MICHIGAN DEPARTMENT OF TRANSPORTATION
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INSTRUMENTATION OF CONCRETE T-BEAM BRIDGE DURING DECK REPLACEMENT PARTELLO ROAD OVER I-94 STRUCTURE NO. S04 OF 13083

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Research Project 94 TI-1737
Research Report No. R-1349

Lansing, February 1997

Introduction

In the State of Michigan there are 703 reinforced concrete T-beam bridges: 419 on the interstate system and M routes, which are maintained by the Michigan Department of Transportation (MDOT), and 284 on local roadways, which are maintained by local agencies such as counties and cities. Of Michigan's 703 T-beam bridges, 498 are short and medium span, simply supported structures, while the remaining 205 structures are continuous-span, haunched T-beam bridges as shown in Figure 1.

Reinforced concrete T-beam bridge decks are composite with the beams and are built with falsework in place to take the dead load during concrete curing. When the structure has continuous spans, reinforcing steel is placed in the deck to account for negative moments over the piers. This becomes a problem to designers when the deck needs to be removed or replaced because the steel is relied upon to support the beams. Designers have questioned whether temporary supports should be required in contract plans for projects involving superstructure removal or when doing full-depth deck replacements. In this case, temporary supports were not necessary, but were included in the contract plans because of the experimental nature of the project.

In November 1994, the Structural Research Unit received a request from the Design Division to monitor the response of a continuous span, reinforced concrete T-beam bridge while it underwent a deck replacement and widening. The bridge, shown in Figures 1 and 2, carries Partello Road over I-94 near Marshall. Since the construction method used on this project was an experimental procedure, the designers wanted the stresses in the beams and loads in the temporary supports to be monitored during the deck removal sequence and during the forming and casting of the new deck. The Federal Highway Administration was concerned about how the structure would react during the deck replacement and how much *locked-in* stress would result in the beams' steel reinforcement from the construction procedure.

A meeting was held to assess the designer's concerns and to develop a plan-of-action for the project. Resulting from the meeting were the following action items:

1. Monitor the bridge beams during the deck replacement to determine if the beams are over stressed at any time because of the construction activity. Of particular interest are the negative moment regions over piers 1 and 3 where removing the deck reduces the tensile reinforcement from fourteen to six 32 mm (1-1/4 inch) square bars, a 57 percent decrease in reinforcement.

2. Compare the dead load stress level in the beams before and after construction to determine if any *locked-in* stress resulted from the construction sequence which would reduce the bridge's live load carrying capacity.

3. Monitor the temporary support loads during the deck replacement. The actual load would be compared to the allowable load each temporary support can carry. This

information could be used to adjust the temporary support design or to determine whether the temporary supports are required on future projects of this type.

Please note: In this report all details (beams, piers, span...etc.) are referenced according to the rehabilitation plans (Job Number 31365A) and are shown in the accompanying figures. U.S. customary units are shown in parenthesis.

Partello Road Bridge Rehabilitation

As shown in the General Plan of Structure (Figure 2), the Partello Road bridge has four spans and crosses I-94 at an angle of 39 degrees. The span lengths are 20.6 meters (67.5 feet), 34.0 meters (111.7 feet), 34.0 meters (111.7 feet), and 20.6 meters (67.5 feet) for spans 1 through 4, respectively. The total bridge length is 109.2 meters (358.3 feet).

The deck replacement and widening of the Partello Road bridge were done using part-width construction. During the 1995 construction season, traffic was diverted to the southeast side of the bridge, so work could begin on the northwest side. Temporary supports were used near piers 1 and 3, as shown in Figure 2, to ensure the beams were not over stressed when the deck was removed. The parapet railing, sidewalk, and the deck between beams G1, G2, and G3 were all removed, as shown in Figure 3. The intermediate diaphragms, which were originally scheduled for removal, were left in place. The deck directly over the beams was intentionally left in place to preserve some of the negative moment reinforcement which was needed to support the structure. Figure 4 shows the structure after the old deck was removed and the forms for the new deck being placed in span 4.

The intermediate diaphragms were originally scheduled for removal. The designers were concerned that, if left in place, the diaphragms would restrict the beams from deflecting upwards when the old deck was removed, thus *locking-in* stress into the beams. They were also concerned distress could result in the diaphragms or in the beams near center span. During construction, the contractor requested the diaphragms be left in place because they (the diaphragms) were in good condition and replacing them would be an extensive job. Since we were closely monitoring the beams, the designers agreed to allow the diaphragms to remain.

A new 200 mm (8 inch) deck was placed, as shown in Figure 5. Because the original deck was not removed directly over the beams, the total beam height and the elevation of the new roadway increased by 200 mm (8 inches) after the new deck was cast. The new deck was made composite to the beams by epoxy-anchoring bent steel reinforcing bars into the beams to act as shear developers. The deck over the piers was cast first to increase the moment of inertia of the section and to help reduce stress in the negative moment reinforcement when the center spans were poured (Figure 6). This is contrary to our normal procedure of pouring positive moment areas of continuous spans first to preload the negative moment areas.

In 1996, after all bridge and approach work had been completed on the northwest side, traffic was rerouted, and a similar removal and construction procedure was used on the southeast side. The major difference in procedures between the two years was that temporary supports were not used in 1996 due to our 1995 test results. This aspect is discussed in the conclusion of this report.

Methodology

Several techniques were used to achieve the objectives of this study, including testing the beam's material properties, doing a detailed structural analysis of the continuous spans, and monitoring the actual response of the beams during the deck replacement. The following sections describe each:

Structural Analysis

Using formulas in the AASHTO Bridge Code[1], Section 8.13, we calculated the beam's gross and effective moment of inertia at discrete sections for the existing beam, and also for the beam with the deck removed. The maximum moment in the member, M_a, was calculated for Michigan's largest legal load: a 77-ton, two unit truck. The bridge beam was divided into 35 sections, as shown in the original construction plans. The resulting gross and effective moments of inertia along with the percent reduction of the effective moments of inertia from the gross are shown in Tables 1 and 2. These values were used to calculate the expected strain and the deflection of the specified beam section during the deck removal and casting the new deck. We also calculated the final gross moment of inertia of the bridge beam. In Table 3, this is compared to the original section to demonstrate the beam's additional final capacity.

Material Properties

Knowing the material properties of the reinforced concrete beams helps us estimate the response of the structure to load. Samples of the bridge's reinforcing steel and cores from the original concrete deck were taken. The results of steel sample testing are shown in Table 4, and the results of the concrete core testing are shown in Table 5.

Allowable Stress

Knowing the compressive strength of the concrete and the yield point of the steel, we compared the measured stress in the beam during the construction activity to the allowable stress calculated using the working stress method.

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[1]American Association of State Highway and Transportation Officials Standard Specificaitons for Highway Bridges, Fifteenth Edition, 1992

The allowable stress in the reinforcing steel according to the AASHTO Bridge Code is:

$$f_s = 0.5\, f_y \text{ for Grade 276 (40,000 psi) Steel}$$

$$f_s = 0.4\, f_y \text{ for Grade 414 (60,000 psi) Steel}$$

From the tensile tests, we found the lower bound (95 percent confidence) tensile strength was 350 MPa (51,000 psi.), so $f_s = 0.4\ F_y = 140$ MPa ($f_s = 20{,}000$ psi.). With the modulus of elasticity for the steel = 200 x 10^3 MPa (29 x 10^6 psi.), the allowable strain in the steel equals:

$$\epsilon_s = \frac{140\ MPa.}{200\ x10^3\ MPa.}(1x10^6 \mu\epsilon/\epsilon) \;=\; 700\ \mu\epsilon$$

The allowable compressive stress in the concrete according to the AASHTO Bridge code is:

$$f_c = 0.4\, f'_c$$

From concrete cores, we found the concrete's average modulus of elasticity is 32.1 x 10^3 MPa (4.65 x 10^6 psi) and the lower bound (95 percent confidence) compressive strength of the concrete (f'_c) is 39 MPa (5,600 psi). Dividing the allowable stress by the modulus of elasticity, we get the allowable strain in the concrete:

$$\epsilon_c = \frac{(0.4)(39\ MPa)}{32.1\ x10^3\ MPa}(1x10^6 \mu\epsilon/\epsilon) \;=\; 486\ \mu\epsilon$$

In the following sections, ϵ_s and ϵ_c will be used to relate the measured strain to the allowable strain in the section.

Strain Measurements with Vibrating Wire Gages

Vibrating wire gages work well for long term static strain collection by electrically plucking a stretched wire and measuring its resonant frequency. Figure 7 shows three vibrating wire gages attached to a bridge beam. An electrical coil for plucking the stretched wire is attached to the flat part of the gage in the center to take a strain measurement.

Before construction started, we took several strain readings without any live load on the deck to establish an initial baseline that we could reference during the deck replacement. We also used these data to determine the accuracy of the gages and to see if the strain in the beams changed with temperature. From the data and prior experience with this type of gage, we feel the accuracy of strain measurements during this investigation is approximately 10 microstrain.

Deflection Measurements

Monitoring deflections is useful since it reflects upon the overall condition of the beam, whereas strain monitoring devices indicate the response of the beam in a discrete location. A Wild NA2000 electronic level was used to take elevation readings. Elevation pins were set so the readings could be taken at the same location each time.

Similar to our procedure for strain readings, we took several elevation readings before the construction work started to determine the accuracy of our equipment and to see if the bridge fluctuates from temperature effects and other environmental conditions. From these data, we feel the accuracy of deflection measurements for this investigation is approximately 6 mm (1/4 inch).

Temporary Support Loads

Figures 2 and 5 show the location of the temporary supports. Load cells (Figure 8), placed on top of the supports, were used to measure the loads transferred to the supports during the deck replacement. Because the load cells were placed on the temporary supports as the contractor erected them, we were not able to take readings prior to the construction. From calibration tests in the lab, we feel the load cells are accurate to 500 Newtons (100 pounds).

Bridge Response During the 1995 Construction Season

In 1995, we monitored strain, deflection, and temporary support loads, as well as a vertical/diagonal crack in the fascia beam. We monitored the second beam from northwest fascia (beam G2) throughout the deck removal and placement of the new deck. Table 6 shows the progression of the construction work and monitoring dates. We did not see a definitive relationship between strain readings and temperature.

Strain Measurements with Vibrating Wire Gages

We placed vibrating wire gages in three locations, shown in Figures 9 and 10. The following sections discuss the placement of the gages at each location and the test results.

Location 1

Location 1 was chosen to observe change in strain in the negative moment region next to pier 1 and temporary support 1. As shown in Figure 11, three gages where placed at set depths of the beam stem in an attempt to determine the strain gradient in the beam section. We had hoped that data from each gage could be used to estimate the strain in the critical locations of the section; i.e., in the tensile reinforcement located in the deck and in the concrete along the bottom of the beam. Unfortunately, Bernoulli's hypothesis that *plane sections before bending remain plane and perpendicular to the neutral axis after bending* was not valid for this section, since there are stress concentrations caused by the pier support bearing load and the temporary support load along with stress concentrations caused by the cracked section. As a result, our test data do not show a linear relationship and cannot be used to estimate the strain in the steel reinforcement placed in the deck. Figures 12, 13, and 14 show the strain readings recorded in vibrating wire gages 1, 2, and 3, respectively, during the deck replacement. The strain values are shown as the divergence from the average of the initial readings. All graphs are shown with the same scale for comparison.

Results

As shown in the methodology, the allowable compressive strain in the concrete is 486 micro strain, and the allowable tensile strain in the steel is 700 microstrain. When the deck was removed, 25 percent of the allowable strain in the beam was still being used by the remaining dead load of the beam, thus leaving a reserve capacity equal to 75 percent or 0.75(700) = 530 microstrain in the tensile reinforcement, and 0.75(486) = 360 compressive microstrain in the concrete.

Vibrating wire gage 1 (Figure 11), located at the bottom of the beam in a negative moment region, was expected to have a compressive strain from the dead load. By analysis, we expected that when the old deck was removed, the strain readings would increase 40 microstrain, indicating reduced compression. When the new deck was placed, we estimated the strain readings would decrease 20 microstrain (increased compression) for a net strain change of +20 microstrain (reduced compression).

Figure 12 shows the compressive strain relaxed 50 microstrain as the old deck, barrier and sidewalk were removed. The compressive strain increased very little as the new deck was cast. We found the largest variations of strain (range of 40 microstrain) occurred when construction equipment was on the deck, as shown in Figures 15 and 16, although none of the loads caused compressive strains greater than the initial readings. Comparing the measured strain to the allowable and the expected, we found it was close to what we expected, and this is a very small fraction of the beam's reserve capacity; i.e., 50 microstrain is only 12 percent of the beam's reserve capacity for live load compression.

Gages 2 and 3 (Figure 13 and 14) measured negligible strain variation during the deck replacement. This shows there is an extended area of zero strain near the center of the beam.

Location 2

Location 2 is shown in Figure 9. Two vibrating wire gages were placed on the beam stem, as shown in Figure 10 (Detail B) and Figure 17. Gages were placed here to observe strain at the maximum positive moment region (center span). The strain values, shown in Figures 18 and 19, are the divergence from the average of the initial readings.

Results

As expected, gage 4 showed the most activity since it was located on the bottom of the beam at the center of span 2, which is the extreme tension fiber in the maximum positive moment region. The allowable tensile strain in the steel is 700 microstrain. When the deck was removed, the dead load of the beam was still using 18 percent of the allowable strain, thus, the reserve capacity was 82 percent or (.82)700 = 574 microstrain in the tensile reinforcement. By our analysis, we expected that when the deck was removed the strain would decrease by approximately 76 microstrain, and when the new deck was cast, the strain in this gage would return to its initial value as shown in Figure 18.

When the deck was removed, the tensile strain in gage 4 decreased approximately 70 microstrain, which is very close to what we expected. When the contractor was forming the deck, the average strain measurements dropped another 70 microstrain and varied considerably, the range being 120 microstrain during the deck removal and forming the new deck. We feel the strain gage became loose or it was knocked into causing the apparent strain reduction when the new deck was being formed.

The strain variations seen during deck removal and forming the new deck may have been caused by construction equipment being on various locations on the bridge deck. For example, when construction equipment is on spans 1 or 3, strain in gage 4 will decrease, and when the equipment is in span 2 the strain values will increase.

When the new deck was cast, the strain values in gage 4 were approximately 105 microstrain lower than the initial values. We expected the values to return to the initial readings (shown as zero in Figure 18).

Although the strains in gage 4 deviated from what we expected, all readings were less than the initial strain value and the maximum range of the data are a small fraction of the reserve capacity of the beam.

The readings of vibrating wire gage 5 were expected to change very little during the deck replacement since it was located near the section's neutral axis. Figure 19 shows the readings remained fairly constant throughout the construction with some reduction of tensile strain during forming of the new deck. When the new deck was cast, the final strain in the gage was approximately 30 microstrain lower than the initial value.

Location 3

Location 3 was chosen to observe the strain in the negative moment region near pier 2. Three gages where placed at set depths of the beam stem as shown in Figures 9 (Detail C) and 20. Unfortunately, the concrete beneath gage 6 spalled, causing the data to be invalid; therefore, the graph is not shown. We discovered that gage 8, which showed significant drift, was not tightened properly during installation; therefore, the graph for this gage is also not shown. Gage 7, located near the neutral axis, showed little activity, as expected (Figure 21).

Results

As indicated above, the results of location 3 are inconclusive. The readings taken for gages 6 and 8 are invalid, and the graph for gage 7 shows little activity as expected.

Deflections

We placed elevation pins in the deck over the fascia beam (beam G1) and the first interior beam (beam G2) to monitor deflections at the center of spans 2 and 3, as shown in Figure 22.

Figures 23 through 26 show the deflections of the beams as the divergence from the average of the initial readings. All graphs are shown with the same scale for comparison.

We had several problems with our elevation measurements. Often measurements were difficult to take because the construction equipment interfered with our line-of-sight. On some days we could read the rod electronically and on other days we had to read the rod visually. Reading the rod electronically yields more accurate data. The elevation pins were set below the surface of the original deck so they would not be lost during construction. This worked well for saving the points, but it also made it difficult to align the survey rod on the point. On September 9, 1995, we had to relocate point 1 to avoid construction activity.

Results

Deflections for points 2, 3, and 4, presented in Figures 24 through 26, show the bridge's final dead load deflection is very close to its original deflection. Conversely, data for point 1, presented in Figure 23, show approximately 15 mm (0.6 inch) final deflection. A deflection of this amount in the beam would be accompanied by large strains. Since our data did not reveal large strains, and there was no sign of distress in the beam, we believe point 1 was disturbed during construction before we could transfer the point.

Temporary Support Loads

According to the design plans, the supports were designed for 220 kN (50 kips) vertical girder load. Figures 27 and 28 show the temporary support loads we recorded using the load cells. For comparison, the design load for the temporary supports is shown. Load cell 1 had a maximum load equal to 59 kN (13.3 kips) or 27 percent of the design load. Load cell 2 measured a maximum load of 29 kN (6.5 kips) or 13 percent of the design load. We compared the load increase in the temporary supports during the new deck pour. We found that the readings in load cell 1 increased by 20 kN (4.5 kips) or 9 percent of the design load, and the load in cell 2 increased by 9 kN (2 kips) or 4 percent of the design load.

Crack Measurements

During a site visit, the designers became concerned with a vertical/diagonal crack in the fascia beam about 7.6 meters (25 feet) from pier 2 in span 2 (shown in Figure 29). The Structural Research Unit was asked to investigate and monitor this crack. A close inspection revealed rust stains and efflorescence, indicating the crack had been there for some time. Using pins mounted on each side of the crack, we monitored its width and found no change during the construction.

Bridge Response During the 1996 Construction Season

From our analysis of the data we collected during the 1995 construction season and from the designer's original calculations showing that the temporary supports were not needed, we concluded the supports would not be required for the second half of construction (1996 construction season). We also believed the beams were not over stressed and they had minimal *locked-in* stress due to the construction. Unfortunately, we were not able to directly monitor the stress in the negative moment steel over the piers. However, the construction of the opposing side of the bridge provided us with another opportunity to monitor the negative moment steel over the piers, compare strains in the beams without the supports, and learn more about how beams react to unloading and reloading. For these reasons, our research continued into the 1996 construction. Table 7 shows the calendar of events for 1996 and monitoring dates.

Strain Measurements with Vibrating Wire Gages

Once again, vibrating wire strain gages were used. We placed gages at the bottom of the beam in the positive and negative moment regions of the bridge, as shown in Figure 30. Gage 9 was placed at location 4, and gage 10 was placed at location 5. We monitored the gages throughout the deck replacement. We also placed two gages on the deck reinforcing steel over pier 3 (location 6) to measure strain in the negative moment region as the bridge was loaded with a 47,000 kg (104,000 pounds) crane.

Location 4

Location 4 was on the bottom of beam G5 near the center of span 3. Similar to location 2, this area was chosen to observe the strain at the maximum positive moment region. Two vibrating wire gages were used at this location (Figure 31), but only one of these gave reliable readings. This gage is labeled gage 9. Figure 32 shows a section view of the beam with the vibrating wire gage. The strain values shown in Figure 33 are the divergence from the initial reading.

Results

As expected, the strain readings in gage 9 decreased as the old deck was removed, and the readings increased as the new deck was poured. When the old deck was removed, the strain readings were slightly less than calculated values. Strain readings did not vary as the new deck was formed as they did in gage 4 during the 1995 construction season. After the new deck was cast, the final strain in the beam was 40 microstrain less than the initial value. The strain readings in this gage compared fairly well to those in gage 4 (1995 data) and similar to gage 4, all strain readings were a small fraction of the beam's reserve capacity.

Location 5

Location 5 was chosen to observe strain in the negative moment region near pier 3, which is similar to location 1. Since gage 10 was placed near the bottom of beam G5 (shown in Figures 34 and 35), it was expected that the strain readings would increase (decreasing compression) as the old deck was removed and the readings would decrease (increased compression) as the new deck was poured.

Results

As shown in Figure 36, the strain readings in gage 10 changed only slightly when the old deck was removed. When the new deck was poured, we took several readings. We saw a 100 microstrain increase of strain as the deck was cast. The final strain readings showed a 40 microstrain increase in compression from the initial readings. Similar to gage 1, all strain readings were a small fraction of the beam's reserve capacity.

Location 6

One of the goals of this study was to record the strain readings in the negative moment deck reinforcement over the piers. We could not monitor strains here during the deck removal and replacement because the gage would interfere with the construction activity. Therefore, to compare actual strain in this location, we installed two strain gages on the deck, and we monitored these gages as the bridge was loaded with a crane. Gage 11 was placed directly over pier 3, and gage 12 was placed over the first diaphragm in span 4, about 5 meters (16.5 feet) from the center of the pier 3. The locations of the gages are shown in Figure 30, and pictures of each are shown in Figures 37 and 38. While we performed this test, the deck was removed in span 4. We began the test by taking initial strain readings in the two gages and then with a 47,000 kg (104,000 pounds) crane parked at the center of span 3, as shown in Figure 39. The crane was driven off the bridge and we took another set of strain readings. We recorded a change of 62 microstrain in gage 11 and 25 microstrain in gage 12. By analysis, we estimated gage 11 should have changed 79 microstrain (comparing very well) during the test, and gage 12 should have changed 42 microstrain (the actual strain was less).

Conclusions

The project objectives, as defined at the beginning of the project, are shown in italics and are followed by the respective conclusions.

1. *Monitor the bridge beams during the deck replacement to determine if the beams are over stressed at any time because of the construction activity. Of particular interest are the negative moment regions over piers 1 and 3, where removing the deck reduces the tensile reinforcement from fourteen to six 32 mm (1-1/4 inches) square bars, a 57 percent decrease in reinforcement.*

The vibrating wire gage data show the beam was not over stressed at any time during the deck replacement. In fact, the strain relieved in the beam when the original deck, sidewalk and parapet rail were removed was usually greater than any strains resulting from the applied loads; i.e., construction equipment, false decking, forms, or the new deck pour. The beam greatest response was when construction equipment was on the deck, and the response was less than expected when the new deck was cast. The design calculations showed the beam had adequate reinforcement in the deck over the piers during the deck replacement. We verified this when the beam responded as estimated when the bridge was loaded with a 47,000 kg (104,000 pounds) crane.

The deflection data, although lacking sufficient resolution for a detailed analysis, corroborated the vibrating wire gages, showing the greatest response of the beam occurred when construction equipment was on the deck.

Our investigation of a suspect crack in the fascia beam near pier 2 showed the crack was not a new crack, and its width did not change during the construction activity.

In Alberta, Canada, a continuous span, haunched T-beam bridge, very similar to Michigan's, was load tested to failure by Scanlon[2]. The structure was successfully loaded well beyond the elastic limit. Scanlon found *the critical-moment section exhibited a high degree of ductility and appeared to have developed the ultimate tensile strength of the reinforcement before failure of the section by crushing of concrete at the compression face*. During their test, the structure reached an ultimate capacity of 1,900 kN (427 kips), a load well above any expected to cross a highway structure.

2. *Compare the dead load stress level in the beams before and after construction to determine if any locked-in stress resulted from the construction sequence, which would reduce the bridge's live load carrying capacity.*

Our measurements show minimal locked-in stress occurred in the beam from the deck replacement. In each of the vibrating wire gages, the at-rest stress level in the beams is less with the new deck than the old. Also, the final elevations for points 2, 3, and 4 were very close to their initial elevations; these results were as expected. By analysis, we did not expect *lock-in* stress because the weight removed (old deck, sidewalk, and parapet rail) was only slightly less than the weight applied (new deck), and because the negative moment regions (over the piers) were cast prior to the positive moment regions (at center span) added additional support to the beams before the new deck was cast over the center spans.

3. *Monitor the temporary support loads during the deck replacement. The actual loads would be compared to the allowable load that each temporary support can carry. This information can be used to adjust the temporary support design or to determine whether the temporary supports are required on future projects of this type.*

Loads did not redistribute to the temporary supports as expected. We measured the maximum loads in temporary supports 1 and 2 to be 27 percent and 13 percent, respectively, of the design load. These loads occurred during deck removal. The maximum load in the temporary supports was expected to occur during the casting of the new deck. The measured values when the deck was poured were only 9 percent and 4 percent of the design load for temporary supports 1 and 2, respectively.

Conclusions from General Observations

The design plans originally called for the diaphragms between the beams to be replaced. This would have been a difficult and expensive item. During the deck replacement, we frequently inspected the diaphragms and found no signs of distress.

[2]Scanlon, A. And Mikhailovsky, L., "Full-Scale Test of Three-Span Concrete Highway Bridge', Canadian Journal of Civil Engineering, Vol. 14, No. 1, February 1987. Pp 19-23.

The diaphragms on this type of bridge add considerable stiffness and lateral support to the beams, and they distribute loads to the beams, even when the deck is removed.

With the diaphragms in place, the deck removal procedure involved cutting out rectangular portions of the deck. This required many intersecting cuts and considerable care by the contractor. In a few instances the contractor overextended the transverse saw-cut, thus cutting longitudinal reinforcement in the bridge deck that was supposed to remain. This necessitated a welded splice of the cut rebar. Figure 40 shows a location with an over-extended saw cut.

Recommendations

Michigan's continuous haunched T-beam bridges are unique and asesthetically appealing structures that have proven themselves to be sturdy and long lasting. Designers should continue their efforts to develop procedures to rehabilitate this type of bridge. In this study, and in past studies by others, this bridge has performed well in load tests. Therefore, designers do not need to make special or conservative assumptions when calculating beam capacity. We believe well-established analysis procedures will adequately predict beam response.

Although the temporary supports were not used during Phase 2 construction of the Partello Road bridge, this does not mean they will not be needed on future projects. For the Partello Road bridge, the design calculations showed the supports were not necessary. This was verified by our field monitoring devices placed on the beams and the temporary supports. The decision to use temporary supports should continue to be made by an experienced bridge design engineer.

TABLES