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# Assessing Bridge Conditions by Performing Static Load Tests

**Daniel Wendichansky, Ph.D.**
Proffesor, Department of Civil Engineering and Surveying, University of Puerto Rico at Mayagüez, Mayagüez, Puerto Rico, daniel@uprm.edu

**Yazmin Seda Sanabria, M.S.C.E.**
Research Engineer, US Corps of Engineers, Vicksburg, Mississippi, United States, sedasay@wes.army.mil

**Jorge L. Ayala Burgos, M.S.C.E.**
Ph.D Student, Department of Civil Engineering and Surveying, University of Puerto Rico at Mayagüez, Mayagüez, Puerto Rico, jabpr@yahoo.com

## Abstract
The principal objective of this work was the development of an experimental-analytical methodology that can be used by field engineers to determine bridge condition by performing static load tests. The methodology proposed and employed for this study is conceptually straightforward and can be stated as it follow: If experimental displacement values are obtained during static load tests at different bridge locations, and if a simple analytical model is constructed for this structure, then the set of bridge properties values that provide the best fitting can be found using some appropriate optimization technique. The analytical models were constructed using the finite difference method. Comparisons between the results obtained using the proposed methodology and the ones obtained following the procedure specified by the American Association of State Highway and Transportation Officials (AASHTO) was performed. As the result of this investigation it was concluded that the proposed methodology can predict with enough accuracy not only the displacement but also the forces acting in existing bridges when they are under static loads. The main advantage of the proposed methodology is that it is possible to find the displacements and the load carried by each girder by using only the commercial version of Excel.

## Keywords
Bridge, Distribution Factor, Finite Difference

## 1. Introduction

It is well known, that one of the most common functionally inadequate structural components of the bridges, are the bridge decks. The causes of a bridge deck's deterioration can be attributed to the sum of different factors such as deficiency in construction practice, overloads, amplification of the dynamics forces due to poor approach slabs, etc. Since budgetary restraints limit, the replacement of the deficient bridge decks has become more difficult. Therefore there is a general feeling that the direction and purpose at this time should be to initiate and develop procedures for extending the service life of the existing bridge components. Extending the service life of the existing bridge decks, will require in a first step to identify which of the existing bridge are in good condition and which ones require to be replaced. The problem of identifying the most vulnerable bridges, becomes more complicated if there is scarce structural information available to be used during the bridge evaluation or as it is commonly known "rated".

The principal objective of this work was therefore the development of a very simple experimental-analytical methodology that can be used by field engineers to determine bridge girder conditions by performing static load tests. The methodology proposed here do not intent to replace the classical approach used for bridge rating, but provide an alternative approach when there is practically no information about the properties of the bridge under study. The methodology developed and employed in this study is conceptually straightforward and can be stated as it follow: If experimental displacement values are obtained during static load tests at different bridge locations, and if a simple analytical model is constructed for this structure, then the set of bridge properties values that provide the best fitting can be found using some appropriate optimization technique. The proposed methodology presented in this work was used to determine the load carried by each beam of the bridge superstructure. Having this information, the field engineers can easily obtain the bridge rate of this structural component.

The vertical loads acting over the bridge decks; are not only their self weight but also the loads produced by the passing vehicles. The self weight of the bridge decks and their effect over some of the structural members can easily obtained by using the geometric characteristic of the bridge. The most complicated part of the problem is how the load produced by the passing vehicles affect certain structural elements. The design approach to find this effect has been for many years through the use of the live load distribution factor. The live load distribution factor from a design point of view pretend to reduce a three dimensional problem to a two dimensional problem. Therefore the analysis of a multiple girder deck superstructure is reduced with the introduction of live load distribution factor to the analysis of single girders.

## 2. Experimental Tests

In order to validate the methodology proposed here-in, an experimental program was designed to collect field data from four existing bridges. This experimental program is well described in reference Ayala (2004). Two of the tested bridges are located in the western portion of the Puerto Rico Island and the other two in the state of Virginia. Of the two bridges located in the Puerto Rico Island one in located in the municipality of Rincón and the other in the municipality of Aguada. **The Rincón Bridge** was designed in 2001 in accordance with the AASHTO 1996 Specifications for a HS30-44 Truck. The bridge was under construction at the moment that this project started, therefore, it was possible to measure the strain in the concrete and in the reinforcing bars during the test by installing strain gages prior to pouring the concrete mix. The Rincón Bridge is a slab bridge of two spans of 10.10 m each, for a total length of 20.20 m. These spans are supported by 1 pier (shear wall type) and two abutments. Each support (pier or abutments) has been built monolithically with the slab. The thickness of the slab varies across the 13.40 m width of the bridge this variation goes from 0.55 m at the edges to 0.75 m at the center line. Since this bridge is located close to the sea, a concrete compressive strength of 34474 KPa (5 Ksi) and a minimum concrete protection of 0.08 m was specified. The Aguada Bridge was designed in 1958 for an H15-44 using the 1953 AASHTO Specifications for the Design of Bridge Structures. **The Aguada Bridge** is a simple-span slab on concrete girder structure, it carries one lane of traffic. The bridge has a total length of 12.00 m and a total width of 4.44 m. The superstructure of this bridge consists of a concrete slab with a thickness of 0.15 m and 3 concrete beams spaced 1.82 m (center to center of beam), the three beams are simply supported at the abutments. The design compressive strength of the concrete was 20684 KPa (3 Ksi). The experimental tests of Virginias's bridges were carried out by the U.S. Corps of Engineers. The **Franklin County Bridge** carries Virginia Route 697 over Mill Creek in Franklin County near the town of

Rocky Mount. This three simple span bridge was built in 1979, rests on steel bearing pads, and has no skew with the roadway. The plan drawings indicate that the T-beams were constructed of concrete with a compressive strength of 27579 KPa (4 ksi) and yield strength equal to 413685 KPa (60 ksi) steel reinforcing bars. The **Patrick County Bridge** carries VA Route 40 over Little Widgeon Creek near the town of Woolwine, Virginia in Patrick County. This one span structure was constructed in 1947 and also has no skew with the roadway. However, unlike the Franklin County Bridge which is on a relatively straight stretch of road, the Patrick County Bridge is on a rather sharp curve. Another difference between the two bridges lies in the bearing details. The Patrick County Bridge super-structure bears directly on the abutment shelf instead of steel bearing pads. The compressive strength of the concrete and yield strength of the reinforcing steel used to construct this bridge were unavailable from original design drawings.

Instrumentation used in the test carried out in the Rincón Bridge consisted of: 1) linear potentiometers to measure the vertical displacements in the bridge slab, 2) strain gages on the steel reinforcement and on the concrete to measure strain at various locations on the bridge. The instrument used in Aguada Bridge consisted of: 1) linear potentiometers to measure the vertical displacements in the girders of the bridge, 2) clip gages to measure strain in the surface at various locations. The Franklin County Bridge and the Patrick County Bridge instrumentation consisted of: LVDTs located at midspan of all four beams in span one.

All of the bridges were tested under static loads, by using typical construction trucks. Before the test begins, the trucks were weighted using a special scale which was able to provide information about the weight of each wheel. The trucks used for testing the bridges, were parked during the test at different locations in order to simulate different scenarios. Each time that the trucks were parked at a different location generate a load cases. During the tests, the truck enters to the bridge slab at approximately 2mph and it was parked at the pre-assigned location. When the truck was in place it stay in the position for at least 1 minute. After this time measurement were taken and the trucks were ordered to leave the bridge. This procedure was repeated but parking the trucks at different location. The objective of the tests was obtaining the displacements in the bridge slab and girders to validate the mathematical approach proposed here. For the Rincón and the Aguada bridges a number of four load cases were carried out. The Rincón Bridge was tested using two dump trucks loaded with granular material to obtain the desired weight. The weights of the trucks for this bridge were: total weight of Truck A 385.7 KN (86.7 kips) and total weight of Truck B 389.7 KN (87.6 kips). The Aguada Bridge was tested using one unloaded dump truck. The total weight of the truck was 116.5 KN (26.2 kips). For the Franklin and for the Patrick County bridges a total of two load cases were used in each bridge. Case 1 corresponds to the condition when the truck wheels were located over the edge beam of the bridge and case 2 when the truck wheels were located over the center beams. The VDOT dumb truck used for testing the Franklin County Bridge weighted 222.4 KN (50 kips). The Patrick County Bridge was tested using a procedure similar to the procedure used on the Franklin County Bridge with one exception. For the static crossings, the wheel lines of the truck were placed 0.305 m from the curb. Because of the bridge geometry, it was physically impossible to center the wheel line over the exterior girders. The VDOT dumb truck used in this bridge weighted 221.3 KN (49.76 kips).

## 3. Mathematical Model – The Approach of using Finite Difference Method (FDM)

The mathematical approach used in this project uses a numerical tool to determine the vertical displacements and loads in the bridge slab. The FDM is a numerical alternative to solve differential equation. Using this approach it is possible to obtain displacement or stress resultant for each node in a structural model. If the value of the function and its derivative in a state is known, values of the function can be obtained for neighboring states. The method relates the displacements of each node to the external applied loads using a finite difference equation where the displacement is unknown. In this project an analysis of sensibility was carried out to determine a suitable mesh size for the FDM approach. Using the mesh size obtained from the sensibility tests the use of an Excel worksheet was justified.

The conventional program knows as Excel was used in this work in order to avoid unnecessary complication in the analysis of bridges. Excel has some limitations regarding the size of the stiffness matrix. The regular version of EXCEL invert matrix which are not greater than 51x51, for that reason a simple 49x49 stiffness matrix and a simplified bridge model was used. Table 1 presents a comparison between the 49x49 stiffness matrix using a worksheet in Excel with the 325x325 stiffness matrix using a program developed in Matlab, the displacements obtained by the worksheet in Excel are practically similar to those obtained with the program in Matlab. This shows that the use of Excel to obtain the live load distribution factors is acceptable.

**Table 1: Excel predictions vs. Matlab predictions**

| **Slab Simply Suported at Two Parallel Edges and a Load of 100Kips in the Center** | | | |
|---|---|---|---|
| Location | Excel [cm] | Matlab [cm] | % Error |
| Center | 27.2839 | 26.4947 | 3.0 |
| Edge | 27.0828 | 26.4084 | 2.6 |

For constructing the FDM model only the geometric information such as, the slab thickness, beam dimensions, span length, and the space between beams is needed. If this information is not available, it can be extracted easily from the field. Having the geometric information the only unknown of the problem is the stiffness of the deck system represented by the product of the inertia by the modulus of elasticity of the material. This value as it can be seen later it can be preliminarily assumed. Using the geometric information and a preliminary value for the stiffness, a simplified FDM model can be created.

## 4. Proposed Approach for Finding the Distribution Factor

Having created a simplified model which can be solved using a well known EXCEL spreadsheet and obtained experimental information from a typical bridge static loads tests is it possible to state the methodology as it follow: 1) Using the mathematical model as described above and with the location and value of the load acting over the bridge deck obtained from the experimental tests, compute as a first trial the vertical displacements of the bridge at different locations. These values obtained using the mathematical prediction was later compared with the values location where instruments were installed 2) compute the error between the experimental deformations and the analytical prediction. 3) Using the EXCEL solver and the moment of inertia of the elements as a variable, proceed using a least square approach to minimize the error obtained in step 2 by changing their moment of inertia. The method obtains some moments of inertia that make the displacements in the bridge converge to the experimental displacements. The moments of inertia had some limits. The maximum is the moment of inertia with the contribution of the steel reinforcement ($\boldsymbol{I_{gt}}$) if the information of the steel reinforcement is unavailable use 1.3 times the gross moment of inertia ($\boldsymbol{I}$) and as minimum a 0.5 percent of the gross moment of inertia ($\boldsymbol{I}$). 4) After having minimized the error between the analytic prediction and the experimental results compute the live load distribution factor in each beam of the bridge. To do so compute first the reaction by using the FDM approach and later the distribution factor by using equation 1 which represents the reaction in each beam divided the truck weight (the sum of the reactions in the beam) result in the distribution factor for each beam.

$$DF_{FDM} = \frac{R_{Bn}}{\sum R_{Bn}} \qquad \text{(Eq. 1)}$$

Where $\boldsymbol{R_{Bn}}$ are the reactions in the beam $\boldsymbol{n}$ due to live load.

## 5. Application of the FDM Approach

The experimental displacement values obtained at different location of the Rincón Bridge and the Aguada Bridge were used to evaluate the error in predicting the displacement using the proposed methodology. The 8 points where displacement transducers were installed in the Rincón Bridge and the 8 point in the Aguada Bridge were used at the same time during the minimization process. Table 2 compares the experimental displacements at the center of the slab vs. the FDM predictions for the Rincón Bridge.

**Table 2: Exp. Displacements vs. FDM Predictions (Maximum Displacements, Center of the Slab)**

| | LOAD CASE | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | **1** | | **2** | | **3** | | **4** | |
| | Displacement [mm] | | Displacement [mm] | | Displacement [mm] | | Displacement [mm] | |
| | Exp. | FDM | Exp. | FDM | Exp. | FDM | Exp. | FDM |
| P1 | 1.0335 | 1.0008 | 1.3102 | 1.2395 | 1.4301 | 1.3995 | -0.2998 | -0.2667 |
| P2 | 0.9997 | 0.9576 | 1.2347 | 1.1887 | 1.3361 | 1.3437 | -0.2585 | -0.2642 |
| P3 | 0.9176 | 0.8306 | 1.1006 | 1.0262 | 1.1792 | 1.1506 | -0.2586 | -0.2540 |

The maximum % error obtained for the displacements was 11% for the potentiometer 1 in the load case 4, the rest of the error were less than 10%.

Table 3 presents the experimental displacements vs. the analytical predictions using FDM for the Aguada Bridge using an f’c = 27579 KPa (4 Ksi) for the concrete in the superstructure of the bridge.

**Table 3: Exp. Displacements vs. FDM Predictions (Maximum Displacements, Center of the Slab)**

| | LOAD CASE | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | **1** | | **2** | | **3** | | **4** | |
| | Displacement [mm] | | Displacement [mm] | | Displacement [mm] | | Displacement [mm] | |
| | Exp. | FDM | Exp. | FDM | Exp. | FDM | Exp. | FDM |
| P1 | 0.2321 | 0.2108 | 0.1706 | 0.1803 | 0.1081 | 0.1194 | 0.1246 | 0.1321 |
| P2 | 0.2574 | 0.2438 | 0.2406 | 0.2388 | 0.1432 | 0.1600 | 0.2295 | 0.2540 |
| P3 | 0.2521 | 0.2184 | 0.2644 | 0.2311 | 0.1802 | 0.1575 | 0.3557 | 0.3302 |

The maximum % error obtained for the displacements was 13% for the potentiometer 3 in the load case one, 12% were obtained for the potentiometer 3 in the load case 2 and 3, 11% was obtained for the potentiometer 3 in the load cases 3 and 4, the rest of the error are less than 10%.

In the Franklin County Bridge only four instruments were installed during the test. The instruments were intalled at the center of the beams: Table 4 presents a comparison between the experimental results vs. the analytical predictions for the Franklin County Bridge using an f’c = 34474 KPa (5 Ksi) for the concrete in the superstructure. Figure 3 presents a comparison between the experimental measures and the analytical predictions for the load case 2 (truck above the interior girders). Equation 3 presents the percent of error multiplied by an importance factor calculated using the experimental results. This importance factor was introduced to force to the least square minimization process to gives more weight to those beams which have larger displacements. This importance factor has values between cero and one, for example for the beam with the maximum experimental displacement the importance factor is equal to 1, which means that the percent error doesn’t need to be reduced.

$$\%Error_{Bn} = \frac{FDM_{Bn} - EXP_{Bn}}{EXP_{Bn}} \times 100 \times \left( \frac{EXP_{Bn}}{EXP_{\max}} \right) \qquad \text{(Eq. 3)}$$

Where $FDM_{Bn}$ is the analytical prediction for the beam $n$, $EXP_{Bn}$ is the experimental displacement lecture for the beam $n$, and $EXP_{max}$ is the maximum experimental displacement lecture of the beams.

**Table 4: Experimental Results vs. FDM Approach (Maximum Displacement, Center of the Slab)**

| | **LOAD CASE** | | | | | |
|---|---|---|---|---|---|---|
| | **1** | | | **2** | | |
| | **Maximum Displacement [mm]** | | | | | |
| Beam | Experimental | FDM | % Error [%] | Experimental | FDM | % Error [%] |
| 1 | -0.8890 | -0.9796 | 10.19 | -0.3175 | -0.3719 | 6.76 |
| 2 | -0.7590 | -0.7051 | 6.06 | -0.8052 | -0.7486 | 7.03 |
| 3 | -0.3353 | -0.3454 | 1.14 | -0.6833 | -0.7486 | 8.11 |
| 4 | -0.1036 | 0.0033 | 12.03 | -0.3454 | -0.3719 | 3.29 |

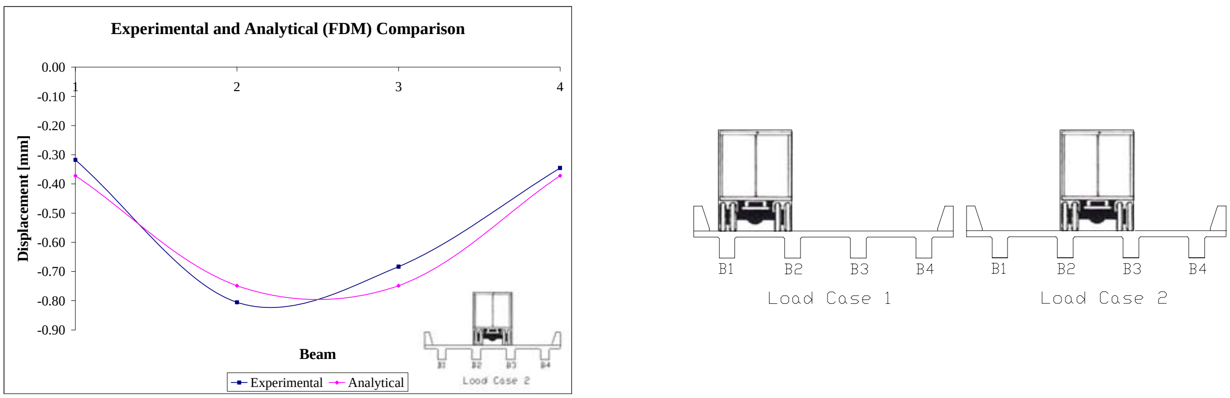
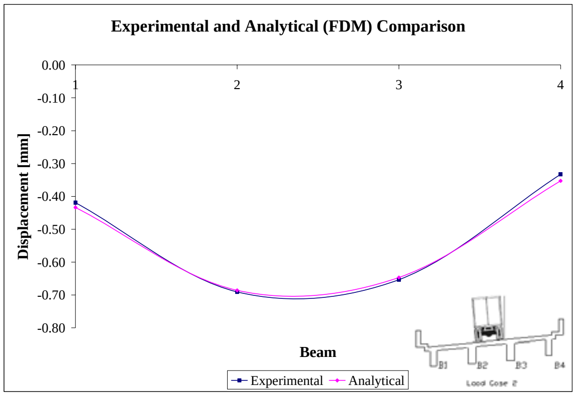


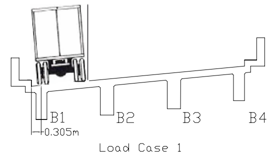

**Figure 3: Experimental vs. Analytical Comparison**

The same approach used in the Franklin County Bridge was used in the Patrick County Bridge. Table 5 presents a comparison between the experimental results vs. the FDM approach for the Patrick County Bridge. Figure 4 presents a comparison between the experimental measures and the analytical predictions for the load case 2 (truck above the interior girders).

**Table 5: Experimental Results vs. FDM Approach (Maximum Displacement, Center of the Slab)**

| | **LOAD CASE** | | | | | |
|---|---|---|---|---|---|---|
| | **1** | | | **2** | | |
| | **Maximum Displacement [mm]** | | | | | |
| Beam | Experimental | FDM | % Error [%] | Experimental | FDM | % Error [%] |
| 1 | -1.1430 | -1.1319 | 0.97 | -0.4191 | -0.4338 | 2.13 |
| 2 | -0.7620 | -0.7293 | 2.86 | -0.6909 | -0.6861 | 0.69 |
| 3 | -0.2957 | -0.3035 | 0.68 | -0.6536 | -0.6470 | 0.96 |
| 4 | -0.1036 | 0.0736 | 15.51 | -0.3327 | -0.3528 | 2.91 |



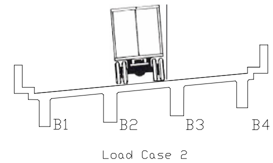




**Figure 4: Experimental and Analytical Comparison (Load Case 2)**

### 5.1 Comparison of Live Load Distribution Factors

The accepted procedure for determining transverse live load distribution factors for concrete bridges for design purpose is outlined in the AASHTO Standard Specification (SS) and in the AASHTO Load Resistance Factor Design (LRFD) Standard Specifications [AASHTO 1996 and AASHTO 1998]. The procedure described in the AASHTO provides simple and conservative formulas for live load distribution factors for interior and exterior beams. Table 6 shows some of the formulas used by AASHTO.

**Table 6: AASHTO Live Load Distribution Factors for One Lane Loaded**

| **.AASHTO Live Load Distribution Factors** | | |
|---|---|---|
| **Interior Beams** | | |
| | SS | LRFD |
| One Lane Loaded | S/6.5 | $0.06+(S/14)^{0.4}(S/L)^{0.3}(Kg/12.0Lts^{3})^{0.1}$ |
| **Exterior Beams** | | |
| | SS | LRFD |
| One Lane Loaded | Lever Rule | Lever Rule |

$$K_g = n\left(I + Ae_g^{\ 2}\right) \qquad \text{(Eq. 4)}$$

Where ***S*** is the beam spacing, ***L*** is the span length, ***n*** is the modular ratio of beam concrete to deck concrete (Eb/Ed), ***I*** is the gross moment of inertia, $\boldsymbol{e_g}$ is the distance between c.g's of deck and concrete T-beam stem, $\boldsymbol{t_s}$ is the slab thickness, and ***A*** is the cross sectional area of T-beam stem. If S > 1.83 m (6 ft) in the SS assume the flooring between stringers acts as a simple beam with the load on each stringer being the wheel load reaction. For the exterior beams use the Lever Rule. The Lever Rule is a method of computing the distribution factor by summing moments about the first interior girder to get the reaction at the exterior girder, assuming there is a rotational hinge in the bridge deck directly above the first interior girder.

The following tables present a comparison of the distribution factors obtained analytically with the distribution factors obtained using the AASHTO Specifications and the experimental distribution factors based on displacement measures. To obtain a simple experimental distribution factor based on displacement measures the following equations is proposed.

$$DF_{EXP} = \frac{\delta_n}{\sum \delta_n} \qquad \text{(Eq. 5)}$$

Where $\delta_n$ is the maximum displacement at the beam $\boldsymbol{n}$.

**Franklin County Bridge:** Table 7 presents the analytical and experimental distribution factor for computed for each the beams of the Franklin County Bridge (load case 1 and 2). Notice that the sum of the distribution factor for each beam is equal to 1

**Table 7: Distribution Factors, Analytical ($DF_{FDM}$) and Exp. Values ($DF_{EXP}$) - Load Case 1 and 2**

| | Distribution Factor | | | |
|---|---|---|---|---|
| | Case 1 | | Case 2 | |
| **Beam** | **Analytical** | **Measured** | **Analytical** | **Measured** |
| B1 | 0.49 | 0.43 | 0.16 | 0.15 |
| B2 | 0.40 | 0.36 | 0.34 | 0.37 |
| B3 | 0.11 | 0.16 | 0.34 | 0.32 |
| B4 | 0.00 | 0.05 | 0.16 | 0.16 |

A comparison between the AASHTO Distribution Factors, the measured distribution factors using the displacement lectures ($DF_{EXP}$), and the maximum analytical distribution factors ($DF_{FDM}$) for exterior and interior beams are presented in the Table 8.

**Table 8: Comparison of Distribution Factors Based on ASSHTO, Measured ($DF_{EXP}$), and Analytical ($DF_{FDM}$)**

| | Load Case 1 | | | | Load Case 2 | | | |
|---|---|---|---|---|---|---|---|---|
| | **Interior Girder [DF]** | | | | **Interior Girder [DF]** | | | |
| | **SS** | **LRFD** | **Measured** | **Analytical** | **SS** | **LRFD** | **Measured** | **Analytical** |
| One Lane | 0.57 | 0.50 | 0.36 | 0.40 | 0.57 | 0.50 | 0.37 | 0.34 |
| | **Exterior Girder [DF]** | | | | **Exterior Girder [DF]** | | | |
| | **SS** | **LRFD** | **Measured** | **Analytical** | **SS** | **LRFD** | **Measured** | **Analytical** |
| One Lane | 0.59 | 0.59 | 0.43 | 0.49 | 0.59 | 0.59 | 0.16 | 0.16 |

As expected the distribution factors obtained using the AASHTO Specifications are conservative compared with the real behavior of the bridge, for the studied cases. The AASHTO Specifications for the load case 1 has a minimum error of 39% and in the exterior girder a minimum error of 37% compared with the experimental lectures. For the load case 2 has a minimum error of 35% compared with the experimental lectures. The error between the analytical predictions and the experimental results for the load case 1 in the interior and exterior girder are 11% and 13% respectively and for the load case 2 in the interior and exterior girder the errors are 8% and 0% respectively.

**Patrick County Bridge:** Table 9 presents the experimental and analytical distribution factors of the Patrick County Bridge (load case 1 and 2).

**Table 9: Distribution Factors, Analytical ($DF_{FDM}$) and Exp. values ($DF_{EXP}$) - Load Case 1 and 2**

| | Distribution Factor | | | |
|---|---|---|---|---|
| | Case 1 | | Case 2 | |
| **Beam** | **Analitical** | **Measured** | **Analitical** | **Measured** |
| B1 | 0.56 | 0.50 | 0.20 | 0.20 |
| B2 | 0.38 | 0.33 | 0.32 | 0.33 |
| B3 | 0.10 | 0.13 | 0.33 | 0.31 |
| B4 | 0.03 | 0.04 | 0.16 | 0.16 |

Table 10 presents a comparison between the AASHTO DF, the measured distribution factors, and the maximum analytical DF for exterior and interior beams.

**Table 10: Comparison of Distribution Factors Based on ASSHTO, Measured ($DF_{EXP}$), and Analytical ($DF_{FDM}$)**

| | Load Case 1 | | | | Load Case 2 | | | |
|---|---|---|---|---|---|---|---|---|
| | **Interior Girder [DF]** | | | | **Interior Girder [DF]** | | | |
| | **SS** | **LRFD** | **Measured** | **Analytical** | **SS** | **LRFD** | **Measured** | **Analytical** |
| One Lane | 0.63 | 0.57 | 0.33 | 0.38 | 0.63 | 0.57 | 0.33 | 0.33 |
| | **Exterior Girder [DF]** | | | | **Exterior Girder [DF]** | | | |
| | **SS** | **LRFD** | **Measured** | **Analytical** | **SS** | **LRFD** | **Measured** | **Analytical** |
| One Lane | 0.63 | 0.63 | 0.50 | 0.56 | 0.63 | 0.63 | 0.20 | 0.20 |

The AASHTO Specifications for the load case 1 has a minimum error of 73% and in the exterior girder an error of 26% compared with the experimental lectures. For the load case 2 in the interior girder has a minimum error of 73% compared with the experimental lectures. The error between the analytical predictions and the experimental results for the load case 1 in the interior and exterior girder are 15% and 12% respectively and for the load case 2 in the interior and exterior girder the errors are 0% in both cases.

## 6. Conclusions

Based in achievement of the objectives of this work, the following conclusions can be inferred:

**Proposed Methodology**

1. The FDM is an acceptable method to obtain displacements and reactions for simply supported T-beam bridges. The displacements obtained using FDM compared with the experimental measures result in percent errors, in the majority of the cases, is lower than 10%.
2. The use of a worksheet in EXCEL is a simple an acceptable tool in the bridge analysis and the results obtained are relatively accurate as demonstrated in this paper.

**Distribution Factors**

1. The distribution factor obtained using an analytical approach ($DF_{FDM}$) and the values obtained using the experimental displacement ($DF_{EXP}$) in general do not differ in more than 10% for the load case 1 (truck above one exterior and one interior girder). For the load case 2 (truck above the interior girders) the error were 0% in most of the cases.
2. As expected, the distribution factors obtained by the code result in conservative values when compared with the analytical and experimental distribution factors. The error percent for the distribution factors obtained by the code were 39% in the interior girder and 37% for the exterior girder for the Franklin County Bridge. Similarly, Patrick County Bridge’s code-obtained distribution factors were 73% and 26% for the interior and exterior girders, respectively, for the first load case. For the second load

case, Franklin County Bridge's code-obtained distribution factors were 35% for the interior girder and in Patrick County Bridge 73% for the interior girder compared with the experimental distribution factors. Even though the use of the code distribution factor will produce conservative bridge girder designs, in the other side, the use of this distribution factors could result in the unnecessary replacement of an existing bridge deck.

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