

Field Test of a Cable-Stayed Bridge

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This paper describes the results of a field test on a cable stayed bridge subjected to vehicle loads. The test was conducted as part of an extensive research program concerned with the overall field response of a cable stayed bridge under a variety of loading conditions. The test structure is a concrete twin box-girder bridge with a 630-foot main span and two approach spans on either side supported by a single plane of stay cables. The vehicle loading was applied by placing a heavily loaded truck at specific locations in both the interior and exterior lanes of the main span and adjacent spans of the bridge. Response information was obtained from an extensive array of electrical resistance strain gages mounted on dummy reinforcing bars imbedded in the concrete and placed within selected deck segments. Data from the electrical resistance strain gages were automatically recorded using a data acquisition system. Results from these load tests are presented and discussed. The response information is also compared with that predicted from a detailed finite element model of the structure.

INTRODUCTION

Segmentally erected post-tensioned box girders and cable-stayed construction are two relatively recent developments in bridge technology which can lead to aesthetically pleasing and economical structures. In spite

of their clean exterior lines and pleasing form, structures using these technologies are challenging to analyze and design. Much about the behavior of these complex structures in service environments is still unknown. Determining the response under load requires consideration of multi-point and multi-stage post-tensioning, cable stay connection and support, time-dependent creep and shrinkage deflections, cable stay nonlinearity, variation in residual stress and cable preload throughout the structure, and complex live and thermal loadings.

Considerable effort has been expended in developing suitable techniques for the analysis of segmentally prestressed and cable stayed bridges [1]. Present computational and modelling capabilities permit the development of sophisticated and detailed finite element models of complex bridge structures from which a wealth of behavioral information can be easily generated. However, the response information obtained from these computer models is only as good as the model representation of the actual bridge. In addition, response information of a local nature such as transverse shear lag in the deck elements, load transfer through cable connecting mechanisms, and local stress concentrations is generally unavailable from finite element analysis, even when large complex models are employed. To understand the actual behavior of cable stayed bridges under vehicle loadings and to provide response information to assist in the development of more reliable computer models, field testing of large complex bridge structures employing new and innovative design concepts is essential.

Beginning in the summer of 1986 and continuing until the present time, a research team from the University of Virginia and the Virginia Transportation Research Council has undertaken an ambitious and extensive

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program involving the field testing of one segmental, cable-stayed, box girder bridge in Virginia in which construction was recently completed. These field studies have utilized an extensive array of strain gage instrumentation on a number of key elements in this bridge along with an automated data acquisition system for recording response information resulting from a variety of loadings. Previous studies reported elsewhere [2,3] have been concerned with bridge behavior during the construction phase and with the dynamic characteristics of the structure.

The objective of the study described in this paper was to measure and evaluate the response of this completed cable-stayed bridge when it was subjected to a known vehicle load placed at locations along the main and approach spans. The response data was obtained from a series of strain gages placed in several deck segments of the main span. In addition, analytical response data was also provided from a detailed finite element model of the bridge. Comparison of the measured and predicted response data facilitated evaluation of the measured response and also made it possible to better define the format of future field tests on this bridge.

The structure which was the subject of this investigation is a segmentally erected, precast, post-tensioned, cable-stayed box girder bridge which carries Interstate 295 over the James River near Richmond, Virginia. The bridge consists of 28 individual spans including the approach spans. That portion of the structure which is the focus of this particular phase of the overall investigation consists of the central seven-span continuous section of the bridge which

includes the 630-ft. main span over the river and the three approach spans on either side. The approach span segments are 20 ft. long and weigh approximately 145 tons, while the main span segments are each 10 ft. in length and weigh approximately 70 tons. An elevation sketch of the bridge is shown in Fig. 1.

As indicated in the figure, the middle five spans of the bridge including the main span are supported by a total of 52 cable stays arranged in a single plane harp configuration and emanating from two pylons, one on either side of the river. The forces from the cable stays are transferred to the twin box girders through a series of precast delta-frame assemblies located between the girder segments at each stay location as shown in the cross-section sketch of Fig. 2. The main span over the river was constructed as two cantilevers extending from piers located adjacent to the pylons and made continuous by a midspan closure pour. Posttensioning consisted of two parts, temporary post-tensioning bars used during construction, and permanent strand post-tensioning located inside the box segments after the erection was completed. Construction of the bridge was completed in April 1990 and it was opened to traffic in July 1990.

EXPERIMENTAL PROGRAM

Instrumentation and Data Acquisition

Instrumentation utilized in this particular study consisted of an extensive array of electrical resistance strain gages installed in three deck segments of the main span. These

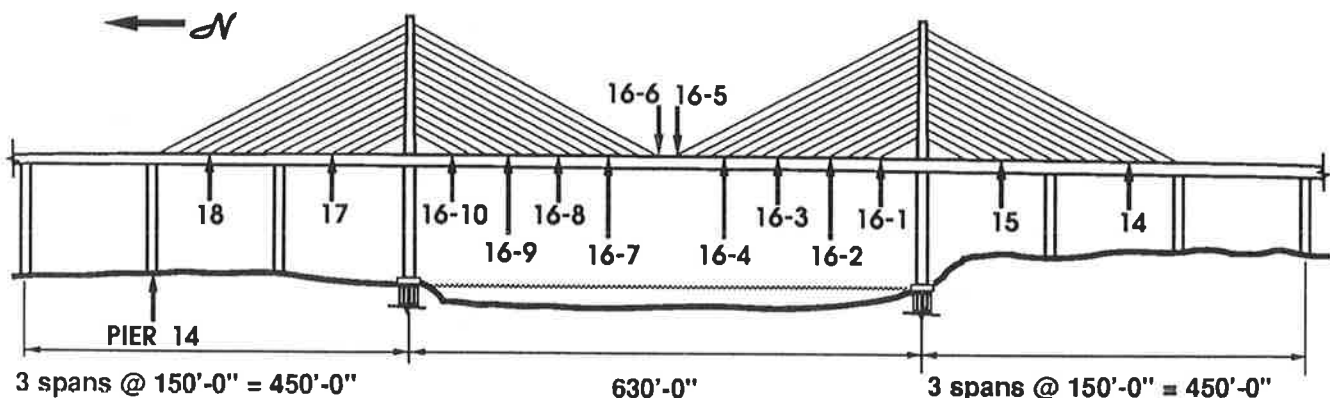


FIG. 1 ELEVATION VIEW OF THE I-295 BRIDGE SHOWING LONGITUDINAL LOCATIONS OF TEST VEHICLE

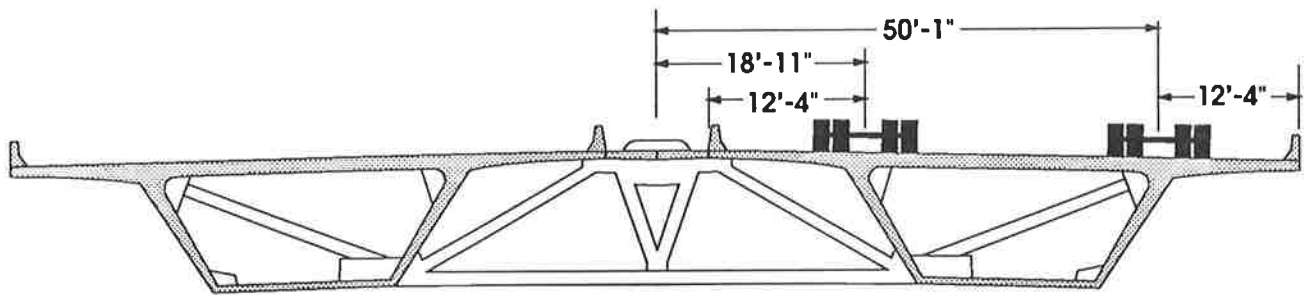


FIG. 2 CROSS SECTION OF TWIN BOX GIRDER SHOWING TRANSVERSE LOCATIONS OF TEST VEHICLE

strain gages were placed in six different deck segments on the south side of the main span corresponding to three different longitudinal locations along the roadway. Specifically, sections corresponding to the north and south bound lanes in segments 33, 48 and 62 in the main span, span 16, were instrumented with gages installed either as single gages or in a rosette configuration. Segment 33 is adjacent to the south pier, segment 48 is approximately at quarter-span and segment 62 is at midspan. The locations of the strain gages within each of the segments is shown in Fig. 3. Each of the gages and rosettes are identified with a number in the figure to facilitate discussion of the measured strains in a subsequent section of this paper.

During installation, each of the gages was mounted on four-foot lengths of reinforcing bar. The gaged reinforcing bars were then placed in the reinforcing cages of the deck segments during the period between cage construction and placement of the cage in the precasting forms. The gaged bars were tied to one of the reinforcing bars in the cage and the lead wires were run along the cage to a block-out in the wall of the box segment. After casting and placement of the segment, the lead wires were retrieved and run directly to a data acquisition system located in

one of the box segments in the vicinity of the instrumentation.

The data acquisition system is a unit manufactured by the John Fluke Company. This system uses a Helios main controller to communicate with a number of individual remote scanning units each of which is located near an instrumented segment. The data is then transmitted from the scanning units to the main controller in digital form and from there to a personal computer where the data is recorded and stored on a hard disk. This data is periodically retrieved and transferred to a computer in the laboratory in spreadsheet format for subsequent analysis and manipulation. The personal computer in the field and the main controller unit are located in a specially designed and fabricated cabinet which is provided with complete environmental controls.

Loading

Loading was applied to the bridge during the field test by means of a heavily loaded dump truck which was loaded to the legal maximum. The total weight of the truck was determined to be approximately 49,720 pounds with 16,820 pounds on the front axle and 32,900 pounds on

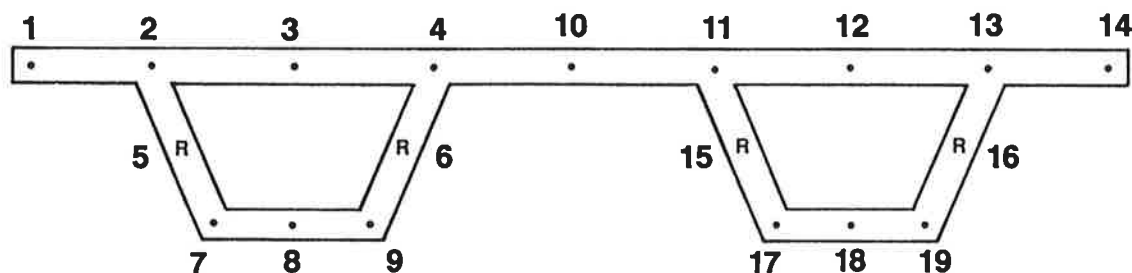


FIG. 3 LOCATION OF STRAIN GAGES IN INSTRUMENTED SEGMENTS

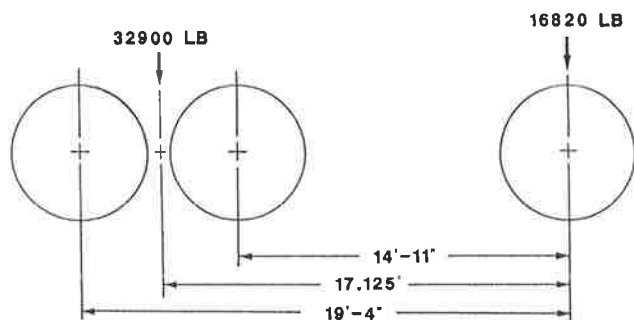


FIG. 4 TEST VEHICLE DIMENSIONS AND LOADS

the rear tandem axle. The axle loads and dimensions between front and rear wheels are shown in Fig. 4. The truck was placed at a variety of locations along the main span and the two adjacent approach spans on either side of the main span. A total of fourteen longitudinal locations were selected along the bridge, one in each of the approach spans and ten in the main span. These longitudinal load locations are identified in Fig. 1 where the number of each longitudinal load position corresponds to the span number and, within the main span, to a position along the span. As indicated in the figure, each load position along the deck also corresponded to the location of a cable stay connection to the deck segments through a delta frame assembly. For each load position, the truck was placed such that the centerline of the rear axles was at the load point with the truck always headed in the southerly direction.

As noted, there were fourteen different longitudinal load locations along the span. At each longitudinal position, there were also four different transverse load positions. These transverse positions, shown in the sketch in Fig. 2, correspond approximately to the two traffic lanes in each of the north and south bound lanes of the bridge. Also, as shown in the figure, these transverse positions place the test vehicle over the webs of the box sections. Thus, during this field test, the test vehicle was located at a total of 56 different positions on the bridge.

At each load position, at least two complete sets of strain data were recorded. Zero readings, with the test vehicle completely off the bridge, were also recorded before the beginning of the test, at the completion of the test, and at six approximately equal intervals during the test. This procedure was adopted to ensure a reliable zero reference for use in

calculating strain increments due to the application of the load. Because of temperature variation during the test procedure, the zero readings did vary slightly and it was assumed that the small variation was linear between readings.

The strains produced by the load application were calculated as the difference between the recorded strain and the zero reading at the time of each test load. Since all strain readings were recorded in spreadsheet format, this calculation of strain due to load application was a straightforward procedure. Stresses corresponding to load application could then be calculated using a modulus of the concrete of approximately 4.5 million psi and also assuming the strains in the top and bottom flanges were predominantly uniaxial in nature.

Results

A careful evaluation and analysis of the recorded strain data indicated several data values that had to be discarded. For example, an apparently defective board in the data acquisition system controlled the majority of data input from segment 33. Consequently, only the strain gages in the bottom flanges of this segment were able to record reliable strain data. Also, analysis of the complete set of data recorded at the quarter-span of the bridge at segment 48 indicated an almost random oscillation of strains even though the data appeared to be consistent between gages. Thus, since reliable strain data was available from two segments, segment 33 and segment 62, the response information from segment 48 was not included in this discussion.

In the discussion that follows, only strain data obtained from segments 33 and 62 are presented and discussed. Also, since data patterns were similar for loads in the north and south bound lanes, only the northbound lane loadings are discussed. It should be noted here that the test loading applied, a tandem-axle dump truck loaded to the maximum legal limit, is a loading of relatively small magnitude compared to the self weight of the bridge. The total vehicle weight was approximately 50,000 lbs while the weight of each of the 124 single segments making up the main span was approximately 135,000 lbs. The corresponding strains and stresses produced by this load were also small in absolute magnitude. For example, maximum

strain values recorded, due to the application of the test load, were on the order of 5 to 10 microinches per inch. Although small, the reliability of these values is attested to by the fact that the readings were reproduced with little error under repeated load applications and recorded values of strain were consistent between different gages and different locations.

As noted, only the strain gages in the lower flanges of segment 33 were operative. Recorded strains from gages 17, 18 and 19, which are located in the lower flange of the northbound lane of segment 33 (see Fig. 3), are presented in Figs. 5 and 6 for loads in the exterior and interior lanes respectively. The strains are plotted as a function of vehicle location where the distance denoting the vehicle position is measured from pier 14. This reference point for location of the test load was chosen arbitrarily and represents the origin of the first span in which the load was placed. Thus, the first load point, at midspan of span 18, is plotted at a distance of 75 ft. From Figs. 5 and 6, it may be observed that the recorded strains are essentially negligible until the vehicle enters the main span. The strain responses, which are similar for all three gages shown, are of the form that would be expected since segment 33 is located adjacent to the south pier in the negative moment region of the continuous span. Relative to the reference point for the distance parameter, the gages in segment 33 are located approximately 920 ft from pier 14. For the vehicle in the exterior traffic lane, the maximum strain recorded was on the order of 8.0 microin./in. corresponding to a

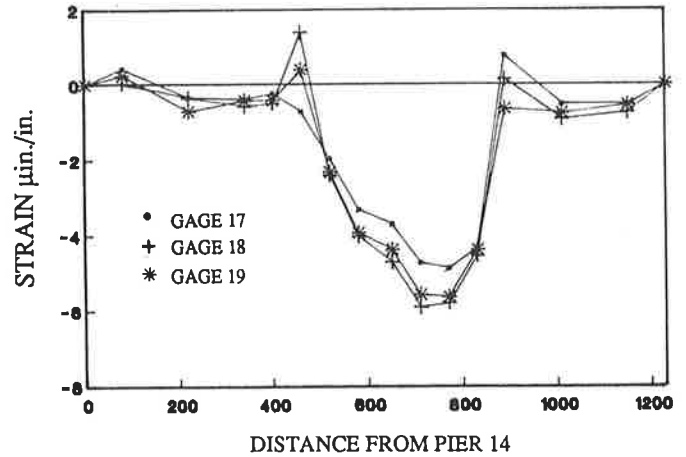


FIG. 6 LOWER FLANGE STRAINS, SEG 33 (NBL), FOR VEHICLE LOCATIONS IN INTERIOR LANE

stress value of approximately 36 psi. With the load in the interior lane, the variation of strain as a function of load position is the same but the maximum strain value recorded is approximately 6 microin./inch.

Strains measured in the top flange, or deck region, of segment 62 are shown plotted in Figs. 7 through 10. Figure 7 represents strain response for gages 1 through 4 of segment 62 for the load in the exterior lane while Fig. 8 provides the same data from gages 11 through 14. Figures 9 and 10 show strain data for the same gages as Figs. 7 and 8 respectively, but for the load traversing the interior lane. There appears to be considerable scatter in these plots but the actual strain values plotted are small and variations are actually less than 1.0 microin./inch. These gages are located longitudinally within a few feet of midspan and the results as shown in the figures

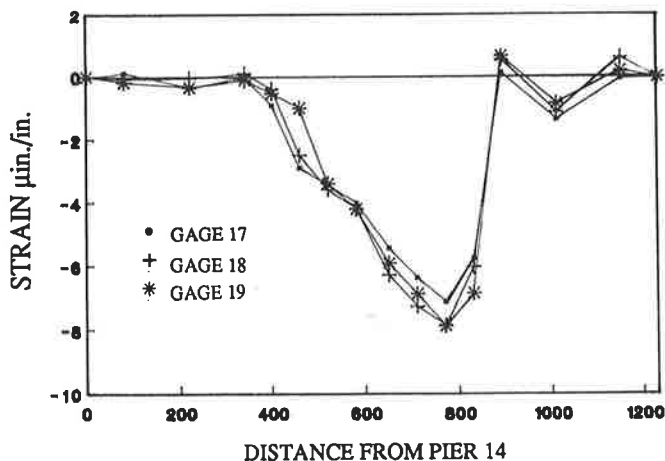


FIG. 5 LOWER FLANGE STRAINS, SEG 33 (NBL), FOR VEHICLE LOCATIONS IN EXTERIOR LANE

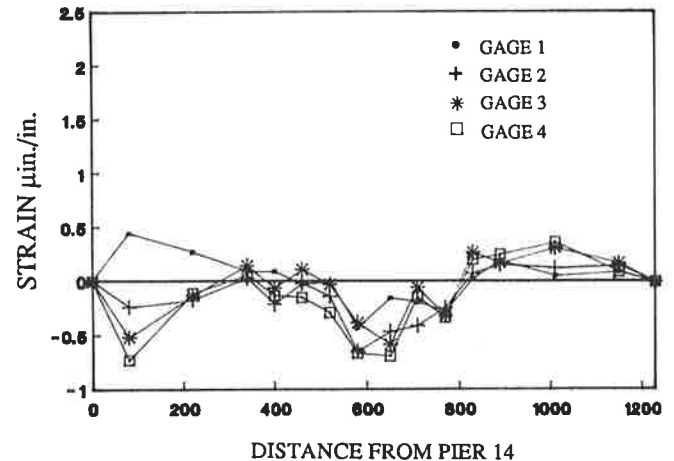


FIG. 7 UPPER FLANGE STRAINS, SEG 62 (SBL), FOR VEHICLE LOCATIONS IN EXTERIOR LANE

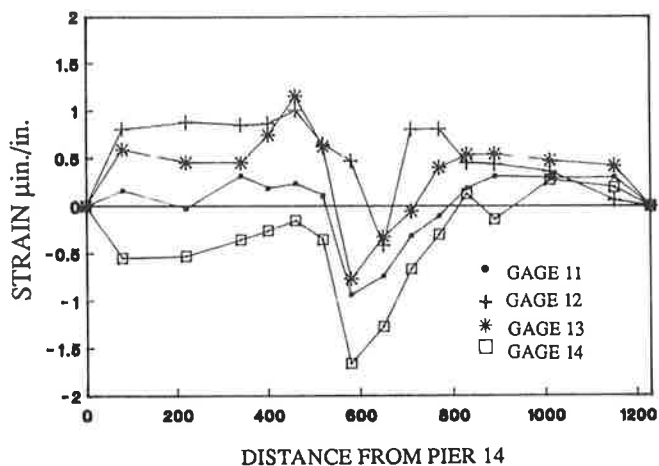


FIG. 8 UPPER FLANGE STRAINS, SEG 62 (NBL), FOR VEHICLE LOCATIONS IN EXTERIOR LANE

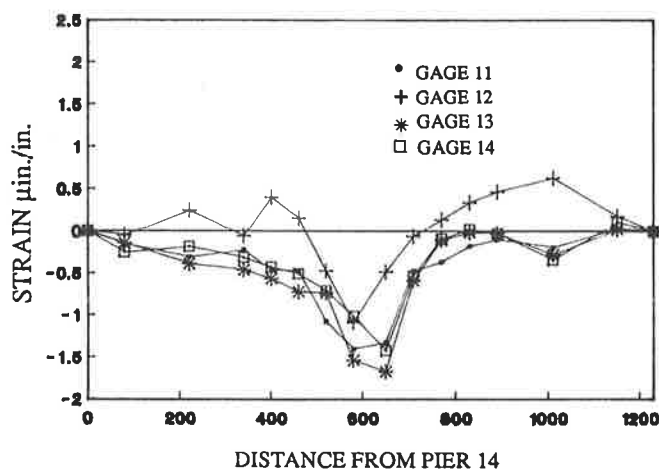


FIG. 10 UPPER FLANGE STRAINS, SEG 62 (NBL), FOR VEHICLE LOCATIONS IN INTERIOR LANE

indicate small compressive strains for the load at midspan as expected. The neutral axis for the box segments in the deck is located much closer to the top flange and hence the measured strains would be expected to be smaller than those observed in the lower flange. The maximum compressive strains recorded are on the order of 1.0 to 1.5 microin./in., even less than might be expected. This apparent anomaly is considered further in the next section. Although there is somewhat more scatter in these data than in those observed in segment 33, the trends are the same and the values are relatively consistent.

Strain data from gages located in the lower flange of segment 62 are plotted against longitudinal load position in Figs. 11 through 14. Figures 11 and 13 provide strain data

recorded from gages 8 and 9 in the southbound portion of the segment as a result of loading in the exterior and interior lane respectively. Figures 12 and 14 provide the same data from gages 17, 18 and 19 located in the lower flange of the northbound portion of segment 62. It may be seen that all of these plots have the same general character with compressive strains recorded for the load in the approach spans and larger tensile strains recorded for the load in the main span. For gages 8 and 9, located in the unloaded side of the segment, maximum strains of approximately 2.0 to 2.5 microin./in. were observed while gages 17, 18 and 19 recorded maximum strains of approximately 2.5 to 3.0 microin./inch. This would indicate that although slightly more load is carried by the section on which the load is placed, there is

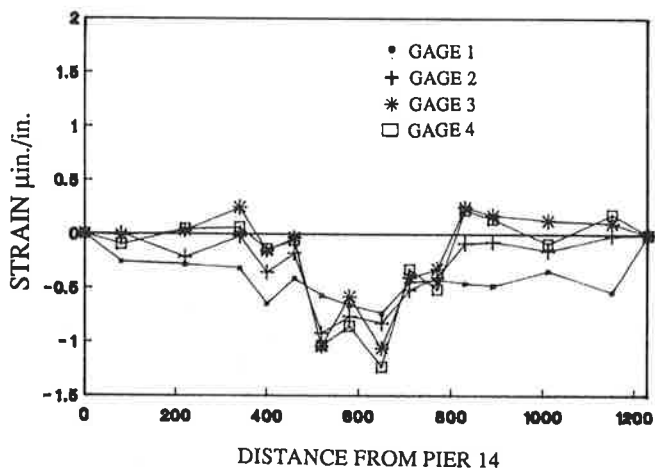


FIG. 9 UPPER FLANGE STRAINS, SEG 62 (SBL), FOR VEHICLE LOCATIONS IN INTERIOR LANE

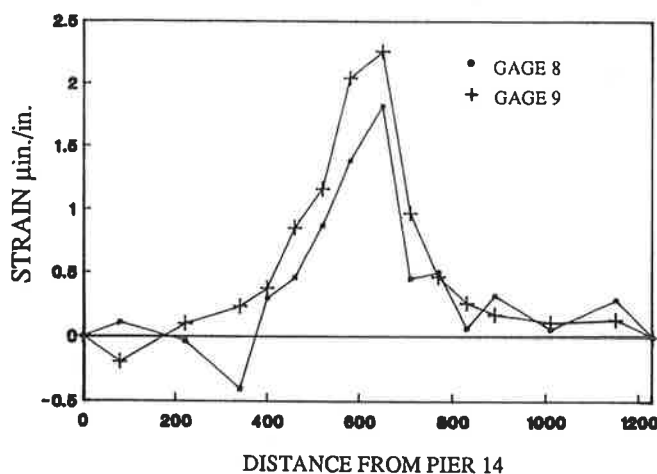


FIG. 11 LOWER FLANGE STRAINS, SEG 62 (SBL), FOR VEHICLE LOCATIONS IN EXTERIOR LANE

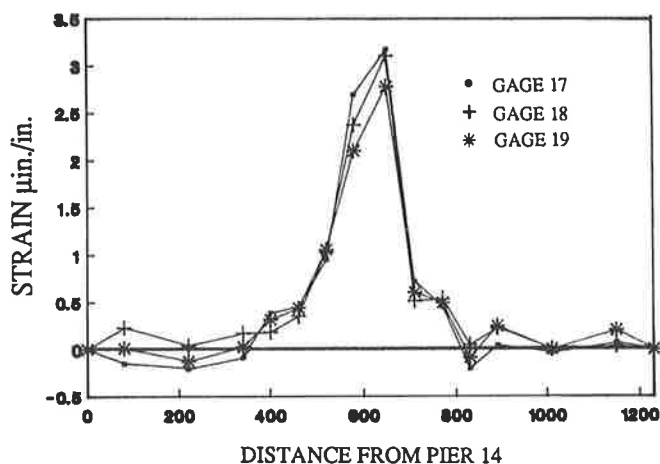


FIG. 12 LOWER FLANGE STRAINS, SEG 62 (NBL), FOR VEHICLE LOCATIONS IN EXTERIOR LANE

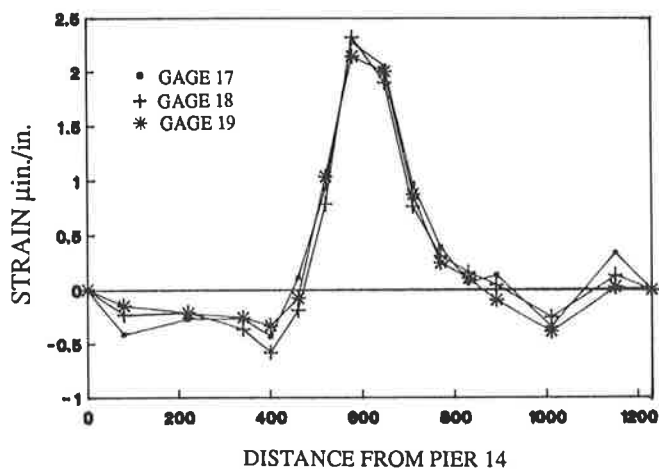


FIG. 14 LOWER FLANGE STRAINS, SEG 62 (NBL), FOR VEHICLE LOCATIONS IN INTERIOR LANE

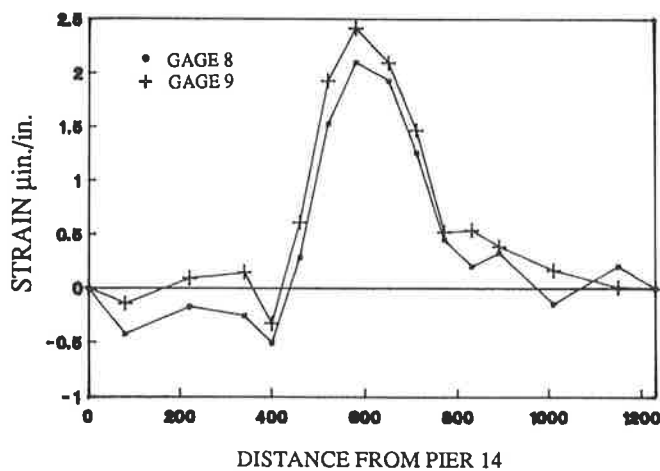


FIG. 13 LOWER FLANGE STRAINS, SEG 62 (SBL), FOR VEHICLE LOCATIONS IN INTERIOR LANE

almost uniform lateral distribution of the load throughout the segment. For all of these gages, the strain behavior is similar regardless of load or gage location.

Considerable strain data was obtained from these tests and this data provides a basis for predicting the behavior of this complex structure due to a single vehicle load. Nevertheless, field testing for every possible loading condition is not possible and it would be desirable to have a computer model to predict such stresses. In addition, there is definitely a need to validate the strain values measured during the field tests described. For these reasons, an analytical study was undertaken to develop a finite element model which could be used to validate field test data as well as provide a mechanism for predicting response under other more complex loadings.

ANALYTICAL STUDY

The emphasis of this paper was on presentation and discussion of experimental response. Consequently, only a brief description of the analytical model and limited results will be provided here. A more detailed description of the finite element model may be found elsewhere in the study by Lissenden [3]. The finite element model used in this study, primarily for validation purposes, was a three-dimensional model in which the deck segments were represented in a two-dimensional sense as a series of beam elements. Loadings were applied to the model as concentrated loads and moments at the locations corresponding to the actual position of the vehicle. The magnitudes of the model loads were calculated to represent the wheel loads of the test vehicle. Properties of the cross section of the deck segments were used to define the element properties and stresses in the top and bottom fibers of the beam elements, corresponding to the top and bottom flanges of the box segments, were calculated for all positions of the test load.

Stresses in the top and bottom flanges of the deck segment in the finite element model are plotted as a function of load position in Figs. 15 and 16. The magnitudes of the plotted stresses represent the average stress in the top and bottom flange for segments 33 and 62. For segment 33, the maximum tensile stress in the top flange was calculated to be approximately 8 psi while the maximum compressive stress in the bottom flange was approximately 28 psi. For segment 62, the

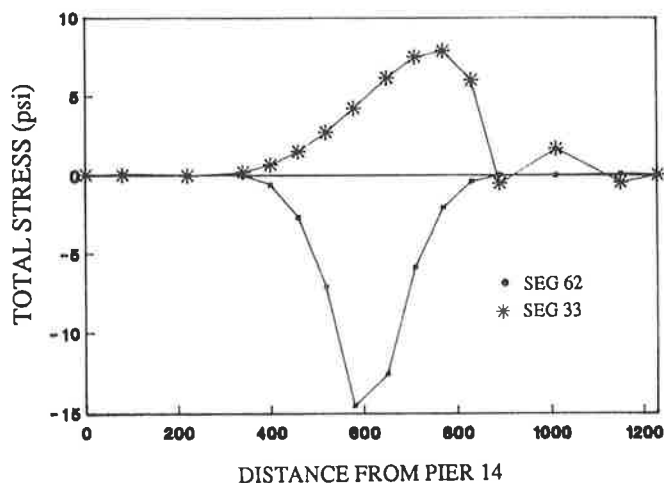


FIG. 15 UPPER FLANGE STRESSES FOR FINITE ELEMENT MODEL, SEGMENTS 33 & 62

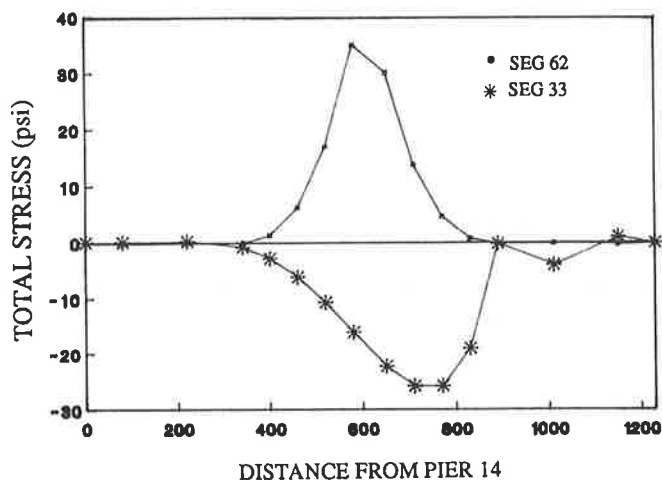


FIG. 16 LOWER FLANGE STRESSES FOR FINITE ELEMENT MODEL, SEGMENTS 33 & 62

corresponding stresses were calculated to be approximately 15 and 37 psi respectively. For the midspan location, stresses were determined to be essentially zero for the load anywhere in the approach spans while small stresses were produced in the location corresponding to segment 33 for the load in the spans adjacent to the segment.

For validation purposes, it is of interest to compare the stresses determined from the strains measured during the field test with those stresses predicted from the finite element model. In Figs. 5 and 6, the variation in strain in the bottom flange of segment 33 is plotted as a function of load position. The corresponding values of stress predicted by the analytical model are plotted in Fig. 16. Comparison of the two figures indicate close

agreement in terms of the overall shape and magnitudes of the response. Both measured and predicted response are essentially zero for load positions from zero to approximately 350 feet when the load enters the main span. Similar variations in response are also observed for distances greater than approximately 900 feet when the load traverses the final two approach spans. Furthermore, the maximum stress as determined from the experimental strain data was approximately 30 psi while the predicted maximum for the same loading was 28 psi, extremely close agreement.

Comparisons between measured and predicted stresses in the vicinity of the bridge midspan may be made by examination of Figs. 7, 8, 11 and 12 for measured data and Figs. 15 and 16 for analytical data. These comparisons raise some interesting questions that have yet to be satisfactorily answered. Data measured in the top flange of segment 62 as shown in Figs. 7 and 8 show some scatter but an average value of maximum compressive stress may be estimated to be on the order of 4 to 6 psi. The corresponding value of stress predicted by the finite element model was calculated to be 14.5 psi as indicated in Fig. 15. These measured and predicted values of maximum stress differ by a factor of approximately 3. Similar differences were observed to exist between measured and predicted stresses in the lower flange of the box segments at midspan. From Fig. 11, the value of maximum tensile stress as determined from the measured strains in segment 62 was found to be approximately 12 psi while the corresponding stress value predicted by the model was calculated to be 37 psi, as shown in Fig. 16. Again, the measured and calculated values of response differ by a factor of approximately 3.

A careful review and evaluation of the experimental procedure for measuring and recording strains have indicated no source of error in the experimental data. Likewise, the finite element model appears to be a reasonable representation of the bridge according to the plans. If, in fact, both measured and predicted response values are to be accepted, this would suggest the possibility that there is some difference in the behavior of the actual structure that is not adequately represented in the finite element model. For example, one factor under consideration is the possibility that complete continuity does not exist at midspan of the

bridge. This main span was constructed as two cantilevers from either side of the river and these two sections were then joined by a closure pour at midspan. The subsequent post-tensioning was intended to render the completed span continuous but there is the possibility that the continuity may not be complete. This could lead to lower stresses at midspan. This and other possible factors which could lead to differences between measured and predicted values of stress are currently being investigated.

In spite of certain differences that exist, the similarities between the stresses predicted by the finite element model and those measured in the field test indicate that finite element models can be a useful analysis tool for predicting the behavior of complex structures such as cable stayed bridges. However, care should be taken that the models used in the analyses accurately represent the actual structure as built. This can only be achieved through the use of carefully controlled and monitored field tests.

SUMMARY AND CONCLUSIONS

A load test of a cable stayed bridge was conducted by placing a known vehicle load at various positions on several spans of the structure and measuring the response from strain gages imbedded in deck segments at three different locations of the main span. Variations in strain as a function of load position were plotted for strain gages in the top and bottom flanges of the segments. Similar response data was calculated from a finite element model developed as a part of the study. Comparisons of measured and calculated response were made as a means of validating the computer model and to provide credibility to the experimental data collected and the system used to collect and record the data.

Results from the field test provided considerable data regarding the behavior of the bridge under a specific vehicle load. For example, the study indicated that reliable experimental strain data can indeed be measured even when the applied load is only

approximately 0.5 percent of the self weight of the bridge. The measured strain data, when plotted as a function of load position, was found to vary in a manner generally consistent with expected response. Results from the analytical study indicated that maximum values of stress in the vicinity of the pier varied from approximately 8 psi in tension to 28 psi in compression. At midspan of the bridge, maximum values of stress were found to vary from 13 psi in compression to approximately 38 psi in tension. Comparison between measured and calculated stresses indicated close agreement with regard to overall trends and magnitudes in most regions of the bridge but differences on the order of a factor of 3 were observed at midspan. Various explanations for this apparent anomaly are currently under study.

ACKNOWLEDGEMENT

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