Experimental Investigation and Modelling of Spread Slab Beam Bridges

Tevfik Terzioglu  Mary Beth D. Hueste  John B. Mander
Postdoctoral Researcher, Texas A&M University, College Station, TX
Transportation Institute, College Station, TX
Professor, Texas A&M University, College Station, TX
Professor, Texas A&M University, College Station, TX

Abstract  A new bridge system was recently developed for short span bridges in low clearance areas using the same concept as spread box beam bridges in which the standard TxDOT slab beams are spaced apart. This paper presents an evaluation of spread slab beam bridges in terms of design, constructability, and performance. Forty-four bridge geometries were designed using standard TxDOT slab beam types to determine the feasible design space. One of the most aggressive geometries with widely spaced slab beams was constructed at full-scale. The bridge was tested under static and dynamic vehicular loads to obtain important insight into its behaviour under vehicular load. The load distribution behaviour was investigated during field testing and the measured data was utilized to validate computational modelling techniques. Based on the research findings, it was concluded that spread slab beam bridges that utilize precast concrete panels with a cast-in-place concrete deck provide a viable construction method for short-span bridges. The desired performance was achieved for in-service loading. Experimental live load distribution factors (LLDFs) were evaluated and LLDF equations for spread box beams were reviewed for applicability to spread slab beams. The measured response under dynamic loads was larger compared to the static values.

1 Introduction

Precast prestressed concrete girders have been used effectively to provide economical bridge superstructures for short to medium spans [1]. The majority of these prestressed concrete bridges are simply supported spans where the cast-in-place (CIP) deck slab is made composite with precast pretensioned girders. The Texas Department of Transportation (TxDOT) often uses prestressed concrete slab beam bridges as a common alternative for short span bridges. The conventional approach consists of placing the slab beams side-by-side and casting a 127 mm CIP reinforced concrete deck on top of the slab beams. This shallow bridge
superstructure system is attractive in locations where there is a low clearance below
the bridge. However, conventional slab beam bridges are more expensive compared
to standard I-girder bridges that are constructed using PCPs as stay-in-place
formwork between girders. To address this issue TxDOT has shown interest in
exploring new bridge systems that may provide more economical solutions for
short-span bridges. One such idea that has been developed by TxDOT is to modify
the current short span bridge design that uses immediately adjacent prestressed
cement slab beams shown in Figure 1(b). The proposed solution is to spread out
the slab beams and to use a conventional topped panelized deck as shown in
Figure 1(a). It is anticipated that spread slab beam bridges will result in a possible
reduction in the overall bridge cost while providing another design alternative for
short span bridges.

![Spread slab beam bridge](image1.png)  ![Conventional slab beam bridge](image2.png)

Figure 1. Slab beam bridge cross-section

Viable spread slab beam bridge geometries were chosen according to practical
beam spacing and bridge width criteria. A total of 44 spread slab beams were
designed using the maximum permissible concrete design strength. One of the
preliminary designs with a large eccentricity due to a wide beam spacing and a
relatively longer span length was chosen for the full-scale bridge construction and
field testing. A full-scale spread slab beam bridge was constructed and field tested
at the Texas A&M University Riverside Campus. The measured response was then
used to validate computational finite element method (FEM) and grillage analysis
modeling techniques to evaluate the accuracy of alternative methods for modeling
spread slab beam bridges.

One key issue for developing a new bridge superstructure system is identifying
appropriate LLDFs. Although there are other viable methods of analysis for
calculating moment and shear demands, such as the grillage and the FEM, bridge
design engineers prefer using approximate LDFs that are provided in the American
Association of State Highway and Transportation Officials (AASHTO) Load and
Resistance Factor Design (LRFD) Bridge Design Specifications [2] for several
bridge types. There are no approximate formulas provided for spread slab beams;
part of this study is to determine whether the spread box beam formulas might also
be applicable to spread slab beams.
2 Full-Scale Bridge Construction and Field Testing

A detailed parametric study was conducted to investigate the potential benefits of using spread slab beam bridges and to develop preliminary designs for alternative design parameters and geometries. The preliminary designs were carried out following the AASHTO LRFD Specifications and TxDOT Bridge Design Manual [3]. All geometric combinations listed in Table 1 were investigated to determine the maximum span length versus number of strands provided. More detailed design procedures may be found in [4, 5]. For the parametric study, each slab beam was designed based on a given number of strands. The maximum achievable span lengths for eight different limit states were calculated at each step. The considered limit states include tension and compression stress limits at release, at the time of deck placement, and at service; along with ultimate flexural strength and the live load deflection limit at service.

Figure 2(a) shows an example chart, where the eight curves shown indicate an upper bound span length solution for the limit state considered and blue shaded region is the feasible solution domain. Figure 2(b) summarizes the maximum achievable span length for all the investigated bridge geometries versus the beam spacing. The results of the parametric study indicated that the beam depth is the most prominent parameter for achieving longer span lengths, and in general, smaller beam spacing results in a greater span length.

Table 1. Alternative Geometries and Design Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description/Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Beams</td>
<td>3, 4, 5, or 6</td>
</tr>
<tr>
<td>Bridge Width, (w) (m)</td>
<td>7.9, 9.1, 10.4, 12.2, 12.8, 14</td>
</tr>
<tr>
<td>Slab Beam Type</td>
<td>4SB12, 4SB15, 5SB12, 5SB15</td>
</tr>
<tr>
<td>Clear Beam Spacing</td>
<td>Varies from 0.6 to 1.8 m</td>
</tr>
<tr>
<td>Deck Thickness, (t_s)</td>
<td>200 mm</td>
</tr>
<tr>
<td>Precast Concrete Strength at Release, (f_{ci})</td>
<td>41 MPa</td>
</tr>
<tr>
<td>Precast Concrete Strength at Service, (f_c)</td>
<td>59 MPa</td>
</tr>
<tr>
<td>Deck Concrete Strength, (f_{cd})</td>
<td>28 MPa</td>
</tr>
<tr>
<td>Prestressing Strand Diameter, (d_p)</td>
<td>12.7 mm</td>
</tr>
</tbody>
</table>

The experimental part of the research included building a full-scale spread slab beam bridge and testing it under service loads to evaluate the performance and determine experimental LLDFs. A challenging spread slab beam bridge geometry with a relatively large beam spacing and the longest possible span length for this spacing was designed and constructed. The bridge has 14.2 m total span length, 10.4 m width and utilizes four standard TxDOT slab beams (5SB15: 1.5 m wide 380 mm deep). All 56 strand locations within the 5SB15 slab beam section were used to meet the tension stress limit at service. This aggressive design for this bridge system introduced several design and construction challenges including requirements for
interface shear reinforcement and higher than predicted camber. Figure 3 depicts the elevation view drawings and photograph together with slab beam erection process.

Figure 2. Preliminary design summary for considered spread slab beam geometries

Figure 3. Description and construction of the Riverside Bridge
The bridge was instrumented to better observe the behaviour under service loading. One objective was to evaluate the shear LLDFs and was achieved by placing load cells at both ends of each slab beam. Another important response for designing the prestressed girders is the distribution of moments between girders. The maximum moment reactions was calculated from the data measured by strain gages attached at midspan of each beam and also by measuring the deflection curve along the length of each beam using a total of 40 string potentiometer. In addition a total of eight accelerometers were attached on the bottom surface of the slab beams to capture dynamic properties of the bridge.

For the static tests the vehicles (a dump truck or a water tanker) were placed at the critical position for creating the maximum moment effect, and they were placed at a member depth away from the centerline of the bearing pads for creating the maximum shear effect. Transverse positions were determined for creating critical loading when two lanes are loaded. Two alignment couples were defined by considering the minimum distance between vehicles as 1.2 m. A total of four transverse alignments were loaded where one alignment couple created the critical loading for an exterior girder and the other created critical loading for an interior girder. Figure 4 presents moment and shear LLDFs when the exterior girder critical alignments are loaded with dump trucks.

Experimental shear LLDFs for both interior and exterior girders were about 5 percent higher when the Bridge was loaded with the rear axle of the water tanker compared to the dump truck loading. This may be due to more concentrated loading achieved with the water tanker. Experimental moment LLDFs were similar for both the dump truck and the water tanker loadings.

The observed bridge responses under dynamic loads were larger when compared to the static counterparts. Evidently, for short-span bridges, the dynamic impact may exceed the AASHTO LRFD Specifications design value of 33 percent. However, the observed impact depended upon the position of the approach bump as well as the dynamic characteristics of the vehicle and the bridge.

![Figure 4. LLDFs measured during static loading of the Riverside Bridge](image-url)
3 Computational Modelling

3.1 Model Descriptions

The experimental results obtained from the field testing of the Riverside Bridge were used to investigate different modelling approaches. These modelling techniques include, grillage analysis and the finite element method (FEM). Moment and shear predictions from computational models were compared with experimentally obtained values. The FEM modelling technique, which gave good agreement with the test results, was then utilized for further investigation in the parametric study for developing moment and shear LDF formulas.

Grillage analysis was first introduced by Lightfoot and Sawko [6] and provides a simplified approach by reducing the number of degrees of freedom. Grillage analysis is historically the most basic type of computational modelling technique for analysing slab and beam bridges. This method idealizes the bridge superstructure by assuming that it may be represented by a mesh of frame elements in each of the two orthogonal directions. The Riverside Bridge grillage model was developed following the guidelines provided by Hambly [7] and Zokaie et al. [8].

FEM provides a powerful and versatile computational approach for modeling the exact geometry of the bridge necessitating very few simplifying assumptions. A 3D finite element model that uses solid brick elements enables representation of the correct bridge geometry including the vertical positions of the boundary conditions. Two different commercial software were utilized to compare analysis accuracy. One of them is Abaqus, which is a general purpose FEM software for solving a broad range of engineering problems. The second one is CSiBridge, which is more specific to bridge engineering. Figure 5 presents the developed grillage and finite element models of the riverside bridge with an example loading condition.
Three-dimensional eight-node solid brick elements with three degrees of freedom at each node were utilized for the FEM models. The slab beams were seated on bearing pads at the support locations. One sample bearing pad was tested under cyclic axial load and the vertical stiffness at the supports were assigned based on test results. The lateral stiffness of these pads were calculated based on the manufacturer provided shear modulus. The rotational stiffness were taken as zero due to the load cell setup, which is very close to ideal pin support conditions.

3.1 Results of Computational Analysis

Figure 6 shows the deflection fields obtained from the Abaqus and CSiBridge FEM software when the dump truck was located at two different alignments. The experimentally observed deflections were compared to those predicted by the two commercial FEM programs. The comparative results indicate that both FEM models can predict the deflection profiles reasonably well. The maximum difference between the measured and predicted deflections is 0.25 mm for Abaqus and 0.3 mm for CSiBridge results. It should be noted that string potentiometers work best within a 0.125 mm resolution.

![Deflections – dump truck on Alignment 1](image1)

![Deflections – dump truck on Alignment 2](image2)

Figure 6. Comparison of experimental deflection profiles with FEM results

It is important to accurately model the dynamic characteristics of the bridge to ensure it is properly modelled by the FEM software. The experimentally observed modal properties were compared with the FEM predictions of Abaqus and CSiBridge software. Figure 7 shows the mode shapes obtained from FEM analysis and their comparison to the experimentally derived ones. Table 2 lists the experimental and computational natural frequencies for the first three modes. The predicted natural frequencies and mode shapes from both programs are in good agreement with the test results.
Figure 7. Comparison of computational and experimental mode shapes

Table 2. Experimental and Computational Frequencies.

<table>
<thead>
<tr>
<th>Description</th>
<th>1&lt;sup&gt;st&lt;/sup&gt; Mode (Hz)</th>
<th>2&lt;sup&gt;nd&lt;/sup&gt; Mode (Hz)</th>
<th>3&lt;sup&gt;rd&lt;/sup&gt; Mode (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>5.5</td>
<td>8.2</td>
<td>13.8</td>
</tr>
<tr>
<td>Abaqus</td>
<td>5.6</td>
<td>8.3</td>
<td>14.6</td>
</tr>
<tr>
<td>CSiBridge</td>
<td>5.9</td>
<td>8.6</td>
<td>14.8</td>
</tr>
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</table>

Maximum moment and shear responses of each slab beam for moment and shear critical longitudinal positions of the dump truck were estimated using grillage and FEM models. The lateral distribution of live loads between girders were then calculated from these moment and shear estimates. The maximum of these responses controls the design of an interior or exterior beam. Maximum values are plotted as bar charts in Figure 8 for visual inspection of the accuracy of the computational methods. It is evident that the grillage model provides slightly conservative estimates for critical moment results, whereas the FEM model estimates for moments are slightly unconservative. Shear predictions obtained from both FEM programs and grillage analysis are in close agreement (within 5 percent) with the test results for most of the maximum shear cases.
4 Conclusions

Forty-four variations of spread slab beam bridge geometries were designed using standard TxDOT slab beam types to determine the feasible design space. One of the most aggressive designs with widely spaced slab beams was constructed at full-scale. Alternative modeling approaches including finite element analysis, and grillage analysis were evaluated.

Code-based preliminary parametric designs indicated that it is a safe approach to provide two beams per design lane as a rule of thumb. A smaller beam spacing always results in a longer span length, while beam depth and beam width have a more prominent effect on the maximum span length as compared to the number of beams.

Field testing of the bridge revealed that the spread slab beam bridge is a viable superstructure option which performed linearly under service loads up to 334 kN total vehicle load. The observed bridge responses under dynamic loads were larger when compared to the static counterparts exceeding 33 percent in some cases.

Deflection predictions obtained from both FEM analysis programs show moderately good agreement with the experimental results. At the location of maximum deflection the difference was 0.25 mm with an associated 0.125 mm accuracy in experimental measurements.

Estimated natural frequencies from both FEM programs were very close to the experimentally observed results. Mode shapes obtained from the FEM models also compare well with the mode shapes inferred from experimental observations.

Moment and shear LDFs calculated from the moment and shear predictions of both FEM programs were in good agreement with the test results. When carefully developed, the grillage model also predicts the moment and shear response accurately.
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