FINAL REPORT

**EXPERIMENTAL AND ANALYTICAL EVALUATION
OF AN ALUMINUM DECK BRIDGE**

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ABSTRACT

In response to a growing number of deteriorated and obsolete highway bridge decks, the Reynolds Metal Company has developed a multi-voided aluminum bridge deck system. The Virginia Department of Transportation agreed to implement the new aluminum deck system in a demonstration project. The Reynolds deck was employed to rehabilitate a functionally obsolete Route 58 highway bridge that spanned the Little Buffalo Creek near Clarksville, Virginia. Because the Virginia Department of Transportation considered the project as experimental, the Virginia Transportation Research Council implemented a study to evaluate the structural performance and serviceability of the Reynolds system. This study involved both experimental and analytical evaluations of the rehabilitated Route 58 Bridge.

The Virginia Transportation Research Council performed two field tests to evaluate bridge response. Instrumentation, consisting of various gages and transducers, measured deck and girder displacements, strains, and accelerations during controlled static and dynamic vehicular load tests.

Analytical evaluations were performed with the commercial, general-purpose finite element code ABAQUS. Three-dimensional finite element models were developed to predict the overall structural response of the Route 58 Bridge. Model accuracy was verified by using response data acquired from field testing. Girder displacements, uniaxial deck strains, longitudinal normal girder strains, and natural frequencies of vibration were predicted with reasonable accuracy. The validated finite element models of the structure were utilized to perform evaluations of the aluminum deck and steel girder response, composite action, natural frequencies of vibration, and mode shapes.

Results from this study clearly demonstrate that aluminum bridge decks are a feasible alternative to reinforced concrete decks from the standpoint of strength and serviceability.

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INTRODUCTION

Deck deterioration is responsible for the majority of deficient bridge ratings in the United States (Sotiropoulos & GangaRao, 1993). Subject to dynamic loading, cyclic loading, and occasional overloading, bridge decks are the most severely stressed elements in a bridge (Wolchuk, 1987). Typically, the usable life of a bridge's deck will only average one half of the useful life of the bridge (Bettigole, 1990). Combined with the stresses induced by environmental effects, such as temperature and moisture variations and freeze-thaw cycles, it is easy to see why the Federal Highway Administration estimates that 7,000 bridge decks are in need of immediate replacement (The Aluminum Association, 1996).

Deck deterioration is accelerated by corrosion problems. De-icing salts applied to bridge decks eventually penetrate the concrete and corrode the reinforcing steel. The corroding steel, in turn, causes the deck to crack, spall, and delaminate. This damaged concrete is more susceptible to additional permeation of harmful chemicals, thus accelerating the process of deck deterioration. Steel decks, although not as common as concrete, are also prone to corrosion.

Engineers have had limited success in their efforts to combat the deterioration of concrete bridge decks. The use of coatings and other protective systems to mitigate the corrosive effects of chlorides have yielded less than desirable results (O'Connor, 1995). To reduce the excessive financial expenses created by the ever growing number of deteriorated decks, engineers are now considering alternative materials such as aluminum for use as a bridge deck material.

Aluminum holds considerable promise due to its ease of manufacture and its excellent corrosion resistance. However, limited experience with aluminum as a bridge deck material and uncertainties associated with the field performance of such decks raises several issues. For instance, the dynamic performance of a bridge's deck directly affects user comfort, design of concrete parapets, choice of wearing surfaces, and connections in the superstructure. Since the late 1950s and the early 1960s, engineers have been studying the dynamic behavior of bridges with reinforced concrete decks, and therefore, are familiar with the basic dynamic characteristics of these type of bridges. However, bridges with aluminum decks are much lighter than bridges with concrete decks, and consequently, greater dynamic responses are anticipated. Additional questions concerning aluminum bridge decks, such as their ability to withstand loads from heavy traffic, resist fatigue effects of cyclic loads, ease of field installation, and affordability must be investigated to ensure that acceptable serviceability can be provided in a safe, efficient, and satisfactory manner.

Recognizing the potential benefits that aluminum could offer the transportation industry, the Reynolds Metal Company (Reynolds) has developed an aluminum bridge deck system. Reynolds approached the Virginia Department of Transportation (VDOT) with their new system, and an agreement was reached to employ the new deck system on a single-span bridge located on U.S. Route 58 in Mecklenburg County, Virginia. The existing bridge, previously listed by VDOT as functionally obsolete, made an ideal candidate for the first employment of an aluminum bridge deck in Virginia. Because the implementation of the new aluminum deck was considered an experimental project, a thorough evaluation was required. Consequently, the Virginia Transportation Research Council (VTRC), together with support from the Federal Highway Administration (FHWA), initiated a three-phase research study.

The first phase of this study took place at FHWA's Turner Fairbank Structural Laboratory in the fall of 1996. The static response of a prefabricated 2.74 m x 3.66 m (9 ft x 12 ft) section of the Reynolds deck system was analyzed by subjecting a representative bridge deck panel to a series of service-load and ultimate-load tests. Using the data collected from the FHWA load tests, an extensive analytical evaluation of the deck panel was conducted (Dobmeier et al., 1999).

The second phase of the study (which is reported here), involved an experimental and analytical evaluation of the deck system that was installed on the Route 58 Bridge. After bridge construction was completed in May 1997, VTRC engineers subjected the structure to static and dynamic field testing and generated analytical responses from computer models of the bridge-deck system.

Phase three of the study will focus on the long-term durability and performance of the deck system and its thin epoxy-type wearing surface and will be the subject of a separate report. In addition, fatigue testing of a second-generation aluminum deck panel will be conducted throughout 1999 at the Virginia Polytechnic Institute and State University in Blacksburg, Virginia.

PURPOSE AND SCOPE

The purpose of Phase II of this project was to investigate, both experimentally and analytically, the static and dynamic response of the Route 58 Bridge and thereby provide an improved understanding of the behavior and performance of aluminum deck bridges. Investigations into the following facets of the aluminum deck bridge were performed:

- Aluminum deck and steel girder response

- Composite action between girders and aluminum deck

- Natural frequencies of aluminum deck bridge

- Mode shapes of aluminum deck bridge.

METHODOLOGY

This study focused on a single bridge employing the Reynolds aluminum deck system. Specific objectives of the experimental evaluations included the measurement of deck and girder strains, accelerations, and displacements under several controlled static and dynamic vehicular loads. Experimental data collected from the bridge were also used to validate the performance and accuracy of finite element models.

The commercial finite element code, ABAQUS, was used for the computational analyses. Specific objectives of the analytical evaluations included the development of finite element models that could represent bridge behavior and accurately predict strains, stresses, displacements, natural frequencies of vibration, and mode shapes.

Reynolds Deck System

The deck system analyzed in this study was comprised of three separate deck panels that were assembled in the field. Each deck panel consisted of an assemblage of the two-voided extrusion shown in Figure 1. The extrusions were fabricated from a 6063-T6 aluminum alloy, which has a minimum tensile yield strength of 172 MPa (25 ksi), a minimum tensile ultimate strength of 207 MPa (30 ksi), and a modulus of elasticity of 69 GPa (10,000 ksi). AASHTO HS-20 loads were used in the design of the extrusion. The deck panels were fabricated by shop-welding individual extrusions together at the top and bottom flanges to achieve the desired dimensions. Figure 2 illustrates the geometry of a deck panel formed by welding the extrusions together.
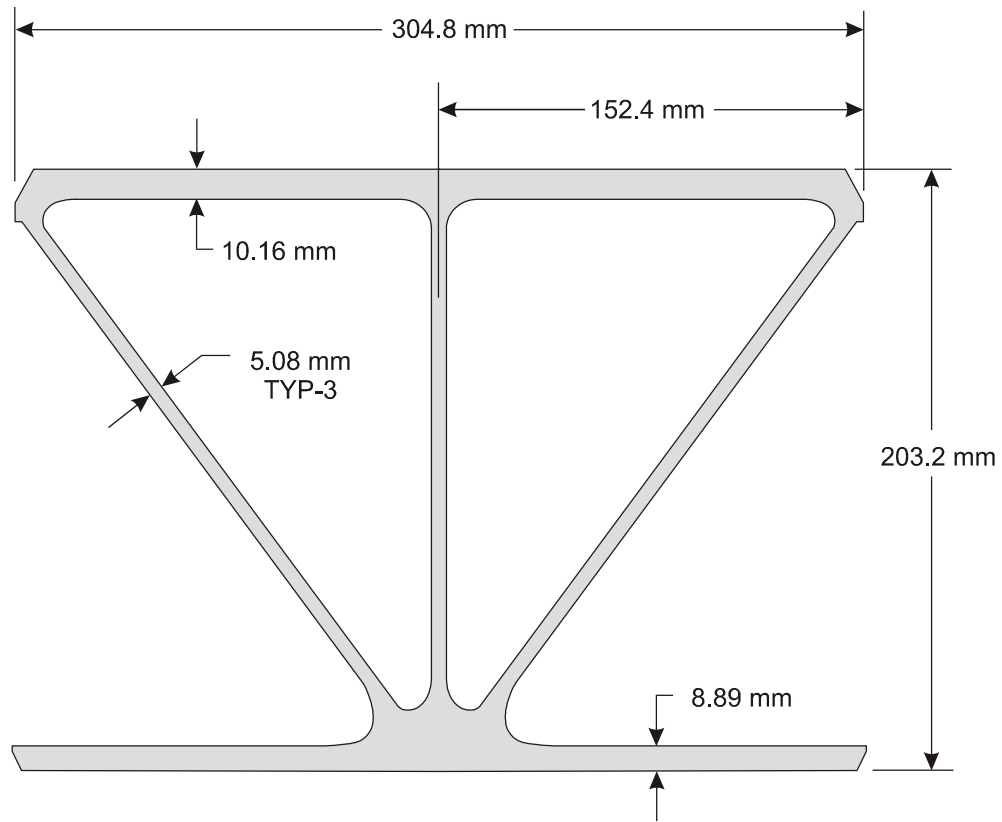
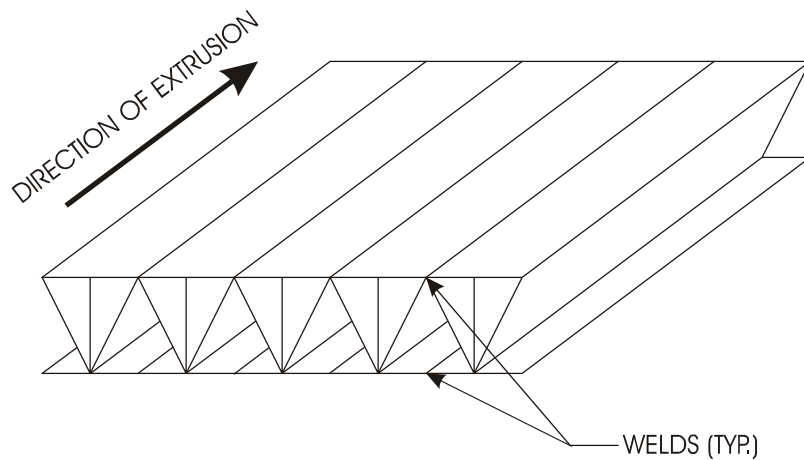**Figure 1. Reynolds Two-Voided Extrusion**

Figure 2. Aluminum Deck System

The deck system was essentially isotropic, which provides significant bending strength in both the longitudinal and transverse directions (Matteo, et al., 1997). Because the deck system is slightly stiffer in the direction of the extrusions, the deck is normally oriented with the extrusions running parallel to the bridge girders. When installed in this manner, the stresses developed under loading can be categorized into three stress systems indicated schematically in Figure 3.

System I stresses are generated by the longitudinal bending of the composite aluminum deck/steel girder section. System II stresses are produced from transverse bending of the deck between girders, and System III stresses result from the local transverse bending of the top deck flanges due to wheel loads.
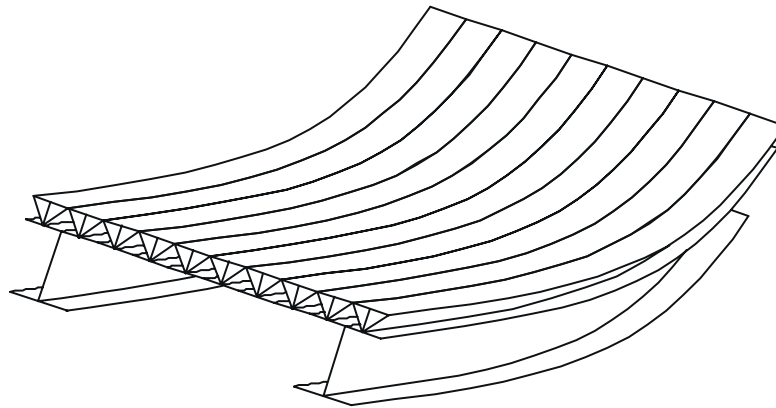
System I Stresses -- Longitudinal Bending of the
Composite Deck Girder Section

System II Stresses -- Transverse Bending
of the Deck Between Girders
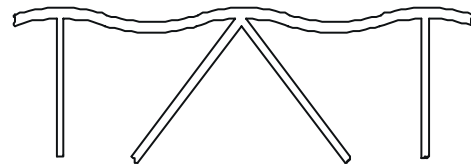
System III Stresses -- Localized, Transverse
Bending of the Top Deck Flange

Figure 3. Stress Systems in Deck

Experimental Evaluation

Description of Test Structure

The Route 58 bridge, outfitted with the Reynolds aluminum deck, is 16.64 m (54.6 ft) long and has a total width of 9.75 m (32 ft), providing two 4.27 m (14 ft) eastbound lanes. The bridge deck was constructed of three prefabricated rectangular aluminum panels. The panels spanned the entire length of the bridge, but varied in width and were delivered to the site by tractor trailers. A 4.27 m (14 ft) wide center deck panel was installed first, and then two 2.74 m (9 ft) wide panels were installed on each side of the center panel. Splice plates were used to join the deck panels together along their longitudinal lengths.

Four welded-plate girders with eight- and nine-foot spacings support the deck panels. Figure 4 illustrates the cross section of the bridge, facing east towards Clarksville, VA, in which the girders are labeled A through D. The plate girders are made of A709 steel of Grade 50 and have an overall depth of 0.91 m (36 in). The top flanges of the plate girders are equipped with 19.05 mm (0.75 in) diameter shear studs spaced at 229 mm (9 in) along the central 7.9 m (26 ft) of each girder and at 165 mm (6.5 in) along the end regions. The diaphragms, made of A709 Grade 36 steel, are C15x33.9 sections, and are located at the ends and the one-third points along the girders.
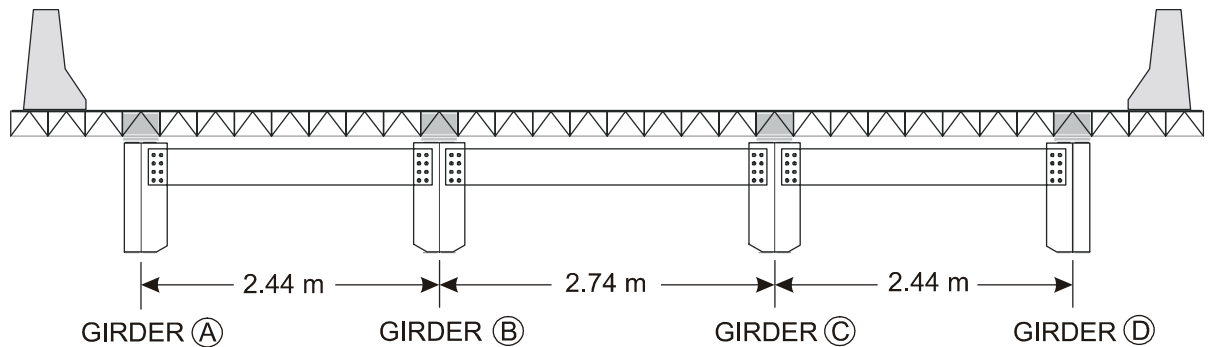

Figure 4. Cross-Section of Route 58 Bridge

To achieve composite action between the aluminum deck and the steel girders, and in order to prevent galvanic action between the two materials, a haunch with a nominal thickness of 50.8 mm (2 in) was constructed on the girders' top flanges. Shear studs on the girders penetrated into the interior of the aluminum deck. Bond was achieved between the deck and shear studs by injecting magnesium phosphate grout into the full length of the extrusions located above each girder. Figure 5 shows details of the composite construction between the deck and a typical girder. The girders are supported on laminated, elastomeric rubber-bearing pads.

Field Test I

The first field test on the Route 58 Bridge was conducted in June 1997. The primary objectives of this initial field test were to experimentally measure deck and girder strains and girder displacements in response to static vehicular loads. Limited dynamic data were also recorded.

Various transducers were utilized to obtain strain and deflection data from the aluminum bridge deck and steel-plate girders. At midspan, half-bridge foil strain gages were placed on the bottom surface of the aluminum deck to record longitudinal and transverse uniaxial strains. The longitudinal and transverse gages were positioned on the deck halfway between each girder, and gages were placed on each side of the longitudinal deck splice located between girders C and D.
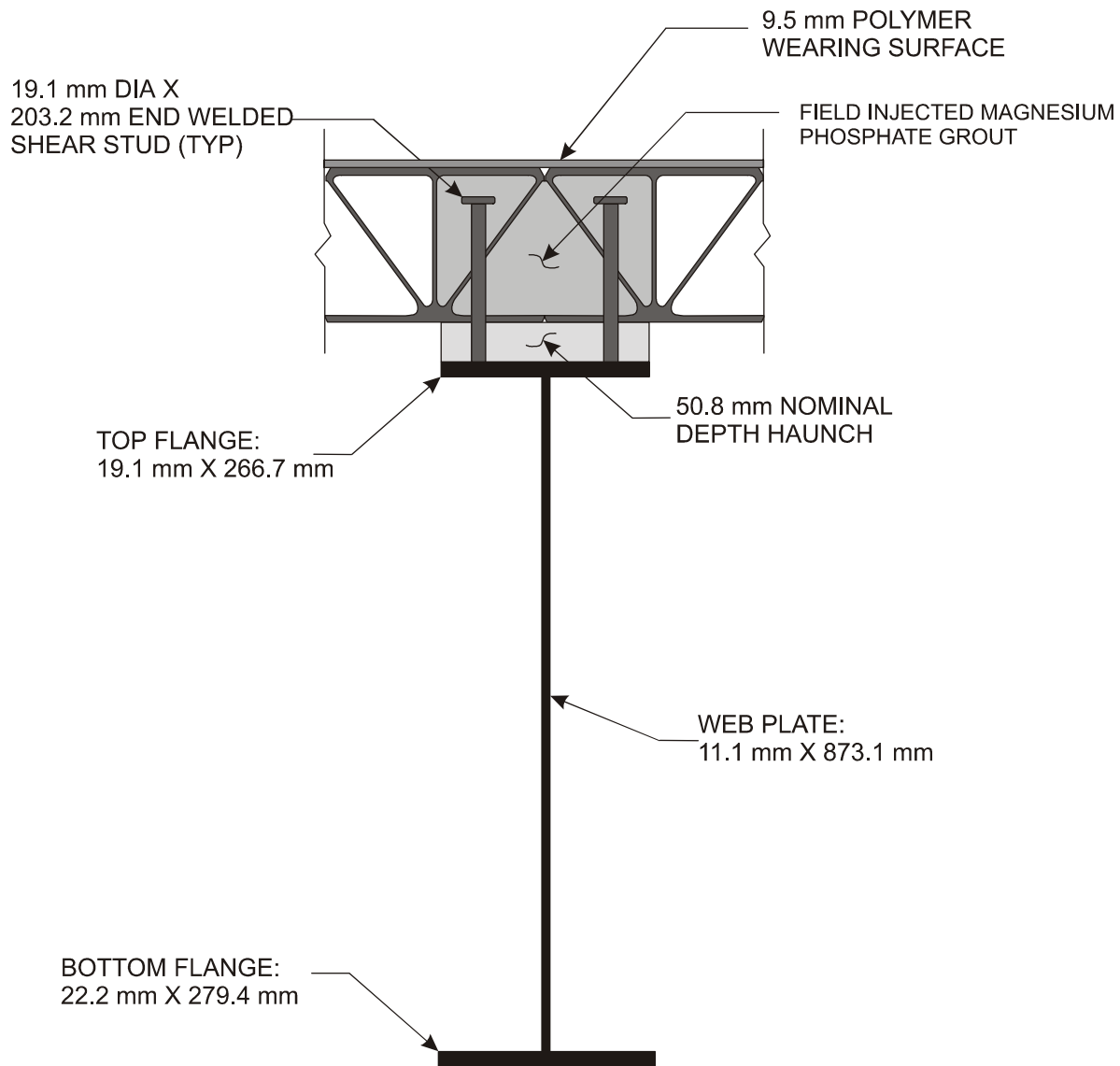
Figure 5. Composite Construction

Figure 6 illustrates the strain gage layout; the drawing assumes a viewpoint of being underneath the bridge, looking up, and facing east. Weigh-in-motion (WIM) gages were utilized to record longitudinal bending strains in the top and bottom flanges of the steel plate girders at midspan. Deflections of the plate girders at quarter and midspan were also obtained by utilizing cantilever deflection gages (CDG).

At the time of the field test, the Route 58 Bridge was not yet open to traffic, which provided VTRC engineers with full access to the structure and circumvented the need for traffic control. VDOT dump trucks, filled with gravel, were used for applying loads to the bridge during the tests. Figure 7 indicates the trucks' model type, dimensions, and weight.

Splice

TS1: Trans. Strain Gage
Between Girders A and B at Mid Span

TS5: Trans. Strain Gage
Left of Splice

LS5: Long. Strain Gage
Right of Splice

TS4: Trans. Strain Gage
Left of Splice Near Girder C

TS6: Trans. Strain Gage
Right of Splice

LS4: Long. Strain Gage
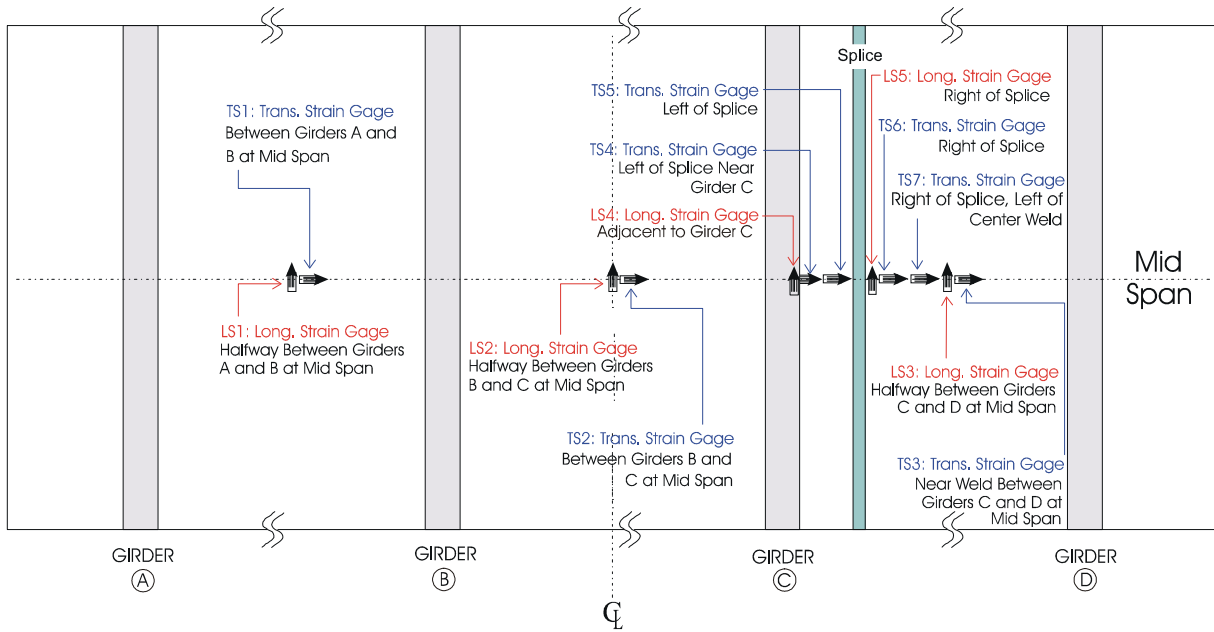Adjacent to Girder C

TS7: Trans. Strain Gage
Right of Splice, Left of Center Weld

LS1: Long. Strain Gage
Halfway Between Girders A and B at Mid Span

LS2: Long. Strain Gage
Halfway Between Girders B and C at Mid Span

LS3: Long. Strain Gage
Halfway Between Girders C and D at Mid Span

TS2: Trans. Strain Gage
Between Girders B and C at Mid Span

TS3: Trans. Strain Gage
Near Weld Between Girders C and D at Mid Span

Mid Span

GIRDER Ⓐ   GIRDER Ⓑ   GIRDER Ⓒ   GIRDER Ⓓ

C_L

Figure 6. Strain Gage Instrumentation−Test 1

A

FRONT

E

REAR

D

B

C
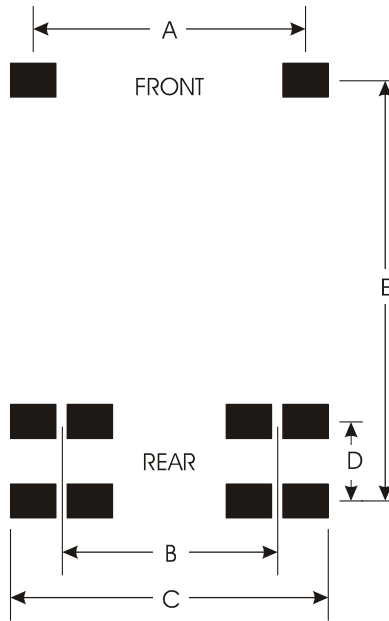
Truck	Closest Girder Position	Dimension A	Dimension B	Dimension C	Dimension D	Dimension E	Total Weight (kg)	Front Axle Weight (kg)
Ford (#58138)	Girder B	2.06 m	1.83 m	2.36 m	1.35 m	5.87 m	25220	N/A
International (R00560)	Girder C	2.06 m	1.83 m	2.36 m	1.36 m	5.73 m	26245	6477

Figure 7. VDOT Dump Trucks

8

Static Tests

A total of six static load configurations were applied to the bridge in an effort to maximize the strains in the deck or the girders. During the static tests, the trucks were always positioned at the same longitudinal position (i.e., side by side) on the deck; however, the transverse distance between the two trucks varied according to the particular load case. In addition, the trucks faced east, towards Clarksville, VA, for all of the tests. For each static load case, data were collected at a rate of ten samples per second for approximately five seconds; the mean of the data collected for each gage was computed and reported as the experimental static response.

For the first three load cases, the trucks straddled girders B and C in an effort to maximize the strain in the aluminum deck. The transverse distance between the two trucks for these load cases was approximately 0.61 m (2 ft). For the first load case, the trucks' first rear wheels were positioned at the quarter point on the span. For the second load case, the trucks' first rear wheels were positioned at midspan, and the trucks first rear wheels were then positioned at three-quarter span for the third load case. For the three-quarter span load case, the front wheels of the trucks were positioned off of the bridge deck. These load cases are designated as Static Load Cases 1, 2, and 3.

For the remaining three static tests, one truck had its right wheels positioned over girder B, while the second truck had its left wheels positioned over girder C in an effort to maximize the strain in the girders. The trucks were again positioned side by side, but were separated by a transverse distance of approximately 2.44 m (8 ft). Once again, the trucks' first rear wheels were positioned at the one-quarter, one-half, and three-quarter span locations. These load cases are referred to as Static Load Cases 4, 5, and 6.

Dynamic Tests

Dynamic load tests were conducted using only one (Ford) dump truck. Strain and deflection data were collected from the gages at a rate of 200 samples per second. For each load case, data collection began as the truck entered the bridge's approach slab and continued for several seconds following the truck's departure. Therefore, the length of data collection for each load case was dependent on vehicle speed. A total of four tests with the traveling dump truck were conducted. For each load case, the truck traveled east, towards Clarksville, VA; Table 1 lists the positions of the truck on the bridge and its various speeds.

Table 1. Dynamic Load Cases–Field Test I

Load Case	Truck Position	Truck Speed (km/hr)
1	Right Lane	76
2	Right Lane	32
3	Middle of Bridge	76
4	Middle of Bridge	40

Field Test II

The second field test of the Route 58 Bridge took place in November 1997. The purpose of the second field test was to subject the structure to traveling loads of various velocities using a single VDOT dump truck and to collect acceleration data from the deck and girders. The acceleration data collected during the test were utilized to determine the bridge's natural frequencies of vibration and provide additional data for the development of analytical models.

Instrumentation during the second field test was limited to girders C and D and the deck region located between these girders. A total of four piezoelectric accelerometers were used to measure the accelerations of the deck and girders. Accelerometers were placed at the following positions on the bridge: at the midspan of girders C and D, at the quarter-span of girder C, and on the bottom of the deck between girders C and D at the midspan. The accelerometers that were affixed to the girders were positioned on the top of the girders' bottom flanges.

The local Clarksville Area Headquarters again provided a three-axle dump truck (International) for field testing. The truck, loaded with gravel, had a mass of 26,317 kg (58,020 lb). Since the structure was open to traffic during the second field test, traffic control measures were established to provide the VDOT dump truck with access to the right lane of the bridge (heading east towards Clarksville, VA). A total of eight tests were performed using the single dump truck. The first two tests performed were "pseudo-static" tests, meaning that the truck crossed the bridge at very slow speeds of approximately 0.9 m/s (2 mph). The two pseudo-static tests provided VTRC engineers with additional "static" data to complement the data collected during the first field test.

Load cases 3 through 8 consisted of the International dump truck traveling in the right lane at various speeds. In addition to the eight dynamic load tests performed by the VDOT vehicle, the bridge response was also recorded for three separate random trucks with estimated speeds of 24.6 m/s (55 mph). Table 2 lists truck speeds for load cases 1 through 11.

Table 2. Dynamic Load Cases–Field Test II

Load Case	Truck Description	Truck Speed (km/hr)
1 and 2	VDOT Truck	< 3
3 and 4	VDOT Truck	40
5 and 6	VDOT Truck	72
7 and 8	VDOT Truck	16
9, 10, and 11	Random	89

Analytical Evaluation

Three-dimensional finite element models of the Route 58 Bridge, with the capability of accurately predicting deflections, strains, stresses, natural frequencies, and mode shapes, were developed using the commercial finite element code ABAQUS. In the model development, the effects of the deck slope, wearing surface, haunch material, and deck splices were ignored. Also, it was assumed that the girders were simply supported at each end and that full composite action existed between the deck and the girders.

Deck strain data collected during the dynamic load cases of Field Test I indicated that loads transferred from one prefabricated deck panel to another were not affected by the presence of the longitudinal splices. From an analytical perspective, reasonable strain continuity across the longitudinal deck splices nullify the need to incorporate splice plates and bolts (that make up the splices) into the finite element models.

Quadrilateral shell elements were used to represent the components of the Reynolds deck system. The top and bottom deck surfaces were discretized with 152.4 mm x 152.4 mm (6 in x 6 in) elements, and each vertical and inclined web stiffener of the deck was represented with a single row of shell elements. Figure 8 illustrates this discretization scheme for a typical section of the Reynolds deck system.
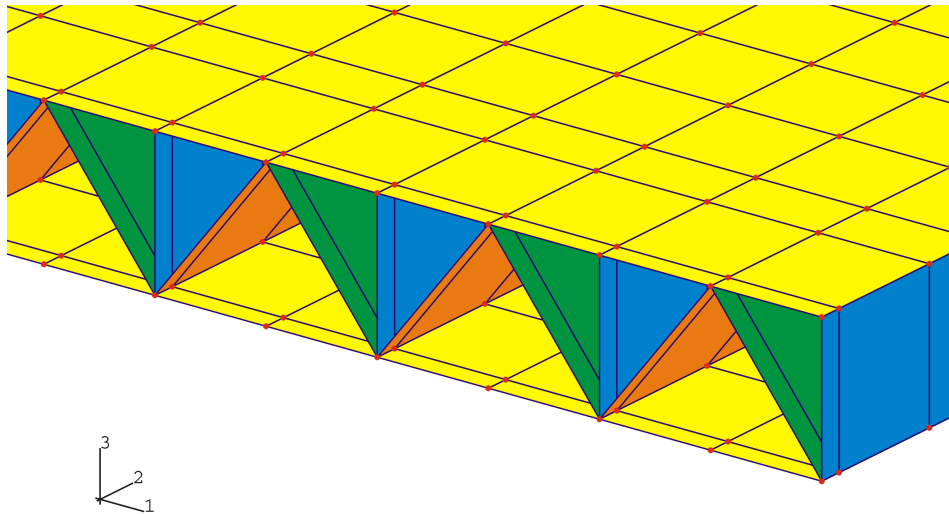

Figure 8. Finite Element Model of Deck

The welded-plate girders were represented with three-dimensional, first-order beam elements that had the capability for representing transverse shear deformation. Composite action was represented in the models by using multi-point constraints to define "rigid beam links" between the deck and girder nodal layers.

Diaphragms in the Route 58 Bridge were represented by three-dimensional truss elements that were connected to the girder nodes. Parapets were defined in the finite element models using beam elements in which element properties such as area, moment of inertia, and torsional

rigidity could be defined. Beam elements representing the parapets were defined on a separate nodal layer, and rigid beam links (multi-point constraints) were utilized to provide connectivity to the shell elements representing the deck. Since this connectivity provided full composite action between the deck and parapets, partial continuity was approximated by reducing the stiffness of the parapets. Through trial-and-error iterations, 30 percent of the parapets' modulus of elasticity and moments of inertia were finally used.

The magnesium phosphate grout injected into three voids of the deck above each line of girders was represented as three-dimensional, solid elements. Since the grout had a mass of approximately 40 to 50 percent of the aluminum deck, the mass of the grout would be expected to have a significant effect on dynamic response. Figure 9 illustrates all of the elements and the discretization scheme used to represent the Route 58 Bridge.

Static Analysis

Static loads from the VDOT dump trucks were defined as surface pressure loads in the finite element models. These loads, representing both the Ford and International trucks, were applied to sets of deck elements to represent a particular load case. During static analysis, ABAQUS neglects inertia effects and time-dependent material effects, such as creep or swelling. Model output was obtained from nodes and elements that closely matched the locations of the field instruments.
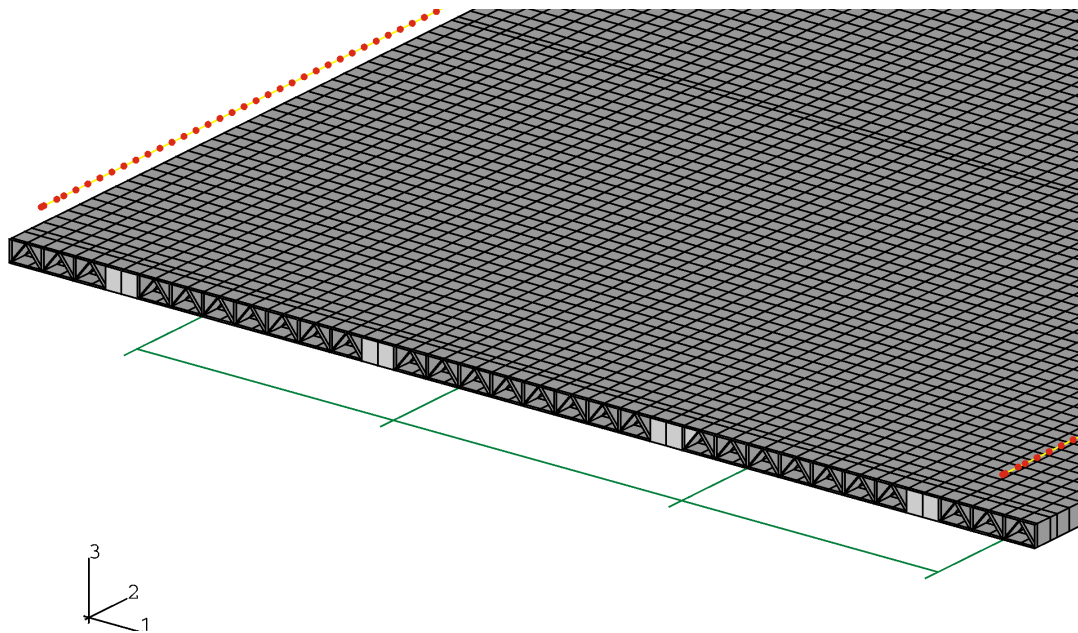

Figure 9. Route 58 Bridge Model

Dynamic Analysis

The specific dynamic analysis procedure performed was a natural frequency extraction that provided natural frequencies and corresponding mode shapes of the structure. The frequency extraction procedure performed by ABAQUS calculates natural frequencies and corresponding mode shapes through a subspace iteration procedure.

RESULTS AND DISCUSSION

In order to demonstrate the capabilities of the validated bridge models, various finite element predictions for deck and girder response are presented. Girder displacements, uniaxial deck and girder strains, and composite neutral axis locations are presented from static analyses. Analytical predictions for the natural frequencies of vibration and mode shapes are also provided. The presented analytical results are intended to demonstrate the capability of the finite element models to predict the general structural response of the Route 58 Bridge with a reasonable degree of accuracy.

Static Analysis

Static response data was acquired at a rate of ten samples per second for approximately five seconds. The mean of the data was computed and reported as the experimental static response. Stress and strain output from each element was acquired at the integration points of the element, the location of which is dependent upon model discretization. It was not possible to require the location of strain gages in the field to coincide with the individual elemental integration points in the models. Therefore, stress and strain output was obtained from elements nearest the actual strain gage locations. On the other hand, model discretization did permit the nodes in the models to be defined at locations very near the positions of the deflection gages. The displacement output was obtained from nodes positioned at the girders' quarter and midspan locations.

Selected results from the six static load cases of Field Test I are presented in Tables 3, 4 and 5. Table 3 provides midspan girder deflections as measured from the field tests and as predicted from the finite element models. The maximum absolute difference between field and finite element results for the girder deflections was 1.27 mm (0.05 in), and the average absolute difference was 0.51 mm (0.02 in). As may be observed from the table, the predicted displacements were slightly larger than the measured displacements for the interior girders and slightly less for the exterior girders. The agreement, however, between predicted and measured displacements for all girders was considered excellent.

Table 3. Midspan Girder Deflections (Millimeters)

Static Load Case	Girder A Midspan		Girder B Midspan		Girder C Midspan		Girder D Midspan	
	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.
1	3.6	3.3	4.6	5.1	4.8	5.3	2.8	3.6
2	4.8	4.3	6.1	7.1	6.6	7.1	3.8	4.6
3	2.8	2.8	3.8	4.3	4.3	4.3	2.5	2.8
4	4.3	3.8	3.8	4.6	4.3	4.6	4.3	4.1
5	5.6	5.1	4.8	6.1	5.8	6.4	6.1	5.3
6	3.3	3.0	3.0	3.8	3.8	3.8	3.8	3.3

Table 4 reports longitudinal and transverse strains on the bottom deck surface at midspan, which were measured halfway between each girder. The maximum absolute difference between the experimental and analytical midspan deck strain data was approximately 34 microstrain, and the average absolute difference was approximately 10 microstrain. It should be noted that the measured strains are quite small, probably due to the fact that the location where strains were measured is close to the neutral axis. Given these conditions, the agreement between predicted and measured strains was considered excellent.

Table 4. Longitudinal and Transverse Deck Strains at Midspan (Microstrain)

Static Load Case	Long. Strain Between Girders A & B		Long. Strain Between Girders B & C		Long. Strain Between Girders C & D	
	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.
1	-7.71	-6.66	-12.68	-7.11	-3.21	4.91
2	16.59	11.67	-0.18	17.05	-0.58	12.72
3	-15.47	-13.86	-32.24	-18.70	-17.06	-14.14
4	-14.02	-7.85	-29.51	-15.22	-11.39	-5.82
5	-13.83	-2.94	-42.22	-21.61	-10.95	-2.95
6	-27.85	-13.48	-29.60	-13.50	-23.67	-13.87

Static Load Case	Transverse Strain Between Girders A & B		Transverse Strain Between Girders B & C		Transverse Strain Between Girders C & D	
	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.
1	55.50	40.09	98.30	67.37	56.87	53.34
2	102.77	72.74	120.95	125.76	81.25	77.55
3	17.41	19.16	44.29	46.40	28.75	20.29
4	20.95	24.45	10.45	23.58	26.87	25.37
5	39.95	48.55	-10.58	23.78	53.43	52.26
6	8.71	16.28	-5.35	21.49	16.26	17.53

Table 5 reports longitudinal normal girder strains at midspan measured at the bottom flange of the girders. For the longitudinal normal girder strains, the maximum absolute difference was approximately 87 microstrain, and the average absolute difference was approximately 18 microstrain. Strain measurements were also recorded on the top flanges. However, since this location was close to the neutral axis, strains were small and little useful information was obtained. Review of the data indicates that predicted girder displacements were within 10 percent of the measured displacements, whereas the predicted deck strains and normal girder strains were all within 21 percent of the measured strains.

Table 5. Longitudinal Normal Girder Strains at Midspan (Microstrain)

Static Load Case	WIM Strain Bottom of Gir. A		WIM Strain Bottom of Gir. B		WIM Strain Bottom of Gir. C		WIM Strain Bottom of Gir. D	
	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.	Field	F.E.M.
1	97.45	95.66	95.62	137.87	115.84	143.93	89.38	102.35
2	125.22	132.99	138.96	226.00	184.62	229.41	115.10	136.83
3	66.27	76.33	68.08	110.31	93.56	111.15	71.18	78.00
4	112.91	112.20	78.17	128.30	107.26	130.28	129.19	115.35
5	151.84	161.04	108.33	191.37	157.61	195.61	185.35	167.58
6	78.47	85.00	56.33	97.51	84.17	98.74	98.44	87.72

Dynamic Analysis

The second field test subjected the Route 58 Bridge to traveling loads. Deck and girder accelerations were recorded and used to determine natural frequencies and mode shapes and to provide a basis for verifying the model predictions. A primary objective of the analytical analyses was to provide finite element predictions of the natural frequencies of vibration of the Route 58 Bridge and the corresponding mode shapes.

Figure 10 illustrates a typical acceleration record acquired during Field Test II. The deck and girder acceleration data collected by the MEGADAC was transferred into ASCII format and then imported into DaDiSP, a PC-based data analysis program. To minimize the complicating effects of vehicle-structure interaction, only the portion of the acceleration record representing the free vibration of the bridge was used to analyze the dynamic response. The frequency response analysis performed by DaDiSP made it possible to identify natural frequencies, and the relative amplitude of the response peaks at different locations suggested characteristics of the mode shapes. Figure 11 is a typical frequency response plot provided by DaDiSP. The peaks in the response curve correspond to natural frequencies of the bridge. All acceleration data, recorded during Field Test II, were analyzed and carefully reviewed in order to determine (experimentally) the structure's natural frequencies of vibration. A total of six natural frequencies were identified experimentally.
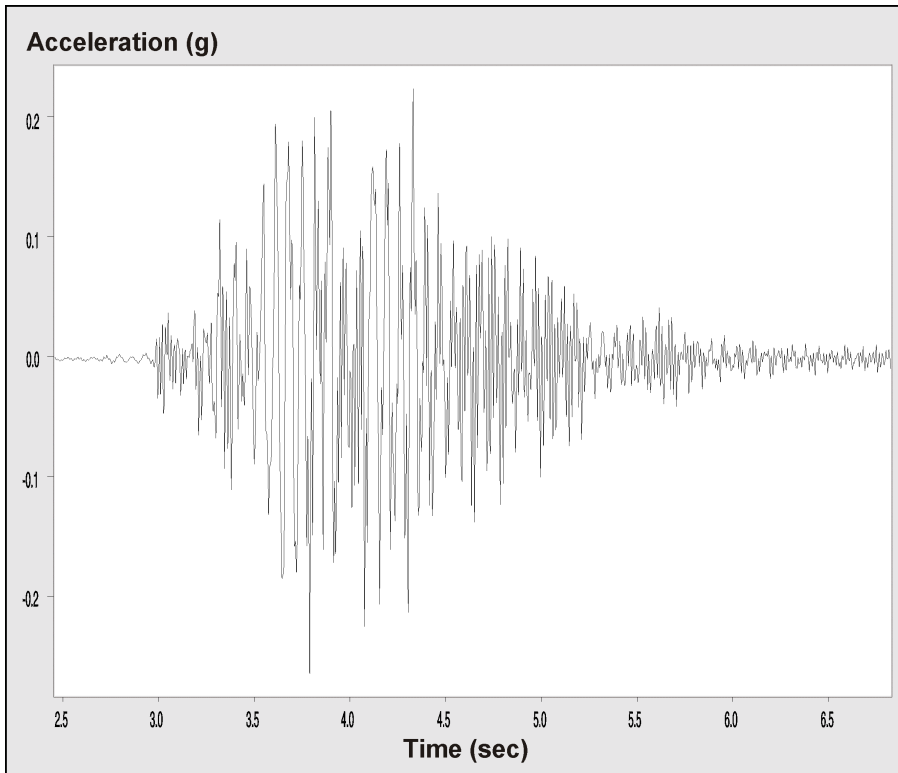
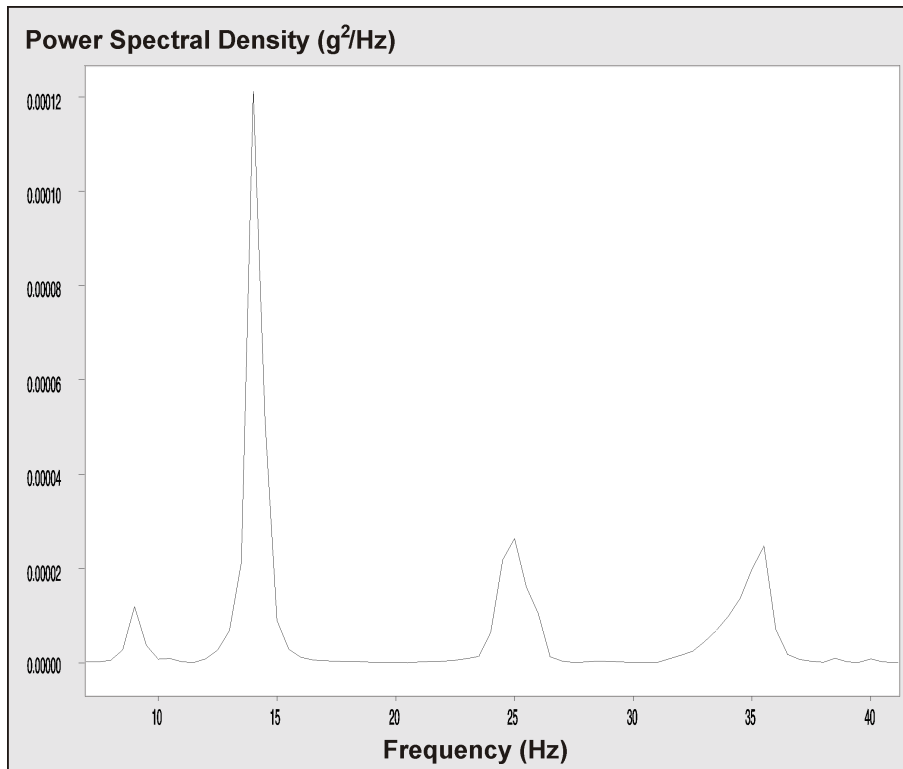**Figure 10. Typical Acceleration Record**

Figure 11. Typical Response Spectrum Plot

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Table 6 provides natural frequencies of vibration obtained both analytically from finite element analysis and experimentally from the field test. The first nine natural frequencies, predicted by the finite element analysis, are listed in the table. A thorough examination of the mode shapes and use of modal participation factors assisted in identifying the predicted frequencies with those frequencies that were measured experimentally. Three of the predicted frequencies, namely the fourth, fifth, and eighth, were judged to be a result of local behavior of the deck and, thus, unlikely to be observed experimentally. The remaining analytically predicted frequencies and mode shapes were found to compare favorably with those frequencies that were determined experimentally.

Table 6. Analytically and Experimentally Determined Natural Frequencies of Vibration (Hz)

	MODE								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
F.E.M.	8.01	8.13	14.94	22.46	24.20	24.36	30.54	30.73	35.09
Experimental	8.0	9.0	14.0		25.0		30.0		35.0

The first two mode shapes of the Route 58 Bridge, as determined analytically, are illustrated in Figures 12 and 13. The first mode shape appears to correspond to the first longitudinal flexural mode, while the second mode shape appears to be the first torsional mode. The deflections and rotations in these illustrations are greatly exaggerated.
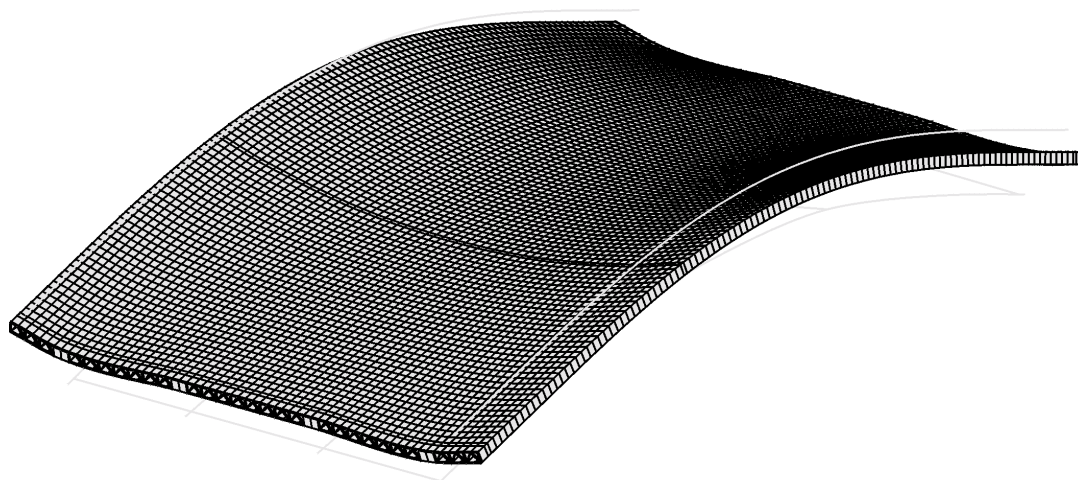

```
3
  2         DISPLACEMENT MAGNIFICATION FACTOR =   78.3
            RESTART FILE = natfreq   STEP 1  INCREMENT 1
     1      EIGENMODE 1     FREQUENCY =   8.01    (CYCLES/TIME)
            ABAQUS VERSION: 5.6-1    DATE: 28-AUG-98  TIME: 16:02:02
```

Figure 12. First Mode Shape

```
          DISPLACEMENT MAGNIFICATION FACTOR =   78.0
          RESTART FILE = natfreq   STEP 1  INCREMENT 1
          EIGENMODE 2     FREQUENCY =   8.13     (CYCLES/TIME)
          ABAQUS VERSION: 5.6-1    DATE: 28-AUG-98  TIME: 16:02:02
```
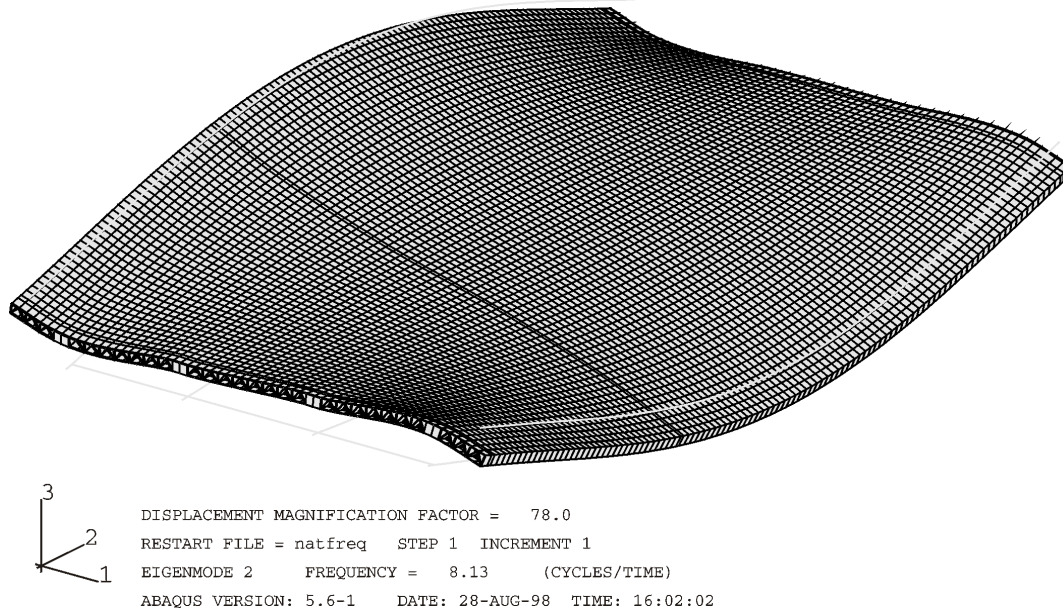
Figure 13. Second Mode Shape

CONCLUSIONS

- The structural behavior of the Reynolds aluminum deck system on the Route 58 Bridge, as recorded during the two field tests, was considered satisfactory in terms of the AASHTO Design Recommendations and was well within the design parameters of the bridge. There was no unusual or unexpected behavior of the aluminum deck detected during the testing.

- Analytical evaluations demonstrated that the static response, natural frequencies of vibration, and mode shapes of a multi-voided, aluminum deck bridge could be accurately predicted with a commercial, general purpose, finite element code.

- Finite element models developed for this project demonstrated the need for significant numbers of elements to accurately represent the geometry and response of a multi-voided deck system. In general, model development for multi-voided deck bridges requires significantly more computational resources than traditional concrete slab and plate-girder bridge models.

- Results from this study clearly demonstrate that aluminum bridge decks are a feasible alternative to reinforced concrete decks from the standpoint of strength and serviceability.

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