Modeling and dynamic analysis of bridges designed with AASHTO and Florida I-beam girders

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Abstract. A comparative analysis of the dynamic behavior of two prestressed concrete girder bridges was implemented. The first bridge was designed with AASHTO (American Association of State Highway and Transportation Officials) type III girders while the second bridge was designed with the new (FIB) Florida I-Beam girders. Both bridges have the same length, width and girder depth. However, six AASHTO Type III girders were used in the first bridge versus only four FIB girders for the second bridge. The bridges response to dynamic loads was obtained with the aid of a sophisticated finite element model. Modal and time history analyses were performed for the bridges to identify the dynamic properties, behavior, and responses of the two bridges. The dynamic properties and the deflections of the bridges were obtained and compared for different girders and spans. Outputs from the finite element model indicated that although FIB bridge has less number of girders, the deflections of FIB bridge were less than their corresponding values from AASHTO bridge.

1. Introduction

Highway bridges are one of the most important components in the transportation systems. They help people to connect and reduce the congestion of traffic. However, in recent years, there was a sharp increase in the traffic volume and traffic flow, in addition to the other limitations associated with building a new bridge, such as construction cost and time. Therefore, there was a need to improve the design codes and standards to fulfill the recent traffic requirements. Accordingly, FDOT (Florida Department of Transportation) developed a new pre-stressed concrete beam called Florida I-beam) in partnership with an academic researcher [1]. FIBs are designed to replace the conventional AASHTO and Florida Bulb T-Beams, which have been widely used in the Florida state. FIBs offer many advantages over the AASHTO and Florida Bulb T-Beams, such as higher load carrying capacity, more efficient fabrication, and safer construction. In addition, FIBs appear to provide an increase in the lateral stiffness as a result of its thicker top and bottom flanges and to reduce the total cost of bridges compared with the traditional pre-stressed beams. FIBs are characterized by their high strength, usually in the range from 8 to 10 ksi. Owing to the wide bottom flange section of FIBs compared with other pre-stressing beams, more pre-stressing strands can be used, which are commonly required in girders with longer span or wider spacing. Another advantage of FIBs is that they can provide larger vertical clearance when using the shallower FIBs sections instead of using their AASHTO equivalents with deeper sections. Nowadays, with the aid of FIBs, bridge designers are able to reduce the number of beams needed for bridges, which eventually results in a reduction in the bridge cost.
Several studies on the performance and behaviour of bridges built with AASHTO and Bulb T-Beams can be found in the literature. However, only few studies have focused on bridges designed with the use of FIBs and compared them with the AASHTO and Bulb T-Beams. Generally, most of the implemented works were prepared by FDOT, such as a cost comparison analysis between two bridges; one was designed with six AASHTO type III girders while the other was designed with four FIB girders. It was found that by using the FIB girders, the bridge cost can be lowered by 24% of the cost when using AASHTO type III girders instead [1].

Catbas et al [2] also developed a 3D finite element model using the commercial software CSiBridge [3], [4] to study the load rating of the same bridge. It was shown that when using FIBs, an increase in the live load capacity can be achieved (about 20% for interior girders and 40% for exterior girders) in comparison with AASHTO type III girders. The same bridges were further studied by [5]. The bridges were analysed for the Florida legal trucks, which are C5 and SU4. The CSiBridge software was also used in the analysis. These loads were applied separately to study the deflections, moment envelopes, section capacity and live load rating of the two bridges. Simulation results demonstrated the higher capacity and the vertical stiffness of FIB girders over their equivalents of AASHTO type III girders.

Another study was implemented by [6] to investigate the load rating and the reliability of the critical members for the same bridges under C5 and SU4 truck loads. In that study, a comparative analysis was conducted and it was observed that the interior bridge girder designed with AASHTO girders has a probability of failure about 6 times that of the FIB bridge girder.

Generally, studying the dynamic response of bridges is of interest. However, there is no study that has actually considered the dynamic response and properties of bridges built with FIB girders and compare them to their AASHTO equivalents. This paper presents a three-dimensional finite element model that is capable to describe the dynamic behaviour of two bridges. The first bridge comprised of four FIB girders while the second bridge consists of six AASHTO type III girders. The CSiBridge software was used to create both models. Modal and time history analyses were carried out. The dynamic properties and responses were determined and compared for both bridges. Results from the analysis confirmed the effectiveness of using FIB girders in bridges instead of AASHTO girders.

2. Bridge Specifications
The bridges investigated in this study are comprised of three equal spans with each span being 90-ft. The bridges bent have three circular columns supporting a 43.08-ft long beam cap. The first bridge has six AASHTO type III girders spaced at 7 ft-6 in. each with 26-0.6 in. low-relaxation pre-stressing strands while the second bridge has four FIB girders spaced at 11 ft-9 in. with 42-0.6 in. low-relaxation strands, as shown in Figure (1) and (2). Both girders are 45 in. deep. The bridges have two sidewalks with a width of 6ft for the left side and 10ft for the right side. The physical properties of the concrete and girders sections that were used in the finite element analysis are summarized in table (1). Figure 1. Typical FIB cross-section. Figure 2. Typical AASHTO Type III girder cross-section.
Figure 1. Typical FIB cross-section.  
Figure 2. Typical AASHTO Type III cross-section.  

Table 1. List of assumptions.

| Property                     | Value                                           |
|------------------------------|-------------------------------------------------|
| Slab thickness               | 8 in                                            |
| Haunch thickness             | 2 in                                            |
| Deck compressive strength    | 4 ksi                                           |
| Girder compressive strength  | 8.5 ksi                                         |
| Barrier load                 | 0.32 kips/ft length                            |
| Wearing load                 | 0.035 kip/ft²                                  |
| Sidewalk load                | 0.03 kip/ft²                                   |
| Column dimension             | 3 circular column 4.5ft dia. (20 - #8 grade 60 steel) |
| Beam Cab dimension           | Depth (56 in), width (60 in)                    |
| Prestress steel              | 0.6 in low relaxation strands                   |
| Fpu                          | 270 ksi                                         |
| Jacking force                | $0.7 f_{pu}$                                    |

Figure 3. CSiBridge FE model: AASHTO type III (Left), FIB (Right).

3. The Finite Element Model  
The use of FE programs for modelling 3D bridges has become more popular over the last few decades [7], [8]. This is due to the increase in the efficiency and effectiveness of these programs. Hence, a 3D
finite element model was created to model the two types of bridges by using the commercial software CSiBridge (v21.2.0), and as shown in Figure 3. Shell elements with three degree of freedom were used to model the deck section while frame elements were used to model the precast pre-tensioned girders, the columns, and the beam cap. Eighteen precast pre-tensioned AASHTO type III girders were defined in the first model with 156 tendons in each span, i.e., 468 pre-stressed strands for the entire bridge. As for the second bridge, 12 FIB-45 girders were defined with 168 tendons in each span, namely, 504 strands total. Pre-tensioned tendons were modeled as separate elements with 44 kips force embedded in the precast girders to satisfy the design specifications for the strength limit state [9]. It should be noted that the force value in the tendons represents the force before pre-stress losses as CSiBridge accounts for the pre-stress losses in the tendons. The jacking force was applied from both sides of the tendons and zero value was specified for the curvature loss coefficient, wobble loss coefficient, and anchorage slip loss coefficient. Each bent has three piers, in which the piers along with the abutment foundations were assumed to be fixed. In addition, intermediate cross diaphragms with 19 in depth and 12 in width were used every one third points of each span while end diaphragms with 40 in depth and 12 in width were used at the ends of each span. Furthermore, only two (12 in.) lanes were loaded in the analysis.

4. Dynamic analysis

In order to assess the dynamic properties, behavior, and responses of both bridges, modal and dynamic analysis were implemented, in which HS20-44 truck was used to load the bridges, Figure (4). Two different configurations were considered in the time history analysis, where each traffic lane was loaded separately with three trucks, but with different speeds. That is, at speed 40, 60, and 75 mph, as these high values of speed are expected to cause a significant excitation to the bridge. The loading time required for the vehicles to pass over the bridge is 16 seconds. The damping ratio was assumed as a constant value equal to 5% according to the recommended damping values by [10].

5. Results and Discussion

5.1. Natural frequencies and mode shapes

The dynamic properties of both bridges were obtained by implementing a modal analysis. The natural frequencies and periods of the first twelve modes are listed in Table 2. From the results given in table 1, it can be noticed that the natural frequencies of AASHTO type III bridge are close to their corresponding values from FIB bridge, this is to be expected as the aspect ratio along with other parameters, such as vehicle speed and mass ratio of both bridges are the same [11]. In addition, the fundamental natural mode of both bridges has a natural frequency of approximately 6 rad/sec and the frequencies of the 2nd to the 10th modes range from 14 rad/sec to 40 rad/sec. Figure (5) and (6) show the first six vibration modes of AASHTO type III and FIB bridges. It is clear that the dominant vibration mode for both bridges is the longitudinal mode, as shown in Figure (5).
5.2. Deflection
In order to predict the dynamic response of concrete bridges, a time history analysis was performed, in which the deflection at the exterior and interior girders of both bridges was determined. Figures (7) and (8) show the displacement at the mid-span of the exterior girders for the first and second spans of both bridges. In addition, the displacements of the interior girders are shown in Figures (9) and (10).

| Mode No. | Natural Frequency $\omega_n$ rad/sec | Period $T$ sec | Type of mode | Natural Frequency $\omega_n$ rad/sec | Period $T$ sec | Type of mode |
|---------|-------------------------------------|---------------|--------------|-------------------------------------|---------------|--------------|
| 1       | 6.033                               | 1.041         | Longitudinal | 6.016                               | 1.044         | Longitudinal |
| 2       | 14.759                              | 0.425         | Vertical     | 15.078                              | 0.416         | Vertical     |
| 3       | 14.890                              | 0.422         | Vertical     | 15.248                              | 0.412         | Vertical     |
| 4       | 15.323                              | 0.410         | Vertical     | 15.637                              | 0.402         | Vertical     |
| 5       | 16.705                              | 0.376         | Torsional    | 17.513                              | 0.358         | Torsional    |
| 6       | 17.078                              | 0.368         | Torsional    | 17.730                              | 0.354         | Torsional    |
| 7       | 18.903                              | 0.332         | Transverse   | 18.752                              | 0.335         | Transverse   |
| 8       | 22.162                              | 0.283         | Transverse   | 22.631                              | 0.277         | Transverse   |
| 9       | 32.696                              | 0.192         | Torsional    | 32.612                              | 0.193         | Torsional    |
| 10      | 34.593                              | 0.182         | Transverse   | 33.379                              | 0.188         | Transverse   |
| 11      | 35.787                              | 0.175         | Vertical-Torsional | 33.918                          | 0.185         | Vertical-Torsional |
| 12      | 40.353                              | 0.156         | Vertical-Torsional | 38.630                            | 0.163         | Vertical-Torsional |

Table 2. Modal frequencies and periods of the AASHTO and FIB bridges.

for the first and second spans, respectively. Generally, it can be seen that the dynamic response of the two bridges is quite similar. Yet, one can observe that the difference between the displacements peaks increase for the exterior girders. Figures (9) and (10) also illustrate one of the advantages of using FIB girders, in which it may be noticed that the deflection values of interior girders are relatively close although four AASHTO girders were used versus only two FIB girders. That indicates the efficiency and effectiveness of using FIB girders.

Moreover, the results show that the maximum difference between the displacements of the two bridges occur at the exterior girders, which is about 12.29% for the first span and 8.6% for the second span. Furthermore, it is of interest to note that the displacement peaks are attributed to the passing vehicles with different speeds. It may be observed that the effect of the last passing vehicle that corresponds the greatest speed (75 mph) influences the dynamic response of the bridge more significantly than the other two speeds. This is expected since the dynamic response of bridges tends to increase with the increase in vehicle speed.

The maximum deflections of the first span when the first and second lanes are loaded were also illustrated in Figures (11) and (12). The deflections were obtained at mid-span sections along the width of the two bridges. It can be seen that the maximum deflections values of the AASHTO bridge are higher than their corresponding values from FIB bridge. This difference becomes more obvious at the exterior girders and less noticeable at the interior ones. That behavior may be attributed to the fact that the loaded lanes are closer to those girders.

Furthermore, by cross referencing the curves of maximum deflection of both bridges, namely, Figure (11) or Figure (12), it can be noted that the location of the interior girders of the two bridges is not fully aligned. Hence, the deflection values illustrated in Figures (9) and (10) don’t actually reflect the deflection at the exact same location, which in turn contributes to the similarity of the two curves. However, higher deflection values of AASHTO bridge can be noticed when comparing the deflection at the same point, as shown in Figures (11) and (12).
Figure 5. First three vibration modes of the bridges: AASHTO type III (Left), FIB (Right)
Figure 6. Second three vibration modes of the bridges: AASHTO type III (Left), FIB (Right)
Figure 7. Mid-span deflection of exterior girder (1st span).

Figure 8. Mid-span deflection of exterior girders (2nd span).

Figure 9. Mid-span deflection of interior girders (1st span).
Figure 10. Mid-span deflection of interior girders (2nd span).

Figure 11. Maximum deflection at mid-span (section cut across span 1 of the bridge) when lane 1 is loaded.

Figure 12. Maximum deflection at mid-span (section cut across span 1 of the bridge) when lane 2 is loaded.
6. Conclusions
A three dimensional finite element model to simulate the dynamic effects on two bridges was developed. The first bridge comprised of four FIB girders while the second bridge consists of six AASHTO type III girders. The bridge models were created by using CSiBridge software. Modal and time history analyses were implemented by loading the bridges with three trucks but with different speeds. Namely, 40, 60, and 75 mph speeds to effectively induce a dynamic excitation to the bridges and hence generate vibration of the bridge structure.
Based on the results obtained from the finite element simulation, the dynamic behavior of both bridges is quite similar. This can be attributed to the similarity of their properties, such as aspect and mass ratios, and their loading conditions. However, the FIB bridge deflections were less than the deflections values of AAHTO type III bridge. Simulation results also confirmed the effectiveness of using FIB girders in bridges instead of AASHTO girders.

7. References
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