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KTC-06-05 /FRT81-82-97-1F

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PERFORMANCE EVALUATION OF BRIDGES WITH STRUCTURAL BRIDGE DECK OVERLAYS (SBDO)

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PERFORMANCE EVALUATION OF BRIDGES WITH STRUCTURAL BRIDGE DECK OVERLAYS (SBDO)

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16. Abstract

Structural Bridge Deck Overlay (SBDO) involves applying 6 to 10 inches (150 to 200 mm) of normal weight, class AA, reinforced concrete directly to a bridge's original slab. The overlay is designed to increase the deck elevation to an extent that standard highway resurfacing procedures can continue uninterrupted up the edges of the bridge. Otherwise, excavation along the bridge approaches or jacking of the superstructure is required to insure proper elevation.

Experimental static field tests were conducted on three different bridges: (1) Simply supported prestressed concrete I-girder bridge; (2) cast-in-place reinforced concrete continuous haunched girder bridge; and (3) cast-in-place reinforced concrete simple span bridge. Field tests were conducted prior to the concrete overlay process and following the application of the concrete overlay.

Based on the results obtained in this research study, a significant advantage is noted due to the additional deck thickness. The addition of the SBDO increases the load carrying capacity of the bridge in addition to providing a wider bridge deck and new and code compliant barrier walls.

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EXECUTIVE SUMMARY

Structural Bridge Deck Overlay (SBDO) involves applying 6 to 10 inches (150 to 250 mm) of normal weight, class AA, reinforced concrete directly to a bridge's original slab. The procedure was developed by the Kentucky Transportation Cabinet in 1992. The overlay is designed to enhance deck elevations to an extent that standard highway resurfacing procedures can continue uninterrupted up to the edges of a bridge. Prior to the SBDO, extensive excavation was required at the approaches of a bridge to facilitate an adequate grade transition while repaving. This work involved removing tons of existing surface material and could cost in excess of 300,000 dollars. In eliminating the need for exaction, the SBDO can utilize existing grade funds to provide a structure with a new deck, crash tested barrier walls, and an estimated 40 additional years of service to the tax payers.

The effect on load carrying capacity of the SBDO is reported for bridges with: (1) prestressed concrete I-girder span (I64 over KY32), (2) cast-in-place reinforced concrete continuous haunched girder span (I64 over Triplett Creek), and (3) cast-in-place reinforced concrete simple girder span (I64 over Triplett Creek). The assessment on load carrying capacity is accomplished by correlating the results from static, experimental testing with analytical findings

Reusable strain gauges were placed on the girders and on the bridge deck at midspan and over the pier. Linear Variable Differentiable Transducers (LVDTs) were also used to measure the vertical deflection of the girders at midspan. Static testing was accomplished by using two fully-loaded, tandem axle trucks to induce the displacements and strains on the bridges. Testing stations were designated along the longitudinal direction of the bridge that would maximize the response of the bridge components.

After testing, the strains obtained from the gauges on the concrete decks were combined with the strains recorded along the cross section of the girders to determine the distribution of stress across the composite cross section. Using this procedure, the location of the neutral axis under the static test loads could readily be obtained, and an assessment of the study hypothesis could be made. Likewise, the vertical displacement records offered insight into the response of the girders both before and after the SBDO. A relatively smooth trace of the strain value across the face also points to an adequate bond between the two slab surfaces.

Also significant was the apparent contribution to the load distribution offered by the stiffer transverse slab element after the SBDO. Both the strain and displacement values show the exterior girders (beams opposite those which were loaded) contributing more to the overall response of the structure under static testing.

Two conclusions were drawn from the experimental results of the girder strains and vertical displacements: (1) SBDO does offer increased load carrying capacity through the new composite section, and (2) load distribution between the adjacent girders is improved due to the stiffer transverse slab element.

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1.0 INTRODUCTION

1.1. STRUCTURAL BRIDGE DECK OVERLAY

1.1.1. Introduction

As the Commonwealth of Kentucky completed asphalt resurfacing on the interstate system and updated its bridge inventory to meet the new Federal Highway Administration's requirements for traffic barriers, some consideration had to be given to the impact on the bridge deck elevation. A cost effective technique for marrying the components is critical. One such procedure makes use of a concrete deck overlay (ACI 1999). This technique is particularly attractive due to the relative ease by which new or existing grades can be utilized. Structural Bridge Deck Overlay (SBDO) requires little excavation at the road to bridge interface thereby allowing traditional asphalt paving techniques (i.e., built-up resurfacing) to proceed with minimal preparation.

However, SBDO involves a significant increase in the load the structure must support. Recognizing this effect, the Kentucky Transportation Cabinet commissioned research to verify the procedure is an acceptable rehabilitation technique. As a secondary benefit, the strength gain associated with the deck overlay can be ascertained through experimental and analytical study. In fact, as concrete bridges reach the latter stages of their service life, consideration must be given to repair, rehabilitation, or replacement of the structure. When replacement is not feasible and/or cost effective, transportation agencies throughout the United States are turning to innovative rehabilitation techniques, such as concrete overlays.

Several types of concrete bridge deck overlays are available. The American Association of State Highway and Transportation Officials (AASHTO 1996) mentions some of the more common overlay options: latex-modified concrete overlays, low slump concrete overlays, and thin resin-based mortar overlays. These thin type overlays concentrate more on increasing the wear capabilities of bridge decks (i.e., are types of resurfacing techniques) or preventing detrimental effects from environmental exposure rather than improving load transfer or the load carrying capacity of the bridge (ACI 1987). The focus of this report is on the SBDO option, which is discussed in detail by Blakeman (1998). A summary of the technique is presented below as a means of reference for this report.

1.1.2. Description of Technique

Structural Bridge Deck Overlay relies on the application of six to ten inches (157.4 to 254.0 mm) of normal weight concrete directly onto the existing bridge deck surface. The existing surface is prepared by scarifying the original slab, paying particular attention to areas where delamination has occurred. Steel reinforcement is placed on the newly prepared surface using bar chairs or slab bolsters, and the new concrete wearing surface is poured into place. Since the overlay is reinforced, the steel area in the compression zone across the girder and deck cross-section is increased. The net result is a bridge with improved cross-sectional properties that outweighs the additional load applied, and matches adjacent grades where resurfacing operations have been completed.

1.2. LITERATURE REVIEW

The objectives of this research were centered on an experimental and analytical study into the behavior of bridges before and after placement of a Structural bridge deck overlay. A literature review was conducted to investigate aspects of the work that would be required to complete the experimental portion of the study. The following topics were identified as major areas where previous research information would be important: (1) load distribution and load rating of highway bridges, and (2) experimental testing of bridges. A summary of the literature review for each topic is presented below.

1.2.1. Load Distribution

Background information on the development of wheel load distribution factors can be found in Culham and Ghali (1977), Hays et al. (1986), Sanders and Elleby (1970), and Stanton and Mattock (1986). In work completed prior to the new AASHTO formulas, Tabsh (1994) presented a simple method for the computation of live load distribution factors for highway girder bridges, taking into account the longitudinal and transverse effect of the truck loads on the bridge. Verification of the proposed equations was completed by comparing results on non-composite and composite steel girder bridges.

Chen (1995a and 1995b) studied load distribution in bridges with unequally spaced girders. AASHTO empirical formulas for estimating live load distribution factors were compared to results from the refined method. Parametric studies were conducted with a number of real bridge examples that were simply supported, non-skewed, and had no intermediate diaphragms between the girders. Refined load distribution equations were proposed. Subsequent work by Chen and Aswad (1996) sought to review the accuracy of the formulas for live load distribution in flexure contained in the LRFD Specifications (AASHTO 1994) for prestressed concrete I-girder bridges. It was concluded that the use of a refined method, namely finite element analysis, generally leads to a reduction of the lateral load distribution factor in I-beams when compared to the simplified LRFD guidelines. Fu et al. (1996) conducted comparable work by field testing four steel I-girder bridge structures under the effect of real moving truck loads. The results indicated that all the code methods (AASHTO, LRFD, and the Ontario Highway Bridge Design Code [OHBDC]) produced higher distribution factors.

Further revisions to load distribution equations were presented by Tarhini and Frederick (1995). Whereas the AASHTO procedure made assumptions aimed at simplifying the analysis required, their finite element analysis revealed that the entire bridge superstructure acts as a unit rather than a collection of individual structural elements. The paper correlated distribution factor results obtained from published field test data, the proposed formulas, and the AASHTO method. Of importance to this study is the fact that the influence of the concrete deck in load distribution was confirmed after the effect of cross bracing on the wheel load distribution factor was found to be negligible.

Ghosn and Moses (1996) reported on the capability of typical prestressed concrete I-beam bridge systems to continue to carry loads after the failure or the damage of one or more of the bridge's main load carrying members. The effect of Structural truck loads was considered. Analytical results from the study were compared to those obtained from full scale field tests.

Bakht and Jaeger (1985) concluded when the skew angle of a bridge is small (e.g., less than 20^{o}), it is frequently considered safe to ignore the angle of skew, and analyze the bridge as a right bridge whose span is equal to the skew span. The research demonstrated other factors influenced response characteristics when comparing skew and right bridges. The simple procedure presented was intended mainly for the determination of longitudinal moments in slab-on-girder bridges.

1.2.2. Experimental Testing

In recent years, several studies have been published on load testing of bridges in either a controlled (i.e., known weight trucks) or ambient traffic condition. However, the focus of the individual research efforts has been many and varied. For example, in research on nondestructive testing of a concrete slab bridge, Aktan et al. (1992) reported on the use of known weight trucks to obtain static bridge response as a basis for nondestructive bridge evaluation (NDE). Experimental data taken from the static and dynamic testing of the bridge were used to calibrate a finite element model. A similar study was conducted by Cook et al. (1993) on a prestressed flat slab bridge. Experimental and analytical research was conducted with the primary objectives of: testing the bridge for service, fatigue, and ultimate loads; developing analytical models to predict the performance of the system; and verifying the analytical results by comparing them with those obtained from experimental data.

In Helba and Kennedy’s (1995) study, equations for the design and analysis of skew bridges were developed from the analysis of a prototype composite bridge subjected to Ontario Highway Bridge Design Code (OHBDC) truck loading. Previous research by Griffin (1997) used known truck loads in order to analyze prestressed concrete I-girder bridges by comparing experimental data with analytical models.

1.3. SCOPE OF RESEARCH

1.3.1. Location of the Bridges

Both bridges are located in the United States interstate system. Interstate 64 is a major artery in Kentucky's highway transportation system, linking the Eastern portion of the Commonwealth, namely Ashland, with Central Kentucky (Lexington and Louisville) as well as Southern Indiana (Jeffersonville and New Albany). Beyond Kentucky, Interstate 64 links the central United States and St. Louis, Missouri, with the eastern seaboard and Richmond, Virginia. As such, the bridges serve a vital role in the interstate commerce of the neighboring states while providing the residents of Kentucky with an efficient travel route to major metropolitan areas. To maintain this vital role, it is critical each element of the system is operational, functional, and safe.

The I-64 Bridge over KY 32 is located in Rowan County near the county seat of Morehead. The I-64 Bridge over Triplett Creek is located to the Southwest of Morehead. Figure 1 illustrates the location of Rowan County in the Northeastern portion of the Commonwealth. Figure 2 highlights the location of the study bridges within Rowan County.

1.3.2. Bridge Descriptions

The two bridges are of varying composition and design philosophies, reflecting the evolution of bridge construction in Kentucky within the past few decades. The I-64 Bridge over KY 32 was built in 1967 and is a three span bridge with mixed construction. The West Span (Span 1) has six prestressed concrete I-girders with concrete intermediate diaphragms located at the midspan and measures 50-ft (15.24-m). An illustration of an AASHTO prestressed concrete I-girder and cast-in-place concrete intermediate diaphragm are given in Figures 3 and 4, respectively. The Center Span (Span 2) has eight prestressed concrete I-girders with concrete intermediate diaphragms located at the third points and measures 70-ft (21.34-m). Finally, the East Span (Span 3) has six cast-in-place concrete girders with no concrete intermediate diaphragms and measures 35-ft (10.67-m). Each span is simply supported. The original bridge deck measured 7.5-in (190.5-mm) thick. The bridge has a slight skew of approximately three degrees. The skew of a bridge is measured as the angle bounded by the centerline of the pier and a line perpendicular to the girders. Figure 5 illustrates the skew of a bridge girder relative to the pier support.

The I-64 Bridge over Triplett Creek also began service in 1967 and is a six span bridge with four cast-in-place concrete girders. The interior spans (Spans 2, 3, and 4) are of continuous haunched-girder design, with the end spans (Spans 1, 5, and 6) being simply supported. The bridge was constructed with a 30 degree skew angle. Each of the three end spans has no concrete intermediate diaphragms. Concrete intermediate diaphragms are located at the midspan of Spans 2 and 4, while Span 3 has concrete intermediate diaphragms at the third points. The intermediate diaphragms were not staggered to account for the skew. The bridge deck above the continuous cast-in-place girders was 7.5-in (190.5-mm) thick prior to placement of the overlay.

1.3.3. Traffic Loading Profile

As mentioned previously, Interstate 64 is a vital cross-state route through Central Kentucky. As such, both bridges can be expected to experience normal passenger vehicle and commercial truck traffic patterns. Dump and tractor-trailer type trucks will be in the normal traffic loadings. Interstate 64 is not within the extended-weight coal haul road system created by Kentucky's General Assembly in 1986, therefore routine and frequent overload or permit loadings are not expected in the traffic profile.

Since these bridges were constructed in the late 1960s, the design considerations parallel those dictated by AASHTO for current bridge projects. In fact, the current lane load guidelines are derived from the truck train loadings originally outlined in the 1935 AASHO Specifications (AASHTO 1996). AASHTO equivalent lane and truck loads are pictured in Figures 6 and 7, respectively (the HS-group represents a tractor-trailer configuration). The gross weight of the H truck group is 42,000 lbs (178-kN) while the gross weight of the HS truck group is 72,000 lbs (320-kN). Since the legal load allowed in Kentucky without permit is 80,000-lbs (356-kN), the need for additional load carrying capacity in the bridge inventory is apparent.

1.3.4. Research Objectives

Analytical studies summarized in later sections indicate a 20 to 25 percent increase in the load carrying capacity of the sample bridges rehabilitated with SBDO. This study seeks to verify these analytical findings by reporting on the experimental testing of bridges before and after SBDO. Secondary to this objective will be an investigation into the effect on load distribution the SBDO may provide. As an added benefit, the bridges selected for this research are of mixed construction. Therefore, the effect on load carrying capacity of the SBDO technique will be illustrated for bridges with: (1) simply-supported precast prestressed concrete girders, (2) continuous cast-in-place reinforced concrete haunched girders, and (3) simply-supported cast-in-place reinforced concrete girders. Among these three types of construction, the majority of the bridge inventory within Kentucky will be covered.

1.4. RESEARCH SIGNIFICANCE

As the bridge inventory in the United States matures, significant consideration must be given to the cost-effectiveness of repair versus replacement schedules. With the increased dependence on the interstate system for commercial and personal travel, alternatives which focus on bridge replacement often are not feasible. Inconvenience to the motorists and long construction schedules at the mercy of environmental conditions are factors weighing against replacement options. Consequently, repair or rehabilitation techniques which offer extended service life while maintaining the same level of safety can be extremely attractive given that they are often completed in much less time with fewer materials.

One such technique which serves this purpose is the use of a Structural Bridge Deck Overlay (SBDO). This research summarizes the findings of experimental studies on two bridges in the Commonwealth of Kentucky which have undergone rehabilitation through the implementation of a SBDO treatment. The results obtained from these investigations can be used to substantiate analytical tools and procedures for predicting the benefit to the load carrying capacity of bridges offered by SBDO.

2.0 I-64 BRIDGE OVER KY 32: SIMPLY SUPPORTED PRESTRESSED CONCRETE I-GIRDERS

2.1. ANALYTICAL RESULTS

2.1.1. Introduction

Using the stress-strain relationship of concrete and steel in combination with the basic principles of mechanics, an evaluation of the bending moment capacity at the critical section of the bridge span can be made. For the simply-supported span of the I-64 Bridge over KY 32, the critical section is located at the midspan. The analytical evaluation to be made is based on the following set of assumptions:

1. Strain distribution remains linear in the elastic range. This assumption is based on the theory plane sections prior to bending remain plane and perpendicular to the neutral axis after bending, and
2. Strain in the reinforcing steel bars and surrounding concrete are equal prior to the concrete cracking.

2.1.2. Moment-Curvature Relationship

Due to the non-prismatic nature of the cross-section at the critical location - prestressed concrete I-girder with reinforced concrete bridge deck - three different locations of the neutral axis must be considered in the calculations. The three locations are the neutral axis in: (1) the web of the girder, (2) the flange of the girder, and (3) the bridge deck. The depth to the resultant compressive force in the cross-section is calculated by equating the value obtained from integrating the parabolic compressive stress block to the value taken from the moment equilibrium equation about the neutral axis for each location. If k and k_2 are defined as coefficients relating the depth to the centroid of the tension steel in the girder (*d-distance*) to the location of the neutral axis and the location of the resultant compressive force, respectively, then expressions for the moment (M) and curvature (ϕ) of the cross-section can be written as follows:

$$M = Td\left(1 - \frac{k_2\, kd}{d}\right) \tag{1}$$

$$\phi = \left(\frac{\varepsilon_c}{kd}\right) \tag{2}$$

where ε_c is the compressive strain in the outermost fiber of the concrete. Having derived the basic equations for analysis, the moment-curvature diagram for the critical section (i.e., midspan) is obtained by considering four strain levels:

1. Maximum concrete strain in tension (cracking point): $\varepsilon_c = \varepsilon_{cr}$,
2. Maximum steel strain in tension (yield point): $\varepsilon_s = \varepsilon_y$,
3. Intermediate concrete strain in compression: ε_c=0.0015, and
4. Maximum concrete strain in compression (ultimate point): $\varepsilon_c = \varepsilon_{cu} = 0.0030$.

Figure 8 depicts the moment-curvature diagram for the I-64 Bridge over KY 32 both before and after SBDO. As can be seen from the graph, the moment capacity of the cross-section has increased approximately 23 percent after the SBDO.

2.2. INSTRUMENTATION

2.2.1. Introduction

Once this bridge had been identified as the experimental subject of study, an instrumentation plan was prepared to provide guidelines for comprehensive static testing. Planning for the instrumentation began before the rehabilitation work on the bridge was started. In cooperation with Kentucky Transportation Cabinet personnel, instrumentation was proposed which would take advantage of the opportunity to study both the "before" and "after" load carrying capacity of the bridge. The experimental study offers the ability to substantiate the analytical findings outlined above. The information in this section is a summary of the instrumentation plan and record of how this plan was implemented in the field. The experimental testing allowed for the influence of SBDO to be studied for simply-supported, precast prestressed concrete girders. This type of construction can be considered a representative of a significant number of the bridge inventory found in Kentucky.

The strain diagram for the composite cross section can be determined from the information obtained by the three girder strain gauges and the strain gauges mounted on the deck. Strain data across a girder cross section is essential for determining the neutral axis of the composite cross section under various loadings and how the neutral axis varies as the load traverses along the longitudinal axis of the bridge. Strain comparisons between the bridges before and after the placement of the SBDO is used to first investigate if the technique produces a new section which acts compositely. Further analysis will then show how forces (stresses) are transferred among the girders and ascertain the contribution to load carrying capacity, if any, achieved by the SBDO.

2.2.2. Static Testing Instrumentation

Two sets of static tests were completed on the bridge. The first was conducted on the "as-built" bridge on August 5, 1997, prior to the placement of the SBDO. The second test was conducted on October 20, 1997, after the concrete in the SBDO achieved its 28-day design compressive strength. Static testing was conducted on the bridge using trucks of known weight which were positioned at various stations on the bridge. The testing provided an opportunity to determine the deflections and stresses induced by normal traffic loading. The results from the static testing were used to quantify neutral axis location and load distribution characteristics in the bridge, as will be discussed in later sections.

In order to minimize the time required to mount concrete strain gauges in the field, reusable strain gauges manufactured by Bridge Diagnostics, Inc., were used to complete the testing of the bridge. These gauges required little surface preparation and could be easily moved to different locations, thus preventing the need to use an inordinate amount of the typical foil gauges requiring extensive surface preparation and adhesive procedure. The reusable gauges have holes at either end, spaced 3-in (76.2-mm) apart, through which the threaded posts of the mounting tabs pass.

A mounting template fabricated for a previous experimental bridge study was employed to expedite the placement of the mounting tabs on the bridge girders. Once locations were identified and marked, the surface was prepared by cleaning away any loose materials with a wire brush and sandpaper. The tabs were inserted into the template and glued to the girders with an industrial strength adhesive. A catalyst was used to reduce the adhesive curing time. The process of mounting the strain gauges was then just a matter of placing the gauge on the tabs and tightening the nut on the threaded post. Figure 9 shows the reusable strain gauges in place on prestressed concrete I-girders. The use of reusable strain gauge also allowed the measurement of strain in the exact same girder locations before and after the placement of the concrete deck overlay since the mounting tabs could remain.

Reusable strain gauges were placed on the bridge deck directly above the transverse center of the girders and oriented along the longitudinal direction of the bridge. Since the I-64 Bridge over KY 32 is essentially a right bridge, only four transverse positions on the deck were considered. By symmetry, instrument positions directly above only four girders would provide enough information for all eight. The gauges were located at midspan in the longitudinal direction of the simply supported Center Span (Span 2) of the bridge carrying the westbound traffic. Three strain gauges were also placed vertically along the cross section of the girders directly below these deck gauges. The locations described are depicted in Figure 10. Placement of the gauges along the girder cross section was consistent with the locations outlined for the deck and the dimensions between each gauge were determined in the field.

In a similar manner, Linear Variable Differential Transformers (LVDTs) were placed at the midspan of the spans discussed above and as indicated in Figure 11 in order to measure the vertical deflections of the girders. Vertical displacements records also will provide a means to ascertain the difference in load distribution characteristics of the girders and the deck before and after the concrete overlay. The sample bridge is pictured in Figure 12.

2.2.3. Total Instrumentation

Table 1 summarizes the instruments (strain gauges and LVDTs) listed in the above sections to complete the experimental static testing of the bridge. The cumulative required number of instruments is also reported. Tables 2 and 3 list the vertical and horizontal location of the test instruments, the calibration factor associated with the instrument, and the channel number of the data acquisition system for each phase of the static testing plan.

2.3. EXPERIMENTAL TESTING

Static testing provided an opportunity to determine the deflections and stresses induced by normal truck traffic under a controlled situation. The results from each test can be used to correlate analytical findings reported above on the effectiveness of SBDO.

Static testing was accomplished by using two fully-loaded, tandem dump trucks (see Figure 13) to induce the displacements and strains on the bridges. The footprints of the respective truck tires are given in Figure 14 for the trucks used prior to the deck overlay and in Figure 15 for the trucks used after SBDO.

For the test setup on the KY 32 Bridge, the trucks were positioned with one line of tires along a beam line, and the rear axle next to the strain gauge locations. In test pass one, the passenger's side wheel line of Truck 1 was positioned over beam 1 and the passenger's side wheel line of Truck 2 was positioned over beam 3. In test pass two, the trucks were shifted to accomplish the same configuration over beams 2 and 4. Due to the relatively small skew, the trucks were side-by-side at each stage of testing. This orientation was chosen to maximize the output to the recording equipment without regard to traffic lanes on the interstate.

Static test data was sampled and recorded at four longitudinal stations within the span under investigation. Stations corresponded to the quarter points along the span which translated to 210-in (5334-mm) intervals along each beam. The tandem set of axles was centered over the test position except at midspan where the presence of the deck gauge required an alternate arrangement. At midspan the rear axle of the tandem set was located as close to the gauge as possible. Strain gauge and LVDT data were obtained for the trucks at each station along the instrumented beams in the bridge.

Using a procedure developed in previous field testing (Griffin 1997), all data acquisition channels were read using a sampling rate of 200 Hz while the trucks were positioned at these locations. Subsequent data readings were made by incrementing the truck positions to the next station. One static test pass was complete once each station had been sampled for trucks along beams 1 and 3. The second and final static test pass sampled trucks at each station along beams 2 and 4. This procedure was repeated after the SBDO was placed.

2.3.1. Data Acquisition

An IBM-compatible portable (laptop) computer with docking station was used to record the data from a Keithley-Metrobyte data acquisition system. Simultaneous sample and hold capability enabled all channels to be sampled and recorded concurrently instead of sequentially. Signal conditioners were not used since the LVDTs and the reusable strain gauges from Bridge Diagnostics, Inc., did not require signal conditioning. The data were obtained from the static tests using the software VIEWDAC©. The data were stored in binary format, requiring one byte of computer storage per point.

Typical static testing using foil gauges requires a procedure employing "dummy" gauges to compensate for any temperature variations throughout the testing process (a description of this procedure is given in Dunnicliff [1993]). However, the reusable strain gauges had self-contained temperature compensators and did not require the use of this "dummy" gauge procedure.

Sampling rates are critical to the quality of the instrument readings. Obviously, the higher the rate, the more often an instrument is sampled within a one second interval. However, this rate must be balanced with time and storage capacities. For example if a sampling rate of 200 Hz was chosen, a test which lasted for 20 seconds would require the computer to store approximately one megabyte of information per station (200 points per second x 20 seconds x 4 bytes per point x 64 channels = 1,024,000 bytes). The storage capacities required would far outweigh the benefit of recording a data point every 0.05 second to minimize secondary influences. For the static tests on the study bridges, the sampling duration was relatively short using a 200 Hz sampling rate to keep the information recorded per station within a reasonable file size.

The packaged software DaDisp was used to process the static test data binary files and report the average values for data recorded on each channel number. Only the average value is necessary since change in strain with time was not measured (i.e., only static testing was conducted) and sufficient time was given for dynamic effects to dissipate before reading the gauges. "Zero" readings were taken before loading the bridges to establish a baseline measurement of the strain gauges and LVDTs. This reading was subtracted from the experimental reading during the data analysis stage to determine the appropriate strain or displacement sampled.

2.3.2. Calibration Factors

Data obtained from the static testing was merely a reflection of a change in voltage read by the data acquisition board. In the case of the strain gauges, the change in voltage output was due to a fluctuation in electrical resistance caused by the strain on a particular gauge. Voltage output on the LVDTs changed as the deflecting core altered the electric field within the instrument. Assessment of the strains and deflections associated with the static tests for each bridge required calibration factors to convert these voltage changes to quantities of microstrain (1×10^{-6} ε or $\mu\varepsilon$) or inches (millimeters).

Based on data reported by the manufacturer, every one volt change in the LVDTs corresponded to an approximate deflection of 0.049-in (1.25-mm). Even though the reusable strain gauges appeared to be of the same construction, each has unique gauge factors leading to different calibration factors. These calibration factors (for microstrain per volt) had been previously determined in the laboratory for another research study (Griffin 1997) and were calculated from the following equation:

$$CF = \frac{(GF)\times 1000}{(excitation\,voltage)\times(voltage\,gain)} \tag{3}$$

Tables 2 through 3 list the calibration factors for each reusable strain gauge based on a voltage gain of 100 volts.

2.4. EXPERIMENTAL RESULTS

Throughout the discussion of the static test results, any mention of a station is based upon the static test "station" specification described in Section 2.3 above. The stations on the I-64 Bridge over KY 32 were not staggered due to the small skew of the bridge. Much of the experimental data offers insight to the behavior of the bridge when subjected to tandem axle truck loads without the need for extensive analytical studies. However, this experimental study is meant to corroborate the analytical findings presented in Section 2.1. A summary of the experimental data obtained during the static testing phase is presented below.

2.4.1. Instrumentation on the Deck

After applying the appropriate calibration factors and subtracting out the zero reading, strain values for the reusable strain gauges on the deck were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 2 and 3.

Illustrations of the strain gauge readings oriented along the longitudinal axis of the bridge deck are presented in Figures 16 through 19 for Test Pass #1. Unfortunately, data for Test Pass #2 where the trucks were oriented along beams 2 and 4 were suspect and have been omitted from this section.

As expected in a simply supported beam, the highest readings were obtained with the trucks positioned at the centerline of the span. Marked decreases in concrete strain on the slab were noted after placement of the Structural Bridge Deck Overlay (SBDO). A peak response of 31.30μ strain was recorded above Beam 3 prior to the overlay. The response at the same location was reduced to 10.96μ strain after the rehabilitation work had been completed, representing a site-specific reduction of 65 percent in compressive stress in the slab. The deck did not experience an increase in compressive concrete strain after the SBDO at any of the four test locations.

Of interest to this study is the influence the SBDO appears to have had on load distribution among the girders. Prior to the SBDO, deck gauges above Beams 1 and 3 experienced strains of greater magnitude than the other two. This was expected since the static test positioned the trucks directly over these two girder locations. However, once the SBDO had been placed, dramatic differences in strain magnitudes between all four test beams were not noted. In fact the thicker concrete slab appears to distribute more of the loading into the interior, more flexible portion of the bridge (e.g., slab strain above girders G3 and G4 reported in Figures 16 and 17). Although Figures 16 and 18 illustrated strain values at quarter points on the bridge span, the test vehicles were pointed the same direction for both readings. Therefore, the weight of the front axle had more of an impact for the results at three-quarter span (Figure 18).

Naturally this trend is not observed in Figure 19. Since the moment at the end of a simply supported span is zero, relatively little response can be expected at the midspan gauges when the trucks were positioned over the pier. The order of magnitude difference is deceiving considering the small strain values recorded (less than 10μ strain). A further discussion of the longitudinal strains on the bridge deck for both test conditions is given below when dealing with the strains obtained from the girder cross section.

2.4.2. Instrumentation on the Girders – Displacements

After applying the appropriate calibration factors and subtracting out the zero reading, displacement values for the vertical LVDTs on the girders were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 2 and 3.

In general, the bridge tended to deflect less after the SBDO placement under the same static test loads. Figures 20 and 21 illustrate the deflection at each girder with the trucks positioned at midspan and at the pier support, respectively. Displacements paralleled the trend observed in Figure 20 - maximum displacement at Beam 2. This is in direct contrast to the results observed for the slab strains as reported above. A maximum vertical displacement of 0.12-in (3.13-mm) was recorded for the condition prior to the deck overlay, and a maximum vertical displacement of 0.08-in (1.95-mm) was recorded for the bridge after SBDO. Figure 21 again demonstrates the dependence of the truck (and load) position (or lack thereof) on achieving significant response in a simply supported bridge span.

If all of the bridge components remained constant, smaller deflections would be a reflection of smaller moments on the bridge girder, as the two are related in the following manner:

$$\Delta \ \alpha \ \frac{M}{I} \tag{4}$$

where Δ is the deflection, M is the applied moment, and I is the moment of inertia of the cross-section. However, since the applied moment can be said to be equal for the before and after testing conditions (i.e., same truck loads, same testing positions), smaller recorded displacements after the SBDO must be indicative of higher load capacities. This is reasonable since the SBDO effectively increases the cross-sectional properties of the beam and slab bridge, namely the moment of inertia. With the 33 percent reduction in the maximum deflection noted above, a moment capacity increase for this particular study is apparent. Furthermore, the results from Figure 8 seem to be conservative compared to the experimental findings based on vertical deflection.

The influence proposed on load distribution due to the deck strain analysis above is not substantiated by the vertical displacements as the same deflection pattern is generally noted between the two test conditions. According to the LVDT data, the SBDO did not appear to shift more of the load to adjacent girders.

2.4.3. Instrumentation on the Girders - Strains

After applying the appropriate calibration factors and subtracting out the zero reading, strain values for the reusable strain gauges on the girders were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 2 and 3.

The strains obtained from the gauges on the deck were combined with the strains recorded along the cross section of the girder to determine the distribution of stress across the composite cross section and the shift in the neutral axis caused by the SBDO. Figures 22 through 25 are representative of the strain distribution witnessed in the experimental testing of the I-64 Bridge over KY 32. Each figure plots the actual test readings for before and after the SBDO as well as a linear fit of each data set. Two conclusions can be drawn from the figures.

First, Figures 22 and 23 show the reaction of Girder 4 to the test vehicles. This beam is at the interior of the bridge span. Both figures illustrate that the neutral axis on the beam shifts upward toward the deck after the SBDO has been placed. Using the working stress method, the effect on moment *capacity* can be identified. The reinforcement ratio is calculated from ACI (1999) as:

$$\rho = \frac{A_s}{bd} \qquad (5)$$

If the ratio of Young's moduli for steel and concrete is taken η, the expression for *k* can be written as:

$$\kappa = \sqrt{(\rho\eta)^2 + 2\rho\eta} - \rho\eta \qquad (6)$$

and

$$j = \left(1 - \frac{\kappa}{3}\right) \qquad (7)$$

and the expression for moment is taken as:

$$M = f_s A_s j d \qquad (8)$$

Since the steel strength at ultimate is the same for both the before and after condition, an increase in moment *capacity* is noted after the SBDO by the following. As both Figures 22 and 23 show the neutral axis shifts toward the deck, which translates to an increase in the distance *jd* (the moment arm between the tensile and compressive force in the cross-section).

This can also be shown empirically by tracking through the equations above. As the distance from extreme compression fiber to centroid of steel, *d*, increases, the reinforcement ratio decreases. Consequently, the value of *k* decreases and j approaches one. Therefore, with $j_b d_b < j_a d_a$, where the subscripts *b* and *a* refer to "before" and "after" the SBDO, respectively, it is readily apparent $M_b < M_a$.

The second conclusion focuses on the response of Girder 2 to the test vehicles, or the *applied* moment. During the test the trucks were positioned along Beams 1 and 3, thereby straddling Beam 2. As Figures 24 and 25 depict, the neutral axis actually shifts downward toward the bottom flange of the girder. At first glance, this would seem to indicate a reduction in moment *capacity*. However, the deck after the SBDO now serves as a stiffer transverse element. The effect from the experimental data illustrates a better distribution of wheel loads among the adjacent girders in the bridge. Strains in both instances were reduced after the rehabilitation work had been completed. Since the same loads were applied at the same locations, this suggests less moment experienced in the test girder. Improved load distribution characteristics are a natural conclusion from this observation.

3.0 I-64 BRIDGE OVER TRIPLETT CREEK: CONTINUOUS HAUNCHED REINFORCED CONCRETE GIRDERS

3.1. ANALYTICAL RESULTS

3.1.1. Introduction

Using the stress-strain relationship of concrete and steel in combination with the basic principles of mechanics, an evaluation of the bending moment capacity at the critical sections along the identified bridge spans can be made. For the simply-supported span of the I-64 Bridge over Triplett Creek, the critical section is located at the midspan. For the continuous haunched girder spans of the I-64 bridge over Triplett Creek, there are two critical sections: midspan (for positive moment) and over the pier support (for negative moment). The chapter presents the results of the cast-in-place concrete continuous spans of the I-64 Bridge over Triplett Creek. The results of the cast-in-place concrete simply-supported span of the I-64 Bridge over Triplett Creek will be presented in Chapter 4.

The analytical evaluation to be made is based on the following set of assumptions:

1. Strain distribution remains linear in the elastic range. This assumption is based on the theory plane sections prior to bending remain plane and perpendicular to the neutral axis after bending, and

2. Strain in the reinforcing steel bars and surrounding concrete are equal prior to the concrete cracking.

3.1.2. Moment-Curvature Relationship

Since the haunched girder spans are continuous, relationships for both the negative and positive bending of the girder and deck cross-section can be developed. For the positive moment analysis, two different locations of the neutral axis must be considered in the calculations: a) in the flange of the equivalent T-section, and b) in the web of the equivalent T-section. The depth to the resultant compressive force in the cross-section is calculated by equating the value obtained from integrating the parabolic compressive stress block to the value taken from the moment equilibrium equation about the neutral axis for each location. If k and k_2 are defined as coefficients relating the depth to the centroid of the tension steel in the girder (*d-distance*) to the location of the neutral axis and the location of the resultant compressive force, respectively, then expressions for the moment (M) and curvature (ϕ) of the cross-section can be written as follows:

$$M = Td\left(1 - \frac{k_2\,kd}{d}\right) \tag{9}$$

$$\phi = \left(\frac{\varepsilon_c}{kd} \right) \tag{10}$$

where ε_c is the compressive strain in the outermost fiber of the concrete. Having derived the basic equations for analysis, the moment-curvature diagram for the critical section (i.e., midspan) is obtained by considering four strain levels:

1. Maximum concrete strain in tension (cracking point): $\varepsilon_c = \varepsilon_{cr}$,
2. Maximum steel strain in tension (yield point): $\varepsilon_s = \varepsilon_y$,
3. Intermediate concrete strain in compression: $\varepsilon_c = 0.0015$, and
4. Maximum concrete strain in compression (ultimate point): $\varepsilon_c = \varepsilon_{cu} = 0.0030$.

Figures 26 and 27 depict the moment-curvature diagrams for the I-64 Bridge over Triplett Creek both before and after SBDO. Figure 26 illustrates the positive moment at midspan of the continuous Span 3, and Figure 27 illustrates the negative moment over the pier support between Spans 2 and 3. As can be seen from the graph, the positive moment capacity of the cross-section has increased approximately 17 after the SBDO for the continuous span. The near 80 percent increase in the negative bending moment capacity over the pier support is indicative of the benefit the additional steel reinforcing in the SBDO provides.

3.2. INSTRUMENTATION

3.2.1. Introduction

Once this bridge had been identified as the experimental subject of study, an instrumentation plan was prepared to provide guidelines for comprehensive static testing. Planning for the instrumentation began before the rehabilitation of the bridge. In cooperation with Kentucky Transportation Cabinet personnel, instrumentation was proposed which would take advantage of the opportunity to study both the "before" and "after" load carrying capacity of the bridge. The experimental study offers the ability to substantiate the analytical findings outlined above. The information in this section is a summary of the instrumentation plan and record of how this plan was implemented in the field. Due to the unique nature of the bridge, the experimental testing allowed for the influence of SBDO to be studied for (a) continuous, cast-in-place reinforced concrete girders, and (b) simply-supported, cast-in-place reinforced concrete girders (Chapter 4). These two types of construction are typical of a large portion of the older, shorter span bridges found in Kentucky.

The strain diagram for the composite cross section can be determined from the information obtained by the three girder strain gauges and the strain gauges mounted on the deck. Strain data across a girder cross section is essential for determining the neutral axis of the composite cross section under various loadings and how the neutral axis varies as the load traverses along the longitudinal axis of the bridge. Strain comparison between the bridges before and after the Structural Bridge Deck Overlay is used to first investigate if the technique produces a new section which acts compositely. Further analysis will then shed light on how forces (stresses) are transferred among the girders and ascertain the contribution to load carrying capacity, if any, achieved by the SBDO.

3.2.2. Static Testing Instrumentation

Two sets of static tests were completed on the bridge. The first was conducted on the "as-built" bridge on August 5 and 6, 1997, prior to the placement of the SBDO. The second test was conducted on October 17 and 20, 1997, after the concrete in the SBDO achieved its 28-day design compressive strength. Static testing was conducted on the bridge using trucks of known weight which were positioned at various stations on the bridge. The testing provided an opportunity to determine the deflections and stresses induced by normal traffic loading. The results from the static testing were used to quantify neutral axis location and load distribution characteristics in the bridge, as will be discussed in later sections.

In order to minimize the time required to mount concrete strain gauges in the field, reusable strain gauges manufactured by Bridge Diagnostics, Inc., were used to complete the testing of the bridge. These gauges required little surface preparation and could be easily moved to different locations, thus preventing the need to use an inordinate amount of the typical foil gauges requiring extensive surface preparation and adhesive procedure. The reusable gauges have holes at either end, spaced 3-in (76.2-mm) apart, through which the threaded posts of the mounting tabs pass.

A mounting template fabricated for a previous experimental bridge testing study was employed to expedite the placement of the mounting tabs on the bridge girders. Once locations were identified and marked, the surface was prepared by cleaning away any loose materials with a wire brush and sandpaper. The tabs were inserted into the template and glued to the girders with an industrial strength adhesive. A catalyst was used to reduce the adhesive curing time. The process of mounting the strain gauges was then just a matter of placing the gauge on the tabs and tightening the nut on the threaded post. Figure 9 shows the reusable strain gauges in place on prestressed concrete I-girders. The use of reusable strain gauge also allowed the measurement of strain in the exact same girder locations before and after the placement of the concrete deck overlay since the mounting tabs could remain.

Reusable strain gauges were placed on the bridge deck directly above the transverse center of the girders and oriented along the longitudinal direction of the bridge. Since the I-64 bridge over Triplett Creek has a significant skew, four transverse positions on the deck were instrumented which correspond to positions directly above all four girders. The orientations of the gauges for the continuous spans in the longitudinal direction were as follows:

(1) slightly East of the midspan of Span 2,
(2) at the Eastern haunch of Span 3, and
(3) at the midspan of Span 3.

Three strain gauges were placed vertically along the cross section of the girders directly below these deck gauges. The locations described are depicted in Figure 29. Placement of the gauges along the girder cross section was consistent with the location outlined above and the dimensions between each gauge were determined in the field.

Likewise, LVDTs were placed at the midspan of the spans discussed above and as indicated in Figure 30 in order to measure the vertical deflections of the girders. The sample bridge is pictured in Figure 31. As with the I-64 Bridge over KY 32, data records from both instruments were used to evaluate the benefit of SBDO.

3.2.3. TOTAL INSTRUMENTATION

Table 1 summarizes the instruments (strain gauges and LVDTs) listed in the above sections to complete the experimental static testing of the bridges. The cumulative required number of instruments is also reported. Tables 4 through 6 list the vertical and horizontal location of the test instruments, the calibration factor associated with the instrument, and the channel number of the data acquisition system for the continuous span and phase of the static testing plan.

3.3. EXPERIMENTAL TESTING

Static testing provided an opportunity to determine the deflections and stresses induced by normal truck traffic under a controlled situation. The results from each test can be used to correlate analytical findings reported above on the effectiveness of SBDO.

Static testing was accomplished by using two fully-loaded, tandem dump trucks (see Figure 13) to induce the displacements and strains on the bridges. The footprints of the respective truck tires are given in Figure 14 for the trucks used prior to the deck overlay and in Figure 15 for the trucks used after SBDO.

For the test setup on the Triplett Creek Bridge, Truck 1 was positioned with one line of tires along a beam line while Truck 2 was positioned as close to the other as possible, and the rear axle of each truck next to the strain gauge locations. In test pass one, the passenger's side wheel line of Truck 1 was positioned over the outermost beam. In test pass two, the passenger's side wheel line of Truck 2 was positioned over the opposite outermost beam. Due to the skew of the girders, the trucks were also positioned with rear axle end-to-end to perform an opposing centerline test. Side-by-side tests were likewise staggered parallel to the skew of the bridge piers. As with the I-64 Bridge over KY32, this orientation was chosen to maximize the output to the recording equipment without regard to traffic lanes on the interstate.

Static test data was sampled and recorded at four longitudinal stations within the span under investigation. Stations corresponded to the quarter points along Span 3 which translated to 300-in (7620-mm) intervals. Span 2 was sampled at 210-in (5334-mm) intervals. Opposing centerline tests were conducted at the midspan of each test span.

The tandem set of axles was centered over the test position except where gauge locations precluded such an arrangement. In these incidences the rear axle of the tandem set was located as close to the gauge as possible. Strain gauge and LVDT data were obtained for the trucks at each station along the instrumented beams in the bridge for each test setup. Due to the nature of the bridge composition, four test setups were necessary to obtain data for each desired girder location. Data readings were made by incrementing the truck positions to the next station along the outermost beams as described above. Once a full set of data had been saved, a new gauge setup was completed, and the process repeated.

3.3.1. Data Acquisition

An IBM-compatible portable (laptop) computer with docking station was used to record the data from a Keithley-Metrobyte data acquisition system. Simultaneous sample and hold capability enabled all channels to be sampled and recorded concurrently instead of sequentially. Signal conditioners were not used since the LVDTs and the reusable strain gauges from Bridge Diagnostics, Inc., did not require signal conditioning. The data were obtained from the static tests using the software VIEWDAC©. The data were stored in binary format, requiring one byte of computer storage per point.

Typical static testing using foil gauges requires a procedure employing "dummy" gauges to compensate for any temperature variations throughout the testing process [a description of this procedure is given in Dunnicliff (1993)]. However, the reusable strain gauges had self-contained temperature compensators and did not require the use of this "dummy" gauge procedure.

Sampling rates are critical to the quality of the instrument readings. Obviously, the higher the rate, the more often an instrument is sampled within a one second interval. However, this rate must be balanced with time and storage capacities. For example if a sampling rate of 200 Hz was chosen, a test which lasted for 20 seconds would require the computer to store approximately one megabyte of information per station (200 points per second x 20 seconds x 4 bytes per point x 64 channels = 1,024,000 bytes). The storage capacities required would far outweigh the benefit of recording a data point every 0.05 second to minimize secondary influences. For the static tests on the study bridges, the sampling duration was relatively short using a 200 Hz sampling rate to keep the information recorded per station within a reasonable file size.

The packaged software DaDisp was used to process the static test data binary files and report the average values for data recorded on each channel number. Only the average value is necessary since change in strain with time was not measured (only static testing conducted) and sufficient time for dynamic effects to dissipate was given before reading the gauges. "Zero" readings were taken before loading the bridges to establish a baseline measurement of the strain gauges and LVDTs. This reading was subtracted from the experimental reading during the data analysis stage to determine the appropriate strain or displacement sampled.

3.3.2. Calibration Factors

Data obtained from the static testing was merely a reflection of a change in voltage read by the data acquisition board. In the case of the strain gauges, the change in voltage output was due to a fluctuation in electrical resistance caused by the strain on a particular gauge. Voltage output on the LVDTs changed as the deflecting core altered the electric field within the instrument. Assessment of the strains and deflections associated with the static tests for each bridge required calibration factors to convert these voltage changes to quantities of microstrain (1 x 10^{-6} ε or $\mu\varepsilon$) or inches (millimeters).

Based on data reported by the manufacturer, every one volt change in the LVDTs corresponded to an approximate deflection of 0.049-in (1.25-mm). Even though the reusable strain gauges appeared to be of the same construction, each has unique gauge factors leading to different calibration factors. These calibration factors (for microstrain per volt) had been previously determined in the laboratory for another research study (Griffin [1997]) and were calculated from the following equation:

$$CF = \frac{(GF)\times 1000}{(excitation\,voltage)\times(voltage\,gain)} \qquad (11)$$

Tables 4 through 6 (continuous span) list the calibration factors for each reusable strain gauge based on a voltage gain of 100 volts.

3.4. EXPERIMENTAL RESULTS

Throughout the discussion of the static test results, any mention of a station is based upon the static test "station" specification described in Section 3.3 above. The stations on the I-64 Bridge over Triplett Creek were staggered to account for the skew of the bridge piers relative to the girders. Much of the experimental data offers insight to the behavior of the bridges when subjected to tandem axle truck loads without the need for extensive analytical studies. However, this study is meant to be a companion to analytical studies already performed and to corroborate the analytical findings presented in Section 3.1. A summary of the experimental data obtained during the static testing phase is presented below.

3.4.1. Instrumentation on the Deck

After applying the appropriate calibration factors and subtracting out the zero reading, strain values for the reusable strain gauges on the deck were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 4 through 6 (continuous span). The continuous span, haunched girder construction of the bridge, depicted in Figure 31, does not allow for deflection data to be taken at the pier location. Hence the LVDT rows are blank. Since the bridge has a continuous span, though, strain data is available for analysis of the effect of SBDO on negative moment capacity.

Representative illustrations of the strain gauge readings oriented along the longitudinal axis of the bridge deck are not presented in this section. No conclusions can be drawn from the data, and it is not consistent with the results expected from the experimental study. For example, strain readings over the pier should demonstrate tension due to the negative moment experienced in the slab. However, as Figure 32 shows, not only does the before and after data not agree, but the after test charts compression strain readings. The discrepancy is significant to the point that it does not invalidate the hypothesis of the SBDO study, but rather indicates an error in data collection.

3.4.2. Instrumentation on the Girders – Displacements

After applying the appropriate calibration factors and subtracting out the zero reading, displacement values for the vertical LVDTs on the girders were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 4 through 6 (continuous span).

The displacement data yielded far better results than the strain readings on the bridge deck. In general, the bridge tended to deflect less after the SBDO placement under the same static test loads. Figures 33 through 36 show plots the displacement values of each girder for the continuous span condition, and illustrate the same effect. Whether the girder deflection was from positive moment (trucks in the same span) or negative moment (trucks in adjacent span), the SBDO served to reduce the net effect. A maximum vertical deflection for positive moment on Girder 3 of 0.19-in (4.78-mm) was recorded for the condition prior to the deck overlay, and a maximum vertical deflection of 0.15-in (3.89-mm) was recorded for the bridge after SBDO. This represents a reduction of 19% in the deflection of Girder 3 under similar load for the continuous span. For negative moment, Girder 2 had maximum displacement values of 0.07-in (1.74-mm) and 0.05-in (1.18-mm) for the before and after condition, respectively. This represents a decrease of 32% in upward deflection of Girder 2 under similar load for the continuous span.

All things being equal, smaller deflections would be a reflection of smaller moments on the bridge girder, as the two are related in the following manner:

$$\Delta \ \alpha \ \frac{M}{I} \tag{12}$$

where Δ is the deflection, M is the applied moment, and I is the moment of inertia of the cross-section. However, since the applied moment can be said to be equal for the before and after testing conditions (i.e., same truck loads, same testing positions), smaller recorded displacements after the SBDO must be indicative of higher load capacities. This is reasonable since the SBDO effectively increases the cross-section properties of the beam and slab bridge, namely the moment of inertia. The experimental data, therefore, suggest a 23% increase in the positive moment load carrying capacity of the bridge with a corresponding 47% increase in the negative moment load carrying capacity.

The graphs also depict a contribution from the SBDO to load distribution as the girders directly below the load location attracted more of the applied force after the stiffened deck was in place. For example, the displacement changes from maximum before the overlay to less than the value for Girder 3 after the SBDO. Similarly, more of the load seems to be distributed to the exterior girder (i.e., girder opposite the load) as witnessed in the difference for displacements in Girder 1.

3.4.3. Instrumentation on the Girders – Strains

After applying the appropriate calibration factors and subtracting out the zero reading, strain values for the reusable strain gauges on the girders were obtained and tabulated for each test scenario. Any location codes referenced in the following paragraphs, figures, and/or tables correspond to those previously defined above and in Tables 4 through 6 (continuous span).

Since the deck strains are an integral portion in the analysis of the stress across the composite cross section, using the girder strains alone to draw conclusions on the effectiveness of the SBDO would not be recommended. The discrepancy in strain distribution is consistent with the phenomena recorded in the slab data. For example, Figure 37 illustrates a strain record where compression is indicated in between tension readings. The discrepancy is significant to the point that it does not invalidate the hypothesis of the SBDO study, but rather indicates an error in data collection.

4.0 I-64 BRIDGE OVER TRIPLETT CREEK: SIMPLY-SUPPORTED REINFORCED CONCRETE GIRDERS

4.1. ANALYTICAL RESULTS

4.1.1. Introduction

The assumptions presented in 3.1.1 can be used to analyze the mid-span (critical section) of the simply-supported span of the I-64 Bridge over Triplett Creek. The results of the cast-in-place concrete simply-supported span of the I-64 Bridge over Triplett Creek are presented herein.

4.1.2. Moment-Curvature Relationship

For simply-supported span, relationships for positive bending of the girder and deck cross-section are of interest. For the positive moment analysis, two different locations of the neutral axis are considered in the calculations: (a) in the flange of the equivalent T-section, and (b) in the web of the equivalent T-section. The depth to the resultant compressive force in the cross-section is calculated by equating the value obtained from integrating the parabolic compressive stress block to the value taken from the moment equilibrium equation about the neutral axis for each location. If k and k_2 are defined as coefficients relating the depth to the centroid of the tension steel in the girder (*d-distance*) to the location of the neutral axis and the location of the resultant compressive force, respectively, then expressions for the moment (M) and curvature (ϕ) of the cross-section can be written as follows:

$$M = Td\left(1 - \frac{k_2\, kd}{d}\right) \tag{9}$$

$$\phi = \left(\frac{\varepsilon_c}{kd}\right) \tag{10}$$

where ε_c is the compressive strain in the outermost fiber of the concrete. Having derived the basic equations for analysis, the moment-curvature diagram for the critical section (i.e., midspan) is obtained by considering four strain levels:

1. Maximum concrete strain in tension (cracking point): $\varepsilon_c = \varepsilon_{cr}$,
2. Maximum steel strain in tension (yield point): $\varepsilon_s = \varepsilon_y$,
3. Intermediate concrete strain in compression: $\varepsilon_c = 0.0015$, and
4. Maximum concrete strain in compression (ultimate point): $\varepsilon_c = \varepsilon_{cu} = 0.0030$.

Figure 28 illustrates the analytical moment-curvature diagrams of the simply-supported Span 1 for the I-64 Bridge over Triplett Creek both before and after SBDO. As can be seen from the graph, the moment capacity of the cross-section has increased approximately 23 percent after the SBDO for the simply supported span.

4.2. INSTRUMENTATION

4.2.1. Introduction

Similar to the continuous haunched girder spans of the I-64 Bridge over Triplett Creek, an instrumentation plan was devised for the simply-supported girder span of the I-64 Bridge over Triplett Creek to provide guidelines for comprehensive static testing.

As indicated in Section 3.2.1, the strain diagram for the composite cross section can be determined from the information obtained by the three girder strain gauges and the strain gauges mounted on the deck. Strain data across a girder cross section is essential for determining the neutral axis of the composite cross section under various loadings and how the neutral axis varies as the load traverses along the longitudinal axis of the bridge. Strain comparison between the bridges before and after the SBDO is used to first investigate if the technique produces a new section which acts compositely. Further analysis will then shed light on how forces (stresses) are transferred among the girders and ascertain the contribution to load carrying capacity, if any, achieved by the SBDO.

4.2.2. Static Testing Instrumentation

Two sets of static tests were completed on the bridge. The first was conducted on the "as-built" bridge on August 5 and 6, 1997, prior to the placement of the SBDO. The second test was conducted on October 17 and 20, 1997, after the concrete in the SBDO achieved its 28-day design compressive strength. Static testing was conducted on the bridge using trucks of known weight which were positioned at various stations on the bridge. The testing provided an opportunity to determine the deflections and stresses induced by normal traffic loading. The results from the static testing were used to quantify neutral axis location and load distribution characteristics in the bridge, as will be discussed in later sections.

In order to minimize the time required to mount concrete strain gauges in the field, reusable strain gauges manufactured by Bridge Diagnostics, Inc., were used to complete the testing of the bridge. These gauges required little surface preparation and could be easily moved to different locations, thus preventing the need to use an inordinate amount of the typical foil gauges requiring extensive surface preparation and adhesive procedure. The reusable gauges have holes at either end, spaced 3-in (76.2-mm) apart, through which the threaded posts of the mounting tabs pass.

A mounting template fabricated for a previous experimental bridge testing study was employed to expedite the placement of the mounting tabs on the bridge girders. Once locations were identified and marked, the surface was prepared by cleaning away any loose materials with a wire brush and sandpaper. The tabs were inserted into the template and glued to the girders with an industrial strength adhesive. A catalyst was used to reduce the adhesive curing time. The process of mounting the strain gauges was then just a matter of placing the gauge on the tabs and tightening the nut on the threaded post. Figure 9 shows the reusable strain gauges in place on prestressed concrete I-girders.

The use of reusable strain gauge also allowed the measurement of strain in the exact same girder locations before and after the placement of the concrete deck overlay since the mounting tabs could remain.

Reusable strain gauges were placed on the bridge deck directly above the transverse center of the girders and oriented along the longitudinal direction of the bridge. Since the I-64 bridge over Triplett Creek has a significant skew, four transverse positions on the deck were instrumented which correspond to positions directly above all four girders. The orientation of the gauges in the longitudinal direction at the midspan of Span 1 (simply-supported span) is presented in Figure 29. Placement of the gauges along the girder cross section was consistent with the location outlined above and the dimensions between each gauge were determined in the field.

Likewise, LVDTs were placed at the midspan of the spans discussed above and as indicated in Figure 30 in order to measure the vertical deflections of the girders. The sample bridge is pictured in Figure 31. As with the I-64 Bridge over KY 32, data records from both instruments were used to evaluate the benefit of SBDO.

4.2.3. TOTAL INSTRUMENTATION

Table 1 summarizes the instruments (strain gauges and LVDTs) listed in the above sections to complete the experimental static testing of the bridges. The cumulative required number of instruments is also reported. Table 7 lists the vertical and horizontal location of the test instruments, the calibration factor associated with the instrument, and the channel number of the data acquisition system for the simply-supported span and phase of the static testing plan.

4.3. EXPERIMENTAL TESTING

Static testing provided an opportunity to determine the deflections and stresses induced by normal truck traffic under a controlled situation. The results from each test can be used to correlate analytical findings reported above on the effectiveness of SBDO.

Static testing for the simply-supported span was accomplished using similar procedures as for the continuous spans. A complete description of the static testing procedure can be found in Section 3.3.

Static test data was sampled and recorded at four longitudinal stations within the span under investigation. For Span 1 (simply-supported span), stations were situated at 150-in (3810-mm) intervals. Opposing centerline tests were conducted at the midspan of each test span.

4.3.1. Data Acquisition

Data acquisition and reduction were based on the same system described for the continuous girder span, presented in 3.3.1.

4.3.2. Calibration Factors

The calibration factors (for microstrain per volt) had been previously determined in the laboratory for another research study [Griffin (1997)] and were calculated from the following equation:

$$CF = \frac{(GF)\times 1000}{(excitation\,voltage)\times(voltage\,gain)} \tag{11}$$

Table 7 lists the calibration factors for each reusable strain gauge based on a voltage gain of 100 volts.

4.4. EXPERIMENTAL RESULTS

4.4.1. Instrumentation on the Girders – Displacements

In general, the bridge tended to deflect less after the SBDO placement under the same static test loads. Figure 38 illustrates the deflection at each girder with the trucks positioned at the midspan. A maximum vertical displacement of 0.05-in (1.36-mm) was recorded for the condition prior to the deck overlay, and a maximum vertical displacement of 0.04-in (1.10-mm) was recorded for the bridge after SBDO. This represents a reduction of 19% in the deflection of Girder 3 under similar load for the simply supported span.

All things being equal, smaller deflections would be a reflection of smaller moments on the bridge girder, as the two are related in the following manner:

$$\Delta \;\alpha\; \frac{M}{I} \tag{12}$$

where Δ is the deflection, M is the applied moment, E is the elastic modulus of the beam and I is the moment of inertia of the cross-section. However, since the applied moment can be said to be equal for the before and after testing conditions (i.e., same truck loads, same testing positions), smaller recorded displacements after the SBDO must be indicative of higher load capacities. This is reasonable since the SBDO effectively increases the cross-section properties of the beam and slab bridge, namely the moment of inertia. The experimental data, therefore, suggest a 23% increase in the positive moment load carrying capacity of the simply-supported span.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1. GENERAL SUMMARY

As concrete bridges reach the latter stages of their service life, consideration must be given to repair, rehabilitation, or replacement of the structure. When replacement is neither feasible nor cost effective, innovative rehabilitation techniques need to be considered. One such procedure makes use of a concrete deck overlay (ACI 1999).

Structural Bridge Deck Overlay relies on the application of six to ten inches (157.4 to 254.0 mm) of normal weight concrete directly onto the existing bridge deck surface. Since the overlay is reinforced, the steel area in the compression zone across the girder and deck cross-section is increased. The net result is a bridge with improved cross-sectional properties that outweighs the additional load applied. This report has presented analytical findings and experimental results which seek to substantiate this statement through the investigation of two study bridges.

5.2. RESEARCH OBJECTIVES

Analytical procedures indicated a 17 to 23 percent increase in the load carrying capacity of bridges rehabilitated with SBDO. This study seeks to verify these analytical findings by reporting on the experimental testing of bridges before and after SBDO. As an added benefit, the bridges selected for this research are of mixed construction. Therefore, the effect on load carrying capacity of the SBDO technique will be illustrated for bridges with: (1) simply-supported precast prestressed concrete girders, (2) continuous cast-in-place reinforced concrete haunched girders, and (3) simply-supported cast-in-place reinforced concrete girders. The majority of the bridge inventories in Kentucky are of these construction types.

5.3. RESEARCH SIGNIFICANCE

As the bridge inventory in the United States matures, significant consideration must be given to the cost-effectiveness of repair *versus* replacement schedules. With the increased dependence on interstate travel, alternatives which focus on bridge replacement often are not feasible due to the inconvenience experienced by motorists and the long construction schedules which can span seasons with adversarial weather. On the other hand, repair and/or rehabilitation techniques which offer extended service life while maintaining the same level of safety can be extremely attractive given that they often are completed in much less time with fewer materials.

Beyond simply the repair and rehabilitation issue, bridges in Kentucky must resist higher loads than prescribed by the AASHTO design trucks. As seen previously, AASHTO H-type trucks only weigh on the order of 42,000 to 72,000-lbs. Trucks in Kentucky are allowed to transport loads up to 80,000-lbs without special permits. Bridges in the commonwealth must be capable of handling these increased demands.

If the improved load carrying capacity assumption can be substantiated, the SBDO technique is attractive due to the relative ease by which existing grades can be utilized. Structural Bridge Deck Overlay requires little excavation at the road to bridge interface thereby allowing traditional construction methods to proceed with minimal preparation.

5.4. ANALYTICAL RESULTS

Using the stress-strain relationship of concrete and steel in combination with the basic principles of mechanics, an evaluation of the bending moment capacity at the critical sections - positive or negative moment regions - of the study bridges was made. This analytical evaluation was based on the assumption that strain distribution remains linear in the elastic range and the strain in the reinforcing steel bars and surrounding concrete were equal prior to the concrete cracking.

The numerical analysis required different considerations for the position of the neutral axis in the girder and slab cross-section, both before and after the SBDO. Expressions for the bending moment capacity and curvature of the cross-section were derived by equating the value obtained from integrating the parabolic compressive stress block to the value taken from the moment equilibrium equation about the neutral axis for each location. Having derived the basic equations for analysis, the moment-curvature diagram for the critical section was obtained by considering four strain levels:

1. maximum concrete strain in tension (cracking point): $\varepsilon_c=\varepsilon_{cr}$,
2. maximum steel strain in tension (yield point): $\varepsilon_s=\varepsilon_y$,
3. intermediate concrete strain in compression: $\varepsilon_c=0.0015$, and
4. maximum concrete strain in compression (ultimate point): $\varepsilon_c=\varepsilon_{cu}=0.0030$.

The *positive* moment capacity of the girder and slab cross-section generally increased 23 percent after the SBDO for a simply-supported span, whether it was the prestressed concrete I-girder or the cast-in-place girder. The continuous, cast-in-place girder case generally witnessed a slightly lower increase of 17 percent for the *positive* bending moment capacity. A much higher response to the SBDO was calculated for the negative bending moment capacity - nearly an 81 percent increase - and is indicative of the benefit the additional reinforcing steel bars provide in the deck overlay.

5.5. EXPERIMENTAL TESTING

An instrumentation plan was prepared to conduct static testing on two I-64 Bridges, one over KY 32 and the other over Triplett Creek. Testing equipment was placed on the bridges in locations where the maximum effect of the static load from the test vehicles would be experienced. For simply-supported spans, this location was the midspan. The continuous haunched girder design of the I-64 Bridge over Triplett Creek also afforded an opportunity to investigate negative moment effects over the pier support.

Reusable strain gauges for static testing were placed on the girders and on the bridge deck at midspan and over the pier. Linear Variable Differentiable Transducers (LVDTs) were also used to measure the vertical deflection of the girders at midspan. The instrumentation was the same for the before and after SBDO condition so that a comparison could be made between the structural response of the bridges with the new concrete overlay. The details of the instrumentation used in the static and dynamic testing of both bridges were given previously in Sections 2 and 3.

Static testing was accomplished by using two fully-loaded, tandem axle trucks to induce the displacements and strains on the bridges. The gross weights of the two trucks were 80.1 and 82.4 kips (356.3-kN and 366.5-kN). Testing stations were designated along the longitudinal direction of the bridge without regard to traffic lane. This method was employed to maximize the response of the bridge components. All data acquisition channels were read for seven seconds using a sampling rate of 200 Hz while the trucks were each testing location. Subsequent data readings were made by incrementing the truck positions to the next station.

Static testing provided an opportunity to determine the deflections and stresses induced by normal truck traffic under a controlled situation. The results from each test were used to correlate analytical findings on the effectiveness of SBDO.

5.6. EXPERIMENTAL RESULTS

The strains obtained from the gauges on the longitudinal deck reinforcement were combined with the strains recorded along the cross section of the girder to determine the distribution of stress across the composite cross section. Using this procedure, the location of the neutral axis under the static test loads could readily be obtained, and an assessment of the study hypothesis could be made. Likewise, the vertical displacement records offered insight into the response of the girders both before and after the SBDO.

5.6.1. I-64 Bridge Over KY 32: Simply Supported PC Girders

Strain records for the I-64 Bridge over KY 32 did demonstrate a shift in the neutral axis after the concrete overlay for the girder cross section considered. The neutral axis was higher in the girder and slab cross-section after the SBDO. The relatively smooth trace of the strain value across the face also points to an adequate bond between the two slab surfaces, thereby inducing composite action.

A peak response of 31.30μ strain was recorded on the bridge deck above Beam 3 prior to the overlay. The response at the same location was reduced to 10.96μ strain after the rehabilitation work had been completed, representing a 65 percent reduction in compressive stress in the slab. Even though this is a selective reading and represents the best response, the deck never witnessed an increase in strain after the SBDO at any of the four test locations.

A maximum vertical displacement of 0.12-in (3.13-mm) was recorded for the condition prior to the deck overlay, and a maximum vertical displacement of 0.08-in (1.95-mm) was recorded for the bridge after SBDO. This is indicative of a 33% decrease in the deflection of the structure.

Also significant was the apparent contribution to the load distribution offered by the stiffener transverse slab element after the SBDO. Both the strain and displacement values show the exterior girders (beams opposite those which were loaded) contributing more to the overall response of the structure under static testing.

5.6.2. I-64 Bridge Over Triplett Creek: Continuous Haunched RC Girders

Unfortunately, due to the nature of the strain data for the I-64 Bridge over Triplett Creek, an analysis of the strain across the girder and slab cross section cannot be made. It would, therefore, be inappropriate to also comment on whether the SBDO acted compositely with the existing structure.

The vertical displacement data, however, were indicative of the expected results. Generally, a reduction in the displacements of the girders occurred for both the positive and negative moment condition. A maximum vertical deflection for positive moment on Girder 3 of 0.19-in (4.78-mm) was recorded for the condition prior to the deck overlay, and a maximum vertical deflection of 0.15-in (3.89-mm) was recorded for the bridge after SBDO. This represents a reduction of 19% in the deflection of Girder 3 under similar load for the continuous span.

For negative moment, Girder 2 had maximum displacement values of 0.07-in (1.74-mm) and 0.05-in (1.18-mm) for the before and after condition, respectively. This represents a decrease of 32% in upward deflection of Girder 2 under similar load for the continuous span. Also noteworthy is how the displacement in a particular span seemed to be relatively independent of the load position in the adjacent span. This suggests the bridge deck has sufficient stiffness to evenly distribute the load effect to all four girders when subjected to negative moment (upward deflection).

5.6.3. I-64 Bridge over Triplett Creek: Simply Supported RC Girders

Similar to continuous cast-in-place reinforced concrete spans in Section 5.6.2, an increase in the flexural capacity (Fig. 28) of the simply-supported reinforced concrete girders was observed, coupled with a reduction in strains and displacements of the respective girders (Fig. 38), after the application of SBDO.

5.7. EFFECTIVENESS OF STRUCTURAL BRIDGE DECK OVERLAY

A significant advantage in structural response is generally noted due to the SBDO technique. In this study, structural responses such as strains and displacements of three different bridge structures were recorded and compared, before and after SBDO. It is generally observed that the strain and displacement of the bridges were significantly reduced after SBDO, indicating that the bridges' stiffness and overall flexural capacity were markedly improved: (1) strain and displacement readings for the I-64 Bridge over KY 32 (simply-supported, prestressed precast concrete I-girder bridge) indicated an increase in positive moment capacity on the order of 50 percent, (2) strain and displacement readings for the I-64 Bridge over Triplett Creek (continuous haunched reinforced concrete girder bridge) indicated an approximately 23% increase in positive moment capacity and an approximately 47% increase in negative moment capacity, respectively, and (3) strain and displacement reading for the I-64 Bridge over Triplett Creek (simply supported, reinforced concrete girder bridge) indicated an approximately 23% increase in positive moment capacity.

In addition to the overall increase in the stiffness and load carrying capacity because of the new composite sections of the bridges as observed in the experimental data and analytical investigation, the implementation of SBDO also enhanced the load distribution between the adjacent girders due the stiffer transverse slab element.

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Table 1: Total instrumentation required for structural testing

Type of Instrument	Number of Instruments	
	I-64 Bridge over KY 32	I-64 Bridge over Triplett Creek[a]
Concrete strain gauges on deck	4	4
Concrete strain gauges on girders	12	12
Concrete Strain Gauge Total	**16**	**16**
LVDTs	4	4
Static Testing Total	**20**	**20**

[a] total number of instrumentation for the continuous and simply-supported spans.

Table 2: Instrumentation Summary for Static Testing of I-64 Bridge Over KY 32 Prior to Structural Bridge Deck Overlay.

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G1B	726.1	Bottom	34.00
1	322	G1M	630.8	2.88	34.00
2	323	G1T	681.5	24.75	34.00
3	324	G1D	683.6	Deck	34.00
4	325	G2B	656.7	Bottom	34.25
5	326	G2M	652.2	3.00	34.25
6	327	G2T	702.4	25.00	34.25
7	328	G2D	709.6	Deck	34.25
8	329	G3B	611.9	Bottom	34.63
9	330	G3M	634.6	3.00	34.63
10	331	G3T	693.0	24.25	34.63
11	332	G3D	662.6	Deck	34.63
12	333	G4B	666.3	Bottom	34.79
13	334	G4M	611.3	3.50	34.79
14	335	G4T	656.4	25.14	34.79
15	290	G4D	617.0	Deck	34.79
16	11	G2	20.427	Bottom	34.71
17	12	G1	20.481	Bottom	34.96
18	13	G3	20.506	Bottom	35.33
19	14	G4	20.396	Bottom	35.50

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Table 3: Instrumentation Summary for Static Testing of I-64 Bridge Over KY 32 After Structural Bridge Deck Overlay.

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G1B	726.1	Bottom	34.00
1	322	G1M	630.8	2.88	34.00
2	323	G1T	681.5	24.75	34.00
3	324	G1D	683.6	Deck	34.00
4	325	G2B	656.7	Bottom	34.25
5	326	G2M	652.2	3.00	34.25
6	327	G2T	702.4	25.00	34.25
7	328	G2D	709.6	Deck	34.25
8	329	G3B	611.9	Bottom	34.63
9	330	G3M	634.6	3.00	34.63
10	331	G3T	693.0	24.25	34.63
11	332	G3D	662.6	Deck	34.63
12	333	G4B	666.3	Bottom	34.79
13	334	G4M	611.3	3.50	34.79
14	335	G4T	656.4	25.14	34.79
15	290	G4D	617.0	Deck	34.79
16	12	G2	20.481	Bottom	34.71
17	11	G1	20.427	Bottom	34.96
18	13	G3	20.506	Bottom	35.33
19	14	G4	20.396	Bottom	35.50

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Table 4: Instrumentation Summary for Static Testing of I-64 Bridge Over Triplett Creek Before and After Structural Bridge Deck Overlay Gauges at Midspan of Span 3

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G4B	726.1	Bottom	50.00
1	322	G4M	630.8	6.50	50.00
2	323	G4T	681.5	38.5	50.00
3	324	G4D	683.6	Deck	50.00
4	325	G3B	656.7	Bottom	50.00
5	326	G3M	652.2	6.50	50.00
6	327	G3T	702.4	38.5	50.00
7	328	G3D	709.6	Deck	50.00
8	329	G2B	611.9	Bottom	50.00
9	330	G2M	634.6	6.50	50.00
10	331	G2T	693.0	38.5	50.00
11	332	G2D	662.6	Deck	50.00
12	333	G1B	666.3	Bottom	50.00
13	334	G1M	611.3	6.50	50.00
14	335	G1T	656.4	38.5	50.00
15	290	G1D	617.0	Deck	50.00
16	11	G4	20.427	Bottom	50.00
17	12	G3	20.481	Bottom	50.00
18	13	G2	20.506	Bottom	50.00
19	14	G1	20.396	Bottom	50.00

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Table 5: Instrumentation Summary for Static Testing of I-64 Bridge Over Triplett Creek Before and After Structural Bridge Deck Overlay Gauges at Pier Support

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G4B	726.1	11.50	100.00
1	322	G4M	630.8	49.50	100.00
2	323	G4T	681.5	92.50	100.00
3	324	G4D	683.6	Deck	100.00
4	325	G3B	656.7	9.50	100.00
5	326	G3M	652.2	51.50	100.00
6	327	G3T	702.4	91.50	100.00
7	328	G3D	709.6	Deck	100.00
8	329	G2B	611.9	11.50	100.00
9	330	G2M	634.6	49.50	100.00
10	331	G2T	693.0	91.00	100.00
11	332	G2D	662.6	Deck	100.00
12	333	G1B	666.3	9.50	100.00
13	334	G1M	611.3	49.50	100.00
14	335	G1T	656.4	91.00	100.00
15	290	G1D	617.0	Deck	100.00
16	N/A	N/A	N/A	N/A	N/A
17	N/A	N/A	N/A	N/A	N/A
18	N/A	N/A	N/A	N/A	N/A
19	N/A	N/A	N/A	N/A	N/A

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Table 6: Instrumentation Summary for Static Testing of I-64 Bridge Over Triplett Creek Before and After Structural Bridge Deck Overlay Gauges at Midspan of Span 2

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G4B	726.1	Bottom	35.00
1	322	G4M	630.8	5.50	35.00
2	323	G4T	681.5	38.5	35.00
3	324	G4D	683.6	Deck	35.00
4	325	G3B	656.7	Bottom	35.00
5	326	G3M	652.2	5.50	35.00
6	327	G3T	702.4	38.5	35.00
7	328	G3D	709.6	Deck	35.00
8	329	G2B	611.9	Bottom	35.00
9	330	G2M	634.6	5.50	35.00
10	331	G2T	693.0	38.5	35.00
11	332	G2D	662.6	Deck	35.00
12	333	G1B	666.3	Bottom	35.00
13	334	G1M	611.3	5.50	35.00
14	335	G1T	656.4	38.5	35.00
15	290	G1D	617.0	Deck	35.00
16	11	G4	20.427	Bottom	35.00
17	12	G3	20.481	Bottom	35.00
18	13	G2	20.506	Bottom	35.00
19	14	G1	20.396	Bottom	35.00

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Table 7: Instrumentation Summary for Static Testing of I-64 Bridge Over Triplett Creek Before and After Structural Bridge Deck Overlay Gauges at Midspan of Span 1

Channel	Gauge/ LVDT	Girder Location[a]	Calibration Factor	Vertical Location (in)[b]	Horizontal Location (ft)[c]
0	321	G4B	726.1	Bottom	25.00
1	322	G4M	630.8	7.25	25.00
2	323	G4T	681.5	38.5	25.00
3	324	G4D	683.6	Deck	25.00
4	325	G3B	656.7	Bottom	25.00
5	326	G3M	652.2	7.25	25.00
6	327	G3T	702.4	38.5	25.00
7	328	G3D	709.6	Deck	25.00
8	329	G2B	611.9	Bottom	25.00
9	330	G2M	634.6	7.25	25.00
10	331	G2T	693.0	38.5	25.00
11	332	G2D	662.6	Deck	25.00
12	333	G1B	666.3	Bottom	25.00
13	334	G1M	611.3	7.25	25.00
14	335	G1T	656.4	38.5	25.00
15	290	G1D	617.0	Deck	25.00
16	11	G4	20.427	Bottom	25.00
17	12	G3	20.481	Bottom	25.00
18	13	G2	20.506	Bottom	25.00
19	14	G1	20.396	Bottom	25.00

[a] B=bottom, M=middle, T=top, D=deck.
[b] measured from bottom of girder.
[c] measured from Eastern pier.

Figure 1. Commonwealth of Kentucky Highlighting Study Bridge Locations.

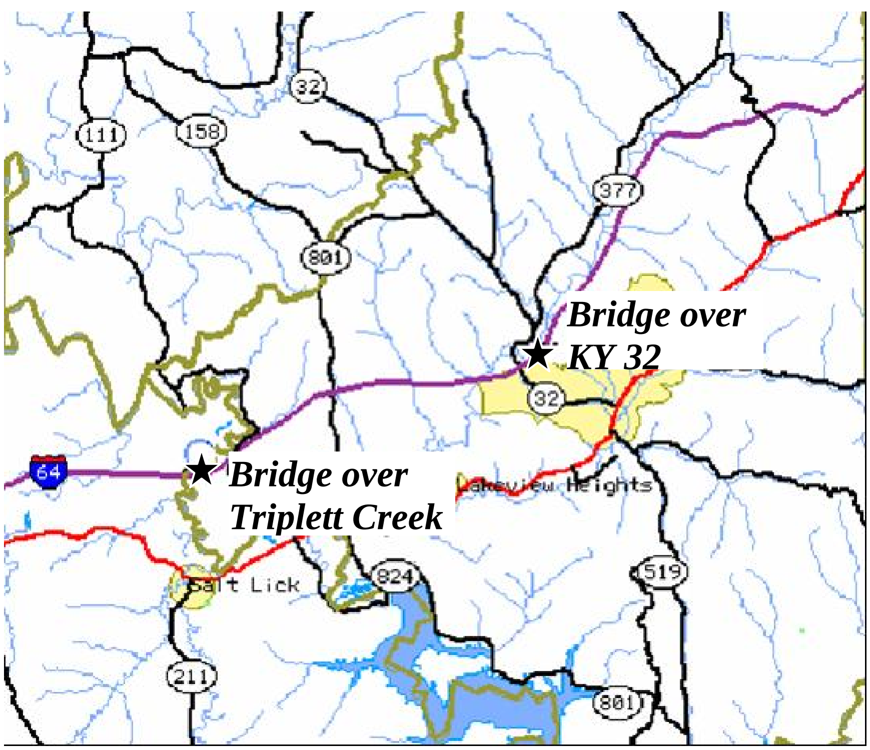


Figure 2. Study Bridge Locations within Rowan County.

Figure 3. Example of AASHTO-Type Prestressed Concrete I-Girder.

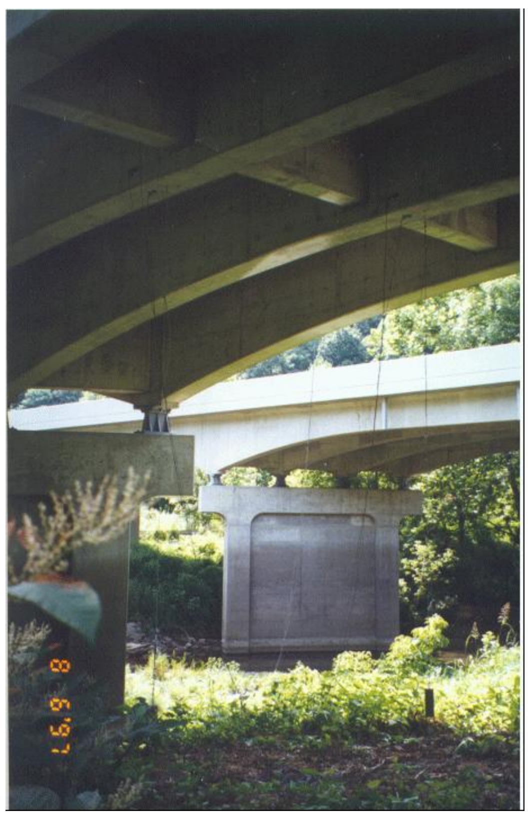


Figure 4. Example of Concrete Intermediate Diaphragms in Bridge with Haunched Girder.

Figure 5. Skew of Bridge Girders
Relative to Pier Support.

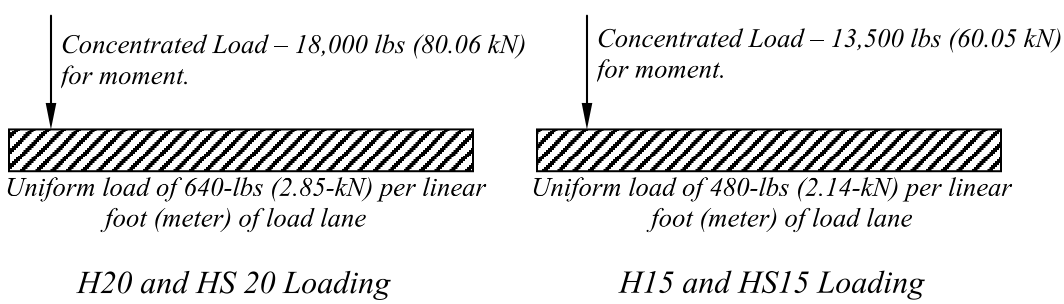


Figure 6. Equivalent Lane Loadings Substituted for the Truck Trains of the 1935 AASHTO Specifications (AASHTP 1996).

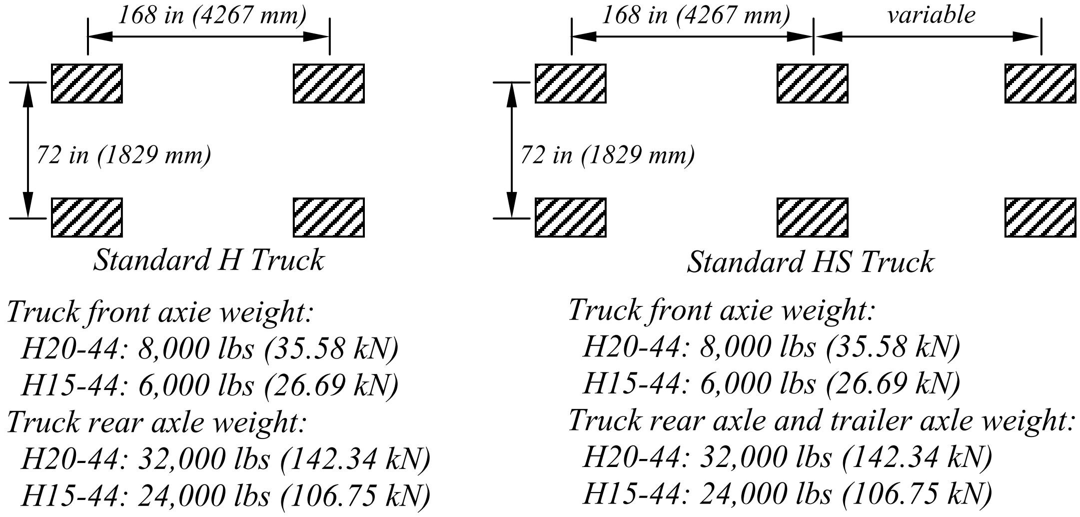


Figure 7. Footprint of the AASHTO H and HS Trucks (AASHTO 1996).

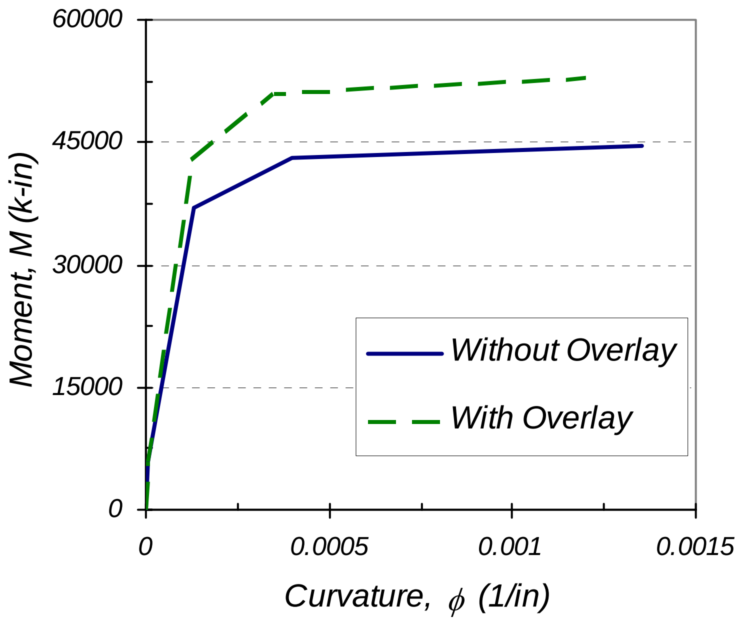


Figure 8. *M*-ϕ Relationship With and Without SBDO at Midspan of Prestressed Concrete I-Girder on I-64 Bridge over KY 32.

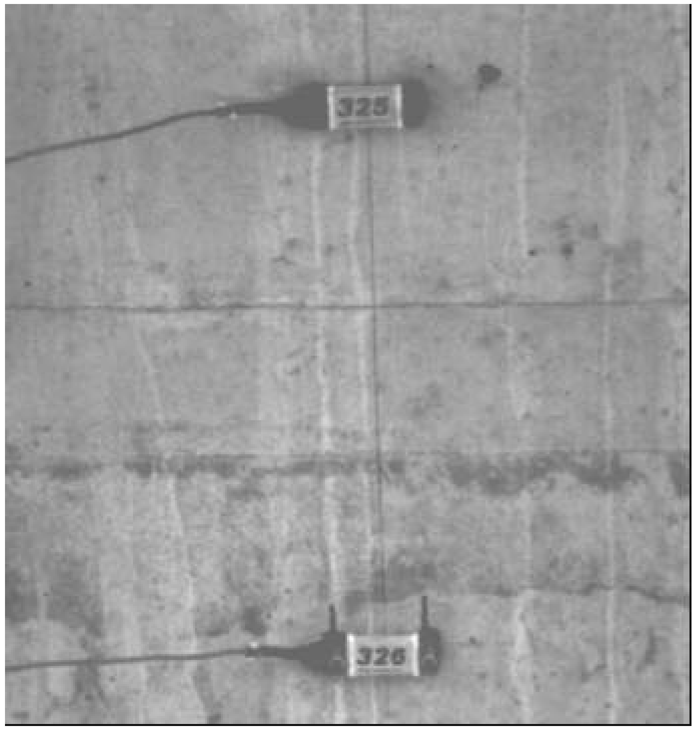


Figure 9. Reusable Strain Gauges
Mounted on a Girder.

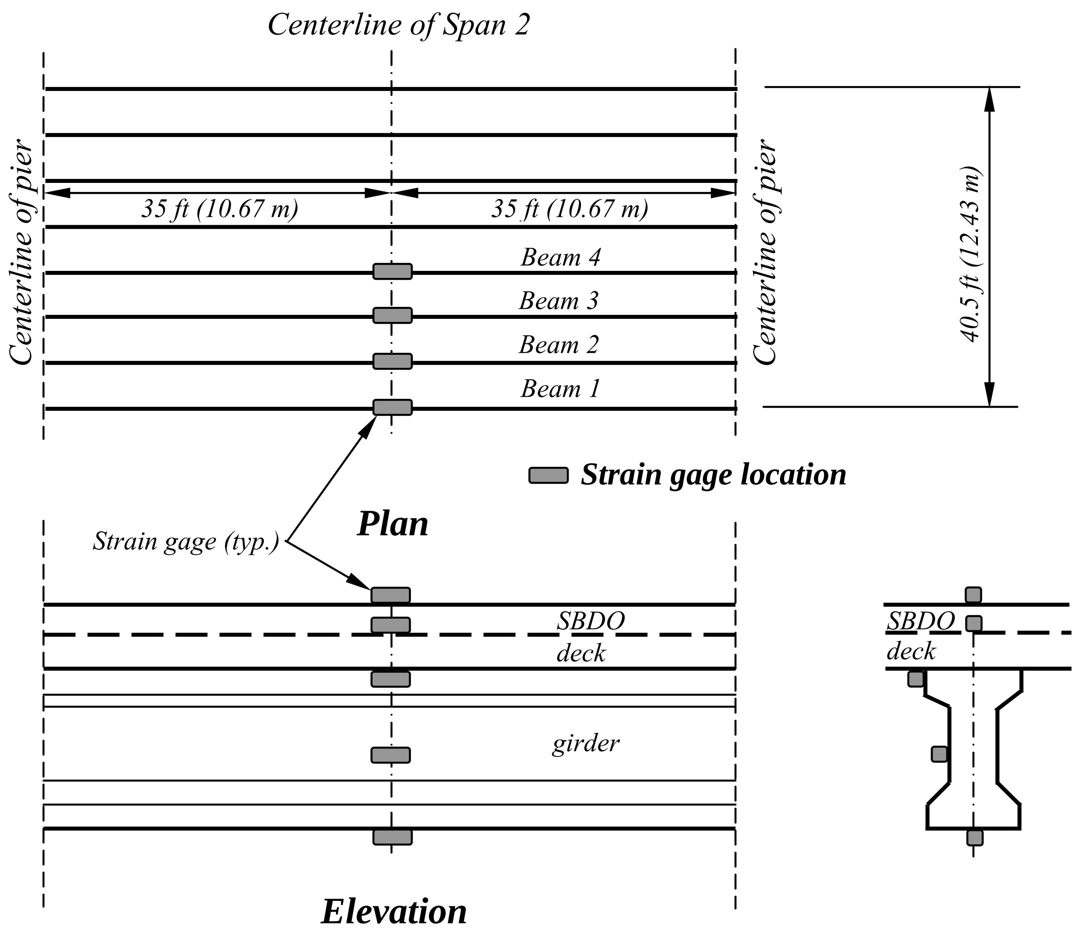


Figure 10. Strain Gauge Placement on the Deck and Girders of the I-64 Bridge over KY32.

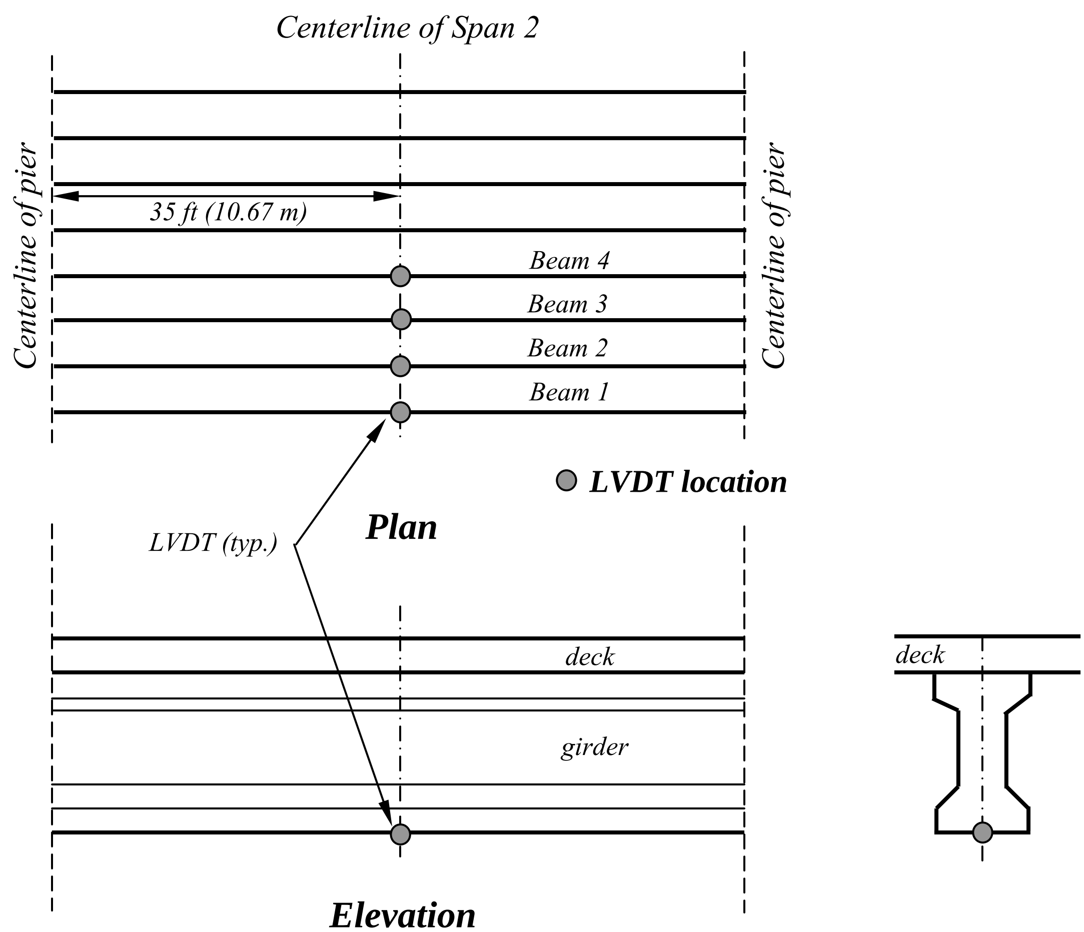
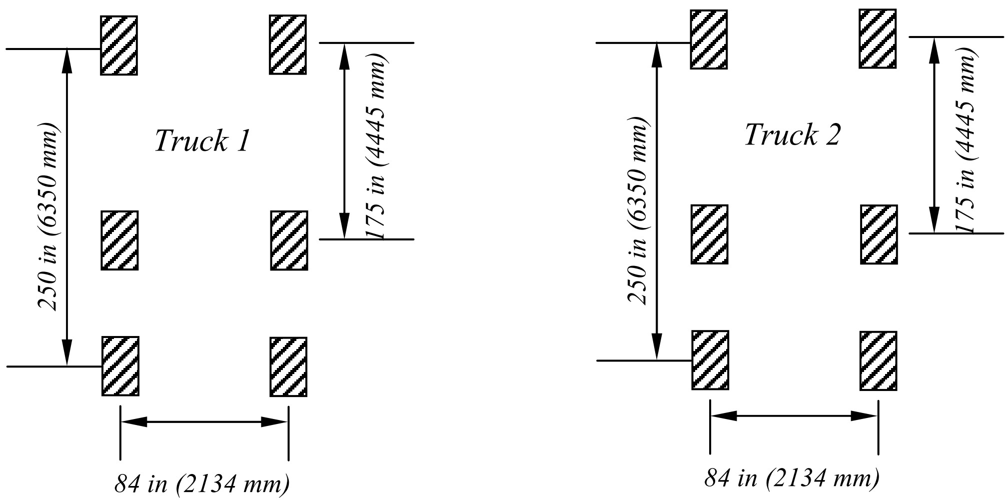


Figure 11. LVDT Locations on the Girders of the I-64 Bridge over KY32.

Figure 12. Prestressed Concrete I-Girder Construction of I-64 Bridge over KY32.

Figure 13. Test Vehicles – Tandem Axle Trucks (Third Rear Axle not Engauged).



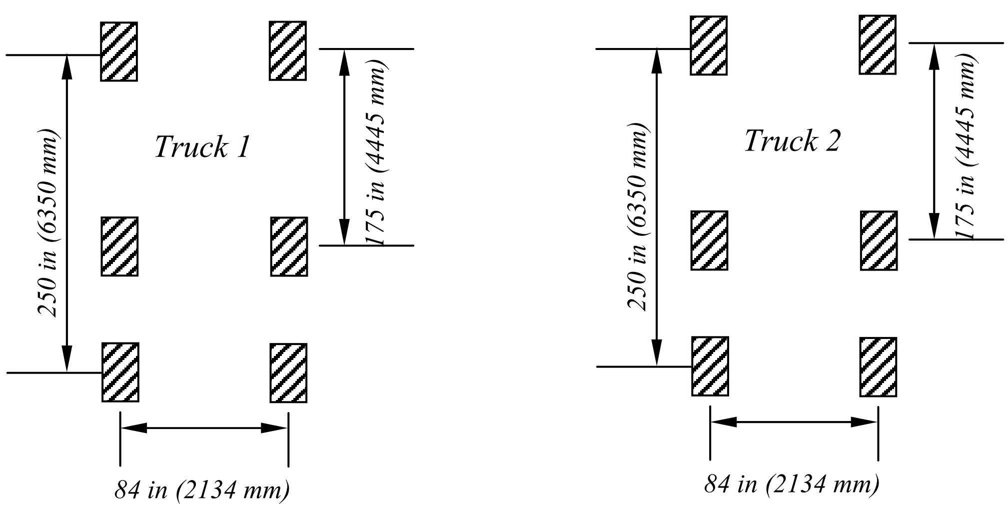
Front Axle = 23.6 kips (105.02 kN)

Rear Tandem = 56.8 kips (252.67 kN)

Front Axle = 23.4 kips (104.13 kN)

Rear Tandem = 59.6 kips (265.09 kN)

Figure 14. Footprints and Axle Weights of Static Test Trucks Before SBDO.



Front Axle = 24.1 kips (107.39 kN)

Rear Tandem = 55.8 kips (248.18 kN)

Front Axle = 24.4 kips (108.58 kN)

Rear Tandem = 57.7 kips (256.77 kN)

Figure 15. Footprints and Axle Weights of Static Test Trucks After SBDO.

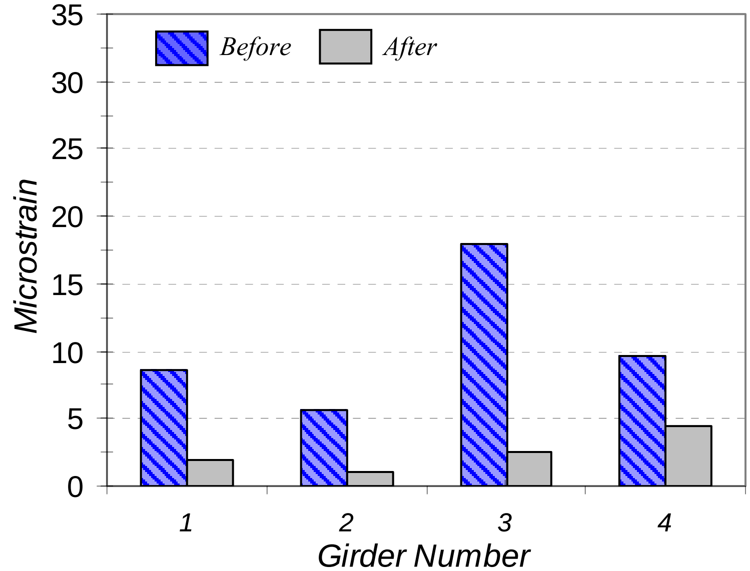


Figure 16. Deck Strain With Trucks at One-Quarter Span Station, I-64 Bridge over KY32.

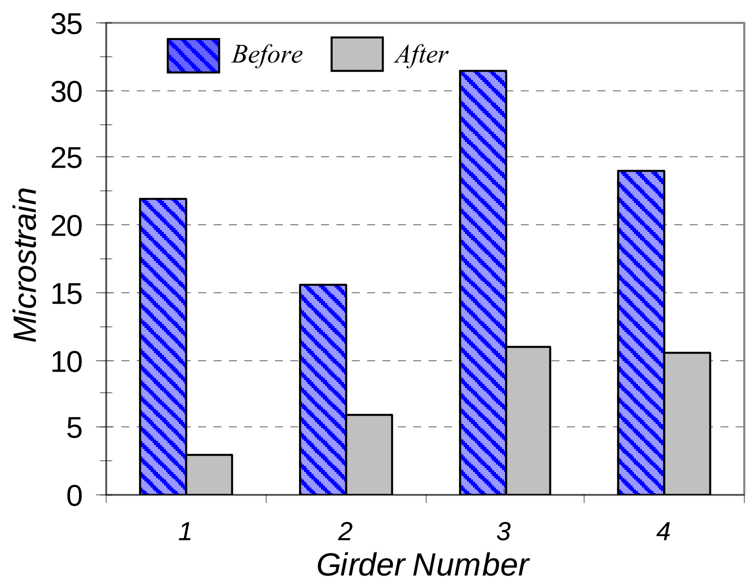


Figure 17. Deck Strain With Trucks at Midspan Station, I-64 Bridge over KY32.

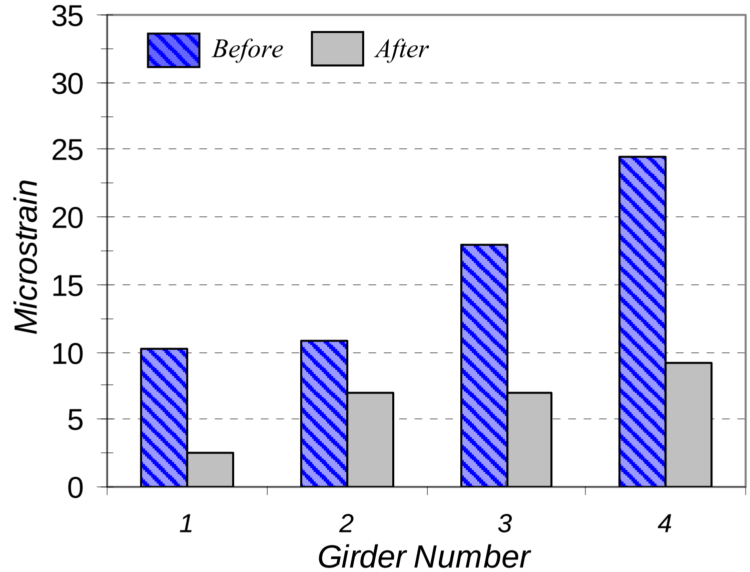


Figure 18. Deck Strain With Trucks at Three-Quarter Span Station, I-64 Bridge over KY32.

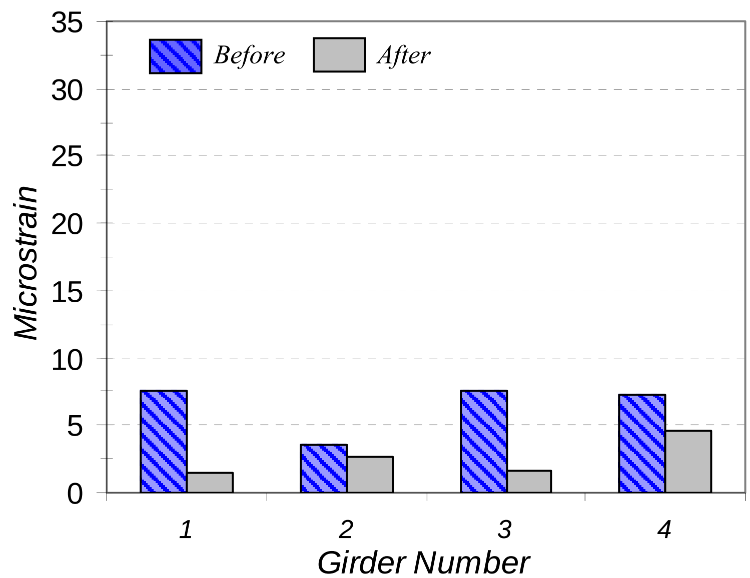


Figure 19. Deck Strain With Trucks at Pier Support, I-64 Bridge over KY32.

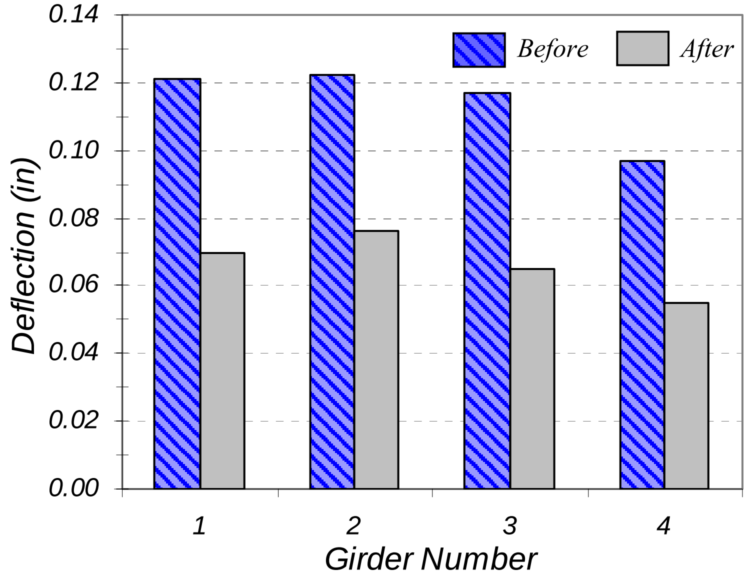


Figure 20. Deflection With Trucks at Midspan Station,
I-64 Bridge over KY32.
Note: 1 inch = 25.4 mm.

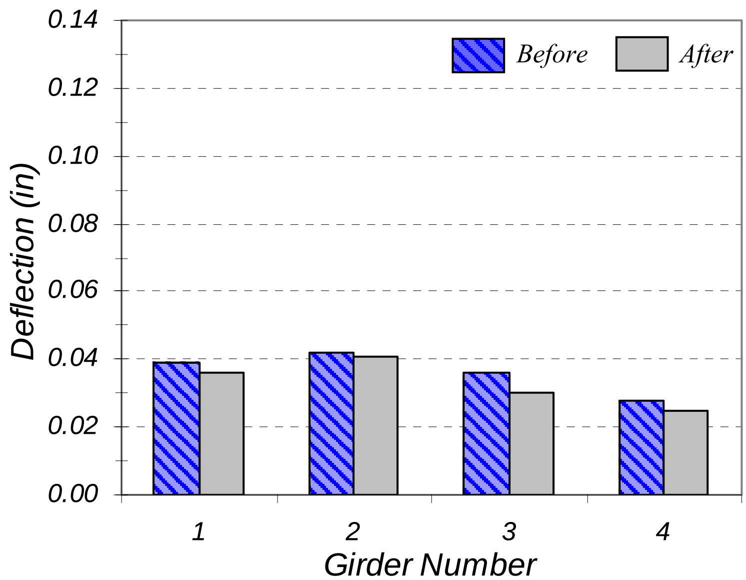


Figure 21. Deflection With Trucks at Pier Support,
I-64 Bridge over KY32.
Note: 1 inch = 25.4 mm.

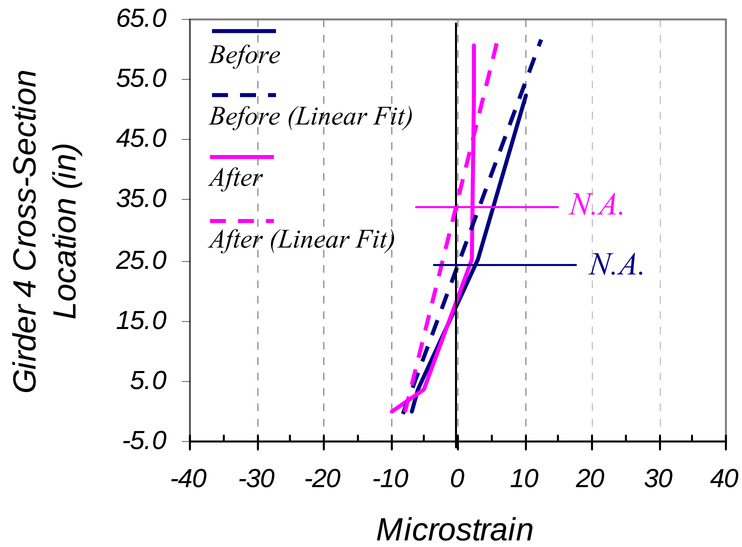


Figure 22. Beam Strain Before and After SBDO With Trucks at One-Quarter Span Station I-64 Bridge over KY32. Note: 1 inch = 25.4 mm

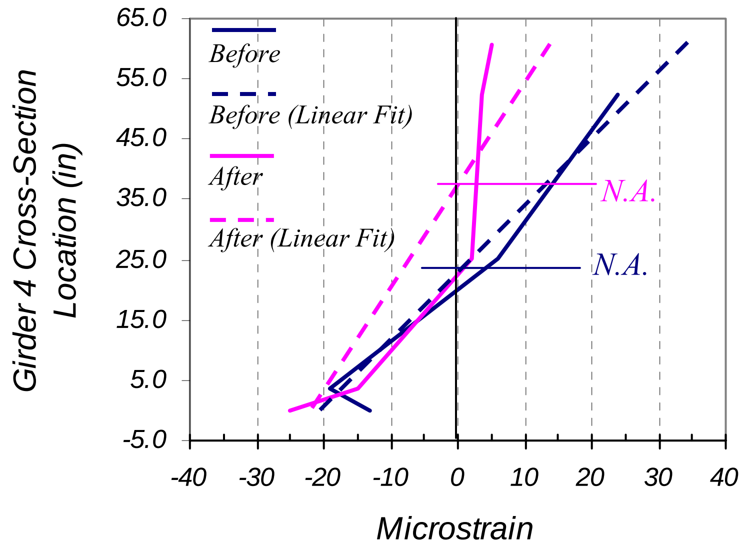


Figure 23. Beam Strain Before and After SBDO With Trucks at Midspan Station I-64 Bridge over KY32. Note: 1 inch = 25.4 mm

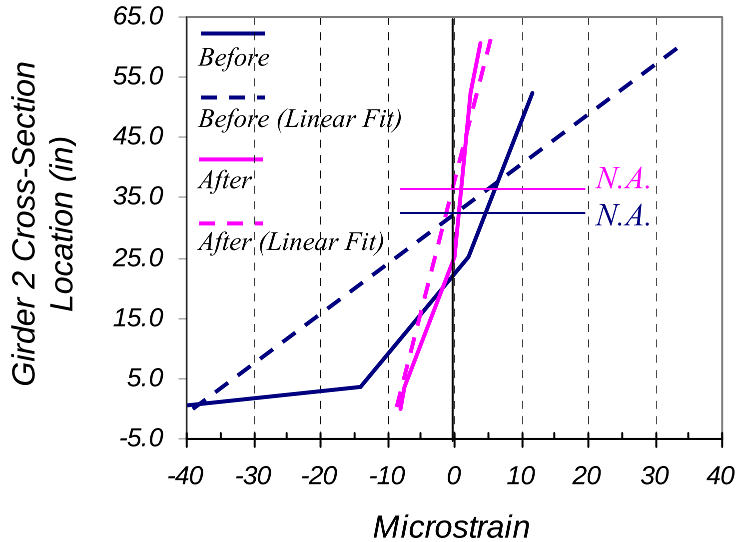


Figure 24. Beam Strain Before and After SBDO With Trucks at Three-Quarter Span Station I-64 Bridge over KY32.
Note: 1 inch = 25.4 mm

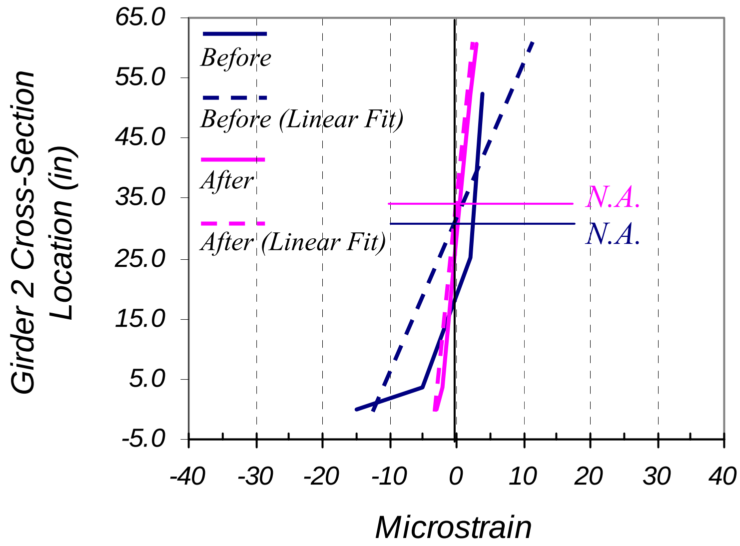


Figure 25. Beam Strain Before and After SBDO With Trucks at Pier Support I-64 Bridge over KY32.
Note: 1 inch = 25.4 mm

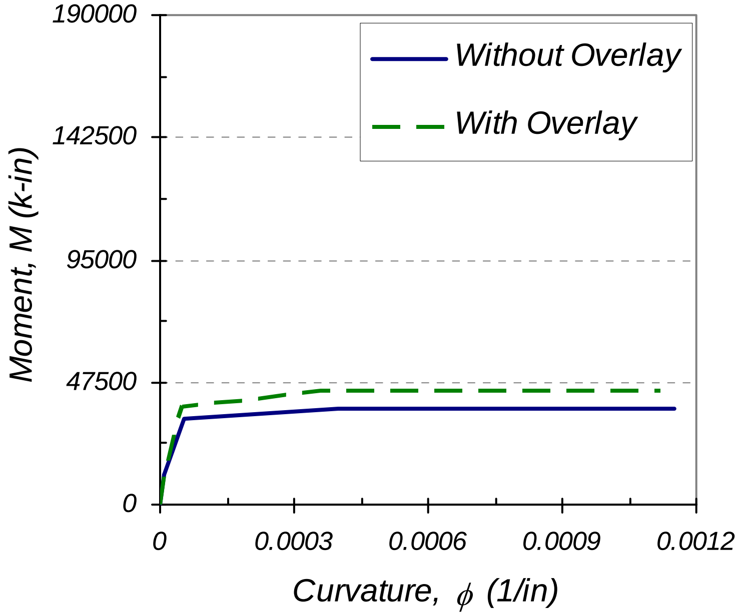


Figure 26. M-ϕ Relationship With and Without SBDO at Midspan of Continuous, Cast-In-Place Concrete Haunched Girder on I-64 Bridge over Triplett Creek.

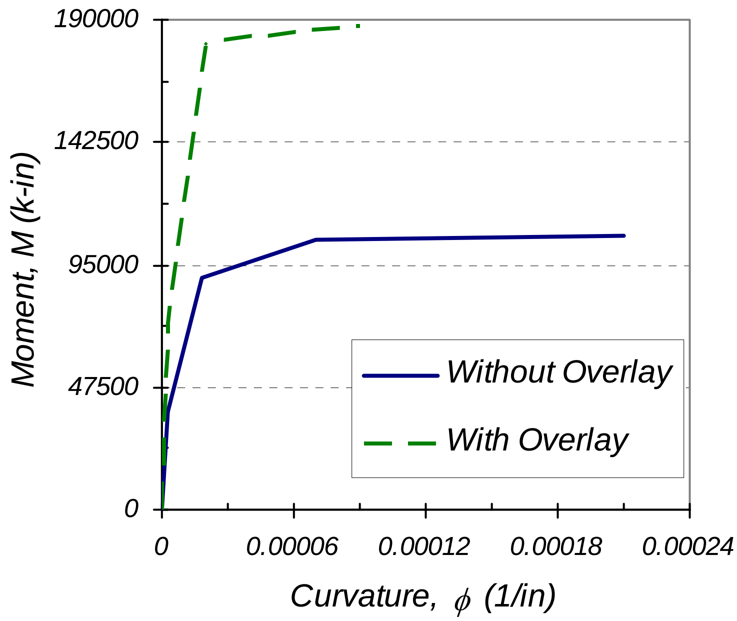


Figure 27. M-ϕ Relationship With and Without SBDO at Pier Support of Continuous, Cast-In-Place Concrete Haunched Girder on I-64 Bridge over Triplett Creek.

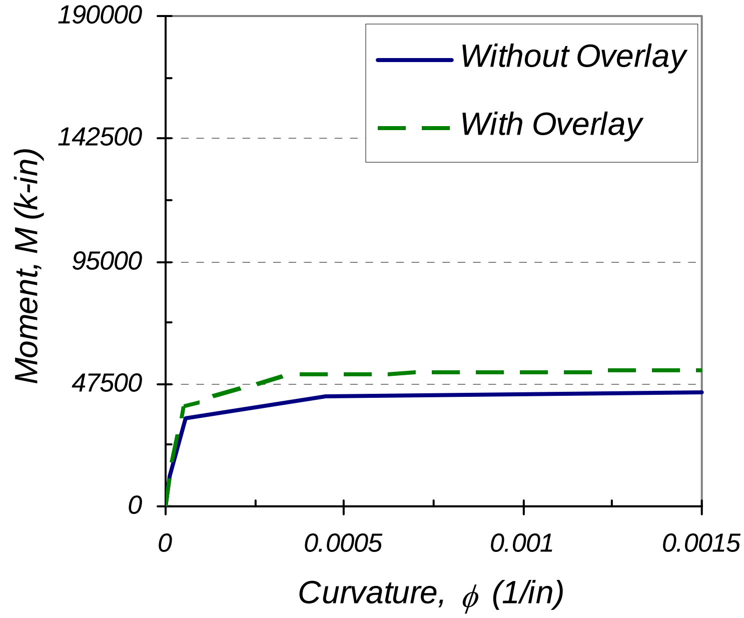


Figure 28. M-ϕ Relationship With and Without SBDO at Midspan of Simply-Supported, Cast-In-Place Concrete Girder on I-64 Bridge over Triplett Creek.

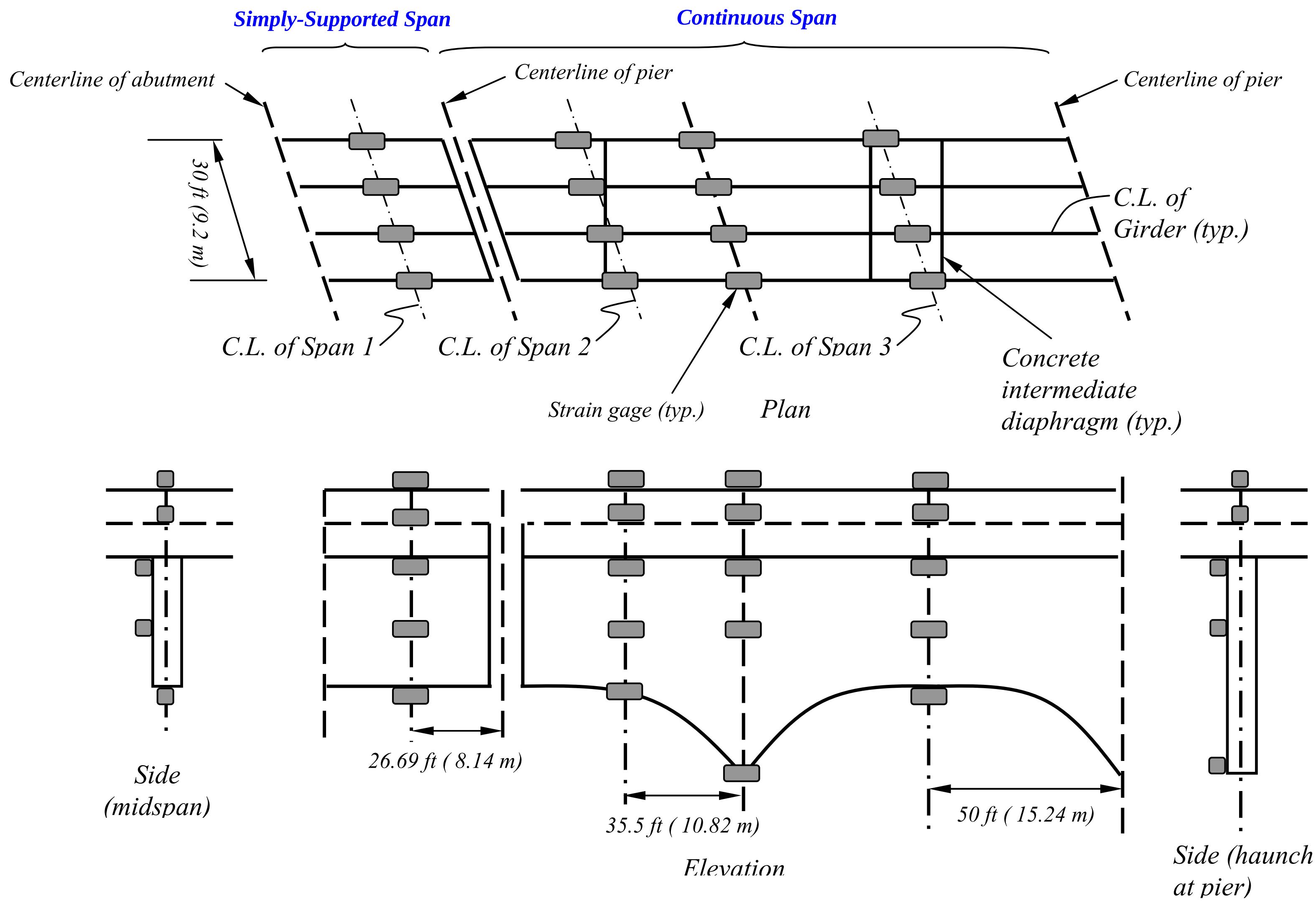


Figure 29. Strain Gage Placement on the Deck and Girders of the I-64 Bridge over Tripplett Creek.

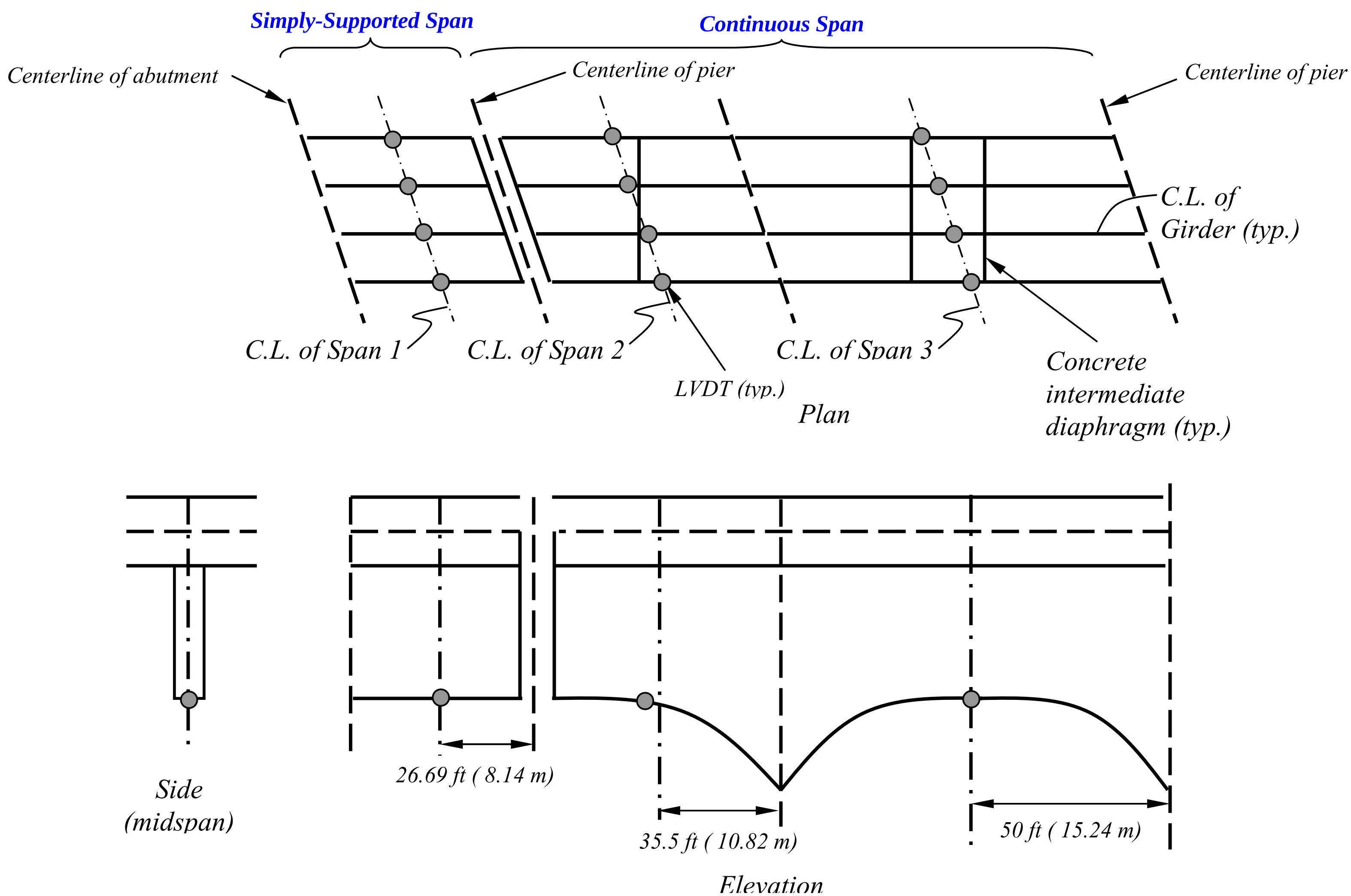


Figure 30. LVDT Locations on the Girders of the I-64 Bridge over Tripplett Creek.

Figure 31. Haunched Girder Construction of I-64 Bridge Over Triplett Creek.

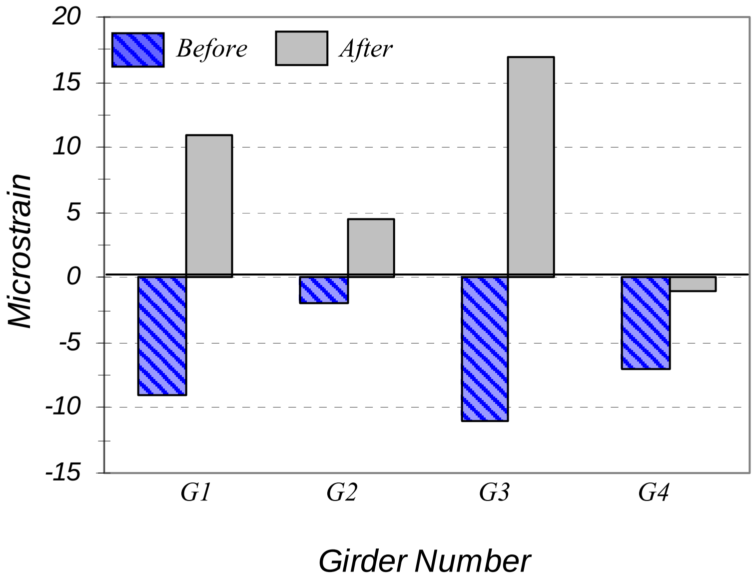


Figure 32. Pier Deck Strain With Trucks at Midspan of Span 3, I-64 Bridge over Triplett Creek
(Test Pass #1 – Rear Axle Next to Gauge Location)

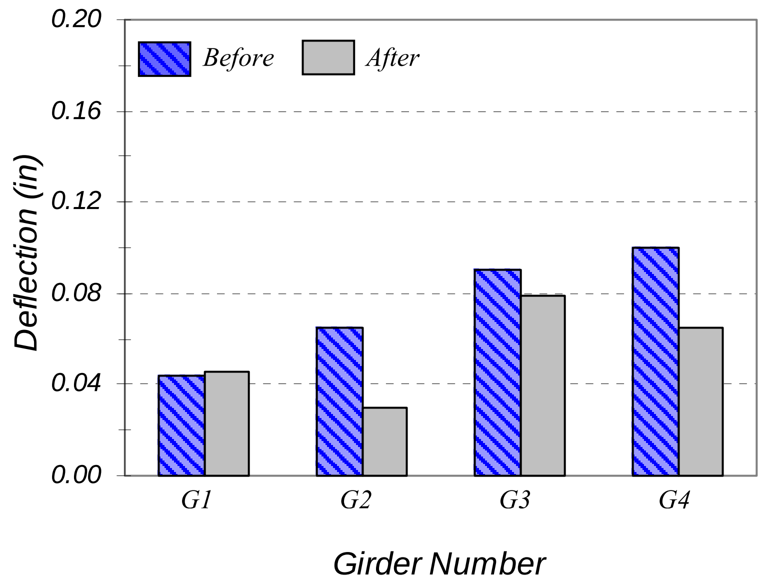


Figure 33. Deflection of Span 2 With Trucks at Midspan of Span 2, I-64 Bridge over Triplett Creek (Rear Axle Next to Gauge Location) Note: 1 inch = 25.4 mm

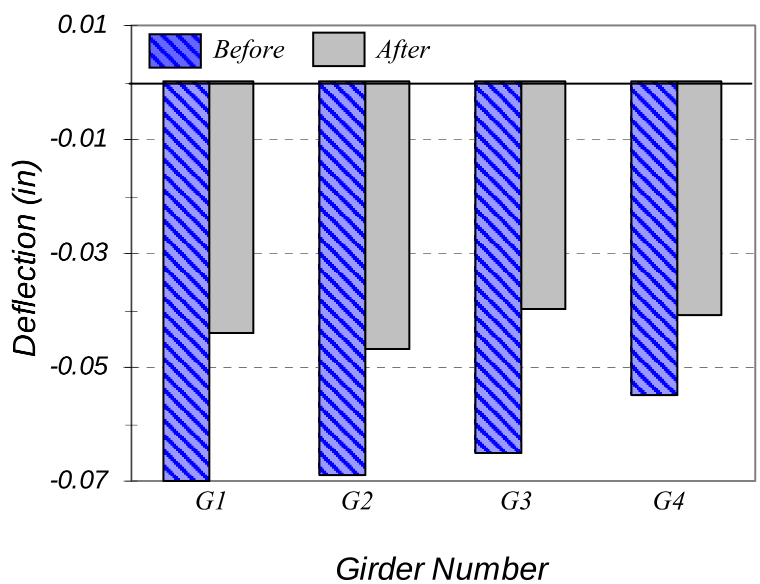


Figure 34. Deflection of Span 2 With Trucks at Midspan of Span 3, I-64 Bridge over Triplett Creek (Rear Axle Next to Gauge Location) Note: 1 inch = 25.4 mm

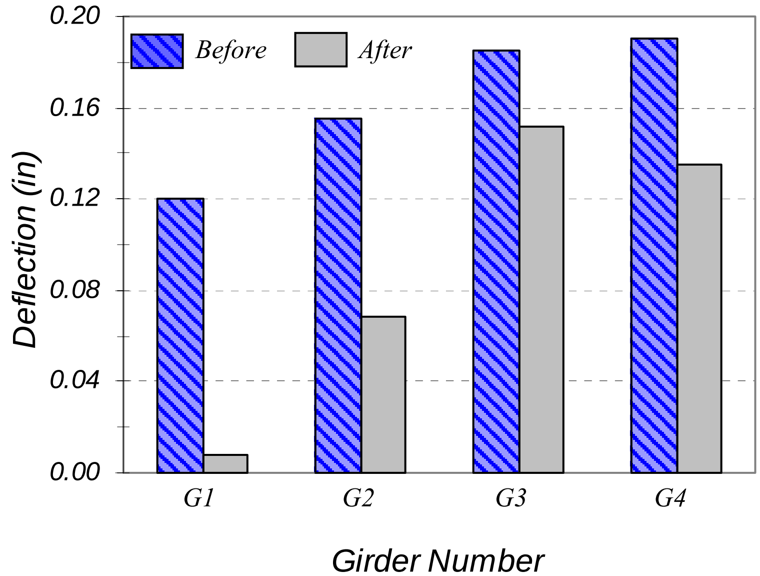


Figure 35. Deflection of Span 3 With Trucks at Midspan of Span 3, I-64 Bridge over Triplett Creek (Rear Axle Next to Gauge Location) Note: 1 inch = 25.4 mm

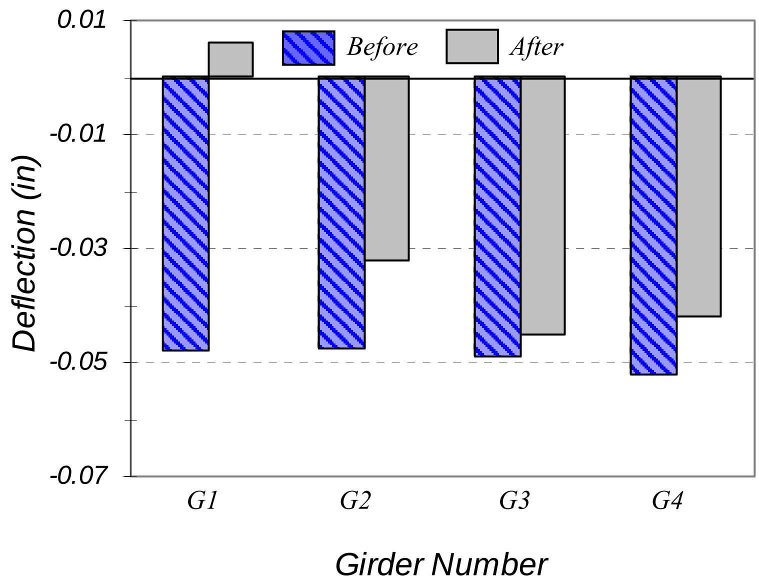


Figure 36. Deflection of Span 3 With Trucks at 3rd Point of Span 2, I-64 Bridge over Triplett Creek (Rear Axle Next to Gauge Location) Note: 1 inch = 25.4 mm

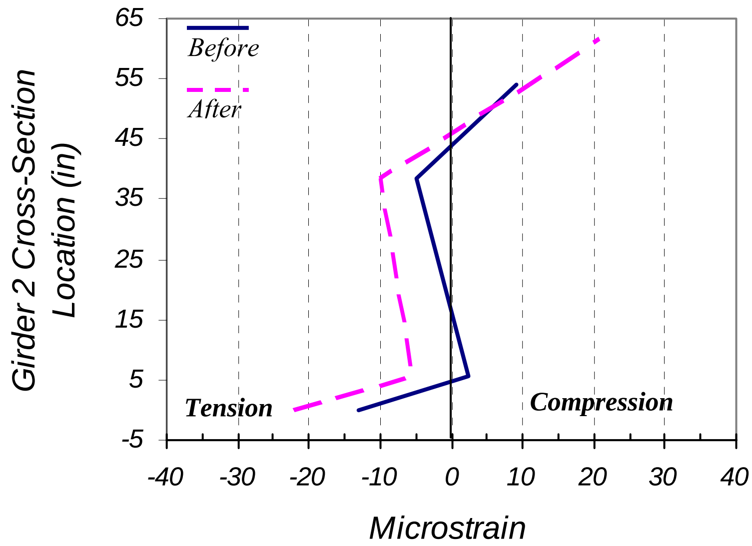


Figure 37. Beam Strain Before and After SBDO With Trucks at Midspan of Span 3, I-64 Bridge over Triplett Creek (Note: 1 inch = 25.4 mm)

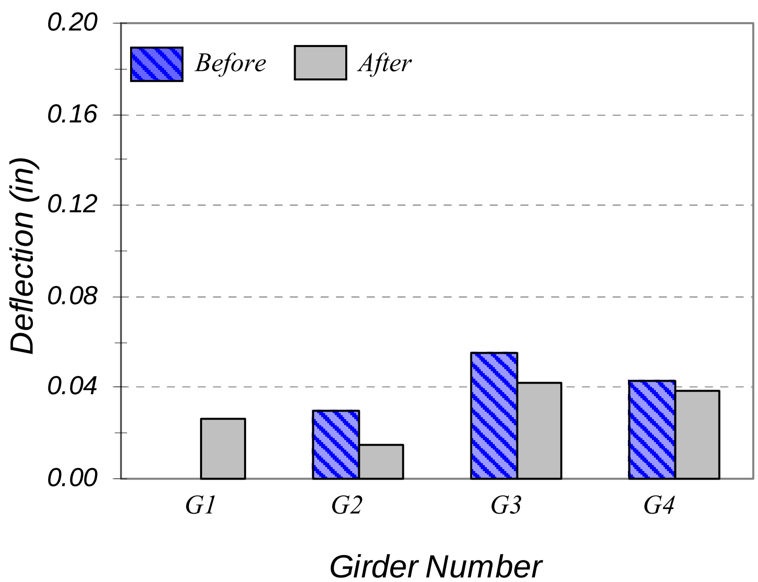


Figure 38. Deflection With Trucks at Midspan of Span 1, I-64 Bridge over Triplett Creek (Rear Axle Next to Gauge Location) Note: 1 inch = 25.4 mm