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EXPERIMENTAL AND ANALYTICAL INVESTIGATION OF T-BEAM CUM SLAB BRIDGE

Rose Enid Teresa Amaladosson[1] and Umarani Chokkalingam[2]

[1]Department of Civil Engineering, Rajalakshmi Engineering College, India
[2]Division of Structural Engineering, Anna University, India
e-mail: enidarose@gmail.com, umarani@annauniv.edu

ABSTRACT: Roads are the lifelines of modern transport and bridges are the most critical parts of transportation systems. Many existing bridges in India are experiencing deterioration, as the bridge codes which were used for construction of those bridges, had no seismic design provisions, and due to aging and the growth of vehicular loads in magnitude and volume. Also, the bridges are vulnerable to environmental corrosion, long term loading or their coupling effects. As the construction of new bridges involves huge time and money, the condition of the existing bridges are to be evaluated, to preserve their load carrying capacity and service performance. In the present study, an existing reinforced cement concrete T-beam cum slab road bridge (Koyambedu bridge) was experimented by conducting a live load test, to investigate the actual behaviour by measuring the flexural responses of the components of the bridge. Nonlinear Time History analysis was conducted and the results were compared with Modal Pushover analysis results.

KEYWORDS: Live load test, flexural response, SAP2000, Modal Analysis, Nonlinear Time History Analysis.

1 INTRODUCTION

Civil infrastructures, such as bridges, tall buildings, large space structures etc., often have a long service period, may be several decades, during which they are inevitable to suffer from environmental corrosion, long term loading, material aging or their coupling effects with extreme loading and most importantly, the failure due to earthquake loading in structures which were built with low seismic design standards. The resulting damage accumulates and the resisting capacity of the structures against disaster gets reduced. Therefore, structural monitoring systems used for investigating the actual behavior of the structural members become more important for the prediction of the structural performance under loading [2,3,6].

Roads are the lifelines of modern transport, and bridges are an integral part thereof. They are susceptible to failure if their structural deficiencies are unidentified. A large number of bridges constructed around the world were designed during the period, when bridge codes had no seismic design provisions, or when these provisions were insufficient according to the current standards.

The 2001 Bhuj Earthquake that shook the Indian province of Gujarat was the most deadly in India's recorded History [11]. The failures of the bridges during the recent earthquakes have created an awareness, to measure the present flexural responses of the components of the bridges which were built before 2001 to evaluate their structural vulnerability. In this paper, the performance assessment of a functional bridge structure is evaluated by both experimental and analytical investigation.

1.1 Objective of the study

The purpose of this study is to assess the performance of a reinforced cement concrete T-Beam cum slab road bridge for its seismic characterization.

To achieve the objective, the following procedures are adopted.

a) To conduct an experimental investigation to measure the flexural responses of the superstructure.
b) To develop a three dimensional nonlinear finite element model of the RCC bridge located in Koyembedu, Chennai.
c) To perform nonlinear analytical investigation to determine the inelastic response of the structure.

1.2 Methodology

The study bridge is a multi-span simply supported reinforced cement concrete T-beam cum slab bridge. Each span has a 16.21m length and the superstructure consists of four longitudinal girders and five cross girders. An experimental investigation was carried out on a single span of the bridge, and the experiment was planned in such a way, that the girders mounted with strain transducers could be subjected to maximum response. The strain transducers were mounted on one of the longitudinal girders and one of the cross girders. The vehicle was allowed to move over the bridge on a selected path. The stiffness parameters of the members were measured using the strain transducers. Nonlinear Analyses were conducted on the bridge model and performance was analyzed.

2 EXPERIMENTAL INVESTIGATION

2.1 Description of the study bridge

The Koyambedu bridge is a multi-span simply supported reinforced cement concrete T-beam cum slab bridge having a total span of 129.7m [4][5][6][12]. The longitudinal view of the bridge and elevation of the bridge bent are shown in Figures 1 and 2 respectively. The bridge was originally designed with M25 concrete and Fe415 steel. Each span consists of eight equal spans of 16.21m, each of which consists of four longitudinal girders and five cross girders. The superstructure is simply supported by multi-column bents over plain elastomeric bearing pads. Each multi-column bent has four columns which are transversely connected by a bent cap beam. The bridge piers and abutments are supported by well foundations.

Figure 1. Longitudinal view of the study bridge

Figure 2. Elevation of the bridge bent

The cross sectional details of the bridge components are shown in Table 1.

Table 1. Cross sectional details of the bridge components

Bridge Component	Description	Size (mm)
Longitudinal girder	Top flange	2500 x 220
	Bottom flange	500 x 250
	Web	200 x 1355
Cross girder	Cross section	200 x 1400
Bent cap beam	Cross section	1400 x 600
	Length	8800
Bent column	Diameter	800
	Height	4867
Bearing pad	Cross section x depth	500 x 320 x 33.5

The cross sectional view of the bridge at the bent location is shown in Figure 3.

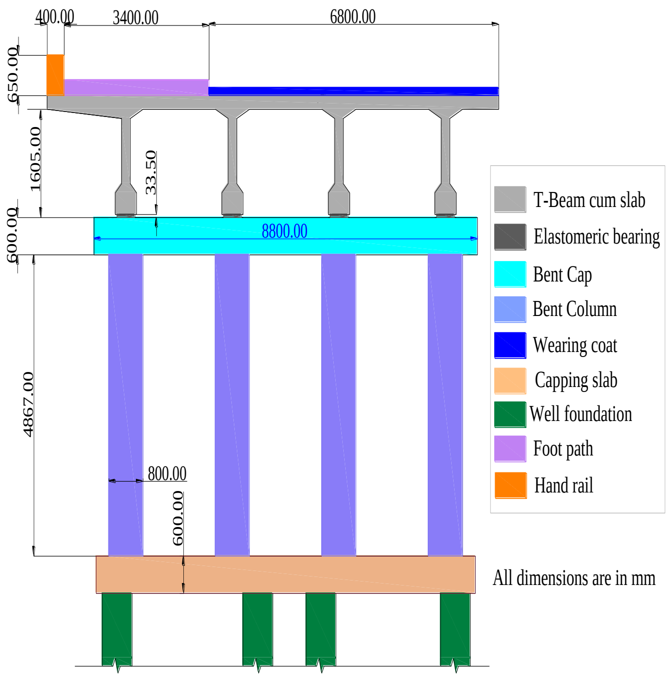


Figure 3. Cross sectional view of the bridge at bent location

2.2 Instrumentation and testing procedures

A live load test on the study bridge was conducted to measure the flexural responses of the longitudinal and cross girders [1][2][3]. The strain transducers were mounted on one of the longitudinal girders and one of the cross girders of a single span (first span) in a completely non-destructive manner. All measurements were made on the surface with three inches long strain-sensors with an extended gauge length of fifteen inches. The purpose of the extensions was to provide an averaged strain value over this gauge length, which reduces the localized effect of the concrete cracks. The positions of the gauges in the longitudinal girder near the abutment, near the midspan and cross girder are shown in Figures 4(a,b), 5(a,b,c) respectively. In the figures, the dimension details of the longitudinal girder (without the deck slab) and cross girder are shown. After the structure was completely instrumented, controlled load tests were performed with multi-axle truck with known axle weights (Figure 6). The autoclicker and reflector arrangement was fixed on the wheel to facilitate the automatic recording of strains corresponding to each wheel rotation (Figure 7). When the truck was driven along a prescribed longitudinal path, for each for each wheel rotation, the strains were automatically measured while the vehicle's position was monitored remotely using the equipment, wireless structural testing system.

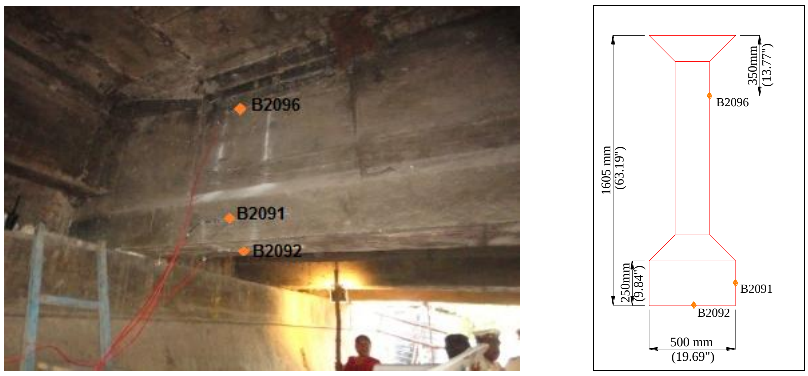


Figure 4(a,b). Position of gauges in the longitudinal girder near the abutment (Photometric view)

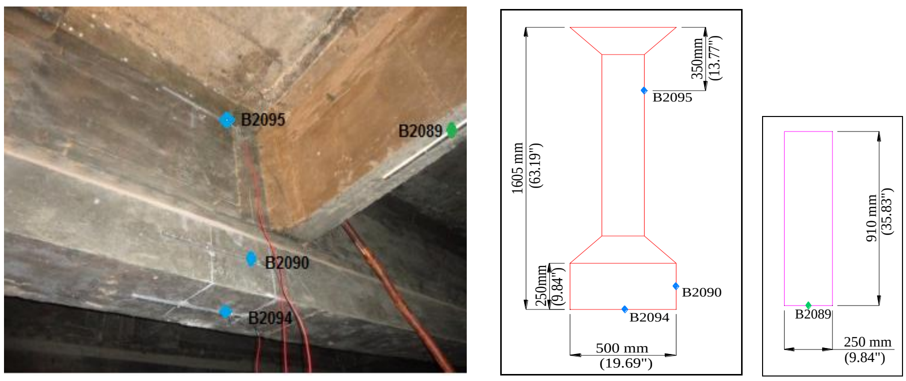


Figure 5(a,b,c). Position of gauges in the longitudinal girder near the midspan and at the bottom of the cross girder (Photometric view)

Figure 6. Truck employed for bridge testing

Figure 7. Auto clicker and reflector arrangement on the wheel

2.3 Modeling and analysis using WINGEN and WINSAC program

The second phase of the investigation was the development of a representative finite element model of the superstructure. The load testing procedures [1] that were used in the field, were reproduced through software after the model was developed. A two-dimensional footprint of the loading vehicle was applied to the model along the same path that the actual test vehicle took across the bridge.

The strain histories obtained from the experimental investigation indicated the nonlinear response of the longitudinal girder and linear response of the cross girder. The initial model was calibrated by modifying the longitudinal girder stiffness, until the results matched with the values measured in the field, as it indicated nonlinear behaviour in the experimental investigation.

The bridge was modeled as a two dimensional (2D) grid consisting of the beam, plate, and spring elements using WINGEN [1], a model generation program that enables to define a planar bridge model with the truck path is shown in Figure 8. The truck dimension and axle weights are shown in Figure 8.

The Bridge Diagnostics, Inc. Structural Testing System (BDI-STS) a suite of analysis and modeling software (WINSAC, WINGEN AND WINGRF) was designed to make the integrated approach a routine process (Figure 9).

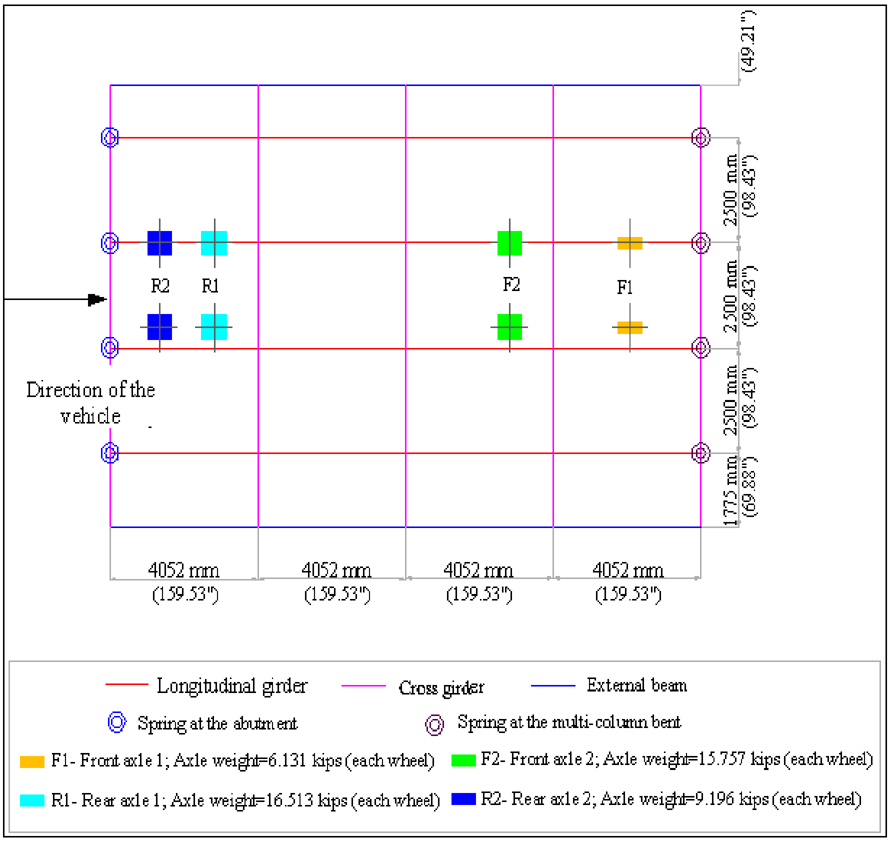


Figure 8. 2D finite element model

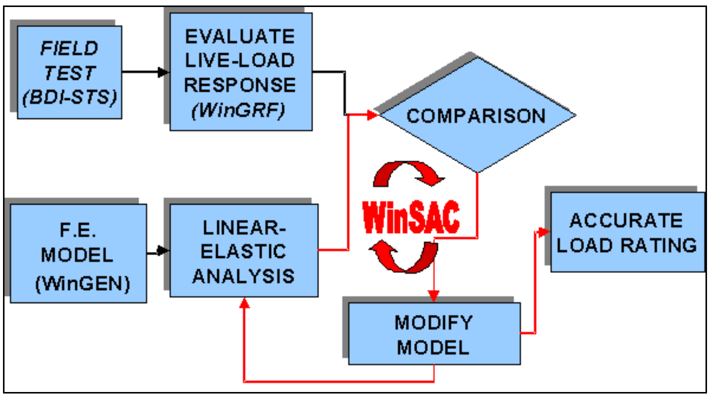


Figure 9. Flow chart of BDI integrated approach

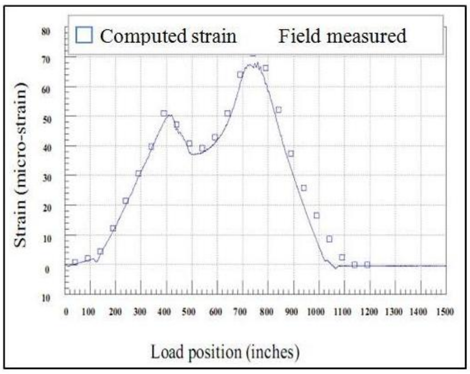


Figure 10a. Strain history plot - B2090

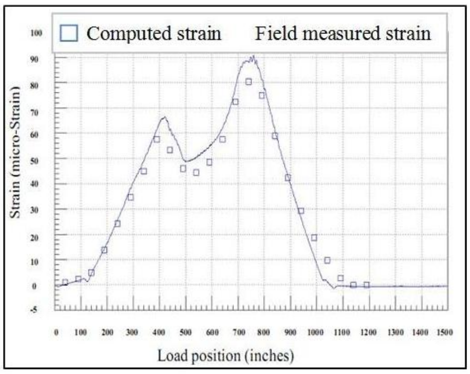


Figure 10b. Strain history plot - B2094

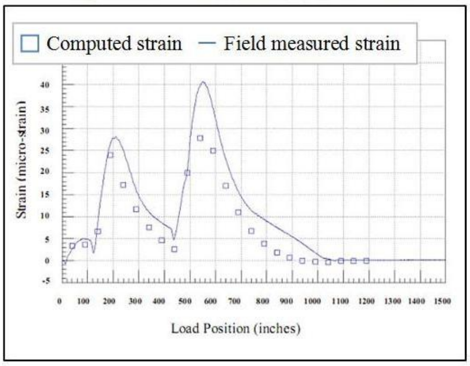


Figure 10c. Strain history plot - B2091

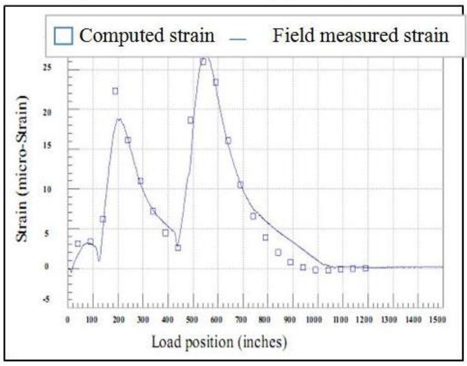


Figure 10d. Strain history plot - B2092

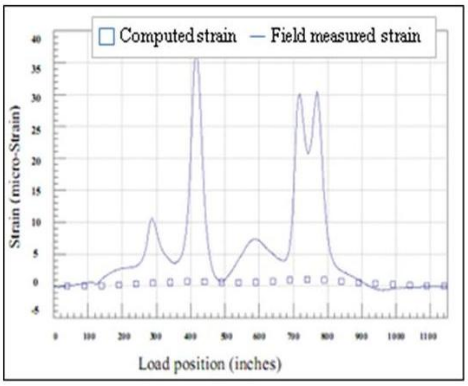


Figure 10e. Strain history plot - B2095

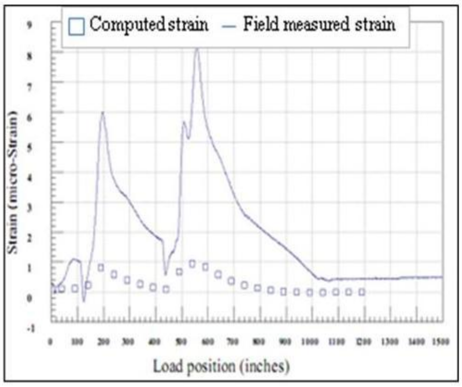


Figure 10f. Strain history plot - B2096

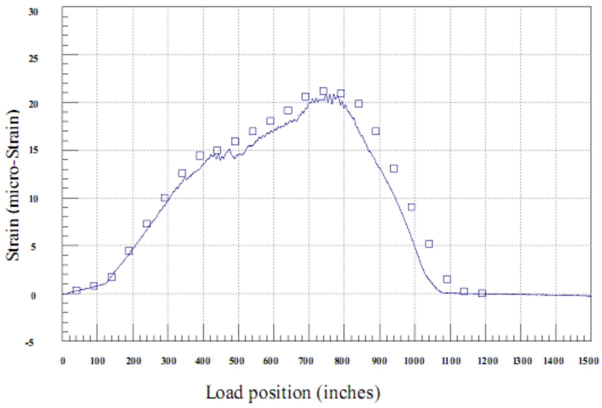


Figure 10g. Strain history plot - B2089

2.4 Test results – Response history plot

The model which was generated using the WINGEN [1] was analyzed with the structural analysis program WINSAC [1]. The WINGRF program is a graphing utility, specifically designed for viewing field measured data from the structural testing system (STS). The field measured data was compared with the responses predicted by the structural analysis program WINSAC. The initial model was calibrated by modifying the longitudinal girder stiffness, until the results matched with the values measured in the field, as it indicated nonlinear behaviour in the experimental investigation. The strain history plots of the transducers are shown in Figure 10a - 10g, and the remarks are tabulated in Table 2.

Table 2. Strain measurements and the behaviour of the members

Sl. No.	Strain Transducer	Position of the transducer	Remarks
1	B2090	Side of the bottom flange of the longitudinal girder near the midspan.	Measured a maximum strain value of 70 micro-strains. This is due to the provision of lesser concrete cover in the longitudinal girder and due to the aging effect. The field measured strain history matched well with the computed strain history. Exhibits nonlinear behaviour.
2	B2094	Bottom of the bottom flange of the longitudinal girder near the midspan.	Measured a maximum strain value of 90 micro-strains. The field measured and analytically computed strain history values matched very well. The strain histories indicated a sudden shift in the magnitude, indicating nonlinear behaviour.

Sl. No.	Strain Transducer	Position of the transducer	Remarks
3	B2091	Side of the bottom flange of the longitudinal girder near the abutment.	Measured a maximum strain value of 40 micro-strains. This is due to the provision of lesser concrete cover in the longitudinal girder and due to the aging effect. Exhibits nonlinear behaviour.
4	B2092	Bottom of the bottom flange of the longitudinal girder near the abutment.	Measured a maximum strain value of 27 micro-strains. The strain histories indicated a sudden shift in the magnitude, indicating a nonlinear behaviour in the member.
5	B2095 & B2096	Top of the longitudinal girder near the midspan and the abutment.	When slipping between the deck and beams occurs in semi-composite conditions, the upper gauges will be heavily influenced by shifts in the neutral axis position. Since this type of behaviour cannot be modeled, gauges that display such irregular shapes were not included in the model correlation.
6	B2089	At the bottom of the cross girder at the midspan.	The strain values measured by the transducer exhibited a linear behaviour.

2.5 Summary of the experimental investigation

For concrete structures, non-linear material behaviour has to be considered as they do not behave strictly linear. Furthermore, defects, as for instance cracks in reinforced concrete, lead to additional non-linearities that increase with the level of damage. Hence, the use of structural monitoring systems to assess the actual behaviour of the structural member is important to consider the nonlinearities in order to avoid misinterpretation. Moreover, the nonlinearities themselves can be explicitly used as damage indicators, as they are dependent on the damage state. This paper presents the results of an experimental analysis of a single span of T-Beam cum Slab concrete road bridge. The strain histories obtained from the experimental investigation (live load test) indicated the nonlinear response of the longitudinal girder and linear response of the cross girder of the T-Beam cum slab bridge. Examining the measured and computed strain data, the stiffness parameters of the longitudinal girder was changed by the heuristic method to improve the model. By improving the model, the effective stiffness (EI_{eff}) property of the longitudinal girders evaluated from the experimental investigation was found to be 0.8 times the gross stiffness (EI_g). The stiffness parameter obtained from the experimental investigation was used while modeling the same bridge using SAP2000 using the nonlinear analysis.

3 ANALYTICAL INVESTIGATION

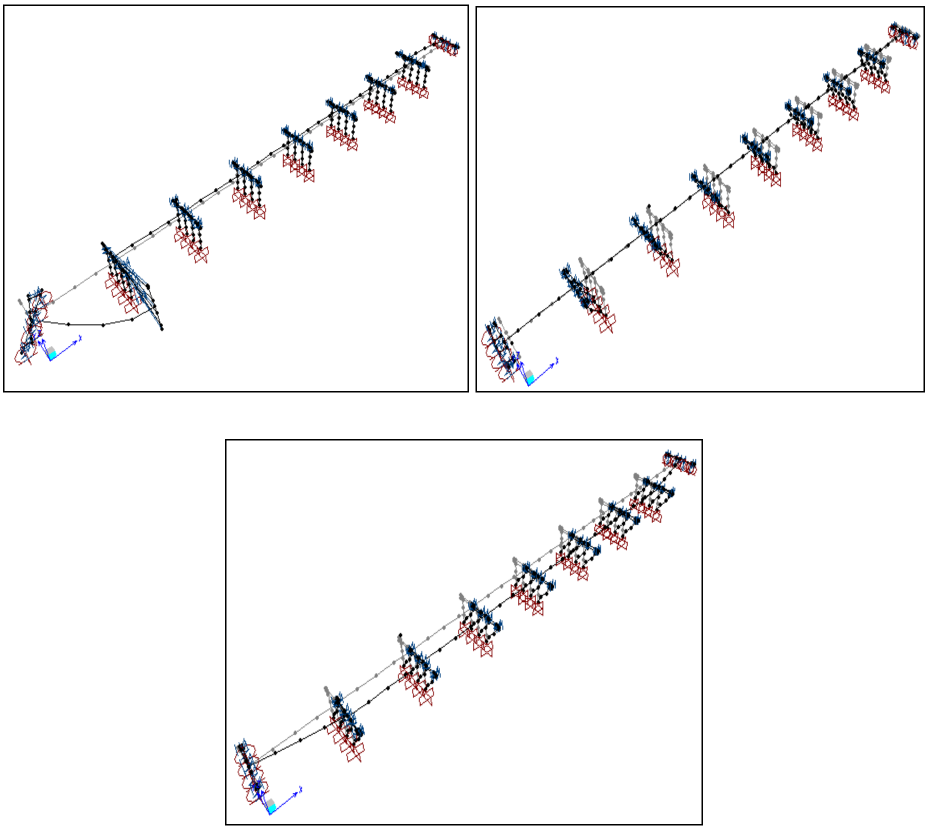
3.1 Modelling of the bridge

A three dimensional (3D) finite element model (FEM) of the bridge was created using Structural Analysis and Program Software SAP2000. Spine model (a type of superstructure model) was employed for modelling the superstructure [8][9]. The deck edges in each simply supported span were considered rigid. Due to the large in-plane rigidity, the superstructure was assumed as a rigid body for lateral loadings [8][10]. The bridge consists of seven multicolumn bents and every bent was modelled as a plane frame. The framing action and coupling between columns in the multi-column bent provides seismic resistance in terms of strength and stiffness.

The bent cap and the columns were modeled as beam-column elements. Effective moment of inertia was taken as 0.7Ig [10] for reinforced concrete columns which were modeled using Section Designer (Sub programme in SAP2000). The interface between each column and the corresponding geometric centre of the bent cap was considered rigid. The default hinge properties (PMM – P stands for axial force, M stands for M2 moment, and M stands for M3 moment in SAP2000) were assigned to each end of the columns. The base of the column was assumed as fixed. The girders of the bridge are simply supported over plain elastomeric bearing pads. The horizontal sliding behavior of the interface between the bearing and girder or cap beam was modeled using linear spring element [7].

3.2 Modal analysis

The modal analysis of the study bridge [4][5][6] was performed to find the dynamic characteristics of the bridge, such as mode shapes, modal mass participation, natural frequencies etc. In the fundamental mode (mode#1), 84.32% of the total mass of the bridge structure participated in the vibration of the structure in the transverse direction. In the second mode, 93.57% of the total mass participated in vibrating the bridge structure in the longitudinal direction. In the 3^{rd}, 4^{th}, 5^{th}, 6^{th} and 7^{th} modes, it was observed that there was no additional mass participation in exciting the bridge structure in the longitudinal and transverse directions. In the 8^{th} mode an additional mass participation of 1.4% was observed in the transverse direction. In the 9^{th} mode there was 0.3% of additional mass participation in the transverse direction. In bridge structures, higher modes may have a significant effect and therefore to evaluate the seismic response of the structure in the higher mode, the 8^{th} mode was also considered in this study. The mode shapes of the bridge structure in the fundamental (first), second and eighth modes are shown in Figures 11 (a), (b) and (c) respectively.



Figures 11a,b,c. Mode shapes of the bridge structure in the fundamental, second and eighth modes

4 NONLINEAR TIME-HISTORY AND MODAL PUSHOVER ANALYSES – RESULTS AND COMPARISON

4.1 Displacement of the deck at each bent location in the transverse direction

The displacement of the deck calculated at each bent location, when the modal pushover analysis and the time-history analysis El Centro Earthquake record were carried out in the transverse direction of the bridge structure, is shown in Figure 12. From the pushover analysis results [5][6], it was found that for the fundamental mode, the center of the mass of the superstructure directly above bent B4 experienced a maximum deck displacement of 87 mm, whereas in the higher mode (eighth mode), the center of the mass of the superstructure underwent a maximum displacement of 84mm.

As both the fundamental mode and higher mode experienced more or less the same deck displacement the total responses of the deck at each bent location, by using the modal combination rule (SRSS), was found to be of a considerably larger value. The results of the modal pushover analysis, which accounts for the two transverse modes (fundamental mode and eighth mode), were not closer to those of the time-history analysis, due to the estimation of the total response by using the modal combination rule (SRSS) (Figure 12).

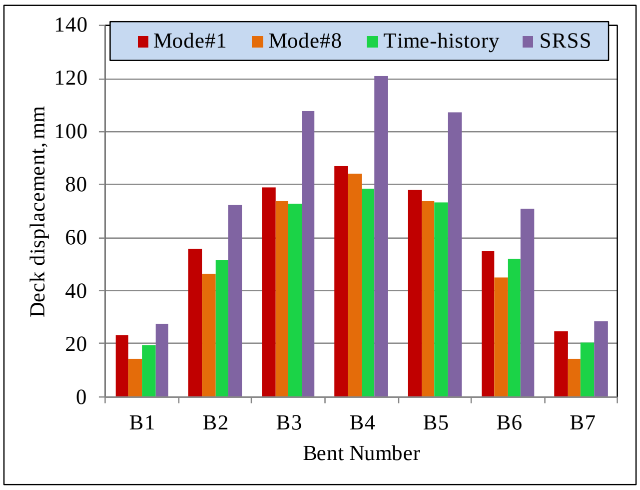


Figure 12. Displacement of the deck in the transverse direction

From Figure 12, it was observed that the SRSS overestimates the transverse displacement of the deck of the bridge (120mm), compared to the more accurate approach of the nonlinear time-history (78.3mm).

On the other hand, from both the results of the independent pushover analysis of mode#1 and mode#8, it was found that the displacement of the deck at all the bent locations in the higher mode (mode#8) were much closer to the nonlinear time-history analysis results, indicating the significance of the higher mode.

4.2 Bent top displacement in the transverse direction

The bent top displacements determined by the standard pushover analysis (SPA) for the fundamental mode, modal pushover analyses (mode#1 and mode#8) and the SRSS results, were compared with those from the nonlinear time history analysis, and are shown in Figure 13.

From the pushover analysis results it was found that for the fundamental mode, the middle bent B4 experienced a maximum displacement of 79.2mm, whereas in the higher mode (eighth mode), the middle bent underwent a maximum displacement of 75.9mm.

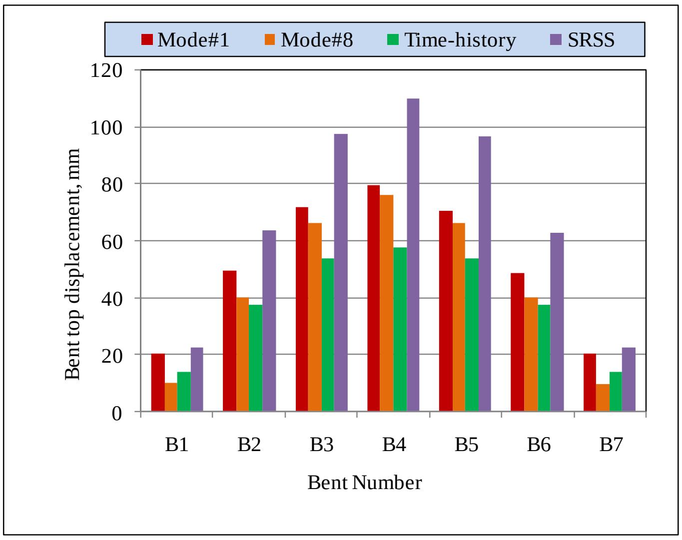


Figure 13. Bent top displacement in the transverse direction

From the results of the independent pushover analysis of mode#1 and mode#8, it was found that the bent top displacements observed in both the fundamental mode and the higher mode (mode#8) overestimate the displacement observed with the nonlinear time-history analysis. The bent top displacement calculated using the SRSS overestimates the results compared to mode#1, mode#8 and the time-history results.

4.3 Drift capacity and demand in the transverse direction

Drift capacity is defined as the global drift of the bent, which is obtained from the pushover analysis. The drift demand is defined as the average maximum bent top drift, when subjected to an earthquake load. The global drift capacity and demand of the bent in the transverse direction is shown in Table 3. The global drift capacity of the bent was found to be greater than the drift demand.

Table 3. Drift capacity and demand in the transverse direction

Sl. No.	Drift capacity	Drift demand
1.	1.79	1.61

5 CONCLUSION

The strain histories obtained from the experimental investigation (live load test) indicated the nonlinear response of the longitudinal girder and linear response of the cross girder of the T-Beam cum slab bridge. Examining the measured and computed strain data, the stiffness parameters of the longitudinal girder was changed by the heuristic method to improve the model. By improving the model, the effective stiffness (EI_{eff}) property of the longitudinal girders evaluated from the experimental investigation was found to be 0.8 times the gross stiffness (EI_g).

The stiffness parameter obtained from the experimental investigation was used while modeling the same bridge using SAP2000. The modal analysis of the study bridge was performed to find the dynamic characteristics of the bridge. In the fundamental mode (mode#1), 84.32% of the total mass of the bridge structure participated in the vibration of the structure in the transverse direction. In the second mode, 93.57% of the total mass participated in vibrating the bridge structure in the longitudinal direction. In the 3rd, 4th, 5th, 6th and 7th modes, it was observed that there was no additional mass participation in exciting the bridge structure in the longitudinal and transverse directions. In the 8th mode an additional mass participation of 1.4% was observed in the transverse direction. In the 9th mode there was 0.3% of additional mass participation in the transverse direction. In bridge structures, higher modes may have a significant effect and therefore to evaluate the seismic response of the structure in the higher mode, the 8th mode was also considered in this study.

Nonlinear time-history analysis was performed in transverse directions of the bridge structure using El Centro Earthquake record and results are compared with modal pushover analyses results. The modal combination rule (SRSS) overestimates the displacement of each span at all the bent locations in the transverse direction. The transverse displacement of each span at all the bent locations in the higher mode (mode#8) was much closer to the nonlinear time-history analysis results, indicating the significance of the higher mode. Modal pushover analysis and the SRSS have overestimated the bent top displacements compared to time-history analysis.

6 ACKNOWLEDGEMENT

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