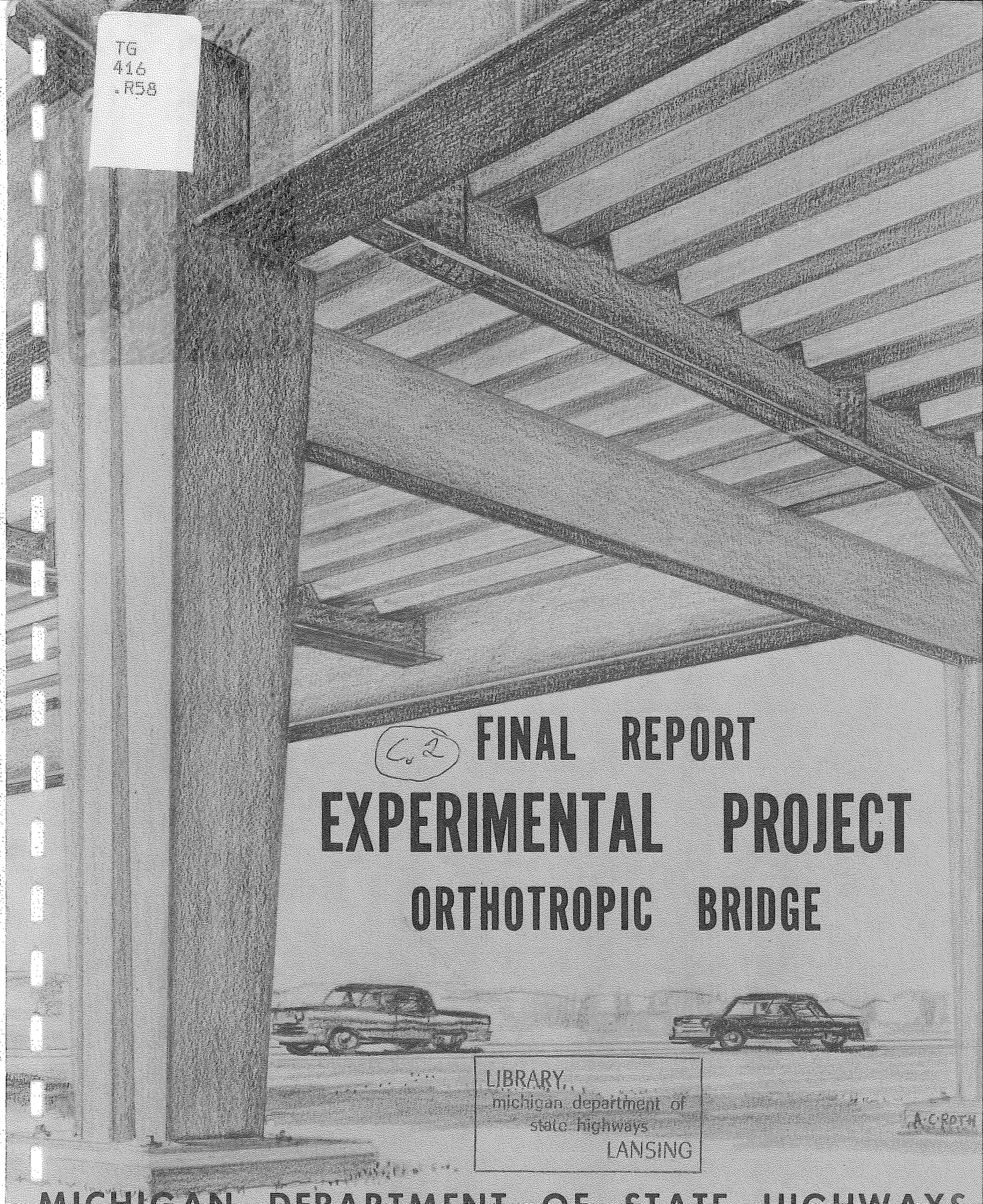


C. 2

FINAL REPORT
EXPERIMENTAL PROJECT
ORTHOTROPIC BRIDGE

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

FINAL REPORT

OF

EXPERIMENTAL ORTHOTROPIC BRIDGE

S05 of 23081 A

Crietz Road crossing over I-496
three miles west of the city limits
of Lansing

FEDERAL PROJECT I 496 - 7(21) 138

STATE RESEARCH PROJECT 67 G - 157

TO

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

Compiled by

John E. Risch

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

January 1970
Revised November 1971

TABLE OF CONTENTS

INTRODUCTION TO MICHIGAN'S FIRST ORTHOTROPIC BRIDGE

F. D. RIEGER, SQUAD LEADER

DEFINITION OF ORTHOTROPIC

An orthotropic steel deck plate bridge has a thin steel deck plate stiffened in two mutually perpendicular directions by a system of longitudinal ribs and transverse floor beams that are welded to the deck plate. The steel deck is considered as an integral part of the main carrying members of the bridge and acts as their flange.

As the rigidities of the ribs and floor beams are generally of unequal magnitude, elastic behavior is different in each of these two main directions. (Anisotropy). Because the ribs and floor beams are at right angles (orthogonal) the whole system has become known as orthogonal - anisotropic; or, briefly, "orthotropic".

WHY ORTHOTROPIC?

During the past few years there has been increased emphasis placed on traffic safety and aesthetics. One trend that has developed from this is the elimination of obstructions along the highways. This has affected the design of grade separations in that the side piers have been moved further from the roadway or eliminated, this resulting in longer spans. In addition, the problem of deterioration of concrete bridge decks has been a problem of much concern.

It has therefore become necessary to develop new types of bridges which will provide the longer spans and be more maintenance free as well as more pleasing in appearance. One type of design which has the capabilities of providing these features is the orthotropic deck type bridge.

With optimum use of steel and welding, orthotropic construction results in a superstructure weight of about one-half of a conventional bridge superstructure. This reduction in dead weight is accomplished primarily by the substitution of a relatively light weight steel deck for the heavy reinforced concrete deck. The steel deck is stiffened by means of longitudinal ribs which span between transverse floor beams. These, in turn, are supported by two main longitudinal girders. The deck plate acts as a top flange for the ribs, floor beams and main girders, as well as supporting the live load on the bridge.

This type of design was used for the Crietz Road bridge which crosses I-496 three miles west of Lansing and which has now been opened to traffic. Although the Orthotropic type design is considered to be more economical for longer spans than for the spans required for the Crietz Road bridge, it was felt that a relatively small bridge would be more ideal to point out and identify construction and maintenance problems inherent in orthotropic plate construction. Also, the adjacent conventional structures would offer a unique opportunity to observe the comparative action under traffic.

BRIDGE DESCRIPTION

The Crietz Road bridge is a two-span continuous structure with spans of 96'-0" and a clear roadway of 32'-6" with two 9" wide brush curbs.

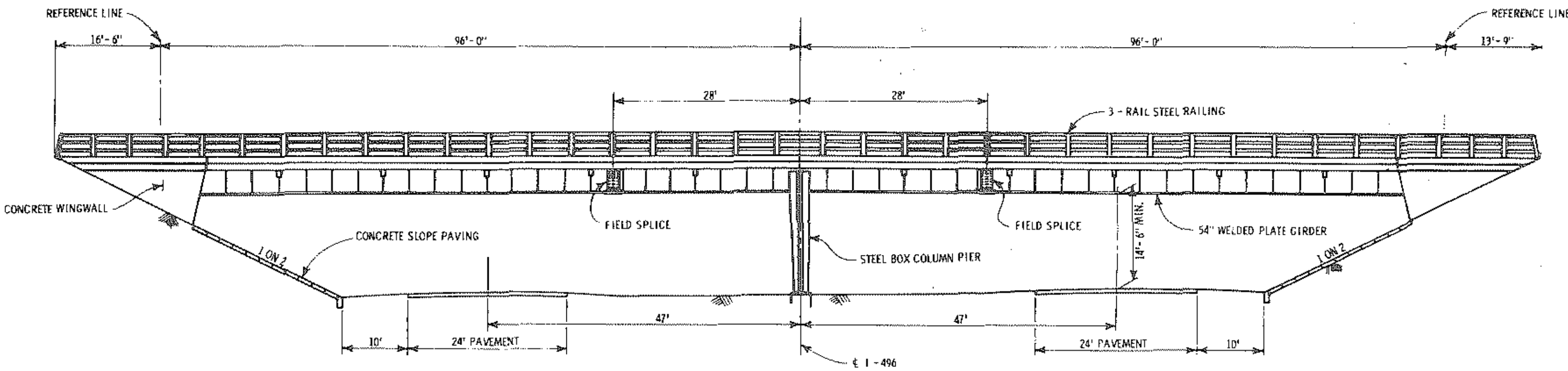
The superstructure consists of a 7/16" steel deck plate supported by two 54" deep welded steel plate girders spaced at 24'-0", and 24" deep transverse floor beams spaced at 15'-7½" on centers. The deck plate is stiffened by 5/16" thick by 9" deep longitudinal trapezoidal shaped stiffeners on 2'-0" centers. The floor beams are notched to allow the stiffeners to run continuous throughout the length of the bridge. The stiffeners are shop welded to the floor beams and the deck plate which, in turn, are shop welded to the two main girders. A 12" fascia channel is shop welded to the outside ends of the floor beams. Web stiffeners are welded on both sides of the girder webs and knee braces are provided between the girder and floor beams at alternate floor beams. Approximately 2-3/4 miles of shop welding was required for the fabrication of this structure.

The shop fabricated structure, which is made from A-441 steel was fabricated in nine 12' wide by 68', 56' and 68' long sections. The largest of the prefabricated sections weighs approximately 22 tons. The girders, floor beams and stiffeners were spliced in the field by high strength bolts and the deck plate was spliced by automatic submerged arc welding. Approximately 3,400 bolts and 460 feet of field welding were required for the splices.

Two 10" high concrete brush curbs, secured by 3/4" round by 7" long steel studs welded to the deck plate, support a three tube galvanized steel railing.

The wearing course consists of a 5/8" thick epoxy mortar placed on the sandblasted steel deck. In order to gain as much information as possible on surfacing steel plate decks, two types of epoxy mortar were specified to be used. Guard-Kote 250 was placed on one-half of the deck and Epon 815 Epoxy was placed on the other half. Filler material for the epoxy consisted of dry 2NS sand.

The substructure consists of concrete curtain wall abutments supported by 12" cast-in-place concrete piles and a welded-steel-box-column type pier supported by concrete spread footings. The main girders of the bridge rest on elastomeric bearing pads at the pier and self lubricating bronze bearings at the abutments. Expansion was provided at the abutments by means of preformed neoprene joint seals.



ELEVATION

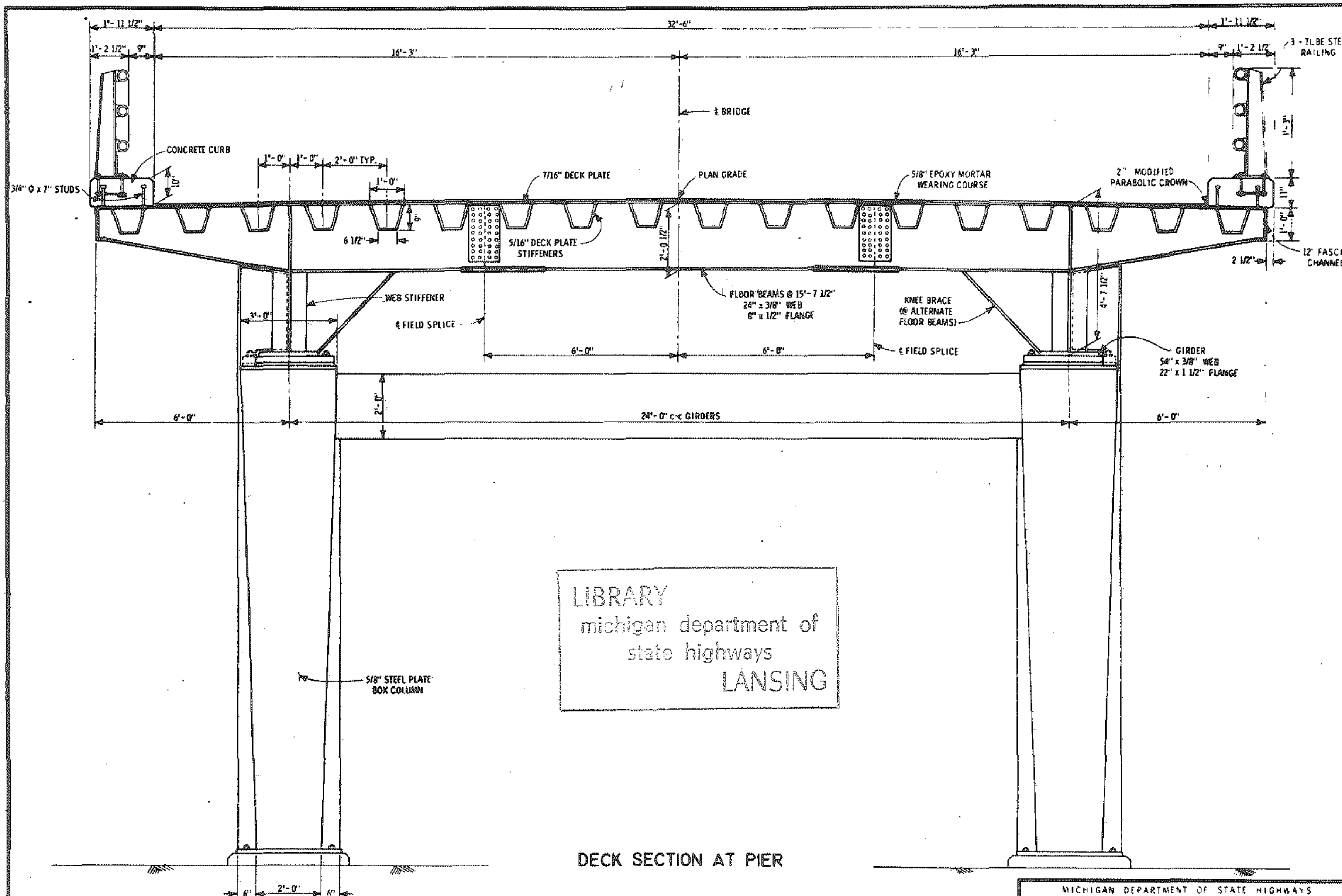
MICHIGAN DEPARTMENT OF STATE HIGHWAYS

CONTINUOUS ORTHOTROPIC STEEL PLATE DECK BRIDGE

CRIETZ ROAD OVER I-496

3 MILES WEST OF LANSING

S05 OF 23081A



DECK SECTION AT PIER

MICHIGAN DEPARTMENT OF STATE HIGHWAYS
CONTINUOUS ORTHOTROPIC
STEEL PLATE DECK BRIDGE
CRIETZ ROAD OVER I-496
3 MILES WEST OF LANSING

S05 OF 23081A

FIELD CONSTRUCTION TIMETABLE

July 9, 1968 - Started excavation for pier subfootings. Encountered soft clay which had to be undercut and backfilled with Class III Sand Gravel Material.

July 10, 1968 - Poured concrete for pier subfootings. Bored holes for piles at Abutment B.

July 11, 1968 - Poured concrete for pier subfootings. Drove piles at Abutment B.

July 12, 1968 - Poured concrete for pier pedestals. Drove Abutment B wingwall piles. Cut Abutment B piles off to correct elevation and filled them with concrete.

July 15, 1968 - Poured concrete for Abutment B subfooting. Bored pile holes and drove piles at Abutment A. Cut Abutment A piles off to correct elevation and filled them with concrete.

July 16, 1968 - Poured concrete for Abutment A subfooting. Started forming Abutment B curtainwall.

July 17, 1968 - Poured concrete for Abutment B curtainwall.

July 18, 1968 - Started forming Abutment A curtainwall.

July 19, 1968 - Finished forming Abutment A curtainwall. Started forming Abutment B return wingwalls.

July 22, 1968 - Poured concrete for Abutment A curtainwall. Finished forming Abutment B return wingwalls.

July 23, 1968 - Poured concrete for Abutment B return wingwalls.

July 29, 1968 - Stripped forms from Abutment B.

July 30, 1968 - Poured concrete for Abutment A return wingwalls.

Aug. 5, 1968 - Stripped forms from Abutment A.

Oct. 4, 1968 - Bennett Industries in Peotone, Illinois started shop fabrication of the deck sections.

Dec. 24, 1968 - Shop fabrication of the deck units and steel pier columns completed.

Dec. 26, 1968 - Loaded all units on railroad cars at Bennett Industries and shipped to Lansing, Michigan.

Jan. 14, 1969 - Erected the steel pier and the west fascia center pier unit 1.

Jan. 15, 1969 - Erected the east fascia center pier unit 2 and the middle center pier unit 3.

Jan. 16, 1969 - Erected the southeast fascia unit 8 at Abutment A.

Jan. 17, 1969 - Erected the southeast fascia unit 7 and the middle center unit 9 at Abutment A.

Jan. 20, 1969 - Erected the northeast fascia unit 5, the northwest fascia unit 4, and the middle center unit 6, at Abutment B.

May 15, 1969 - Grouted slope paving in front of Abutment B.

May 16, 1969 - Grouted slope paving in front of Abutment A.

Jun. 16, 1969 - Poured concrete for backwall and top lift of return wingwalls for Abutment B.

Jun. 20, 1969 - Poured concrete for backwall and top lift of return wingwalls for Abutment A.

Jun. 24, 1969 - Poured concrete curb along west side of bridge.

Jun. 26, 1969 - Began field welding of deck plate seams.

July 1, 1969 - Poured concrete curb along the east side of the bridge.

July 30, 1969 - Finished field welding of the deck plate seams. Began placing the curb railing.

Aug. 8, 1969 - Finished placing the curb railing.

Aug. 11, 1969 - Tightened all rib splice bolts.

Sept. 29, 1969 - Keith Wolf began applying the epoxy wearing surface to the steel deck.

Nov. 7, 1969 - Keith Wolf finished applying the epoxy wearing surface to the steel deck.

Nov. 25, 1969 - Bridge was opened to traffic.

FABRICATION OF STEEL FOR ORTHOTROPIC BRIDGE

A pre-fabrication meeting was held at the fabricator's plant to clarify issues and assure that uniform fabrication procedures would prevail. The meeting included representatives from Design, Construction - including shop inspectors and field supervisors - and fabricating supervisors.

The fabricator developed several unique innovations to perform this work. He devised an "A" frame for pressing the ribs to the deck plate at which time they were tacked in place. During the final welding phase, the fabricator produced a machine that provided a flux backing, this assuring 100% penetration for all the continuous rib to deck plate welds.

There were problems of distortion in at least two of the nine sections which required the consultation of a flame heating expert to provide the necessary corrections. The deck plate of one section was grossly laminated which did not become apparent until after all welding was completed. This was due to the fact that all sections were fabricated upside down, and only upon turning the entire section over, was the defect discovered. The lamination was probed with ultrasound to determine its extent and then removed. A 14 foot length of full width deck plate was removed and replaced. The amount of torch cutting necessary for removal was quite extensive and tedious. Due to the amount of heating and rewelding, a certain amount of distortion is noticeable in the finished structure in the field.

The fabricator used smaller bolts during assembly than those used in the field, thus causing field problems in alignment which necessitated field reaming in awkward positions.

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS
CONSTRUCTION DIVISION

SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176
(Rev. 3/69)

Report No. 3

Fabricator's Contract No. 7824

Date 7-12-68

Shop Inspection of Structural Steel

At Bennett Industries, Peotone, Ill.

Project No. S05 23081 002 I 496-7(21)138

Status of fabrication for period ending 7-12-68

Date approved drawings received	4-28-68
Estimated tonnage	344,600#
Material received from mill	100 %
Material laid out	--- %
Material fitted and welded	--- %
No. of radiographs required	100 %
No. of radiographs completed and accepted	---
Material fabricated, not shipped	--- %
Material shipped	--- %

Remarks: No Progress.

Plant Mgr., Mr. Ed Bosak has received no word from the mill as to what they anticipate to do with the rib's.

Inspector Samuel C. Lee

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Testing and Research Division
District Construction Engineer
Project Engineer
Construction File

STATE OF MICHIGAN DEPARTMENT OF STATE HIGHWAYS CONSTRUCTION DIVISION	SHOP INSPECTION OF STRUCTURAL STEEL	Form 1176 (Rev. 3/69)

Report No. 9

Fabricator's Contract No. 7824

Date 10-5-68

Shop Inspection of Structural Steel

At Bennett Industries, Peotone, Ill.

Project No. S05 23081 003 I 496-7(21)138

Status of fabrication for period ending 10-4-68

Date approved drawings received	4-8-68
Estimated tonnage	126
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	50 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	50 %
Material shipped	0 %

Remarks: Center section SPan A-3 of this project now in process of preassembly - rib to deck plate. Will witness runoff tab procedures Oct. 5, 1968 for weld operation.

No other fabrication in process as of this date.

Inspector Andrew Jones

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SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176
(Rev. 3/69)

Report No. 2 (10)

Fabricator's Contract No. 7824

Date 10-11-68

Shop Inspection of Structural Steel

At Bennett Industries, Peotone, Ill.

Project No. S05-23081A002 I-496(21)138

Status of fabrication for period ending 10-11-68

Date approved drawings received	4-8-68
Estimated tonnage	178
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	50 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	50 %
Material shipped	0 %

Remarks: Center sections A-3, B03, C-3, for this project now complete. Thru assembly of rib to deck plate.
Sec. A-3 in also being fitted with floor beams. All work being performed in accordance to A.W.S. & M.D.S.H. spec.

Inspector Andrew Jones

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DEPARTMENT OF STATE HIGHWAYS
OFFICE OF CONSTRUCTION

SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. 3 (11)

Fabricator's Contract No. 7824

Date 10-19-68

Shop Inspection of Structural Steel

At Bennett Industries- Peotone, Ill.

Project No. S05-23081A ~~00~~ I 496(21) 138

Status of fabrication for period ending Oct. 19, 1968

Date approved drawings received	4-8-68
Estimated tonnage	172
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	50 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	50 %
Material shipped	0 %

Remarks: Sections A-5, C-5 assembled with the following exception. These assemblies carry a girder in their centers. These girders have been tacked in, and the floor beams mounted on the left and right end to allow continous welding of girder to deckplates as per discussed with Charlie Ellis 10-11-68.

Sections B-4, B-5 layed out for girder to deckplate. All 9 sections are complete through tacking of rib to deckplates. All samples of microetchings accepted to date with penetration of 90 to 100%. All work performed in accordance to A.W.S. and M.D.S.H. specs.

Inspector Andrew T. Jones

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SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. 4 (12)

Fabricator's Contract No. 7924

Date October. 26,1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone,Illinois

Project No. S05-23081 I 496(21)138 ~~(80?)~~

Status of fabrication for period ending October,26,1968

Date approved drawings received	4/8/68	
Estimated tonnage	178	
Material received from mill	100	%
Material laid out	100	%
Material fitted and welded	90	%
No. of radiographs required	0	
No. of radiographs completed and accepted	0	
Material fabricated, not shipped	90	%
Material shipped	0	%

Remarks: Items A-3,A-2,and B-3 which are the center sections of this project are now complete thru weld fabrication.

Items A-4,A-5 and D-5 are now complete thru welding of the girder to deckplate assembly. All work performed in accordance to A.W.S.and M.D.S.H. spec.

Inspector Andrew [signature]

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SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. 5 (13)

Fabricator's Contract No. 7824

Date November 3, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Ill.

Project No. S 0 5-23081 A 002 I 496 (21) 138

Status of fabrication for period ending ~~November 3, 1968~~

Date approved drawings received	4-8-68	
Estimated tonnage	126	
Material received from mill	100	%
Material laid out	100	%
Material fitted and welded	80	%
No. of radiographs required	0	
No. of radiographs completed and accepted	0	
Material fabricated, not shipped	80	%
Material shipped	0	%

Remarks: Sections A-5 and B-4 are now as of this date in hold. Reasons for this are as follows: Section A-5 has a defect in the deck plate. There is a lamination rolled in from the mill that was not detected until said item was 80% thru weld fabrication. The fabricator contacted the mill and the mill sent their representative out to look at the defective piece. Note pictures of defect.(Mr. John Janis, Metallurgist)

Section B-4 was fabricated with a twist effect in the deck plate lengthwise, causing a rib to raise and making a proper fit impossible. Mr. J.R. Stitt was called in by the fabricator to correct said condition. Mr. Stitts arrived Saturday 10am to

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correct said defect in this item. Note pictures of defect.

Items B-5, B-3 and D-5 are now being painted inside of ribs. No paint to be applied to outside until Mag. Par. Test are made. The method of applying paint proves to be very satisfactory. Sample ran first, and the machine applys a coating a appr. 1.8 to 2 ml. wet paint evenly.

All work and repairs in accordance to A.W.S. and M.D.S.H. specs.

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS
OFFICE OF CONSTRUCTION

SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176
Mich. Dept. of State Hwys.

NOV 12 1968

Report No. 6 (14) Office of Construction

Fabricator's Contract No. 7824

Date Nov.9,1968

Shop Inspection of Structural Steel

At Bennett Industries ,Peotone,Illinois

Project No. S05-23081A 002 I 496(21)138

Status of fabrication for period ending Nov.9,1968

Date approved drawings received	4/8/68
Estimated tonnage	126
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	100 %
Material shipped	0 %

Remarks: Mr. J.R.Stitt of the Mahon Co.Arrived Sat.5,Nov. 10 am.Left on Sun.8pm Nov.6. Mr. Stitt was consulted on a procedure to correct the twist in section C-4 of this project. Defect corrected to within 1/8 of an inch.

As of this date there are 6 sections fitted for drill splicing. Mr. G.Hill and Mr. R. Clensy arrived in the plant on Wed. Nov.6. concerning the defect in section of deckplate in previous report. The correction of said deckplate will start on Monday. Nov. 11. All work to date in accordance to A.W.S. and the M.D.S.H. spec.

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OFFICE OF CONSTRUCTION

SHOP INSPECTION OF STRUCTURAL STEEL

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Report No. 16

Fabricator's Contract No. 7824

Date Nov. 23, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Illinois

Project No. S05-23081 A 002 I 496-7(21)138

Status of fabrication for period ending Nov. 23, 1968

Date approved drawings received	4/8/68
Estimated tonnage	178
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	100 %
Material shipped	0 %

Remarks: Sec. D-5 of this project repaired as of this date up to the point of deck plate to rib welding. Trimming of excess material to be done on mon. 11/25/68 Transverse splice X-rays made and accepted. The other sec. of this project now in process of drilling for splices. All work and repairs done in accordance to A.W.S. and M.D.S.H. spec.

Inspector Archie D. Jones

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SHOP INSPECTION OF STRUCTURAL STEEL

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Report No. #6 (17)

Fabricator's Contract No. 7824

Date November 27, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Illinois

Project No. S 0 5-23081 A 002 I 496-7(21)138

Status of fabrication for period ending November 27, 1968

Date approved drawings received	7-8-68
Estimated tonnage	126
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	100 %
Material shipped	0 %

Remarks: Repairs are complete as of this date on sec. D-5. There were some heats applied without authorization to the rib sec. on the repaired end. Cons. office notified of this infraction of spec. The splices are now drilled in sec. C-5, B-4 and A-5. All other work up to this point in accordance to authorization or A.W.S. and M.D.S.H. spec.

Inspector [illegible signature]

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Mich. Dept. of State Hwys.
DEC 11 1968
Office of Construction

Report No. 10 (18)

Fabricator's Contract No. 7824

Date December 7, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Illinois

Project No. S05-23081 A 002 I 496-7(21)138

Status of fabrication for period ending

Date approved drawings received	
Estimated tonnage	
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	100 %
Material shipped	0 %

Remarks: This project is complete thr the splicing operation. Sections D5 and B3 have been mag. par. inspected. There were no defects found. Sec B3 also painted on December 6. All work on this project in accordence to A.W.S. and M.D.S.H. spec.

Inspector Andrew T. Jones

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SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. 11 (19)

Fabricator's Contract No. 7824

Date December 14, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Ill.

Project No. S 0 5-23081 A 002 I 496-7(21)138

Status of fabrication for period ending December 14, 1968

Date approved drawings received	7-17-68
Estimated tonnage	197
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required Transverse splice for repair	3
No. of radiographs completed and accepted	3
Material fabricated, not shipped	100 %
Material shipped	12 %

Remarks: This project is complete thru fabrication and all other work except for minor repairs to sections B-5, B-3, D-5, sections C-5, A-3, A-5, B-4, A-2 are finished and painted. Section B-5 loaded for shippment to site 12-14-68. Work and repairs made in accordance to A.W.S. and M.D.S.H. or by authorization.

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OFFICE OF CONSTRUCTION

SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. 20 20

Fabricator's Contract No. 7824

Date December 21,1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Ill.

Project No. SO 5-23081-A 002 I 496-7 (21) 138

Status of fabrication for period ending December 21, 1968

Date approved drawings received	4-8-68	
Estimated tonnage	197	
Material received from mill	100	%
Material laid out	100	%
Material fitted and welded	100	%
No. of radiographs required	0	
No. of radiographs completed and accepted	0	
Material fabricated, not shipped	100	%
Material shipped	0	%

Remarks: Five sections of this project are loaded at present awaiting shipment to site. Mag. par. is being performed in accordance to special notations in proposal. Work proceeding in accordance to A.W.S. & M.D.S.H. specs.

Inspector Andrew T. Jones

dl

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SHOP INSPECTION OF STRUCTURAL STEEL

Form 1176

Report No. ~~#21~~ Final

Fabricator's Contract No. 7824

Date December 28, 1968

Shop Inspection of Structural Steel

At Bennett Industries Peotone, Ill.

Project No. S 05-23081 A 002 I 496-7(21) 138

Status of fabrication for period ending December 28, 1968

Date approved drawings received	4-8-68
Estimated tonnage	197
Material received from mill	100 %
Material laid out	100 %
Material fitted and welded	100 %
No. of radiographs required	0
No. of radiographs completed and accepted	0
Material fabricated, not shipped	0 %
Material shipped	100 %

Remarks: This project is complete, loaded on 12-24-68 en route to site on 12-26-68. All work performed in accordance to A.W.S. and M.D.S.H.

Inspector [signature]

dl

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Contractor
Bureau of Public Roads
Office of Testing and Research
District Construction Engineer
Project Engineer
Construction File

Form 1000

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS
Testing and Research Division
Research Laboratory Section
735 East Saginaw Street
Lansing, Michigan

REPORT OF TEST

Project: S05 of 23081 A (Crietz Rd over I 496 3 Miles West of Lansing)
Laboratory No.: 69 E-109 to 120
Date: November 6, 1969

Report on sample of: EPOXY MORTAR CYLINDERS (3" Diam. x 6")
Date sampled: As Stated Below
Date received:
Source of material: C. L. Wolff Co. (Midwest Bridge Co)
Sampled from: Epoxy Mortar Mixes
Quantity represented: Not Stated
Submitted by: Sampled by Research Laboratory Personnel
Intended use: Orthotropic Bridge Deck Surfacing
Specification: MDSH Supp. (5-29-67)

TEST RESULTS

Laboratory Number	Cylinder Number	Date Cast	Test Date	Trapped Air* (percent)	Max. Unit Compressive Stress (psi)
	Guard Kote 250 Epoxy Mortar (14 Gal. Binder to 660 lb 2NS Sand) Part A, Lot No. 5LHJ752 Part B, Lot No. 7KHJ676				
69 E-109	1	10-1-69	10-8-69	25	1400
110	2	Southwest		22	1270
111	3	Panel		28	1420
112	4	10-3-69	10-10-69	24	1180
113	5	South-center		22	1200
114	6	panel		22	1140
	Gen Epoxy-E 15 & Versamid 140 (14 Gal. Binder to 660 lb 2NS Sand) Part A (E15, Lot No. 7KHJ205 Part B (V140), Lot No. 8FF537				
69 E-115	7	10-7-69	10-14-69	20	4920
116	8	Northwest		26	5100
117	9	Panel		29	4840
118	10	10-9-69	10-16-69	29	6190
119	11	Northeast		24	5800
120	12	Panel		33	5890

*Linear Traverse Method on Top 1/2" ASTM C457

REMARKS: Tested for Information.

cc: ATL
File
F. D. Rieger
J. E. Risch
R. C. Martin
J. J. Michels (2)

Signed ______
Engineer of Testing and Research

REPORT ON

CRIETZ ROAD BRIDGE ERECTION
SO5 OF 23081

Submitted to

Mr. John Risch
Chairman, Joint Committee

by

Gary W. Grimes
Assistant Project Engineer
and Senior Bridge Inspector

January 7, 1970

CRIETZ ROAD BRIDGE ERECTION

CONSTRUCTION REPORT
(S05 OF 23081)

Introduction

The experimental orthotropic bridge carrying Crietz Road traffic over I-496 in Eaton County, was erected in 1969. This report concerns the problems and difficulties noted by construction crews, which may be correctable at earlier stages of the project.

Procedure

The data for this report was taken from the diary, daily reports, and memory of the Senior Bridge Inspector.

Discussion

The staking and unclassified earth excavating were similar to the ordinary structure, and offered no problems peculiar to this bridge.

The bearing surfaces of pier 1 pedestals were to be finished to exact elevation, to be in accordance with the plans.

The standard specifications for roads and bridges already state that areas under bearing plates will be finished to 1/16 inch of plan grade (5.01.11d). The extra floating of the concrete required to obtain a more accurate grade than this might result in excess surface mortar. Extra hand floating was done on these surfaces, and there did appear to be some excess mortar. It is felt that after the extra work, the elevation was still not "exact."

The pier consists of two separate units. Each unit has a 3 ft. x 3 ft. 9 in. pedestal on a 12 ft. x 12 ft. footing. Each pedestal has four pairs of anchor bolts. These bolts are 3/16' less diameter than the holes in the pier column base plate. It is very difficult to arrange eight anchor bolts to a combined accuracy of this tolerance, and for them all to be perfectly plumb. The bolts on the pier were measured to 1/16 inch accuracy at the bearing surface, but it is unlikely that they were exactly vertical. Some small adjustment was made by bending the bolts at the bearing surface, but before the pier column could be started over the anchor bolts, the holes in the base plate had to be enlarged.

Construction of the abutments was similar to other structures on this project, and offered no special problems.

The superstructure was prefabricated, and arrived in nine units. Three units were attached over each of the two girders, and the three middle units were to be suspended between the fascia units by floor beam splices. False-work piers supported the splice ends of the fascia units until they were spliced together. This falsework was not set to an accurate grade, and the bridge units were accordingly placed to inaccurate line and grade. It was the feeling of the contractor that proper positioning would automatically occur as all the field splices were made. This did not happen.

A minor erection problem was the lack of enough clearance between the top of the column anchor bolts, and the bottom of the center knee brace. The nut could not be started on the bolt until the deck unit was slightly raised.

Notches were cut into the side of the interior splice plates of the longitudinal ribs. This was necessary to allow the deck splice backup bar to pass through the rib. The cause of this problem appeared to be incorrect location of the bolt holes through the splice plates.

Another problem related to the backup bar, was the trapping of moisture and dirt between the top of the bar and the bottom of the deck plate. The bar was welded to one side of the deck splice during fabrication. Moisture, rust, and dirt became trapped here. The runoff of every rain between initial field erection in January, and deck splice welding in

August added moisture and dirt. During field welding, the moisture and impurities were trapped by the shop weld. They escaped through the field weld, causing porosity.

Root opening between the deck units varied from zero to 11/16 inches. Some of the variation was in irregular edges of the deck plates. However, it is suggested that part of the cause is the lack of proper alignment before the field splices were begun.

A cutting torch and a power saw were used to widen the narrow openings. The wide roots required additional passes with the welder.

The epoxy mortar wearing surface was the least troublesome operation. The greatest difficulty was the temperature limitation.

A large overrun of material was expected, and experienced, due to the vertical irregularity of the deck plate surface. In order to approach the 5/8 inch minimum thickness, the average depth was held at more than 3/4 inch. This extra thickness was unavoidable, and resulted in a better surface.

The proposed priming of the deck steel with epoxy resin at the rate of 150 sft. per gallon, proved to be impossible. At this rate, excess resin flowed under the forms. Also, the mortar was transported from the mixer to the screed, then dumped on the prime in a pile. The screed was charged by scooping the mortar from the pile and spreading it with a shovel. The action of the shovel scooping through the prime into the pile effectively mixed the excess prime into the mortar. This resulted in a resin rich mix, indicated by mortar flowing under the forms, and by resin bubbles appearing on the epoxy surface. The prime rate was reduced to about 300 sft. per gallon, to make the final product acceptable.

Recommendations

It is recommended that:

1. The noting of an elevation as "exact" be replaced by noting a tolerance.
2. The number of anchor bolts be limited to one in each corner, or the size of the bolt holes in the base plate be increased.

3. The plans or proposal state that each unit be brought to proper line and grade before the adjoining unit is erected.
4. Splice plates be drilled while in their exact final location. It is thought that the interior splice plates for the longitudinal ribs were drilled while in the outside position.
5. Back up bars be left off the longitudinal deck joints until just before they are to be field welded. If the transverse back up bars did not go through ribs, it would also be advantageous to weld them in the field.
6. The pay units for Epoxy Mortar Wearing Surface be in volume instead of area. In this project, the overrun of epoxy mortar exceeded 20 percent by volume. The pay units remained constant. It is the feeling of the inspection personnel that the contractor could be more adequately compensated for his work and materials if the pay item had been volume units.
7. A substantial portion of the erection problems in regard to alignment, were caused by the short spans of the structure. It is felt that any future use of orthotropic spans should be confined to lengths in excess of 200' in individual spans. The reason for this is that every section on the Crietz Road structure was founded upon a portion of the substructure. There was no possibility in the erection of these deck units to adjust alignment within the span as there would be if there were three or four units being assembled in the same span.

EPOXY MORTAR SURFACING OF THE STEEL PLATE DECK

The 7/16 inch steel plate deck was surfaced with two experimental types of epoxy mortar mixtures after the nine deck sections were field welded and the concrete curb had been poured. The two types of epoxy resin systems were selected as a result of successful laboratory tests performed at Battelle Memorial Institute in Columbus, Ohio. The selection was made at a meeting November 17, 1966, between F. F. Fondriest of Battelle and members of the Design and Testing and Research Divisions. The two types of epoxy binders selected were: (1) Guardkote 250, a lower strength and flexible oil modified epoxy, made by Shell Oil of St. Louis, Missouri and (2) a combination of E15 resin and Versamid 140 polyamide curing agent made by General Mills of Kankakee, Illinois. The latter epoxy system is a higher strength, moderately flexible, and slower curing binder. Both of the epoxy binder's two components were mixed at a 1:1 ratio by volume.

The fine aggregate, mixture design, and application procedure for both epoxy systems are described in a supplemental specification dated 5-29-67 which was included in the contract proposal. The surfacing operation was done by C. L. Wolff and Sons, Inc., of Benton Harbor, Michigan. Batching and mixing of the epoxy mortar was done at the north approach area and transported onto the deck in a small, powered, scoot-crete buggy. Most of the batches were of about 7.5 cubic feet in size and consisted of 660 pounds of bone dry graded quartz sand and 13 to 14 gallons of epoxy binder. The mixes were found to contain about 20 percent air. The dry quartz sand was obtained in 100 pound bags from Sewanee, Tennessee, and was batched at a ratio of four bags of a No. 6 (coarse) to two bags of No. 20 (fine). About 60 pounds of banding sand was added to each batch to increase the fine fractions. The final 660 pound aggregate blend had the following gradation:

Standard Sieve Size	Cummulative % Passing
No. 4	100
No. 8	99
No. 16	66
No. 30	42
No. 50	21
No. 100	3

The sand blend was slightly finer than the specified 2NS grading in the coarse sizes but this actually proved beneficial in the screeding properties at the 5/8 inch minimum thickness used. The use of the dry bagged aggregate blend was approved at a pre-construction meeting held September 9, 1969, with the contractor and members of Design, Construction and T&R Divisions.

The two epoxy binders, as mentioned earlier, were chosen for their strength and flexibility properties. The Guardkote 250, however, is much more flexible and has lower bond and tensile strengths than the E15-V140 combination. The acceptance tests performed in the Research Laboratory on the cured binders with no aggregate were as follows:

	Guardkote 250	E15-V140
Tensile Strength, psi	950	5640
Elongation, percent	41.7	14.8
Shear Bond to Steel, psi	765	4130
Water Absorption, 24 hr, %	0.35	0.45

The epoxy mortar surfacing was applied in six pours, each done on separate days. Guardkote 250 was used on the south half and E15-V140 was used on the north half, each 94.5 feet long. A 12 foot wide area was outlined by two steel screed rails in the southeast quadrant of the deck leaving about a six inch gap at the east curb. The area was thoroughly sandblasted in the AM and primed and surfaced in the afternoon of September 30, 1969. A similar 12 foot wide panel was done in the southwest quadrant on October 1 and the center 7-1/2 feet was surfaced on October 3. The 13 to 14 gallons of epoxy binder's two components were measured by volume and premixed about three minutes. The mixed binder was then blended about three minutes more with the 660 pounds of dry quartz sand in an 11 cubic foot Essex mortar mixer. The mortar mixture was then placed immediately ahead of a Maginniss vibrating screed pulled forward by a self propelled concrete saw. About 6 to 8 feet of the sandblasted steel plate was kept primed with premixed epoxy binder using a paint roller. The screed was moved at about one foot per minute, taking about 90 minutes to do the 94-1/2 foot span. A light application of dry sand was applied to the finished mortar as a top dressing to assure good skid resistance.

Three similar panels in the north half using an E15 - Versamid 140 mortar were done on October 7, 9, and 30 for the NW, NE, and middle pours, respectively. The concrete curbs were sandblasted and coated with the respective clear epoxy binder and a 3/4 inch epoxy mortar fillet was placed at the base of the curb line after the surfacing had been finished.

Uniformity of epoxy mortar surfacing thickness was checked on November 25, 1969, the day the structure was opened to traffic. A model E4 Swiss Pachometer was used, with modifications, to measure the surfacing thickness at about 4 foot intervals longitudinally and on 2 foot centers transversely. No measurements were made over transverse beams or the two main plate girders. A total of 312 measurements were made in each half of the deck with only 15 readings in the north half and 18 readings in the south half indicating thicknesses less than the specified 5/8 inch minimum. Six readings in the south half were slightly under 9/16 inches and all readings in the north half were equal to or greater than 9/16 inch. The average thickness of surfacing was found to be about 3/4 inch for both halves. A maximum of 1-3/8 inch thickness was found at one point over a longitudinal weld.

Initial 40 mph skid tests were run on December 2, 1969, using the Research Laboratory skidometer. An average wet coefficient of friction of 0.72 was obtained on the north half and 0.67 on the south half. Initial Rapid Travel Profilometer runs were made on December 5 in all four wheel track areas and including 200 feet of approach pavement at each end. Subsequent skid and profilometer tets will be run and an experimental load-strain analysis will be conducted in the spring of 1970. These various measurements along with condition surveys will be included in subsequent reports.

PROBLEMS TO BE CORRECTED ON FUTURE ORTHOTROPIC JOBS

COMMENTARY

The experience gained in the design, fabrication, construction, and maintenance of Michigan's first orthotropic bridge will pave the way for more economical orthotropic bridges in the future.

The hand solution of design calculations for any orthotropic design are long and tedious. Our design engineers have set up a computer design program to supplement these hand calculations for the closed rib type orthotropic deck. Future designs of this nature can now be done in a relatively short period.

A few of the more important fabrication and construction problems encountered with the Crietz Road structure are described in the following pages.

1. ROLLED RIB SHAPES:

The trapezoidal shaped rib sections for the Crietz Road structure were formed by passing universal mill plates through special rolling mills at the Bethlehem Steel Corporation.

However, the uneven edges and sometimes considerable amount of sweep present along the edges of universal mill plates, caused a substantial amount of variation in the rib dimensions after passing through the rolling mills.

To prevent this problem in the future, Bennett Industries suggested that we allow the fabricator to pre-cut the rib plates to exact size using a "plasma arc cutting process" prior to the trapezoidal shape rolling procedure.

In this process a multi-torch burning machine would automatically flame-cut several rib plates to precise width in one operation. The plates would then be cold-rolled to form the trapezoidal shaped "trough" sections of the ribs.

In addition, the cutting arc of this process cuts through the plate at a 7° to 10° arc, thus leaving the edges of the plate beveled by this same amount, in contrast to the 90° cut edge of our standard cutting torches. The beveled edge left by the plasma arc process would guarantee a 100% weld penetration when the rib sections are welded to the bottom of the deck plate.

The length of rib plates prior to the rolling process should be increased a few feet at each end to allow for easier handling and also a more accurately rolled section at the design cutoff point.

2. GIRDER KNEE BRACES:

Extra care had to be used by the fabricator in handling the extremely flexible superstructure units where permanent deformations of the thin sections coule easily occur.

The fabricator felt that box-type girders would have given him much more rigidity while fabricating the girder units of this structure; especially, the fascia portion of the deck extending beyond the main girders under the sidewalks. Box-type girders would also facilitate more automatic welding.

GIRDER KNEE BRACES: (Cont.)

On other structures with longer spans and possibly wider roadway widths, the design and economy might justify using box-type girders.

However, on future orthotropic structures using girders similar to the Crietz Road project, we might add extra knee braces to the exterior part of the girder web in addition to those detailed on the interior. This would greatly stiffen the fascia cantilevered portion of the deck and also help reduce distortion problems caused by the welding.

3. TRANSVERSE JOINT SPLICING:

It was impossible for the fabricator to hold a true transverse crown in the deck plate at the end of the deck units. Distortions due to welding the deck plate to the ribs cause the deck plate to shrink and form short chords across the tops of the ribs. This rippling effect in the deck plate is quite a problem when trying to fit the units together in the field prior to splicing.'

On future jobs, Bennett Industries would favor welding transverse channels or plates across the end of the ribs with the top of these transverse plates cut to fit the crown of the roadway and welded to the bottom of the deck plate. Holes could be drilled through these plates between the ribs and short bolts used for splicing the deck units together.

In addition to holding a true crown of the deck plate, fitting and splicing the deck units together in the field would be much easier and faster.

4. HAND CUTTING FLOOR BEAM WEBS:

Future design might dictate a wider cut in the floor beam webs where the ribs pas s through, this providing more allowance for small variations in rolled rib dimensions.

Automatic machine cutting would then replace a substantial amount of hand cutting, and the bottom of the rib would be welded to the floor beam web, leaving the sides of the rib weld free.

This process would greatly reduce future fabrication costs.

5. <u>DECK PLATE BACK UP BARS</u>:

The fabricator felt it would be easier and less costly to drill and tap the back-up bars to the deck plate after the deck units have been completed and shop fitted.

After field erection of the deck units, the back-up bars could be brought to their proper position and bolted in place.

This procedure would facilitate fitting and handling costs on future jobs.

6. <u>FLOOR BEAM COPE SIZE</u>:

Our plans called for a 1 inch radial cope in the top of the floor beam web where it met the top of the rib and the deck plate.

This cope allowed the three welds to meet properly and it also provided stress relief in the plates at this junction.

During fabrication, however, it was found necessary to increase the cope depth to at least 1-1/4 inches at the point where the top of the end floor beam web of each deck unit met the top of the main girder web. This deeper cope allowed a proper hand weld where the automatic welding rig could not pass through the floor beam web when making the longitudinal weld along the main girder.

7. <u>BEARING STIFFENER WELDING AT ABUTMENTS</u>:

It was very difficult for the fabricator to weld pairs of wide bearing stiffeners at the abutments. Clearances were too small; especially where the knee braces were joined to the main girder.

We allowed the fabricator to bevel the outside edge of the bearing stiffener next to the girder web and then provide a full 3/8 inch weld (same as girder web thickness) along the outside edge of the bearing stiffeners. The 3/8 inch weld was still adequate to transmit the shear load of the girder web to the bottom of the bearing stiffeners as design required.

8. <u>ANCHOR BOLT HOLES</u>:

It was difficult to place the steel pier columns over the anchor bolts projecting from the concrete pier pedestal.

The plans called for holes 3/16" larger in diameter than the anchor bolts. However, there were eight anchor bolts for each column and the base plate was1-1/4" thick. Even the slightest variation in anchor bolt horizontal placement could cause difficulty when placing the steel columns on the concrete pedestals.

A cutting torch was used in the field to cut the holes larger where necessary to allow proper fitting of the pier base plates over the anchor bolts.

In the future we should increase the diameter of the anchor bolt holes in the pier column base plates and also the size of the main girder sole plate slots at the abutments and top of pier columns. Consideration has also been given to casting a base plate in the top of the pier pedestal.

This will not only facilitate easier placement of the pier columns but also better alignment of the superstructure joints prior to completing the deck splices.

The size of the main girder sole plate slots will also be increased at the abutments as well as the pier to allow better alignment of the superstructure joints prior to completing the splices.

9. <u>ANCHOR BOLT CLEARANCE</u>:

The contractor had difficulty placing the nuts on the top of the pier anchor bolts because the haunched beams came down too low to permit placing the nuts directly on the bolts after the girder was set.

To solve this problem the men raised the units a little with the cranes, then slid the nuts into position, and finally lowered the units a little at a time as the nuts were turned down upon the projecting bolts.

On future jobs, wider bearing and masonry plates should be provided with corresponding wider anchor bolt spacing; thus insuring the proper clearance for placing and tightening the nuts upon the anchor bolts.

10. PLACING THE PIER STRUT:

The strut was detailed to fit between clip angles that were shop welded to the steel columns. Later, after correctly placing the strut, the plans called for field welding the strut to the angles.

However, extensive pounding was necessary to drive the strut down between the clip angles that were welded to the pier columns. Evidently, the steel fabricator failed to allow the proper clearance necessary between the strut and the clip angles for field erection or there remained enough twist in the pier columns caused by the anchor bolt problem to prevent the close fit planned by the fabricator.

11. GIRDER WEB STIFFENING DIAPHRAGMS:

Near the main girders, the local stress conditions in the deck plate are complicated by the fact that the girder web acts as a rigid support for the deck plate, while the adjoining ribs act as elastic supports because of their flexibility under wheel loads.

To help prevent these excessive local deck plate stresses over the main girder webs, small stiffening plate diaphrams were welded between the girder web and the first rib at the third-points of the rib spans to provide a more gradual transition between the "hard" girder web and the "soft" ribs.

On future jobs of this nature, it would be more economical and better looking to detail these small stiffening plate diaphrams as being an integral part of the main girder web stiffeners which are placed near the third-points of the rib spans.

12. RIB SEALER PLATES:

The foreman in charge of steel erection suggested that in the future, the sealer plates located inside the ribs be placed farther back from the handhole.

This would allow the inside rib splice plates to be brought up through the handhole after the deck units have been correctly positioned for splicing.

13. BACK UP BAR NOTCHES:

Another item that should be considered on future orthotropic jobs is beveling the cut-out portion of the rib walls directly beneath the deck plate for receiving the transverse joint back-up bars.

Our present structure had level cut-out notches, whereas if these receiving slots were beveled, the deck units would slip into position much more easily during erection.

14. WATER IN THE RIB SECTIONS:

On August 11, 1969, our field engineer noticed water running down through the expansion joint and over the abutment face at the north end of the bridge. After further examination it was determined that water had managed to get into the rib sections and was leaking out of a thin open seam left between the deck plate and the end plate welded across the ends of the ribs.

The resident engineer was notified and he authorized drilling a 1/4 inch diameter hole in the bottom of two ribs in the leaky area. Water immediately flowed out and continued to run for as long as 1-1/2 hours. Adjacent ribs were then drilled and water also flowed out of these.

After considerable examination and discussion it was determined that heavy seasonal rains had caused a large amount of water to flow over the low end of the bridge (North end) and then seep into the open seam between the deck plate and the end plate, eventually filling the ribs with the water.

To correct the situation the ribs were drained and the open seam was hand welded with two passes and groun flush. The curb blocks were removed at the fascias to facilitate welding the seam across the fascia ribs and then replaced with new concrete.

The small drain hole drilled into the bottom of each rib in the leaky area was tapped and a threaded plug inserted.

The identical seam on the south end of the bridge had been properly welded and ground flush just after erection. The ribs on this end were checked for leakage and no water was found to be present in them.

15. <u>SLIGHT EXTERIOR CRACKS ON RIB SECTIONS</u>:

Small exterior hairline cracks appeared on various ribs after construction of the bridge had been completed. Some of the more prominent cracks were on the exterior rib adjacent to the west girder web near Abutment B.

Three representatives from Bethlehem Steel Company (including a metallurgist) examined the areas of rib cracking at the bridge site. After filing smooth a cracked area, a die penetrate was used to prove the cracks had a very shallow depth. They explained that this "slivering effect" is quite common in rolled plates and results from rolling thin traces of foreign matter into the exterior side of the plate during the rolling process. Later, slight surface cracks appear in these areas but are not serious enough to affect the structural strength of the member.

As a remedy, all affected areas on the rib sections are to be ground smooth and painted with a proper water-proofing paint to protect these areas from moisture deterioration.

A complete report of the condition and the corrective procedure to follow will be submitted by the metal-lurgist. A copy will be attached to this report.

Bethlehem Steel Corporation

DISTRICT SALES OFFICE: 3-235 GENERAL MOTORS BUILDING

DETROIT, MICH. 48202

D. C. TURNER
MANAGER OF SALES
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BETHLEHEM STEEL

PHONE: 875-9720
AREA CODE 313

January 5, 1970

Michigan Department of State Highways
Highway Building, Drawer K
Lansing, Michigan 48904

Attention: Mr. Gerald Hill
Construction Division

Gentlemen:

Subject: Orthotropic Rib Sections
Creitz Road Bridge
Lansing, Michigan

Regarding our telephone conversation on this subject and your request for a letter related to the jobsite visit by Mssrs. Ffield, Maykovich and the writer, the following information should suffice.

> "The investigation at the jobsite disclosed that all so-called "cracks" in the two ribs were due to slivers in the plate material from which the section was fabricated. During roll forming, the slivers opened up leaving an indication of a crack.
>
> Further investigation of the surface of the plate by Mr. Ffield showed that the "cracks" were extremely shallow. None of the "cracks" could be considered as structurally weakening the section.
>
> In our opinion, based on our investigation of the two ribs of the orthotropic sections in the Creitz Road bridge, these ribs were structurally sound."

We believe the above answers your request. If not, please advise what additional information is required.

Very truly yours,

BETHLEHEM STEEL CORPORATION
R. C. Wakefield, Contracting Manager

By: C. A. Bengston

CABengston:cjp

JOINT COMMITTEE REPORTS

COMMENTARY:

In November of 1967, John E. Meyer, Chief of the Bureau of Engineering for the Michigan Department of State Highways, formed a joint committee to follow and to report on the experimental features of Michigan's first orthotropic bridge project (S05 of 23081A) Research Project 67G-157, carrying Crietz Road over I-496 West of Lansing.

Mr. Meyer designated three Divisions of our Department to be represented by this joint committee and they were as follows: The Design Division, the Construction Division, and the Testing & Research Division.

The following joint committee reports are included as part of this final report, to illustrate and record certain events and observations that this joint committee reported during the construction of Michigan's first orthotropic bridge.

INITIAL - PRECONSTRUCTION REPORT

OF

EXPERIMENTAL BRIDGE PROJECT

S05 of 23081A

ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD OVER I-496 WEST OF LANSING

December 20, 1967

NATURE AND OBJECTIVE OF EXPERIMENT

Michigan's first orthotropic steel deck plate bridge is now under contract and is expected to be open to traffic in the Fall of 1969.

PURPOSE OF EXPERIMENT

The orthotropic type deck was selected for the Crietz Road bridge for three main reasons:

(1) To obtain information on this type of construction which will be useful for the design and construction of future bridges of this type.

(2) To observe and compare the performance of this type of structure under traffic, and its maintenance requirements with the conventional type bridge, as well as to identify construction and maintenance problems inherent in orthotropic plate construction.

(3) To provide an esthetically pleasing structure.

ADVANTAGES AND FEASIBLE APPLICATIONS

(1) Extremely adaptable for very long spans due to its unique type of design.

(2) The light weight of the superstructure results in a savings in substructure which can be particularly advantageous in locations with extremely poor soil conditions.

(3) In addition to the advantage of eliminating the heavy concrete deck which could deteriorate, the orthotropic type bridge can be erected in a matter of days, since practically the entire superstructure is welded together in the shop and shipped to the building site in a few units.

(4) We anticipate the deck will require much less maintenance which has been quite a problem area with the conventional concrete bridge decks.

(5) It has a better load carrying capacity, as the deck, girders, and floor beams are integrated into one structural element, making it a highly indeterminate structure which has the ability to transmit load effect in different directions, thus resulting in a very efficient design.

(6) Elimination of side piers, thus giving wider traffic clearance under the structure and resulting in much better safety features.

ESSENTIAL FEATURES

This orthotropic steel deck bridge is a two-span continuous structure and will have spans of 96'-0", and will have a clear roadway width of 32'-6" with two 9" wide brush blocks.

The superstructure will consist of a 7/16" steel deck plate supported by two 54" deep welded steel plate girders spaced at 24'-0" and 24" deep transverse floor beams spaced at 15'-7" on centers. The deck plate, which is stiffened by 5/16" thick longitudinal trapezoidal shaped stiffeners on 2'-0" centers, serves both as the bridge deck and the top flange for the main girders and floor beams. The floor beams will be notched to allow the stiffeners to run continuously throughout the length of the bridge. These stiffeners will be shop welded to the floor beams and the deck plate which, in turn, are shop welded to the two main girders. A 12" fascia channel will be shop welded to the outside ends of the floor beams. Web stiffeners will be welded on both sides of the girder webs and knee braces will be provided between the girders and floor beams at alternate floor beams.

The shop fabricated structure will be fabricated in six, 12 foot wide by 68 foot long sections and three 12 foot wide by 56 foot long sections.

Two 10" high concrete safety curbs, secured by 3/4 round by 7" steel studs welded to the deck plate, will support a three-tube bridge railing.

The substructure will consist of concrete curtain wall abutments supported by 12" cast-in-place concrete piles and a two, welded-steel-box-column pier supported by concrete spread footings. The main girders of the bridge will rest on elastomeric bearing pads at the pier and self-lubricating bronze bearings at the abutments. Expansion will be provided at the abutments by means of preformed neoprene joint seals.

DESIGN

This structure was designed basically in accordance with the Design Manual for Orthotropic Steel Plate Deck Bridges published by the American Institute of Steel Construction, and also the current AASHO Standard Specifications for Highway Bridges.

A design-history report is presently being written and the final report will include all the reports (design and construction) with an analysis of the completed project.

A sketch of the "Elevation" and the "Deck Section at the Pier" are attached to the back of this report for a brief illustration of the structure.

PLAN OF STUDY AND EVALUATION

The following is a brief outline of the fabrication, construction and post construction testing and observation phases of this experimental structure which will be conducted and reported on.

FABRICATION

1. A prefabrication meeting will be held at the fabricating plant with representatives present from Design, Construction, Testing & Research, and the fabricators themselves. This meeting will be held as soon as an approved set of shop drawings has been received. The complete fabrication process will be gone over at this meeting.

2. A welding procedure will be submitted prior to welding and as the welding progresses.

3. The Michigan Department of State Highways will have a man inspecting at the plant and he will send a weekly report on the fabrication progress and development.

4. A final and complete fabrication report will be compiled when the fabrication is completed.

CONSTRUCTION

1. Observations of the field welding of the prefabricated deck sections. (Features of this operation may affect the application of the epoxy mortar surfacing.)

CONSTRUCTION (Cont'd.)

2. Observations of placement of concrete curbing and anchoring to the steel deck.

3. Application of the epoxy mortar surfacing.

    (a) Preliminary sampling and testing of epoxy binder material.

    (b) Initial appraisal of all equipment to be used in sand blasting, measuring, mixing, placing, and finishing operations.

    (c) Blast cleaning of steel deck plate and concrete curbs.

    (d) Application of prime coating to deck plate.

    (e) Mixing and placing of epoxy mortar surfacing with surface application of dry sand.

    (f) Spot sampling of epoxy mortar mixes to be included for laboratory tests.

    (g) Curbing of surfacing mixture.

4. Placement of preformed neoprene seal in expansion dams.

The project engineer will issue a more detailed weekly construction report for this experimental bridge rather than the usual bi-weekly report on normal jobs.

A final and complete construction report will be made after the entire construction of this project is completed.

POST CONSTRUCTION

1. Initial and annual surface roughness and profile determinations along several longitudinal axes.

2. Initial skid tests, followed by additional skid tests at 3 month intervals during the first year with subsequent tests to be conducted twice a year thereafter.

3. Initial and semi-annual condition surveys with particular attention to the epoxy mortar surfacing.

POST CONSTRUCTION (Cont'd.)

4. Observations on frequency and severity of deck surface icing supplemented with possible skid testing (without water).

5. Periodic inspections of the whole structure to observe the performance of the many weld details.

It is also planned to keep Mr. F. Fondriest of Battelle Memorial Institute informed of the construction progress as he is planning to be at the job site during surfacing operations.

In addition, an experimental stress analysis involving strain measurements on the various deck components under actual as well as controlled loading sequences for comparison with theoretical analysis will be conducted. A detailed outline of this phase of study will be provided in a later report.

A visual record of the fabrication and construction of this project in the form of slides and/or a small film will be prepared by the Graphic Presentation Unit of our Testing and Research Division.

MATERIALS TO BE USED

All of the structural steel will be A441.

The girders, floor beams, and stiffeners will be spliced in the field by A-325 high strength bolts.

The bridge railing will consist of a three-tube galvanized steel railing.

The wearing course will consist of a two types of epoxy mortar. Guard Kote 250 epoxy will be placed on one-half of the deck and Epon 815 epoxy will be placed on the other half. Filler material will consist of dry 2NS sand.

EQUIPMENT TO BE USED

Standard shop welding procedures in accordance with the latest AWS specifications will be used throughout the entire fabrication of this structure.

The deck plate will be spliced in the field by automatic submerged-arc welding equipment.

EQUIPMENT TO BE USED (Cont'd.)

The equipment used in placing and finishing the epoxy wearing surface will be mechanically vibrated screeds and power trowels as described in the Supplemental Specifications of the Proposal.

ESTIMATED COST OF STRUCTURE

The Crietz Road Bridge contract was let at approximately $202,000, which is about 40% more than a two-span continuous steel girder bridge with a concrete deck and concrete pier.

TRAFFIC

VOLUME

The present average daily traffic count is estimated at 300.

The future average daily traffic count is estimated at 13,000 for 1988.

CHARACTER

The character of travel over the structure is "passenger", with about 10% commercial.

LOADING: This structure has been designed for an H20 Live Loading.

DISCUSSION

Other projects involving similar experimental deck plate features include California's "San Matao" Bridge and their Dublin Bridge. Recently, the "Poplar Street" Bridge across the Mississippi River between Illinois and St. Louis has also been completed.

ESTIMATED CONSTRUCTION STARTING TIME

Construction on the Crietz Road bridge will probably begin after the adjacent Snow Road bridge has been completed. The fabrication of the Crietz Road job will take place in the shop during the Winter and Spring months of 1968 and actual erection of the structure will take place sometime in the Fall of 1968. The bridge is to be open to traffic no later than Fall of 1969.

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

Report of 1st. Joint Committee Meeting
of Experimental Bridge Project
S05 of 23081A
Orthotropic Bridge Carrying Crietz Road
Over I-96 West of Lansing

Date of Meeting - December 1, 1967

Place of Meeting - Conference Room, 8th Floor, Mason Bldg.

Time of Meeting - 9:00 A.M. to 10:40 A.M.

Members Present Were:

John Risch - Design Division - Chairman
Fred Rieger - Design Division
Gene Cudney - Testing & Research Division
Charles Ellis - Construction Division
Jerry Hill - Construction Division

A brief appraisal of the structure and its construction was examined and a plan of action for the committee was set.

The following items were discussed and agreed upon:

FABRICATION:

(1) A pre-fabrication meeting will be held at the fabricating plant in the near future with representatives present from Design, Construction, Testing and Research, and the fabricators themselves. This meeting will be held as soon as an approved set of shop drawings has been received. The complete fabrication process will be gone over at this meeting.

(2) A welding procedure will be submitted prior to welding and as the welding progresses.

(3) We will have our own man inspecting at the plant and he will send us a weekly report on the fabrication progress and development.

(4) A final and complete fabrication report will be compiled by Jerry Hill when the fabrication is completed.

CONSTRUCTION:

(1) Construction on this Crietz Road bridge will probably begin after the adjacent Snow Road bridge has been completed, thus keeping the area open to traffic. The Snow Road bridge project will most likely be constructed quite soon while fabrication of the Crietz Road job will take place in the shop during the Winter and Spring months.

(2) Charles Ellis will ask his men to issue a more detailed weekly construction report for the Crietz Road bridge rather than the usual bi-weekly report on normal jobs.

(3) A final and complete construction report will be made after the entire construction of this project completed.

TESTING AND RESEARCH:

(1) A brief discussion was held on the feasibility of examining the structure or certain portions of it for stress analysis to compare the actual results with the design calculations. Since a few design discrepancies were observed by our Design Section in comparison with the orthotropic design manual, and since this is an experimental bridge, it was agreed that Testing and Research should conduct a short stress analysis to compare with Design.

(2) Mr. D. E. Jones of the Bureau of Public Roads has asked that we measure and evaluate the initial skid resistant properties and roughness of the epoxy deck wearing surface and perform periodic checks thereafter to ascertain what effects, if any, traffic loading will have on the determination or failure of the relatively thin wearing surface. He also thought it would be desirable to relate these measurements to traffic counts, if this is possible. Gene Cudney has assured us that a complete report on this matter will be submitted.

(3) The possibility of a visual record of the fabrication and construction of this project in the form of slides or a small film was discussed and it was agreed that for an experimental bridge of this nature, a visual record would be interesting as well as of some value in the future. The committee agreed to contact R. L. Greenman on this matter and request the cooperation of the Graphic Presentation Unit.

DESIGN:

(1) John Risch of the Design Unit (who is acting as Chairman of this committee) will coordinate the reports of the committee as they come in and furnish copies to Mr. Meyer, Mr. Jones, etc., and all individual members of the committee. Field trips will be organized and dates of future meetings will be set as the work on the project advances.

(2) A Design Report is already being written and the final report will include all the reports with an analysis of the completed project.

(3) A Preconstruction Report is presently being prepared by John Risch and copies will be sent to the Bureau of Public Roads as soon as it is completed.

John E. Risch
JOHN E. RISCH, Chairman

DD-JER:dc

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

Report of Joint Committee Fabrication
Inspection Trip of Experimental Bridge Project
S05 of 23081A
Orthotropic Bridge Carrying Crietz Road
Over I-496 West of Lansing

Date of Inspection Trip - October 15 and 16, 1968

Place of Inspection Trip - Bennett Industries, Incorporated
Peotone, Illinois

Members in Party Were:

John Risch - Design Division - Chairman
Charles Ellis - Construction Division
Sam Lee - Construction Division
Fred Cassel - Testing & Research Division

PURPOSE OF INSPECTION TRIP

Our Committee felt that an actual inspection of the fabrication process of this unusual type structure would allow us to review the complete fabrication procedure under shop conditions and enable the fabricator to present some of his problems to us.

Our Testing and Research Division also had pictures and slides taken of the fabrication procedure for our Department's future reference and audience presentation.

SUBJECTS DISCUSSED

The main subject of this fabrication meeting concerned the fabrication procedure, problems involved in fabrication, and possible changes in future designs of this nature resulting from present experience.

Numerous items were discussed and examined in detail but a few of the more important items were as follows;

(1.) PROBLEM OF ROLLED RIB SHAPES

It has been difficult for the rolling mills to accurately form the trapezoidal shaped ribs to their proper dimensions using pre-cut plates in their rollers.

PROBLEM OF ROLLED RIB SHAPES (Cont'd)

A suggestion was made by Andy Roush (who is in charge of the fabrication of this structure at Bennett Industries, Inc.) that in the future we specify the rolling mill to perform a "roll and shear" type process whereby oversize rib plates would be put through the shaping rollers and the tops of the ribs simultaneously cut at a 7^{o} to 10^{o} bevel.

This type of process would result in a very straight edge along the top of the rib section and with a 7^{o} to 10^{o} bevel we could expect a 100% weld penetration where the deck plate is welded to the tops of the ribs.

(2.) FLOOR BEAM COPE SIZE

Our present plans call for a 1 inch radial cope in the top of the floor beam web where it meets the top of the rib and the deck plate.

This cope allowed the three welds to meet properly and it also provides stress relief in the plates at this junction.

During fabrication, however, it was found necessary to increase the cope depth to at least 1-1/4 inches at the point where the top of the end floor beam web of each deck unit met the top of the main girder web. This deeper cope allowed a proper hand weld where the automatic welding rig could not pass through the floor beam web when making the longitudinal weld along the main girder.

(3.) BEARING STIFFENER WELDING AT ABUTMENTS

It is very difficult for the fabricator to weld pairs of wide bearing stiffeners at the abutments. Clearances are too small; especially, where the knee braces join to the main girder.

We have allowed the fabricator to bevel the outside edge of the bearing stiffener next to the girder web and then provide a full 3/8 inch weld (same as girder web thickness) along the outside edge of the bearing stiffeners. The 3/8 inch weld is still adequate to transmit the shear load of the girder web to the bottom of the bearing stiffeners as design requires.

(4.) SPLICE HOLES IN THE RIBS

The question of using a template and punching the holes in the sides of the ribs at the splice joints instead of drilling was raised.

After some consideration, it was decided that punching the holes would deform the sides of the ribs and then the holes would have to be ground flush on the inside of the ribs. Also, it would be hard to align and place a large punching press accurately on the sides of the ribs.

(5.) MISCELLANEOUS ITEMS OF INTEREST

(a) All sections and portions of the superstructure were initially tack welded in correct position prior to final welding.

(b) 1100 lbs. to 1500 lbs. of powdered welding flux were blown into the ribs of each unit to prevent burn through and spattering when welding the ribs to the deck plate.

(c) As the ribs were drawn through the welding jig for their final weld to the deck plate, a torch was run along the bottom of the rib ahead of the submerged arc nozzles to help counteract the shrinkage and camber effect caused by welding the top of the ribs to the deck plate.

(d) The interior of the ribs and underside of the deck plate were sandblasted prior to welding.

(e) A template was formed to mark the floor beams for cutting around the ribs. Each floor beam was marked and then hand-cut to fit its particular location and then hand welded in place.

(f) The pier columns and connecting strut were fabricated early in 1968 and were entirely completed and painted at this time.

(g) Pictures and slides were taken of most of the fabrication and welding procedures with some close up detail pictures included.

John E. Rich
Design Division

October 23, 1968

Minutes of Meeting to Discuss Erection and Field Welding of the Structural Steel for the Creitz Road Orthotropic Bridge, I 23081 002 (S05)

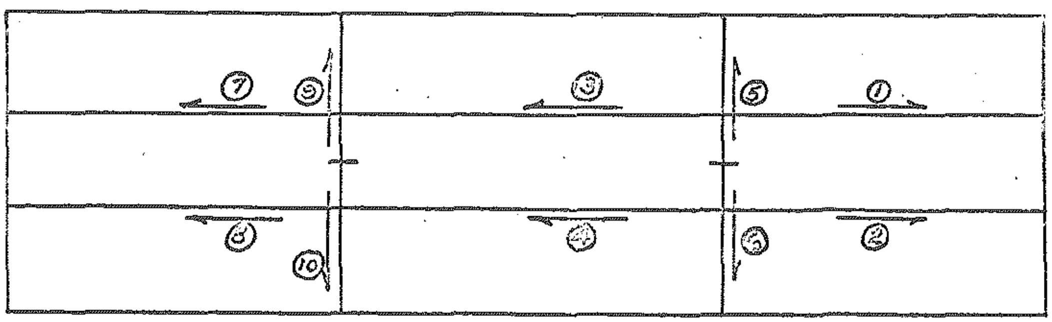
The meeting was held on October 23, 1968 in the Bureau of Operations Conference Room on the First Floor of the New Highway Building.

Attending were: Jim Lancaster - Midwest Bridge Division
Charles Bodary - "
C. M. Ellis - Department of State Highways
F. D. Rieger - "
J. E. Risch - "
G. R. Cannell - "
S. Sanders - "
C. R. Clensy - "
G. J. Hill - "

The first item discussed was the fastening of lifting lugs for erection purposes. It was decided that Mr. Lancaster would contact Bennett Industries in an effort to have the lugs shop welded on the nine sections at specified locations with special emphasis on providing for longitudinal, that is parallel to center line of the bridge, welding of the lugs in all cases. The lifting lugs should provide for more wrap around on the hook end to decrease the chances of pulling the cable out with a sideward lift. See detail attached.

The second item of discussion related to the erection of all nine sections of the bridge prior to field welding. Bolts in all splices shall be snugly tightened prior to welding. This differs from that required by supplemental specification, but will provide for a much more desirable fit and welding sequence.

The following is the welding sequence:



All joints will be power wire brushed prior to tacking with 5/32" diameter E7018 electrodes. Power wire brushing will also follow the tacking operation, or be completed just prior to the automatic welding process. Since power brushes tend to throw small pieces of wire, care should be observed by all concerned during this operation.

Any temporary welding for fit-up purposes will be done in a longitudinal direction and ground flush upon removal.

Minutes of Meeting October 23, 1968

All longitudinal seam welds shall start or end 6" from each transverse seam weld. These 1' segments will be the last sections welded.

The sequence of welding the back-up bars is very important and must be adhered to without exception. The butt welds of the transverse back-up bars shall be made after welds 1 and 2, 3 and 4 are complete, but prior to welds 5 and 6. No welding shall be done on the longitudinal back-up bars at this time. When welds 5 and 6 are complete, the longitudinal back-up bars are to be made continuous by welding complete to the transverse bars. This same process should be followed on the other portion of the bridge.

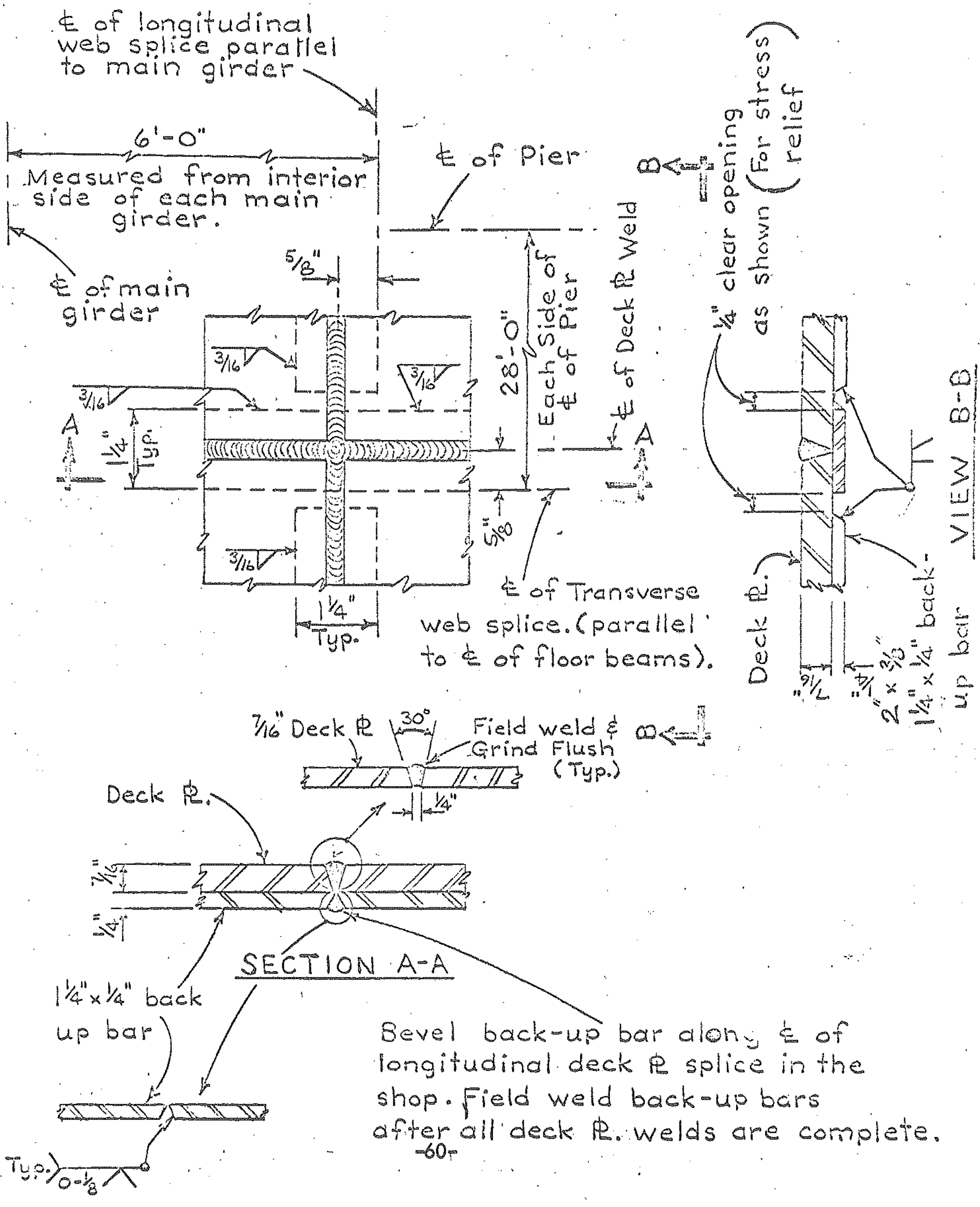
Back-up bars are to be increased in size from ¼" x 1¼" to 3/8" x 2" to eliminate the possibility of burn through or side blow during field welding. The Department will pay for the increase in weight of the heavier bar.

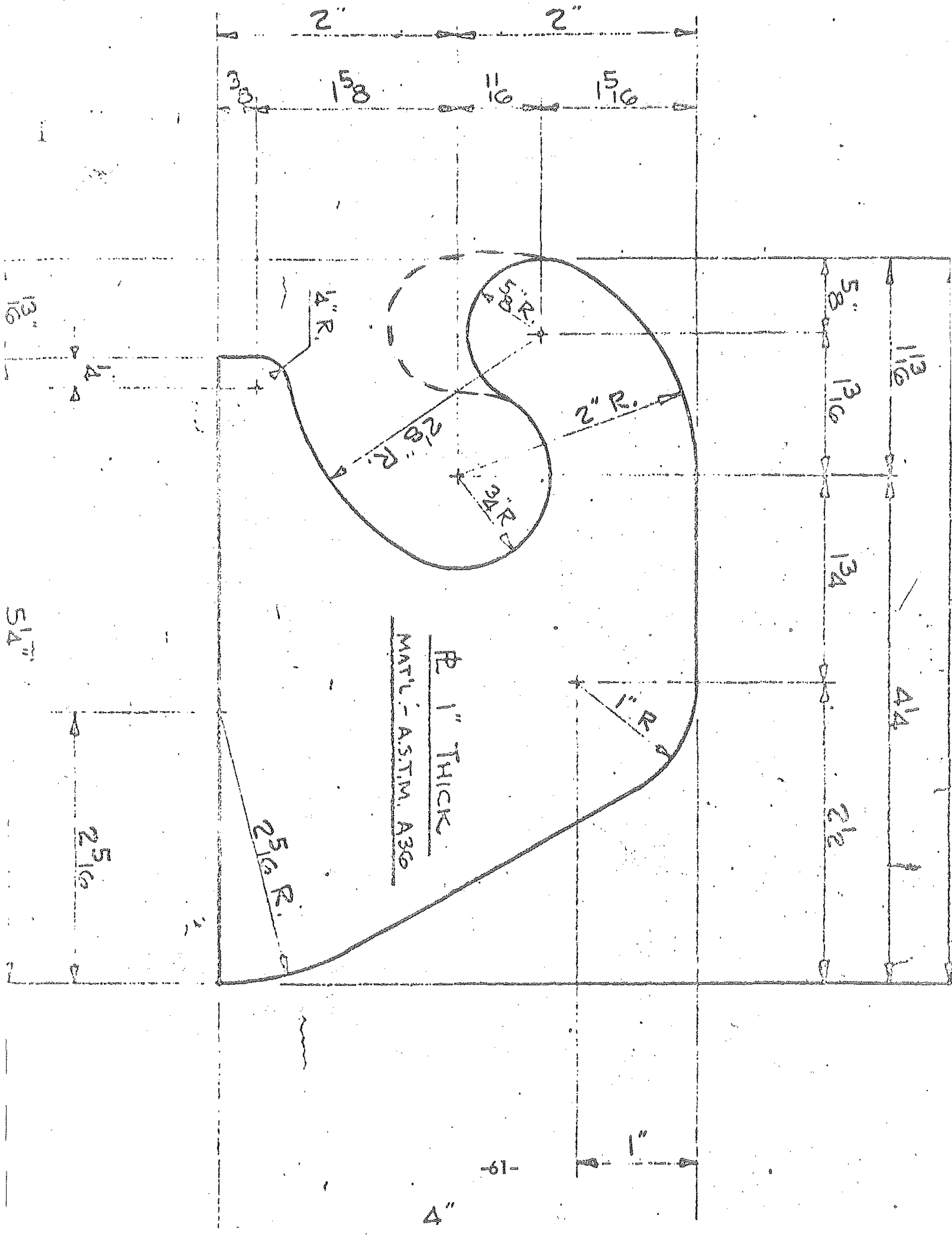
Mr. Lancaster will make arrangements and coordinate the radiography for this project.

Gerald J. Hill
Construction Staff Engineer

GJH:dal
cc: Bureau of Public Roads
E. Bosak
A. H. Jones

DECK SPLICE DETAILS





2"
2"
3/8
1 5/8
1/16
1 5/16
13/16"
1/4"
1/4" R.
5/8" R.
2 1/8" R
2" R.
3/4" R
5 1/4"
PL 1" THICK
MATL.- A.S.T.M. A36
1" R
2 5/16 R.
2 5/16
5/8"
1 13/16
1 3/16
1 3/4
4 1/4
2 1/2
1"
4"

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

REPORT OF PRE-ERECTION MEETING
OF EXPERIMENTAL BRIDGE PROJECT

S05 of 23081A

ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD
OVER I-496 WEST OF LANSING

DATE OF MEETING:	OCTOBER 23, 1968
PLACE OF MEETING:	CONFERENCE ROOM OF CONSTRUCTION DIVISION
TIME OF MEETING:	1:30 PM to 4:15 PM

MEMBERS PRESENT WERE:

Charlie Ellis	Construction Division
Jerry Hill	Construction Division
Ray Clensy	Construction Division
George Cannell	Construction Division
Sam Sanders	Construction Division
Chuck Bodary	Midwest Bridge Company
Jim Lancaster	Midwest Bridge Company
Fred Rieger	Design Division
John Risch	Design Division

The following items were discussed and acted upon:

1. Using a small model of our orthotropic structure, Jim Lancaster illustrated how he would erect the three deck units over the pier first and then shore them adequately with temporary pile bents from below. Bolts in the transverse splices would be brought to a snug tight condition at this time.

   The three units at Abutment A would then be placed next and the bolts in the connecting splices be brought to a snug condition.

   The three remaining units at Abutment B would be placed last and in a similar manner to the units at Abutment A.

   In all cases the fascia units containing the main girder would be placed first and the center deck unit lifted into position between the fascia units.

2. The temporary pile bents will consist of timber piles with adequate cross members to support the structure. All temporary shoring will be removed later in accordance with our present specifications.

   It was pointed out that in a few instances, these temporary bents may actually act as tie-down units for tipping, wind forces, etc., while erecting adjacent units.

3. The bolts in all the splices will be brought to a snug condition before any deck plate welding takes place.

4. Lifting lugs are to be shop welded to the deck units and welded at a point on the deck plate where the top of a rib meets the web of a floor beam.

   The lug is to be placed parallel to the side of the rib so its weld to the deck plate will be in a longitudinal direction.

   If open hooks are used, the open portion shall face toward the interior of the unit.

5. Two lifting cranes will be used to lift each deck unit into position on the structure.

6. The Supplemental Specifications that we arranged for this structure specify that the field welding of the deck plate seams be completed for each three units before proceeding to the next three units. This specification applies to cantilever type erection (such as the Poplar St. Bridge in St. Louis) rather than our particular situation.

   This specification will be revised as follows: "All splice bolts in the splices will be brought to a snug condition before beginning deck plate welding. The longitudinal seams of the three deck units at Abutment "A" and the three deck units over the pier shall be welded first. Then the transverse seam joining the Abutment "A" units and the pier units shall be welded.

   The remaining longitudinal seams of the three deck units at Abutment "B" shall follow. Finally the transverse seam joining the three Abutment "B" units and the pier units shall be welded.

7. The deck plate will be preheated before welding the deck plate seams.

8. Discussion was held concerning the scheduling of radiographers. Jim Lancaster agreed to handle this matter and have enough men and scaffolding to assist the radiographers.

9. There is a strong possibility the epoxy wearing surface of this structure will be placed next Spring.

   A No. 6 Commercial Type sandblasting operation is specified to be performed prior to placing the epoxy wearing surface.

   For this reason the members present at this meeting felt protective treatment for the deck during the Winter months would not be necessary.

   However, the Construction Division agreed to contact the Bureau of Public Roads and obtain a decision on this matter.

10. Jerry Hill agreed to submit welding details consisting of back-up-bar details, joint details, welding deck unit sequence, etc., to the members attending this meeting.

    The Bureau of Public Roads will be contacted and agreement reached with them before Jerry submits his details to us.

John E. Risch

Design Division

DD-JR:dc

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

REPORT OF POST-FABRICATION MEETING OF
EXPERIMENTAL BRIDGE PROJECT

S05 of 23081A

ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD
OVER I-496 WEST OF LANSING

DATE OF MEETING:	January 22, 1969
PLACE OF MEETING:	Conference Room of Construction Division
TIME OF MEETING:	9:00 AM to 11:45 AM

MEMBERS PRESENT WERE:

Charles Ellis	Construction Division
Gerald Hill	" "
Gary Grimes	" "
George Cannel	" "
John Michaels	" "
Fred Rieger	Design Division
John Risch	" "
Ed Bosack	Bennett Industries
Andy Roush	" "
Rick Lidberg	" "
	" "

A substantial amount of experience was gained by Bennett Industries while fabricating their first orthotropic bridge.

Following, is a list of suggestions they would like to advise us of in the event we may design and fabricate another orthotropic bridge.

(1) ROLLED RIB SHAPES:

The trapezoidal shaped rib sections for the Crietz Road structure were formed by passing universal mill plates through special rolling mills at the Bethlehem Steel Corporation.

However, the uneven edges and sometimes considerable amount of sweep present along the edges of universal mill plates, caused a substantial amount of variation in the rib dimensions after passing through the rolling mills.

To prevent this problem in the future, Bennett Industries suggested that we allow the fabricator to pre-cut the rib plates to exact size using a "plasma arc cutting process" prior to the trapezoidal shape rolling procedure.

In this process a multi-torch burning machine would automatically flame-cut several rib plates to precise width in one operation. The plates would then be cold-rolled to form the trapezoidal shaped "trough" sections of the ribs.

In addition, the cutting arc of this process cuts through the plate at a 7° to 10° arc, thus leaving the edges of the plate beveled by this same amount, in contrast to the 90° cut edge of our standard cutting torches. The beveled edge left by the plasma arc process would guarantee a 100% weld penetration when the rib sections are welded to the bottom of the deck plate.

The length of rib plates prior to the rolling process should be increased a few feet at each end to allow for easier handling and also a more accurately rolled section at the design cutoff point.

(2) ANCHOR BOLT HOLES:

On future orthotropic structures we will increase the diameter of the anchor bolt holes at the pier column base for easier field placement of the units over the anchor bolts projecting from the concrete pedestals.

The size of the main girder sole plate slots will also be increased at the abutments as well as the pier to allow better alignment of the superstructure joints prior to completing the splices.

(3) GIRDER KNEE BRACES:

Extra care had to be used by the fabricator in handling the extremely flexible superstructure units where permanent deformations of the thin sections could easily occur.

The fabricator felt that box-type girders would have given him much more rigidity while fabricating the girder units of this structure; especially, the fascia portion of the deck extending beyond the main girders under the sidewalks. Box-type girders would also facilitate more automatic welding.

On other structures with longer spans and possibly wider roadway widths, the design and economy might justify using box-type girders.

However, on future orthotropic structures using girders similar to the Crietz Road project, we might add extra knee braces to the exterior part of the girder web in addition to those detailed on the interior. This would greatly stiffen the fascia cantilevered portion of the deck and also help reduce distortion problems caused by the welding.

(4) RIB SPLICES:

It was impossible for the fabricator to hold a true transverse crown in the deck plate at the end of the deck units. Distortions due to welding the deck plate to the ribs cause the deck plate to shrink and form short chords across the tops of the ribs. This rippling effect in the deck plate is quite a problem when trying to fit the units together in the field prior to splicing.

On future jobs, Bennett Industries would favor welding transverse channels or plates across the end of the ribs with the top of these transverse plates cut to fit the crown of the roadway and welded to the bottom of the deck plate. Holes could be drilled through these plates between the ribs and short bolts used for splicing the deck units together.

In addition to holding a true crown of the deck plate, fitting and splicing the deck units together in the field would be much easier and faster.

(5) COSTLY FABRICATION PROCESS:

Future design might dictate a wider cut in the floor beam webs where the ribs pass through, thus providing more allowance for small variations in rolled rib dimensions.

Automatic machine cutting would then replace a substantial amount of hand cutting, and the bottom of the rib would be welded to the floor beam web, leaving the sides of the rib weld free.

This process would greatly reduce future fabrication costs.

(6) DECK PLATE BACK-UP BARS:

The fabricator felt it would be easier and less costly to drill and tap the back-up bars to the deck plate after the deck units have been completed and shop fitted.

After field erection of the deck units, the back-up bars could be brought to their proper position and bolted in place.

This procedure would facilitate fitting and handling costs on future jobs.

John E. Fisch

Design Division

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

JOINT COMMITTEE REPORT
OF FIELD ERECTION PROGRESS

EXPERIMENTAL BRIDGE PROJECT

S05 of 23081 A

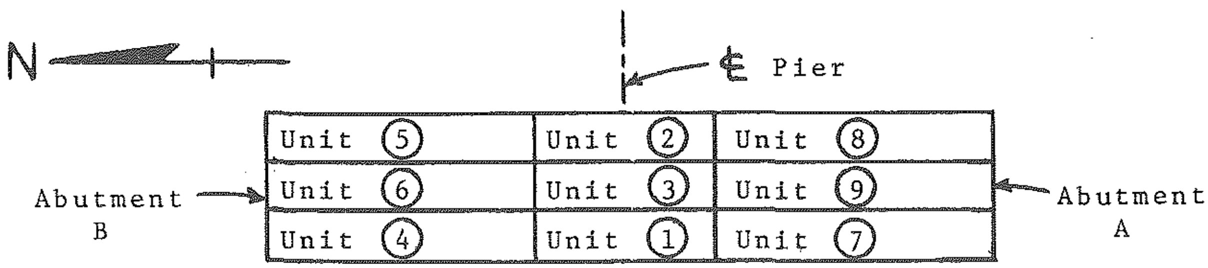
ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD
OVER I-496 WEST OF LANSING

MARCH 10, 1969

PURPOSE OF REPORT

The information presented in this report is intended to describe the field erection of the main deck units and point out a few items observed during erection that might possibly be changed on future jobs of this nature.

DECK UNIT NUMBERING SYSTEM



ERECTION UNIT TIMETABLE

January 14, 1969 - Erected the steel pier and the west fascia center pier unit 1.

January 15, 1969 - Erected the east fascia center pier unit 2 and the middle center pier unit 3.

January 16, 1969 - Erected southeast fascia unit 8 at Abutment A.

January 17, 1969 - Erected southeast fascia unit 7 and the middle center unit 9 at Abutment A.

January 20, 1969 - Erected northeast fascia unit 5, the northwest fascia unit 4, and the middle center unit 6 at Abutment B.

COMMENTS

(1) The steel pier and superstructure units were shipped on railroad flat cars from Bennett Industries in Peotone, Illinois to the railroad yard on the southwest side of Lansing. They arrived in early January of 1969.

(2) Each unit was loaded upright and trucked individually to the job site at Crietz Road and erected in the sequence as shown in the timetable above.

(3) Two cranes were on the job site for lifting the units from the trucks into their correct deck position using the lifting lugs provided in the shop.

(4) Each unit was held in correct position by the cranes while they were being braced against the temporary pile bents below. At this time high strength bolts were placed in the web and bottom flange plate splices of the main girders and floor beams. A few of the rib splices were bolted together at this time also.

(5) It was difficult to place the steel pier columns over the anchor bolts projecting from the concrete pier pedestal.

The plans called for holes 3/16" larger in diameter than the anchor bolts. However, there were eight anchor bolts for each column and the base plate was 1-1/4" thick. Even the slightest variation in anchor bolt horizontal placement could cause difficulty when placing the steel columns on the concrete pedestals.

A cutting torch was used in the field to cut the holes larger where necessary to allow proper fitting of the pier base plates over the anchor bolts.

In the future we should increase the diameter of the anchor bolt holes in the pier column base plates and also the size of the main girder sole plate slots at the abutments and top of pier columns. Consideration has also been given to casting a base plate in the top of the pier pedestal.

This will not only facilitate easier placement of the pier columns but also better alignment of the superstructure joints prior to completing the deck splices.

(6) There was considerable difficulty encountered in placing the steel rectangular shaped strut between the top of the pier columns.

The strut was detailed to fit between clip angles that were shop welded to the steel columns. Later, after correctly placing the strut, the plans called for field welding the strut to the angles.

However, extensive pounding was necessary to drive the strut down between the clip angles that were welded to the pier columns. Evidently, the steel fabricator failed to allow the proper clearance necessary between the strut and the clip angles for field erection or there remained enough twist in the pier columns caused by the anchor bolt problem to prevent the close fit planned by the fabricator.

(7) Near the main girders, the local stress conditions in the deck plate are complicated by the fact that the girder web acts as a rigid support for the deck plate, while the adjoining ribs act as elastic supports because of their flexibility under wheel loads.

To help prevent these excessive local deck plate stresses over the main girder webs, small stiffening plate diaphrams were welded between the girder web and the first rib at the third-points of the rib spans to provide a more gradual transition between the "hard" girder web and the "soft" ribs.

On future jobs of this nature, it would be more economical and better looking to detail these small stiffening plate diaphrams as being an integral part of the main girder web stiffeners which are placed near the third-points of the rib spans.

(8) The contractor had difficulty placing the nuts on the top of the pier anchor bolts because the haunched beams came down too low to permit placing the nuts directly on the bolts after the girder was set.

To solve this problem the men raised the units a little with the cranes, then slid the nuts into position, and finally lowered the units a little at a time as the nuts were turned down upon the projecting bolts.

On future jobs, wider bearing and masonry plates should be provided with corresponding wider anchor bolt spacing; thus insuring the proper clearance for placing and tightening the nuts upon the anchor bolts.

(9) The foreman in charge of steel erection suggested that in the future, the sealer plates located inside the ribs be placed farther back from the handhole.

This would allow the inside rib splice plates to be brought up through the handhole after the deck units have been correctly positioned for splicing.

(10) Another item that should be considered on future orthotropic jobs is beveling the cut-out portion of the rib walls directly beneath the deck plate for receiving the transverse joint back-up bars.

Our present structure had level cut-out notches whereas if these receiving slots were beveled, the deck units would slip into position much more easily during erection.

John E. Risch

John E. Risch, Chairman
Design Division

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

REPORT OF PRE-APPLICATION EPOXY MEETING
OF EXPERIMENTAL BRIDGE PROJECT

S05 of 23081A

ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD
OVER I-496 WEST OF LANSING

DATE OF MEETING:	September 9, 1969
PLACE OF MEETING:	Conference Room of Design Division
TIME OF MEETING:	9:00 AM to 12:45 PM

MEMBERS PRESENT WERE:

Charles Ellis	Construction Division
John Michaels	" "
Gary Grimes	" "
George Cannel	" "
Fred Rieger	Design Division
John Risch	" "
Keith Wolf	Epoxy Contractor
Bob Morris	Bureau of Public Roads
Myron Brown	Testing & Research
Gene Cudney	" "
Jim Lancaster	Midwest Bridge Company

Following, is a list of items discussed at our meeting.

1. The Epoxy Contractor, Keith Wolf, plans to begin applying the epoxy wearing surface to the orthotropic bridge by September 22.

2. Keith found that "Swanee Silica" sand shipped from Indiana gave the best results on his epoxy jobs. He ordered #20 sand and they shipped him #14 sand containing 2% finer sand than the #20 sand.

3. The Shell Company recommended "rolling" after placing the epoxy material to obtain a better density. This was not part of our contract so it is doubtful if this would be allowed. This decision would be up to T & R and would also result in an extra cost if adopted.

4. The tracks upon which the epoxy screeding equipment travels, consist of metal bars simply resting on the deck plate. No fastening would be necessary to hold them in place. After the epoxy mortar begins to set up the bars are slightly pulled to one side to they do not bond to the mortar.

5. Cold joints will be made in all cases instead of placing the mortar against steel bars or other types of bulkheads.

6. The deck area will be divided into six individual poured sections.

7. Each section is to be prepared and sandblasted in the morning and then poured in the afternoon if weather conditions permit.

8. The main purpose of the prime coat of epoxy is to provide the deck with a waterproof cover and proper bonding agent between the deck and the surfacing material. However, this was desirable for the older types of epoxy where a zinc deck coating was used.

   The Shell Oil Company and Keith Wolf's company both agree that a priming coat is not necessary with these types of epoxy mortar. It would also be extremely difficult to apply in the correct manner and obtain a tight bond to the final wearing course.

   Our Testing and Research Division is currently engaged in actual testing of the priming layer of epoxy and will soon be able to report on its performance. A decision will then be made concerning its application on our Crietz Road Structure.

9. Keith Wolf anticipates an over-run in mortar quantities due to warpage of the deck plate in the field. He feels that a 20% over-run is a reasonable amount and is willing to apply this extra amount without any additional cost to the State. However, Keith would like written assurance of payment for any quantities exceeding 20% of the contract quantity. He proposed two possibilities of providing for this.

9. (a) Change our present bid quantity from "square feet" to "cubic feet".

(b) Keep our present bid "square feet" the same with the same bid price per square foot but allow an increase in over-run exceeding 20% of the contract quantity to be paid for as "cubic feet" at a lesser price than the original bid price.

Keith also felt that it would be easier to place a 3/4" thick layer of epoxy with his screed type machine than the contract thickness of 5/8".

Charlie Ellis felt that if Testing and Research could justify changing our epoxy thickness from 5/8" to 3/4" we might obtain a new bid price per square foot and this obtain compensation for Keith's over-run. The State would have to ask for this change but the Bureau of Public Roads may not go along with it. Also, the end bars on top of the deck plate at the abutments and piers are planned size of 5/8" for the 5/8" thick mortar.

John E. Risch

John E. Risch, Chairman
Design Division

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

JOINT COMMITTEE REPORT
OF FIELD ERECTION PROGRESS

EXPERIMENTAL BRIDGE PROJECT

S05 of 23081A

ORTHOTROPIC BRIDGE CARRYING CRIETZ ROAD
OVER I-496 WEST OF LANSING

SEPTEMBER 11, 1969

PURPOSE OF REPORT

The information presented in this report is intended to describe two problems encountered with the rib sections after construction of the bridge had been completed, but prior to placing the epoxy wearing course.

(1) WATER IN THE RIB SECTIONS:

On August 11, 1969, our field engineer noticed water running down through the expansion joint and over the abutment face at the north end of the bridge. After further examination it was determined that water had managed to get into the rib sections and was leaking out of a thin open seam left between the deck plate and the end plate welded across the ends of the ribs.

The resident engineer was notified and he authorized drilling a 1/4 inch diameter hole in the bottom of two ribs in the leaky area. Water immediately flowed out and continued to run for as long as 1-1/2 hours. Adjacent ribs were then drilled and water also flowed out of these.

After considerable examination and discussion it was determined that heavy seasonal rains had caused a large amount of water to flow over the low end of the bridge (North end) and then seep into the open seam between the deck plate and the end plate eventually filling the ribs with the water.

To correct the situation the ribs were drained and the open seam was hand welded with two passes and ground flush. The curb blocks were removed at the fascias to facilitate welding the seam across the fascia ribs and then replaced with new concrete.

The small drain hole drilled into the bottom of each rib in the leaky area was tapped and a threaded plug inserted.

The identical seam on the south end of the bridge had been properly welded and ground flush just after erection.. The ribs on this end were checked for leakage and no water was found to be present in them.

(2) SLIGHT EXTERIOR CRACKS ON RIB SECTIONS:

Small exterior hairline cracks appeared on various ribs after construction of the bridge had been completed. Some of the more prominent cracks were on the exterior rib adjacent to the west girder web near Abutment B.

(2) Cont'd)

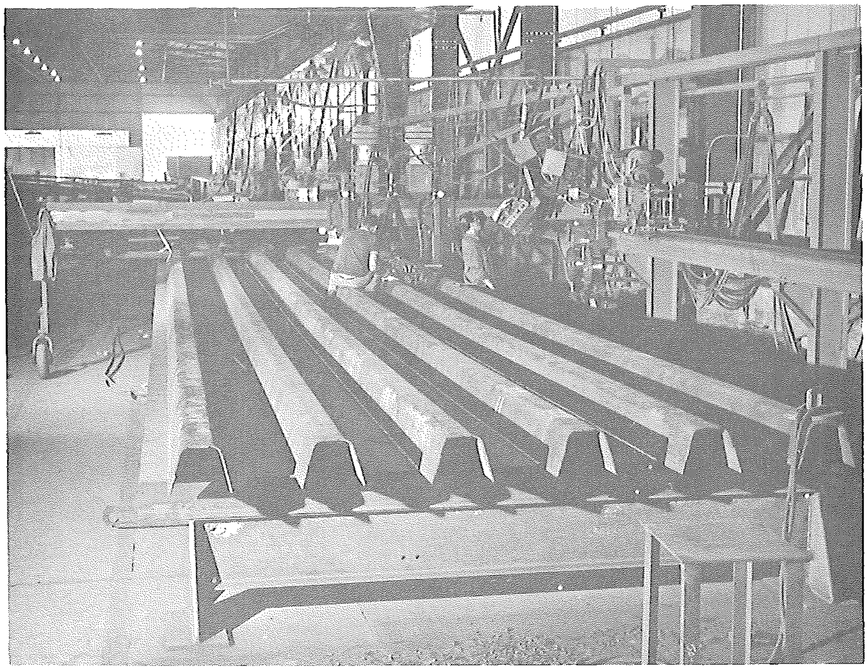
Three representatives from Bethlehem Steel Company (including a metallurgist) examined the areas of rib cracking at the bridge site. After filing smooth a cracked area, a die penetrate was used to prove the cracks had a very shallow depth. They explained that this "slivering effect" is quite common in rolled plates and results from rolling thin traces of foreign matter into the exterior side of the plate during the rolling process. Later, slight surface cracks appear in these areas but are not serious enough to affect the structural strength of the member.

As a remedy, all affected areas on the rib sections are to be ground smooth and painted with a proper waterproofing paint to protect these areas from moisture deterioration.

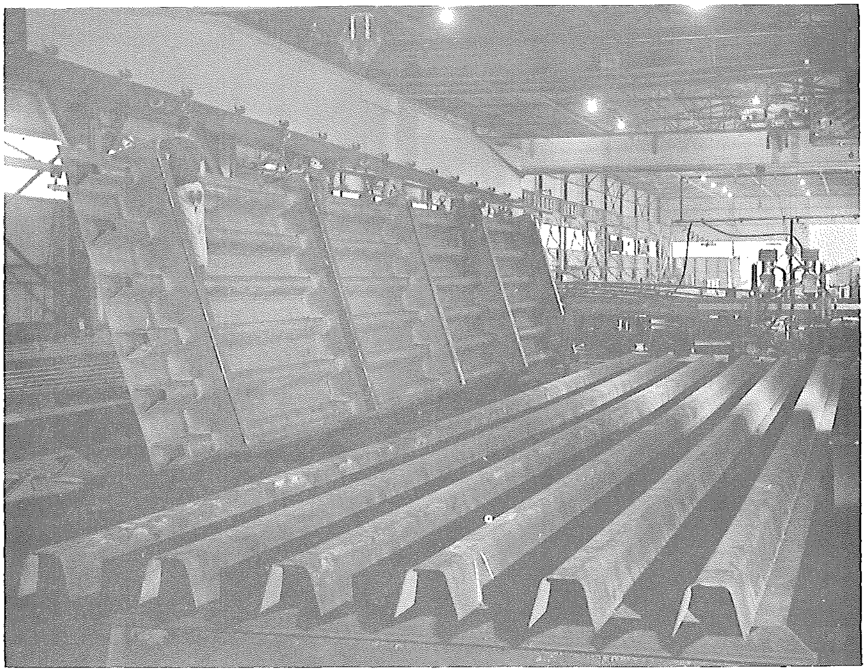
A complete report of the condition and the corrective procedure to follow will be submitted by the metallurgist. A copy will be attached to this report.

John E. Risch

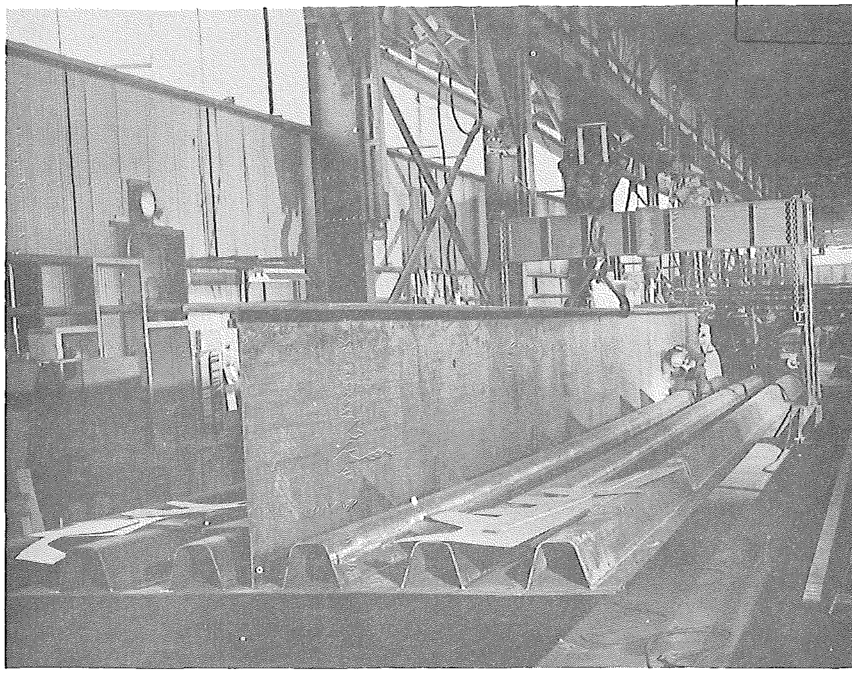
John E. Risch, Chairman
Design Division



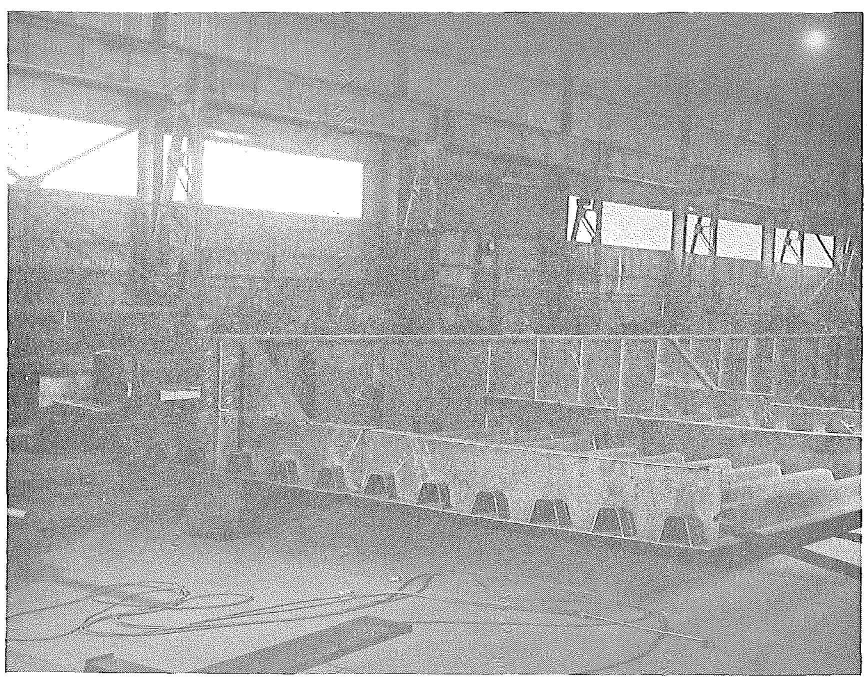
Submerged arc welding of the ribs to the deck plate.



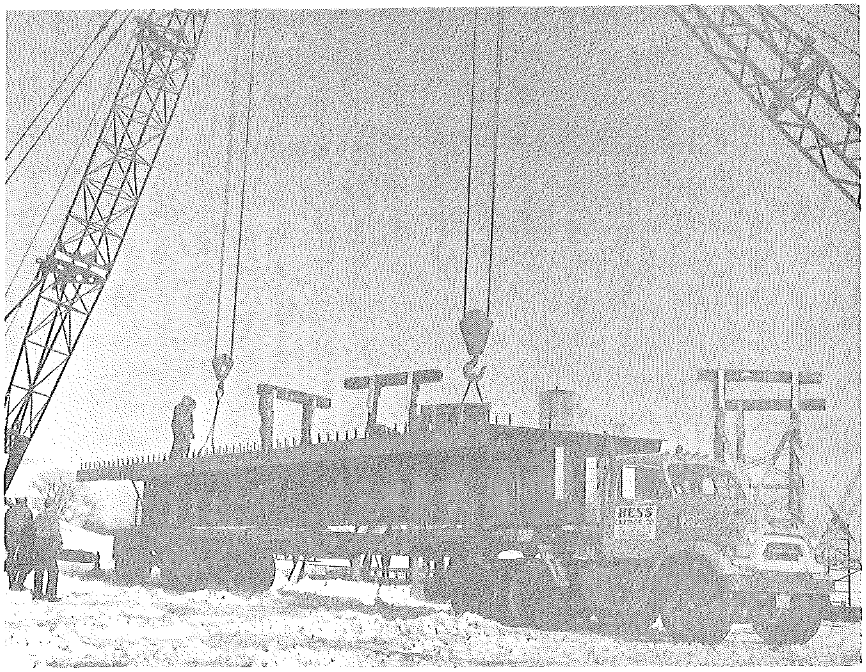
Positioning a deck section for welding the floor beams to the ribs.



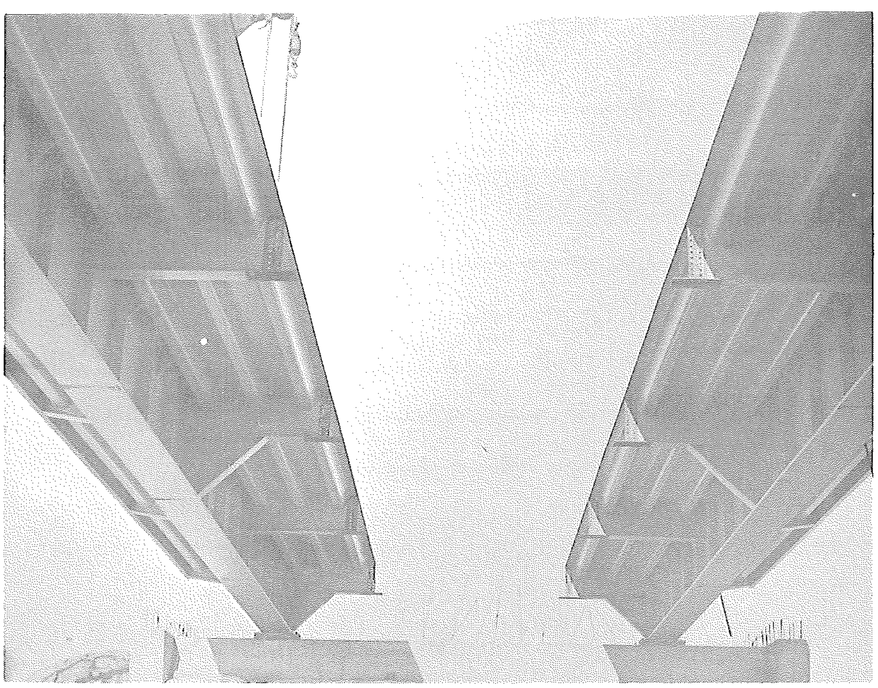
Tack welding the girder to the deck plate. Note the press holding the pieces together and the floor beam patterns in foreground.



Outer and center deck section being matched for repair and later joining in the field.

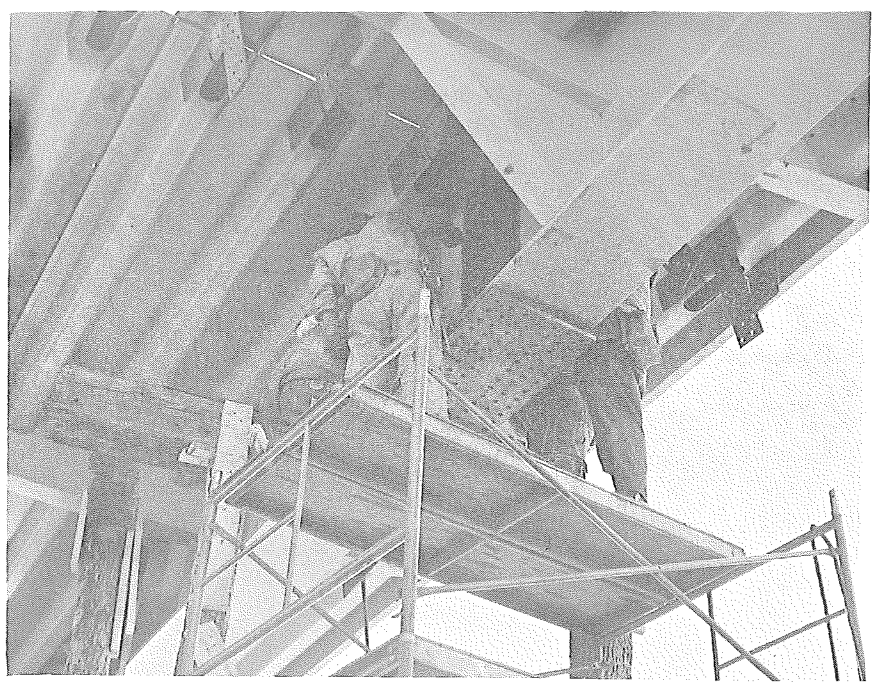


West-Center deck section being lifted for placement on the pier. This was the first deck section to arrive at the bridge site.



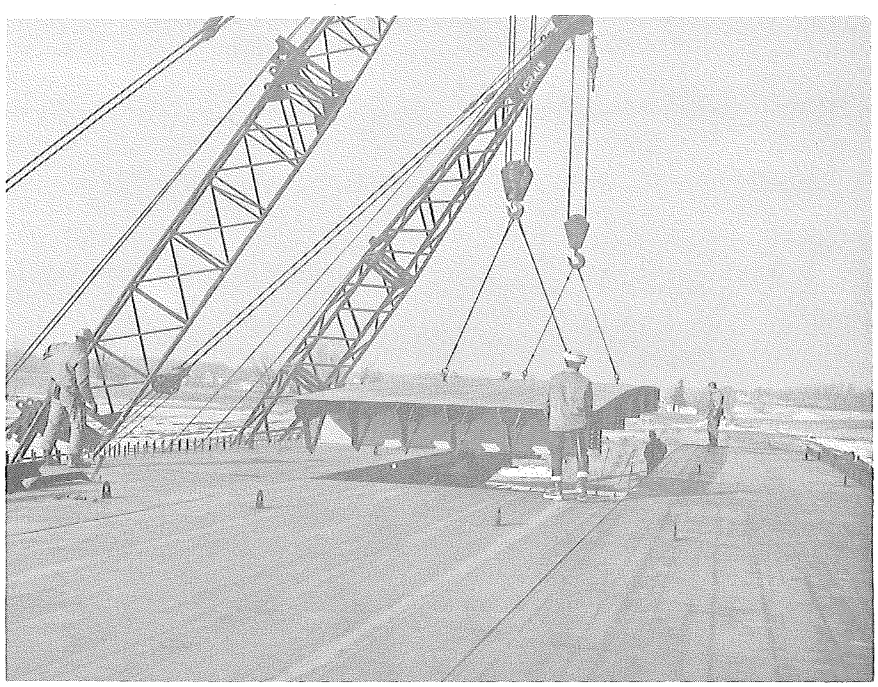
View of the north fascia sections with the splice plates set to receive the center deck section.

General view of five deck section in place, looking to the north.



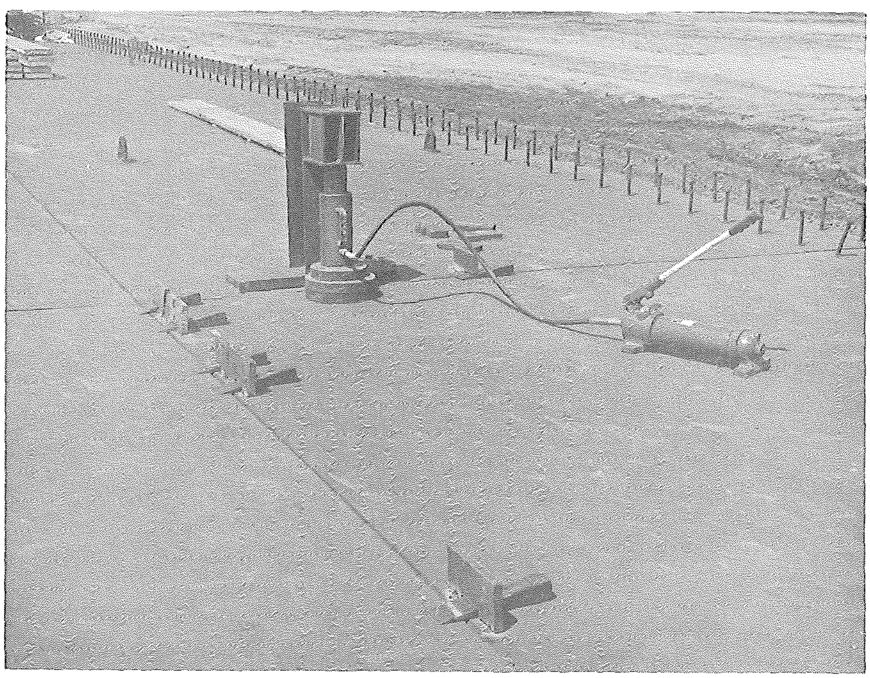
Bolting the field connection in the web of the west girder section of the north span.

View of bolted connections in the stiffening ribs and floor beams of south span.

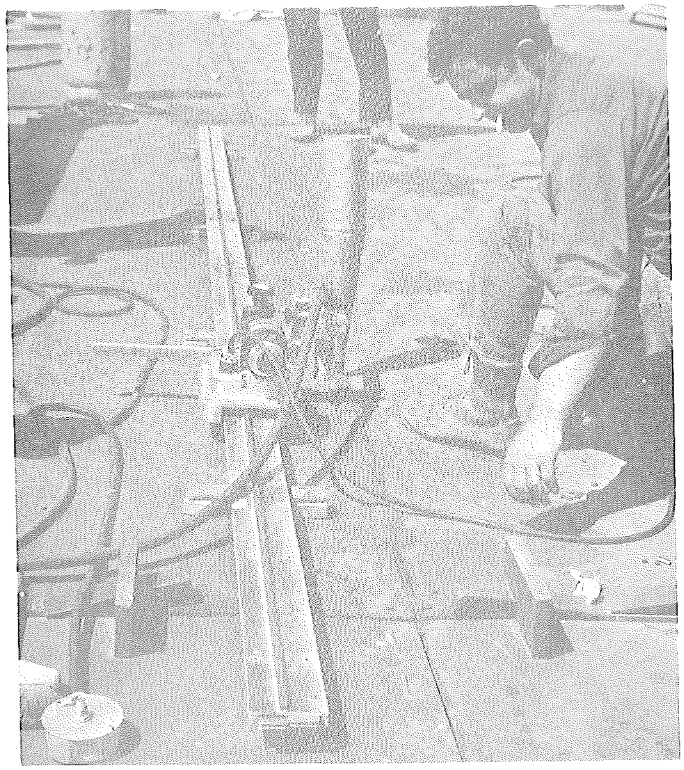


View of positioning the interior deck section in the north span. This was the last deck section to be placed.

General view of the structure with all nine deck sections field bolted.

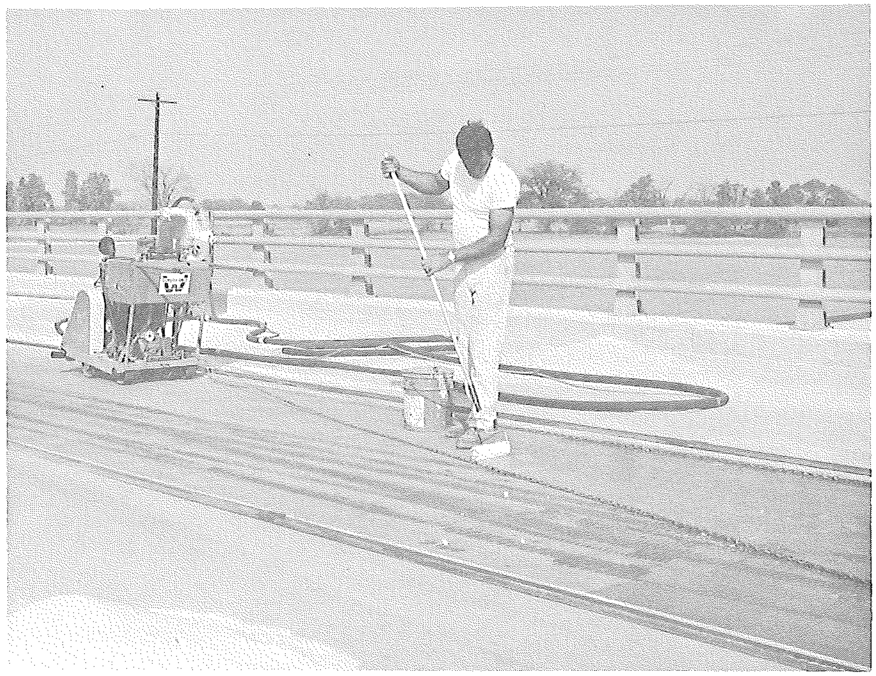


View of hydraulic jack and slot-widge assemblies used to hold down high spots in the deck plate prior to welding.

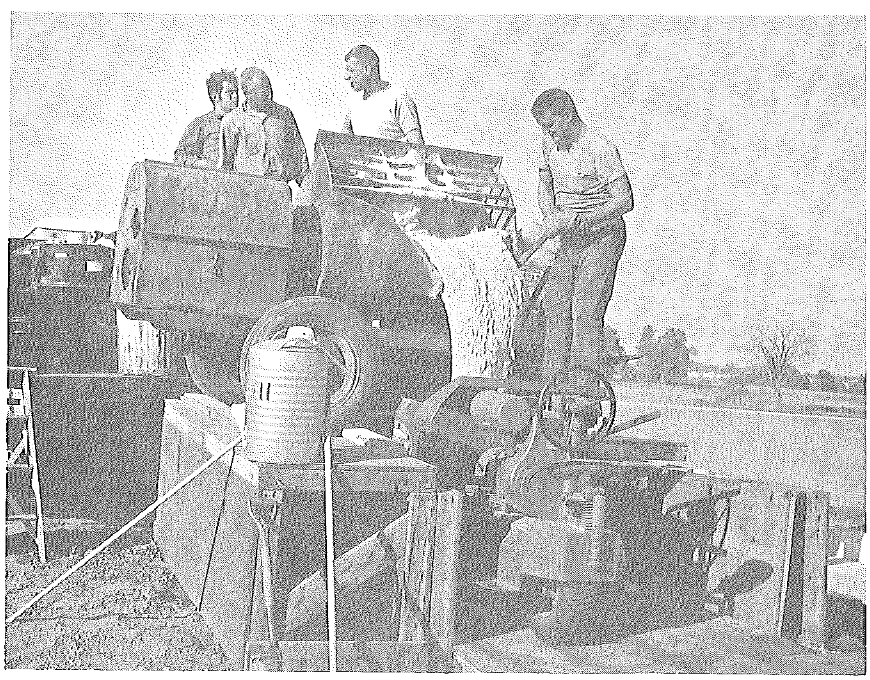


View of the continuous submerged arc welder as it moved along a longitudinal joint. The welder was completely automatic and three passes were made to complete each joint.

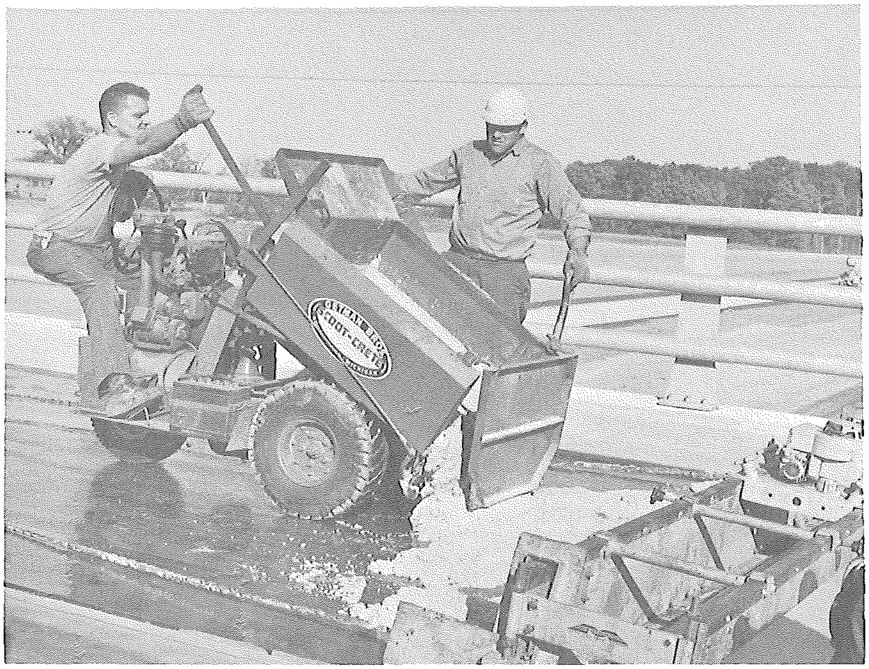
Sandblasting the steel deck plate in preparation for the epoxy primer and mortar.



Priming the sandblasted steel deck plate prior to applying the mortar mix.



Dumping a fresh epoxy mortar batch into the scoot-crete buggy.

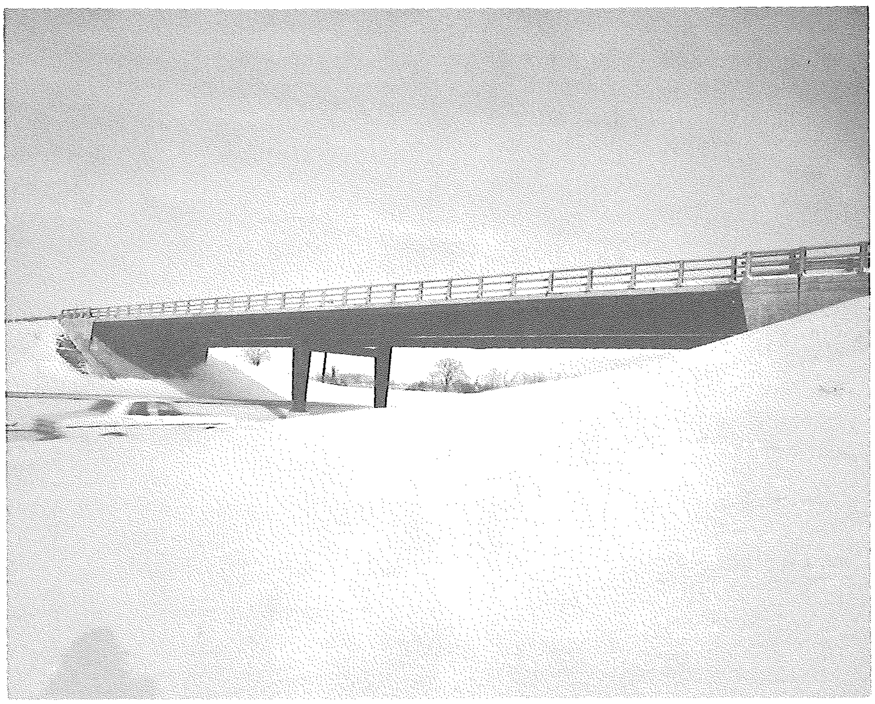


Dumping fresh epoxy mortar ahead of the screed machine.

Distributing the epoxy mortar ahead of the screed.

Rough texture of fresh epoxy mortar immediately behind the screed machine.

Broadcasting silica sand on top of the fresh mortar for maximum skid resistance.



Finished Structure

STRAIN AND DEFLECTION MEASUREMENTS
ON AN EXPERIMENTAL ORTHOTROPIC BRIDGE

C. J. Arnold

Research Laboratory Section
Testing and Research Division
Research Project 67 G-157
Research Report No. R-767

Lansing, May 1971

APPENDIX A

This report covers results of an investigation concerning strain and deflection in an experimental orthotropic bridge (Crietz Rd over I 496 west of Lansing). The bridge is a two-span continuous structure with equal spans and a total length of 187-1/2 ft. All measurements reported herein were made in the north span.

This study represents one phase of the post-construction evaluation and performance characteristics of this structure as proposed by a Joint Departmental Committee following the experimental features of this particular bridge. The purpose of this experiment was to obtain strain and deflection data for evaluation and comparison with the theoretical design analysis.

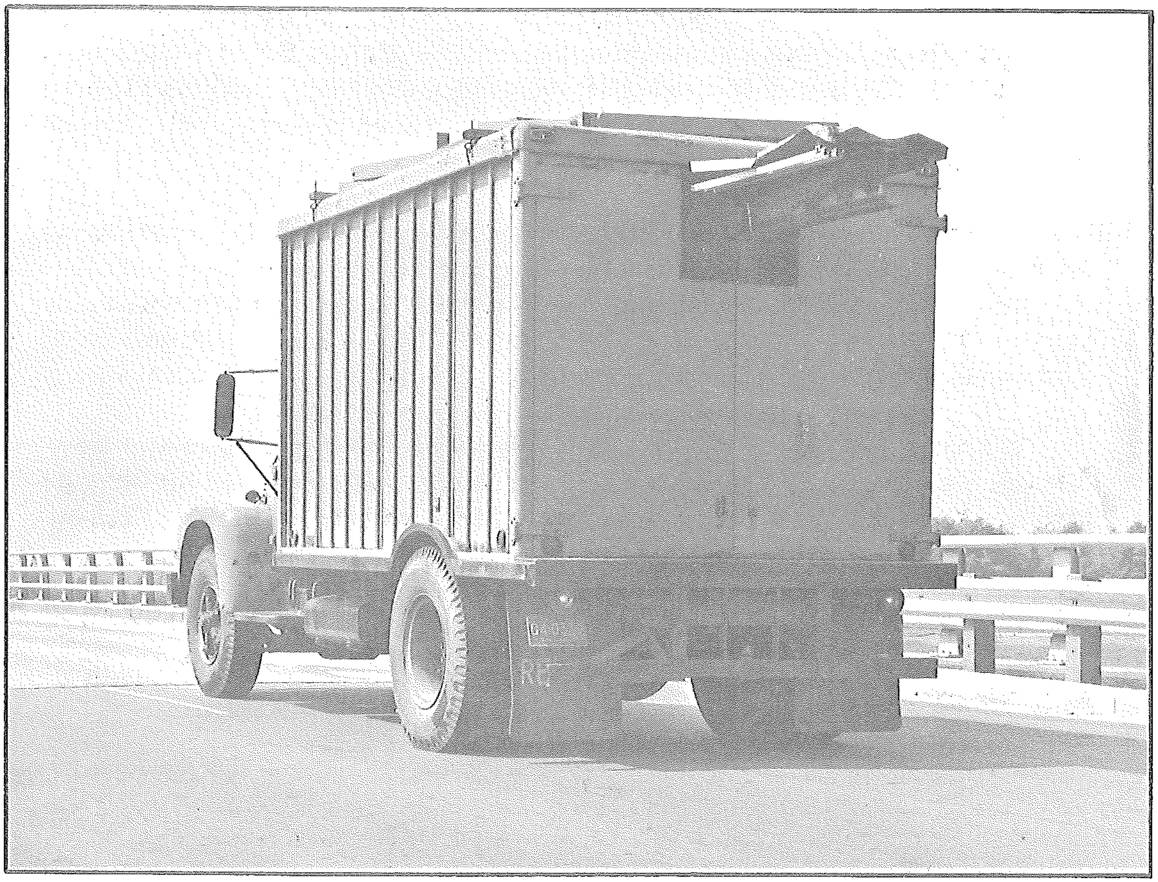
The computed values of strain and deflection shown in this report were furnished by the Design Division. Main girders were designed for H20-44 loading, but computed strains and deflections were based upon a H15-44 vehicle which was approximated in the tests. Strains and relative deflections for floor beams, ribs, and deck plate were based on a pair of 12,000-lb dual wheel loads spaced 6 ft center-to-center. The design analysis was based on the AISC Design Manual for Orthotropic Steel Plate Deck Bridges, with minor variations.

Equipment

Two MDSH test vehicles were used for the experiment and are shown in Figures 1 and 2. The two-axle vehicle approximates the load distribution of a theoretical H15-44 design vehicle, while the semi-trailer provides a trailer axle load far removed from the influence of the weight of the tractor, simulating a pair of isolated dual wheel loads.

Strain gages were applied to the underside of the structure at the locations shown in Figure 3. Foil gages were used at all locations, with 90-degree rosettes applied on the deck plate. Static strain and deflection readings were made with the load vehicles at several different locations, and limited dynamic results were obtained. Dynamic deflections were measured by linearly variable differential transformers and results recorded on an oscillograph, along with dynamic strain information. Static deflections were measured by dial indicators, with 0.001-in. increments. Three separate tests were made for each location and load condition. Values reported are averages for the three trials.

Note: Static strain measurements, under the conditions involved, may be subject to considerable error when the magnitude of the measured strain is low. Reported values below 100 μ in./in. are only approximations and errors of 10 percent or more may be present at values of 150 μ in./in.



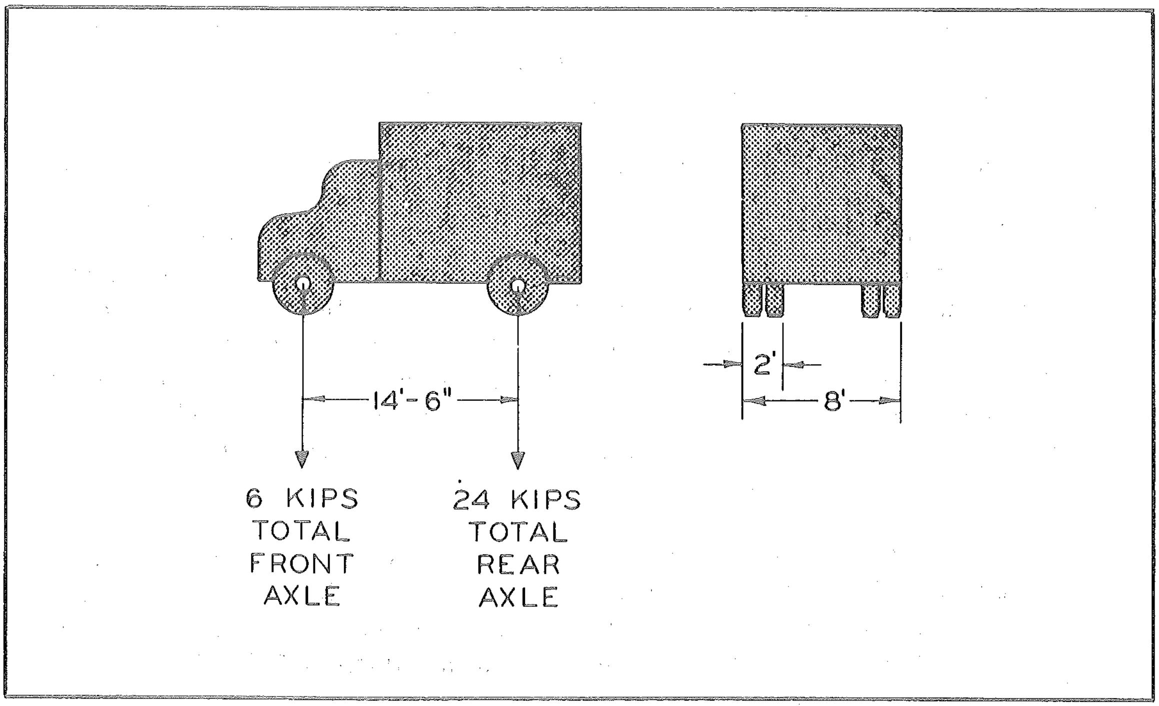
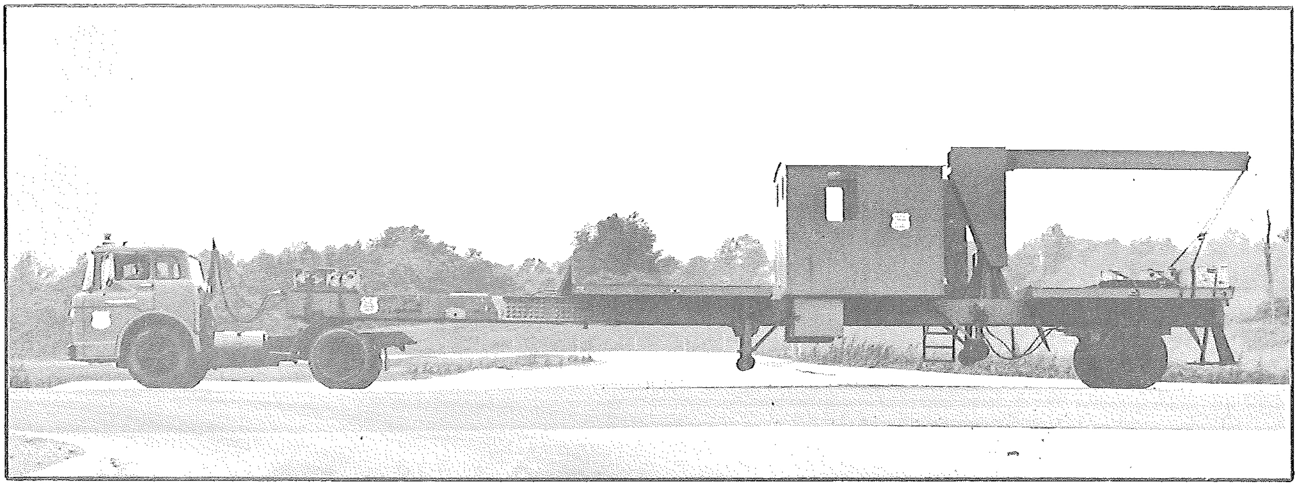
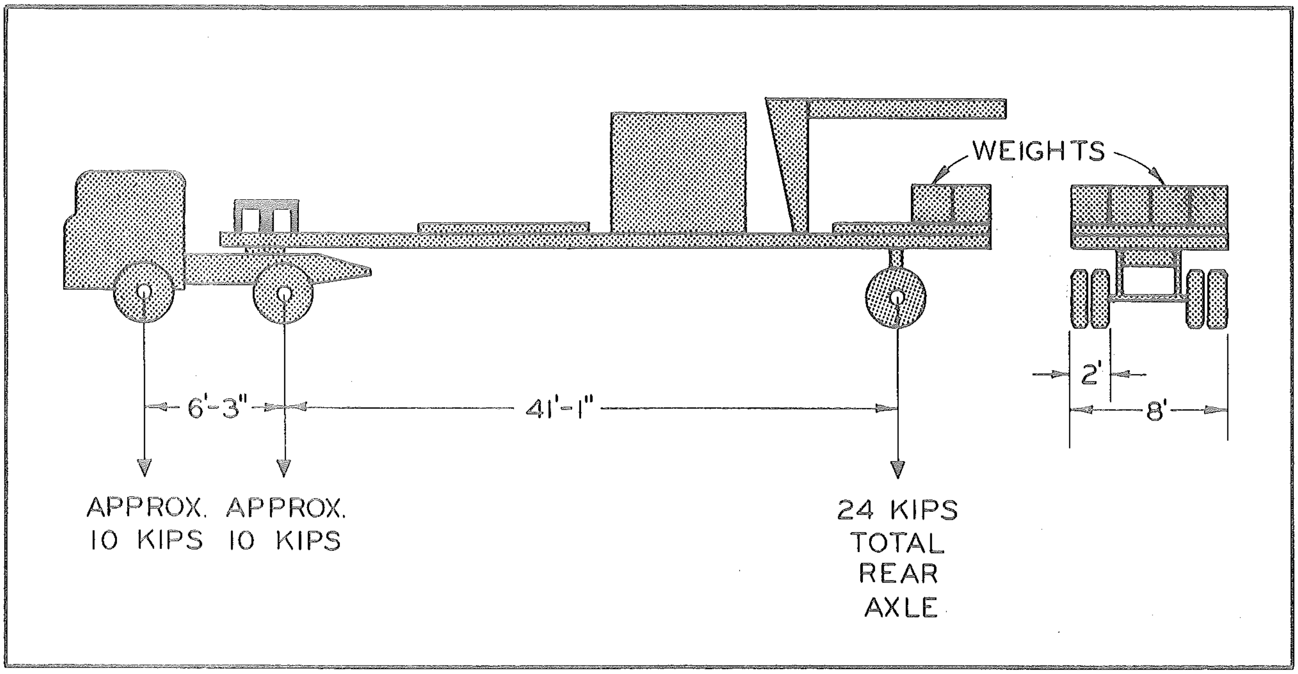


Figure 1. MDSH test vehicle -- Type 2D.





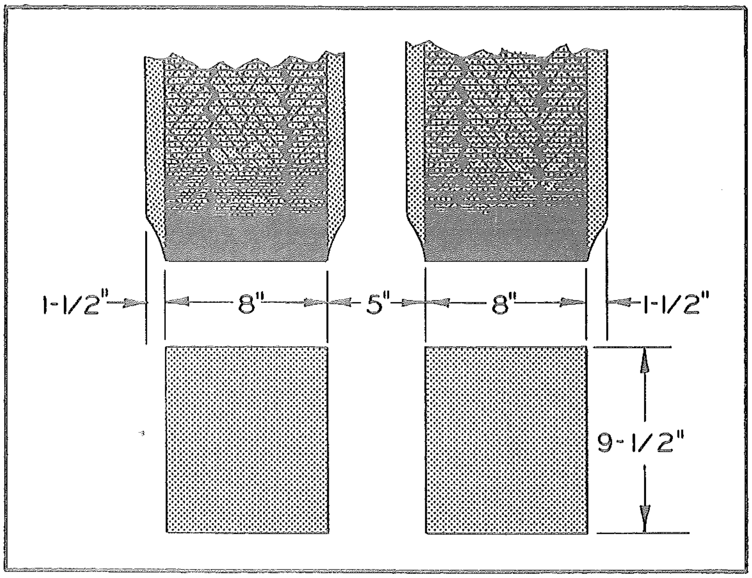


Figure 2. MDSH test vehicle -- Type 2S1 (top). Typical "footprint" for 12,000-lb load on dual tires with 80 psi inflation (right).

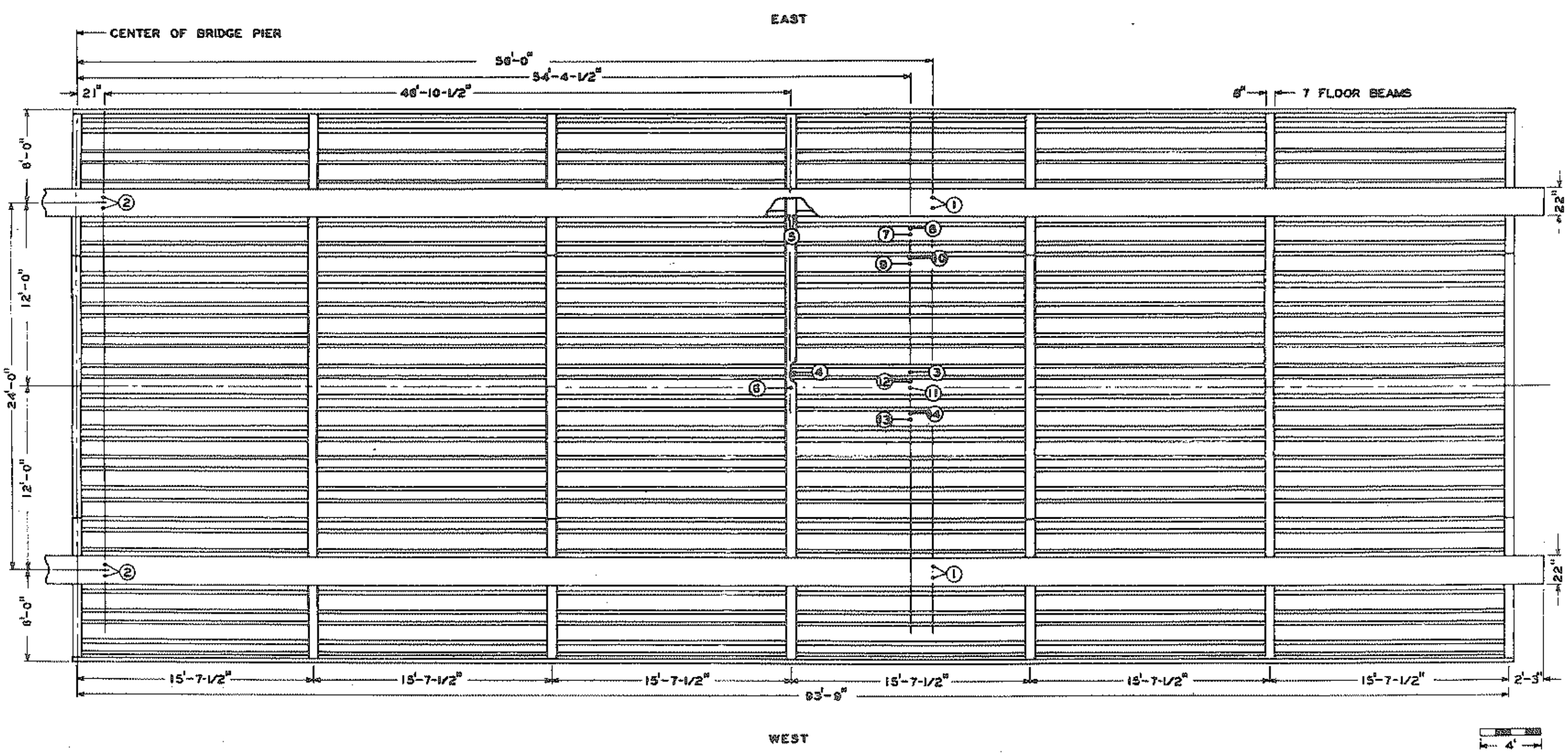


Figure 3. Bottom view of N span showing the location of strain gages.

Discussion of Test Results

1. Main Girders

Results of tests on the main girders are given in Table 1, with corresponding truck locations shown in Figure 4. Comparison of computed and measured values indicates that design assumptions are conservative, as is normally the case with other types of bridges. It was noted during testing that the bridge tends to rotate about the pier. Evidently there is sufficient friction in the system to maintain the elevation of the span slightly higher or lower, depending on which span was the last to be loaded.

Dynamic runs over the bridge at speeds of 15 and 30 mph gave peak strain and deflection values approximately 15 percent higher, due to vibration. Free vibration of the structure after the vehicle had passed was of extremely low amplitude. This is to be expected because of the high stiffness-to-mass ratio of the orthotropic design. Frequency of free vibration was roughly 4 cps.

2. Ribs

Table 2 shows the results of tests on a rib near the bridge centerline. Relative locations of the loading and gages are shown in Figure 5. Measured values are considerably lower than computed values, as was the case with the girders. Again, this indicates conservative assumptions.

The load vehicle was driven over the bridge at creep speed and at 20 mph, with the wheels passing over the instrumented rib. Indicated dynamic strains were slightly below the static values shown in the table. However, since lateral placement of the wheel load is critical in this case, and alignment is more difficult to obtain under dynamic conditions, little significance is attached to the slight difference in strain.

3. Floor Beams

Test results from floor beam tests are shown in Table 3, for loading conditions indicated in Figure 6. Here, again, measured values are considerably lower than predicted.

Dynamic runs at creep speed and at 20 mph indicated less than 10 percent increase in strain due to vibration.

TABLE 1

STATIC STRAIN AND DEFLECTION FOR MAIN GIRDERS (Load Vehicle Type 2-D)

Run No.	Truck Location	Gages Monitored	Strain (μ in./in.)				Deflection (in.)			
			E. Girder		W. Girder		E. Girder		W. Girder	
			Computed	Measured	Computed	Measured	Computed	Measured	Computed	Measured
1	1	1E & 1W	110	70	---	0	0.31	0.21	----	0.04
2	2	1E & 1W	---	30	---	30	----	0.12	----	0.12
3	3	1E & 1W	---	0	110	70	----	0.04	0.31	0.21
4	4	2E & 2W	-50	-40	---	0	----	----	----	----
5	5	2E & 2W	---	-20	---	-20	----	----	----	----
6	6	2E & 2W	---	0	-50	-40	----	----	----	----
7	7	2E & 2W	-50	-40	---	0	----	----	----	----
8	8	2E & 2W	---	-20	---	-20	----	----	----	----
9	9	2E & 2W	---	0	-50	-40	----	----	----	----

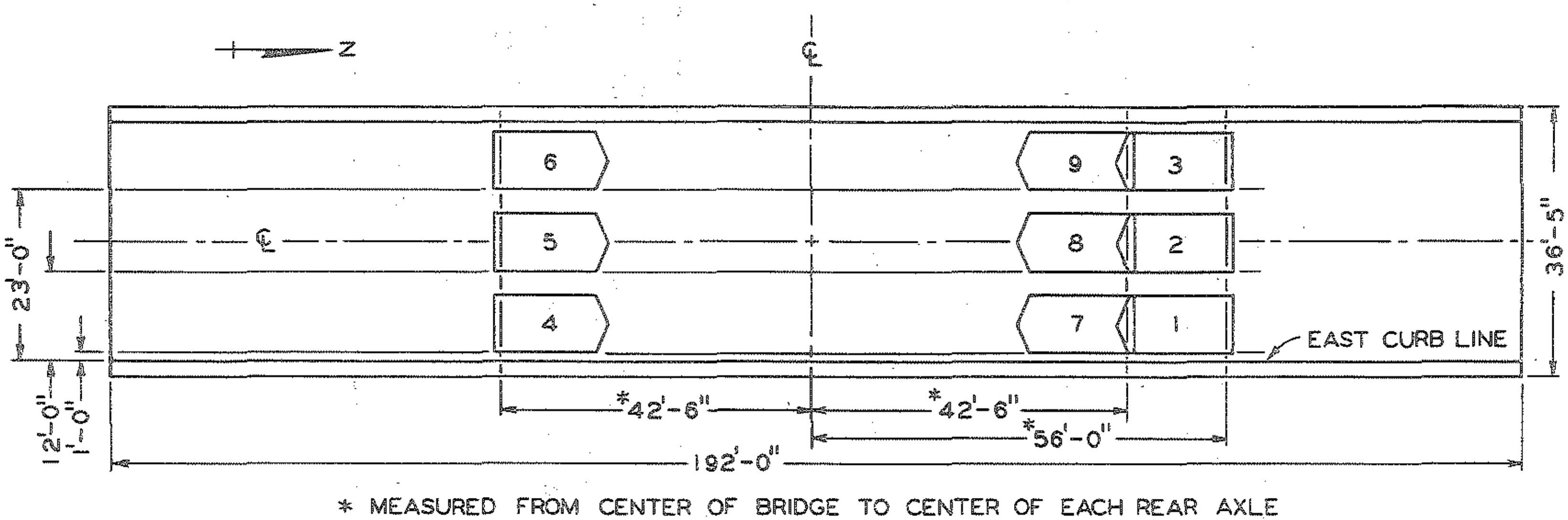


Figure 4. Positions of Type 2D load vehicle for testing main girders.

TABLE 2

STATIC STRAIN AND DEFLECTION FOR RIB (Load Vehicle Type 2S1)

(Tabulated deflection value is the deflection of the rib relative to the two adjacent floor beams)

Run No.	Truck Location	Gage Monitored	Strain (μ in./in.)		Relative Deflection (in.)	
			Computed	Measured	Computed	Measured
1	10	3	375	270	0.12	0.08
2	11	4	-160	-100	----	----
3	12	4	---	- 90	----	----

TABLE 3

STATIC STRAIN AND DEFLECTION FOR FLOOR BEAM (Load Vehicle Type 2S1)

(Deflection value given is the deflection of floor beam relative to the girders)

Run No.	Truck Location	Gages Monitored	Strain (μ in./in.)			Deflection (in.)	
			Gage 5	Gage 6		Computed @ Gage 6	Measured @ Gage 6
				Computed	Measured		
1	13	5 & 6	0	255	140	0.09	0.06
2	14	5 & 6	0	255	140	0.09	0.06

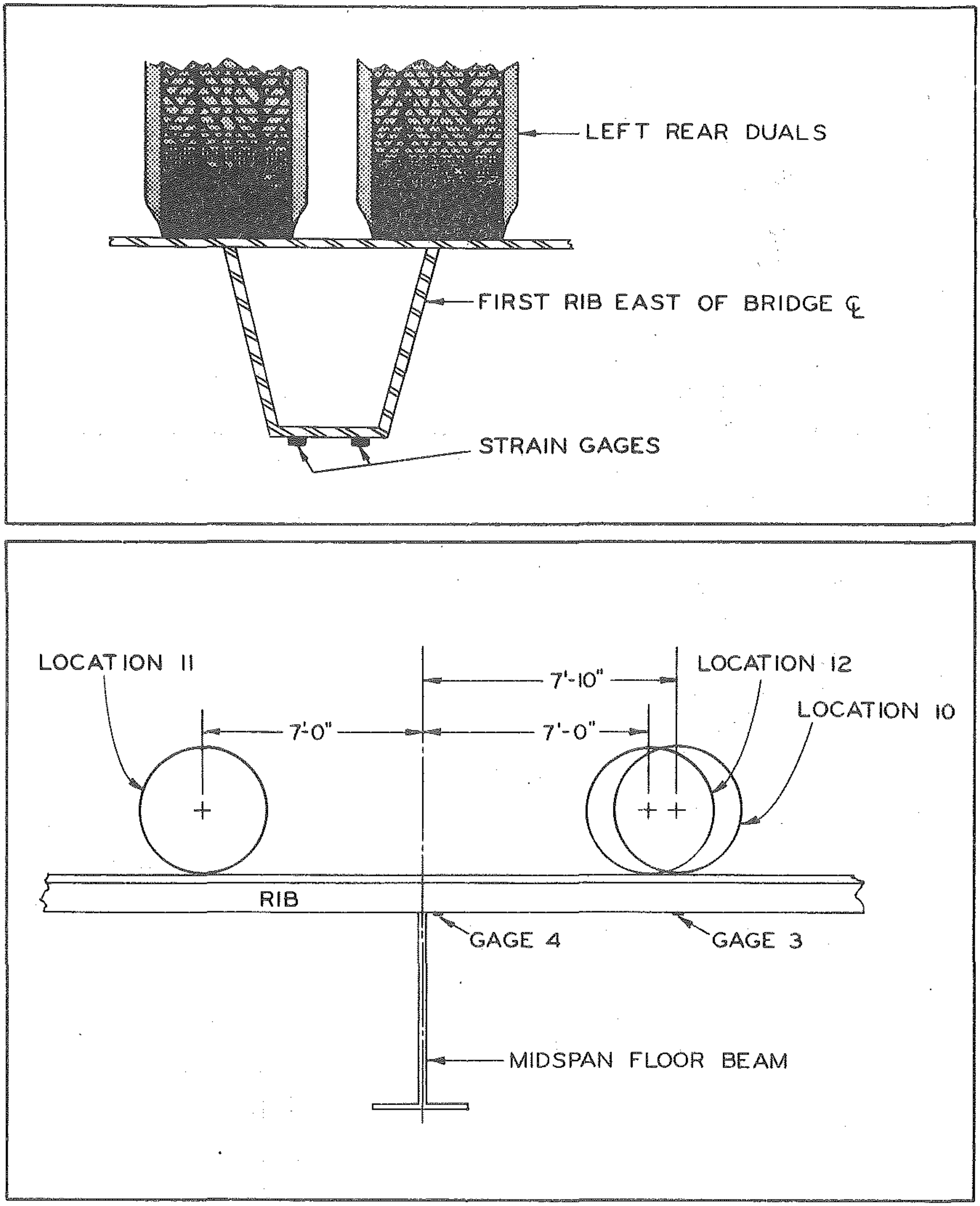
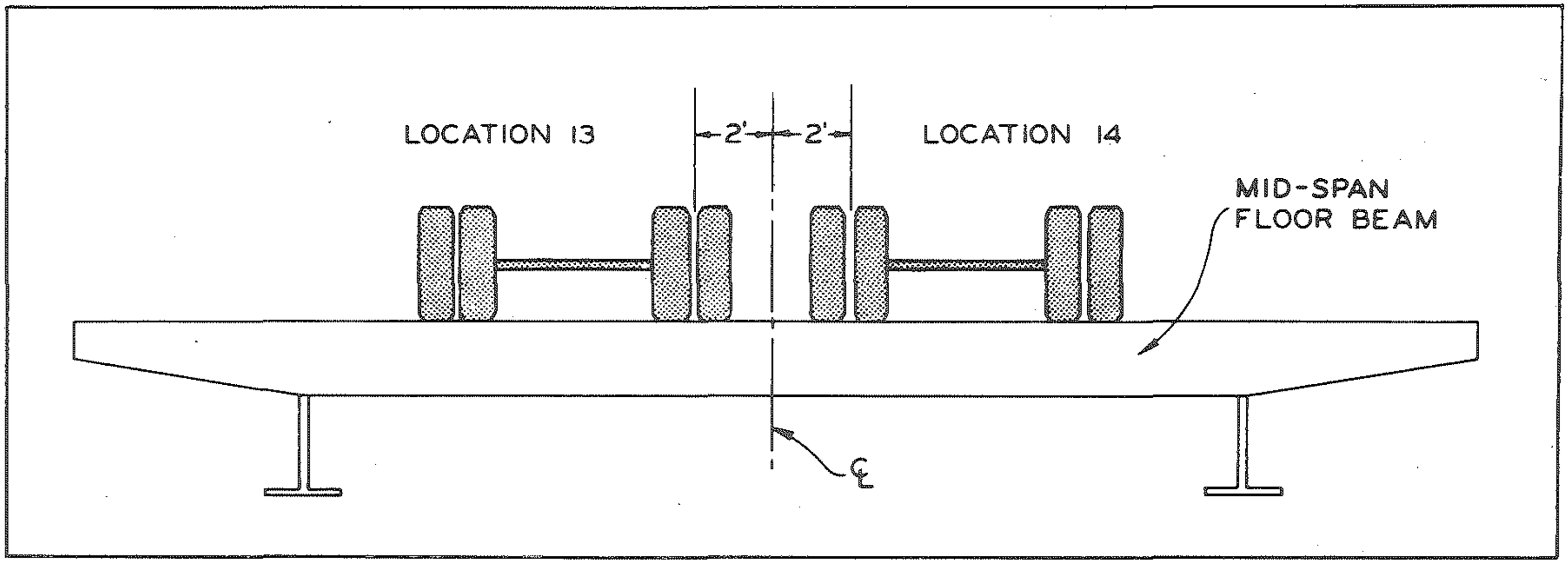


Figure 5. Loading positions for rib tests. Load vehicle Type 2S1 with 12,000-lb wheel load positioned as shown.



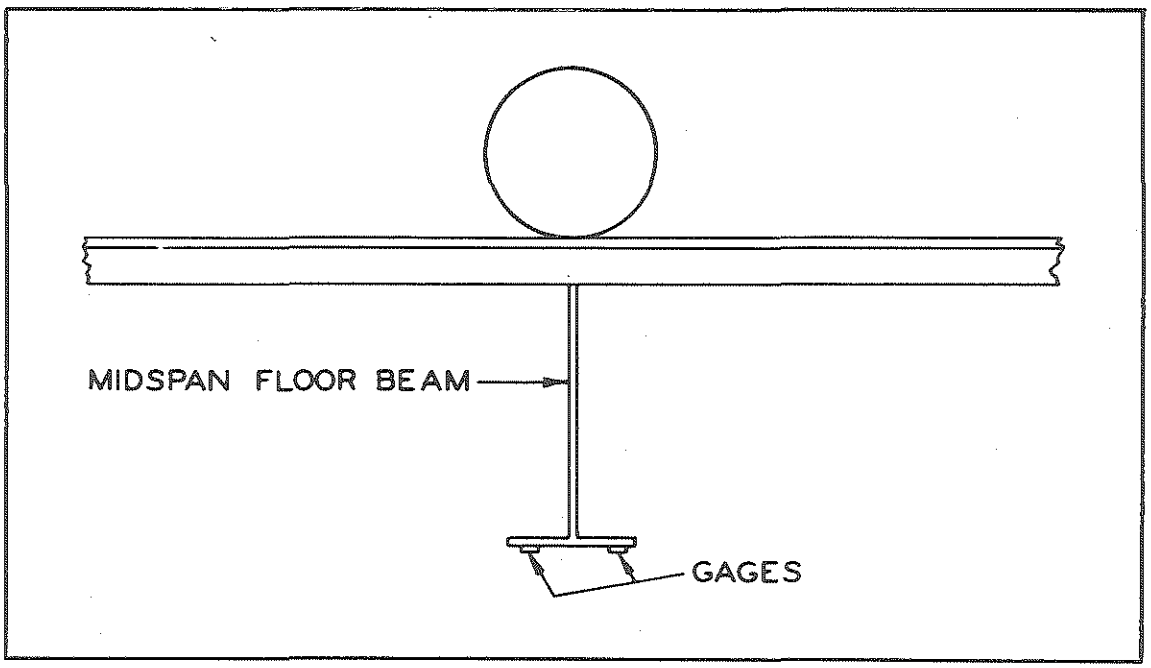


Figure 6. Loading positions for floor beam tests. Load vehicle Type 2S1 with 24,000-lb axle load positioned as shown.

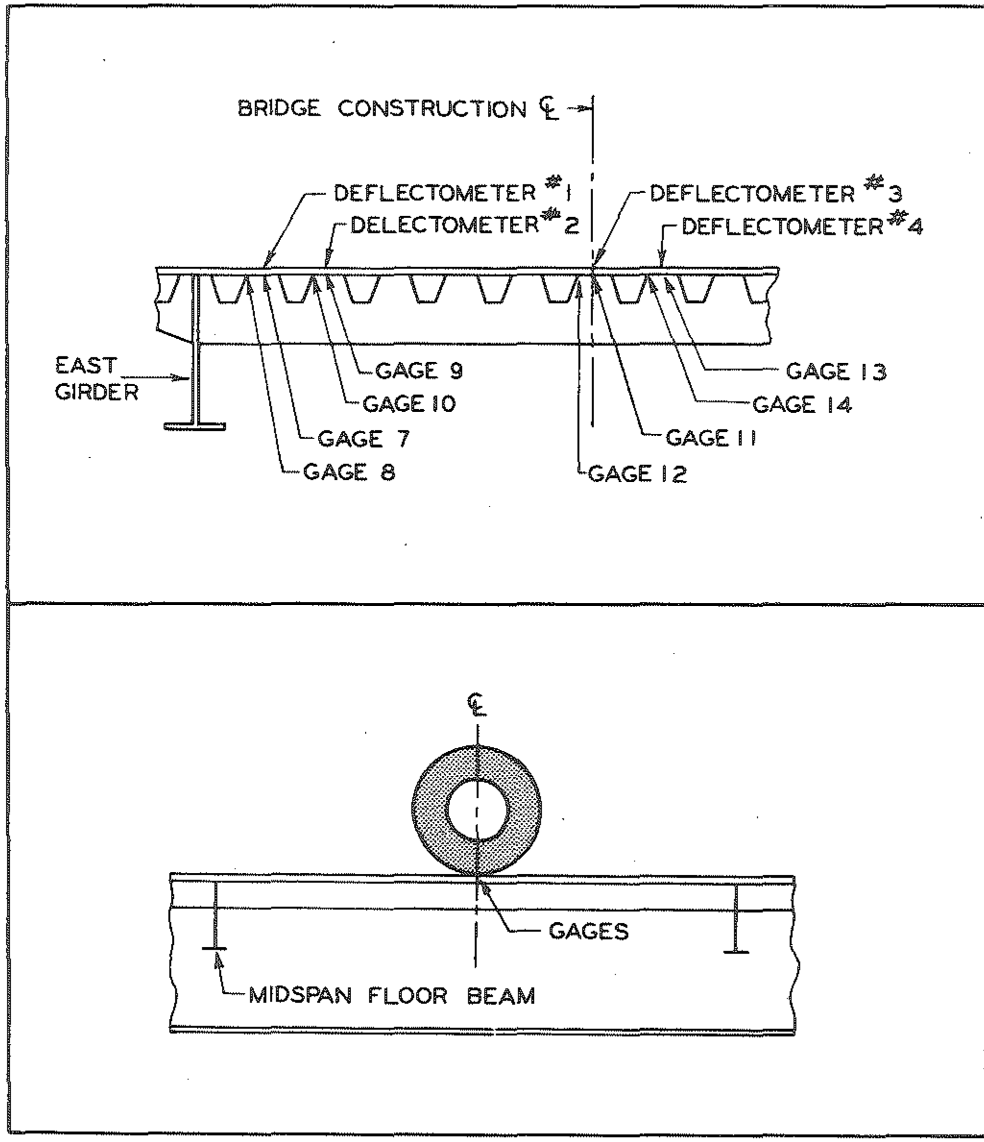
4. Deck Plate

The deck plate proved to be very difficult to test because hot weather occurred during the time the tests were conducted. It was found that the floor beams provided effective restraint to lateral expansion of the deck plate when the plate was heated by the sun. The result was a slight wrinkling or buckling of the plate between the ribs. When the sun was shining brightly, lateral pressures in the plate sometimes were sufficient to hold the plate in a slightly deflected position after removal of the load. It was also noted that the shadow of the load vehicle caused changes in strain and deflection of the plate in the area, evidently due to localized cooling of the heated plate. Temperature changes in the deck occurred quite rapidly because of high thermal conductivity and low mass. Therefore, the plate tests had to be rerun several times, and still were not considered to be entirely satisfactory.

Strain and deflection data presented in this section are believed to be reasonable approximations but should not be taken as absolute, because of the problems noted above.

Gages mounted on the plate were rosettes, with arm No. 1 measuring plate strain in the transverse direction, arm No. 2 longitudinal, and No. 3 at 45° to the other two (bisecting the angle).

Results of the deck plate tests are shown in Table 4 for load positions indicated in Figure 7. The design assumption for plate loading considers the load to be uniformly distributed over a rectangular area approximating the imprint area of the dual tires. Calculations based on this assumption indicated maximum transverse strains of nearly 700 micro-inches per inch, and relative deflections of more than 0.060 in. Test results indicate maximum transverse strains of about 100 micro-inches per inch, with deflections less than 0.010 in. The sketches in Figure 7 show approximately how the load is applied by the tires and it can be seen that the actual load distribution differs from the assumed distribution. Therefore, strains could be expected to be lower than predicted. However, it was noted during testing that the single tire of the steering axle of the truck caused as much or slightly more deflection of the plate than the 12,000-lb dual wheel, even though the steering axle weight was only 10,000 lb. It can be seen from the sketches that the single tire load distribution fits readily within the unsupported span of the plate between the rib side plates, yielding a more critical loading case. Also, since there is no other load nearby to hold down the adjacent plate, the plate can rotate more readily about the ribs when the single-tired load is applied. The end result is more deflection at lower applied load.



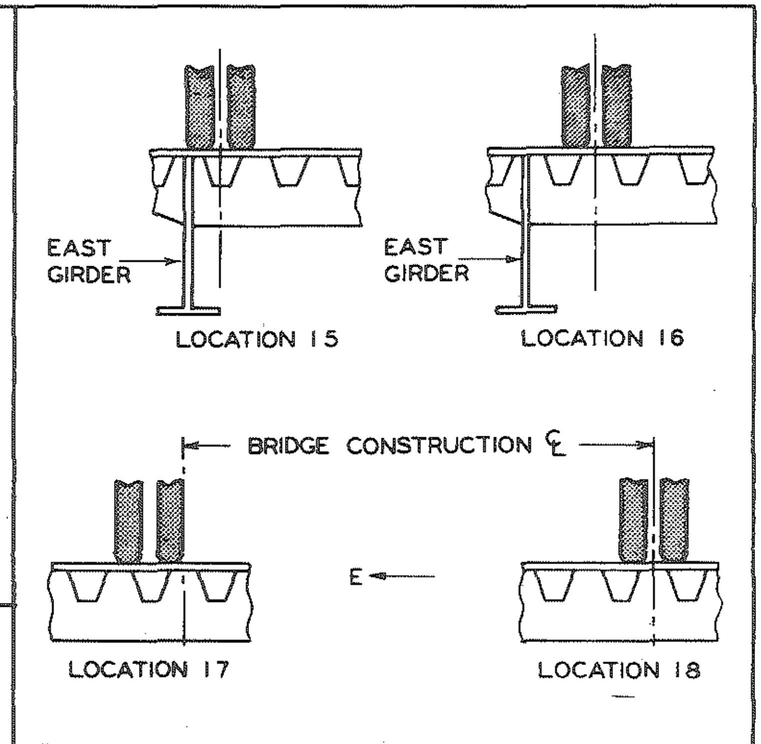


Figure 7. Loading positions for deck plate tests. Load vehicle Type 2S1 with 12,000-lb dual tire load placed as shown.

TABLE 4

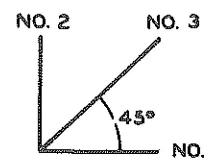
STATIC STRAIN AND DEFLECTION FOR DECK PLATE

(Positive deflection is downward)

Truck Location	Deflection, in.		Gage Location											
			7			8			9			10		
			Arm			Arm			Arm			Arm		
	D_1	D_3	1	2	3	1	2	3	1	2	3	1	2	3
15	0.001	-0.002	20	-60	-40	-40	-60	-80	20	-40	-20	-20	-40	-40
16	0.009	-0.001	120	20	80	0	-40	-80	0	-40	-20	0	-40	-20

Truck Location	Deflection, in.		Gage Location											
			11			12			13			14		
			Arm			Arm			Arm			Arm		
	D_3	D_4	1	2	3	1	2	3	1	2	3	1	2	3
17	0.002	-0.002	0	-60	-40	40	-60	-20	0	-40	-20	0	-40	-20
18	0.008	-0.001	100	0	60	-40	-40	-40	0	-40	-20	40	-40	0

NOTE: Locations 7 through 14 each have 3-gage rosettes. Arm No. 1 is transverse to bridge ℄ , No. 2 parallel to the ℄ , and No. 3 at 45°.



Gages 7 through 10 were intended for comparing deck plate strains near the girder where the rib support conditions are quite stiff; with gages 11 through 14 indicating deck strains near the bridge centerline where the rib support is more flexible. However, the results do not seem to be sufficiently accurate to make a valid conclusion, since the differences to be detected are quite small.

Conclusions

Results of the tests indicate that design assumptions are conservative. All measured strains and deflections were well below calculated values.

Measurements of strain and deflection on the orthotropic bridge bear similar comparisons to design values as do comparable measurements in slab-on-beam bridges of approximately the same size.

EPOXY MORTAR SURFACING PERFORMANCE EVALUATION ON AN EXPERIMENTAL ORTHOTROPIC BRIDGE

H. L. PATTERSON

Research Laboratory Section
Testing and Research Division
Research Project 67 G-157
Research Report No. R-778

MICHIGAN DEPARTMENT OF STATE HIGHWAYS
August 4, 1971

Appendix B

OFFICE MEMORANDUM

MICHIGAN
DEPARTMENT OF STATE HIGHWAYS

August 4, 1971

To: L. T. Oehler
Engineer of Research

From: H. L. Patterson

Subject: Epoxy Mortar Surfacing on the Crietz Rd Bridge over I 496, Lansing (S05 of 23081A): A Report on the First and Second Annual Field Inspections. Research Project 67 G-157. Research Report No. R-778.

INTRODUCTION

This report is part of the performance evaluation outlined by G. R. Cudney in a letter of December 8, 1967 to John Risch. It is primarily concerned with evaluating the epoxy mortar wearing surface on the subject bridge. The bridge was erected on an experimental basis to determine whether the orthotropic design and epoxy wearing surface were practical in this climate. The structure is described in a Design Division report compiled by John Risch ("Final Report of Experimental Orthotropic Bridge," January 1970).

The steel deck and concrete curbs of the bridge were placed in the winter and summer of 1969, respectively, and the epoxy mortar surfacing was applied in September and October of that year. The deck was divided such that the south half and north half contained different epoxy mortars applied at a minimum thickness of 5/8 in. On the south half, the epoxy binder used was "Guardkote 250," an oil-modified epoxy produced by the Shell Oil Co. On the north half, the epoxy binder consisted of compatible components from two sources: "Epon 815," a modified epoxy resin from Shell Oil Co. and "Versamid 140," a modified polyamide curing agent from the Chemical Division of General Mills and carrying their designation "Genepoxy E15." To the best of our knowledge, this application of the latter epoxy was its first use on Michigan highways.

BRIDGE DECK INSPECTION TECHNIQUES

A system of surveillance was established to detect any faults or failures which might occur in the epoxy mortar wearing course. It consisted of a semi-annual series of oblique photographs taken longitudinally down both sides of the bridge, accompanied annually by a closely detailed inspection. All deterioration features were photographed and recorded on a scale drawing of the deck plan. To aid in cross-referencing between the oblique photographs and the scale drawing, the curbs were measured and marked off in 10-ft intervals from north to south. Six-inch stenciled numbers denoting the distance were painted near the marks such that they would be visible in the photographs.

The oblique photographs were taken from an elevation of 10 ft and showed a 35-ft longitudinal distance down the deck; each subsequent photograph in the series overlapped its predecessor by 10 ft to aid in continuity. All photographs taken during the detailed inspection consisted of two pictures of each feature; a general area picture to show the relative location, and a close-up to show the detail.

To supplement the photographs and mapping of deterioration features, tests were run in the spring and fall of each year with the Department's skid testing vehicle. As an additional check on the slipperiness of the deck, weekly inspections were performed during the first operational winter to determine the deck surface icing characteristics. Table 1 gives a summary of the skid tests. Additional discussion of these results are presented at the end of the report.

OBSERVATIONS OF DECK PERFORMANCE

Figure 1 shows the overall view of the deck looking south at the time the initial set of oblique photographs were taken; approximately one month after the bridge was opened to traffic.

Weekly inspections of icing conditions on the deck throughout the winter revealed that this deck was typical of most, in that it became very slippery if snow were allowed to accumulate; however, liberal amounts of deicing salts had been applied in all cases of significant snow accumulation and the deck surface afforded good traction. Icing, resulting from overnight frost conditions, did not seem to produce a slippery surface on the deck.

Inspection After the First Winter

In the latter part of April 1970, the deck was closely inspected and all types of deterioration were shown and noted on a scaled plan drawing. Figure 2 is a diagram of the deck which shows the signs of deterioration found at the time of this inspection.

On the west strip of the north half, where some screeding difficulties were encountered with cold E15-V140 epoxy mortar, several badly pitted areas were observed. This problem was later solved by heating the epoxy. We believe that an excessive amount of air had been trapped during mixing by the cold, higher viscosity binder in these areas. Traffic soon removed the thin cover over the voids, and in their exposed condition they resembled pitting instead of air bubbles.

On the south half of the bridge, the weather was warmer when the Guardkote 250 mortar was placed, and this binder had a lower viscosity so excessive trapped air did not become a problem. However, it appeared that the mortar wearing surface was developing a pattern of craze cracking. Reference to Figure 2 shows that the present pattern was not random but seemed to be developing under the outside wheel track of both the northbound and southbound traffic lanes. Many of these craze crack areas appeared to coincide with live-load negative moment areas over the transverse floor beams.

Inspection After the Second Winter

In mid-May 1971, the deck was again closely inspected and all types of deterioration were noted and recorded as before. Figure 3 is a diagram of the deck which shows the deterioration features found at the time of this inspection.

On the north half of the bridge, where the E15-V140 epoxy mortar was placed, the condition of the mortar surface had changed very little. Some light traffic abrasion had occurred and the west side "pitted" area, located about 35 ft from the north abutment, had enlarged about 50 percent. This pitting, as previously explained, is merely the exposure of large (1/16 to 1/8 in.) air bubbles that were trapped by the cold viscous epoxy binder. Although this is a distinct feature, it appears to be superficial (Fig. 4).

On the south half of the bridge, where the Guardkote 250 mortar was placed, the condition of the mortar surface has continued to deteriorate. Many of the areas that were observed to have obscure craze cracking in 1970 have now become distinct, thus expanding the total area now exhibiting craze cracking. On the east side of the bridge around the 170-ft mark, and on the west side near the 160-ft mark, clusters of craze cracked spots have developed. At first it appeared that this cracking had gone full-depth through the epoxy mortar, because dark "rusty" spots were apparent; however, further investigation established these "rust streaks" to be embedded asphalt. Figure 5 is a view of these clusters around the 170-ft mark on the east side of the deck.

On the west side of the deck near the 150-ft mark, a longitudinal crack pattern is developing about 7 ft from the curb line; presently it consists of three parallel cracks which apparently are the result of flexure in the steel deck. These cracks are located directly above the east edge of the second rib from the west girder. This crack pattern is shown on the diagram in Figure 3 and a close-up view of it appears in Figure 6.

REMARKS

During the time that this bridge was being constructed, several hydraulic patching mortars were being evaluated in the Research Laboratory. A Guardkote 250 epoxy mortar was included in the series as a control material and the subsequent evaluation disclosed that it sustained excessive shrinkage. It was theorized in the report[1] that the shrinkage was caused by a volatilization of the oil with which the epoxy was extended. It would appear from the development of craze cracked areas on this bridge that the Guardkote 250 epoxy mortar surfacing is undergoing the same shrinkage as was observed in the laboratory, but it appears to be a surface feature and does not go full depth.

The parallel longitudinal cracks which were previously described, are probably the result of a combination of stresses. One of the contributing stresses, since the cracks occurred midway between two floor beams, was probably tension caused by differential flexure of the longitudinal ribs as live loads moved across the bridge. Other contributing stresses were caused by shrinkage and thermal contraction.

The light traffic abrasion shown in the diagram in Figure 3 is probably responsible for the loss of traction that is apparent in Table 1. There also appears to be a greater accumulation of road oils and dirt in the surface of the south half. It is believed that the coefficients of friction as measured on June 3, 1971 should be about the minimum that will be reached; additional tests will confirm if any further polishing should develop. Another winter's exposure may require some remedial measures for the fine cracking in the south half, but no recommendations are being made at this time.

TESTING AND RESEARCH DIVISION

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TABLE 1
DECK SURFACE SKID TEST SUMMARY
(Wet sliding tests at 40 mph)

Mortar Type and Location	Coefficient of Friction* and Dates Tested			
	12-2-69 (initial)	5-4-70	10-14-70	6-3-71
North Half (E15 - V140)				
Northbound Lane	0.67	0.52	0.57	0.41
Southbound Lane	0.66	0.53	0.55	0.38
Average	0.67	0.53	0.56	0.40
South Half (Guardkote 250)				
Northbound Lane	0.75	0.48	0.56	0.40
Southbound Lane	0.69	0.46	0.51	0.31
Average	0.72	0.47	0.54	0.36

* Each test value is the average of 3 individual tests in each direction.

(1) "Evaluation of Five Commercial Fast-Setting Hydraulic Patching Mortars," MDSH Research Report No. R-715, October 1969.

Figure 1. View looking south over Crietz Rd Bridge, December 1969.

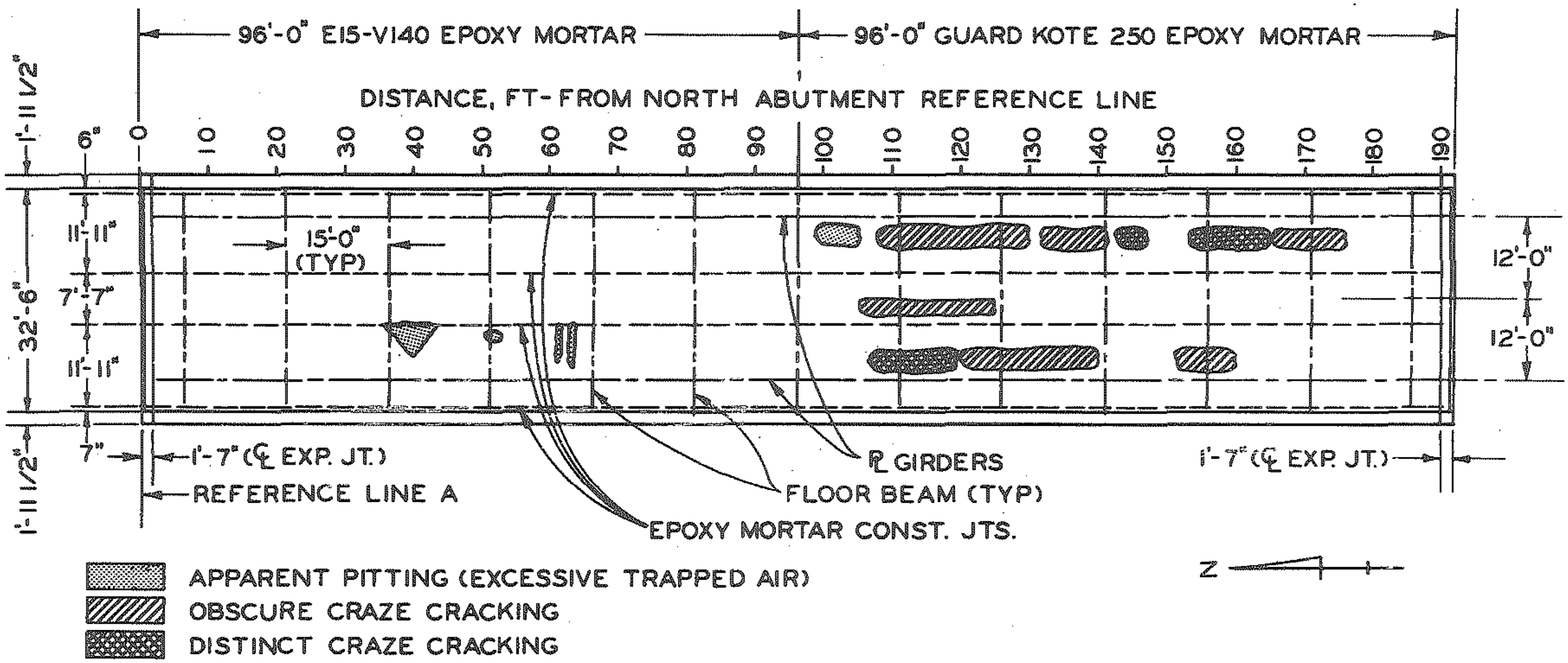


Figure 2. Deterioration features in the epoxy mortar wearing surface as observed on April 29, 1970.

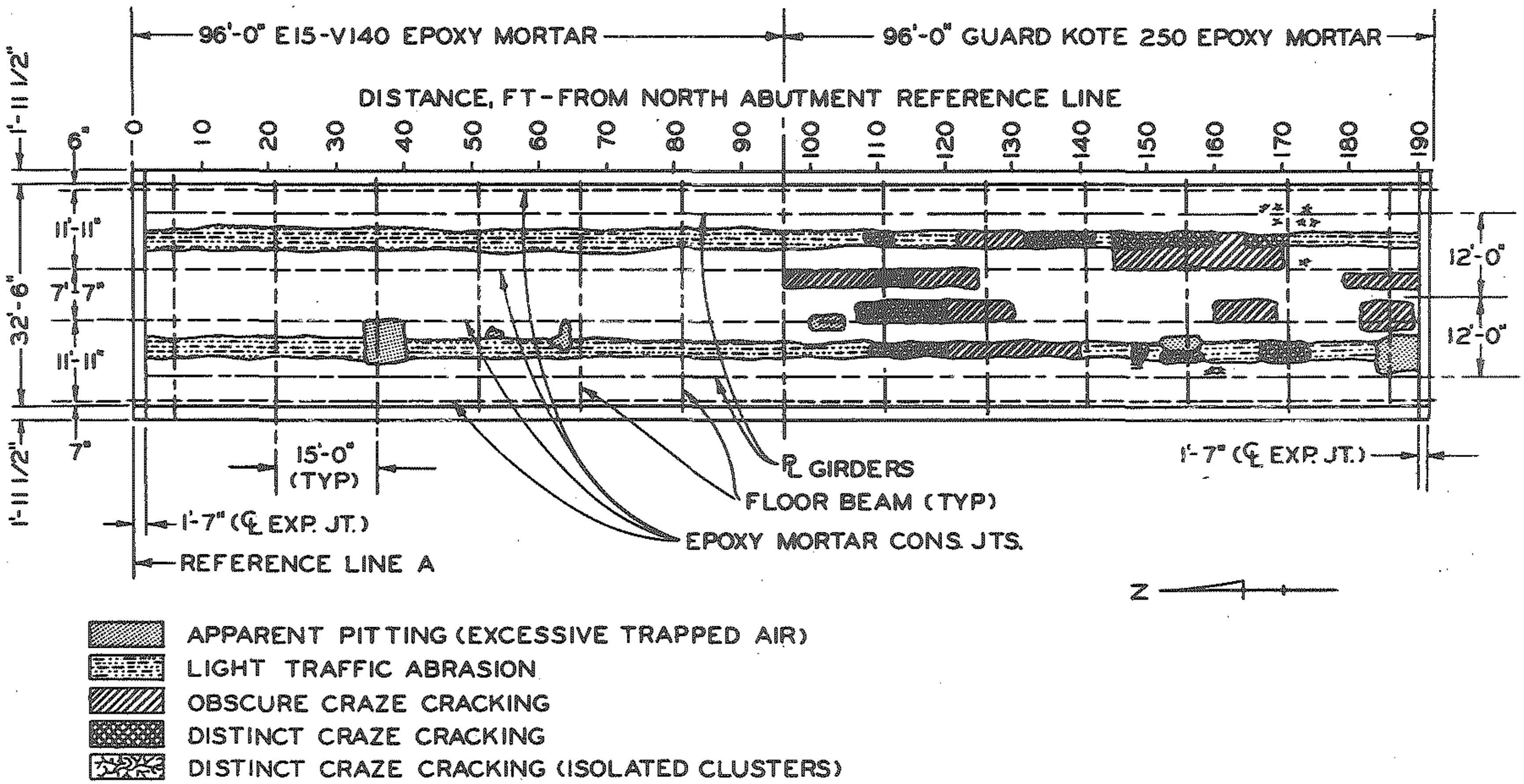
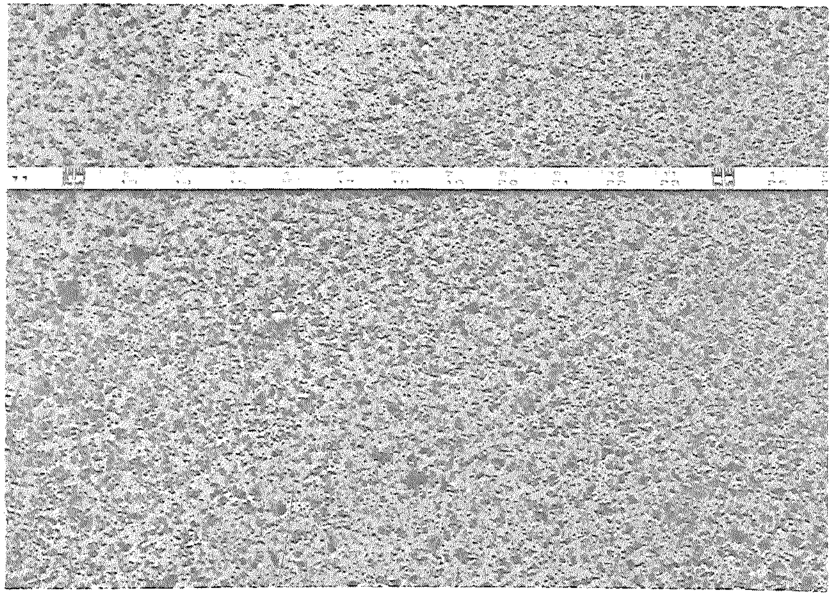
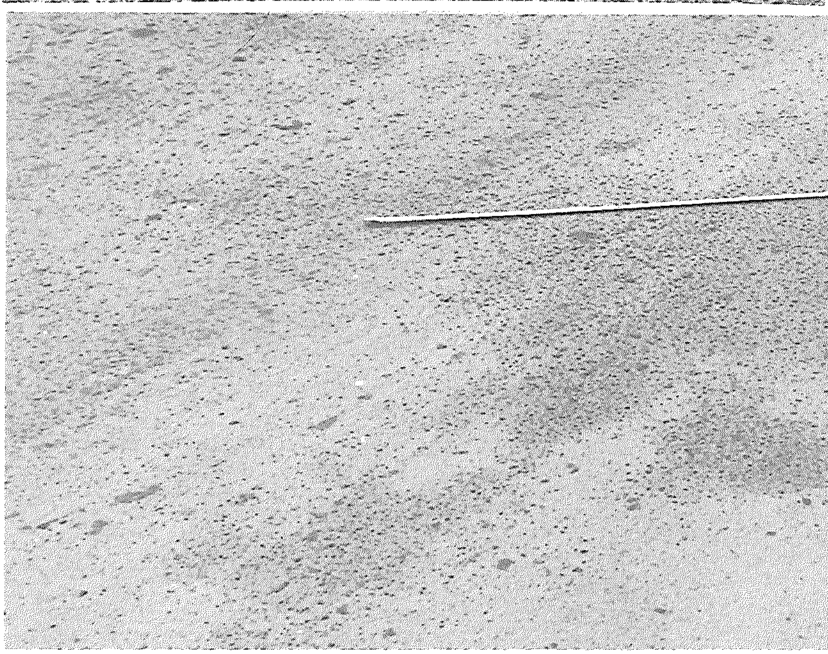


Figure 3. Deterioration features in the epoxy mortar wearing surface as observed on May 6, 1971.

Figure 4. General area and close-up view of apparent pitting in E15-V140 epoxy mortar about 6 ft from the west curb line and at the 40-ft mark from the north abutment reference line.



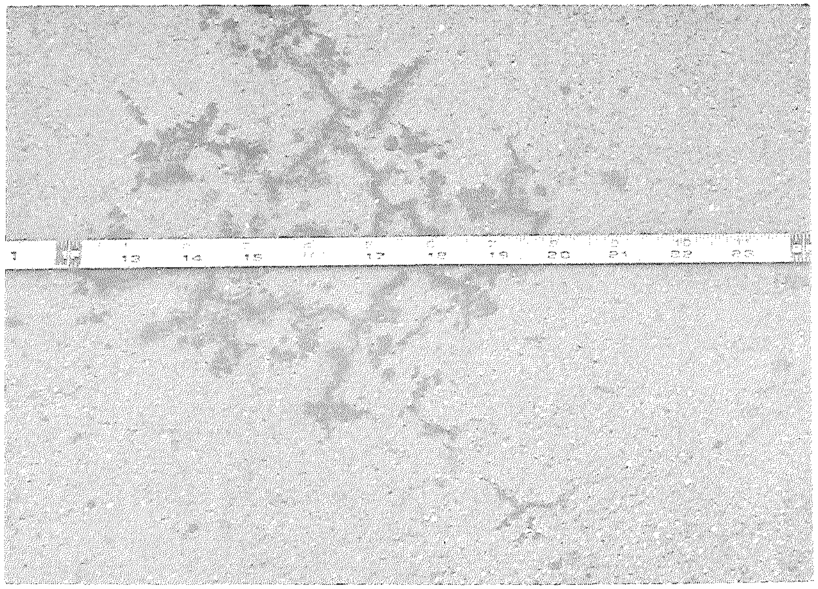


Figure 5. General area and close-up view of isolated distinct craze cracking in the Guardkote 250 epoxy mortar surfacing located 5 ft from the east curb line and at 172 ft from the north abutment reference line. The black streaks originally thought to be rust are actually embedded asphalt probably from approach paving operations.

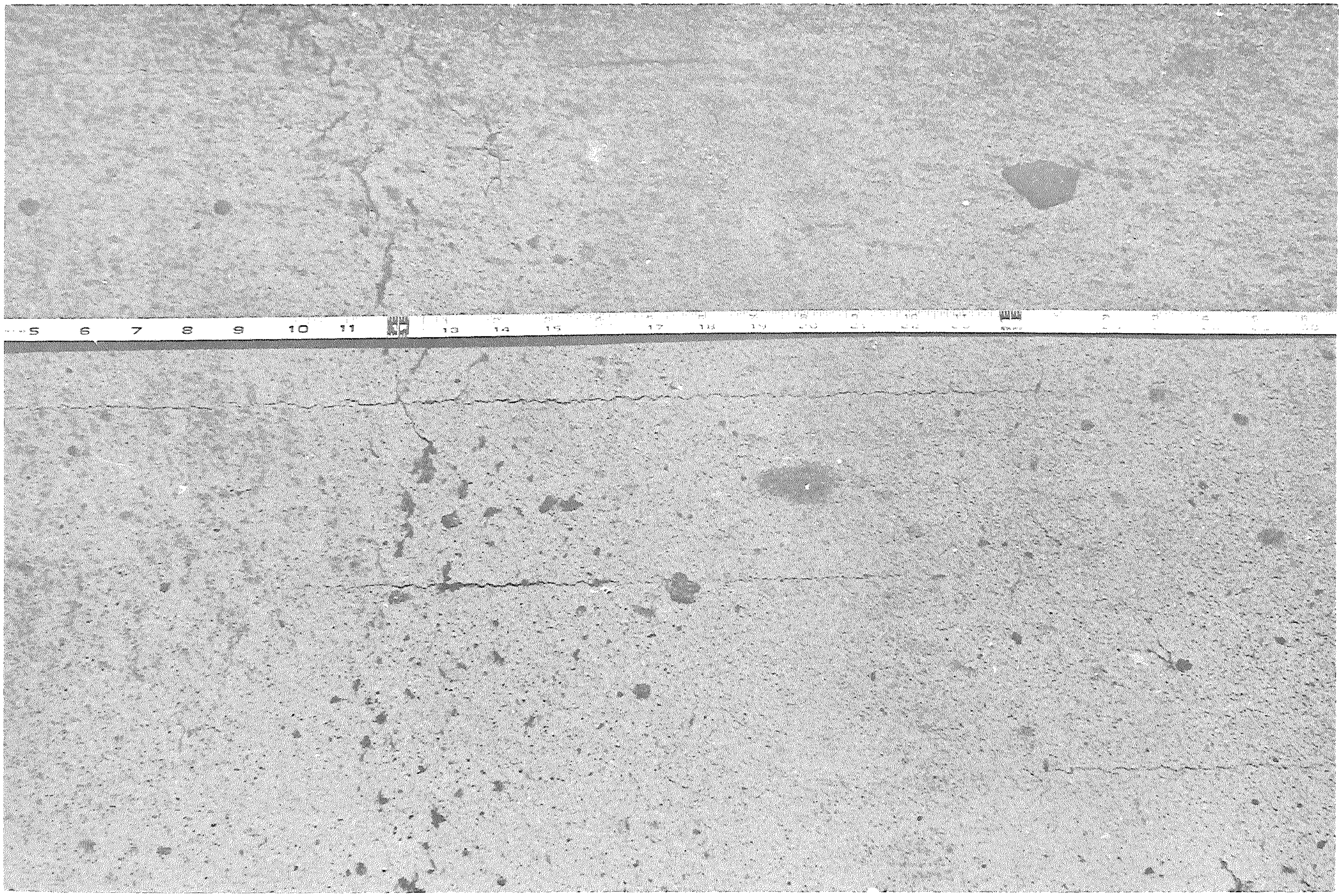


Figure 6. Close-up view of parallel longitudinal cracks which are developing along with a distinct craze crack pattern in the Guardkote 250 epoxy mortar surfacing about 7 ft from the west curb line and at the 150-ft mark from the north abutment reference line.