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AN ANALYTICAL STUDY OF MEGHNA-GUMTI BRIDGE

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ABSTRACT : A computer method for the analysis of segmentally erected concrete bridges has been used to analyze the Meghna-Gumti Bridge. The program is capable of analyzing bridges consisting of any cross section including box section. Depth of girder can vary along the bridge span. Stiffness method of analysis has been used. A program has been developed to perform necessary calculation for dead load, live load, temperature gradient, creep and shrinkage of concrete and prestressing forces. Numerical results obtained for bending moment and shear force by means of the method presented compared favorably with the design values for internal forces of Meghna-Gumti Bridge as available in the design documents.

KEYWORDS : Meghna-Gumti Bridge, box girder bridge, segmental erection, cantilever method, stiffness method, prestressed concrete.

INTRODUCTION

The first application of segmental bridge construction technique in Bangladesh was in the 930 m long cast-in-place segmental bridge with 87 m central spans on the Meghna River in 1991. This was followed by a 1410 m long similar segmental bridge with 87 m central spans crossing the Meghna-Gumti River along Dhaka-Chittagong highway near Daud Kandi about 40 km south-east of Dhaka in 1994 (Islam, 1997). Both of these bridges were constructed by Obayashi Corporation, Japan as turnkey projects. The design report for the Meghna-Gumti Bridge shows the design values for moment and shear at various sections of the bridge. But the details of analysis are not available in the document. In segmental construction method, a bridge structure is made up of concrete elements usually called segments (either precast or cast-in place) assembled by post-tensioning. Cast-in-place segments were used in Meghna-Gumti Bridge

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to construct the entire bridge length. Only a few detailed investigations into the behaviour of segmentally erected bridges have been published. Vlasov (1961), Scordelis (1967), Kristek (1970), Bazant and ElNimeiri (1974, 1975), Danon and Gamble (1977) and Van Zyl and Sordelis (1978) dealt with the analysis of segmentally erected concrete box girder bridges. However, methods suggested in all these literature involve a huge computational effort in setting up of the structure stiffness matrix and solving the equilibrium equations.

In order to make the analysis relatively simple, a study on the analysis of segmentally erected concrete bridges has been carried out using one-dimensional beam element with only three degrees of freedom at each end. Danon and Gamble (1977) found that these provided sufficient accuracy for the slender long-span single box girders under study herein. Furthermore, in the time-dependent analysis the overall longitudinal behaviour is of primary interest and the local transverse behaviour is a much lesser degree of interest. For the analysis of strsses and deflection at various stages of construction, a computer program in FORTRAN 77 has been developed using the stiffness method assuming the bridge logitudinally as a frame. The results of the Meghna-Gumti Bridge as reported by its consultant Pacific Consultants International(1991) have been used to check the reliability of the developed program.

MEGHNA-GUMTI BRIDGE

The Meghna-Gumti Bridge is a post-tensioned seventeen span continuous hinged rigid frame box girder bridge constructed by cantilever erection method. The intermediate piers are fixed with superstructure and movable shoes support the ends of girders at the hinges. The logitudinal variation of deck-profile is a sine curve. Configurations of centre span and end span are shown in Fig. 1 and Fig. 2 respectively. Typical cross-section of the bridge is shown in Fig. 3.

DESIGN CONSIDERATIONS

Though the bridge has seventeen spans, the analysis was performed taking six spans into consideration. The bottom of the piers were considered fixed with footings, and the hinges at the centre of intermediate spans were assumed to transfer only shearing force in the plane frame analysis. The analytical model is shown in Fig. 4. A total 184 nodes with 3.5 m and 4.0 m segment lengths were used in the analysis. The data for self weight, bridge surface load, curb-railing load live load, creep and shrinkage of concrete, temperature gradient and prestressing tendon were taken from the design report of Meghna-Gumti Bridge (Pacific Consultants International;, 1991). A complete list of design data has been presented by Islam (1997).

GENERAL STIFFNESS EQUATIONS INCLUDING INITIAL STRAINS

Details of stiffness formulation have been described by Harrison (1979) and Islam (1997). The equilibrium equations of a member can be written as (Islam, 1997)

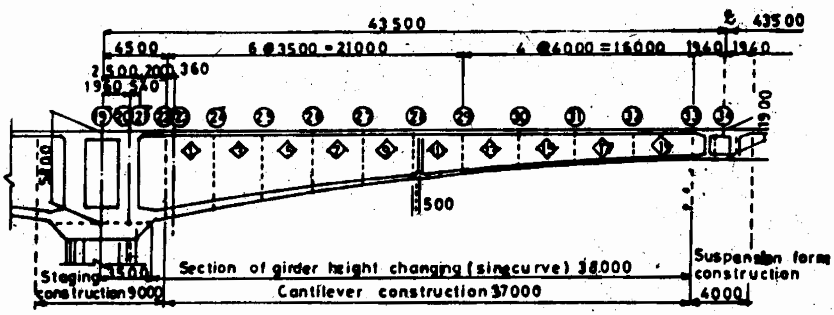

Fig 1. Configuration of Centre Span (All Dimensions in mm)

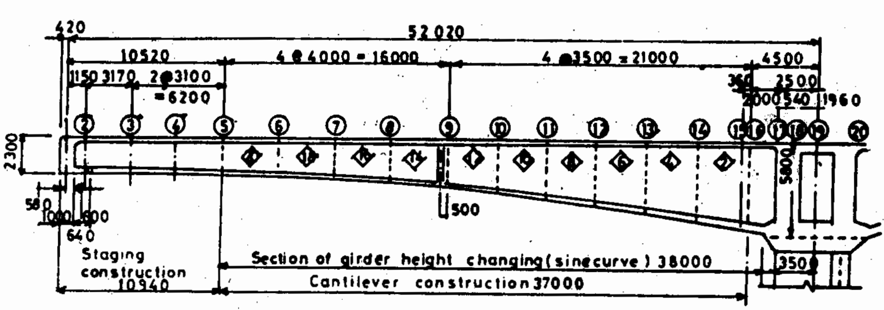

Fig 2. Configuration of End Span (All Dimensions in mm)

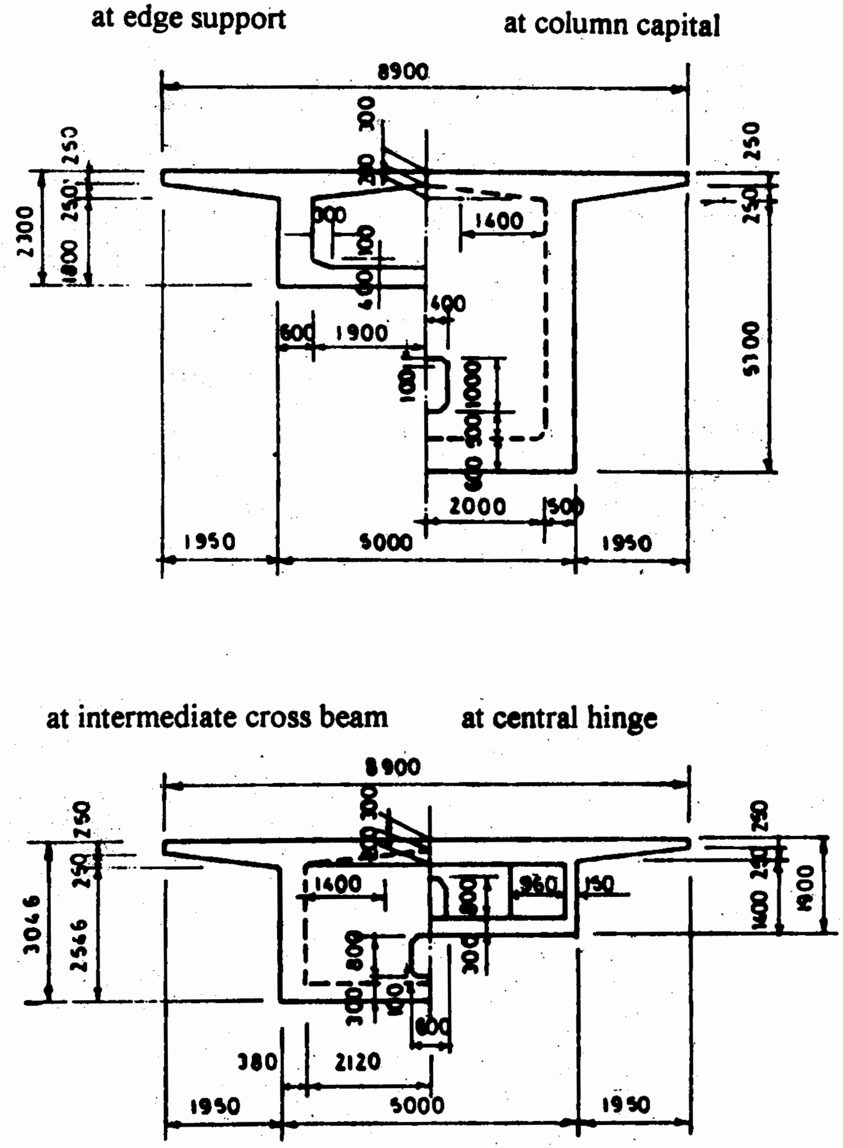

Fig 3. Typical Cross-section of Meghna-Gumti Bridge (All Dimensions in mm)

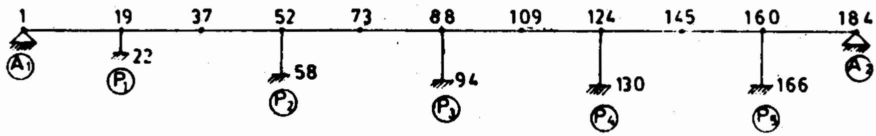

Fig 4. Analytical model of Meghna-Gumti Bridge

$\{W\}=[A].\ \{SR\}$

$=[A].\ [S].\ \{X\}$

$= [A].\ [S].\ [A]^T.\ \{X\}$

$= [K].\ \{X\}$ (1)

Where, {W} is the applied loads at joints; [A] is the statics matrix; {SR} is the vector representing member stress resultants; [S] is the member stiffness matrix; {X} is the vector representing member deformation; $[A]^T$ is the transpose of statics matrix; {X} is the joint displacement vector and [K] is the contribution of one member to the frame stiffenss matrix. After introduction of initial strain vector $\{X^H\}$, the above equation becomes

$\{W\}+ [A].\ [S].\ \{X^H\}=[K].\ \{X\}$ (2)

The elements of member stiffness matrix are assembled in proper sequence to form the frame stiffiness matrix for the analytical model. The joint load Vector and joint displacement vector are also arranged in sequence. The equilibrium equations, thus formed, are solved by using Tripe matrix Decomposition Method (Harrison,1979).

GENERATION OF LOAD VECTOR {W}

Dead load is calculated directly to form the basic components of the load vector. Contribution of live load to the load vector is obtained by first generating the influence line diagrams and then calculating effects of various live load combinations using these diagrams. For computation of loads due to creep and shrinkage of concrete, shrinkage strains and creep coefficients are calculated according to ACI Committee 209 (1970) recommendations. The shrinkage strains are used to generate a pseudo-load vector. For creep analysis, stresses at different levels of deck due to dead and prestressing loads are estimated first. Using these stresses and the creep coefficients, creep strains are obtained. A pseudo load vector due to creep strain, thus generated, is added to the load vector. Temperatre effects are converted into fictious loads at joints in the form of fixed-end moments and axial forces (Weaver and Gere, 1986) and added to the load vector. Prestressing forces are calculated from the data of prestressing steel (Islam, 1997). These forces, in general inclined to the axis of member, are resolved into horizontal components, Vertical componets and moments to obtain the load vector due to prestressing.

COMPARATIVE STUDY

Meghna-Gumti Bridge has been analyzed in this study with the program developed. The results obtained have been presented and compared with values reported by Pacific Consultants International (1991). In the figures, ~Reference" indicates the values reported by Pacific Consultants Internationl.

Bending Moment and Shear force due to Self-weight

Comparison of bending moment due to self-weight for the Meghna-Gumti Bridge as reported by Pacific Consultants International (1991) and as obtained in the present study is shown in Fig. 5. At the interior spans, results show very close agreement. Small variation, however, is observed at the exterior spans.

Comparison of shear force due to self-weight is shown in Fig. 6. Here the results show very close agreement.

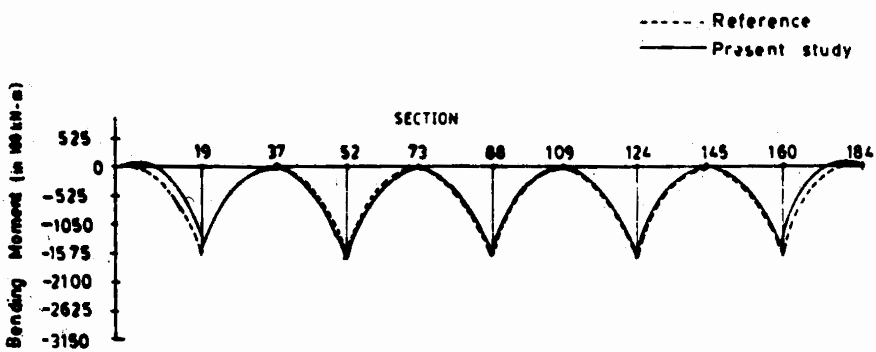

Fig 5. Bending Moment Diagram Due to Self-Weight

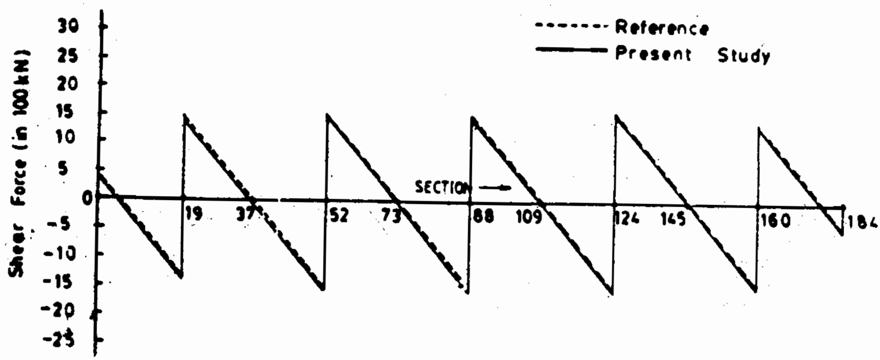

Fig 6. Shear force Diagram Due to Self-weight

Bending Moment and Shear Force Due to Bridge Surface Load

Comparison of bending moment due to bridge surface load is shown in Fig 7. Results of Present study are almost the same as the results of Pacific Consultants International (1991).

In Fig. 8, comparison of shear force due to bridge surface load is shown. It is observed that the variation is negligible.

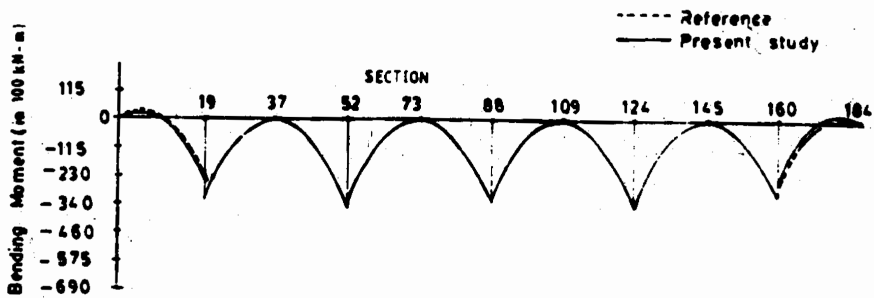

Fig 7. Bending Moment Diagram Due to Bridge Surface Load

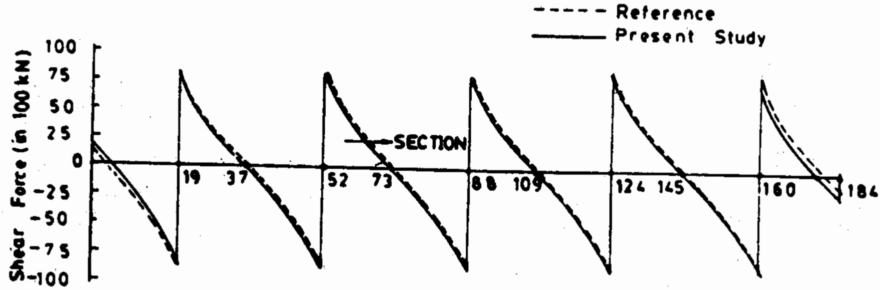

Fig 8. Shear Force Diagram Due to Bridge Surface Load

Bending Moment and shear Force Due to Live Load

Bending moment due to live load is compared in Fig. 9. It is observed that the results obtained in the present study are very close to those of Pacific Consultants International (1991).

Shear force due to live load obtained from the present study is compared with the values of Pacific Consultants International (1991) in Fig. 10 also agreement is very close.

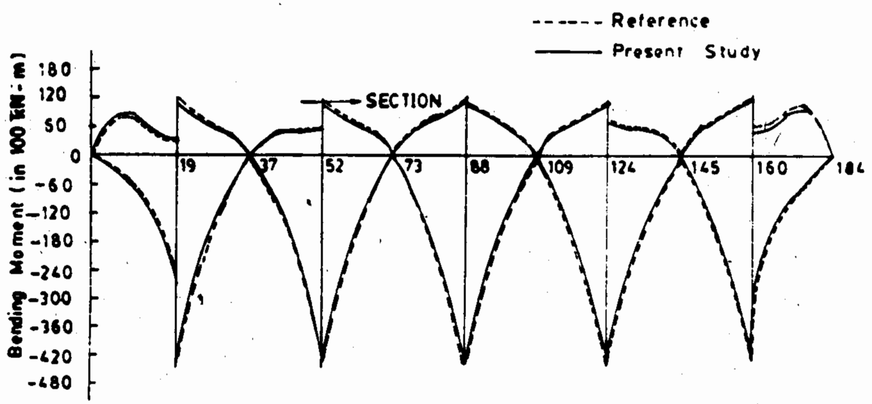

Fig 9. Bending Moment Diagram Due to Live Load

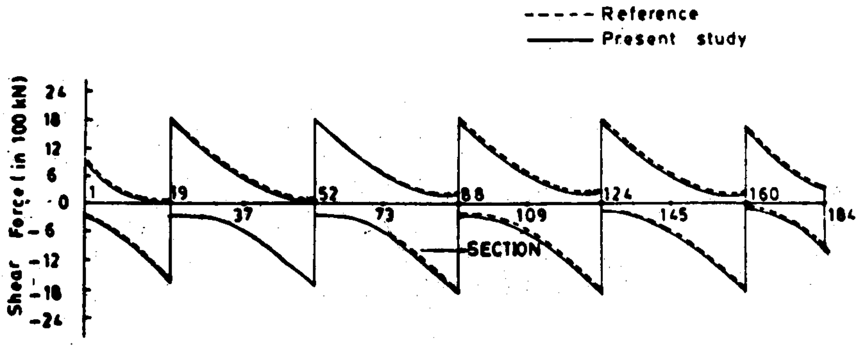

Fig 10. Shear Fore Diagram Due to Live Load

Bending Moment and Shear Force Due to Temperature Gradient

Analysis has been performed for temperature gradient of 5 ^{0} C. The values of bending moment are shown in Fig. 11. At the piers, the present study gives lower values as compared to the values of Pacific Consultants International (1991). This may be due to the non-linear variation of temperature along the depth of the cross-section. In this study, a linear variation of temperature has been assumed along the depth. If the non linearity of temperature variation along the depth were considered, a closer correlation would, perhaps, be obtained.

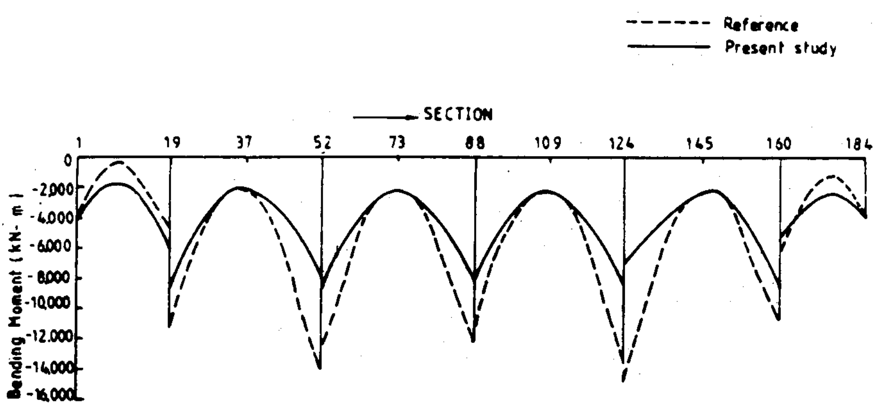

Fig 11. Bending Moment Diagram Due to Temperature Gradient

In Fig. 12, comparison of shear force due to temperature gradient is made. It is observed that the variation is very small except at the right exterior span.

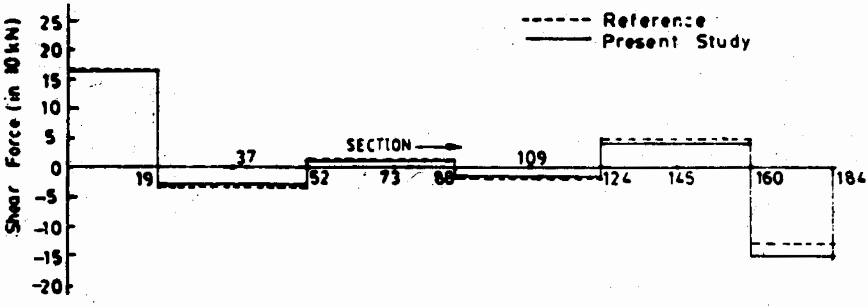

Fig 12. Shear Force Diagram Due to Temperature Gradient

Bending Moment Due to Creep and Shrinkage

From Fig. 13, it is seen that the critical values of bending moments for creep and shrinkage occur at the exterior spans. The present study gives similar values of bending moments to those of Pacific Consultants International (1991). The present study gives relatively higher values especially at the exterior spans. Inaccurate estimation of stiffiness of top segment of pier might be the cause of this difference in two results at the exterior spans as discussed later. Moreover, the use of more than one time step as suggested by Van Zyl and Scordelis (1978) might give better results at the expense of large computer storage and huge computational effort.

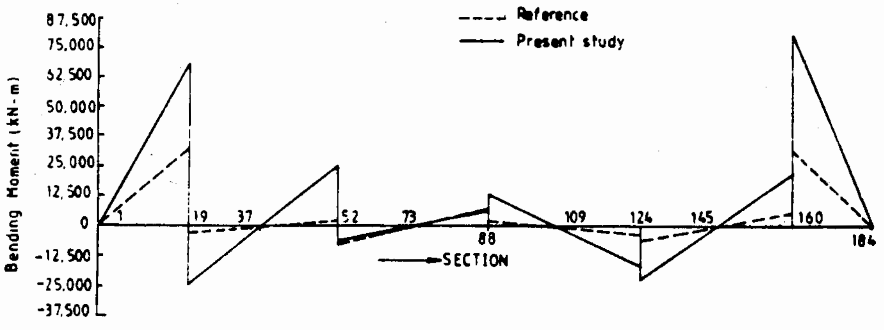

Fig 13. Bending Moment Diagram Due to Creep and Shrinkage

Bending Moment Due to Prestressing

The results of prestressing analysis have been compared in Fig. 14. The results of the present study are similar to those of Pacific Consultants International (1991). From the diagram it is seen that significant disagreement of results occurs only at the left exterior span.

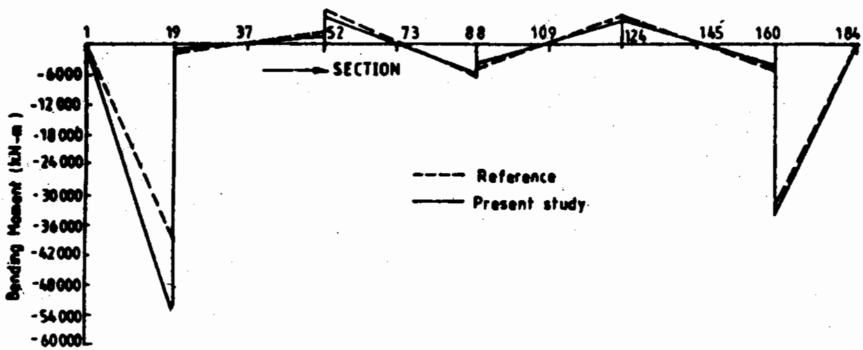

Fig 14. Secondary Moment Due to Prestressing

It may be noted that the values of secondary moments due to prestressing, although opposite in sense to those due to creep and shrinkage, are nearly of the same order of magnitude. Significant values of secondary moments occur at the exterior spans (Fig. 14) which are of negative signs. But in the case of shrinkage and creep these values are of positive signs (Fig.13). The present study gives higher values of both prestressing and creep analyses and these effects, being of opposite sign, seem to counteract each other and hence, perhaps, would not affect the design moment. Moreover, as mentioned earlier, improper estimation of the stiffness of top segment of pier might be the cause of this difference in results as explained in the following article.

Effect of the Variation of Pier Stiffness

During the analysis it was observed that the stiffness value of top segment of pier affects the final results significantly. However, it was difficult to estimate this stiffness value from the complex details as available in the design report of the Meghna-Gumti Bridge. From the estimated value of pier stiffness, trials have been given to investigate the effect of variation of pier stiffness on the bending moment. It has been found that the variation of stiffness value of top segment of pier has a significant effect on the final result at the exterior spans (Islam, 1997). A careful estimation of this stiffness value is, therefore, essential for reliable predication of moments at the exterior spans. For the interior spans, however, the variation of pier stiffness has negligible effect on the values of moment.

CONCLUSIONS

The relatively simple analytical model presented in this paper is capable of analyzing Meghna-Gumti Bridge and similar other segmentally erected concrete bridges with acceptable accuracy. The methods adopted in this study for prestressing analysis and time dependent creep and shrinkage analysis give a fairly reliable representation of the effects of these on internal resisting forces. It was found that the stiffness of top segment of pier has a significant influence on the final results at the exterior spans.

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NOTATION

[A] = member statics matrix

[A] = transpose of statics matrix

[K] = contribution of one member to the structure stiffess matrix

[S] = member stiffness matrix

{SR} = vector representing member stress resultants

{W} = vector representing loads at joints

{X} = member deformation vector

$\{X^H\}$ = initial strain vector of member

{X} = joint deformation vector