Development and Deployment of Aluminum Bridge Decks
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DEVELOPMENT AND DEPLOYMENT OF ALUMINUM BRIDGE DECKS

by

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in cooperation with

Transportation Cabinet
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and

Federal Highway Administration
U.S. Department of Transportation

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DEVELOPMENT AND DEPLOYMENT OF ALUMINUM BRIDGE DECKS

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Prepared in cooperation with the Kentucky Transportation Cabinet and the U.S. Department of Transportation, Federal Highway Administration.

This report contains the analysis and retrofit of a steel truss bridge on KY 974 over Howard Creek in Clark County, Kentucky. The bridge had major corrosion and damage to the steel stringers, along with cracking and leaching occurring in the concrete bridge deck.

The retrofit involves the replacement of the corroded steel stringers and the damaged concrete bridge deck. The original concrete deck was removed and replaced by light-weight high-strength aluminum deck panels. The aluminum deck panels, each 2-m wide, were assembled at the site and connected to each other and to the bottom steel stringers using special clamps and connectors. Following the new deck installation, the top surface was waterproofed and a new asphalt overlay was placed.

The new bridge deck constructed of aluminum material significantly reduces the deck weight, while it also allows rapid construction due to prefabricated components. In addition, the new deck can now carry a HS20-44 truck weight, which the old concrete deck was not designed to carry.

Bridge, Aluminum bridge deck, Stringers

Unlimited with approval of Kentucky Transportation Cabinet

Unclassified

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EXECUTIVE SUMMARY

The KY 974 bridge over Howard Creek in Clark County, Kentucky had major corrosion and damage to the steel stringers, along with cracking and leaching occurring in the concrete bridge deck. This report outlines the repair and retrofit carried out on the bridge. The repair involves the replacement of the deteriorated steel stringers and other related bridge components. The damaged concrete deck was also removed, and has now been replaced with aluminum bridge deck panels.

One major challenge in replacing the concrete bridge deck is the removal of the old deck, due to its sheer size and weight. Therefore, coordinated efforts are required for securing and sawing the deck into smaller pieces, and eventually removing the pieces from the bridge. A closer examination of the steel stringers was performed upon the deck removal to determine the extent of the corrosion and the needs for replacement or repair. All steel stringers at the end spans and the two exterior stringers of all interior spans were badly corroded and therefore were replaced. The gusset plates and the bracings at the two end spans were also replaced.

The Aluminum deck panels, each 2-m wide, are manufactured by Bayards Aluminum Constructies B.V. of the Netherlands. They were imported and shipped to the site. A plastic-type insulating membrane is laid onto the flange of the steel stringers prior to the placement of the aluminum deck to prevent any galvanic action between the two different materials. The panels are then connected to each other and to the bottom steel stringers using special clamps and mechanical connectors. New asphalt overlay and a waterproof layer is then placed after the deck installation process.

The new bridge deck constructed of aluminum material significantly reduces the deck weight, while also allowing rapid construction due to prefabricated components. The aluminum deck can carry the load generated by an AASHTO standard HS20-44 truck, which the old concrete deck was not designed to bear. After four years of monitoring no significant oxidation, discoloration or damage of any nature could be seen in the aluminum panels.
ACKNOWLEDGEMENT

The funding for this project is provided by the Kentucky Transportation Cabinet (KYTC) and the Federal Highway Administration (FHWA). The contribution of Mr. David Steele in this project is greatly appreciated. The support and guidance provided by Bayards Aluminum Constructies B.V. and Mr. Dick de Kluijver are greatly appreciated. The authors also wish to thank Mr. Scott Pabian for his time and effort in preparing this report.
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1 INTRODUCTION

1.1 KY 974 Bridge

The KY 974 Bridge over Howard Creek is located in Clark County, KY. The bridge is a steel truss bridge with a reinforced concrete bridge deck. With only one traffic lane, the bridge is 14’ 8” wide, and has a deck length of 81 feet. The steel reinforced concrete (R/C) bridge deck is supported on six steel stringers. The stringers are placed on transverse floor beams, which are in turn supported by the two planar trusses. The location of the bridge is depicted in Figure 1.1, while an elevation view of the bridge is presented in Figure 1.2.

Figure 1.1: Geographic location of the KY 974 bridge over Howard Creek.
This pony truss bridge had a load posting of 17 tons when inspections revealed that severe cracking and leaching in the concrete deck had taken place. Some of the steel stringers supporting the deck were found to have deteriorated to a point requiring immediate replacement. The bridge in plan view along with the sections before renovation is depicted in Figure 1.3.
Figure 1.3 – Plan and Cross sections of bridge prior to renovation.
1.2 Damage Assessment

The bridge was observed to have severe corrosion damage to the steel stringers, coupled with cracking and leaching in the concrete bridge deck. Severe cracking of concrete along the main bar direction of the deck slab bottom surface was observed, as seen in Figure 1.4. Efflorescence (deposit of minerals due to leaching of the deck) was observed on the rock surfaces directly below these cracks.

Figure 1.4 – Cracking leaching of concrete deck

All the steel stringers in panel points one and five (the two end sections) and all exterior steel stringers in panel points two through four (the interior sections) were corroded and required replacement. A total of 18 such stringers (W10 x 30) were identified for removal and replacement. The lateral steel bracing (a total of four) in panel points one and five at the abutment were also in need of replacement, as seen in Figure 1.5. The top and bottom region of

Figure 1.5 – Corrosion of stringers and lateral bracing
all the floor beams were found to have corroded at the ends, as seen in Figure 1.6. In addition, bearing plates (a total of 12), shown in Figure 1.7, at both abutments were determined to be in need of replacement. Several areas requiring mortar filling were also found in both abutments.

**Figure 1.6 – Corrosion of top and bottom ends of floor beam**

**Figure 1.7 – Bearing plate location**

### 1.3 Repair Method

Due to the deteriorated condition of the R/C concrete deck it was found necessary to replace the entire deck section. In order to increase the load bearing capacity of the structure it was proposed that the dead load of the bridge be reduced by replacing the R/C bridge deck with an aluminum deck of similar dimension (14”-0” wide x 81”-0” long). The aluminum deck is expected to carry the load generated by an AASHTO standard HS20-44 truck, in addition to all dead loads.

Aluminum bridge decks have several advantages over their concrete and steel counterparts. It has a higher strength-to-weight ratio than concrete, and corrosion-resistant aluminum bridge
decks are found to be 80 percent lighter than concrete. While requiring the least maintenance it provides a rapid and cost-effective construction alternative. Compared to steel, aluminum offers lower initial and life-cycle costs with reduced failure points due to less number of welds.

In 1997 a similar successful replacement of an existing, but functionally obsolete, bridge deck by an aluminum bridge deck was carried out by the Virginia Department of Transportation (VDOT) and the Federal Highway Administration (FHWA) on the four-lane bridge that carries Route 58 traffic over Little Buffalo Creek in Mecklenburg County, Va.

Prior to the deck replacement the existing deck, and the structural components identified as requiring replacement during the damage assessment, will be removed. This includes 18 steel stringers and four lateral steel braces. Once this was done, replacement steel could be added to the structure. A cut-away diagram of steel that will be replaced is shown below in Figure 1.8. Steel that is being replaced has been highlighted in red. Cover plates are to be provided for the corroded edges of the floor beams at the top as well as at the bottom. All the bearing plates at both abutments are to be replaced with elastomeric pads. First, the existing deck and corroded steel had to be removed.

![Figure 1.8- Steel replaced on the Howard Creek Bridge](image)

After the steel is replaced, the deck is replaced with an aluminum bridge deck. The deck is designed to carry the load generated by an AASHTO standard HS20-44 truck, in addition to all dead loads.
2 ALUMINUM BRIDGE DECK

2.1 Introduction

The penetration of de-icing salts corrodes reinforcing steel in concrete bridge decks. These lead to cracking of concrete, and ultimately loss of serviceability due to spalling of concrete or even leachate, due to penetration of water. Steel decks are also prone to corrosion even with the numerous types of coatings available for mitigation of chloride formation. To reduce the excessive financial expenses created by the ever growing number of deteriorated decks, engineers are now considering alternative materials such as aluminum for use as a bridge deck material. Aluminum is not only corrosion resistant, but unlike steel it can survive harsh environmental conditions without a protective coating.

One of the major drawbacks in the use of aluminum as deck material is the limited experience and the lack of design specifications; especially in cases, such as seismic loading research on the dynamic behavior of aluminum decks, that are insufficient. Other limitations, such as on site weld-ability, connection to existing superstructure and application of wearing surface, have prevented aluminum decks from becoming more applicable.

In order to understand the performance of aluminum bridge decks it is important to analyze them under field conditions. The bridge over Howard Creek, located in Clark County, KY, provides such an opportunity. Due to the restrictions enforced by the remaining superstructure, it was necessary to construct a bridge deck which provided an increased load bearing capacity while reducing the dead load of the deck. The deployment of the aluminum deck designed according to AASHTO specifications will increase the load posting of the bridge from present the 17 tons to the weight of the HS20-44 truck of 32 tons.

2.2 Deck Design

The new aluminum bridge deck is placed on the existing steel support structure. The profile of the extruded section welded to make approximately 2 m. (6.56 ft.) long sections of the deck is shown in Figure 2.1. The deck is constructed by joining these sections (shown in Figures 2.2 and 2.3) along the traffic directions.

![Figure 2.1 – Cross section of bridge deck](image-url)
The deck was extruded in the direction perpendicular to traffic flow; therefore the span of the deck is the distance between two longitudinal stringers i.e. approx. 900mm (2.95 ft.).

**Figure 2.2 – Extruded aluminum bridge deck panel**

These section elements are to be mounted on the renovated steel super structure of the bridge using special clamps. The deck is made up of 12 elements as seen in the plan view in Figure 2.4, the size being determined by transportation constraints from the fabrication site to the bridge site. Before placing of the aluminum deck, isolators are attached to the steel stringers (Figure 2.4) to prevent galvanic reactions between the two metals.

**Figure 2.3 – Prefabricated Deck Section**
Figure 2.4 – Aluminum Deck in Plan and Section
As seen in Figure 2.3, the elements include a side profile for the positioning of asphalt layer and the control of surface water. A drain pipe (Figure 2.3 and 2.4) has been placed in each prefabricated element for the facilitation of water drainage. Following the laying of the deck an epoxy layer will be applied on the top surface to avoid damage to the deck due to future removal of the asphalt wearing surface.

The aluminum alloy 6005 T6 used in the fabrication of the deck poses a 0.2 percent proof stress of 215 N/mm² (31.18 ksi), with an ultimate tensile strength of heat affected zone (HAZ) of 115 N/mm² (16.68 ksi) (Eurocode EN 1999-1-1). The strength of the webs, upper and lower flanges is taken as 0.2 percent proof stress of the parent material, while the strength of the welds is taken as the ultimate tensile strength of the heat affected zone. The design material factors used were obtained from the Eurocode, with 1.1 for the parent material and 1.25 for the heat affected zone. Thus, the respective design strength values for the aluminum alloy, 0.2 percent proof stress is 195 N/mm² (28.28 ksi) and the ultimate tensile strength of HAZ 92 N/mm² (13.34 ksi). The modulus of elasticity of the chosen alloy is 70,000N/mm² (10153 ksi) with a Poisson ratio of 0.3.

The design of the deck was carried out employing the standard AASHTO HS20-44 truck defined in the standard specifications for highway bridges, 17th edition 2002 and the AASHTO LRFD Bridge/Design specifications (customary L.I.S./third edition 2004). In cases where the above codes do not carry any specifications, Eurocode 9 was used.

2.3 Analysis Results

Using a three-dimensional finite element model the stresses in the deck were determined. The AASHTO HS20-44 truck with a vertical axle load of 142.3 kN (32 kip) was used as the live load. A standard AASHTO HS20-44 truck with the load distribution is shown in Fig. 2.5. The division of the load according to the contact area of the tires was done as stated in the specifications. A dynamic impact factor of 1.3 was used, according to the maximum of 30 percent dynamic impact fraction allowed, according to article 3.8.2 of the specifications. A horizontal live load equaling five percent of the vertical load was considered according to the specifications. The dead load was calculated from the weight of the deck, an epoxy layer and an asphalt top layer.

A load combination factor of 2.6 for live loads, as specified by AASHTO, was considered too conservative, and a load combination factor of 1.5, as found in Eurocode EN 1990, was used in combination with the dynamic impact factor. From different loading positions it was found that the highest moments and deflections occur when the wheel is positioned at the middle of the end field of the deck, between two longitudinal stringers.

The developed model included only two extruded deck sections and one weld section. As the load carrying capacity of the rest of the deck is neglected, this is thought to be a more conservative approach. The modeled section is made up of three spans and four longitudinal beams as seen in Figure 2.6.
Fig. 2.5 – Standard AASHTO HS20-44 truck.

Figure 2.6 - Mesh of Finite Element Model.
The supports were modeled as hinges at the center of the steel stringers. While the actual clamped supports would reduce stresses, they were neglected conservatively. The deck was restricted horizontally in the traffic direction, but not perpendicular to traffic direction. A linear elastic analysis was carried out to evaluate the positions of maximum stress occurrence under load limit state, and maximum deflections in sections of the deck at serviceability limit state.

The highest stresses in the parent material were observed when the wheel contact area was positioned above the weld. The highest stresses were seen to occur at the junction between the bottom flange and the web in the two beams at the middle. While the highest stress level in the HAZ was found when the wheel contact area was just next to the weld.

Although the allowable deflection under AASHTO LRFD Bridge Design Specifications is 1/800 times the span, this was considered to be extremely small for the case of Aluminum bridge decks. Thus, the maximum deflection, which, according to specifications was 1.13mm, was increased up to 1.35mm. The value is thought to be adequate, considering the facts that the actual structure cannot bear a vehicle load of 32 tons, and the dynamic impact will be quite small due to speed restrictions arising from the position of the bridge soon after a bend in the road.

14'-8" or 4.5

$\Delta_{\text{max}} \leq \frac{L}{800}$

$L \approx 3’-0” = 36 \text{ in} \approx 900 \text{ mm}$

Then, $\Delta_{\text{max}} \leq 1.13 \text{ mm}$

Fig. 2.7 – Deflection control of aluminum deck.
3 BRIDGE REPAIR PROCESS

3.1 Concrete Deck Removal

The R/C concrete deck on KY 974 Bridge over Howard Creek was in a deteriorated state and was deemed necessary to replace the entire deck section. In order to increase the load bearing capacity of the structure it was proposed that the dead load of the bridge be reduced by replacing the R/C bridge deck with an aluminum deck of similar dimension (14’-0” wide x 81’-0” long). The aluminum deck is expected to carry the load generated by an AASHTO standard HS20-44 truck, in addition to all dead loads.

![Figure 3.1 – RC deck cut to smaller pieces](image1)

![Figure 3.2 – Removal of RC deck](image2)
Prior to installing the Aluminum deck the existing R/C deck needed to be dismantled and removed. Since the 6” R/C deck was not composite with the steel stringers it was decided that the slab be cut to smaller pieces and then be removed from the bridge site. Replacement of corroded and damaged structural components, such as stringers, was to be carried out soon after. The removal of the RC slab enabled further identification of structural members requiring replacement. The deck was initially cut along its center in the direction of traffic flow. Subsequently it was cut in the transverse direction as shown in Fig. 3.1 into pieces of approximately 7’4” long and 2’ wide. Starting from one end of the bridge, the cut-off panels were removed and hauled away. Fig. 3.2 and Fig. 3.3 show the removal and the removed RC slabs stacked and ready to be transported.

3.2 General Repairs

Once the RC slab was removed the stringers and other structural members became accessible for repair and replacement. The condition of the underlying stringers of one end panel is seen in Fig.3.4.
Fig. 3.5 shows the stringers removed from one end panel of the bridge. The lateral steel bracings seen in the figure were also repaired or replaced in both end panels as seen in Fig. 3.6. The replaced members included 18 steel stringers and four lateral steel bracing.

Figure 3.5 –Stringers removed from end panel

Figure 3.6 –Repair and replacement of damaged members

All stringers in both end panels were replaced, while all the outer stringers in all other panels were replaced, as seen in Fig. 3.7, where the replaced steel is shown in red.
The corroded edges of the floor beams were provided with cover plates at the top as well as at the bottom. As seen in Fig. 3.8, all bearing plates at both abutments were replaced with elastomeric pads. After replacing the steel members, repairing the corroded areas of the floor beams and replacing of the bearing plates the bridge was primed and painted. The condition of the bridge before and after the painting is depicted in Fig. 3.9, where the total duration of work from the start of slab removal to end of painting was 21 days. A period of two weeks was provided for the paint to dry before work on the Aluminum deck panels began.
3.3 Aluminum Deck Placement

After providing ample time for the paint to dry, work on the aluminum deck installment began. Before the placing of the deck panels an isolating rubber sheet was attached to the top flange of each stringer. This would prevent direct contact of stringer steel and deck panel Aluminum. The isolation sheets were glued on to the stringer top flange using epoxy as shown in Fig. 3.10.

Once the isolation sheets were glued, placing of the deck panels was carried out. The panels were lifted and transported from the truck to the bridge and placed consecutively from one end of the bridge, as seen in Fig. 3.11. As seen in the figure, it should be noted that each panel, once placed, immediately became load bearing components of the bridge by facilitating the placement of the succeeding panels.

Each panel was connected to the adjacent panel at the panel edges using bolted connecters as seen in Fig. 3.12. In addition, as seen in Fig. 3.13, a T-section was inserted onto
the space between two panels on the bridge to prevent seepage of water onto the steel sections below. A steel plate was fixed to the edge of both abutments and an isolation sheet was glued on to the steel sheet. Then the edges of both abutments and edge panels were connected using another joint as seen in Fig. 3.14.
Drain pipes were inserted into the built-in holes on either side of each deck panel as seen in Fig. 3.15. The additional length, above the expected asphalt layer height, protruding from the top of the deck was then cut-off. The length of the pipes below the deck was designed so that the draining water would not come in contact with any of the steel components of the bridge super structure, as seen in Fig. 3.15.

The top surface of the bridge following the completion of the previous works is shown in Fig. 3.16. As clearly seen, the bridge was road worthy as soon as the last panel was placed and before any asphalt paving was provided. Before water-proofing and asphalt paving was carried out, the deck panels were fastened on to the stringers using special clamps seen in Fig. 3.17. Two types of clamps were used; one type was for clamping the panels to interior stringers, while the other was for fastening on to edge stringers. The clamps were made of aluminum with a galvanized steel nut and spring to maintain a constant tension at the connection.
3.4 Waterproofing and Asphalting

Waterproofing was laid on top of the wearing course of the aluminum deck panels as seen in Fig. 3.18. The water proofing was cut to accommodate the drain pipes, and also extended...
along the sides to prevent water seepage at the panel edges. Following the laying of the waterproofing, a tack coat was sprayed on to the deck before the laying of asphalt (Fig. 3.19).

Figure 3.19 – Tack coat sprayed before Asphalt overlay

Figure 3.20 and Fig. 3.21 show asphalt being laid on the bridge deck and being compacted according to specifications. Soon after compaction a wearing surface was placed on top, as seen in Fig. 3.22. The completed bridge is seen in Fig. 3.23.
3.5 Post-Construction Monitoring

The installed aluminum deck panels were monitored periodically for four years for possible damages or failures. The results from the inspection provide an assessment as to the performance of the deck panels, as well as the effectiveness of the mechanical fastening system used to tie the deck to the steel stringers supporting the deck panels.

After four years of service the deck aluminum panels appear to be in perfect condition. No significant oxidation, discoloration or damage of any nature could be seen on the underside of the panels (Fig. 3.24). As seen in Fig. 3.24, slight corrosion was seen on the galvanized steel nuts and springs of the clamping mechanism, fixing the panels and the steel beams. A couple of nuts and springs showed more corrosion compared to the rest, as seen in Fig. 3.25.

A crack in the asphalt was observed on both ends of the bridge deck above the location where the end panels meet the abutment. Differential movement of the steel plate spanning the gap between the end panels and abatements are thought to cause these cracks. Some corrosion was observed in the bolt head and some of the bolts of the connectors between panel edges (Fig. 3.26). Surprisingly the nuts, on the outside of the bridge and not facing the roadway, were not affected.
Figure 3.24 – Underside of panels

Figure 3.25 – Significant corrosion on one bolt and spring

Figure 3.26 – Corrosion of connector bolt heads
4 SUMMARY AND CONCLUSIONS

The KY 974 bridge over Howard Creek in Clark County, Kentucky had major corrosion and damage to the steel stringers, along with cracking and leaching occurring in the concrete bridge deck. This report outlines the repair and retrofit carried out on the bridge. The repair involves the replacement of the deteriorated steel stringers and other related bridge components. The damaged concrete deck was also removed, and is now replaced with aluminum bridge deck panels.

One major challenge in replacing the concrete bridge deck was the removal of the old deck due to its sheer size and weight. Therefore, coordinated efforts were required from securing and sawing the deck into smaller pieces, and eventually removing the pieces from the bridge. A closer examination on the steel stringers was performed upon the deck removal to determine the extent of the corrosion and the needs for replacement or repair. All steel stringers at the end spans and the two exterior stringers of all interior spans were badly corroded and therefore were replaced. Some of the gusset plates for bracing were also replaced.

The Aluminum deck panels, each 2-m wide, are manufactured by Bayards Aluminum Constructies B.V. of the Netherlands. They were imported and shipped to the site. A plastic-type insulating membrane is laid onto the flange of the steel stringers prior to the placement of the aluminum deck to prevent any galvanic action between the two different materials. The panels are then connected to each other and to the bottom steel stringers using special clamps and mechanical connectors. New asphalt overlay and a waterproof layer is then placed after the deck installation process.

The new bridge deck constructed of aluminum material significantly reduces the deck weight, while it also allows rapid construction due to prefabricated components. The aluminum deck can carry the load generated by an AASHTO standard HS20-44 truck, which the old concrete deck was not designed to bear. After 4 years of monitoring no significant oxidation, discoloration or damage of any nature could be seen in the aluminum panels.
APPENDIX

Aluminum bridge deck for the bridge over Howard Creek
Aluminum bridge deck for the bridge over Howard Creek
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1 Introduction

The current bridge deck of the bridge over Howard Creek in Clark Country, K.Y. is intended to be replaced by an aluminum bridge deck. The new bridge deck will be constructed by Bayards Aluminum Constructions B.V. This report gives the structural design and code checking of the bridge deck. Stresses in the deck as well as vertical deflection of the deck are checked for static loading.

Chapter two of this report gives the geometry of the bridge and the loads to be applied. The Finite Element model of the bridge deck, which was developed to check stresses and deflections, is elaborated in chapter three. The results of the calculations and the code checks are given in chapter four. Chapter five contains the conclusions.
2 Geometry, loads and material properties

2.1 Overall geometry

The aluminum bridge deck will be placed on the steel supporting structure of the existing bridge. The deck consists of sections with a width of 2 m in traffic direction. The deck is supported by longitudinal beams, so that the extrusion direction is perpendicular to the traffic direction of the bridge. The span of the deck is equal to the distance between the longitudinal beams, i.e. 900 mm. The longitudinal beams are supported by perpendicular beams, which are in turn supported by trusses. A sketch of the structure of the bridge is given in Figure 2.1.

![Figure 2.1 – Sketch of top view of the structure of the bridge (dimensions in mm)](image)

2.2 Deck geometry

The deck consists of extruded profiles that are welded together to a width of approximately 2 m. The cross-section of the extruded profile is given in Figure 2.2.

![Figure 2.2 – Cross-section of the bridge deck (dimensions in mm)](image)
2.3 Loads

The loads on the bridge deck consist of a vertical live load, a horizontal live load and the dead load.

2.3.1 Vertical live load

The loads applied on the bridge deck are according to the standard specifications for highway bridges, 17th edition 2002.

According to these specifications, the heaviest vehicle is a standard AASHTO HS20-44 truck, with maximum vertical axle load of 142.3 kN.

This load of 142.3 kN is divided over two tire contact areas with dimensions according to part 3.30 of the specification. The wheel print area according to part 3.30 is a rectangle with a length in traffic direction of 10 inches (254 mm) and a width of 20 inches (508 mm). The pressure on the deck introduced by the tire contact areas is thus 550 kN/m².

The impact fraction according to article 3.8.2 of the specifications is maximum 30 %. The dynamic impact factor is thus equal to 1.3.

2.3.2 Horizontal live load

The horizontal live load to be considered on the deck is given in part 3.9 of the standard specifications for highway bridges, and is equal to 5% of the vertical load. Per truck, the horizontal force is thus equal to:

\[ F_h = 0.05 \times (35.6 \text{ kN} + 142.3 \text{ kN} + 142.3 \text{ kN}) = 16.01 \text{ kN}. \]

The horizontal load is assumed to be transferred to the deck by one axle, two wheel prints. The horizontal load is thus equal to 124 kN/m².

2.3.3 Dead load

The dead load of the deck consists of the self weight of the deck and an epoxy and asphalt layer on top of the deck.

The bridge deck considered has a self-weight of approximately 0.56 kN/m². For the epoxy and asphalt layer a load of 0.18 kN/m² is assumed (for a system applied regularly in The Netherlands). The total dead load is thus equal to 0.74 kN/m².

2.4 Material properties

The deck is made of aluminum alloy 6005 T6. According to the Euro code EN 1999-1-1, the characteristic values of the strength for this alloy are:

- 0.2 % proof stress: 215 N/mm²;
- Ultimate tensile strength of heat affected zone (HAZ): 115 N/mm².

The strength of the webs, upper flange and lower flange of which the deck consist is taken as the 0.2 % proof stress of the parent material. The strength of the parts of the deck where welds are situated is taken as the ultimate tensile strength of the heat affected zone. (According to the code, the ultimate tensile strength of the weld metal is larger than that of the heat affected zone. Therefore, the HAZ is decisive and not the weld metal.)
Material factors are specified in the Euro code, through which the material strength values should be divided. The material factors are equal to 1.1 for parent material and 1.25 for the heat affected zone. Thus, the design values of the strength for this alloy are:

- 0.2% proof stress: \( \frac{215 \text{ N/mm}^2}{1.1} = 195 \text{ N/mm}^2 \);
- Ultimate tensile strength of HAZ: \( \frac{115 \text{ N/mm}^2}{1.25} = 92 \text{ N/mm}^2 \).

The modulus of elasticity of aluminum according to Euro code EN 1999-1-1 is equal to 70000 N/mm² and the Poisson ratio is equal to 0.3.

### 2.5 Load combination factors

According to part 3.22 of the standard specifications for highway bridges, the combination factors for combination 1A are determined by multiplying factors \( \gamma \) times \( \beta \) and are according to table 1.

#### Table 1 – Combination factors according to part 3.22 of the standard specifications for highway bridges (\( \gamma \beta \))

<table>
<thead>
<tr>
<th></th>
<th>Dead load (self weight)</th>
<th>Live load (trucks)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service limit state</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Strength limit state</td>
<td>1.3</td>
<td>2.6</td>
</tr>
</tbody>
</table>

As the bridge is placed in a curve in the road, the traffic speed will be limited. Also, according to the information provided, the load bearing resistance of the main structure of the bridge is limited to a vehicle of approximately 17 ton. The weight of the specified vehicle (32 ton) in combination with the factors for combinations of actions according to part 3.22 of the code seems therefore too conservative to apply for the bridge deck. Therefore a combination factor of 1.5 was used in combination with the specified vehicle (32 ton) and the dynamic impact factor. The combination factor applied origins from the Euro code EN 1990 (table 2). Besides these combination factors, also the material safety factors in chapter 2.4 are applied.

#### Table 2 – Combination factors applied in the design (with factors for live load according to EN 1990)

<table>
<thead>
<tr>
<th></th>
<th>Dead load (self weight)</th>
<th>Live load (trucks)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service limit state</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Strength limit state</td>
<td>1.3</td>
<td>1.5</td>
</tr>
</tbody>
</table>

In strength limit state, the design pressure load induced by one tire contact area (254 x 508 mm²) on the deck is therefore equal to:

- Vertical live load \( p_{vl} = \text{Pressure x impact factor x combination factor} = 550 \text{ kN/m}^2 \times (1+0.30) \times 1.5 = 1070 \text{ kN/m}^2 \).
- Horizontal live load \( p_{hl} = \text{Horizontal load x combination factor} = 124 \text{ kN/m}^2 \times 1.5 = 186 \text{ kN/m}^2 \).

In combination with this load, the dead load of the deck acts:

- Vertical dead load \( p_{vd} = \text{Self weight x combination factor} \)
In service limit state, the design pressure load induced by one tire contact area (254 x 508 mm$^2$) on the deck is therefore equal to:

- Vertical live load $p_{vl} = \text{Pressure} \times \text{impact factor} \times \text{combination factor}$
  $= 550 \text{ kN/m}^2 \times (1+0.30) \times 1.0 = 715 \text{ kN/m}^2$.
- Horizontal live load $p_{hl} = \text{Horizontal load} \times \text{combination factor}$
  $= 124 \text{ kN/m}^2 \times 1.0 = 124 \text{ kN/m}^2$.

In combination with this load, the dead load of the deck acts:

- Vertical dead load $p_{vd} = \text{Self weight} \times \text{combination factor}$
  $= 0.74 \text{ kN/m}^2 \times 1.0 = 0.74 \text{ kN/m}^2$
3 Description of Finite Element model of the bridge deck

The stresses in the deck and the deflection of the deck are determined with a 3-dimensional Finite Element model consisting of solid elements. The Finite Element program applied is DIANA vs. 9.1.

The longitudinal beams are supported by perpendicular beams, so that the span of the longitudinal beams is 2 m. Because of this short span, it is expected that the deflection of the longitudinal beams between the perpendicular beams is so small, that the stresses induced by this in the bridge deck are negligible to the stresses caused by bending of the bridge deck itself between the longitudinal beams. The longitudinal beams are therefore not modelled and the deck is supported with hinges at the centre of the top flange of the longitudinal beams.

Two load cases are distinguished that may be decisive for the deck:
1. Vehicle positioned in such a way that a wheel is situated exactly in the middle of the end field of the deck, i.e. in the middle between two longitudinal steel beams at the side of the bridge (Figure 3.1). In this case, no space is left for another vehicle in the other traffic direction;
2. Two vehicles that cross the bridge in different directions, so that the middle span of the deck is loaded by wheels of both trucks (Figure 3.2). The minimum distance between the tire contact areas of the trucks was assumed to be 100 mm. (Note that the bridge leaves no space for two of the prescribed vehicles. For this load case the distance between the wheels on one axle was therefore taken smaller than prescribed (conservative approach)).

![Figure 3.1 – Load case 1](image1)

![Figure 3.2 – Load case 2](image2)

With a simple two-dimensional Finite Element model, it was calculated that load case number 1 results in the highest moments (both in the field and at the supports) and the highest deflection. This is shown in Figure 3.3 and Figure 3.4, in which the deflection is scaled with a factor $5 \times 10^4$ and the colours indicate the size of the moments. Therefore the bridge deck is checked for load case number 1. For this load situation, the three-dimensional Finite Element model was made (Figure 3.5).
Figure 3.3 – Moment in bridge deck due to load case 1

Figure 3.4 – Moment in bridge deck due to load case 1
3.1 Geometry

In order to save computation time, only two extruded deck sections and one weld was considered in the model. This is a conservative approach, as the contribution of the rest of the deck in taking the load is neglected. The total width of the model is 1 m, meaning that only one axle of the vehicle is loading the part of the deck considered. The modelled part of the deck consists of three spans and four longitudinal beams. The last two unloaded spans (most right-hand spans in Figure 3.1 and Figure 3.3) are not modelled.

![3D view of the Finite Element model](image)

Figure 3.5 – 3D view of the Finite Element model

3.2 Mesh

Only one of the extruded profiles and the heat affected zone was modelled with sufficient elements in order to accurately determine the stress levels. Checks on the stress levels are based on this part of the model (Figure 3.6). The less dense mesh of the other part of the model is accurately enough to generate an accurate stiffness. In the same way, only the elements of the first span and the first support are modelled with sufficient elements to accurately determine the stress levels (Figure 3.7).

![Cross-section with element division](image)

Figure 3.6 – Cross-section with element division of the Finite Element model (left-hand profile is modelled with sufficient elements)
3.3 Supports

The supports of the deck at the longitudinal beams are modelled as hinges at the centre of the top flange of the beams. A possible clamped supported, generated by the connections between beams and deck, may reduce stresses and deflection. This possible clamped situation is, conservatively, not taken into account. At the hinges, the bridge deck is supported in vertical direction and in horizontal traffic direction (in order to take the breaking force of the vehicle). Horizontal movement of the deck perpendicular to the traffic direction is not restrained.

3.4 Loads

The tire contact area is applied on different positions in traffic direction. The positions are selected in such a way that the maximum stresses are generated in the top flange, bottom flange and webs of the deck, and at the weld in the upper flange of the deck. The deck was analysed applying linear elastic analyses.
4 Results of the analyses and code checks

The checks applied are according to AASHTO LRFD Bridge Design Specifications, 3rd edition 2004. The checks carried out comprise:

- Check of the stresses in the deck section in load limit state;
- Deflections of the bridge deck between the longitudinal beams in service limit state.

4.1 Stresses

For all of the positions of wheels analysed so far, the maximum equivalent (Von Mises) stress was determined. The Von Mises stress is a combination of vertical live load (weight of the truck), horizontal live load (breaking force) and dead load (self weight of deck and asphalt layer). The largest stress level was selected both for the parent material as for the heat affected zone.

4.1.1 Stress level of parent material

The decisive load case causing the highest stress level in the parent material of the deck appeared to be a wheel contact area centred above the weld. The decisive spot in the bridge deck appeared to be in the middle of the first span at the junction between the bottom flange and the webs (indicated with circles in Figure 4.1).
Figure 4.1 – Stresses and deflection at the decisive spot in the deck for a wheel contact area centred above the weld

The stress level in the web and the bottom flange of the deck at this position, including the dynamic impact factor but excluding the combination factor, is 88.2 N/mm². Multiplied by the combination factor of 1.5, the design value of the stress is 132 N/mm². The design value of the 0.2% proof strength of the parent material is equal to 195 N/mm². The unity check is equal to 132 / 195 = 0.67, i.e. smaller than 1.0.

It is noted that high peak stresses occur at the supports at a longitudinal beam. This is due to the node constraints applied in the Finite Element model. In reality, the supports are not point constraints, but cover a certain area. In such a case, the peak stresses will flow off.
4.1.2 Stress level in weld and HAZ

The decisive load case causing the highest stress level in the parent material of the deck appeared to be a wheel contact area just next to the span of the upper flange of the deck section where the weld is situated. The decisive spot in the bridge deck appeared to be in the middle of the first span.

Figure 4.3 - Stresses and deflection at the decisive spot in the heat affected zone for a wheel contact area just next to the weld
The stress level in HAZ of the weld in the upper flange of the deck, including the dynamic impact factor but excluding the combination factor, is 48.1 N/mm². Multiplied by the combination factor of 1.5, the design value of the stress is 72 N/mm². The design value of the ultimate strength of the HAZ material is equal to 92 N/mm². The unity check is equal to 72 / 92 = 0.78, i.e. smaller than 1.0.

4.2 Deflections

According to the AASHTO LRFD Bridge Design Specifications, the allowable deflection of the bridge deck in service limit state is equal to 1/800 times the span. The span of the bridge deck is 900 mm, so that the maximum deflection according to the specifications is equal to 1.13 mm. This is considered to be extremely small for aluminum bridge decks, considering the high strength to stiffness ratio of aluminum alloys. In a discussion between Bayards and the approving authorities, it was concluded that a deflection of 1.35 mm is acceptable.

The deflection was determined for a vehicle with a weight of 32 ton, an axle load of 142.3 kN and a tire contact area of 254 x 508 mm². The axle load is subsequently multiplied with the dynamic impact factor equal to 1.3 and the combination factor equal to 1.0.

The Finite Element analyses resulted in the following maximum deflections:
- For a tire contact area situated in the middle of the section of the deck: 1.18 mm;
- For a tire contact area exactly at the edge of a bridge deck section: 1.33 mm.

These values are slightly larger than the deflection according to the AASHTO LRFD Bridge Design Specifications of 1.13 mm, but smaller than the acceptable deflection of 1.35 mm. According to the information provided, the steel structure of the current bridge cannot bear a vehicle of 32 ton. Besides, as the bridge is situated in a curve in the road, the traffic speed, and therefore the dynamic impact, is expected to be small.

The magnitude of the axle load is calculated for which the deflection of the bridge is equal to the deflection according to the AASHTO LRFD Bridge Design Specifications of 1.13 mm. If the dynamic impact factor of 1.3 is taken into account, this axle load - for which the deflection is 1.13 mm - is equal to 120 kN. Excluding the dynamic impact factor, this axle load is 156 kN.

Based on comparable bridge decks in Europe of which the authors of this document have taken knowledge, a deflection of 1 to 1.5 mm for a span of 900 mm is considered as small for an aluminum bridge deck. The authors consider all deformations as well acceptable.
5 Conclusions

The aluminum bridge deck proposed for the bridge over Howard Creek in Clark Country, K.Y., is checked according to the AASHTO LRFD Bridge Design Specifications. In order to determine the stresses and deflection of the bridge deck, a three-dimensional Finite Element model was made and analysed.

The design value of the maximum stress in the bridge deck is checked for an AASHTO HS20-44 truck, with a weight of 32 ton and a maximum vertical axle load of 142.3 kN. This axle load is multiplied with a dynamic impact factor of 1.3 and a combination factor of 1.5 (according to the European standards). The design value of the maximum stress in the bridge deck is then equal to:
- For the parent material: 132 N/mm$^2$;
- For the heat affected zone near the weld: 72 N/mm$^2$.

The allowed material strength for alloy 6063 T6 are taken according to the European Specification. According to the specification, the given values should be divided by a material factor in order to obtain the design value of the strength. The material factors are 1.1 for the parent material and 1.25 for the heat affected zone, respectively. The design values of the strength are equal to:
- For the parent material: 195 N/mm$^2$;
- For the heat affected zone near the weld: 92 N/mm$^2$.

It is thus concluded that the bridge deck satisfies the strength limit state demand.

According to the AASHTO LRFD Bridge Design Specifications, the maximum deflection of the bridge deck in service limit state is equal to 1.13 mm. This is considered to be extremely small for aluminum bridge decks, as the ratio between strength and stiffness is high for aluminum alloys. In a discussion between Bayards and the approving authorities, it was concluded that a deflection of 1.35 mm is acceptable. The authorities deflection of the bridge deck section for loading by the AASHTO HS20-44 truck multiplied by the dynamic impact factor of 1.3 is equal to:
- For a tire contact area situated in the middle of the section of the deck: 1.18 mm;
- For a tire contact area at the edge of a bridge deck section: 1.33 mm.

These values are slightly larger than the deflection according to the AASHTO LRFD Bridge Design Specifications of 1.13 mm, but smaller than the acceptable deflection of 1.35 mm.

The magnitude of the axle load is calculated for which the deflection of the bridge is equal to the deflection according to the AASHTO LRFD Bridge Design Specifications of 1.13 mm. If the dynamic impact factor of 1.3 is taken into account, this axle load is equal to 120 kN.