The Implementation of Full Depth UHPC Waffle Bridge Deck Panels

Final Report

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16. Abstract

In the U.S. today, there are over 160,000 bridges that are structurally deficient or obsolete, and more than 3,000 new bridges are added each year. Federal, State, and municipal bridge engineers are seeking new ways to build better bridges, reduce travel times, and improve repair techniques, thereby reducing maintenance. Additionally, owners are challenged with replacing critical bridge components, particularly bridge decks, during limited or overnight road closure periods.

State and Federal agencies are gaining significant interest in using full depth ultra high performance concrete (UHPC) waffle deck panels as a possible solution to these challenges. The first implementation of UHPC deck panels on a U.S. roadway was made possible through a grant from the Federal Highway Administration’s Highways for LIFE program. The objective of this research is to confirm that this system is a viable solution to the problems encountered by design engineers. It is hoped that the full depth UHPC bridge deck system will revolutionize the way bridges are designed in North America.

This project was divided into two phases. Phase 1 included the design and testing of a mock-up bridge section for verifying design assumptions, as well as for evaluating the feasibility of manufacturing and installing the deck elements. Phase 2 consisted of the construction of a full scale two-lane bridge on a secondary road in Wapello County, Iowa, using prestressed concrete girders and 14 UHPC deck panels.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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LIST OF ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
BPN	British Pendulum Number
BPT	British Pendulum Tester
CIP	Cast-in-Place
DOT	Department of Transportation
FEA	Finite Element Analysis
FHWA	Federal Highway Administration
LCCA	Life Cycle Cost Analysis
LRFD	Load and Resistance Factor Design
PPE	Personal Protective Equipment
SRV	Skid Resistance Value
TRRL	Transport and Road Research Laboratory
UHPC	Ultra High Performance Concrete

1. INTRODUCTION

In the U.S. today, there are over 160,000 bridges that are structurally deficient or obsolete, and more than 3,000 new bridges are added each year.(1) Federal, State and municipal bridge engineers are seeking new ways to build better bridges, reduce travel times, and improve repair techniques, thereby reducing maintenance. Additionally, owners are challenged with replacing critical bridge components, particularly bridge decks, during limited or overnight road closure periods.

In response to these challenges, researchers at the Federal Highway Administration (FHWA) Turner-Fairbank Highway Research Center began investigating potential solutions in 2000. Prototype designs of full depth ultra high performance concrete (UHPC) waffle deck panel systems have been in development over the past 6 years in both Europe and the U.S.

UHPC provides superior durability against chlorides, freeze-thaw effects, salt scaling, abrasion, accidental impact, fatigue, and overload, thereby extending the useful life of the bridge deck. Combining these positive attributes of UHPC and the efficiency of the waffle panel design provides an extremely durable option that enables faster construction and longer girder spans through the efficient use of materials and reduced weight. In addition to these benefits, the UHPC waffle panel bridge deck system is applicable to both new construction and the rehabilitation of existing deteriorated bridge decks. Using this solution for bridge rehabilitation not only restores the deck but also provides opportunities for upgrading load capacity through the improved strength and reduced deck dead load.

Numerous State departments of transportation (DOT) and the FHWA have expressed significant interest in using full depth UHPC waffle deck panels. By demonstrating this system is a viable solution to the problems encountered by design engineers, it is hoped that it will revolutionize the way bridges are designed in North America.

The research conducted under this project was divided into two phases. Phase 1 included the design and testing of a mock-up bridge section for verifying design assumptions, as well as evaluating the feasibility of manufacturing and installing the deck elements. Phase 2 consisted of the construction of a full scale two-lane bridge on a secondary road in Wapello County, Iowa, using prestressed concrete girders and 14 UHPC deck panels. This report describes the results of both Phase 1 and Phase 2.

2. WORK COMPLETED IN PHASE 1

This chapter summarizes the progress that was made in Phase 1 of the project. As mentioned previously, the objectives of Phase 1 were to prototype and model the demonstration bridge construction planned for Phase 2. These objectives were met by producing two prototype UHPC waffle deck panels based on preliminary design work completed by the Iowa DOT, modeling the section of the demonstration bridge that would undergo testing in Phase 1 using a finite element analysis (FEA) to predict the response of the system, and load testing the prototype waffle slabs to confirm the design assumptions and FEA validity.

WAFFLE PANEL DESIGN AND FABRICATION

The preliminary design of the waffle slab was completed in late August 2009, and shop drawings were completed by early September 2009. The project team consulted on the fabrication process and aspects of the design relating to ease of production and requirements for UHPC joint fill. The prototype panels were 8 feet wide by 9 feet, 9 inches long, with two layers of mild reinforcing in each rib and dowel bars extending out of the slab at the transverse panel-to-panel joint locations (see figure 1). The prototype panel modeled one girder-to-girder span along the length of the bridge. It was determined that two slabs would be necessary to test the transverse panel-to-panel joint.

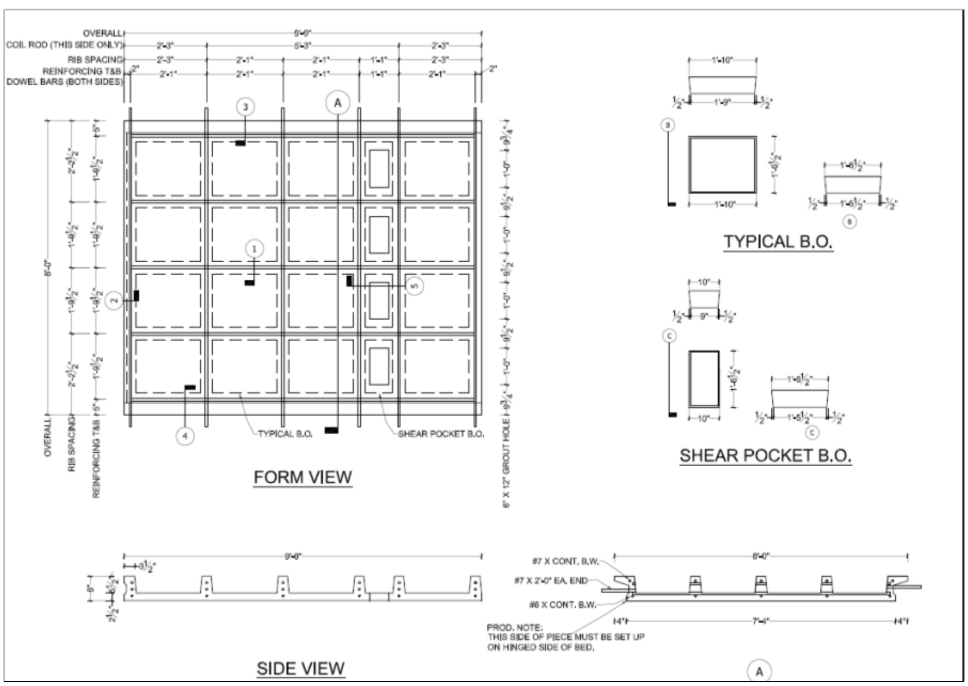

Figure 1. Diagram. Shop drawing of prototype panel.

The two prototype panels were produced in September 2009 at the Coreslab Structures plant in LaPlatte, NE. The panels were cast using a displacement technique where the form was first partially filled with fluid UHPC. Then the voids, which make the ribs of the panel, were forced downward into the UHPC to displace the material, creating the shape of the panel. This technique was used to allow for the removal of the voids once the UHPC reached initial set, which is necessary to allow for unrestrained shrinkage and to maintain the random orientation and consistent meshing of the fiber reinforcement. An additional benefit of casting the panels in an inverted orientation is that the final driving surface will be cast into the panels through the use of a form liner. By eliminating the application of a wearing surface placed in the field, the cost of the system is further reduced, while the durability is increased. Figures 2 through 6 depict the casting and production sequence.

Figure 2 shows the reinforcing layout of the first casting. Uncoated reinforcing was used because the UHPC is effectively impermeable, eliminating the need for epoxy coated, galvanized, or stainless steel reinforcing to resist corrosion. The prototype panel formwork was made of wood to provide an inexpensive and temporary form. This strategy was employed because the design was preliminary, and changes to the rib size, spacing, and joint profiles were possible between the prototype and demonstration panels. The final formwork will be made of steel and adjustable for various panel configurations. In addition to the standard reinforcing, several strain gauges were cast into the panels to monitor the internal responses of the UHPC and reinforcing.

Figure 2. Photo. Prototype panel form setup.

Figure 3 shows the placing of the UHPC into the form. The UHPC was placed with a specially designed bucket that is the same width as the form, which aligns the steel fibers in the UHPC in the longest direction of the panel. This alignment helps to increase the flexural strength of the panels. The bucket was moved along the length of the form and kept behind the leading edge of the flow to eliminate any discontinuity in the fiber orientation. The form was filled to a predetermined level equal to the required volume of the panel, and then the voids were set as an assembly (see figure 4). Each of the prototype panels required approximately 1 cubic yard of UHPC.

Figure 3. Photo. Casting.

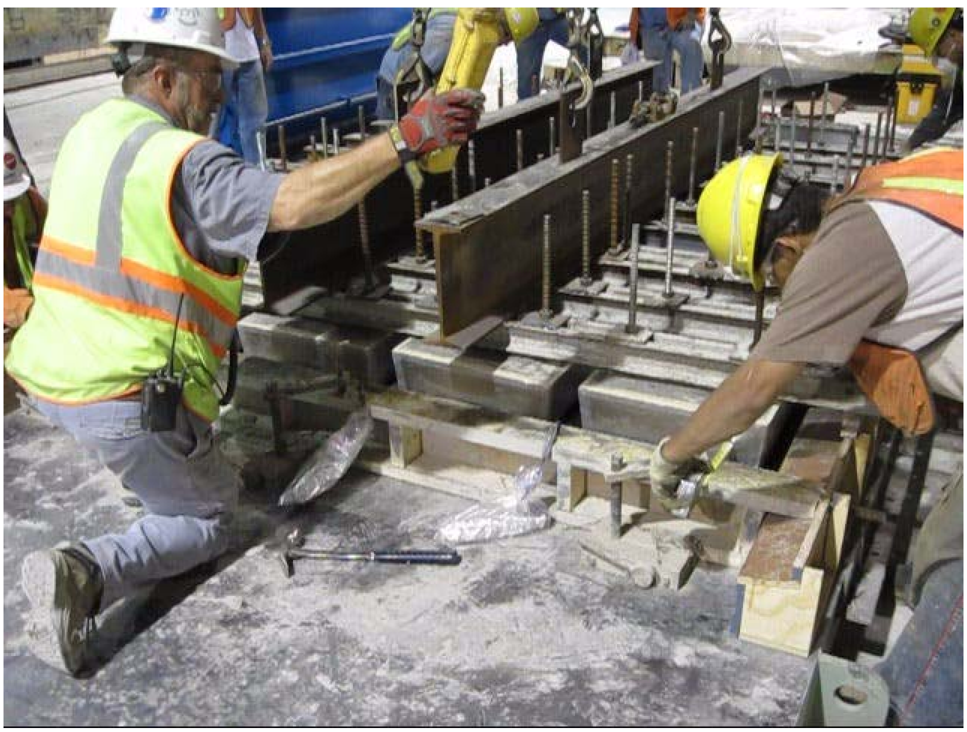
Figure 4. Photo. Placing the voids as they are lowered into position.

By placing the voids as an assembly, the panels can be cast substantially faster and with more precision. The UHPC is displaced by the voids as they are lowered into position to create the final shape of the piece. Placing the voids prior to the casting would create a cold joint effect at the corners of the voids because, as the UHPC flows into itself around the corners of the voids, the fibers will not cross the intersection of the two flows of UHPC.

As mentioned previously, this casting method also allows the voids to be removed after the initial set of the UHPC, which allows the UHPC to shrink without inducing internal stresses. This is important because the UHPC will reach initial set within 12 to 14 hours but will not reach release strength until approximately 40 hours after casting. Damage from shrinkage can occur during this time if the appropriate precautions are not taken. Figure 5 shows the panel after the side forms and voids were removed.

Figure 5. Photo. Formwork removed.

After the panels reached the required 14,000 psi release strength, they were stripped from the form. The panels were lifted into the vertical position by the casting bed, rotated 180 degrees about a vertical axis, and lowered back to the horizontal position in the proper orientation by the casting bed. This technique was used to reduce the handling stresses on the panels. Even though the concrete strength is very high at release, the sections from which the panels are rotated are very thin, limiting the amount of stress that can be taken without cracking. The critical section is at the very thin area where the toe of the transverse joint edge and the longitudinal rib intersect. By rotating the piece with the casting bed, the issue of handling stresses is eliminated.

The panels were moved to the curing area and exposed to an accelerated steam curing cycle for 48 hours at 195 °F, as specified for maximum strength and durability. Before steam curing the panels measured approximately 15,000 psi compressive strength. A test cylinder was broken before the steam was turned off to verify that the panels had cured correctly, and the compressive strength of the UHPC at that time was 29,800 psi, exceeding the required design strength of 24,000 psi. After curing was complete, the panels were loaded on a truck and transported to Iowa State University to be load tested.

Figure 6. Photo. Completed prototype panels.

LABORATORY LOAD TESTING

Laboratory load testing included testing the panels and the transverse joint in service, fatigue, and ultimate loading scenarios, along with investigating friction properties of textures for the driving surface. Table 1 shows the structural test protocols and sequence. The load magnitudes for testing were established using the American Association of State Highway and Transportation Officials (AASHTO) 16-kip service level truck wheel load with a factor of 1.33 for the panel and 1.75 for the joint to account for impact from moving loads.

To represent field conditions as closely as possible, it was determined that a prestressed concrete beam was needed to support the panels during testing. A beam similar to the one required for the demonstration bridge was used.

A UHPC joint casting was required and completed in the laboratory. The transverse joint between the panels, the shear pockets, and the longitudinal area along the length of the support beam were cast with Ductal UHPC in late November 2009. The laboratory joint fill followed the same processes and used the same equipment as planned for the full scale demonstration bridge construction.

Table 1. Structural test protocols and sequence.

Test Number	Test Description	Loading Location	Maximum Load
1	Service load test panel-2	Center of the panel	1.33[a] x 16kips = 21.3 kips
2	Service load test on transverse joint	Center of the joint	1.75[b] x 16 kips = 28 kips
3	Fatigue test on the transverse Joint	Center of the joint	28 kips (1 million cycles)
4	Ultimate load test of transverse joint	Center of the joint	48 kips
5	Fatigue test on the panel-1	Center of the panel	21.3 kips (1 million cycles)
6	Ultimate load test of the panel	Center of the panel	40 kips

a, b –dynamic allowance factors from AASHTO table 3.6.2.1-1(2)

Strength and Serviceability Testing

An FEA model of the completed test setup was created, and several runs were made to locate the worst case loading scenarios before the physical testing began. This helped limit the amount of physical testing required and decrease the amount of time required for the testing phase of the project. It was verified upon completion of the testing that the model closely represented the results of the physical tests and will enable future projects incorporating UHPC to be designed and investigated more efficiently and with less physical testing.

The results of the testing were promising. In summary, the 21.3-kip load placed on the panel caused two hairline cracks in the rib below the loading location, and the 28-kip load applied to the joint caused a barely visible crack to form on the bottom of the joint, as predicted by the FEA model. Fatigue loading applied for the specified 1,000,000 cycles did not have any noticeable effect on the strength or durability of the panels.

Surface Texture and Skid Resistance Testing

Five commercially available form liners were selected as possible driving surface textures, and 12-inch by 12-inch samples of the textures were cast in UHPC. The samples were tested according to ASTM E303(3), Standard Method for Measuring Surface Friction Properties, and the Transport and Road Research Laboratory (TRRL, formerly the Road Research Laboratory) Standard Sand Patch Test for Measuring Surface Texture Depth (4). Table 2 summarizes the results of both tests.

Any texture depth greater than 1/64 inch for the Standard Sand Patch Test is considered "open," according to the TRRL. An open sample indicates the texture can be worn substantially from its current condition before becoming smooth, which means it has good characteristics for use as a long-term wearing surface. The testing showed all samples met the criteria to be classified as open.

Table 2. Sand patch test results.

Sample No.	Texture (Source)	Sand Patch Dia. (in.)	Average Dia. (in.)	Texture Depth (in.)	Texture Characterization
1	2/61 Thames (Rekli)	9.45, 8.66, 8.46, 8.86	8.86	0.05	Open
2	2/102 Parana (Rekli)	7.87, 7.68, 7.87, 8.07	7.87	0.04	Open
3	Broom Finish (Architectural Polymers)	10.04, 9.25, 9.65, 9.06	9.50	0.06	Open
4	Heavy Broom (Architectural Polymers)	6.30, 6.30, 6.10, 5.90	6.15	0.10	Open
5	Anti-Skid (Fitzgerald Formliners)	8.66, 8.86, 9.06, 9.06	8.87	0.05	Open

Texture Depth = $\frac{4V}{\pi d^2} \times 10^3$, V = 3.051187 in^3

The skid resistance value (SRV) was measured using a British Pendulum Tester (BPT), according to ASTM E303. The BPT is a dynamic pendulum impact-type tester used to measure the energy loss when a rubber slider edge is propelled over a test surface. The test was performed on the same set of sample textures as the Sand Patch Test, and four SRV tests were performed on each sample. The SRV test setup is shown in figure 7, and a summary of the SRV tests is presented in table 3.

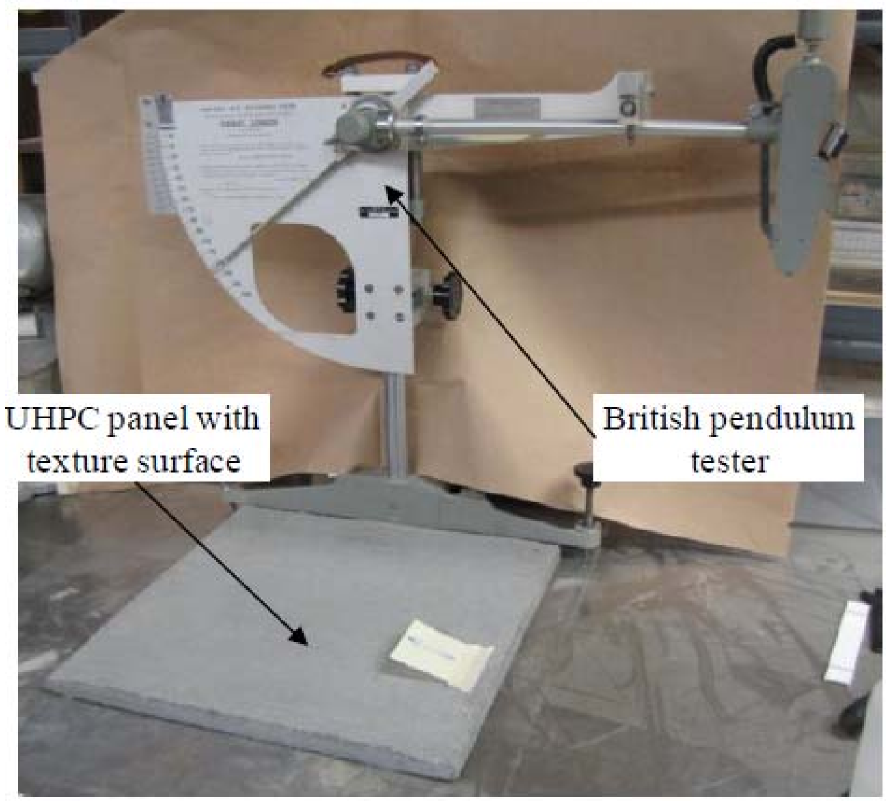

Figure 7. Photo. SRV testing setup.

Table 3. Skid resistance test results.

Sample No.	Texture (Source)	Skid Resistance Value (BPN)	Average (BPN)
1	2/61 Thames (Rekli)	87, 88, 88, 88	87.75
2	2/102 Parana (Rekli)	96, 96, 96, 96	96
3	Broom Finish (Architectural Polymers)	72, 70, 70, 70	70.5
4	Heavy Broom (Architectural Polymers)	80, 81, 80, 81	80.5
5	Anti-Skid (Fitzgerald Formliners)	80, 80, 80, 81	80.25

BPN = British Pendulum Number

The suggested minimum SRV for difficult sites is 65, as determined by the TRRL, so all the tested textures are excellent choices for all possible scenarios and roadway types. Based on the information gathered from these tests, sample 2 was chosen due to its extremely high SRV and good surface texture characteristics.

CONCLUSIONS

The researchers developed a preliminary design that is anticipated to perform well in service.

Casting was successful, and the prototype panels were produced without any major difficulties.

An FEA was developed that accurately represented the material and the overall structural performance.

Only minor cracking was observed during testing, and this amount of cracking is not anticipated to cause any long-term durability issues.

A suitable wearing surface was selected.

In general, the prototype panels performed very well and appeared to be more than capable of holding up to the rigors of use on a public highway. Additional information regarding the technical aspects of the strength and serviceability testing and associated results can be found in separate reports.[5,6]

3. WORK COMPLETED IN PHASE 2

The objectives of Phase 2 included the construction of the full scale demonstration bridge in Wapello County and a life cycle cost analysis (LCCA). These objectives were met by producing 14 UHPC waffle deck panels used in the demonstration bridge, creating an LCCA spreadsheet to compare the cost of conventional construction to the use of UHPC waffle deck panels over the useful life of the structure, and compiling data recorded during the first three months of the in-service performance of the demonstration bridge to determine the best estimation of the long-term performance and durability of the structure.

WAFFLE PANEL DESIGN

Research in the United States and Canada has shown that reinforced concrete bridge decks do not act as flexural members, but rather as low profile tied arches.(7) This research has noted the large factor of safety inherent in typically reinforced bridge decks. Factors from 8 to 10 are usually found with failures in typical decks by block shear and not flexure. Based on this study, bridge design specifications were developed in Ontario, Canada, in the early 1980s that allowed minimal reinforcing steel to be designed into bridge decks meeting required design conditions. (8) In addition, AASHTO adopted similar design specifications with the release of the Load and Resistance Factor Design (LRFD) Bridge Design Specifications in 1994.(9) In the 1990s, the Iowa DOT constructed 16 bridges on the State's primary system using the empirical deck design. The DOT's bridge office has been monitoring these bridges since their construction.

The preliminary design of the reinforcing steel for the waffle slab panels took into consideration the empirical design specifications allowed in the AASHTO LRFD Bridge Design Specifications. The empirical design allows for bridge decks that meet the required "9.7.2.4-Design Conditions" to be designed using four layers of reinforcing steel with a minimum of:

- 0.27 in^2/ft in the bottom mat of reinforcing.
- 0.18 in^2/ft in the top mat of reinforcing.

In the preliminary design for the test panel, no. 7 reinforcing bars were used in the bottom mat of steel at a transverse spacing of 1 foot, 9.5 inches (0.334 in^2/ft) and longitudinal spacing of 2 feet, 1 inch (0.28 in^2/ft). No. 6 reinforcing bars were used in the top mat at a transverse spacing of 1 foot, 9.5 inches (0.246 in^2/ft) and longitudinal spacing of 2 feet, 1 inch (0.211 in^2/ft).

The field cast UHPC joints were developed based on the testing done at the FHWA Turner-Fairbank Highway Research Laboratory and the details used in New York and Canada on bridge projects that had already been completed.(10)

One other factor to note in the basic design was a limit of 8 inches of overall depth. This thickness was chosen to allow the UHPC waffle deck system to be used to replace existing deteriorated bridge decks without altering the road profile.

Initial discussion for the waffle panel design also included designing for a transversely pre-tensioned concrete section that would be longitudinally post-tensioned in the field to eliminate any cracks in the panels, but one goal from the producer’s view was to design the slabs with mild reinforcing. This decision would eliminate the need for prestressing capabilities in the precast plant, limiting the initial cost of the equipment and certification required to produce the slabs. Hopefully this decision will allow more producers and municipalities to adopt the UHPC waffle slab system.

Once the basic design concept was completed, FEA was performed to assist in determining the adequacy of the panel. The FEA verified the design concept, with only minor cracking predicted at the bottom of the rib section, allowing the project to move to full scale testing. At this time it was decided that the connections between the panels would also need to be tested. The connection details were primarily based on ongoing testing at FHWA.[10] The locations for connection rebar dowels were limited based on the physical dimensions of the slab, so it was decided one reinforcing bar would be located at each rib of the panel in the transverse panel-to panel-joint, and two reinforcing bars would be placed in each rib of the panel at the longitudinal panel-to-panel joint in addition to the shear key detail on the panel edge. The acceptability of the panel-to-panel connections was primarily verified through testing.

The project team reviewed the testing of the mock-up bridge section, and the final design of the waffle slab was completed in early April 2010. Shop drawings for use by production staff were complete shortly thereafter. Figures 8 and 9 show the final panel dimensions and reinforcing, respectively.

The demonstration bridge panels were designed identically to the prototype panels with respect to the shape, depth, and spacing of the ribs and voids, but changes were made to the overall size, reinforcing design, and surface texture. The demonstration bridge panels were designed to span from the centerline of the roadway to the outside of the guardrail with an overall size of feet wide by 16 feet, 2.5 inches long. A no. 7 reinforcing bar was used in both the top and bottom of each rib instead of one no. 6 and one no. 7, to eliminate possible confusion and improper installation by the production staff. A section of reinforcing was also added to the outside edge of the panel to splice into the barrier posts that resist the impact forces from a vehicle collision. The 2/102 Parana formliner texture selected from the skid resistance testing in Phase 1 was provided on the top surface of the panel to act as the final driving and wearing surface.

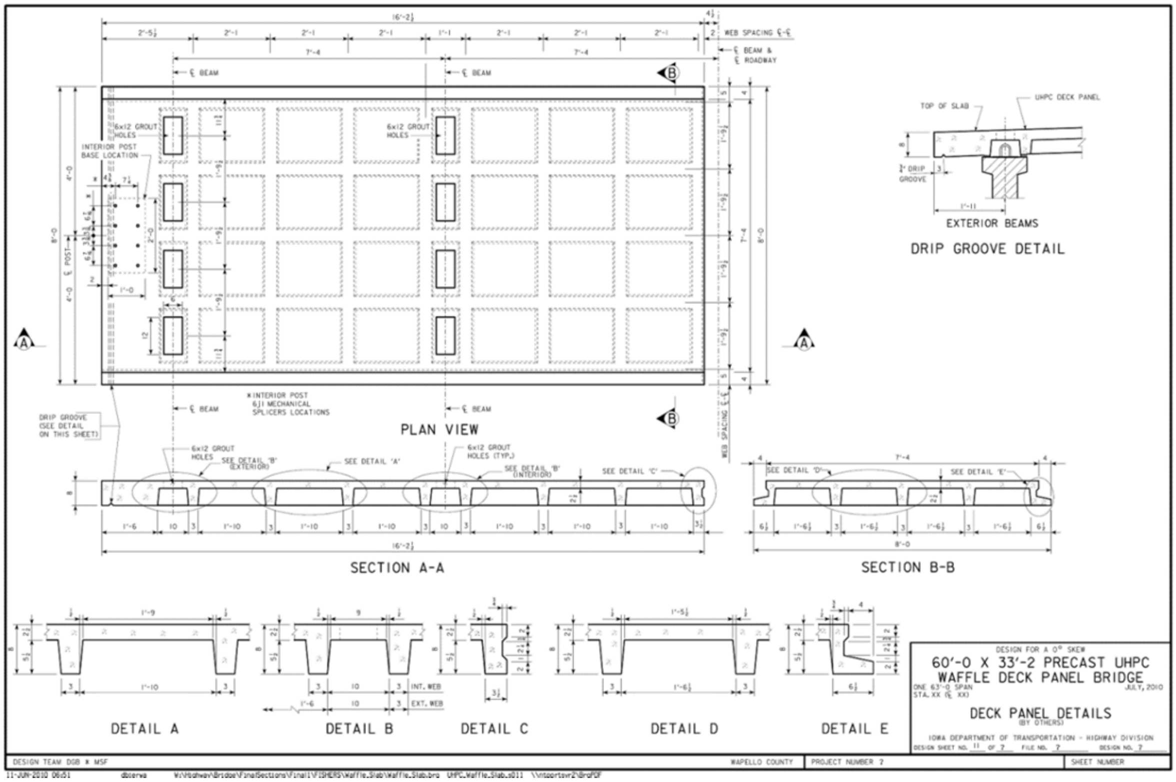

Figure 8. Diagram. Final panel dimensions.

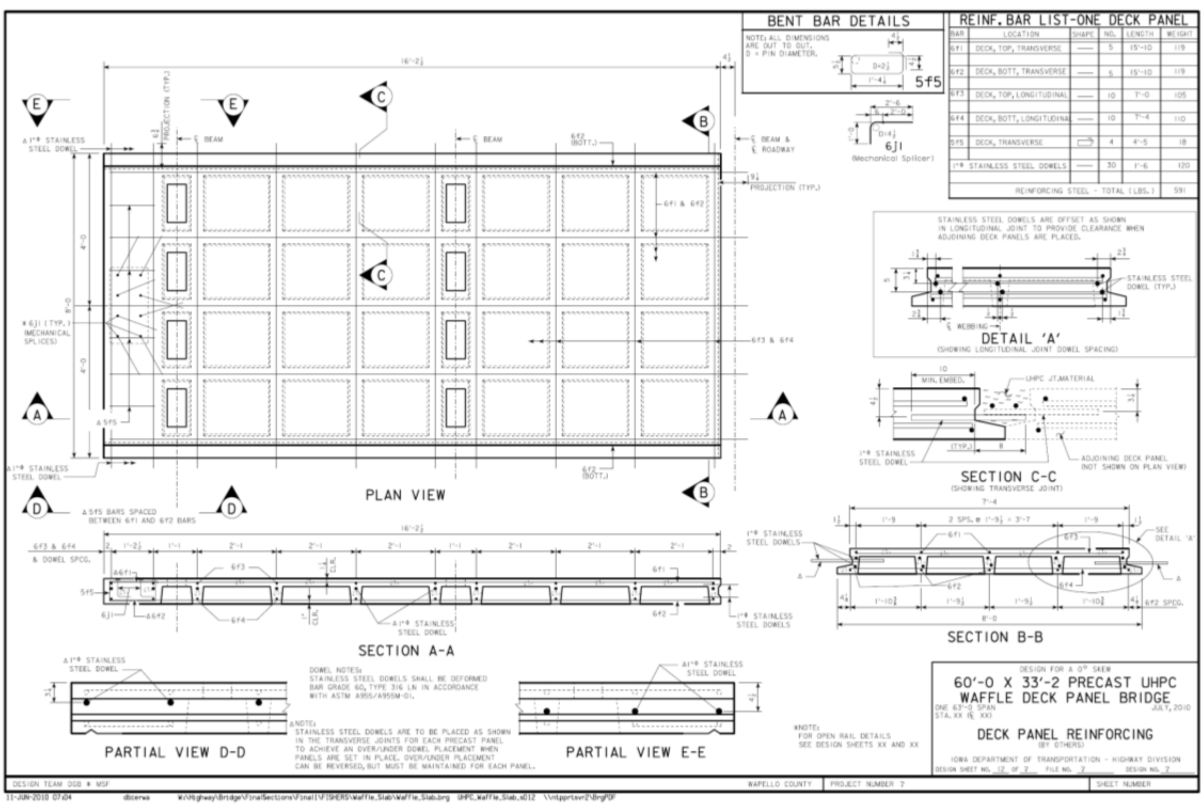

Figure 9. Diagram. Final panel reinforcing.

TESTING EQUIPMENT AND PLANT MODIFICATIONS

Special equipment is required to produce the waffle slabs and to mix, cast, cure, and test the UHPC. Most of the plant modifications required to accommodate the casting of UHPC were completed during Phase 1, including the basic formwork, UHPC mixing and placing equipment, and quality control and testing equipment. The additional modifications that were required to cast the demonstration slabs included the additional formwork used to cast the ribs and voids of the panels, and a computer controlled steam curing system. The equipment necessary to produce the UHPC waffle slabs is described in the sections below.

Formwork

No existing formwork matched the profiles of the waffle slab design or the edge conditions needed for the panel-to-panel joints. New formwork was designed and fabricated to cast the slabs. At the time of the prototype casting, the final design was unknown because testing would be required to validate the design. By researching the typical bridge designs in the Iowa highway system, it was determined that the maximum length would be 25 feet. The slab design was far enough along at the time of prototype casting to determine that the standard width would be 8 feet. The width was chosen primarily due to transportation and handling restrictions. The casting bed was fabricated by a specialized formwork manufacturer to accommodate these dimensions and was utilized to rotate the prototype waffle slabs manufactured in Phase 1. As previously mentioned, the side and void forms for Phase 1 were fabricated out of wood to serve as a temporary form in the event changes to the section needed to be made between Phase 1 and Phase 2.

The remaining formwork for the demonstration slabs was purchased after the final panel design was complete and was fabricated out of steel to allow for a high number of reuses and to conform to the tight tolerances required due to the thin UHPC sections used. The formwork consisted of a bottom form including the side forms, end forms, and the casting table acquired in Phase 1, and a top form which created ribs and voids of the panel. Figures 10 and 11 show the bottom and top form assemblies, respectively.

The formwork was selected primarily to cast the demonstration panels but was also designed to be adaptable to various rib spacings and overall panel configurations for future uses of the waffle panel system. As shown in figure 11, the pans can be removed and replaced with different sizes on the transverse beams, and the transverse beam locations can be adjusted along the length of the longitudinal beams to accommodate unlimited configurations with a limited additional investment. The bottom form is also designed to cast a deck panel up to approximately 25 feet in length to accommodate panels covering multiple lanes of traffic.


Figure 10. Photo. Casting bed and side forms.


Figure 11. Photo. Void form.

Casting and Initial Curing Equipment

A high shear mixer is required to adequately mix the UHPC material, and most precast plants have an existing batch plant capable of mixing UHPC without major modifications. However, typically the capacity of the mixer is reduced by approximately 50 percent due to the amount of energy required to mix the UHPC. During both the Phase 1 and Phase 2 production periods, no modifications were made to the batch plant to add steel fibers to the mixer or to automate the addition of premix bags or admixtures. All components of the mix were weighed and added manually for this project.

The manual addition of steel fibers and other components of the UHPC mixture required additional safety precautions and personal protective equipment (PPE) beyond what is required for conventional concrete production. The most difficult problem to solve was how to protect workers from the steel fibers. A very durable and puncture-resistant glove is needed to handle the fibers. The best solution found was to use a thick rubber glove (to help prevent punctures) under a heavy welding glove (to provide a thick layer of protection).

To place the UHPC in the forms, a concrete bucket was purchased in Phase 1. As mentioned previously, the bucket helped to create the alignment of the steel fibers, but it was also necessary because of the fluidity of the UHPC mix. Rubber gaskets were provided on the bucket gate to seal the opening. Previous experience with UHPC showed this was the only efficient way to transport UHPC from the mixer to the casting bed.

UHPC needs to be moist cured during the initial curing period, but steam curing was not available at the casting bed, so dry heat and moisture retention was used. To cure the panels, the exposed surfaces were covered with a shrink wrap film in Phase 1 and spray applied liquid curing compound in Phase 2 to seal in the internal moisture. Heat was applied by propane heaters for the initial curing period, and the entire form was enclosed with a tarp to contain as much heat and moisture as possible.

Quality Control Equipment

Equipment for testing fresh UHPC was purchased before the prototype panel casting and included a flow table (see figure 12), vibrating table, molds for prisms and cylinders, and scales. These items were essential for producing a high-quality product. Project personnel were trained on the equipment and related procedures before production began. A cylinder end grinder and high capacity compression testing machine are needed to accurately verify compressive strength of cured UHPC. Since this equipment was outside the budget limitations for this project, a local university was contracted to perform the testing of the hardened properties within the specifications outlined by the supplier.

Figure 12. Photo. Flow table.

Steam Curing

To accelerate the curing process and increase the durability of the UHPC a 48-hour steam curing cycle is required. During Phase 1, the steam temperature was controlled manually by adjusting a ball valve approximately every 15 minutes. This process was inefficient and time-consuming, but it was effective in reaching the required design strength for the mock-up panels. An automated steam curing system was purchased to provide precise control and eliminate the constant monitoring and manual adjustment of the temperature for the demonstration panels. A Sure Cure system consisting of a software package, electronic equipment, and thermocouples was used to monitor the internal temperature of the curing enclosure and control the steam flow into the curing enclosure by the use of an electronically actuated valve. The software package also logs the time and temperature curve for the curing cycle to assist with quality control. See figure 13 for an example of the output. Two channels of temperature recording are shown in magenta and orange, and one channel of ambient temperature is shown in black.

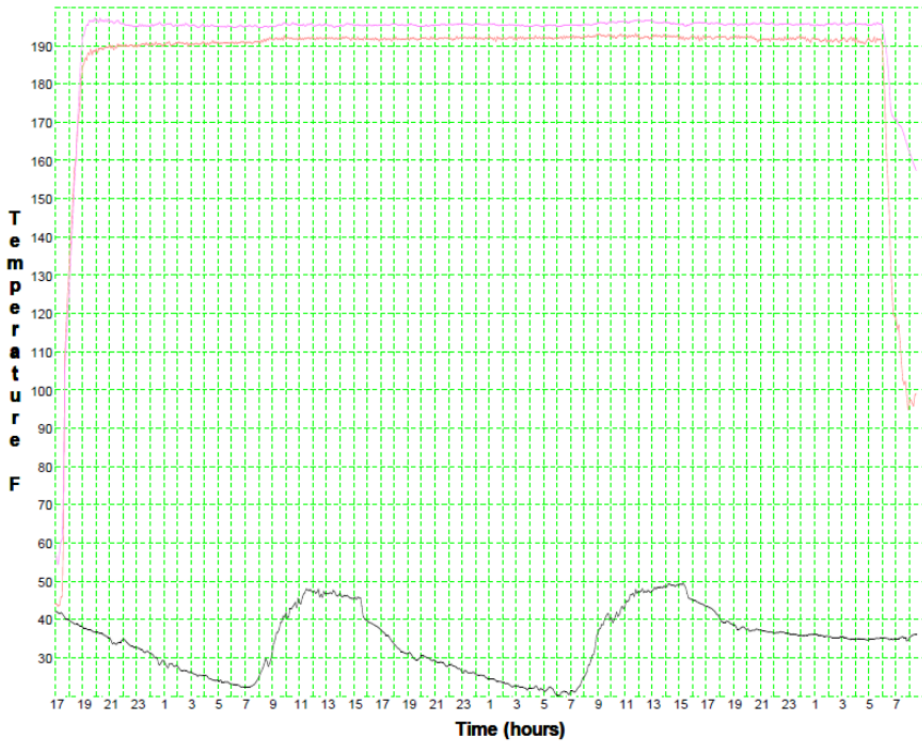

Figure 13. Graph. Time vs. temperature.

Other Plant Modifications

Additional equipment and plant modifications were considered in the original proposal, including a fiber distribution system for mixing UHPC, the installation of an automated bed heat system to cure the fresh UHPC, construction and installation of a permanent steam curing chamber, and the purchase of special equipment to handle the slabs from the edges. Knowledge gained during the course of the project proved that these items were not required and did not prove to be cost-effective for this project alone. Commercial fabrication of UHPC may warrant the purchase of these items based on the volume of product required.

Impact on the Precaster

In general, the modifications required to produce the UHPC waffle deck panels were very limited, and for small quantities of UHPC production, no modifications to the plant would be required beyond formwork. If more than one or two castings are required, it is recommended to invest in the products purchased for this project, including a bucket, testing equipment, and a curing system to improve the efficiency and safety of the operation. The process and timing related to producing UHPC compared to conventional concrete is the biggest change to the production environment. Once a process is implemented and a new mindset of the crew producing the panels is reached, the process runs smoothly.

DEMONSTRATION PROJECT PANEL FABRICATION

The demonstration project panels were produced with the same basic process as the prototype panels. Figure 14 shows the form setup for the first demonstration panel casting. Uncoated reinforcing was used on the demonstration panels because only very minor cracking on the bottom surface of the panel was observed during the laboratory testing of the prototype panels. However, the dowels entering the field cast joints were made of stainless steel to provide an additional factor of safety against the possibility of corrosion at the interface between the plant and field cast UHPC. Unlike the prototype panels, no monitoring instruments were cast into the panels.

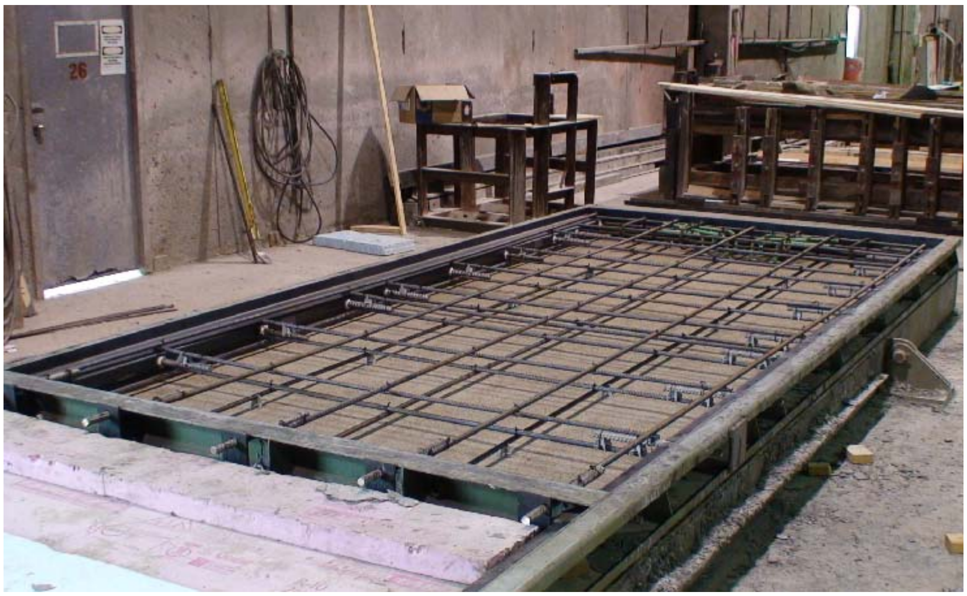

Figure 14. Photo. Initial form setup.

Figure 15 shows the placing of the UHPC into the form. The UHPC was placed with the same specially designed bucket that was used for the prototype panels. Along with being able to transport the extremely fluid material, the bucket helps to align the steel fibers in the UHPC in the longest direction of the panel. The bucket was moved along the length of the form and kept behind the leading edge of the flow to eliminate any discontinuity in the fiber orientation.

The form was filled to a predetermined level based on the total volume of UHPC required for a panel, and then the voids were set as an assembly (see figures 16 and 17). Each of the prototype panels required approximately 2 cubic yards of UHPC. By placing the voids as an assembly, the panels can be cast quickly while keeping the size and spacing of the ribs very accurate. As with the prototype panels, the UHPC is displaced by the voids as they are lowered into position to create the final shape of the piece.

Figure 15. Photo. Placing UHPC.

Figure 16. Photo. UHPC placed to correct level.

Figure 17. Photo. Placing the voids.

As mentioned previously, by forming the panels upside down and using a displacement casting method, the voids can be removed after the initial set of the UHPC, which allows the UHPC to shrink unrestrained. Damage from shrinkage will occur between initial set and stripping if the appropriate precautions are not taken. The use of steel forms increased the need to remove the voids promptly because the form was extremely rigid compared to the prototype panel form made of wood and foam. Using the displacement technique also helps to randomly orient the fibers in the rib sections. Figure 18 shows the panel after the formwork is removed.

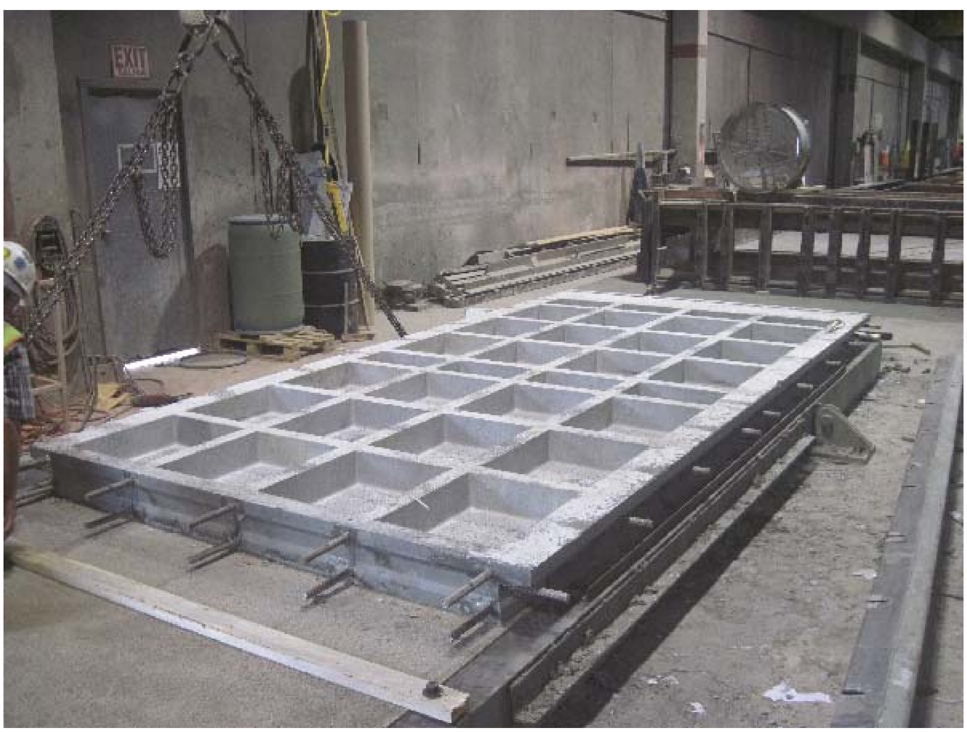

Figure 18. Photo. Formwork removed (panel upside down).

After the required 14,000 psi release strength was verified, the panels were lifted into the vertical position by the casting bed (see figure 19), rotated 180 degrees about a vertical axis, and lowered back to the horizontal position in the proper orientation by the casting bed similar to the prototype panels (see figure 20). The panels were then moved from the casting area to the curing area.

Checking the release strength was one of the challenges faced by the plant when dealing with UHPC. A 3-inch by 6-inch cylinder with 14,000 psi compressive strength could be tested at the precast plant using existing compression machine, but the ends of the cylinders could not be prepared to the flatness tolerance outlined in the specifications. The cylinders were ground flat by hand, not by a machine, so the compressive strength measured was most likely a conservative value due to stress concentrations on the cylinder ends caused by unevenness. The alternative to this conservative method was to deliver individual cylinders to the local university and have them tested every time release strength needed to be verified, but this was not feasible due to time constraints in the production schedule.

Figure 19. Photo. Lifting the panel to vertical.

Figure 20. Photo. Panel rotated back to horizontal (panel right side up).

The panels were cured for 48 hours at 195 °F, as required by the supplier for maximum strength and durability. Prior to steam curing, the panels measured approximately 15,000 psi compressive strength. A series of test cylinders were broken after the steam curing was completed to verify that the panels had been cured correctly, and the compressive strength of the UHPC at that time was an average of 33,700 psi, substantially exceeding the required design strength of 24,000 psi. After curing was complete, the panels were stored until they were needed at the demonstration bridge job site.

DEMONSTRATION PROJECT CONSTRUCTION

The demonstration bridge in Wapello County is 33 feet, 2 inches wide by 60 feet long, consisting of 14 UHPC panels supported on five Iowa "B" beam precast/prestressed concrete girders spaced at 7 feet, 4 inches, with overhangs measuring 1 foot, 11 inches. The panels are jointed with UHPC at the crown longitudinally, the transverse panel-to-panel joints, and the shear pockets over the girders. The demonstration bridge plan and cross section are shown in figures 21 and 22, respectively.

The design of the demonstration bridge and deck panels was completed in early April 2010; however, due to an extraordinary amount of rainfall and flood damage around the bridge site, the construction schedule was delayed by approximately 1 year. The letting for the general contractor occurred in May 2011, and construction began in August 2011. A full set of plans for the demonstration bridge, including the panels and connections, is included in the appendix.

The general contractor proceeded with construction quickly. The existing bridge was removed the week of August 15, 2011, and new substructure and abutments were completed by September

5, 2011. The precast beams and UHPC panels were set in place starting the week of September 12, 2011, followed by the approach slabs and other associated road work. The total time to install the bridge was less than 4 months.

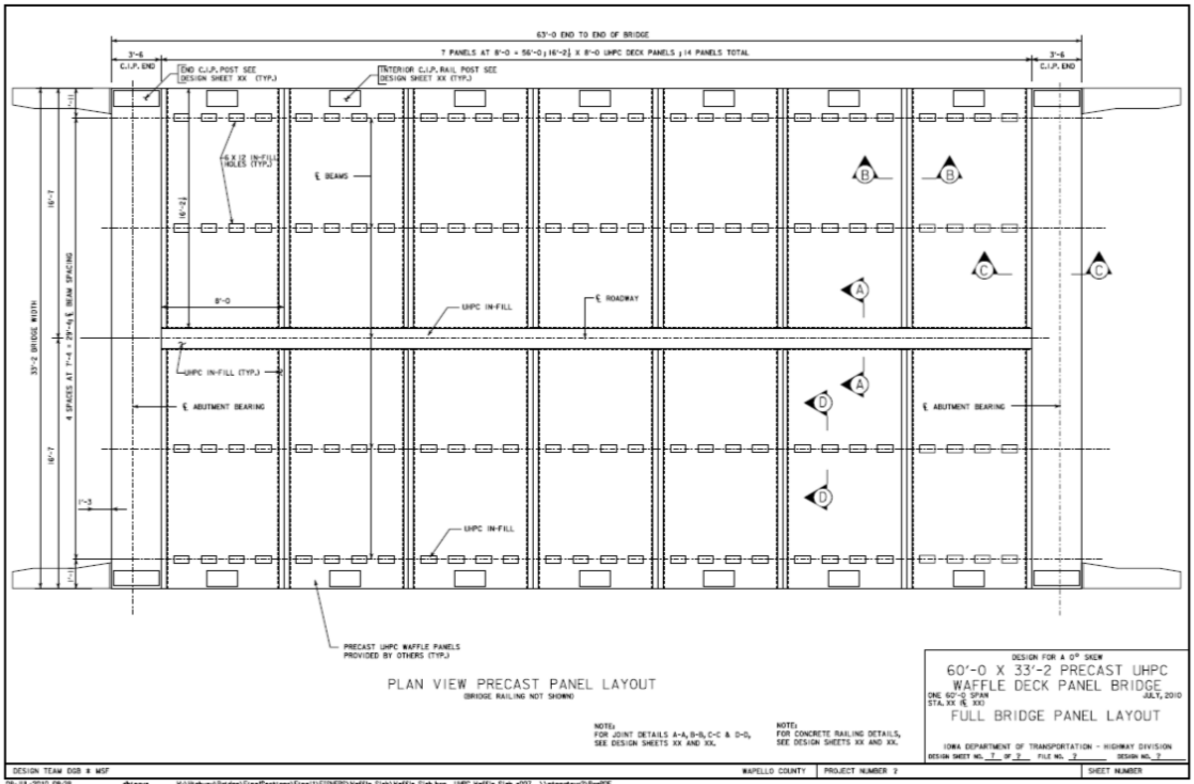

Figure 21. Diagram. Plan of the demonstration bridge.

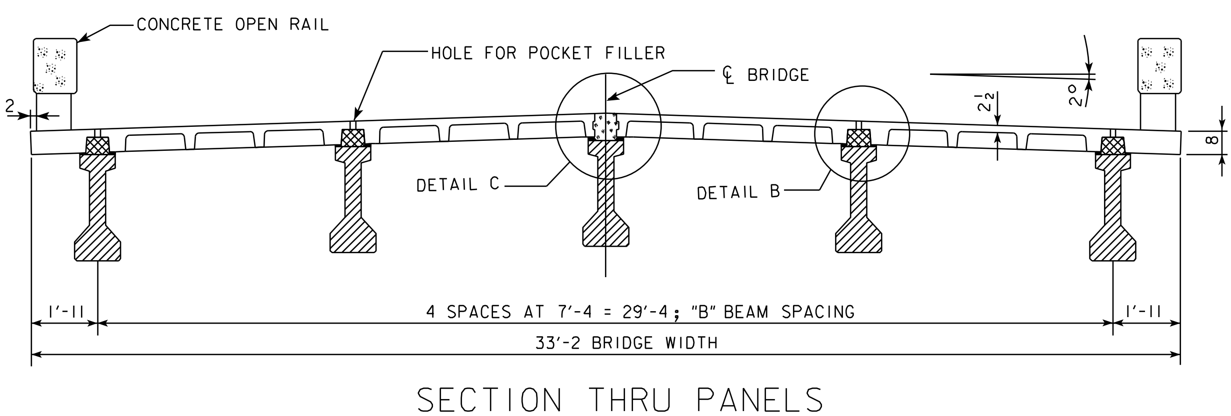

Figure 22. Diagram. Cross section of the demonstration bridge.

Panel Installation

The panels were delivered by truck to the demonstration bridge site, where they were offloaded and stacked until they were needed. The panels were set over a period of 2 days, and simple slings were used for rigging. The sequence of installation was from south to north on the east side and from north to south on the west side, due to the layout of the reinforcing dowels. Steel shims were placed under the panels to align the top surface of the panels and provide a smooth driving surface. Two layers of half-inch compressible foam weather stripping were placed along the top surface of the beams to seal the gap between the beams and panels to contain the field cast UHPC placed in the longitudinal panel joint and shear pockets. Figure 23 shows the deck panels being set, and figure 24 shows a close-up of the panels in place.

The contractor had no major problems installing the panels, and the east side of the bridge was completed in only a few hours since each panel’s transverse joint dowel bars were designed to set on top of the previous panel’s dowel bars. The only minor issue was the installation of the second panel on the west side of the bridge. The panel was difficult to install because of the overlapping reinforcing dowels in both the transverse and longitudinal joints, but after a few adjustments to the rigging, the contractor was able to slide the panel into position. The same rigging technique was used for the remaining panels on the west side of the bridge without any issues.

Figure 23. Photo. Setting the deck panels.

Figure 24. Photo. Close view of the deck panels.

Joint Fill

The field cast UHPC joints were poured on September 27 and 28, 2011. The transverse panel-to-panel joints on the east side were cast on September 27, and the transverse panel-to-panel joints on the west side, along with the longitudinal crown joint, were cast on September 28. The UHPC was mixed on-site with two portable mixers. Approximately 222 feet of joint were cast on September 27, and 282 feet were cast on September 28. The UHPC was placed using wheelbarrows and a funnel system. Figures 25 and 26 illustrate the construction sequence of filling the joints and shear pockets with UHPC.

The transverse panel-to-panel joints were set tight together, but they were also sealed with a bead of silicone caulking to ensure no UHPC leaked out of the joint during casting. The longitudinal joint and the shear pockets were sealed with the weather stripping applied to the beams during construction. Weather stripping was also applied to the top side of the panels at all field cast locations to act as a form extension and ensure there were no areas of the joint that were underfilled.

Figure 25. Photo. Portable mixers for batching joint fill UHPC.

Figure 26. Photo. Transverse joint casting.

A layer of sealed plywood was placed over the weather stripping at the joints and shear pockets after they were filled with UHPC to prevent moisture loss during the curing period. Small risers were placed on each joint and filled with UHPC to provide positive fluid pressure on the joints to force air out of the mix and keep the joint completely filled. Sandbags were placed on top of the plywood to keep the UHPC from leaking out of the joints and onto the deck due to the pressure from the riser and the bridge slope.

According to the supplier, casting went very well. If it had not rained the day before casting delaying form preparation, the entire joint fill process could have been finished within an 8-hour work day. The contractor was also very easy to work with and was open to suggestions relating to the joint fill process. The only negative of the joint fill was one low spot in the UHPC on the first day of casting. The imperfection was noticed prior to casting the second day, and two layers of weather stripping were used instead of the single layer used on the first day.

Figure 27 is a photograph of the completed demonstration bridge.

Figure 27. Photo. Completed demonstration bridge.

IN-SITU TESTING AND EVALUATION

Field testing was used to evaluate the performance of the UHPC waffle deck panels under true service conditions. The demonstration bridge was opened to traffic in November 2011 and field tested in February 2012. The field testing included monitoring of live load deflections and deformations at discrete, critical locations on the bridge superstructure as it was subjected to static and dynamic truck loads. A 3D FEA of the bridge was used to help interpret the results of live load testing, estimate strains due to dead load, and examine live load distribution.

Two UHPC waffle deck panels along the length of the bridge were selected for instrumentation. One of these panels was located near the mid-span, and the other was located adjacent to the south abutment, as illustrated in figure 28. Surface-mounted strain gauges were used on each panel and their adjacent UHPC joints to quantify deformations and identify the likelihood of cracking under service loads. The locations of these strain gauges were selected to coincide with critical locations on the panels and deck joints where stress and strain would likely be extreme.

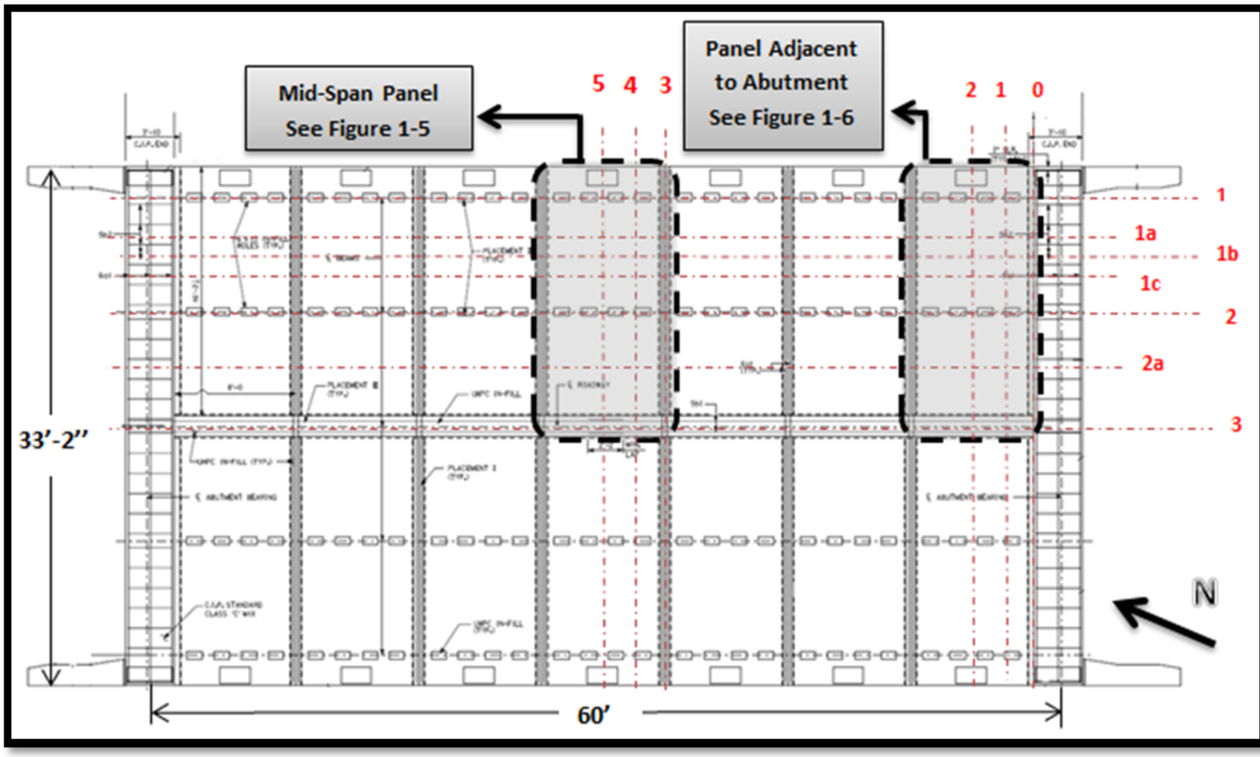

Figure 28. Diagram. Locations of monitored panels.

In addition to the strain gauges on the deck panels, 13 surface-mounted strain gauges and five string potentiometers were attached to the girders to characterize the global bridge behavior, measure mid-span deflections, and quantify lateral live load distribution factors. Using two additional string potentiometers, deflections were also measured at the mid-spans of the deck panel located near the center of the bridge. Top and bottom girder strains were also monitored for three of the girders at mid-span and at a section 2 feet from the south abutment.

Live load was applied by driving a heavily loaded dump truck across the bridge along predetermined paths. The total weight of truck was 60,200 lb, with a front axle weight of 18,150 lb and two rear axles weighing roughly 21,000 lb each. For static tests, the truck was driven across the bridge at a speed of less than 5 miles per hour. Each load path was traversed twice to ensure repeatability of the measured bridge response. For dynamic tests, the truck speed was increased to 30 miles per hour to examine dynamic amplification effects.

The results of the testing were promising. The maximum strains and deflections experienced by the demonstration bridge during the field tests were well within expected performance parameters. No strains recorded on the top of the deck indicated a likelihood of cracking or opening of joint interfaces that might adversely affect durability. The only locations where strains approached the anticipated cracking threshold of the UHPC waffle deck were on the underside of the panel adjacent to the abutment. These cracks were small in width, and the strains recorded during the test were less than those recorded on the laboratory test panels at service load levels.

In general, the UHPC waffle deck panels performed very well and appear to be more than capable of holding up to the rigors of use on a public highway.

Additional details of the Phase 2 field testing are expected to be published in the 2013 proceedings of the PCI Convention and National Bridge Conference (E.H. Gheitanbaf, J.M. Rouse, and S. Sritharan, authors).

4. LIFE CYCLE COST ANALYSIS

LCCA was performed to compare the demonstration bridge to a conventional cast-in-place (CIP) concrete deck bridge of the same size that would be typical for the location of the demonstration bridge. The following sections summarize the methodology and assumptions used to complete the analysis, along with a summary of the results.

LCCA ASSUMPTIONS AND SUMMARY OF RESULTS

The formula in figure 29 was used to calculate the LCCA for both options:

$$LCC = FC + \sum_{t=0}^{t=n} pwf\,[MC+IC+FRC+UC] + pwf\,[S]$$

Figure 29. Equation. LCCA formula.

Where:
FC = First (Initial) Cost
t = Time Period of Analysis
MC = Maintenance Costs
IC = Inspection Costs
FRC = Future Rehabilitation Costs
S = Salvage Values or Costs
pwf = Present Worth Factor
UC = Users Costs

The life cycle cost consists of the summation of all costs incurred over the life of the structure discounted to account for the use of constant dollars. These costs include the initial design and construction costs, periodic maintenance and inspection costs, future rehabilitation costs, any residual value or salvage value, and the user costs associated with all of the previously mentioned activities. The discount rate is represented by the present worth factor, and for this analysis a value of 3 percent was used (typical established values range from 3 to 5 percent). All costs for the LCCA were gathered from actual costs recorded during the demonstration project construction or best professional estimates by the Iowa DOT, Wapello County, or the contractor.

The user costs for both options were compiled using the information shown in table 4, and a spreadsheet prepared by the Pennsylvania DOT available for public use was modified and used to complete the computations.

LCCA summaries for the demonstration bridge and the CIP option are shown in tables 5 and 6, respectively. The LCCA for both options turned out to be similar after all costs were considered.

Table 4. User costs for the demonstration bridge and the CIP bridge.

User Cost Inputs	Demonstration Bridge	CIP Bridge
Length of affected roadway (miles)*	3.00	3.00
Average Daily Traffic*	280	280
Normal traffic speed (mph)	45	45
Construction traffic speed (mph)	45	45
Normal accident rate (per million vehicle miles)	1.9	1.9
Construction accident rate (per million vehicle miles)	2.2	2.2
Number of construction days	120	130

*Total traffic affected by maintenance, inspection, and rehabilitation activity.

As shown in tables 5 and 6, the CIP bridge has a slightly lower life cycle cost than the demonstration bridge. There are a few main causes:

- **The initial cost of the demonstration bridge is high.** A large initial cost is difficult to overcome and is the biggest cause for uncertainty about UHPC. If the initial cost for the UHPC option could be decreased, the LCCA would favor this option easily.

- **The amount of traffic using the bridge is relatively small.** The user costs associated with maintaining and rehabilitating the CIP bridge are one of the main reasons the LCCA is close to being equal for both options. However, the amount of traffic using the bridge on a rural secondary road is considerably less than what would be found on a highway or interstate. If this bridge were placed on a road with double the daily traffic, the LCCA would favor the demonstration bridge.

- **The maintenance and rehabilitation costs for the CIP bridge occur far enough in the future that the costs are discounted substantially to convert to today's dollar value.** While the CIP bridge requires considerably more maintenance than the demonstration bridge, the costs occur at 25, 50, and 100 years in the future. Due to the discount rate, these costs are reduced to a small value in terms of today's dollars. This limits the effect of maintenance costs in the overall LCCA.

Based on the results of the LCCA, it appears the UHPC waffle deck system would be ideally suited for use on a heavily traveled road where impacts to users would be minimized by the shorter construction time and decreased maintenance activity.

Table 5. LCCA summary: demonstration bridge.

Demonstration Bridge		
ITEM	**COST**	**NOTES**
Initial Cost	[illegible]	[illegible] (Provided by Wapello County)
Annual Maintenance	$175 / Year	Estimated Yearly General Maintenance (Provided by Wapello County)
Inspections (Required Every Two Years)	$250 / Occurance	Estimated Inspection Cost (Provided by Wapello County)
Five Year Increment Scheduled Maintenance Crack Repair, Patching, Joint Sealant (Inspect / Repair / Replace)	$0	This Item is Not Needed on UHPC Bridge
25 Year Scheduled Maintenance Surface Grinding and Overlay	$0	This Item is Not Needed on UHPC Bridge
50 Year Scheduled Maintenance Redeck Bridge	$0	This Item is Not Needed on UHPC Bridge
75 Year Scheduled Maintenance Surface Grinding and Overlay	$0	This Item is Not Needed on UHPC Bridge
100 Year CIP Design Life Reached Demolish and Rebuild CIP Bridge	$0	This Item is Not Needed on UHPC Bridge
120 Year UHPC Design Life Reached End of Usefull Life - No Residual Value	$0	Assume girder life is extended by 20 years due to fewer deck rehabilitations and less damage from corrosion due to poor deck conditions.
User Costs Associated with Construction and Maintenance Consists of Driver Delay Costs, Vehicle Operating Costs, and Accident Costs	$168,702	(Calculated from data provided by IDOT)
TOTAL LIFE CYCLE COST	**$680,270**	

Table 6. LCCA summary: CIP bridge.

Estimated Cost of a Similar Bridge with CIP Deck		
ITEM	**COST**	**NOTES**
Initial Cost	$375,642	Estimated Cost of Construction with Typical CIP Deck Design (Provided by Wapello County / IDOT)
Annual Maintenance	$250 / Year	Estimated Yearly General Maintenance (Assume slightly more than UHPC) (Provided by Wapello County)
Inspections (Required Every Two Years)	$200 / Occurance	Estimated Inspection Cost (Assume slightly less than UHPC) (Provided by Wapello County)
Five Year Increment Scheduled Maintenance Crack Repair, Patching, Joint Sealant (Inspect / Repair / Replace)	$1,000 / Occurance	
25 Year Scheduled Maintenance Surface Grinding and Overlay	$25,000	(Provided by IDOT)
50 Year Scheduled Maintenance Redeck Bridge	$45,000	(Provided by IDOT)
75 Year Scheduled Maintenance Surface Grinding and Overlay	$25,000	(Provided by IDOT)
100 Year CIP Design Life Reached Demolish and Rebuild CIP Bridge	$375,642	Assumed Typical Service Life of CIP Bridge is 100 Years
120 Year UHPC Design Life Reached	$0	Not Applicable to the CIP Bridge
120 Year Residual Value of CIP Bridge	$297,313	Credit for 80 years of useful life remaining in the structure (Calculate from construction cost - future maintenance costs)
User Costs Associated with Construction and Maintenance Consists of Driver Delay Costs, Vehicle Operating Costs, and Accident Costs	$233,824	(Calculated from data provided by IDOT)
TOTAL LIFE CYCLE COST	**$662,756**	

CONCLUSIONS

The final design is anticipated to perform well in service.

Casting was a successful experience for Coreslab Structures. The demonstration bridge panels were produced with ease due to the experience of Phase 1.

Construction of the demonstration bridge proceeded smoothly, considering the new construction techniques that were required.

The testing on the completed structure validates the assumptions from Phase 1 testing.

Only minor cracking was observed adjacent to the abutment in the demonstration bridge that poses no threat to the long-term durability of the structure.

The LCCA relating to the UHPC waffle deck system is suited for a roadway where user costs can be decreased by construction speed and reduced maintenance delays.

Overall, the project has been a successful experience, and invaluable knowledge has been gained relating to the application of UHPC in bridge construction.

5. REFERENCES

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APPENDIX: PLANS FOR LITTLE CEDAR CREEK BRIDGE, WAPELLO COUNTY, IA

Figures 30 through 51 are the complete set of plans for the Little Cedar Creek Bridge.

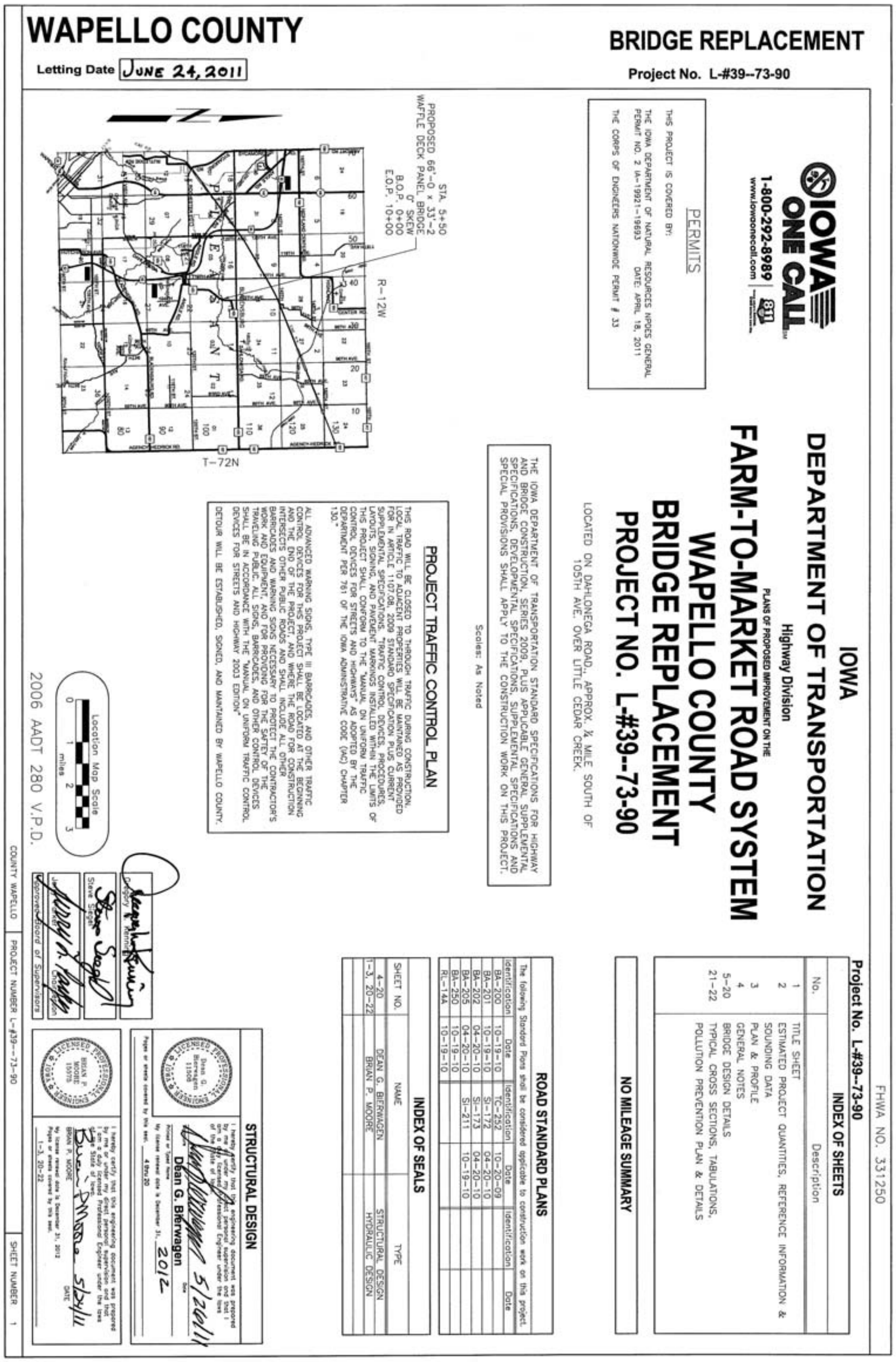

Figure 30. Diagram. Bridge plans, page 1.

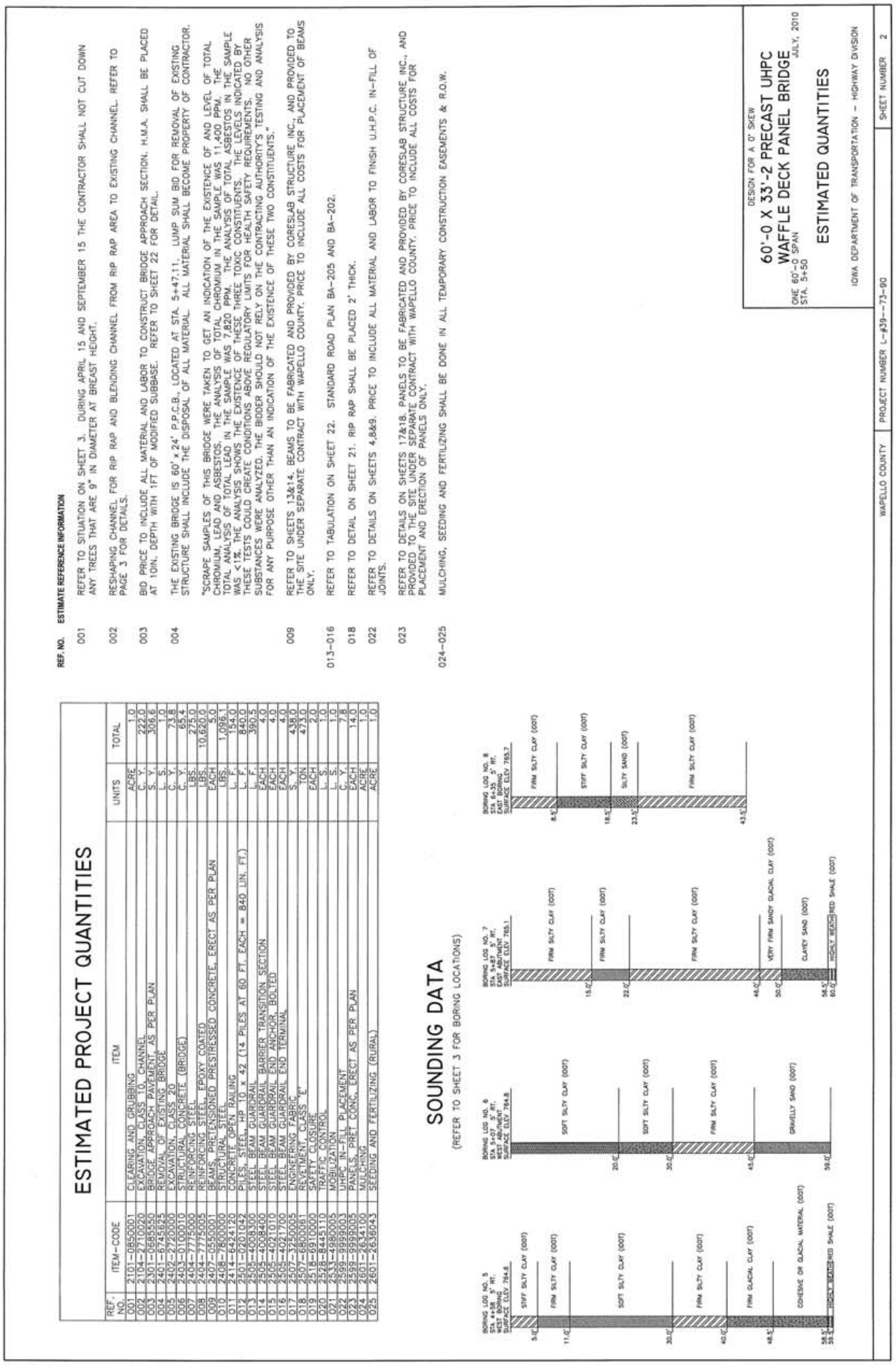

Figure 31. Diagram. Bridge plans, page 2.

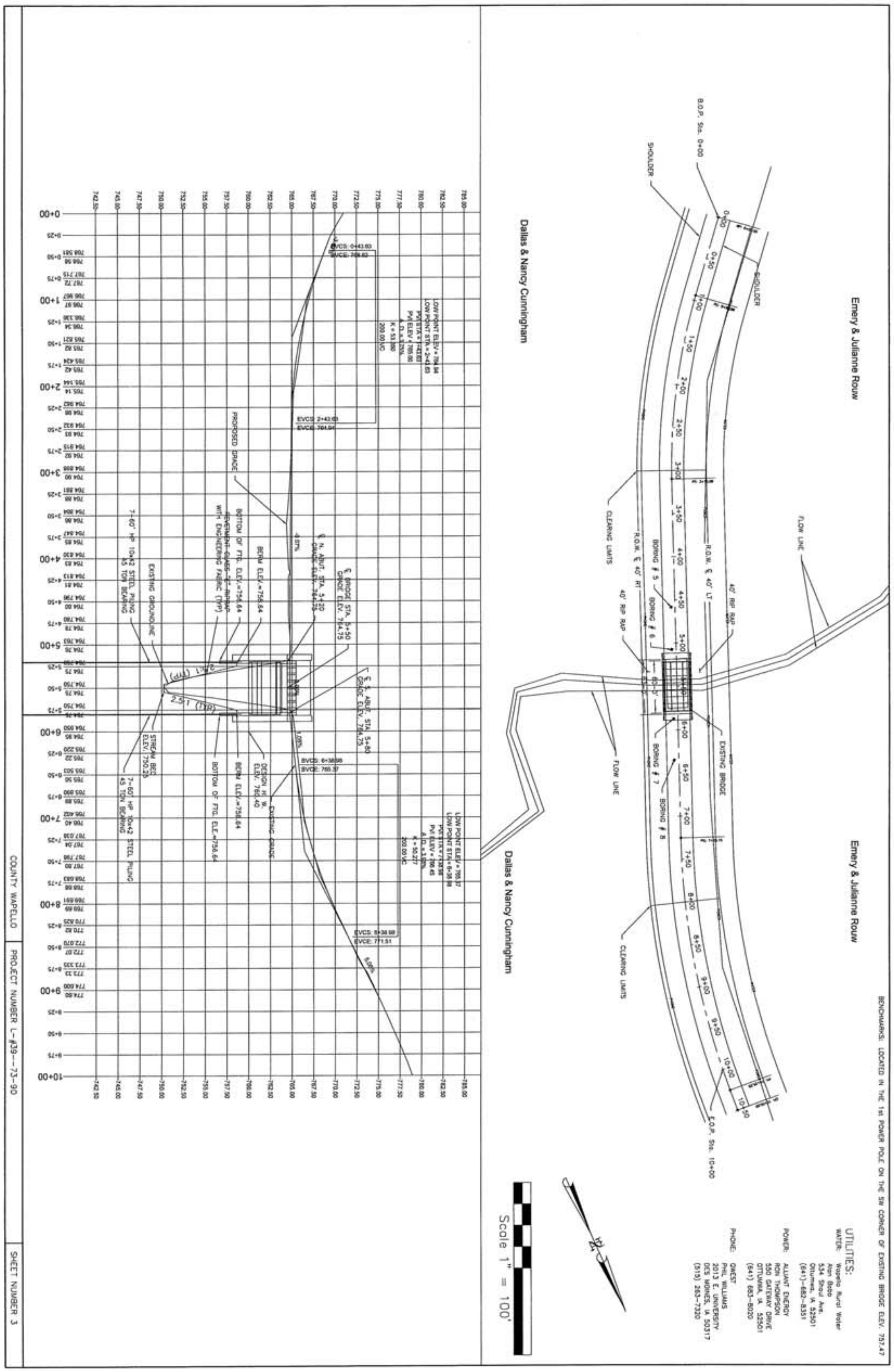

Figure 32. Diagram. Bridge plans, page 3.

GENERAL NOTES:

THIS DESIGN IS FOR THE REPLACEMENT OF THE EXISTING 60 FT. PRESTRESSED CONCRETE BRIDGE. THE EXISTING BRIDGE SUPERSTRUCTURE CONSISTS OF CONCRETE DECK, PRESTRESSED BEAMS AND TIMBER ABUTMENTS. THE REPLACEMENT BRIDGE SHALL BE A SINGLE SPAN PRESTRESSED WITH PRECAST ULTRA HIGH PERFORMANCE DECK PANELS.

THIS BRIDGE IS DESIGNED FOR HL-93 LOADING, PLUS 20 LBS. PER SQUARE FOOT OF ROADWAY FOR FUTURE WEARING SURFACE.

THESE BRIDGE PLANS LABEL ALL REINFORCING STEEL WITH ENGLISH NOTATION (5a1 IS $\frac{5}{8}$ INCH DIAMETER BAR). ENGLISH REINFORCING STEEL RECEIVED IN THE FIELD MAY DISPLAY THE FOLLOWING "BAR DESIGNATION". THE "BAR DESIGNATION" IS THE STAMPED IMPRESSION ON THE REINFORCING BARS, AND IS EQUIVALENT TO THE BAR DIAMETER IN MILLIMETERS.

ENGLISH SIZE	BAR DESIGNATION
3	10
4	13
5	16
6	19
7	22
8	25
9	29
10	32
11	36

THE CONTRACTOR SHALL VISIT THE SITE AND BE FAMILIAR WITH THE EXISTING CONDITIONS OF THE PROJECT. UTILITY LOCATIONS ARE APPROXIMATE ONLY. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATIONS OF UTILITIES PRIOR TO CONSTRUCTION. THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR ANY DAMAGE, WHICH MIGHT OCCUR DUE TO HIS FAILURE TO LOCATED AND PROTECT UNDERGROUND UTILITIES.

ACCESS SHALL BE MAINTAINED TO INDIVIDUAL PROPERTIES DURING CONSTRUCTION. THIS WORK SHALL BE CONSIDERED INCIDENTAL TO THE PROJECT.

THE PRECAST SLAB FOR THE BRIDGE IS CAST FROM ULTRA HIGH STRENGTH CONCRETE CALLED 'DUCTAL' MANUFACTURED BY CORESLAB STRUCTURES, LAPLATTE, NEBRASKA THE SLABS ARE BEING PURCHASED AND SUPPLIED TO THE JOB SITE UNDER A SEPARATE CONTRACT BETWEEN WAPELLO COUNTY AND CORESLAB STRUCTURES. THE CONTRACTOR SHALL CONTACT JOHN HEIMANN OF CORESLAB STRUCTURES, INC, 802 ALLIED ROAD, LAPLATTE, NE, 68123 OFFICE: (402)291-0733 EXT. 407 AND CELL PHONE: (402)960-0644 TO ARRANGE A TIME TO HAVE THE SLABS SHIPPED TO THE JOB SITE.

UPON DELIVERY TO THE JOB SITE EACH PANEL SHALL BE JOINTLY INSPECTED BY THE WAPELLO COUNTY ENGINEER, CORESLAB STRUCTURES AND THE BRIDGE CONTRACTOR FOR STRUCTURAL INTEGRITY AND ANY DAMAGE THAT MAY HAVE BEEN CAUSED BY SHIPMENT ANY DAMAGE SHALL BE REPAIRED BY CORESLAB. ONCE ACCEPTED BY THE WAPELLO COUNTY ENGINEER AND THE CONTRACTOR, THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PANELS DURING CONSTRUCTION AT THE BRIDGE SITE. THE CONTRACTOR SHALL BE RESPONSIBLE FOR REPLACEMENT OR REPAIR OF ANY PANELS DAMAGED DURING THE CONSTRUCTION AT THE BRIDGE SITE, WHICH SHALL INCLUDE THE LIFTING AND PLACEMENT ON THE BRIDGE BEAMS.

MINIMUM CLEAR DISTANCE FROM FACE OF CONCRETE TO NEAR REINFORCING BAR IS TO BE 2" UNLESS OTHERWISE NOTED OR SHOWN.

FAINT LINES ON PLANS INDICATE EXISTING STRUCTURE.

PRECAST PANEL HANDLING

LIFTING ANCHORS:
LIFTING ANCHORS FOR PRECAST PANELS SHALL BE APPROVED BY THE WAPELLO COUNTY ENGINEER PRIOR TO USE.

MECHANICAL SPLICES:
THE OPEN RAIL POSTS SHALL BE SPLICED AT THE DECK LOCATION SHOWN IN THE PLANS USING MECHANICAL SPLICE ASSEMBLIES. THE MECHANICAL SPLICE ASSEMBLY USED SHALL MEET THE REQUIREMENTS OF MATERIALS I.M.451 APPENDIX E.

LIFTING AND HANDLING:
PANELS SHALL BE HANDLED IN SUCH A MANNER AS NOT TO DAMAGE THE PANELS DURING LIFTING OR MOVING. LIFTING ANCHORS CAST INTO THE PANELS SHALL BE USED FOR LIFTING AND MOVING THE PANELS AT THE FABRICATION PLANT AND IN THE FIELD. THE ANGLE BETWEEN THE TOP SURFACE OF THE PANEL AND THE LIFTING LINE SHALL NOT BE LESS THAN SIXTY DEGREES, WHEN MEASURED FROM THE TOP SURFACE OF THE PANEL TO THE LIFTING LINE. DAMAGE CAUSED TO ANY PANEL, SHALL BE REPAIRED AT THE EXPENSE OF THE CONTRACTOR TO THE SATISFACTION OF THE WAPELLO COUNTY ENGINEER.

NOTES FOR UHPC FIELD CAST IN-FILLS JOINTS

REFER TO SPECIAL PROVISIONS SP-090112A.

PANEL INSTALLATION ON SITE:

EQUIPMENT:
THE CONTRACTOR SHALL HAVE ALL EQUIPMENT REQUIRED FOR PANEL INSTALLATION, AND UHPC IN-FILL ON-SITE PRIOR TO BEGINNING PANEL INSTALLATION.

PLACEMENT TECHNIQUE:

PANELS MAY BE PULLED TOGETHER DURING PLACEMENT USING APPROVED TEMPORARY DEVICES. ANY DAMAGE TO THE PANELS CAUSED BY TEMPORARY DEVICES SHALL BE REPAIRED AT THE CONTRACTOR'S EXPENSE TO THE SATISFACTION OF THE WAPELLO COUNTY ENGINEER.

PLACEMENT TOLERANCES:

THE CENTERLINE OF THE PANELS SHALL BE WITHIN $\frac{1}{4}$ INCH OF THE PRE-SURVEYED CENTERLINE MARKED ON THE SURFACE OF THE BASE. VERTICAL ALIGNMENT OF THE PANELS SHALL BE SUCH THAT THE TOP SURFACE OF AN INDIVIDUAL PANEL IS NO MORE THAN $\frac{1}{8}$ INCH HIGHER OR LOWER THAN THE TOP SURFACE OF AN ADJOINING PANEL AT ANY POINT ALONG THE JOINT BETWEEN THE PANELS.

PATCHING AND REPAIRS:

DAMAGE CAUSED TO THE PRECAST PANELS DURING ANY PART OF THE PANEL INSTALLATION PROCESS SHALL BE REPAIRED BY THE CONTRACTOR AT THE CONTRACTOR'S EXPENSE TO THE SATISFACTION OF THE WAPELLO COUNTY ENGINEER USING APPROVED REPAIR METHODS AND MATERIALS. REPAIRS OF DAMAGED AREAS WILL BE ADDRESSED ON A CASE-BY-CASE BASIS BY THE WAPELLO COUNTY ENGINEER. ANY PATCH IF REQUIRED SHALL BE FINISHED FLUSH WITH THE SURFACE OF THE SURROUNDING CONCRETE AND GIVEN A TEXTURE SIMILAR TO THE SURROUNDING CONCRETE.

LEVELING DEVICE:
THE CONTRACTOR SHALL WORK WITH CORESLAB STRUCTURES INC. THE SUPPLIER OF THE PRECAST PANELS TO DEVELOP A LEVELING DEVICE FOR EACH PANEL TO PROPERLY PLACE THE PANELS TO THE CORRECT ELEVATION. COST OF THE LEVELING DEVICES SHALL BE INCLUDED IN THE BID PRICE "PANELS, PRET CONC. AS PER PLAN".

SPECIFICATIONS:

DESIGN: AASHTO LRFD SERIES OF 2007.
CONSTRUCTION: IOWA DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR HIGHWAY AND BRIDGE CONSTRUCTION, SERIES 2009, PLUS APPLICABLE GENERAL SUPPLEMENTAL SPECIFICATIONS, DEVELOPMENTAL SPECIFICATIONS, SUPPLEMENTAL SPECIFICATIONS AND SPECIAL PROVISIONS SHALL APPLY TO CONSTRUCTION WORK ON THIS PROJECT.

DESIGN STRESSES:

DESIGN STRESSES FOR THE FOLLOWING MATERIALS ARE IN ACCORDANCE WITH THE AASHTO LRFD SPECIFICATIONS FOR HIGHWAY BRIDGES, SERIES OF 2007.
REINFORCING STEEL IN ACCORDANCE WITH SECTION 5, GRADE 60.
CAST-IN-PLACE CONCRETE IN ACCORDANCE WITH SECTION 5, f'c = 3,500 PSI.
PRECAST CONCRETE BEAMS IN ACCORDANCE WITH SECTION 5, f'c = 7000 PSI.

DESIGN FOR A 0° SKEW
60'-0 X 33'-2 PRECAST UHPC
WAFFLE DECK PANEL BRIDGE
ONE 60'-0 SPAN
STA. 5+50
JULY, 2010
GENERAL NOTES
IOWA DEPARTMENT OF TRANSPORTATION – HIGHWAY DIVISION

DESIGN TEAM DGB • MSF | WAPELLO COUNTY | PROJECT NUMBER L-#39--73-90 | SHEET NUMBER 4
8/4/2010 8:02:14 AM dbierwa W:\Highway\Bridge\FinalSections\Final1\FISHERS\Waffle_Slab\Waffle_Slab.brg UHPC_Waffle_Slab_s002

Figure 33. Diagram. Bridge plans, page 4.

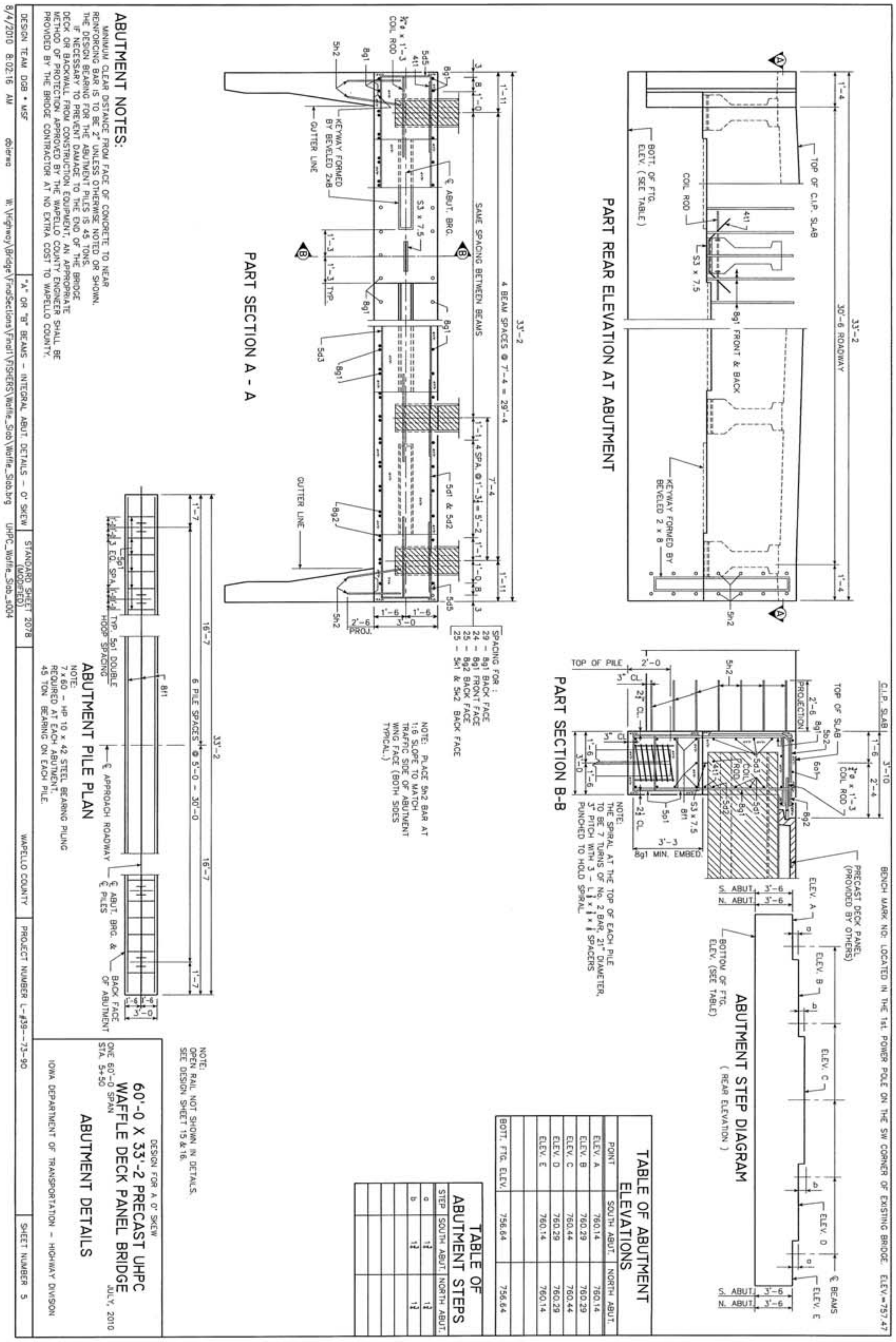

Figure 34. Diagram. Bridge plans, page 5.

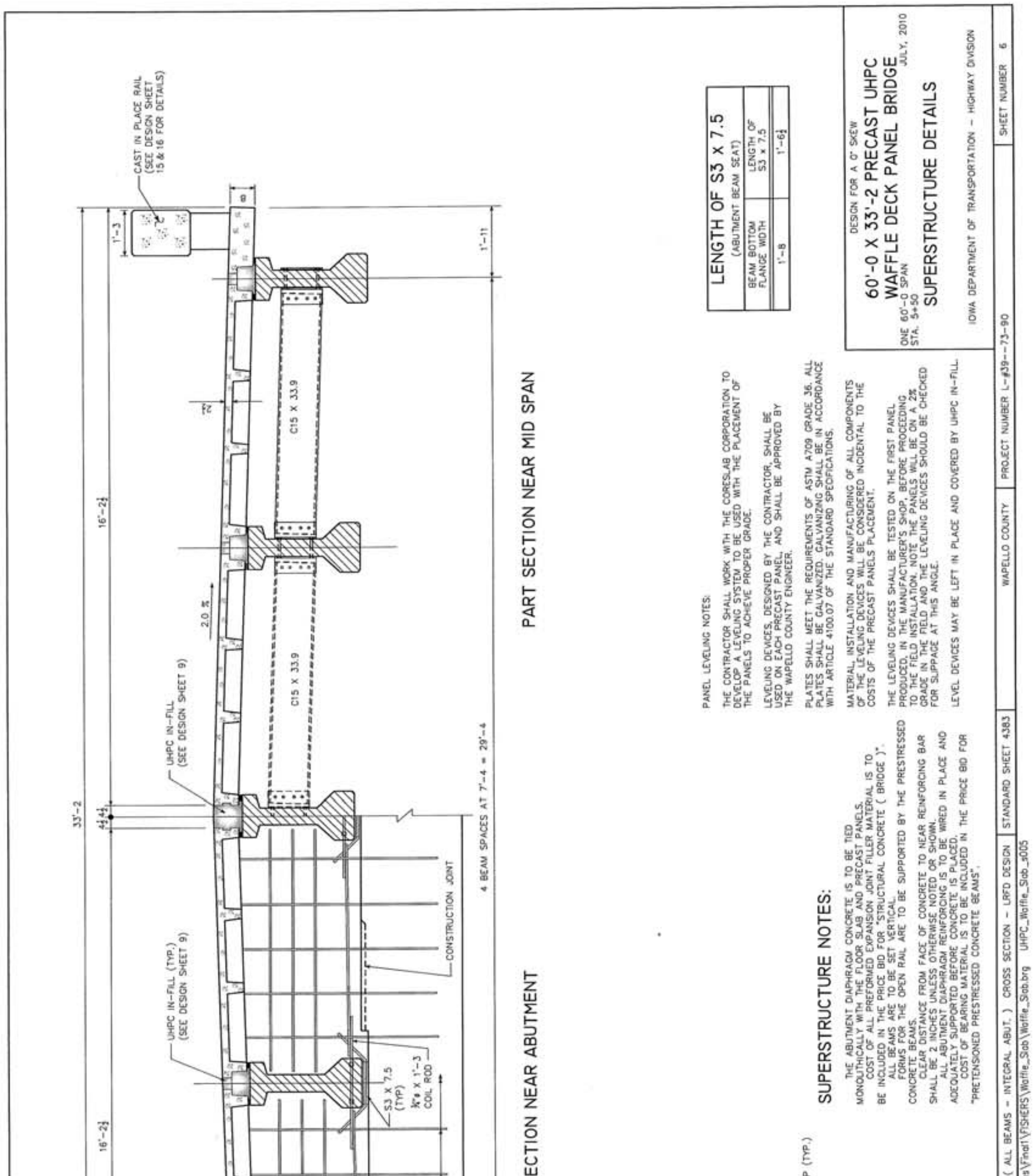

Figure 35. Diagram. Bridge plans, page 6.

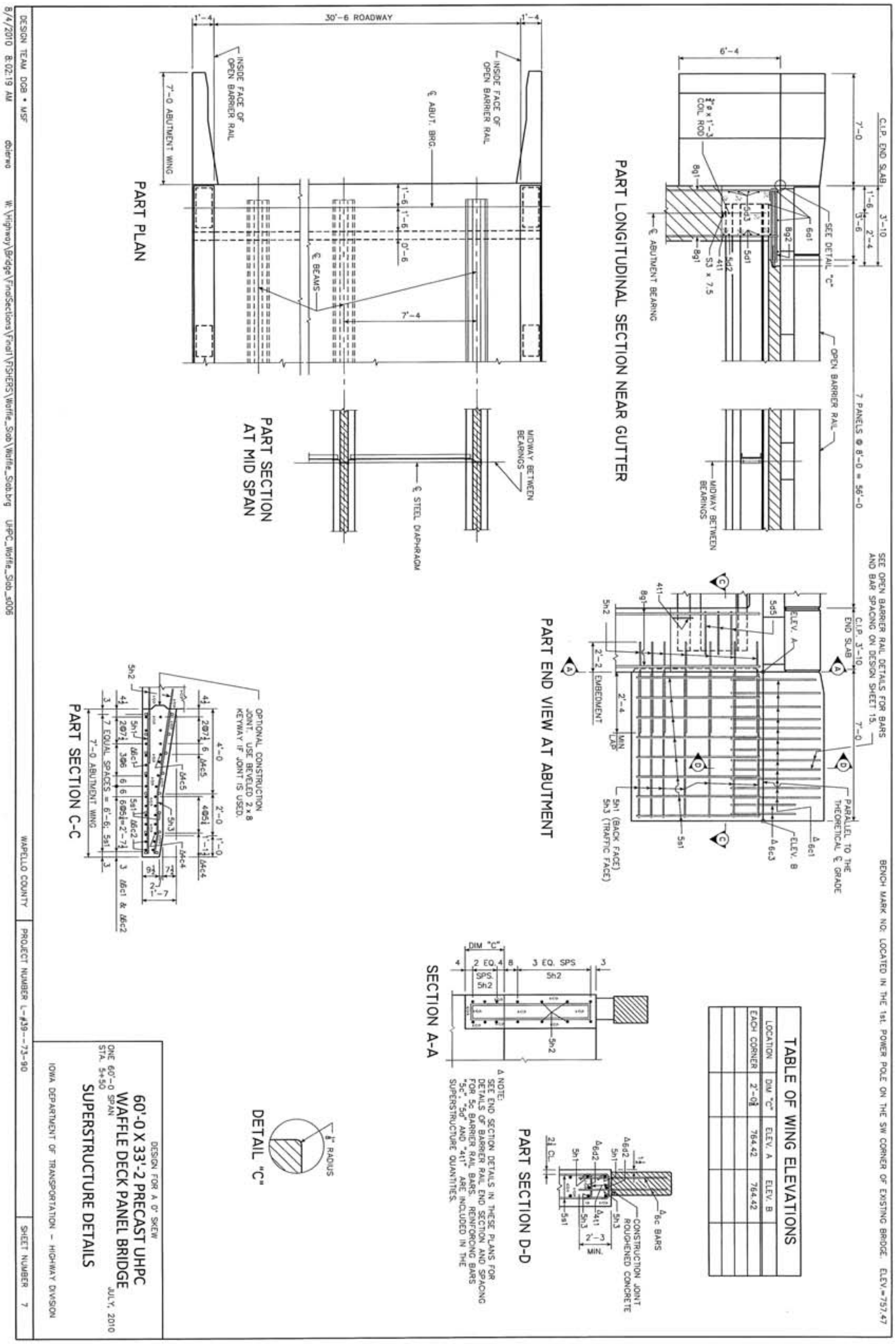

Figure 36. Diagram. Bridge plans, page 7.

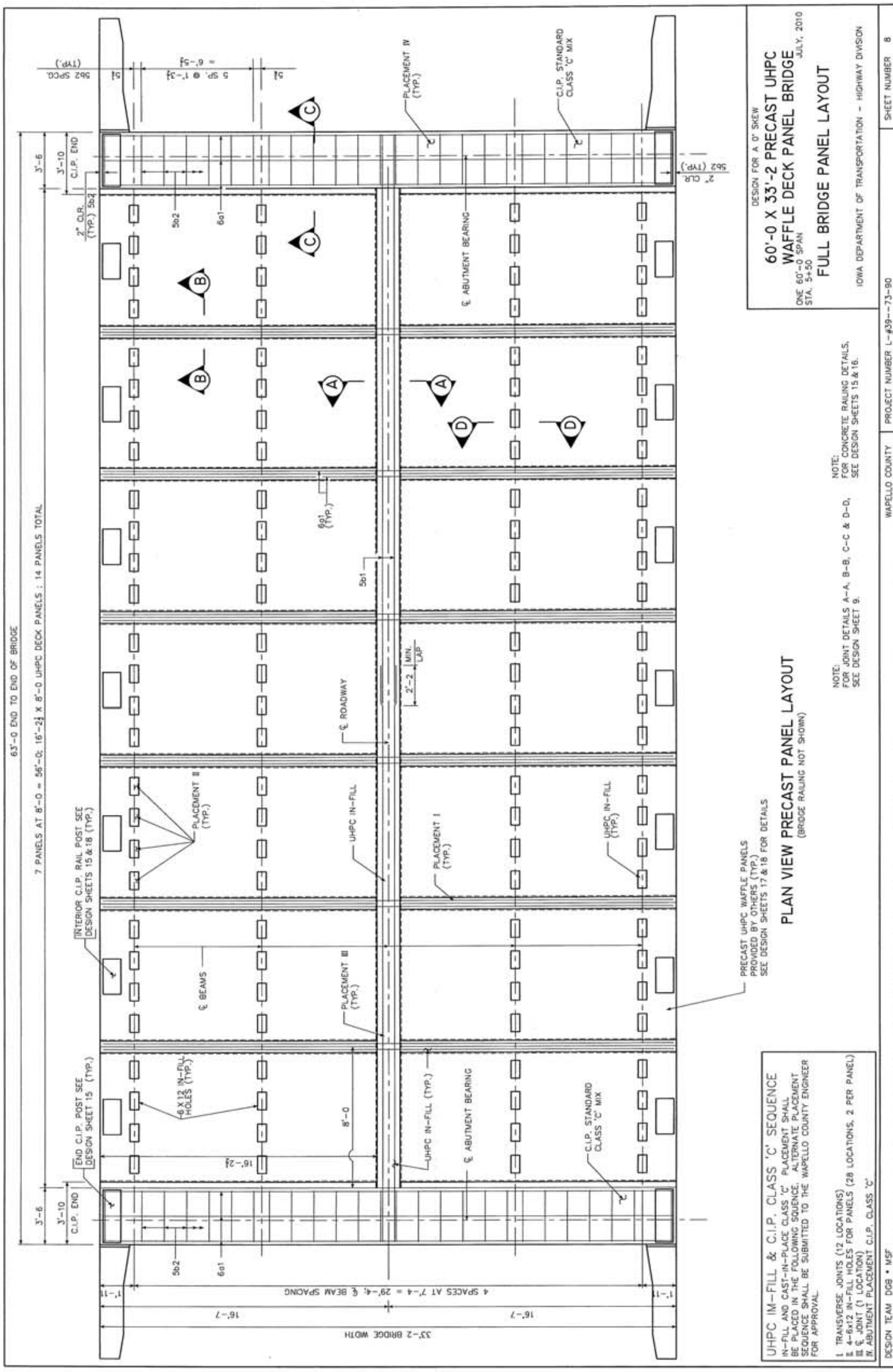

Figure 37. Diagram. Bridge plans, page 8.

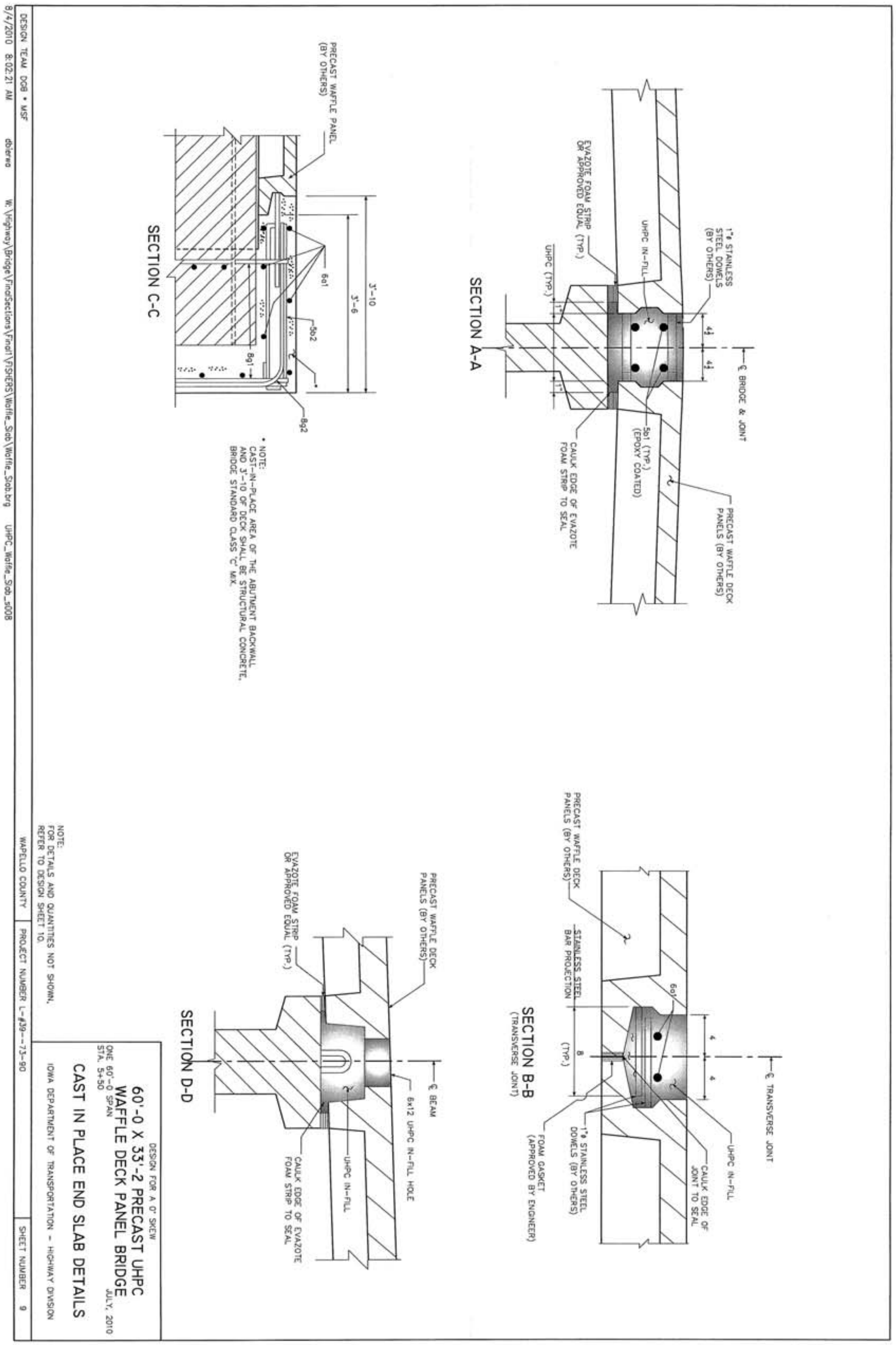

Figure 38. Diagram. Bridge plans, page 9.

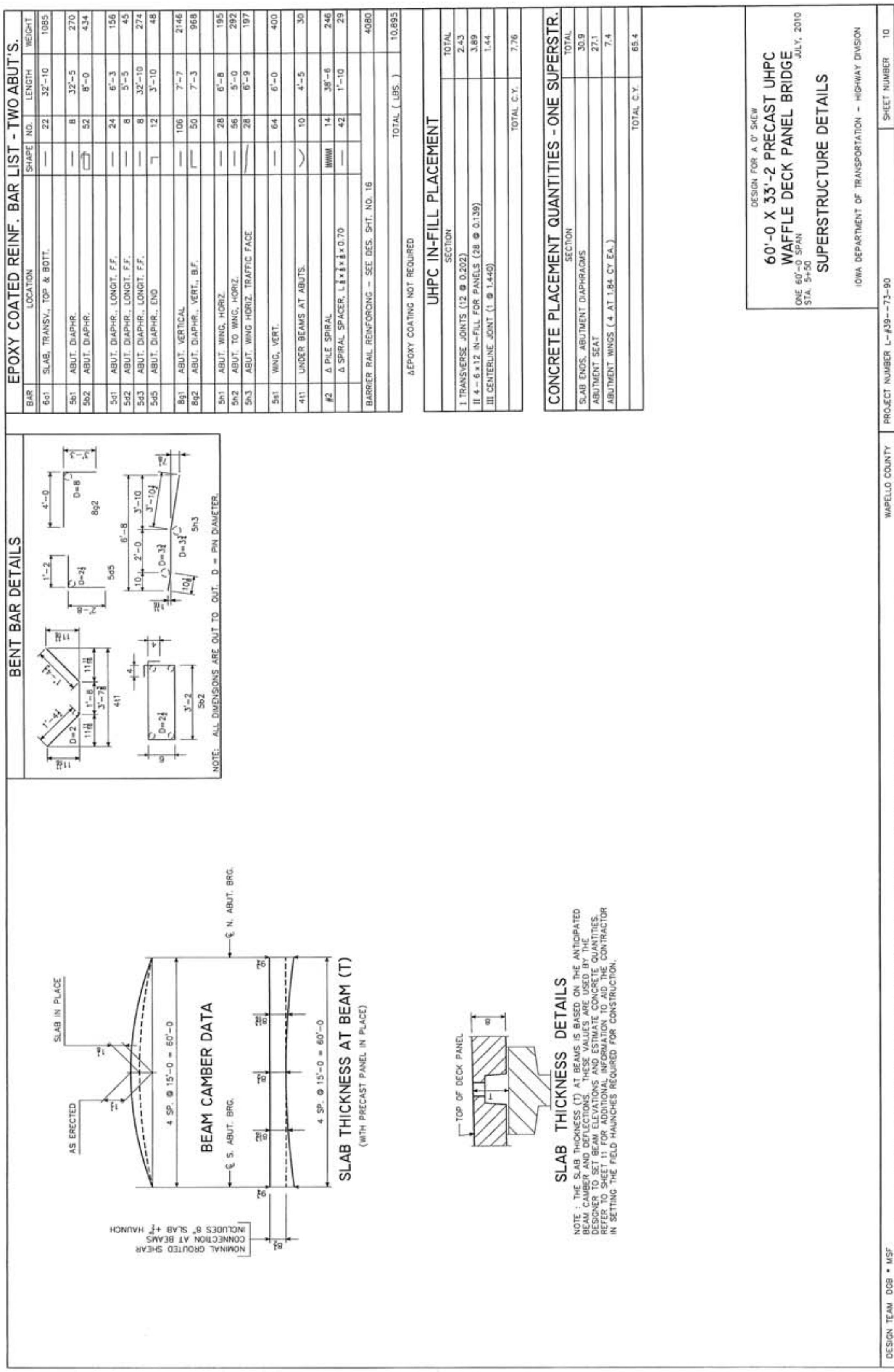

Figure 39. Diagram. Bridge plans, page 10.

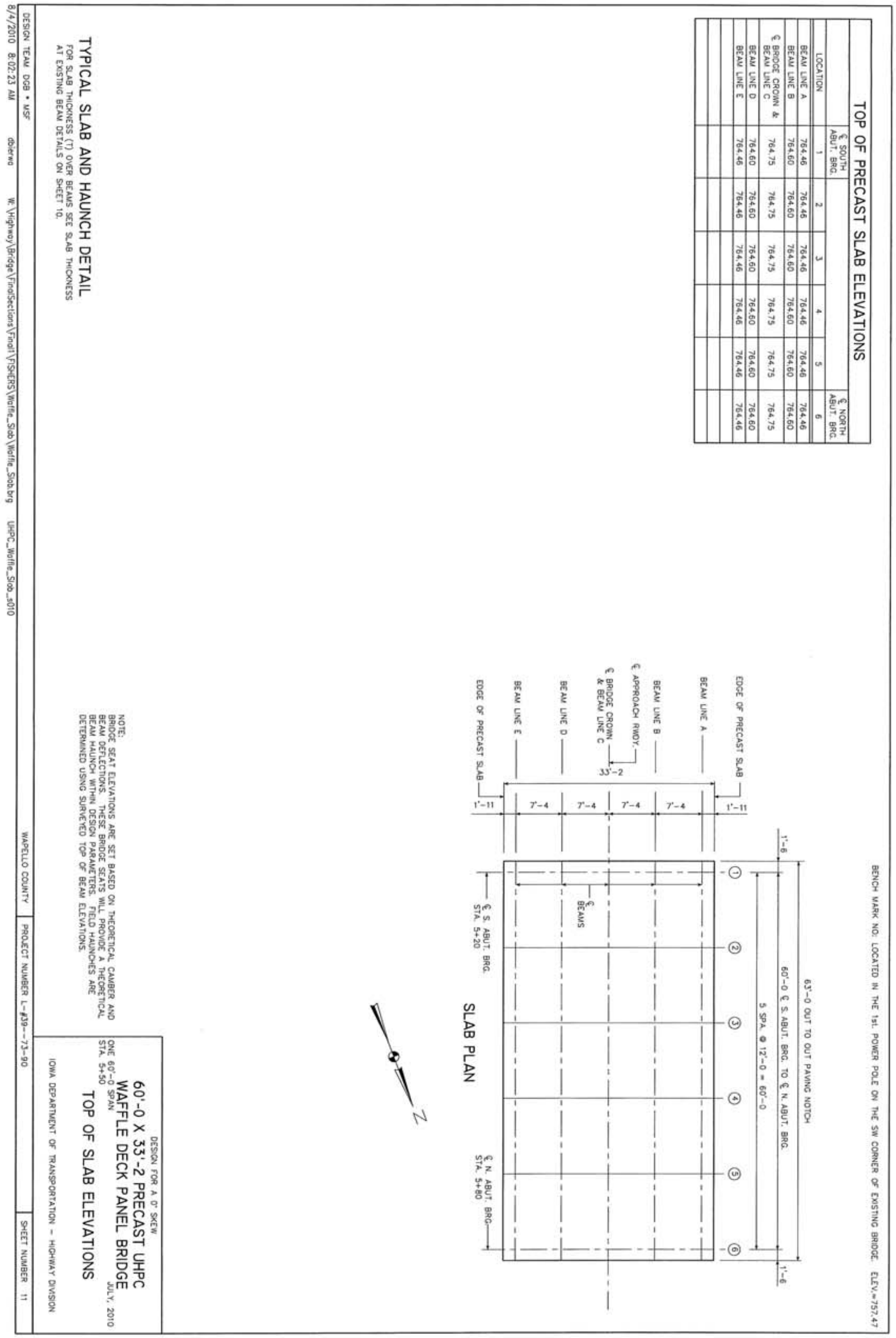

Figure 40. Diagram. Bridge plans, page 11.

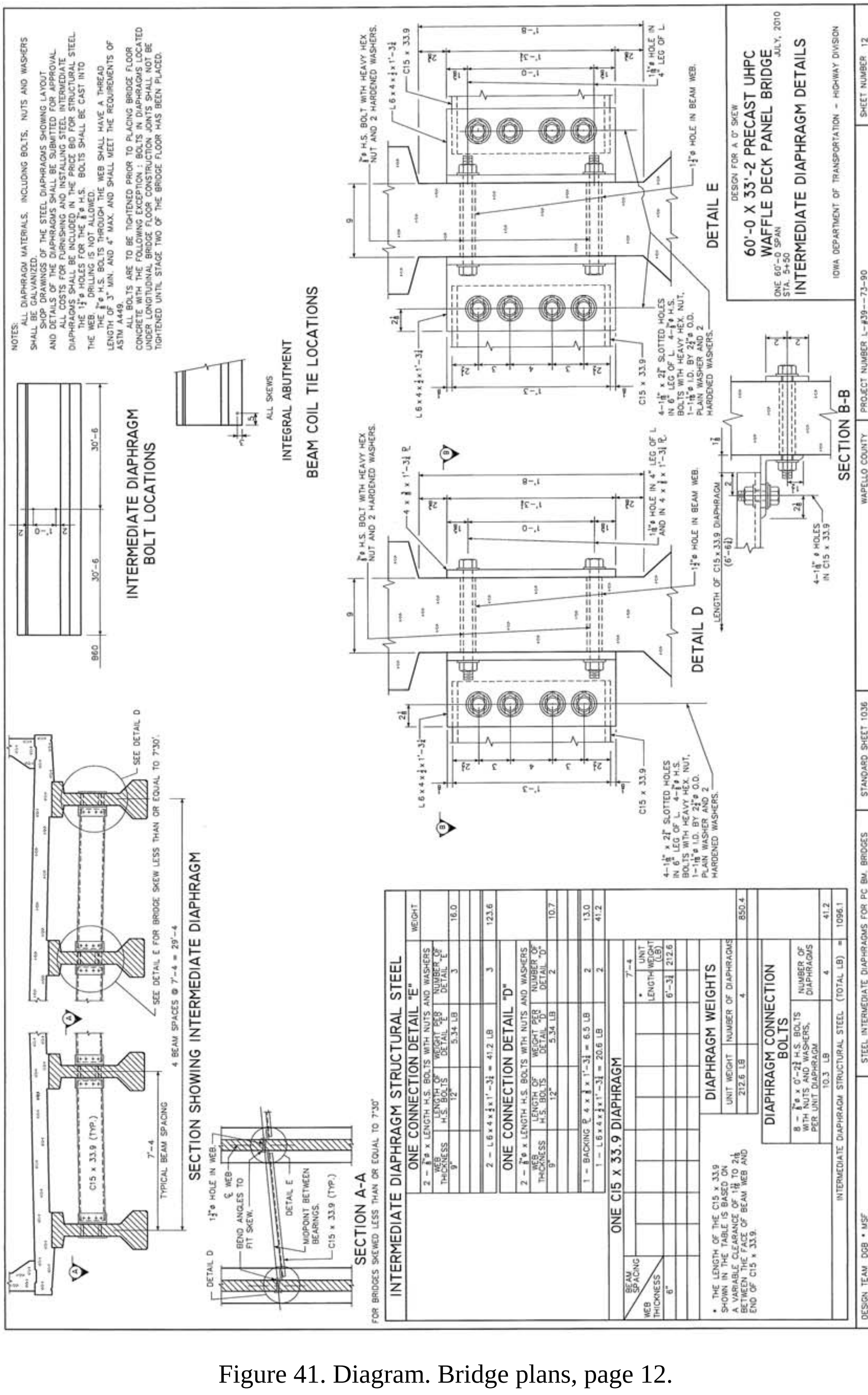

Figure 41. Diagram. Bridge plans, page 12.

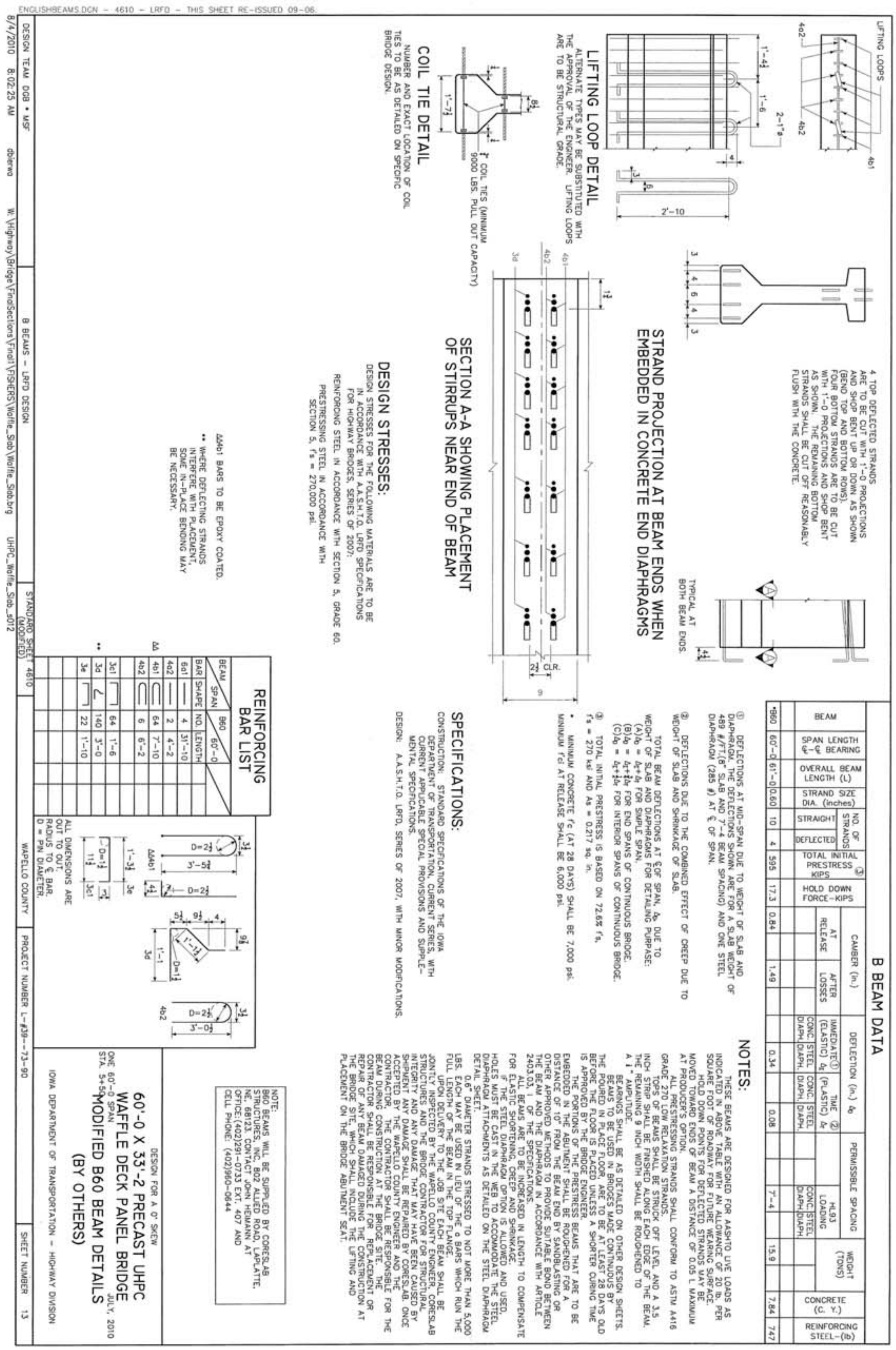

Figure 42. Diagram. Bridge plans, page 13.

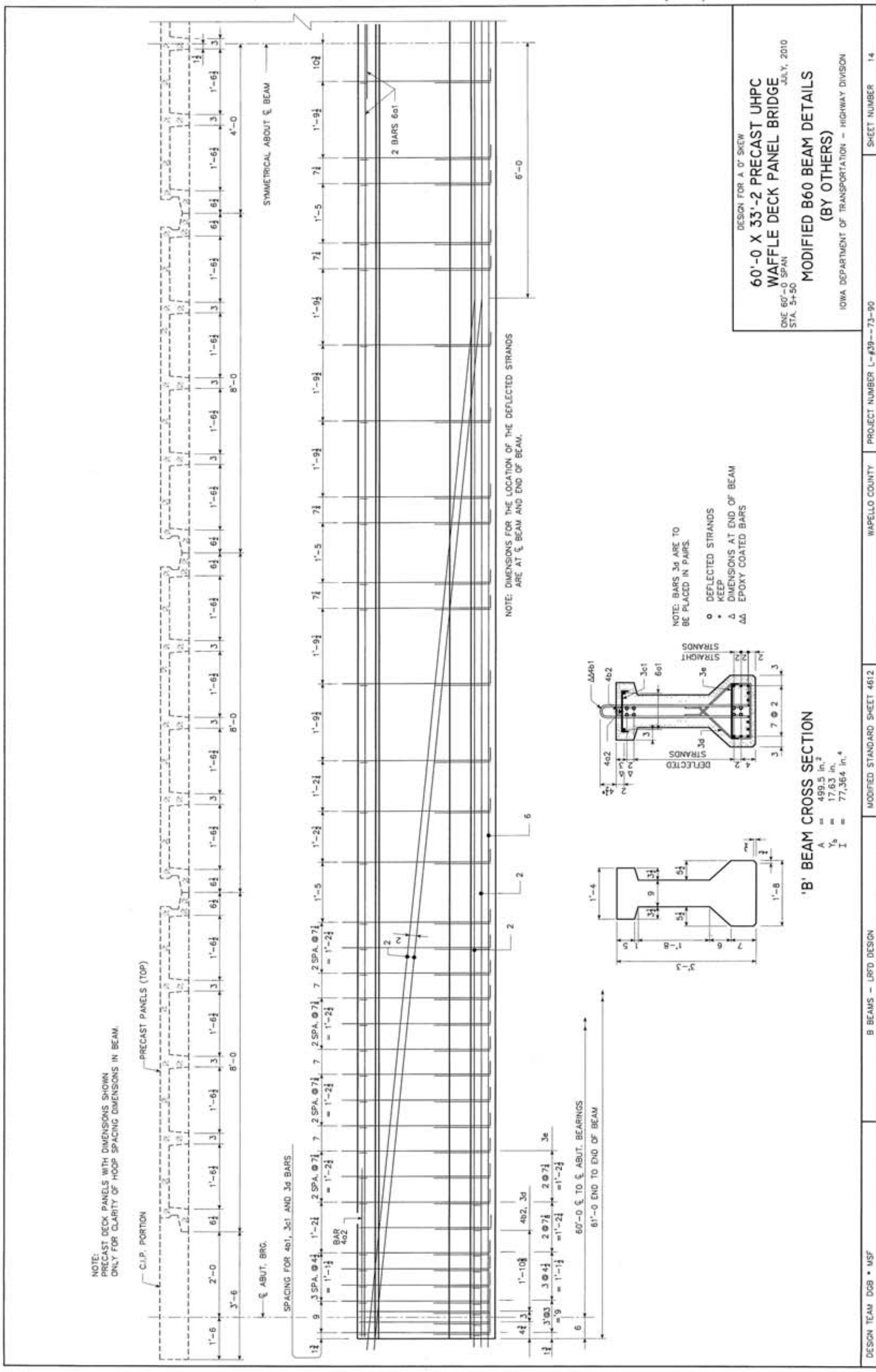

Figure 43. Diagram. Bridge plans, page 14.

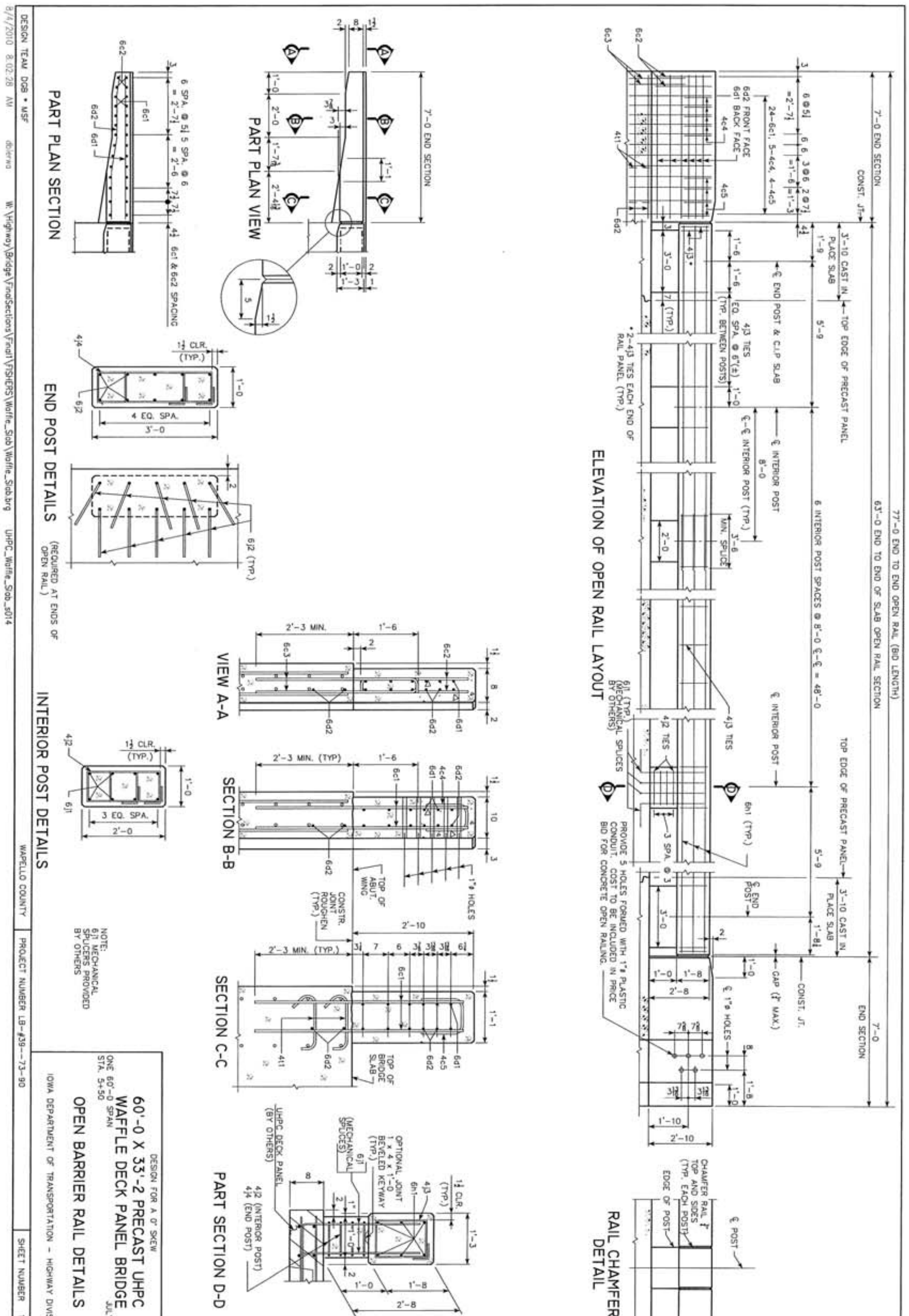

Figure 44. Diagram. Bridge plans, page 15.

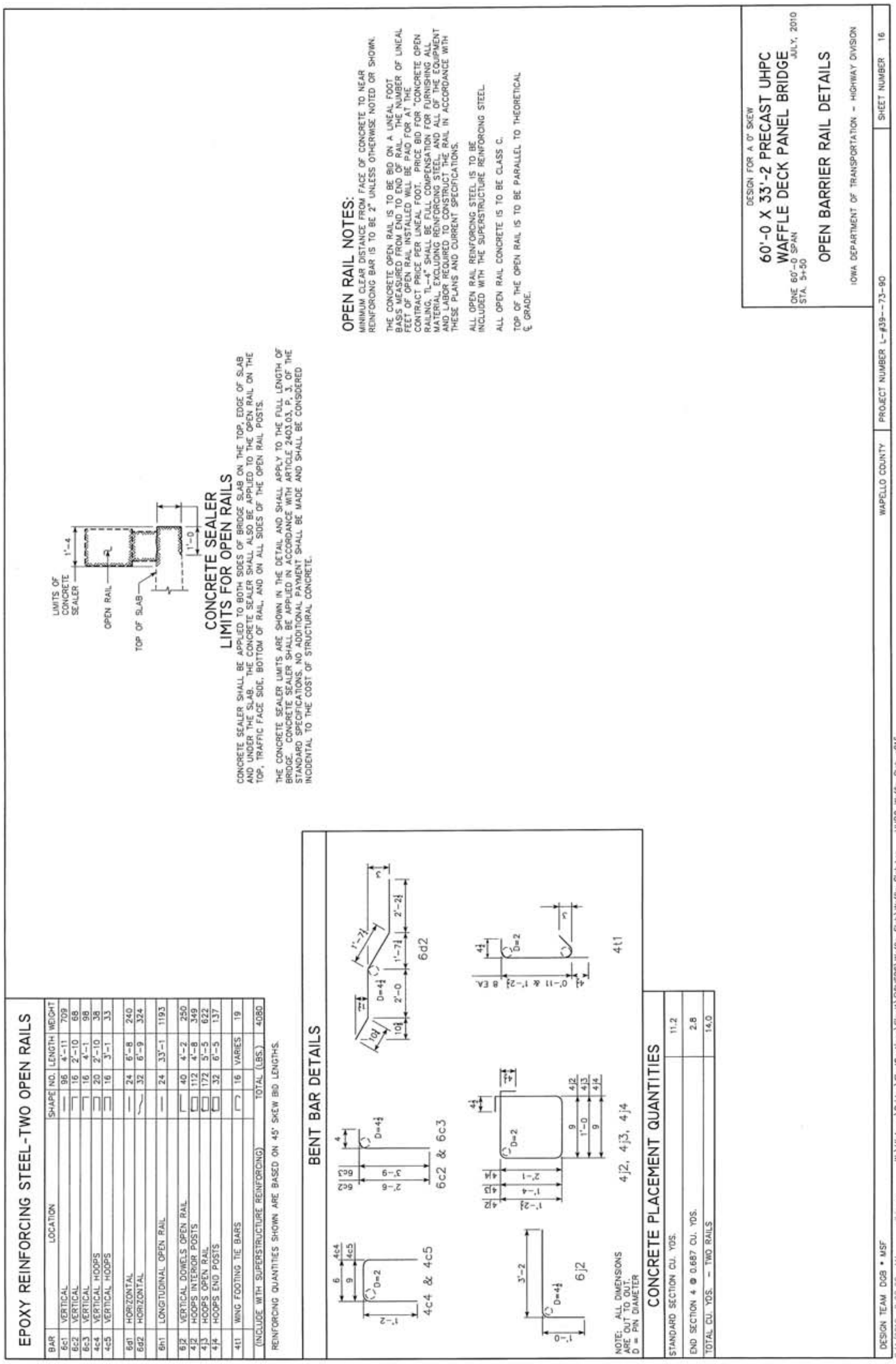

Figure 45. Diagram. Bridge plans, page 16.

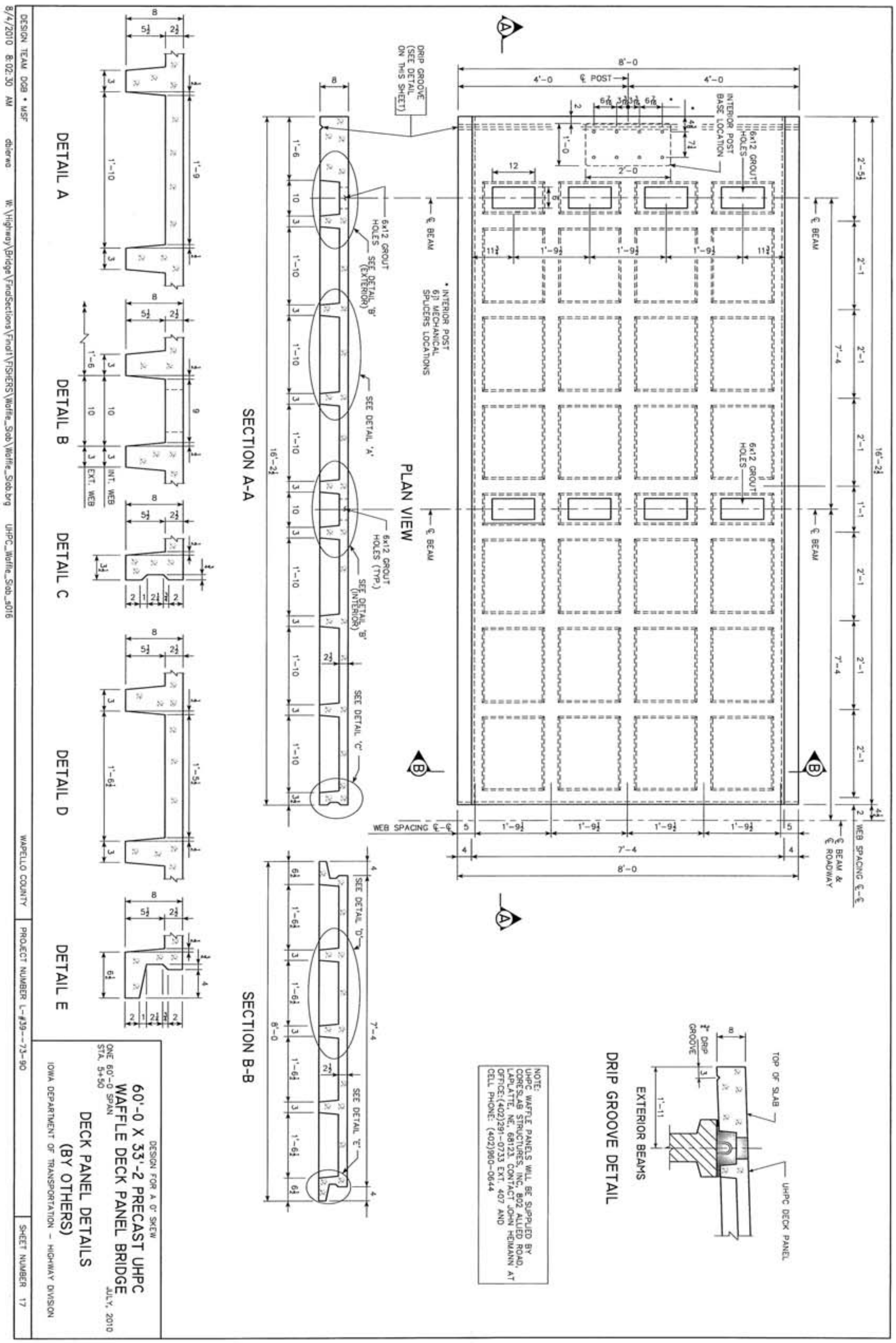

Figure 46. Diagram. Bridge plans, page 17.

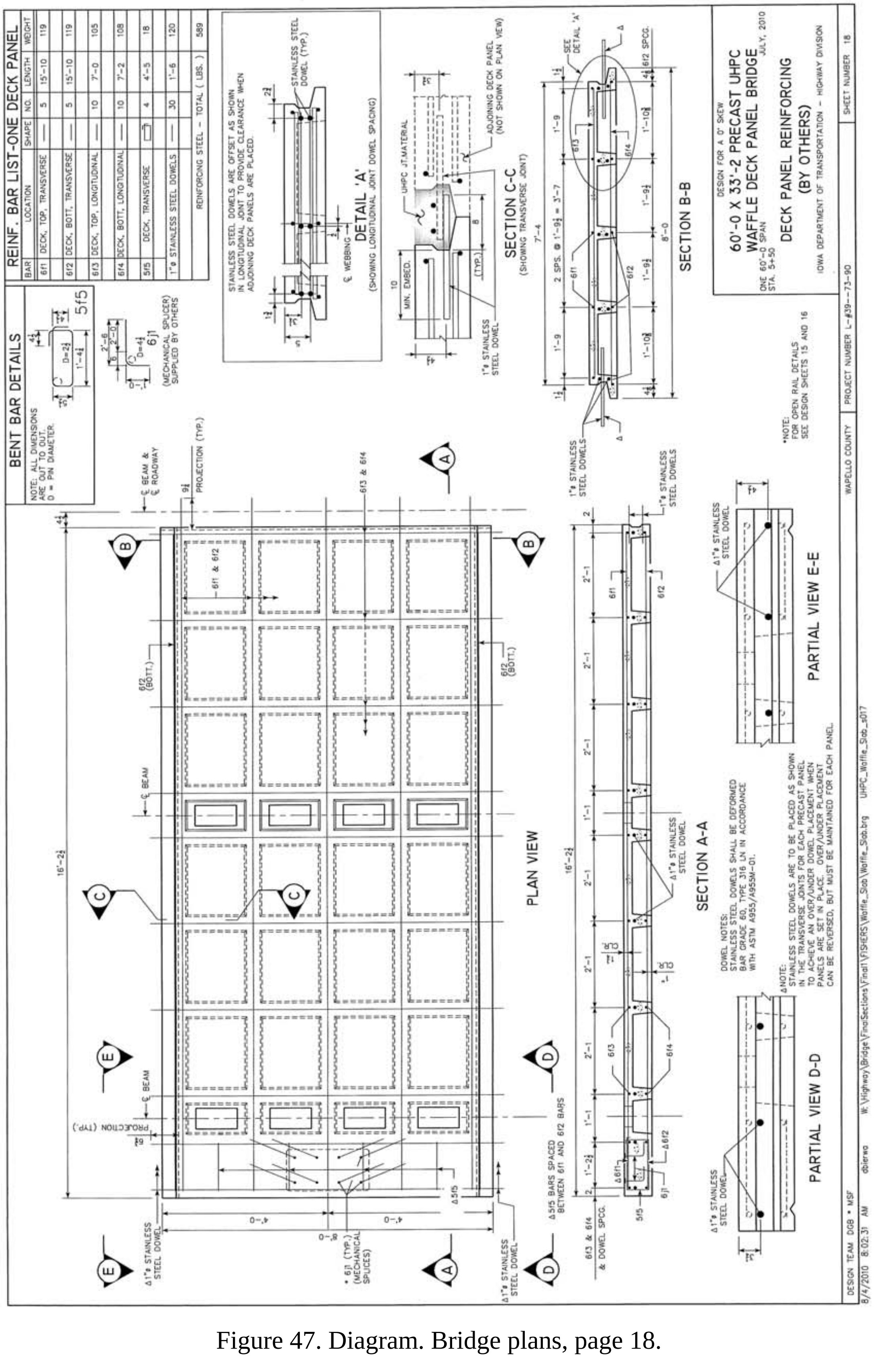

Figure 47. Diagram. Bridge plans, page 18.

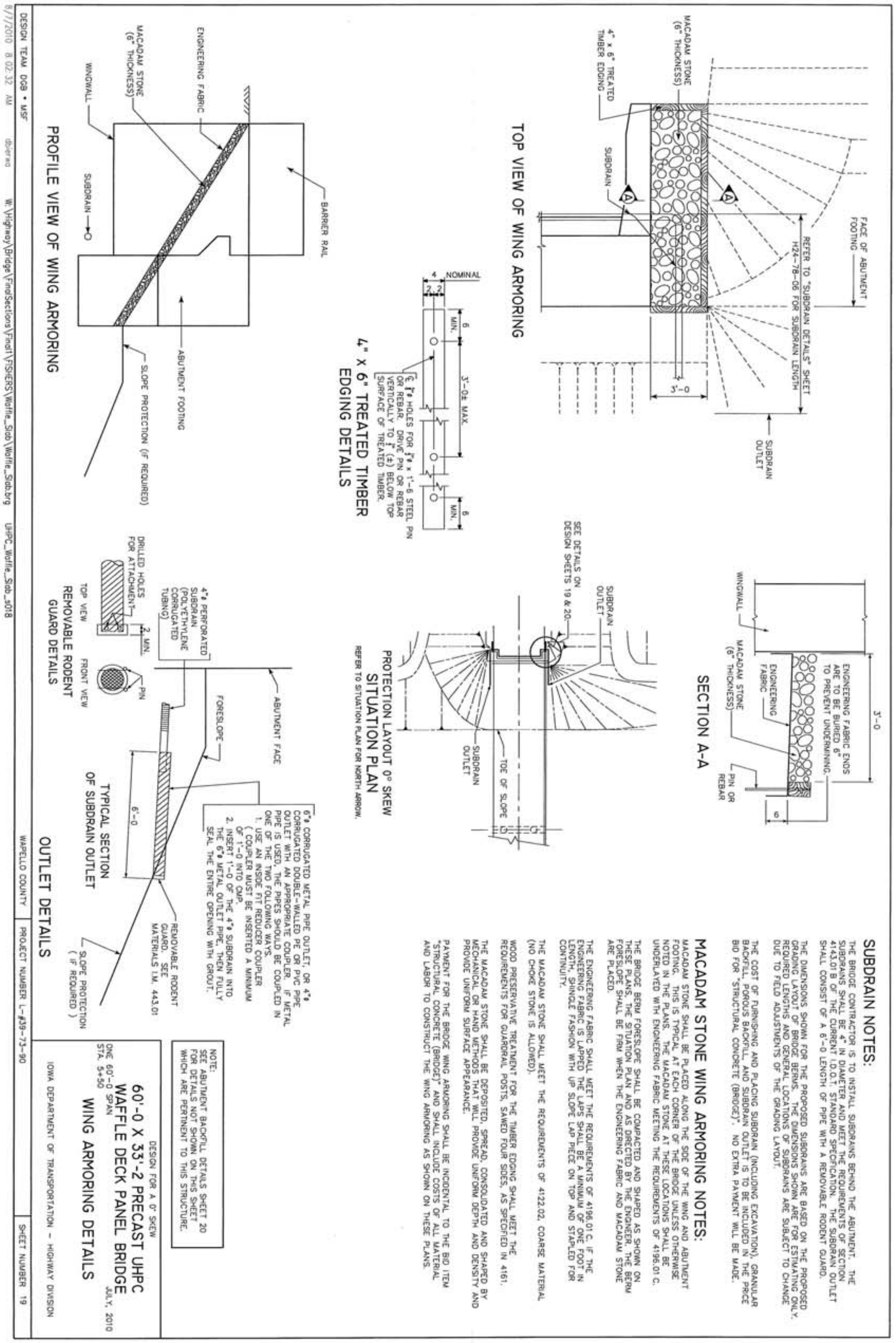

Figure 48. Diagram. Bridge plans, page 19.

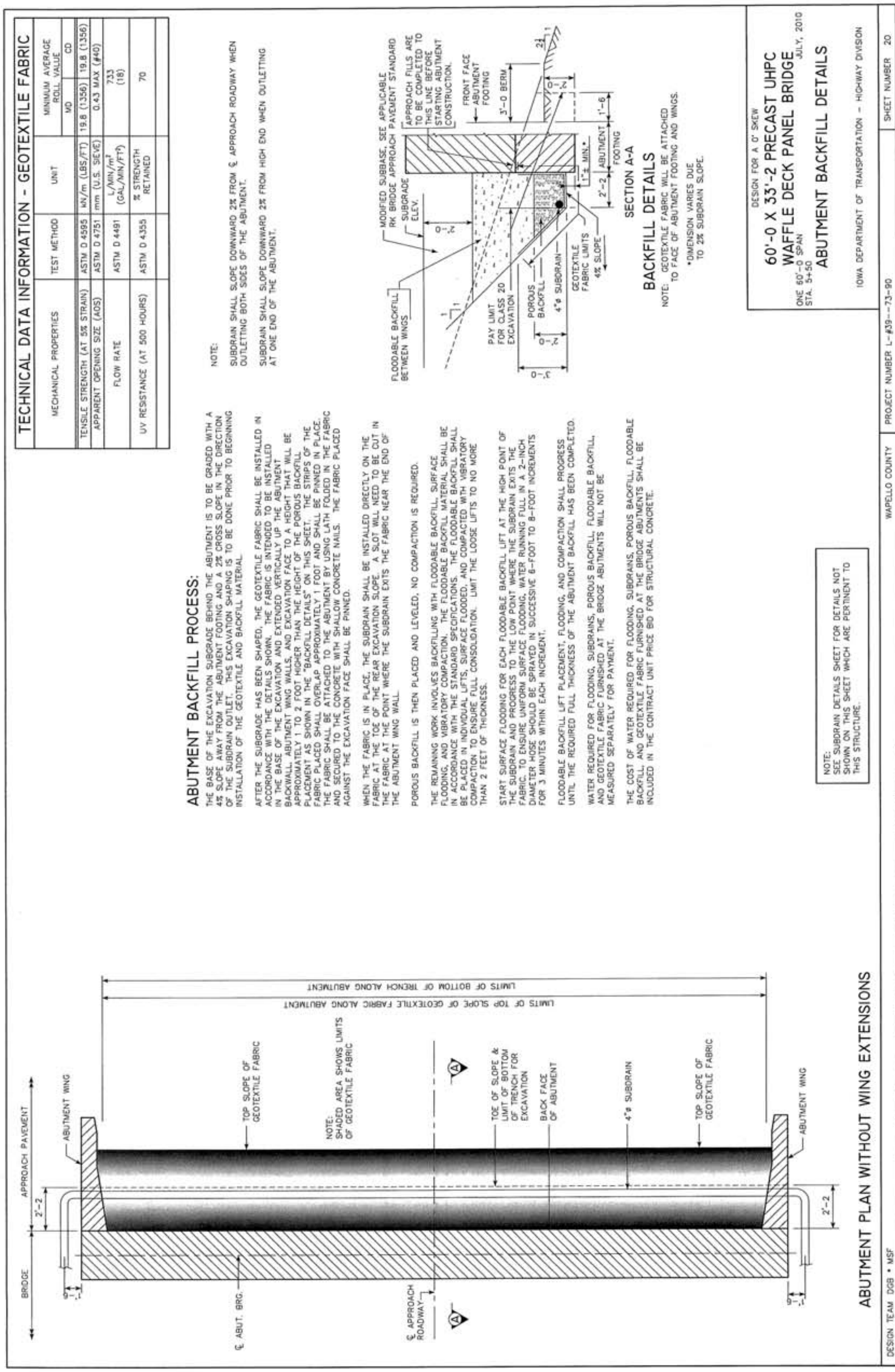

Figure 49. Diagram. Bridge plans, page 20.

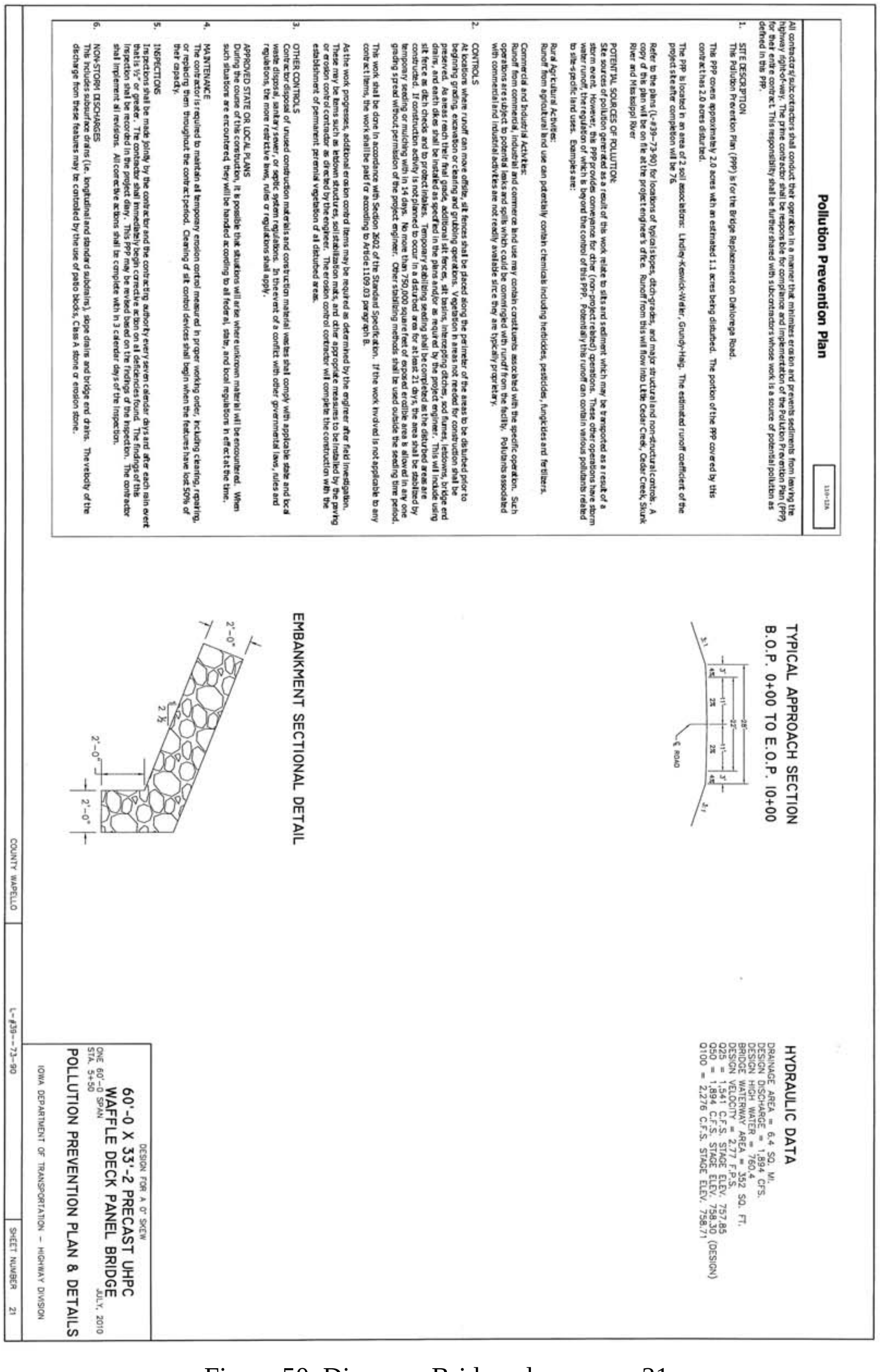

Figure 50. Diagram. Bridge plans, page 21.

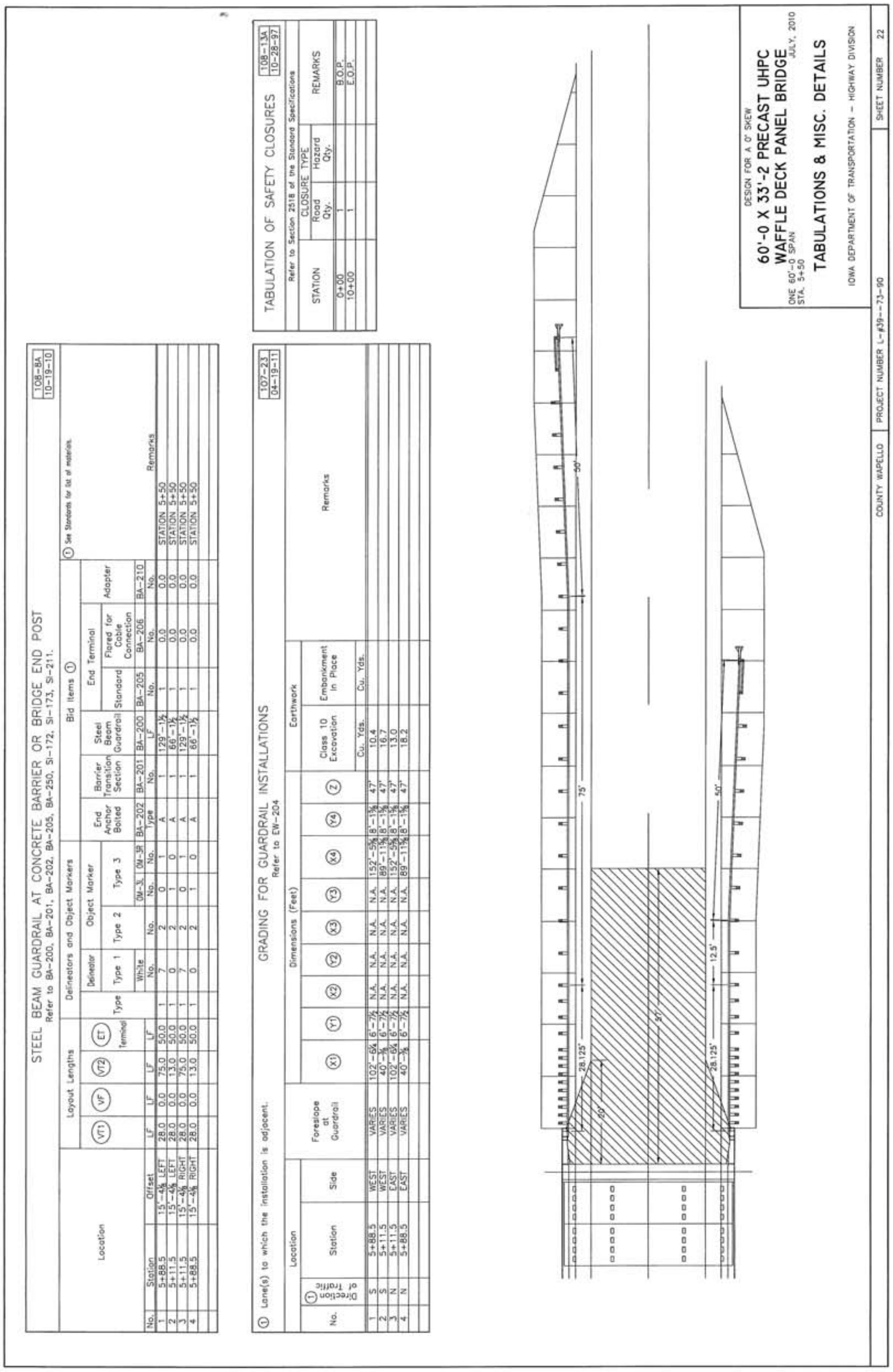

Figure 51. Diagram. Bridge plans, page 22.