Performance and Analysis of a Deck Girder Bridge

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ABSTRACT

The state-of-the-art in the use of HPC materials in bridge design and construction allowed the development of a new deck girder bridge system, fabricated using AASHTO girders with an integral deck. The current research project (and the present paper), funded by the North Carolina Department of Transportation and the Federal Highway Administration through the IBRC program, focuses on the field performance and the analytical study performed using finite element analysis software, as well as commercially available design software. A comparison of LRFD design calculations, measured values, and software model results define the actual and predicted performance of the new bridge system. This project provides a more comprehensive understanding on the performance of these deck girders, including load distribution, flange connection and girder diaphragm forces, and deck girder deformations.

Keywords: Deck girder, Pre-stress, Testing, Distribution factor, Impact factor, Shear stud, Finite element analysis, Connection, AASHTO, NCDOT

INTRODUCTION

No bridge construction of the deck girder systems using the standard AASHTO Type III girder currently exists in North Carolina. Additionally, research pertaining to this type of girder system is very limited. Therefore, an analytical and experimental study was conducted to evaluate design predictions. The deck girders and the connecting elements were modeled using bridge analysis software to yield values at the areas of concern. The contribution of certain elements to the structure as a whole, as well as the adequacy of these elements under loading were then determined. The accuracy of the existing design methods presented in the AASHTO LRFD Bridge Design Specifications 3rd Edition[1] were then validated.

The primary source of existing literature on deck girders were authored by Anderson[2], and Anderson et al.[3,4]. Other related information found contained discussions about pryout capacity and design criteria of headed studs. The parametric study "Deck Bulb Tees Using Standard AASHTO I-beams and the AASHTO LRFD Specifications" [5] was also reviewed to provide background information and concepts pertaining to the bridge design. Similar research by Stanton et al.[6] and Millam et al.[7] concerning HPC and deck girder behavior has been performed. An article concerning the effects of diaphragms on live load distribution was published in the Canadian Journal of Civil Engineering[8].

Connection elements in the deck girder system include embedded angles connected with a welded steel plate. The embedded angles are anchored into the concrete flange using two headed studs. This connection type currently does not have any design method published. The parametric study was conducted to reveal what forces are transmitted through this connection under different loading combinations. The connection was fabricated and tested to ultimate capacity in the UNC-Charlotte civil engineering laboratory in pure tension and longitudinal shear. Initial analyses of the connections were performed using the finite-based LARSA 2000© 4th Dimension[9], and a more precise analysis was completed using ANSYS© [10], a full finite element software package. The connection was modeled as tested in the laboratory and the welded plate was modeled as a component of the entire deck girder bridge. Results from these analyses were compared to computational and load test values from the deck girder design. Additionally, the amount of stress induced in the welded plates was also analyzed to check load and fatigue susceptibility.

The testing of the bridge required the use of two types of instruments: strain and displacement devices. The two loading conditions used for the testing were quasi-static and dynamic loadings. The girders were instrumented to report strains and displacements under multiple loading paths from the two tandem trucks. The connector plates and the diaphragms were also instrumented to provide strain readings resulting from the same truck paths.

ANALYTICAL STUDY

A current initiative by FHWA, the Innovative Bridge Research and Construction (IBRC) Program, focuses on new materials and technologies in bridge design and construction. Under this program, a recent project awarded to the North Carolina Department of

Transportation concentrates on a high performance concrete (HPC) deck girder system, comprising of a modified AASHTO Type III girder with an additional flange (deck) section. The objective of the present research project is to review the design and detailing information, monitor the deck girder fabrication and the bridge construction processes, and finally, load test the completed bridge.

LARSA© 3-D MODELING GEOMETRY

A parametric study was conducted using LARSA© 2000 4th Dimension for Bridges[9], a software developed for bridge analysis and design (this software was provided by LARSA Inc. free of charge for the duration of the present project). This program was chosen to facilitate the parametric study. The program allows the user to assemble the bridge elements as they are fabricated and constructed in the field. This feature allowed all loadings and restraints to be applied in a staged construction order. Once completed, the program reported results (such as, member stresses, camber due to prestress, etc...) that were comparable with the design hand calculations performed by the NCDOT[10].

The first step in completing the LARSA© model was defining the six different girder cross section types. The next step involved extruding the sections to the desired length from bearing to bearing. In this parametric study it was assumed that the bridge is simply supported at both ends (pinned-roller connection). This assumption was also used in the NCDOT design calculations[11]; however, the integral end walls will introduce negative moments close to the supports in the actual bridge. The load test comparisons, presented later in this paper, were made using a fixed-end deck girder version of the same LARSA© model.

A limitation of this model was the angle-to-steel plate connections. The current modeling techniques in LARSA© allowed the plates to be connected to the embedded angles only at the four corners, instead of the fillet weld connection to be performed in the field. Furthermore, the shear keys were not filled with a grout, connecting the deck girder flanges. These limitations resulted in a more flexible and weaker connection details, causing the analytical study to predict higher than anticipated forces in the connecting plates. Therefore, the LARSA© analytical results should be viewed as a qualitative analysis, rather than a quantitative evaluation.

ANSYS© 3-D MODELING GEOMETRY

The deck girder bridge was also modeled using the finite-element analysis program ANSYS©[11]. The ANSYS© program has the ability to provide a more detailed and representative analysis by incorporating a finer degree of precision in the smaller components of the bridge. The plate connectors were more accurately modeled and connected to the adjacent angles as constructed. The first step in creating the ANSYS© model was defining an equivalent girder cross-section. The cross-section used in the ANSYS© model had an equivalent moment of inertia and contained no angled lines, thus the entire model could be meshed with brick elements. The use of brick elements allows the user to define element patterns, whereas tetrahedral meshing is subject to automatic meshing within the program. The element pattern was created to allow the placement of the prestress tendons and the plate connectors, which otherwise would not be allowed with automatic tetrahedral meshing. Since the plate connectors were present in the deck girder flange brick elements were used for the entire cross section. The reinforced concrete was modeled using SOLID65, an element with cracking and crushing capabilities. The concrete material properties that were input in the program were taken from multiple cylinder tests completed at the time of the bridge load test. The ultimate strength of the HPC was found to be approximately 13,000psi. The A36 steel members (plates and diaphragms) were modeled using SOLID45, a standard 8-node multi-purpose element. The prestress strands were modeled using LINK8 elements. The cross-section of one of the five deck girders is shown in Fig. 1.

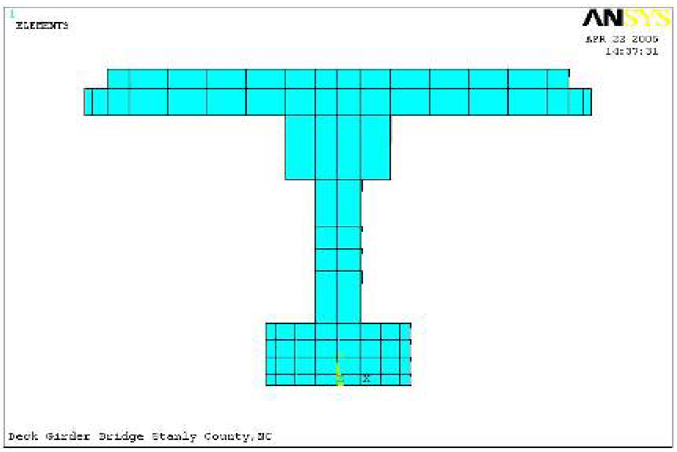

Fig. 1 Equivalent Deck Girder Cross-Section

The five deck girders were generated and joined by connector plates and the steel diaphragms. The model was then prestressed by placing tendons in the bottom flange of the deck girder. All five deck girders were assumed simply supported as designed by the NCDOT[11] to initially calibrate the model. The ANSYS© model was then verified by comparing the displacement and stresses at mid-span of a single deck girder. The comparison includes values obtained from the NCDOT design document[9] and the LARSA© model (see Fig.2 and 3).

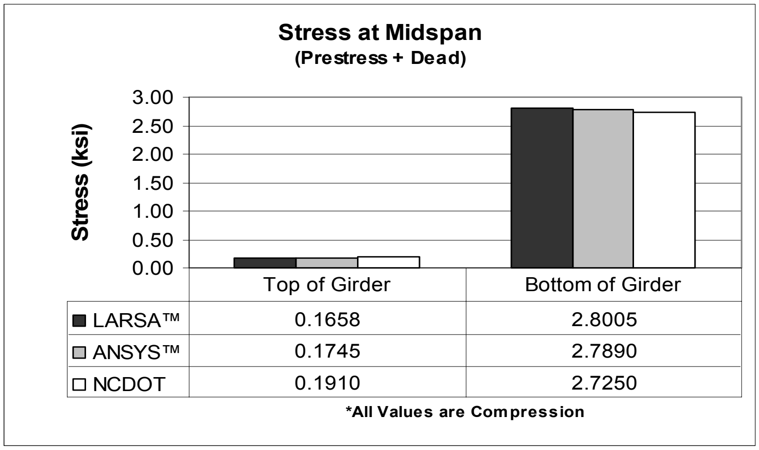

Fig. 2 Stress at Mid-span Comparison

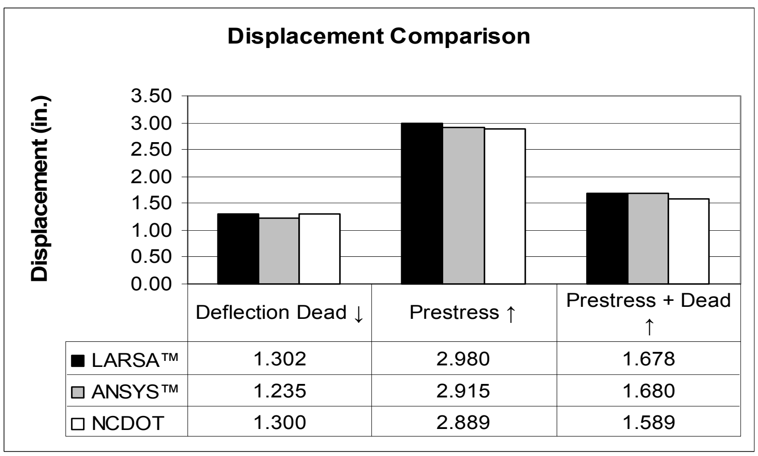

Fig. 3 Displacement at Mid-span Comparison

After the model was calibrated, the actual fixity contribution of the integral end bent was considered by fixing the deck girder areas incased by the end bents. The two trucks used in the actual load test were used in the model. The trucks were represented by an equivalent set of point loads due to the use of symmetry in the program to minimize computational time and memory usage. The equivalent axle loads provided the same moment and deflection values as the actual un-symmetric trucks used in the bridge test when applied to a simply supported deck girder. The equivalent trucks placed on the load paths that corresponded to the actual bridge load test are shown in Fig. 4. The trucks were placed at mid-span in order to agree with the program's symmetry line and to create near maximum moments and displacements in the deck girders.

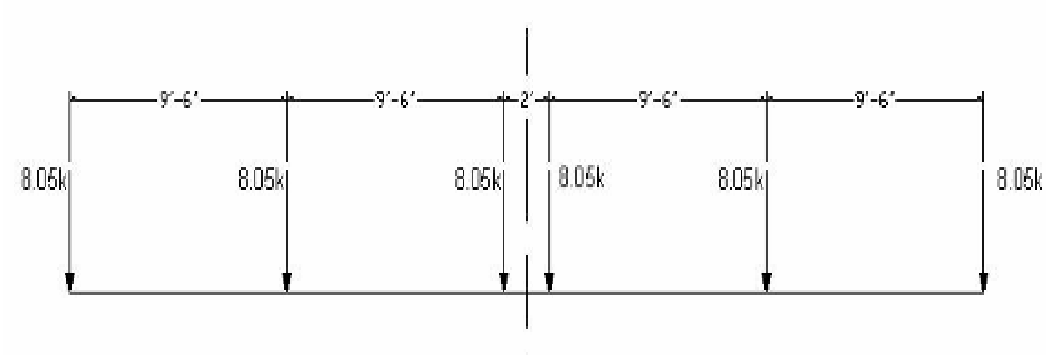

Fig. 4 Equivalent Truck at Mid-span

The ANSYS© model was then subjected to three of the five load paths that were used when the bridge was load tested. The total amount of load paths was not used due to the symmetry of the finite element bridge model. The quasi-static load testing procedure is discussed in greater depth in the bridge load test section of this paper. The five bridge test truck paths are shown in Fig. 5.

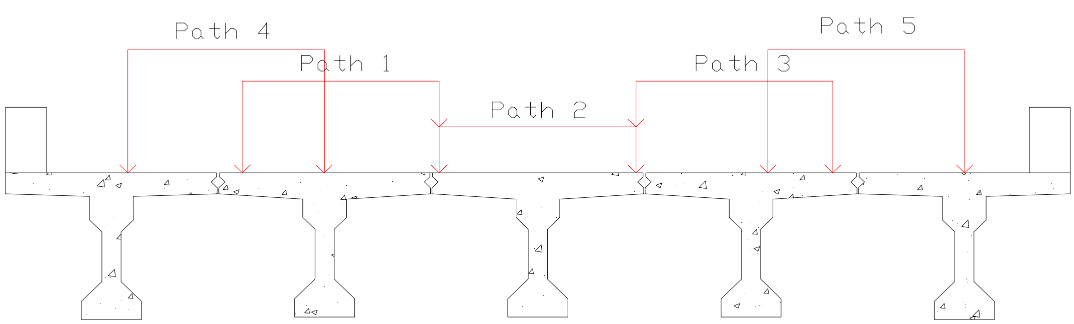

Fig. 5 Bridge Load Test Truck Paths

3-D MODELING RESULTS

The mid-span extreme fiber stresses in the five girders induced by path 2 live loading is shown for the actual load test and the two software models (see Fig. 6). The loading was arranged in the software models in order to produce the maximum girder stress, the maximum plate stress induced by this loading scheme was then found. The maximum displacement at mid-span for the actual load test and the two software models was also compared for each of the three truck paths. The LARSA© model was fixed at both ends to account for the integral end bent contribution for the comparison. The displacement values for path 2 are shown in Fig. 7.

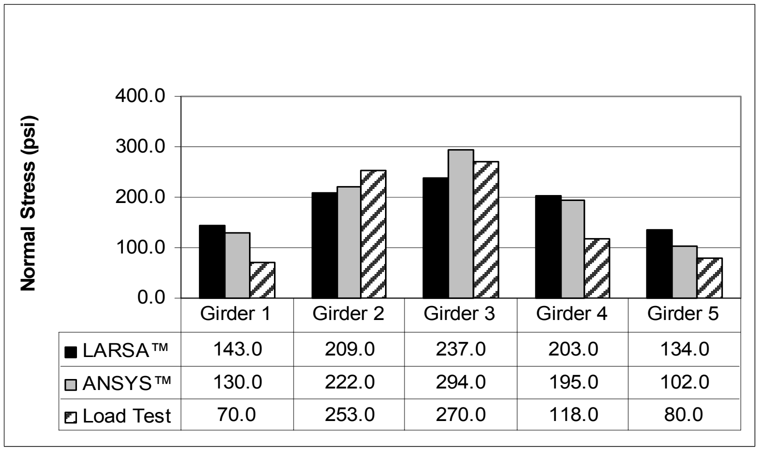

Fig. 6 Path 2 - Normal Stress Comparison (Tension)

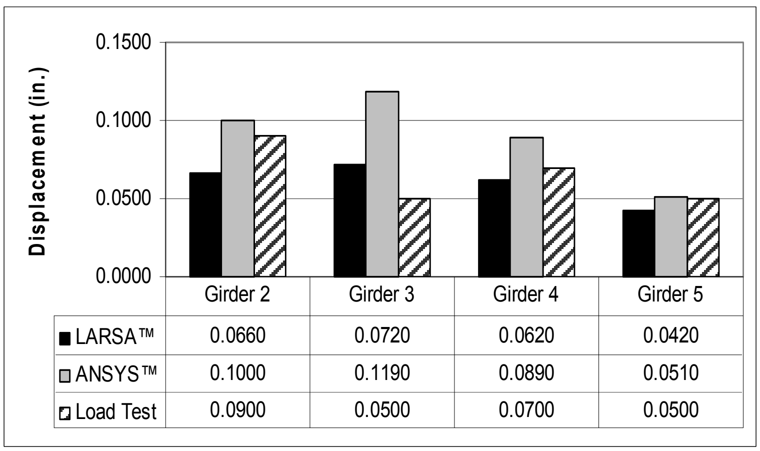

Fig. 7 Path 2 - Mid-span Displacement Comparison

The ANSYS© and LARSA© models produced results that were very comparable. The actual bridge however, proved to have a greater stiffness. This increased stiffness is evident in the lower deflection and stress values in all three loading paths. The asphalt overlay and the shear key components of the bridge were also ignored because the actual material properties for these components were unknown. The load paths that were completed during the actual bridge test may have slightly varied from the paths used in the software modeling, resulting in minor differences.

PLATE STRENGTH ANALYSIS

The plate connection stresses were determined to be maximum at the top extreme fiber in the transverse direction (perpendicular to traffic) according to the LARSA © model. The load test trucks and paths were modeled in the ANSYS© finite element program and the maximum transverse stresses were compared to the maximum values obtained from the load test, shown in Fig.8 and 9.

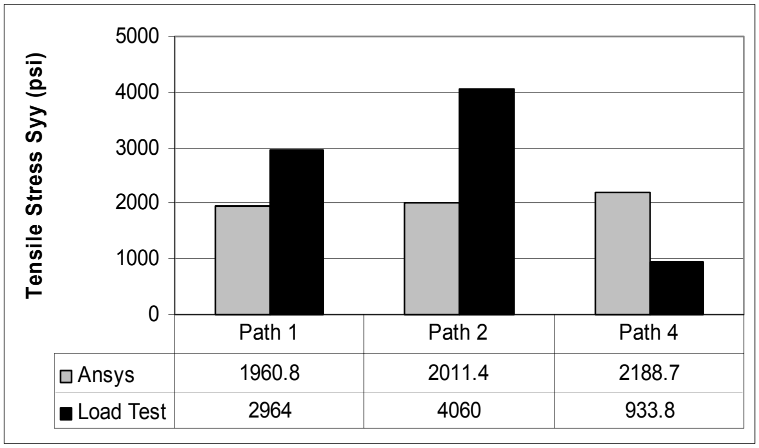

Fig. 8 Maximum Transverse Tensile Plate Stress

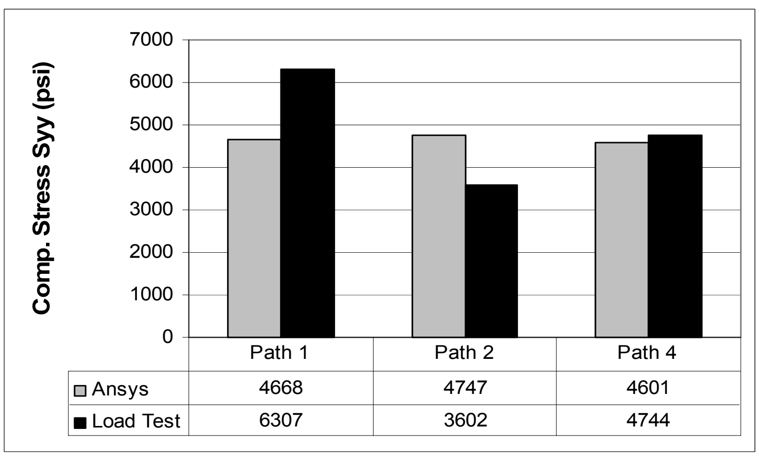

Fig. 9 Maximum Transverse Compressive Plate Stress

On average the plate stresses obtained in the actual load test were found to be higher than the ANSYS© model values. This increased stress in the connector plates could be attributed to the position of the trucks in the actual load test. Only one position from each load path was modeled in ANSYS©. ANSYS© does not have a moving load feature so the placement of the truck load was modeled to result in maximum girder stress and deflection values at mid-span. The maximum plate stress in a certain plate may not occur when the truck is in this position.

BRIDGE LOAD TEST

The finished fabricated girder is shown in Fig. 10 and the final deck girder placement on site is shown in Fig. 11. Please refer to the conference paper "New HPC Deck Girder System: Fabrication, Construction, and Behavior"[12] for the complete fabrication and construction of the deck girders.

Fig. 10 Finished Deck Girder

Fig. 11 Installing the Last Deck Girder

STRESSES AND DISPLACEMENTS

The bridge load testing required the bridge to be instrumented using three different instrument types: strain gages, strain transducers, and displacement transducers. The bridge testing resulted in the generation of hundreds of data points that were analyzed. Fig. 12 shows the resulting strain at the quarter point for path 2 west. Once the deck girder strains were graphed for each load path, the peak values for each instrument were recorded. The strains were then used to calculate the stress values in the deck girder. The strain values were also used to determine the actual distribution factors for the interior and exterior deck girders.

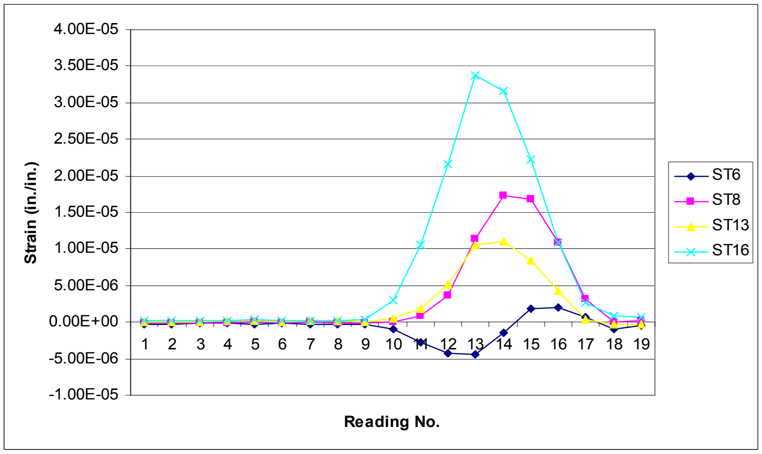

Fig. 12 Strain Readings at the Quarter-points on Path 2 West

The layout of the instruments was designed to determine the transverse and longitudinal effects along the deck girders (see Fig. 13). More detailed information on instrument layout can be found in the project draft final report, Gergely et al.[15]. The A36 steel diaphragms (standard channel sections connected to the deck girders by bolted angle connections) were instrumented to measure strain in the four corners. The plate connections were instrumented with two strain gages in order to measure transverse strain. The results from the strain gages were used to calculate the stress values in the diaphragms and plates. The results show that each diaphragm carried a small amount of moment, but primarily transferred axial load (with some inconsistency).

The graph in Fig. 14 shows that the diaphragms contain very little moment. The maximum strain occurred in Path 2. Path 2 also showed higher strains in other diaphragms as well. This finding suggests that Path 2 controlled the design of the diaphragms. A representation of the axial loads transferred by the diaphragms is shown in Fig. 15.

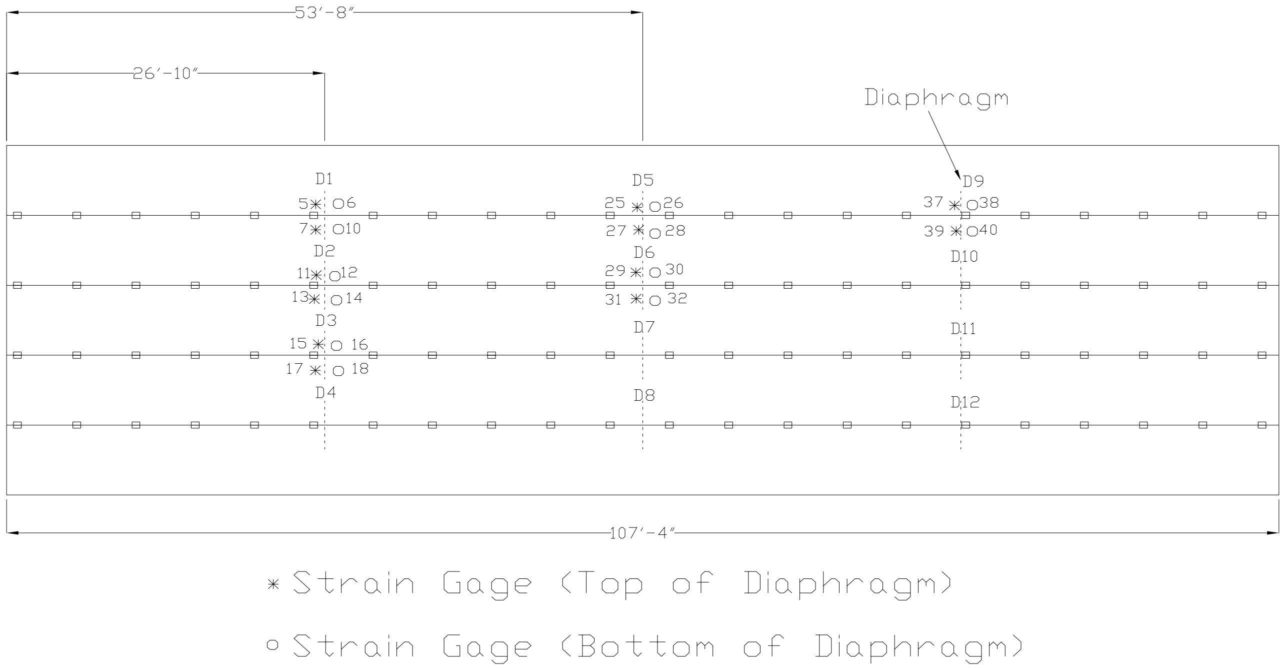

Fig. 13 Diaphragm Strain Gage Layout

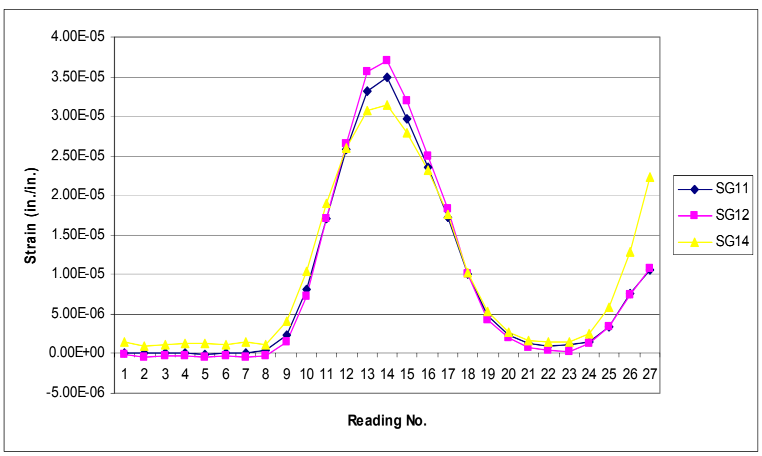

Fig. 14 Strain Readings for Diaphragm D2

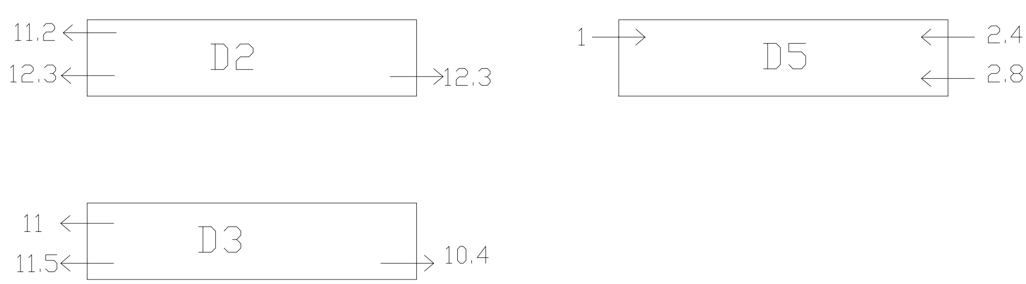

Fig. 15 Diagram of axial forces in diaphragms (kips) for Path 2 West

The graph in Fig. 16 shows the strain values for the two plates located approximately seventy feet from the west end of the bridge. Values for these plates show that one side of the plate is in compression and the opposite side is in tension. This suggests that there was some moment occurring in the plates, in addition to an axial force. Paths 1 and 2 seemed to produce the highest strain in these two plates.

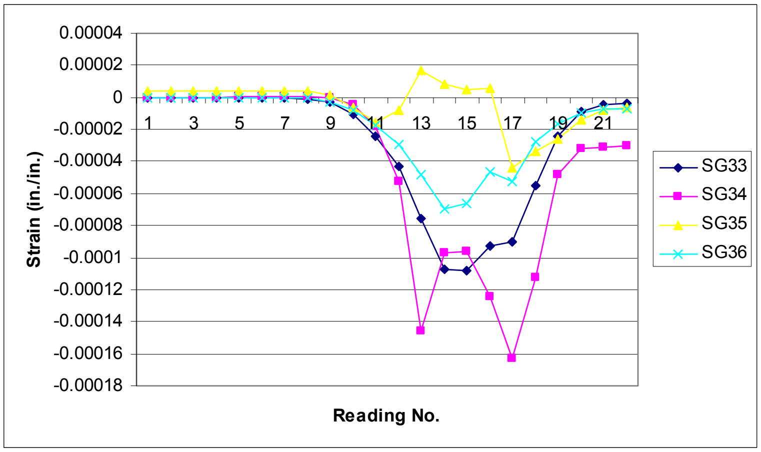

Fig. 16 Plate Strain Values from Path 1E

The deflections from the load testing were very small compared to the deflection limit of L/800 used in the design calculations. This could be a result of the deck girders having a higher stiffness than designed due to the integral end bent, asphalt overlay, and concrete parapet wall. The data showed that, as expected, each girder that was located beneath the corresponding load deflected more than the other deck girders away from the loaded area. The graph in Fig. 17 shows the deflections along deck girder 2 for Path 1 East. The overall maximum deck girder deflection of 1/8” is well below the design limits. The results show that there was not a significant difference amongst adjacent girders. Therefore, a comparison was not considered.

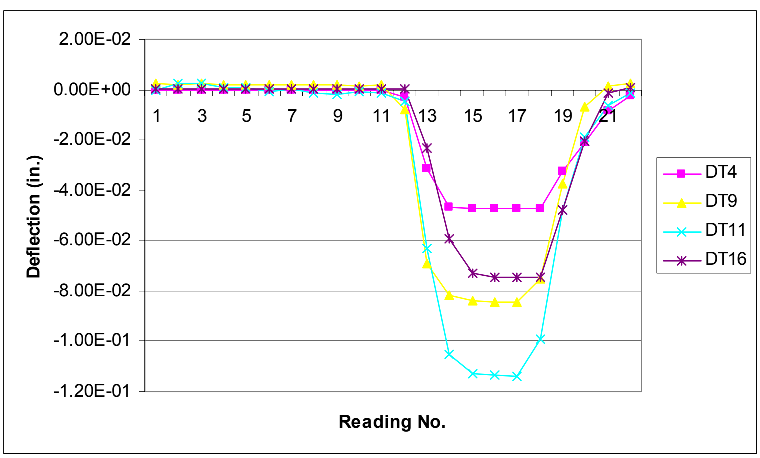

Fig. 17 Deflections in Girder 2 for Path 1-East

DISTRIBUTION FACTORS

The distribution factors of transverse loads to longitudinal members are determined using the girder spacing and the number of design paths loaded. The distribution factors shown in Tables 1 are the transverse distributions at the quarter point of the bridge, a location used because some of the instruments at the center line produced erroneous results for Paths 3 and 5. The LRFD and ASD design distribution factors were calculated according to the AASHTO specifications. The distribution factors from the test data were calculated using Eq.1 developed by Stallings and Yoo[13]. The values in Table 1 were calculated using only 2 (n = 2) wheel lines, to reflect the actual loading condition during the tests. Distribution factors were also calculated using 4 (n = 4) wheel lines, which assumed that while one path was being loaded during the test, the opposite path in the transverse section was being loaded as well.

$$DF_i = \frac{n\varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j w_j} \qquad (1)$$

where:
n – number of wheel lines
ε_i – strain at the bottom of the ith girder
w_j – ratio of the section modulus ratio of the jth girder to the section modulus of typical interior girder
k – number of girders

Table 1. Distribution Factors at the Quarter-line of Bridge Using 2 Wheel lines

	Test Data		ASD		LRFD	
	Interior	Exterior	Interior	Exterior	Interior	Exterior
1E	0.52	0.77	1.08	1.16	0.70	0.65
1W	0.58	0.60				
2E	0.78	0.26				
2W	0.73	0.33				
3E	0.75	0.37				
3W	0.69	0.47				
4E	0.50	0.95				
4W	0.48	0.85				
5E	0.67	0.65				
5W	0.54	0.74				

IMPACT FACTORS

For design purposes, AASHTO specifies an impact factor. This factor is applied to increase the live load effects on a bridge, in order to include dynamic, vibratory, and impact effects[12]. The results from the dynamic loads were used to calculate the actual impact factor for the bridge, and to compare them with the AASHTO value of 1.30. The results summarized in Table 2 are the impact factors calculated from Paths 1, 2, and 3 in both directions, using as single loading vehicle. The west integral abutment of the bridge was located at the bottom of the sag curve, which caused the loading truck to have a larger effect on the bridge, resulting in larger dynamic effects for the East-bound load path. A representative graph associated with these results is shown in Fig. 18 and 19, recorded through strain transducers mounted on the bottom of the deck girders.

Table 2. Impact Factors for Both East and West

Load Path	Test Data	LRFD
1E	0.73	1.30
1W	1.13	
2E	1.10	
2W	1.03	
3E	**1.30**	
3W	1.02	

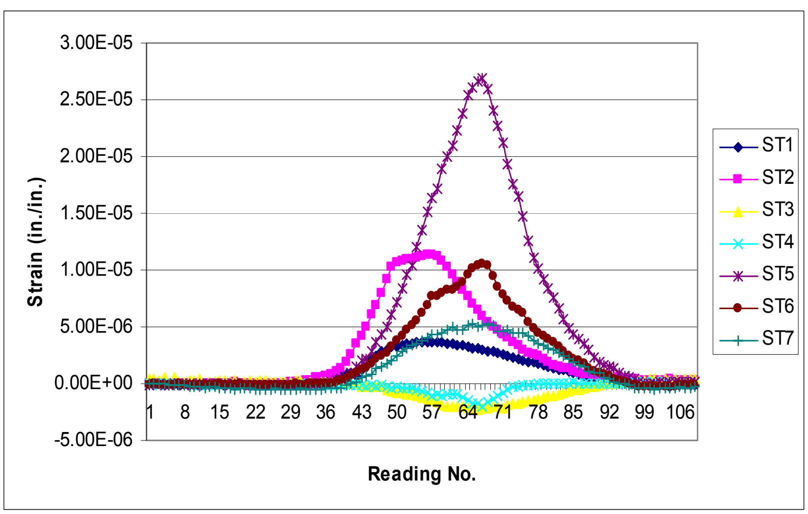

Fig. 18 One Truck (slow) Loading Strains for Path 2E

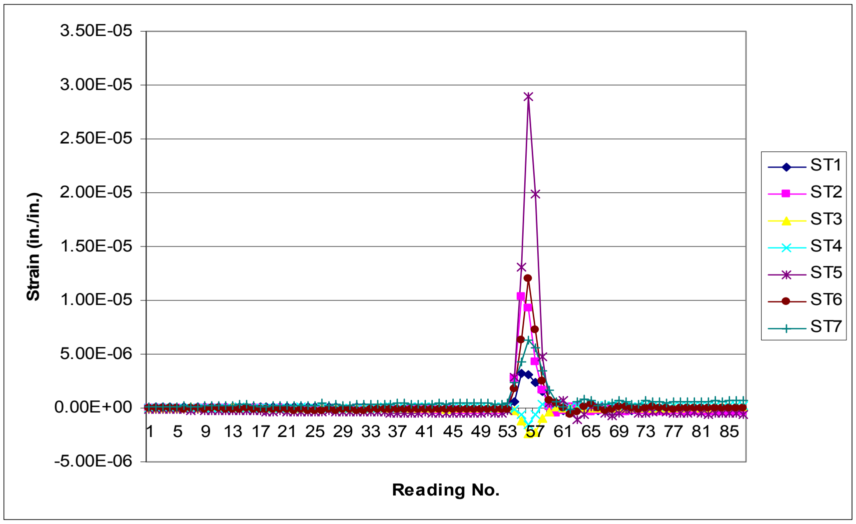

Fig. 19 Dynamic Loading Strains for Path 2E

CONNECTION EVALUATION

SPECIMEN DESIGN

Early in the project it was clear that currently available headed stud connection design methods do not represent well the details of the actual deck girder flange connections. This would not have been critical, however, based on the bridge load test, it was clear that most of the load transfer from deck girder to deck girder was achieved through the flange connections, and not the diaphragms. Therefore, additional experimental and theoretical analyses were performed on the flange connection, considering direct tension and longitudinal shear. The research team also made a decision to neglect the capacity of the grouted shear key between flanges, as this provided some resistance in the flange out-of-plane direction only.

The specimens were designed and constructed based on the top flange connection detail provided in the NCDOT[11] bridge design document. The details concerning specimen size and shape were arranged by using a tributary method to simulate the area of influence of one single connection on the actual bridge. The materials used were within general compliance of the materials used in girder production and were followed as closely as possible regarding rebar size and placement, concrete, and the studded connection. Four tension and four shear (data from one shear test is disregarded here as it did not follow the load-deformation behavior and load capacity of the other three specimens) specimens were tested during the experimental phase of this task. A schematic of the test specimen in plan view is shown in Fig. 20.

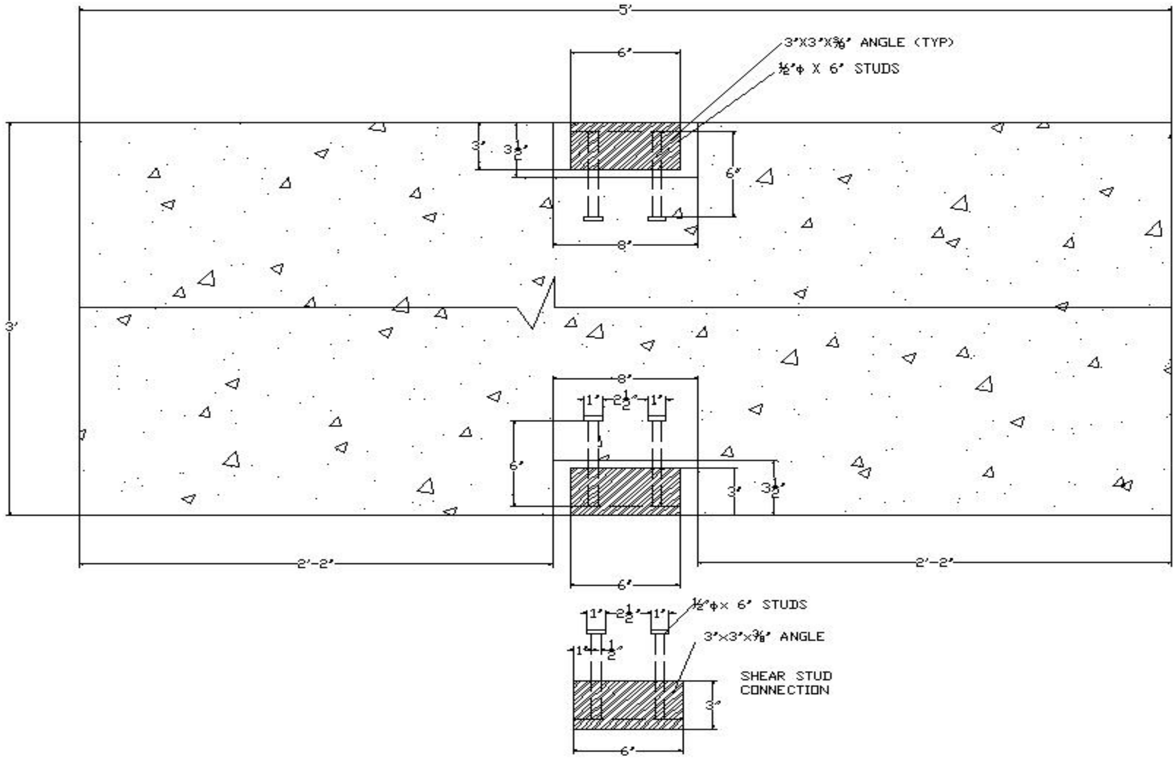

Fig. 20 Test Specimen Schematic – Plan View

LABORATORY TESTING

The testing specimens and setups were designed in order to determine the ultimate capacities of the flange connection in tension and longitudinal shear, approximately following the testing protocol developed by Anderson and Meinheit[3]. The tension specimens were fixed to the load frame to prevent translation and rotation of the specimens (Fig. 21). Furthermore, displacement transducers were attached to both the concrete slab and to the connections in two orthogonal directions, allowing the measurement of net connection deformations.

The load was measured through a load cell, and pressure transducer attached to the hydraulic pump, and was applied to the specimen through a steel plate connected to the hydraulic jack. Strain gages were attached to the plate connection, and both headed studs embedded in concrete. The average load versus displacement graphs for the four tensile test specimens are shown in Fig. 22. It is clear from these curves that the connection angle yielded at an approximate load of 5,000 lbs, transferring the force mostly through tension to the embedded studs. An average of 12,800 lbs capacity was reached, followed by large deformations, and finally, a failure due to stud/angle connection fracture.

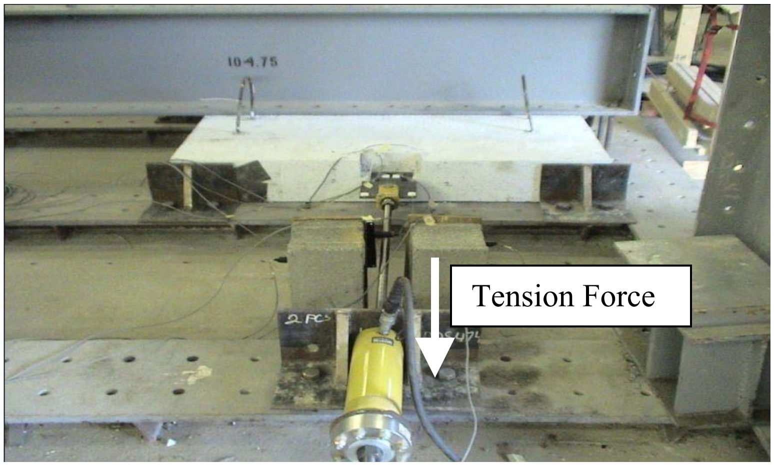

Fig. 21 Tension Connection Test

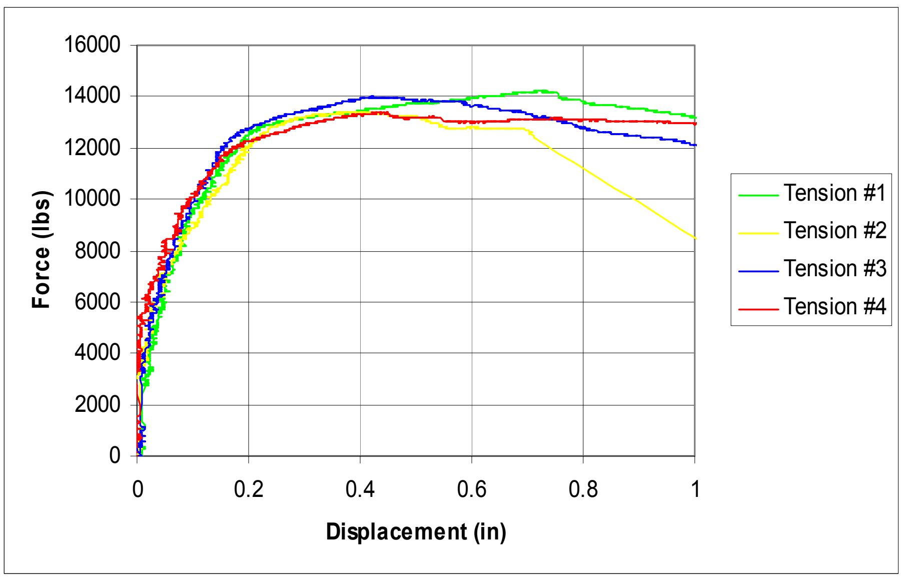

Fig. 22 Load vs. Displacement for Pure Tension Test Specimen

The longitudinal shear test setup had the same specimen details used in the tension testing; however, the specimens were rotated 90 degrees, and the shear force was applied through a steel plate/channel configuration (shown in Fig. 23). This allowed testing of the connection with minimal eccentricity, similarly to the actual bridge details. Similar restraints and instrumentation was applied to the shear specimens, as the ones utilized for the tensile tests.

Even though four shear tests were performed, the results of only three tests are included in Fig. 24. One of the specimens did not produce reliable research data, and was omitted from further discussions.

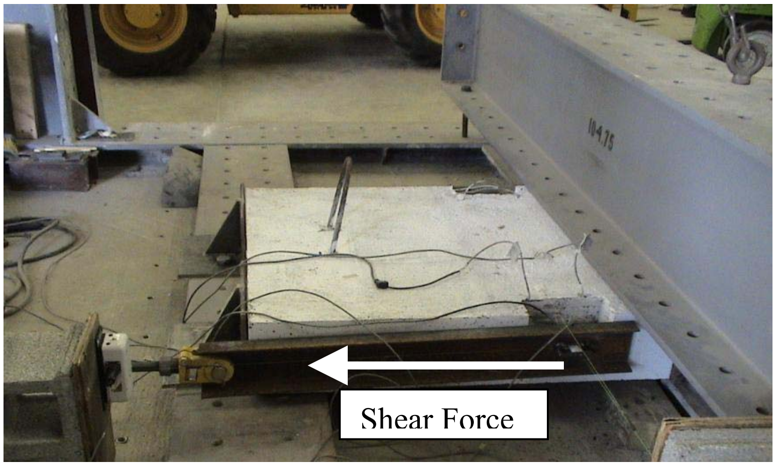

Fig. 23 Shear Connection Test

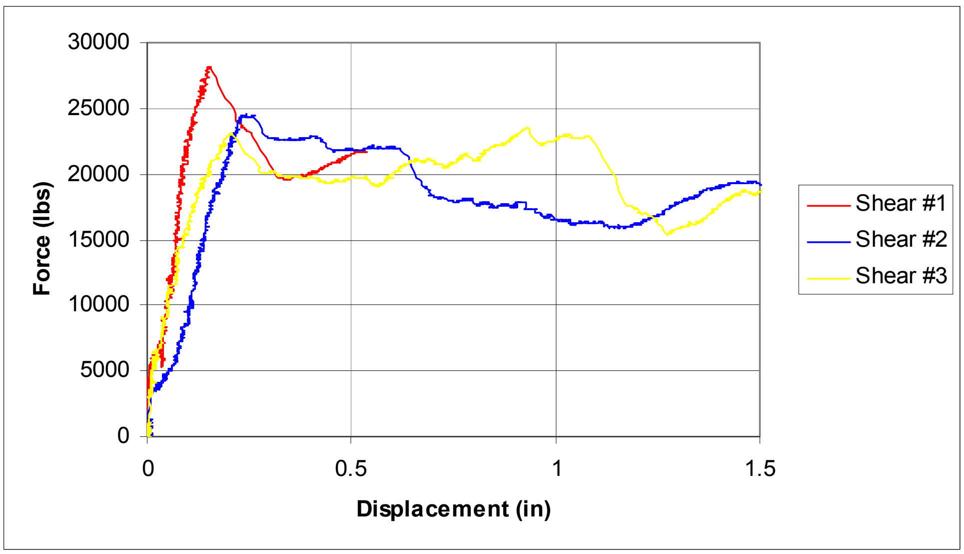

Fig. 24 Load vs. Displacement for Shear Specimen Test

At approximately 5,000 lbs, shifts in the graphs indicate that some of the connection steel members yielded. After steel yielding occurred, load redistribution occurred, which was followed by significant rotation of the connection angle, resulting in a failure governed by stud-to-angle connection capacity. The average ultimate value for the shear tests was 26,400 lbs.

CONCLUSION

In conclusion, the research shows that the new HPC deck girder system could be a new option for bridge construction. The results of the LARSA 2000© 4th Dimension[9] software analysis compared very well with the NCDOT[11] design hand calculations. The ANSYS© finite element model was found to accurately model the behavior of the deck girders and components as compared to the actual load test results. This was completed to validate the software model so that a parametric study on the plates and diaphragms could be completed to aid the NCDOT in the future design of deck girder systems. The load test and the finite element model confirmed that the connection design was adequate.

The tension and shear testing of these connections provided an accurate ultimate capacity for the flange connection, for which, no published design equations are available. The development of an accurate finite element model will aid future design and characterization of this connection type.

The analysis of the testing results showed some good comparisons for the distribution factors, impact factors, stresses, and displacements. The addition of the integral end bent fixity contribution in the ANSYS© and LARSA© software models more accurately captured the behavior of the actual bridge.

ACKNOWLEDGEMENTS

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