A Thesis
titled
SuperLoad Crossing of Millard Avenue Bridges Over Duck Creek and CSX Railroad
by
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Submitted to the Graduate Faculty as partial fulfillment of the requirements for the
Master of Science Degree in Civil Engineering

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The University of Toledo
December 2012
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An Abstract of

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A series of superloads crossed the Millard Avenue Bridges which span over Duck Creek and the CSX railroad. These bridges are located in Toledo and Oregon, Ohio, on the route from the Port of Toledo to a nearby oil refinery. The bridge over Duck Creek is a three span straight slab-on-steel girder bridge and the CSX is a horizontally curved slab-on-steel girder bridge having five continuous spans. The horizontal curve of the steel super structure is achieved by kinking the girder lines at the field splices.

The tasks requested to be performed by the Cities of Toledo and Oregon were load tests and load ratings to verify that twelve super loads could safely traverse the bridges, determine if any measurable damage was caused by the heavy loads and establish load rating procedures for future superloads.

The work presented in this thesis is the comparison of the analytical and field strain measurements for the largest and last superload to cross the bridge, the comparison of the field strain measurements for five diagnostic dump truck tests the development of a calibrated finite element model for the Duck Creek bridge. An examination of the lateral bending strain at the splices for the CSX bridge.
The field and analytical strain fit well for the superload crossing. The diagnostic tests showed no observable change before and after the superload crossing.
For My Parents
Acknowledgements

I thank my advisor, Dr. Nims, for giving me the support, and the guidance. I have learned a lot from him, and it was a great opportunity to do my research under his supervision. I gratefully acknowledge Dr. Victor Hunt for his generous help in this research. I am also thankful to Mr. Howard Blanc and the personnel of Burkhalter Rigging, Mr. Raj Huria formerly of the City of Toledo, Ohio and Mr. Paul Roman of the City of Oregon, Ohio for their assistance in this research. I appreciate the reviews of this thesis by Dr. Frederick and Dr. Randolph. I am grateful to Ms. Christine Lonsway of the University of Toledo, Transportation Center and Mr. Brice Carpenter of Bridge Diagnostics, Inc for their help.

Finally, a special appreciation and thanks to my parents who supported me and encouraged me all the time throughout my Masters Degree Program.
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Chapter 1

Introduction

1.1 Purpose

It was desired to have twelve extremely large loads, referred to as superloads, cross the Millard Ave bridges in Toledo and Oregon Ohio. The goals of the project were to confirm if the bridges could safely carry the superloads, estimate if the superload sequence caused any cumulative damage to the bridges and to develop load rating procedures for future superloads.

1.2 Background

The Cities of Toledo and Oregon, Ohio approached the University of Toledo about investigating the adequacy of the two bridges on Millard Ave. The first bridge is over Duck Creek. It is a three-span, straight steel-girder bridge with a composite deck that carries four lanes, the overall width is approximately 55’-0” (52’-0” roadway width) and the bridge length is approximately 231 ft. with span lengths varying from 71 ft. to 89 ft. The second bridge is over the CSX railroad. It is a five-span, horizontally curved bridge that carries four lanes of Millard Avenue over the CSX railroad interchange. The superstructure consists of seven steel plate girders that are composite with a concrete deck. The overall width is approximately 67’-0” (48’-0” roadway width) and the bridge
length is approximately 650 ft. with span lengths varying from 114 ft. to 152 ft. The
superstructure’s curved geometry was generated in an interesting fashion in that the
girders were essentially kinked at the field splices, which were located near the dead load
flexural inflection points (see figure 1-1). Table 1.1 lists the loads that were proposed to
cross the bridges.

![Image](image.jpg)

**Figure 1-1: Detail of Kinks in the Horizontally Curved Bridge over the CSX**
### Table 1.1: Superloads Series (BDI 2011 A)

<table>
<thead>
<tr>
<th>LOADING</th>
<th>Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Platforming Reactor</td>
<td>728,527</td>
</tr>
<tr>
<td>2 Regenerating Module C</td>
<td>582,579</td>
</tr>
<tr>
<td>3 Regenerating Module B</td>
<td>461,228</td>
</tr>
<tr>
<td>4 Regenerating Module D</td>
<td>575,745</td>
</tr>
<tr>
<td>5 Packinox Exchanger</td>
<td>543,239</td>
</tr>
<tr>
<td>6 Regenerating Module A - No Dollies*</td>
<td>898,170</td>
</tr>
<tr>
<td>7 Lower Stack Section</td>
<td>302,318</td>
</tr>
<tr>
<td>8 Upper Stack Section</td>
<td>328,774</td>
</tr>
<tr>
<td>9 Bottom Convection Module</td>
<td>570,706</td>
</tr>
<tr>
<td>10 Reduction Zone</td>
<td>253,816</td>
</tr>
<tr>
<td>11 Middle Convection Module</td>
<td>658,131</td>
</tr>
<tr>
<td>12 Top Convection Module</td>
<td>745,880</td>
</tr>
</tbody>
</table>

* It was elected that (6) Regenerating Module A not cross the bridges.

The team assembled to work on this project included members from the University of Toledo Department of Civil Engineering, the University of Cincinnati Infrastructure Institute (UCII) and Bridge Diagnostics, Inc. (BDI). BDI led Mobilization 1 and Mobilization 2 and introduced The University of Toledo to their analysis software WinGen, WinGrf, and WinSac. The software has been used to produce a calibrated finite element analysis models and match with the data obtained from the load tests.

Mobilization 3 test data reduction and application of BDI’s calibrated finite element model have been done by the University of Toledo assisted by BDI. The University of Toledo used BDI model (BDI 2011A), assigned the superload to the bridges models, and performed the analysis to predict the bridge behavior for Mobilization 3. Also, they added the dump truck scaled data of Mobilization 3 to the comparison of Mobilization 1.
and 2 dump trucks. The bridge behavior used for comparison was calculated from a modified version of the final calibrated model, the rating model that has been done by BDI.

1.3 General Description of the Mobilizations

The work was conducted in three mobilizations. Mobilization one took place in June 2011. Dump trucks of known weight were used to conduct tests to obtain data to develop the calibrated finite element models of the bridges.

In mobilization one, 56 strain transducers were installed on the Duck Creek Bridge, and 114 reusable surface-mounted strain transducers, 3 tiltmeters, and 7 displacement sensors were installed on the CSX bridge. The sensors were installed in regions of expected high tensile and compressive bending stress, over the piers and near the kinked field splices.

The load used for mobilization one was two dump trucks. The dump truck weights were obtained from certified scales at a local gravel pit, and all vehicle dimensions were measured in the field at the time of testing. Dump trucks moved along paths (Figure 1-2) Y1 to Y6. A single dump truck was moved along each path at a crawl speed. Except path Y3, down the center of the bridge, for which the two dump trucks moved side by side along the center of the bridge as shown in Figure 1-2.

The CSX bridge model was built using AutoCAD and imported to BDI software WinGin. The CSX model is a 3D finite element model. The Duck Creek model was a finite element model with an adjusted girder. The based on the diagnostic load tests BDI developed several calibrated bridge models. BDI calibrated 3-D models were then used to
load rate the bridge for the proposed superloads and determine if these loads could safely cross the bridge (BDI 2011 A).

Figure 1-2: Details of the Dump Trucks paths (BDI 2011 A). Where D is the driver side, and P is the passenger side.

In September 2011, Mobilization 2 occurred, where the first measured superload, the Platforming Reactor, crossed the bridges. Prior to the superload crossing two diagnostic tests were done with two dump trucks of the known weight. The data were collected while the two dump trucks move along the center of the bridge. The platforming reactor was the second largest load to cross the bridges. Sixteen strain transducers were installed on the Duck Creek Bridge, and 48 strain transducers were installed on the CSX Bridge. First, data was collected while two dump trucks traveled side-by-side on the center of the bridges. Second, data was collected during the crossing of the platforming reactor. This
superload weighed 729 kips. Finally, after the superload crossing, the pre-crossing tests with two dump trucks were repeated. Reviewing the results of the load rating and mobilization 2, it was decided by others that the largest superload should not cross the bridges.

In October 2011. Mobilization 3 occurred, the second measured superload, the Top Convention module, crossed the bridges. It was the last and heaviest of the eleven superloads to cross the bridge. The bridge was monitored and measurements made to assess the cumulative damage caused by the superload sequence. The instrumentation was the same as Mobilization 2. First, as in mobilization two, data was collect while two dump trucks drove down the centerline of the bridges. Second, data was collected while the superload, the Top Convention Module, crossed the bridges. Finally, the calibration tests done in Mobilization 1 were repeated.

Thus, the behavior of the bridges were measured at the beginning and end of the superload sequence, under the two largest superloads and five sets of comparable dump truck data to assess the changes in the bridge condition were available. The goodness of fit of the calibrated model for the first and last superload transits and absence of changes in behavior in the dump truck data sets means that the bridges behaved elastically under the superloads and the was no observable cumulative damage.

1.4 Predicted and Measured Response Comparison for the Dump Trucks in Mobilization 1

The gages in Figures 1-2 to 1-4 are the reading of the dump trucks crossing the bridges on path Y1, and Y3, where Y1 located at the east side of the bridge, and Y3 is located at the center of the bridge. These data are for mobilization one, where the solid lines are the field data and the dashed lines are the calibrated finite element model. The Figures
illustrate the fit of the calibrated model to the dump truck data for Mobilization 1 (BDI 2011A).

Figure 1-2: Final Model – Stress Comparison Plot – Abutment – Paths Y1-Y3. (BDI 2011 A)
Figure 1.3 Final Model – Stress Comparison Plot – Piers – Paths Y1-Y3. (BDI 2011 A)

Figure 1.4 Final Model – Displacement Comparison Plot – Piers – Paths Y1-Y3 (BDI 2011 A)
1.5 Work Presented and Thesis Organization:

Chapter 2: Comparison of the Duck Creek results from the collected field data from Mobilization 3 and the analytical results predicted from the calibrated finite element model.

Chapter 3: Comparison of the CSX results from the collected field data from Mobilization 3, the analytical results predicted from the calibrated finite element model.

Chapter 4: Describes the behavior of the beams adjacent to the girder kinks.

Chapter 5: Describes the steps carried out by the author in creating an independent model of the Duck Creek Bridge using the BDI software.

Chapter 6: Conclusions
Mobilization 3 Data Reduction – Duck Creek

On October 28th of 2011, the heaviest permit load to cross the Duck Creek Bridge was transported from the Port of Toledo to the BP-Husky Refinery. This superload was designated as the Top Convection Module, had a gross weight of approximately 745 kips and was the final load in the superload sequence. The data from the superload crossing were examined by two primary methods. First, the responses of the superload crossing were matched with predictions obtained from the calibrated model to confirm that the structure performed as estimated. Overall, the model produced acceptable results and the variation in structural stiffness or applied load was relatively minor.

Second, the bridge was also examined by comparing the responses from five dump truck tests. the second and the third were the pre and post to the superload of Mobilization 2, and fourth and fifth were the pre and post to the superload of Mobilization 3. These data sets were examined for reproducibility and visual checks were to detect differences in response behavior at every gage location. Comparisons from every sensor indicated that the bridge performance before and after the superload crossing was almost identical. Figure 2-1 shows the top convection module approaching the Duck Creek Bridge.
Figure 2-1: Monitoring of the TCM Crossing.

Figure 2-2 shows the strain transducer locations on the Duck Creek Bridge for Mobilizations 2 and 3. The gages are arranged to measure the strains at the integral abutment, the gages in the middle of span 1 are to measure the strains due to the positive moment there and the gages at the pier are used to measure the strains due to the negative moment at the pier. This captures the general behavior of the bridge and is sufficient to verify the performance of the model as well as capture the strains in the bridge area which governed the load rating. All seven girders are instrumented at midspan to confirm that the expected symmetry is actually observed.

The same gage layout was used for Mobilizations 2 and 3. However, because the gages where accessible from the ground they removed immediately after Mobilization 2,
but the anchors were left in place. The gages were reinstalled the day before the TCM crossed the bridge, but some anchors had to be reinstalled. Therefore, the gage numbers on the Duck Creek Bridge are different between the two mobilizations and some locations are very slightly different.
Figure 2-2: Mobilization 3 Instrumentation Plan and Load Configuration. (BDI 2011 A)
Figure 2-3: Instrumentation Section (BDI 2011 A)
Figure 2-4: Top Convection Model (BDI 2011 A)
2.1 Predicted and Measured Response Comparison for the Top Convection Module

The following data plots show the analysis predictions from the “composite rating model” developed by BDI and used by the author, which was slightly more conservative than the original field calibrated model. This composite rating model was an envelope that was used for computation of all load rating factors and the response predictions for the Top Convection Module Superload crossings. This model assumed composite behavior over the pier, but had a reduction in abutment rotational spring stiffness used to simulate the effect of the integral abutments. Computed responses are shown by the markers and the measured responses are shown by the continuous lines.

Figure 2-5 to 2-16 are plotted to the same scale and have been scaled based on the largest strain value. The entire suite of figures conveys the overall fit of the model and the strain data.

The following plots show the comparison between predicted and actual response for the TCM crossing. On the west end figures 2-5 and 2-7 are at the integral abutment. Figures 2-8 to 2-13 present the strain at the midspan and the gages reading are symmetric. Figures 14 and 15 are at the pier, Also, they are showing the comparison between the field and model results for the top and bottom gages in the girders, where the blue is the top gages and the green is the bottom gages. The figures show a good match between the calibrated model and the field data, which means that the model developed for the low level load of the dump trucks has a good fit for the superload and the bridges behaved linearly elastically.
Figure 2-5: Duck Creek Strain History at the Abutment Location 1-C (Abutment Girder C)
Blue is top flange and green is the bottom flange. The moments occur because the gages are away from the bearing and the restraint at the bearing. This good fit means the boundary conditions are adequately modeled.
Figure 2-7: Duck Creek Strain History at Location 1-E (Abutment Girder E)
This gage data are low because they are in the outer girder and the TCM is passing down the center of the bridge. This indicates the magnitude of the lateral load distribution effects.
Figure 2-9: Duck Creek Strain History at Location 2-B (Midspan Girder B)

This gage model data are over predicting the strains.
Figure 2-10: Duck Creek Strain History at Location 2-C (Midspan Girder C)

This gage is near the center of the bridge and the test and analysis matchs better than for the gages farther from the center of the bridge.
Figure 2-11: Duck Creek Strain History at Location 2-D (Midspan Girder D) Gage B3001 Top & 3015 Bottom.

Figure 2-11 presents strains from the top and bottom gages in the middle girder. These are the highest predicted and measured strains. The blue line and symbols are tension at the bottom. The green line and symbols are the strain in the upper flange. The upper flange is near the neutral axis and the stresses are low.
Figure 2-12: Duck Creek Strain History at Location 2-E (Midspan Girder E)

The gages on girder C and E show similar fits confirming the symmetry of the model. The load was at the center on the bridge.
Figure 2-13: Duck Creek Strain History at Location 2-G (Midspan Girder G)
Figure 2-14 shows strain data from the top and bottom gages at the pier. Gage B2994 (blue) is on the top flange and gage (B3030) is on the bottom flange. With no composite action, the strain in the top and bottom flange would be closer to equal (this could be estimated this using the non-composite model or by just considering the area of the tributary rebar). The closeness of fit of this model indicates the extent of composite action captured in the model. AASHTO code does not allow consideration of composite action in negative moment regions, but clearly this beam has it.
Figure 2-15: Duck Creek Strain History at Location 3-D (Pier Girder D) B3023 Top & B3014 Bottom

As in figure 2-14 this reflects the effect of composite action.
Figures 2-16 to 2-18 shows the lateral load distribution, where Figure 2-16 is at the Piers, Figure 2-17 is at the abutment, and Figure 2-18 is at the midspan.

**Figure 2-16: Pier Superload Lateral Load Distribution**
Figure 2-17 shows the match of the test data and model data for gages B3030, B2994, B3014, and B3023, where gages are at the abutment.

Figure 2-17: Abutment Superload Lateral Load Distribution
Figure 2-18: Duck Creek Midspan Superload Lateral Load Distribution
2.2 Comparison of Truck Tests

In order to detect the change of the structure performance and verify instrument performance, tests with two fully loaded dump trucks were run just prior to and immediately after the superload crossing. These tests were each run twice in both cases to ensure reproducibility and to provide a data quality check. Five set of dump truck tests were performed for both bridges and for the three mobilizations as mentioned previously in section 2.1.

During the initial load test on June 27th and the first superload test, similar series of tests were run with two dump trucks. Figures 2-19 to 2-21, show the test data for the top and bottom gages on girder D throughout the bridge. For Figure 2-19, gage B3003 is the top gage and B2987 is the bottom gage. For Figure 2-20, gage B 3015 is the top gage and B 3001 is the bottom gage. For Figure 2-21 gage B3023 is the top gage and B3014 is the bottom gage. Note that in Mobilization 3 the pretest for the dump truck at the center of the bridge has been done twice and the post test at the center of the bridge was done only once; therefore there are two sets of data from the pretest are compared to one set of data from the post test. Plots 2-22 and 2-23 show the dump truck test data for Mobilizations 1, 2, and 3. A basic conclusion that could be made from these tests is that there was no noticeable change in the structure’s measured response.
Figure 2-19: Pre/Post- Double Dump Truck Tests at Girder D Section 1-1 (Abutment).

This figure shows the repeatability of the dump truck data before and after Mobilization 3 at the Abutment.
Figure 2-20: Pre/Post- Double Dump Truck Tests at Girder D Section 2-2 (midspan).

This figure shows the repeatability of the dump truck data before and after Mobilization 3 at the midspan.
Figure 2-21: Pre/Post- Double Dump Truck Tests at Girder D Section 3-3 (near piers)

As shown in Figures 2-22 and 2-23 the test data are repeatable for Mobilization 3
Figure 2-22: Middle Span Dump Truck Comparison for all Mobilizations

Figure 2-22 is for the pre and post dump truck crossings the Duck Creek Bridge at the Midspan. It is for two gages for the three Mobilizations.
Figure 2-23: Pier 1 Dump Truck Comparison for all Mobilization

Figure 2-23 is for the pre and post dump truck crossings the Duck Creek Bridge at the Piers. It is for two gages for the three Mobilizations.
Chapter 3

Mobilization 3 Summary CSX Railroad Bridge

When the Top Convention Model crossed the CSX Bridge, the field measurements from the superload crossing were examined by two methods. First, the measured strains were compared with calibrated model estimates. The models produced good results when compared with the measured responses. This means that the structure’s response behavior was very linear with respect to load magnitude and that the finite element model can realistically predict responses up throughout the load range. Two minor differences were observed in the data comparisons. The measured diaphragm stresses and the measured lateral flexure stresses in the girder’s bottom flange were significantly less than predicted. Physically this is acceptable because the reduced tension on the staggered diagonals reduced the lateral stress near the girder field splices.

The other method of data analysis was to compare responses from two fully loaded dump trucks that crossed prior to the superload, and for a second time after the superload had crossed the CSX Bridge. These data sets were simply examined for reproducibility and visual checks were made to detect differences in response behavior at every gage location. The data set were almost identical.
At the piers, the vertical stress was relatively low. The highest compressive stress induced by the Top Convection Module was approximately 4 ksi. This suggests that the bearing stiffener design should not be the controlling factor in the load ratings.

Figure 3-1: Monitoring of the Reactor Module Crossing at Millard Avenue over CSX Railroad – Oregon, OH (BDI 2011B)
Figure 3-2: Monitoring of the Reactor Module Crossing at Millard Avenue over CSX Railroad – Oregon, OH (BDI 2011 B)
Figure 3-3: Instrumentation Plan for the CSX Bridge for Mobilization 3

This is a half plan. All the instrumentation for this Mobilization was in this half of the bridge.
SECTION 1-1

A  B  C  D  E  F  G
B3012 (13)  B3011 (14)  B3038 (15)  B3040 (16)  B3037 (20)  B3039 (18)  B3036 (17)

SECTION 2-2

A  B  C  D  E  F  G
B3002 (12)  B3022 (11)  B3033 (9)  B3008 (10)

SECTION 3-3

A  B  C  D  E  F  G
B3043 (35)  B3009 (36)  B2992 (33)  B3025 (21)  B2990 (22)  B3044 (23)  B3045 (24)

Figure 3-4: Instrumentation Sections (BDI 2011 B)
Figure 3-5: Instrumentation Sections (BDI 2011 B)
3.1 Predicted and Measured Response Comparison

The following data plots show the analysis predictions from the “composite rating model” developed by BDI and used by the author, which was slightly more conservative than the original field calibrated model. The rating model was used for computation of all load rating factors and the previous response predictions for the Top Convection Module Superload crossings. This model assumed full composite behavior over the piers. Computed responses are shown by the markers and the measured responses are shown by the continuous lines.

Figure 3-6 to 3-28 are plotted to the same scale and have been scaled based on the largest strain value. The entire suite of figures conveys the overall fit of the model and the strain data.

The following plots show the comparison between predicted and actual response for the TCM crossing. On the west end, Figures 3-6 and 3-12 present the strain at the meddle of span 1, section 1-1 and the gages reading are symmetric. Figures 3-13 to 3-21 are at pier 1. Figures 3-22 to 3-24 present the strain at the midspan 2. Figures 26 to 28 are at pier 2. Also, they show the comparison between the field and model results for the top and bottom gages in the girders, where the blue line is the top gages and the green line is the bottom gages. The figures show a good match between the calibrated model and the field data, which means that the model, developed for low level load the dump trucks has a good fit for the superload and the bridges behaved linearly elastically.
Figure 3-6: CSX Stress History at Location 1-A-B (Midspan 1 Girder A)

This gage data are relatively low because this is an outer girder and the TCM is passing down the center of the bridge. This indicates the magnitude of the lateral load distribution effects.
Figure 3-7: CSX Strain History at Location 1-B-B (Midspan 1 Girder B)

Gages in Figures 3-6 and 3-7 for model indicate a good fit with field data. This indicates that the model is predicting the strain accurately. The sold blue lines are the field data and dashed blue lines are the model results.
Figure 3-8: CSX Stress History at Location 1-C-B (Midspan 1 Girder C)
Figure 3-9: CSX Stress History at Location 1-D (Midspan 1 Girder D)

Figure 3-9 is the data from the top and bottom gages in the middle girder. This girder has the highest predicted and measured strains. The blue line and symbols are tension at the bottom. The green line and symbols are the strain in the upper flange. The upper flange is near the neutral axis and the relatively low stress indicates the section is behaving compositely.
The data from the gages plotted in Figures 3-10, 3-11, and 3-12 indicate that the model is over predicting the strain.
Figure 3-11: CSX Stress History at Location 1-F-B (Middle of span 1 Girder F)

Comparing girders B and F, the field data are symmetrical.
Figure 3-12: CSX Stress History at Location 1-G-B (Midspan 1 Girder G)
Figure 3-13: CSX Stress History at Location 2-D (Pier 1 Girder D)

Data plotted in figure 3-13, 3-14, 3-18 and 3-28 are from the top and bottom gages at the pier. Gages B3002, and B3008 (blue) are on the top flange and gages (B3022, and B3033) are on the bottom flange. With no composite action, the strain in the top and bottom flange would be nearly equal. The closeness of fit of this model data indicates the influence of composite action.
Figure 3-14: CSX Stress History at Location 2-E (Pier 1 Girder E)
Figure 3-15: CSX Stress History at Location 3-A-B (Pier 1 Section 3 Girder A)

Figures 3-15 to Figure 3-17 are show a good fit of the model data and field data at the pier.
Figure 3-16: CSX Stress History at Location 3-B-B (Pier 1 Section 3 Girder B)
Figure 3-17: CSX Stress History at Location 3-C-B (Pier 1 Section 3 Girder C)
Figure 3-18: CSX Stress History at Location 3-D (Pier 1 Section 3 Girder D)
Figure 3-19: CSX Stress History at Location 3-E-B (Pier 1 Section 3 Girder E)

The gages in Figures 3-18 and 3-19 indicate the model is over predicting the strain at the pier.
Figure 3-20: CSX Stress History at Location 3-F-B (Pier 1 Section 3 Girder F)

Gages in Figures 3-20 to 3-26 indicate the model is providing a good fit with field data. This indicates that the model is satisfactory predicting the strain.
Figure 3-21: CSX Stress History at Location 3-G-B (Pier 1 Section 4 Girder C)
Figure 3-22: CSX Stress History at Location 4-C-B (Pier 1 Section 4 Girder G)
Figure 3-23: CSX Stress History at Location 4-D-B (Midspan 2 Girder D)
Figure 3-24: CSX Stress History at Location 4-E-B (Midspan 2 Girder E)
Figure 3-25: CSX Stress History at Location 4-F-B
Figure 3-26: CSX Stress History at Location 5-D-B (Pier 2 Section 5 Girder D)
Figure 3-27: CSX Stress History at Location 5-E-B (Pier 2 Section 5 Girder E)
The overall conclusion from monitoring the test data throughout the test sequence was that there was no measurable change in the superstructure stiffness associated with the superloads crossing the bridge.
3.2 Comparison of Pre and Post Dump Truck Tests

Two sets of data have been collected in Mobilization 3 for the dump truck. The first set of data is for prior to the superload crossing the bridge, where two dump trucks crossed the bridge side by side along the center of the bridge. The second set of data is for the same dump trucks crossing the bridge after the superload crossing. Traffic was stopped so that the test trucks the only live load applied. The plots show a good fit between the pre and post dump truck data. Note that plots of the strain gages on the upper flange frequently have spikes (i.e. figures. 3-28 and 3-29). The spikes are the result of local effects when the wheels of the load pass over the gage locations. The spikes do not reflect the major axis bending in the member. Five sets of dump truck tests were done for both bridges and for the three mobilizations as mentioned previously in section 2.1.
Figure 3-29: Pre/Post – Double Dump Truck Tests at Girder D Section 2-2.

This figure shows the repeatability of the dump truck pre/post data at the Piers 1.
Figure 3-30: Pre/Post – Double Dump Truck Tests at Girder D Section 6-6.

This figure shows the repeatability of the dump truck pre/post data at the Piers 2
Figure 3-31: Pre/Post – Double Dump Truck Tests at Girder D Section 1-1.

This figure shows the repeatability of the dump truck pre/post data at the Midspan 1
Figure 3-32: Pre/Post – Double Dump Truck Tests at Girder D Section 4-4.

This figure shows the repeatability of the dump truck pre/post data at the Midspan 2
The comparison plots (identify the three Mobilizations figures) above show the results for the Mobilization 1, Mobilization 2 post superload and Mobilization 3 post superload matched closely. Mobilization 2 and 3 pre-superload have also been reviewed and they are consistent with the post-superload plots. The stress results matched closely and the differences are within the bounds of reasonable experimental error approximately 5%; there was no trend in the behavior of the differences. Therefore, based on the review of this data, there is no sign of structural damage.
Figure 3-33: Middle Span 2 Dump Truck Comparison.
Figure 3-34: Pier 3 Dump Truck Comparison.
Chapter 4

Behavior of the Beams and Diaphragms Adjacent To the Girder Kinks

Because there was no more than general guidance in the literature for kinked beams, it was necessary to develop a procedure for considering the biaxial bending stresses in the girders and the forces in the braces near the kinks in the girder lines in the load rating of the bridge. BDI developed the basic procedure and it was reviewed by Dr. Nims and Dr. Hunt. The strain results from the gages at the diaphragm are presented and discussed here.

The lateral flexure effects of the girders at the kinks were localized whereas the code provided stress limitations based on the radius of curvature. Instead the rating procedure used by considered biaxial flexure from the combined effects of lateral and primary bending (BDI 2011B). This was done by using the axial and lateral flexure flange stresses from the FE model and a stress limit corresponding to the given section capacity to calculate the superstructure’s flexural load rating. Due to the structure’s composite behavior near midspan and the relatively short unbraced lengths of the compression flanges near the interior supports, the nominal flexural stress limit for most of the girder sections was equal to the yield stress, $F_y$. 
During load rating, the field splice capacity was checked by BDI (BDI 2011A) and compared to that of the girder capacity in order to ensure that the splices could develop the full strength of the girders. In all cases, the field splices were as strong as or stronger than the girder components. None of the girder sections immediately adjacent to the splices were found to be critical because the splices were located near the dead-load inflection points. Since the splices could develop the full strength of the girder sections and the girder sections were not critical, the splices were considered adequate and no further ratings were performed. This was done by BDI (BDI 2011B).

4.1 Mobilization 1

Detailed measurements were taken at three field splices in Mobilization 1. Girder G was instrumented adjacent to field splice FS1 and sensors were placed on girder F near FS1 and FS2. Girder G is the outermost girder and FS1 and 2 have the largest angles between the girders. Therefore, it was anticipated that the highest diaphragm forces and lateral flange bending would occur near these splices. The strain gages were placed near the inside and outside edges of the top and bottom flanges.

The flange lateral bending was examined for truck paths near the outside of the curve. The top flange was found to be just above the neutral axis and is embedded in the deck. Therefore, it showed low stresses and little evidence of flange bending. However, the bottom flanges did show lateral bending. Figure 3-1 illustrates the observed lateral flange bending for the bottom flange. Lateral stresses up to 33% of the average normal stress in the flange were observed during the Mobilization 1 testing.
4.2 Mobilizations 2 and 3

Because of the reduced number of channels in the later mobilizations, the determination during Mobilization 1 that the lateral bending in the top flange was not significant, and knowledge of the load path the superload would follow, only the bottom flange of the middle girder (Girder D) at FS2 was monitored for lateral flange bending during the superload crossings. Figures 4-2 show these strain gage locations. From investigating the super load FS2 at girder D is shown in Figure 4-2. The gages at the splices were designated inner gages and outer gages with inner gages, on the concave side of the curve.
Figure 4-2: FS2 Elevation and Section of the Strain Gages on the Beam Bottom Flange.

The sections A-A, B-B, C-C, and D-D have B3021, B3027, B3031, and B3028 for inner gages. B3034, B3016, B3004, B3026 are the corresponding outer gages.

Figure 4-3: FS2 Strain Gages on the Bottom Flange.

These strain gage locations (Figure 4-3) were selected to capture the behavior from one side of the splice to the region on the other side of the splice where two diaphragms connect to opposite sides of the beam with a slight offset. The maximum ratio of the observed lateral flange stress to the mean lower flange was found to be 32%. There was a
complete reversal in the lateral flexural stress between the staggered diaphragms on each side of the field splices. The analysis had significantly higher ratios at all locations with the highest being 113% at section D-D. However, this extreme was an anomaly. Figure 4-4 shows a typical comparison of the test and analytical stresses. The measured diaphragm stresses and the measured lateral flexure stresses in the girder’s bottom flange were significantly less than predicted. One likely cause of this discrepancy was the model assumptions of the rigid body connections between the diaphragms and the girders in this region. Another likely cause was that the superload was rigid enough to create better lateral distribution and shift the load away from the centerline of the bridge as the bridge deck slightly deformed (i.e., the superload was stiff enough to shift load away from the regions of larger deformation) (BDI 2011A). This possible redistribution effect of the superload had a positive influence on the overall performance because it reduced tension on the staggered diagonals and reduced the lateral stress near the girder field splices.
Figure 4-4: Analytical and Test Strain History for Lower Flange Gages at Section AA Mobilization 2 superload.
A stress comparison (Table 4.1) of the superload of the Mobilization 2 field test result and a comparison of the model results were done as shown in Figure 4-5 and Figure 4-6, respectively. The inner and the outer lateral stresses were also compared, as shown in Figure 4-7, to indicate the behavior of the lateral forces at the splices.

**Figure 4-5: Test Stresses for Mobilization 2 Superload**

**Figure 4-6: Model Stresses for Mobilization 2 Superload**
### Table 4.1: Model and Test Stresses Adjacent to FS2 for Superload-Mobilization 2

<table>
<thead>
<tr>
<th>Sections ordered west to east (Toledo end to Oregon end)</th>
<th>test stress (ksi)</th>
<th>model stress</th>
<th>test stress</th>
<th>model stress</th>
<th>Test Ave stress</th>
<th>Model Ave Stress</th>
<th>Test Lateral stress</th>
<th>Model Lateral stress</th>
<th>Test lat/ave</th>
<th>Model lat/ave</th>
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<tbody>
<tr>
<td>Bottom flange</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Test Ave stress</td>
<td>Model Ave Stress</td>
<td>Test Lateral stress</td>
<td>Model Lateral stress</td>
<td>Test lat/ave</td>
<td>Model lat/ave</td>
</tr>
<tr>
<td></td>
<td>Inner</td>
<td>Outer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Section AA</td>
<td>B3021</td>
<td>6.8</td>
<td>B3034</td>
<td>4.4</td>
<td>2.0</td>
<td>5.6</td>
<td>6.4</td>
<td>1.2</td>
<td>3.2</td>
<td>21%</td>
</tr>
<tr>
<td>2 Section BB</td>
<td>B3027</td>
<td>5.9</td>
<td>B3016</td>
<td>5.8</td>
<td>4.4</td>
<td>5.9</td>
<td>6.8</td>
<td>0.1</td>
<td>1.7</td>
<td>1%</td>
</tr>
<tr>
<td>3 Section CC</td>
<td>B3031</td>
<td>7.0</td>
<td>B3004</td>
<td>5.6</td>
<td>1.0</td>
<td>6.3</td>
<td>5.8</td>
<td>0.7</td>
<td>2.5</td>
<td>11%</td>
</tr>
<tr>
<td>4 Section DD</td>
<td>B3028</td>
<td>8.2</td>
<td>B3026</td>
<td>6.0</td>
<td>7.2</td>
<td>7.1</td>
<td>3.2</td>
<td>1.1</td>
<td>-3.4</td>
<td>15%</td>
</tr>
</tbody>
</table>
Figure 4-7: Test and Model Lateral Stress at FS2 Mobilization 2 Superload

The dump truck test results were compared for the inner and the outer gages at same the splices for Mobilization 2. Table 4.2 shows the results of the inner and outer test stress of the dump trucks for Mobilization 2. The test stress and the test lateral stresses of the dump truck, Figure 4-8 and Figure 4-9 respectively, had similar behavior to the test stress and the test lateral stresses of the superload.

Table 4.2: Model and Test Stress for Dump Truck-Mobilization 2

<table>
<thead>
<tr>
<th>MOB2 DUMP TRUCK</th>
<th>FS2</th>
<th>Girder D</th>
<th>Sections ordered west to east (Toledo end to Oregon end)</th>
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<tbody>
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<table>
<thead>
<tr>
<th>Bottom flange</th>
<th>test stress (ksi)</th>
<th>test stress</th>
<th>Test Ave stress</th>
<th>Test Lateral stress</th>
</tr>
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<tbody>
<tr>
<td>Inner</td>
<td>Outer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Section AA</td>
<td>B3021 2.6</td>
<td>B3034 2.2</td>
<td>2.4</td>
<td>0.2</td>
</tr>
<tr>
<td>2 Section BB</td>
<td>B3027 2.5</td>
<td>B3016 2.5</td>
<td>2.5</td>
<td>0.0</td>
</tr>
<tr>
<td>3 Section CC</td>
<td>B3031 3.2</td>
<td>B3004 2.5</td>
<td>2.8</td>
<td>0.4</td>
</tr>
<tr>
<td>4 Section DD</td>
<td>B3028 3.2</td>
<td>B3026 2.5</td>
<td>2.9</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Figure 4-8: Test Stresses for Mobilization 2 Dump Truck

Figure 4-9: Test lateral stress for Mobilization 2 Dump Truck

Table 4.3 of the superload of the Mobilization 3 field test result and another comparison of the model results were done as shown in Figure 4-10, and Figure 4-11, respectively. The inner and the outer stresses were also compared, as shown in figure 4-12.
Table 4.3: Model and Test Stress for Superload-Mobilization 3

<table>
<thead>
<tr>
<th></th>
<th>Bottom flange</th>
<th>Inner Test Stress</th>
<th>Test Stress (ksi)</th>
<th>Model Stress</th>
<th>Model Ave Stress</th>
<th>Test Ave Stress</th>
<th>Model Lateral Stress</th>
<th>Test Lateral Stress</th>
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<tr>
<td></td>
<td></td>
<td>Test strain (in/in)</td>
<td>Test stress (ksi)</td>
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<td>Model stress</td>
<td>Test Ave stress</td>
<td>Model Ave Stress</td>
<td>Test Lateral stress</td>
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<td>1</td>
<td>Section AA</td>
<td>B3021</td>
<td>210</td>
<td>6.1</td>
<td>9.3</td>
<td>145</td>
<td>4.2</td>
<td>75.0</td>
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<td>155.0</td>
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<td>Section CC</td>
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<tr>
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<td>7.5</td>
<td>20.0</td>
<td>190</td>
<td>5.5</td>
<td>245.0</td>
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</table>

Figure 4-10: Test Stresses for Mobilization 3 Superload
Figure 4-11: Model Stresses for Mobilization 3 Superload

Figure 4-12: Test and Model lateral stress for Mobilization 3 Superload

The dump truck test results were compared for the inner and the outer gages at the same splices for Mobilization 3. Table 4.4 shows the results of the inner and outer test stress of the dump trucks.
Table 4.4: Model and Test Stress for Dump Truck-Mobilization 3

<table>
<thead>
<tr>
<th>MOB2 DUMP TRUCK</th>
<th>FS2</th>
<th>Girder</th>
<th>D</th>
<th>Sections ordered west to east (Toledo end to Oregon end)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>Bottom flange</td>
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</tr>
<tr>
<td>Inner</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outer</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Section AA</td>
<td>B3021</td>
<td>24.9</td>
<td>B3034</td>
<td>2.1</td>
</tr>
<tr>
<td>2 Section BB</td>
<td>B3027</td>
<td>24.9</td>
<td>B3016</td>
<td>2.3</td>
</tr>
<tr>
<td>3 Section CC</td>
<td>B3031</td>
<td>2.9</td>
<td>B3004</td>
<td>2.5</td>
</tr>
<tr>
<td>4 Section DD</td>
<td>B3028</td>
<td>2.9</td>
<td>B3026</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The comparison of the superload test stress and the dump truck test stresses Fig. 4-13 show the structure has some nonlinear behavior at the kinks. The model stress and the test stress comparison for the lateral/average indicate that the model has a significant error in one of the regions, as shown in Figure 4-15, and Figure 4-16 that was not considered a critical region.

Figure 4-13: Test Stresses for Mobilization 3 Dump Truck
The model data brackets the actual, but does not accurately track it. BDI was aware of this error and considered ways to correct it. One way was varying the stiffness of the connection and the other was introducing addition plate elements in the region of the splice. Varying the stiffness of the connection did not resolve the issue. The superload tests showed the model was conservative. Modeling with additional plate elements would be slow, and since the conservative model was sufficient to show that the load ratings were not governed by the splice, numerical trials with the plate elements were not conducted.

Figure 4-14: Model and Test Lateral/Average Stresses for Mobilizations 2 & 3 Superload and Dump Truck
The gages in Figures 4-17 to 4-28 show the fit between the field and the model data for the splice in Figure 4-2.
Overall, the modeling and testing of the girders never the splice was sufficient to insure that the behavior was generally understood and was adequate for engineering purposes.
Figure 4-17: CSX Stress History at Diaphragm Location FS2-D1
Figure 4-18: CSX Stress History at Diaphragm Location FS2-D2
Figure 4-19: CSX Stress History at Diaphragm Location FS2-D3
Figure 4-20: CSX Stress History at Diaphragm Location FS2-D4
Figure 4-21: CSX Stress History at Diaphragm Location FS2-AA
Figure 4-22: CSX Stress History at Diaphragm Location FS2-BB
Figure 4-23: CSX Stress History at Diaphragm Location FS2-CC
Figure 4-24: CSX Stress History at Diaphragm Location FS2-DD
Figure 4-25: CSX Stress History at Pier 1 Web a
Figure 4-26: CSX Stress History at Pier 1 Web b
Figure 4-27: CSX Strain History at Pier 1 Left Bearing
Figure 4-28: CSX Strain History at Pier 1 Right Bearing
Chapter 5

Duck Creek Bridge using the BDI software

A model parallel to BDI’s model of the Duck Creek Bridge was created to gain familiarity with BDI’s software and the optimization process. The model was created using BDI’s three bridge analysis programs, WinGin, WinSac, and WinGraf. WinGin is the software that is used to create the 3D Model. Also, the WinGin is used to assign the deck thickness, girder size, pier type, boundary conditions, gage locations, truck position, and bridge type (composite, or non-composite). The WinSac is used to run the analysis and to run the optimization. The WinGraf is used to create the plots.

5.1 WinGen

This program is designed to model straight bridges, and horizontally curved bridges. Also, it has been designed to have the truck moving in a curved path on the horizontally curved bridges, which make the analysis results easier to calculate and more accurate. For the Duck Creek Bridge, the 3D model was created using the WinGin.

The data that have been collected from the test are used in this program to calibrate the model. The truck’s path on the bridge for the field test is assigned in the model, where the model and the test will have the same exact position. Both data, model and test, will be associated to calibrate the model. The optimization was done on some of the bridge
properties to develop the best possible model. For the load rating, the model was optimized using the optimization option in the program (BDI 2011A). For the Duck Creek Bridge the load rating model was optimized and was simulated using the WinSac program.

5.2 **WinSac**

This program is designed to run the finite element model that has been created in WinGin. Also, it simulates the model after the optimization command is assigned at the WinGin. The output is stored in a .str file and is used on WinGraf to create the plots.

5.3 **WinGraf**

This program is designed to create plots from the both the model and the field test data. The data of the model that has been created using the WinSac and the field test data can be compared. The comparison plot between field and analysis data is created by assigning the starting position of the truck and the plot scale.

5.4 **Duck Creek Model**

The Duck Creek Bridge model was created using the WinGen program. Steps that have been used to create the model are as follow:

1. Model Geometry: chose “Beam/Slab bridge” a window of “Beam/Slab Geometry Definition” will open as shown in Figure 5-1:
The following options are used to create the geometry of the bridge:

- “Define plan parameters” as shown in Figure 5-2:

![Define Beam/Slab Plan Geometry](image)

**Figure 5-2: Model Beam/Plan Geometry**

In this option insert the number of spans, number of beam lines, skew angle if the bridge is skewed, and the bridge meshing. Here the number of spans is 3 and the number of beam lines is 9, where the extra two beams represent as the parapet.
• “Span Length/Beam Spacing” as shown in Figure 5-3.

![Define Beam/Slab Dimensions](image1)

**Figure 5-3: Model Beam/Slab Dimension**

In this window, the length of the span and the girder spacing are assigned. Span 1 length is 71 ft. and the first beam spacing is assigned to be 1.5 ft., where the 1.5 ft. is the spacing between the first girder and the parapet.

“Transverse Members” as shown in Figure 5-4.

![WinGen beam/slab ...](image2)

**Figure 5-4: Model Beam/Slab Transverse Member**
In this window, the transverse members are assigned by using the options in Figure 5-4. The transverse members in the model are the bridge bracing.

- Choose “Spring Locations” as shown in Figure 5-5:

![WinGen elastic support dialog](image)

**Figure 5-5: Model Elastic Support Dialog**

In this window, springs can be added or deleted. The springs were added at the Duck Creek bridge at the abutments and the piers.

2. “x-section definition” to define the properties of the bridge’s element as shown in Figure 5-6:

![WinGen X-section Definition Mode](image)

**Figure 5-6: Model Section Define Mode**
• “Create New Cross-section” to define a new cross section

3. Assign X-section: assign the section properties of the bridge’s elements that have been defined using “x-section definition”, as shown in figure 5-7

![WinGen X-section Assignment Mode](image)

**Figure 5-7: Model Section Assigned Mode**

4. “Boundary Conditions” assign the bridge boundary conditions as shown in Figure 5-8. The boundary condition was assigned all fixed at the abutment for the Duck Creek Bridge.

![Boundary Condition Definition Dialog](image)

**Figure 5-8: Boundary Condition Dialog**
5. “Gage Locations” assign the location of the gages on the bridge choosing this option the following window will appear as shown in Figure 5-9.

![Gage Definition Dialog](image)

**Figure 5-9: Model Gage Definition Dialog**

Using the options in figure 5-9 to assign the gage locations as shown in Figure 5-10, Duck Creek Bridge was assigned 56 gages as shown in Figure 5-10. This figure is for the Mobilization 1 model.
Figure 5-10: Duck Creek Model

6. “Define Truck” define the truck that will cross the bridge as shown in Figure 5-11.
Use the options in Figure 5-11 to assign the truck type and the loads on each axle as shown in Figure 5-12
Figure 5-12: Dump Truck Model

7. “Truck Path” assign the path of the truck crossing the bridge and the length of the path (start and end position) as shown in Figure 5-13. Also, it is used to assign the field data with the truck path to calibrate the model in the simulation.
This figure shows each truck path being assigned, where YT is the Truck Position from the starting point of the bridge the left bottom corner in Figure 5-11, and DuckCreek_1.dat is the truck field data for path 1.

- "Save Model" to save the model, then "Save SAC file" to save the model in a form that allows WinSac program to run the simulation and generate the results.
- WinSac program is used to run the analysis of the model that have been created at the WinGen and save as Sac file.

5.5 WinSac

- Open the WinSac file then run the analysis as shown in figure 5-14.
Figure 5-14: WinSAC Runtime Window

Using the run option in Figure 5-14, WinSac will run the analysis and create a results file that can be used at the WinGrf program to create the plots.

- “Load STS data” and “load WinSac Result” To create plots using the WinGrf.

5.6 The University of Toledo model and BDI model

- Figure 5-15 is the University of Toledo model compared to the dump trucks field data from Mobilization 3. The optimization input parameters were different than BDI’s model (BDI 2011A). Figure 5-16 is BDI model compared to Mobilization 3 dump trucks field data. BDI model have the optimization limit for pier 1 and pier 2 internal splice limit FX from 0 to 1500 and internal beam 4 eccentricity from -25 to 0. The University of Toledo optimization parameters are pier 1 and pier 2 internal splice limit FX from 0 to 1700 and internal beam 4 eccentricities
from -20 to 0. The following Figures 5-15 and 5-16 are for two different set of field data compared to the two models.

**Figure 5-15: University of Toledo Model Results**

**Figure 5-16: BDI Model Results**
Chapter 6

Conclusions

This thesis presents the work done at the University of Toledo to examine the effect of a sequence of eleven superloads on two bridges on Millard Ave in Toledo and Oregon, Ohio. The Duck Creek Bridge is a three-span, straight steel-girder bridge with a composite concrete deck that carries four lanes of Millard Avenue over Duck Creek in Toledo, Ohio. The overall width is approximately 55’-0” and the bridge length is approximately 231 ft. with span lengths varying from 71 ft. to 89 ft.

The Millard Ave Over CSX Bridge is a five-span, horizontally curved bridge that carries four lanes of Millard Avenue over the CSX railroad interchange in Oregon, Ohio. The superstructure consists of seven steel plate girders that are composite with a concrete deck. The overall width is approximately 67’-0” and the bridge length is approximately 650 ft. with span lengths varying from 114 ft. to 152 ft. The superstructure’s curved geometry was generated by kinking the girder lines at the field splices, which were located near the dead load flexural inflection points. The effects of the kinked girder splices were of particular interest in the structural evaluation.

The first superload monitored was the “Platforming Reactor Module”. This was the second heaviest to cross the bridges and the first in the sequence. The second superload monitored was the “Top Convection Module”, this was the heaviest to cross the bridge
and the last in the sequence of eleven superloads that crossed the Millard Avenue. The TCM results are discussed in this thesis. A sequence of five diagnostic tests was performed before and after superloads crossings. The first diagnostic test was done to gather data to calibrate finite element models. The second and the third diagnostic test were done before and after the Platforming Reactor Module crossing. The fourth and the fifth diagnostic tests were done before and after the TCM crossing.

6.1 Duck Creek Bridge

A finite element model of TCM superload was run. The results were compared with the field data, and there was a good fit between the model and field strains. The TCM dump truck tests were scaled and compared to the earlier dump truck tests. The strain measurements for all the dump truck tests were consistent. A finite element model similar to BDI’s model was created by the University of Toledo and compared to the field data. The data from the TCM dump truck test was used to create this calibrated finite element model of the bridge using BDI software. The input optimization parameters of the two models were slightly different. Both models predicted the bridge behavior and had a good fit with field data.

6.2 CSX Bridge

BDI’s calibrated finite element model for the CSX Bridge was used by the author to run the analysis for the TCM superload. The analytical and the field strain was compared, there was a good fit between the model results and the field data. The dump truck tests before and after the superloads crossed in TCM were compared and there was a good fit between the two sets of data. The dump trucks test in TCM were scaled and compared to the dump trucks in previous test data and produced a good fit which means the structure
behavior had no observable change. The good fit for the superload and the dump trucks showed the bridge behaved elastically throughout the loading range.

The measured and calculated lateral bending strains in the lower flanges of the girders near the kinked splices for the superloads and the dump truck tests were compared. The finite element model was found to overestimate the lateral flange bending and the measured strains revealed some nonlinearity.

This work performed by author satisfied the goal of understanding the behavior of the bridges and gaining familiarity with BDI’s software. The University of Toledo participating in the field work lead to an understanding of BDI hardware.

6.3 Summary

Overall, based on the work conducted by BDI, UT and UC, the goodness of fit of the calibrated model for the first and last superload transits and absence of changes in behavior in the dump truck data sets means that the bridges behaved elastically under the superloads and the was no observable cumulative damage. The results of the dump truck test before, during and after the superload sequence showed no observable change to the stiffness of the bridge.
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