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16. Abstract

This report describes a full-size field demonstration of a rapid bridge deck replacement concept put forward several years ago. The concept was to utilize a sand mortar made with a rapid setting epoxy to provide the shear tie between precast concrete deck panels and existing steel stringer beams.

The demonstration site was a 50 ft simple span that serves as part of the SPUR 326 overpass over the AT&SF railroad tracks in downtown Lubbock, Texas. The demonstration bridge is one of two identical bridges, side-by-side; the other was redecked using a conventional poured-in-place technique.

Eight precast panels, each 6 ft 3 in. x 45 ft x 8 in., were used to form the experimental deck. Each panel was cast with blockouts (holes) positioned directly over the supporting steel beams. When the original concrete deck was removed, the steel shear studs were cut away leaving the top of the flanges of the steel beams clean and flat. The precast panels were then positioned atop of the steel in an operation lasting less than 5 hours, the new steel studs were welded into place through the blockouts in about 4 hours, and then the panels were epoxied into place in less than 2 full working days.

Subsequent load tests and deflection measurements were compared to results obtained using a 3D computer model. The comparison provided convincing evidence of the composite performance of the completed bridge.

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RAPID BRIDGE DECK REPLACEMENT:
A FIELD DEMONSTRATION AND LOAD TEST

by

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and

James S. Noel

Research Report Number 324-5F

Rapid Bridge Deck Replacement
Research Study Number 2-5-82-324

Sponsored by
Texas State Department of Highways and Public Transportation
in cooperation with
The United States Department of Transportation
Federal Highway Administration

May 1988
Texas Transportation Institute
Texas A&M University
College Station

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac

MASS (weight)

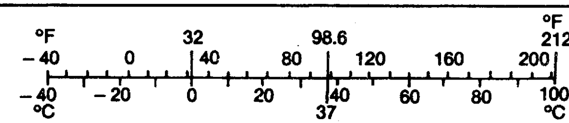
Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the opinions, findings, conclusions and recommendations presented herein. The contents do not reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

KEY WORDS

Bridge Decks, Precast Concrete, Composite Bridges, Epoxy Mortar, Shear Connectors, Load Tests, Bridge Performance, and Deflections.

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Many people graciously and efficiently helped to gather the information contained in this report. The employees of J. D. Abrams Construction Co., the prime contractors, helped by scheduling their construction program around the research instrumentation and testing activities. Even more importantly, they helped by making their expertise and equipment available to us whenever it was needed. The Texas State Department of Highways and Public Transportation, especially the D-5 Bridge Division, and District 5, made substantial contributions to the effort. Mr. Louis Gamboa, the Department's Chief Inspector at the job site, made many observations and suggestions that added greatly to the value of the work.

A special word of appreciation goes to Mr. John Panak, the Project Coordinator. His quiet guidance and counsel are always given in a manner that reflects his thoughtful and professional character.

ABSTRACT

This report describes a full-size demonstration of a rapid bridge deck replacement concept put forward several years ago. The concept was to utilize a sand mortar made with a rapid setting epoxy to provide the shear tie between precast concrete deck panels and existing steel stringer beams.

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TABLE OF CONTENTS

Chapter		Page
I	INTRODUCTION	1
	General	1
	Summary of Earlier Work on Laboratory Model	2
	Description of Laboratory Model	3
	Positive Moment Test	7
	Negative Moment Test	7
	Repetitive Loading Test	8
	Ultimate Load Test	8
	Objectives of Study	9
	Scope of Study	10
II	DESCRIPTION OF PROTOTYPE BRIDGE	11
	General	11
	Description of Precast Decked Span	13
	I-Beams	16
	Precast Panels	16
	Shear Connectors and Deck-Stringer Interface	19
	Epoxy Mortar	19
	Summary	21
III	DESCRIPTION OF CONSTRUCTION	22
	General	22
	Installation of New I-Beams	22
	Casting of Precast Panels	27
	Preparations for Placement of Panels	27
	Placement of Panels	28
	Preparations for Grouting	29
	Grouting of Connections	30
	Description of Grout	30

Chapter Page

	Mixing of Mortar	31
	Placement of Grout	31
	Installation Time	32
	Summary of Deck Installation	33
IV	LOAD TESTS AND RESULTS	35
	General	35
	Loading Vehicles	35
	Description of Instrumentation	37
	Position of Loading Vehicle	39
	Deflection Readings	45
	Summary	48
V	ANALYSIS OF TEST RESULTS	49
	General	49
	Elastic Properties	49
	Composite Geometric Properties	50
	Finite Element Program	50
	Determination of Support Spring Modulus	53
	Measured Versus Computed Deflections	55
	Summary	59
VI	SUMMARY, CONCLUSIONS AND RECOMMENDATIONS	60
	Summary	60
	Conclusions	61
	Design Recommendations	62
	Inspection Recommendations	63
	Research Recommendations	63
	REFERENCES	64

LIST OF TABLES

Table		Page
1	Nomenclature for Loading Vehicle Positioning	44
2	Deflection Readings	46
3	Beam Deflections	47
4	Composite Sectional Properties of Beams	51
5	Comparison of Maximum Measured and Computed Deflections . .	59

LIST OF FIGURES

Figure		Page
1	Layout of 1/3 Scale Model	4
2	Details of Model Stringers	5
3	Details of Model Precast Panels	6
4	Plan View of SPUR 326 Bridge	12
5	Plan View of Precast Decked Span of SPUR 326 Bridge	14
6	Typical Transverse Cross Section of Precast Decked Span	15
7	Precast Panel Details	17
8	Typical Key Way and Bearing Details Between Adjacent Panels	18
9	Typical Shear Connector Detail	20
10a.	Pick-up Spreader	23
10b.	Support Spreader	23
11	Casting Bed with One Panel in Place	24
12	The Next to the Last Panel Being Positioned on the Bridge	24
13	A Typical Bearing Pad to Be Shared By Two Panels	25
14	Mixing the Epoxy Mortar	25
15	Pouring the Mortar Through the Deck Openings	26
16	A Photograph From Under the Bridge Showing the Panels Prior to Grouting	26
17	Loading Vehicle and Its Wheel and Axle Load Distribution	36
18	Deflection Measuring Device	38
19	Location of Installed Dial Gages	40
20	Position of Truck for Loadings L0N and L0S	41
21	Position of Truck for Loadings L2N, L4N and L6N	42
22	Position of Truck for Loadings L2S, L4S and L6S	43
23	Finite Element Layout of Bridge Span	52
24	Neoprene Bearing Pad for Beam Supports	54
25	Measured Versus Computed Deflections for Loading L2N	56
26	Measured Versus Computed Deflections for Loading L4N	57
27	Measured Versus Computed Deflections for Loading L6N	58

CHAPTER I

INTRODUCTION

General

With the increasing problems of bridge deck deterioration, many concrete decks are being or will be replaced. Until recently, the conventional method for bridge deck replacement has been the using of cast-in-place concrete. There are two major problems associated with this method: a) the replacement time is too long, and b) the method greatly interferes with the flow of traffic. These problems are of great concern, especially when the bridge is located in an urban area or in a highly travelled route.

One replacement method that is proving to both save time and minimize traffic interference is the installation of full-depth precast concrete panels. The panels are cast on the site or in a casting yard. The old deck is removed and the panels are placed on, and connected to, the supporting stringers.

Full-depth precast concrete panels placed on simple span bridges can be connected to develop composite action. To assure the horizontal shear force transfer, the modular panels are connected to steel I-beams using epoxy mortar grout and standard shear stud connectors. Epoxy mortar-grouted key ways are employed for the transfer of compressive normal forces between adjacent panels.

Current construction methods used to develop composite action are simple and are typically as follows. First, the deteriorated deck and existing shear connectors are removed such that the top flanges of the I-beam stringers are left bare. The modular panels are then placed on the I-beam floor system with or without bearing pads or strips. Stud shear connectors are then welded to the top flanges through molded openings in the concrete panels. Gaps between the panels and stringers are sealed and subsequently grouted with epoxy or polymer mortar. Finally, the molded openings and the key ways between adjacent panels are also grouted. After the grout has cured, the bond at the panel-stringer interfaces, the shear connectors, and the key way connections make the steel I-beams and the modular precast deck to act as an integral unit in resisting traffic loads.

This study represents the final phase of a research project to demonstrate the implementation of full-depth precast concrete panels for the rapid replacement of deteriorated decks. Earlier studies within the project involved the use of a 1/3 scale laboratory model of a 60-ft span bridge to assess a) the composite action under service loads, b) the applicability of the method for negative moment regions, c) the adequacy of the structure under repetitive loadings, and d) the ultimate flexural capacity of the structure. Earlier findings are summarized in a following section. The culmination of the project resulted in the reconstruction and load testing of a 50-ft simple bridge span redecked using full-depth precast concrete panels.

This report serves a two-fold purpose: 1) it documents the design, construction and installation procedures of the precast deck system of a 50-ft simple span of the SPUR 326 highway overpass over AT&SF Railway in downtown Lubbock, Texas; and 2) it reports the results of full-scale static load tests performed on the precast, redecked span.

Summary of Earlier Work on Laboratory Model

Earlier research involved the testing of a scaled model of a 60-ft prototype bridge. To establish a basis for design of the model, a 1962 Texas standard of a two-lane, 60-ft. nominal span bridge was selected. The prototype bridge included features such as old standard 36WF150 rolled stringers spaced at 8 ft center-to-center with cover plates welded to the top and bottom flanges. It was assumed that the existing deck of such a bridge would be replaced by a series of full-depth precast panels connected to act compositely with the stringers. A 1/3 scale model of a 16 ft typical width of the prototype which included two interior stringers was built. Ref. [6] includes all details of the prototype bridge and the 1/3 scale model. A brief description of the laboratory model is included in the following section.

Description of Laboratory Model

The laboratory model was built by using two modified W12x19 sections, ten precast panels, steel stud connectors and epoxy mortar grout. All components were designed using similitude analyses such that the behavior of the model reflected the actual mechanics of the prototype bridge. Figures 1 through 3 show some of the details of the laboratory model.

Two 20-ft long modified W12x19 sections were used to simulate the old 36WF150 stringers. The beam sections were modified in two ways: a) cover plates, 3/16 in. thick, 2-3/4 in. wide and 13 ft 4 in. long were welded to the top and bottom flanges; and b) both sides of the top and bottom flanges at each end were coped by grinding away a 3/8 in. wide by 34-1/2 in. long strip from each edge. Pairs of 1/4 in. dia. by 2-1/2 in. long studs were welded on the top flanges at a 6-in. spacing.

Ten identical model panels were used in series to form the deck of the model span. The nominal dimensions of each panel were selected to scale the geometry of an 8-in. thick prototype panel. The nominal dimension of the panels were 2 ft long, 64 in. wide and 2-2/3 in. thick. Block-out openings were molded in the panels to accommodate the prewelded headed studs. Also, grooves were molded at the transverse sides to allow the key ways to form. The reinforcement consisted of two layers of welded wire fabric modelling conventional steel requirements.

The concrete mix for the panels was scaled to the extent possible. The gradations of the coarse and fine aggregates that were used were very nearly within the geometrically scaled down limits of AASHTO [2] requirements. Because of the presence of a higher proportion of fines than in usual concrete, a larger amount of water was needed to obtain proper workability. To compensate for strength reduction, a larger cement factor was used. All tests on cylindrical samples reported over 6000 psi of compressive strength at 28 days.

A blend of silica sand and Texas Highway Department Standard Epoxy Binder B-102 was used for the grouting of all connections of the model. Blends of different grades of sandblasting sands were made to fulfill the grading requirements for use with this epoxy. A trowellable mix obtained

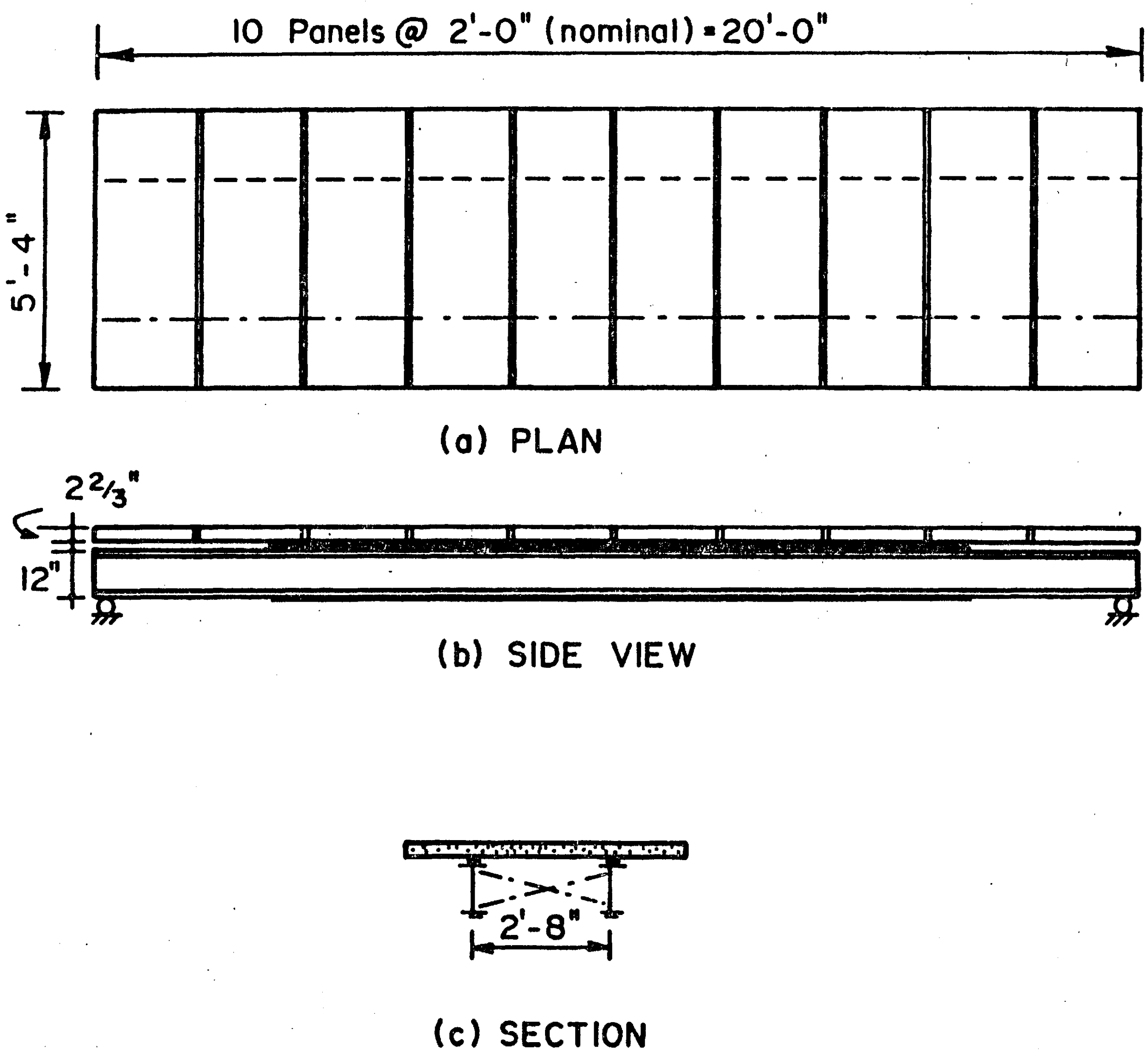


Figure 1. Layout of 1/3 Scale Model.

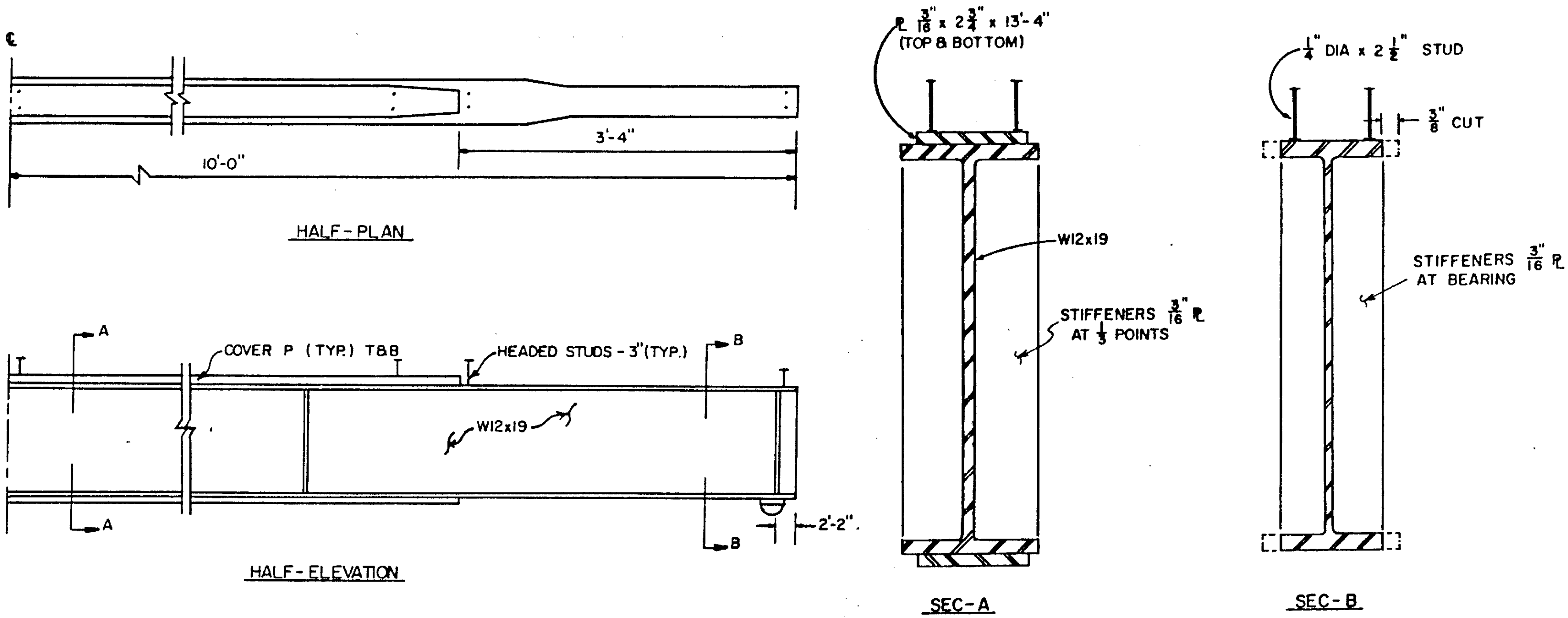


Figure 2. Details of Model Stringers.

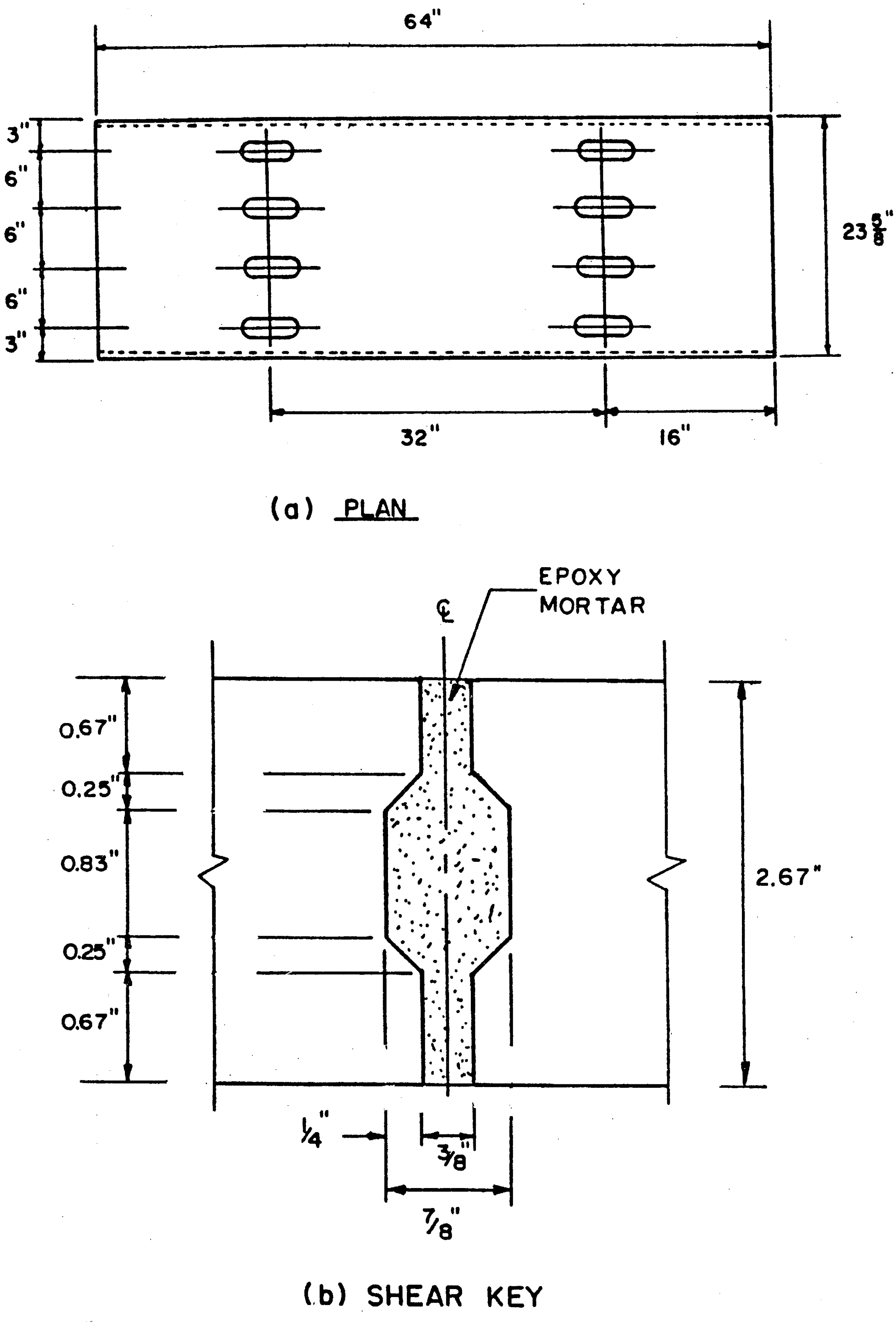


Figure 3. Details of Model Precast Panels.

by a sand-to-epoxy ratio of 3, by weight was used for the casting of the block-out holes around the pairs of shear studs. Meanwhile, a sand-epoxy ratio of 2.75 was used for the grouting of the thin transverse key ways of the model.

Positive Moment Test

Static tests were performed on the laboratory model. The load came in the form of equal concentrated forces applied downward at the third points of each model stringer. The magnitude of the loads was limited to protect the model; however the equivalent service load moments produced by an HS-20 AASHTO truck were surpassed. The details of this study can be found in Ref. [6]. Among the significant findings of this study are: a) the panel-stringer and the panel-to-panel connections are adequate for the development of full composite interaction, b) the relative slip displacement between the precast deck and the stringers was negligible, c) shear stiffness of the panel-to-stringer connection is almost entirely provided by the layer of epoxy mortar at the interface, and d) no evidence of load being carried by the shear stud connectors was observed.

Negative Moment Test

The laboratory model was subjected to negative bending moments by anchoring the ends and applying upward concentrated loads at the midspan. The details of the experiments and the results are reported in Ref. [5]. The results showed that the structure was only capable of developing composite action when the deck was uncracked. The tensile strengths of the key ways and the concrete were not sufficient to maintain the full integrity of the precast deck. Failure first occurred at the vicinity of the key ways, and came in the form of vertical cracks that caused significant reductions in the composite sectional properties. Composite action was lost in a region of longitudinal length of approximately 9 times the slab thickness in each direction from the location of the crack. In Ref. [5] it is recommended that composite action in regions of negative moments be considered for design only when the integrity of the precast deck can be guaranteed by means of longitudinal postensioning. In this event, it is also recommended that the modulus of elasticity of the concrete in tension be taken as 0.67 times the modulus of the concrete in compression. If postensioning is not used, it is recommended that any composite action be neglected for design, including that developed by the longitudinal reinforcement.

Repetitive Loading Test

In an effort to investigate the adequacy of the connections for fatigue, the laboratory model was subjected to two-million load cycles of simulated HS-20 AASHTO truck loading. The conclusions reported in Ref. [7] include: a) the epoxy mortar and steel stud connections performed satisfactorily to hold the precast deck and stringers together, thus preserving composite action during the two-million load cycles, b) there were no losses of rigidities by the connections, and c) the flexural composite properties of the bridge were not decreased by the simulated HS-20 loads. With respect to the design of the mechanical shear connectors for fatigue, it was concluded that the current specifications provided by AASHTO [1] are conservative and are recommended to be used.

Ultimate Load Test

Finally, to investigate the adequacy of the precast deck to stringer connections under ultimate load conditions, the 1/3 model bridge was loaded to its ultimate capacity. Simultaneously, symmetrical push-out tests of full scale shear connector specimens were performed to determine their load capacity. The model was loaded with concentrated forces applied at the third point of the bridge. It must be noted that the laboratory model was designed to fail at the interface connections between the precast deck and the stringers as documented in Ref. [8]. The shear connectors were significantly underdesigned according to AASHTO specifications for the shear capacity of steel stud connectors [1]. The results of the push-out tests revealed that the current AASHTO design formula is conservative, provided that the values for the modulus and strength of the embedding material be used instead of concrete values. From the push-out tests it was observed that if adhesive bond exists at the interface, a brittle debonding failure first occurs and then a plastic failure of the studs proceeds. The same observations were made during the ultimate test of the model. However, the adhesive bond of the interface held up to 95 percent of the flexural capacity. After the debonding failure, the stud connectors began to deform considerably until they yielded. Among the recommendations was that the current AASHTO design formula [1] for the capacity of conventional shear stud connectors be used. But care is suggested when using the formula to assure that reasonable values are selected for the properties of the materials in which the connectors are embedded.

Objectives of Study

The research efforts performed in the laboratory culminated with the reconstruction and load testing of a 50-ft simple span bridge redecked with precast concrete panels. This report has two objectives: 1) it documents the design, construction and installation procedures of the precast deck system of a 50-ft simple span of the SPUR 326 highway overpass over AT&SF Railway in downtown Lubbock, Texas; and 2) it reports the results of full-scale static load tests performed on the bridge using loading vehicles. Within this report, advantages and problems encountered during the construction are documented. Furthermore, the static tests had the objective to verify that full composite behavior was achieved by the redecked bridge. This verification was performed by comparing measured deflections to analytical values obtained by a finite element program that assumed a full interaction between the deck and the supporting stringers.

Scope of Study

No attempt is made here to document similar construction procedures that have been already executed by several highway agencies. In some form or another the design for the precast panels associated with the SPUR 326 work is based on the experiences and designs of these agencies that have been documented in the literature. The authors recognize these efforts conducted by other highway entities, especially the pioneering efforts of the New York Thruway Authority. For a complete survey of deck replacements using full-depth concrete panels, the reader is referred to Refs. [6] and [10].

Furthermore, no efforts were made to develop analytical models that accurately describe the three-dimensional behavior of the bridge under static loads. Instead, an existing finite element program, SLAB49 [9], which has been extensively used by the Bridge Division of the Texas State Department of Highways and Public Transportation (SDHPT), was implemented for the correlation of the measured deflections.

CHAPTER II

DESCRIPTION OF PROTOTYPE BRIDGE

General

The original construction of the SPUR 326 bridge at AT&SF Railway consisted of two separate non-composite structures handling traffic travelling in the North-South direction. Each structure has a total length of 545 ft and is divided into three units: a) a 50-ft simple span, b) a four-span 290 ft continuous unit, and c) a three-span 205 ft continuous unit. The structures were originally completed in 1958 and both included a 33 ft wide (26-ft roadway width) and 6-1/2 in. thick slab supported on four longitudinal steel stringers spaced 8 ft apart.

In 1968, because of deterioration of the concrete deck, the bridge was modified to include a 7-3/4 in. slab. Other modifications, such as the addition of shear connectors for composite action and the welding of cover plates in negative moment regions, resulted from the weight increase of the slab.

In 1986, justified by signs of early deck deterioration and the necessity to widen the roadway width to accommodate increasing traffic volumes, the bridge underwent a second rehabilitation. The roadway width was increased from 26 ft to 36 ft. This resulted in the addition of two I-beams at each of the structures and a slab width of 45 ft. The additional beams were spaced at 7 ft center-to-center. The thickness of the new deck was increased from 7-3/4 in to 8 in. A plan view of the bridge is shown in Figure 4.

The 50-ft span of the west structure was the only span that was redecked using precast concrete panels. The rest of the bridge was redecked using conventional cast-in-place concrete. The reconstruction of this bridge presented some favorable aspects from the research point of view. These aspects include:

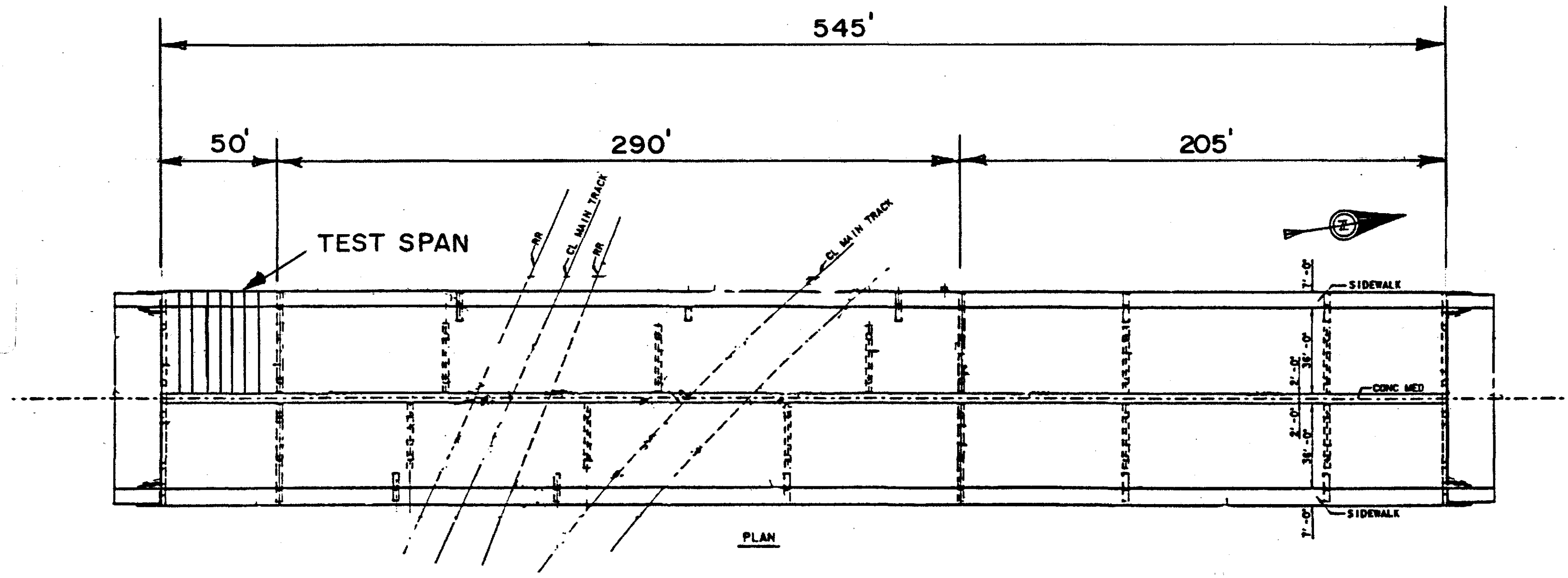


Figure 4. Plan View of SPUR 326 Bridge.

1) The bridge is located in a highly travelled route to and from downtown Lubbock. The influence of traffic on the long-term durability of the precast deck can be investigated; this aspect was essential.
2) The climate of the Lubbock area is one of the most extreme environmental conditions within the state of Texas. The long-term damage to the deck from these conditions, especially to the epoxy connections, can be assessed.
3) The 50-ft span in the west structure is a mirror image of the simple span on the east structure. This element dictated the decision to replace the deck of the west 50-ft simple span with precast panels, while that of the east structure was replaced using conventional cast-in-place concrete. Side by side comparisons on the integrity of both decks can be made in the future.
4) The area under the 50-ft bridge spans is easily accessible and does not represent any hazards to the research staff.

Description of Precast Decked Span

A detailed description of the 50-ft bridge span decked with the precast panels is presented in this section. Figure 5 shows a plan view of the span and Figure 6 shows a typical transverse cross section. The span consisted of eight precast panels connected to six I-beams. The design called for two different sizes of panels, one to be used at the ends, and the other to be used as interior panels. Two exterior I-beams, Beams 1 and 2, were added. This required the reconstruction of the abutments at the south end and the addition of new piers and bents at the north end of the bridge span.

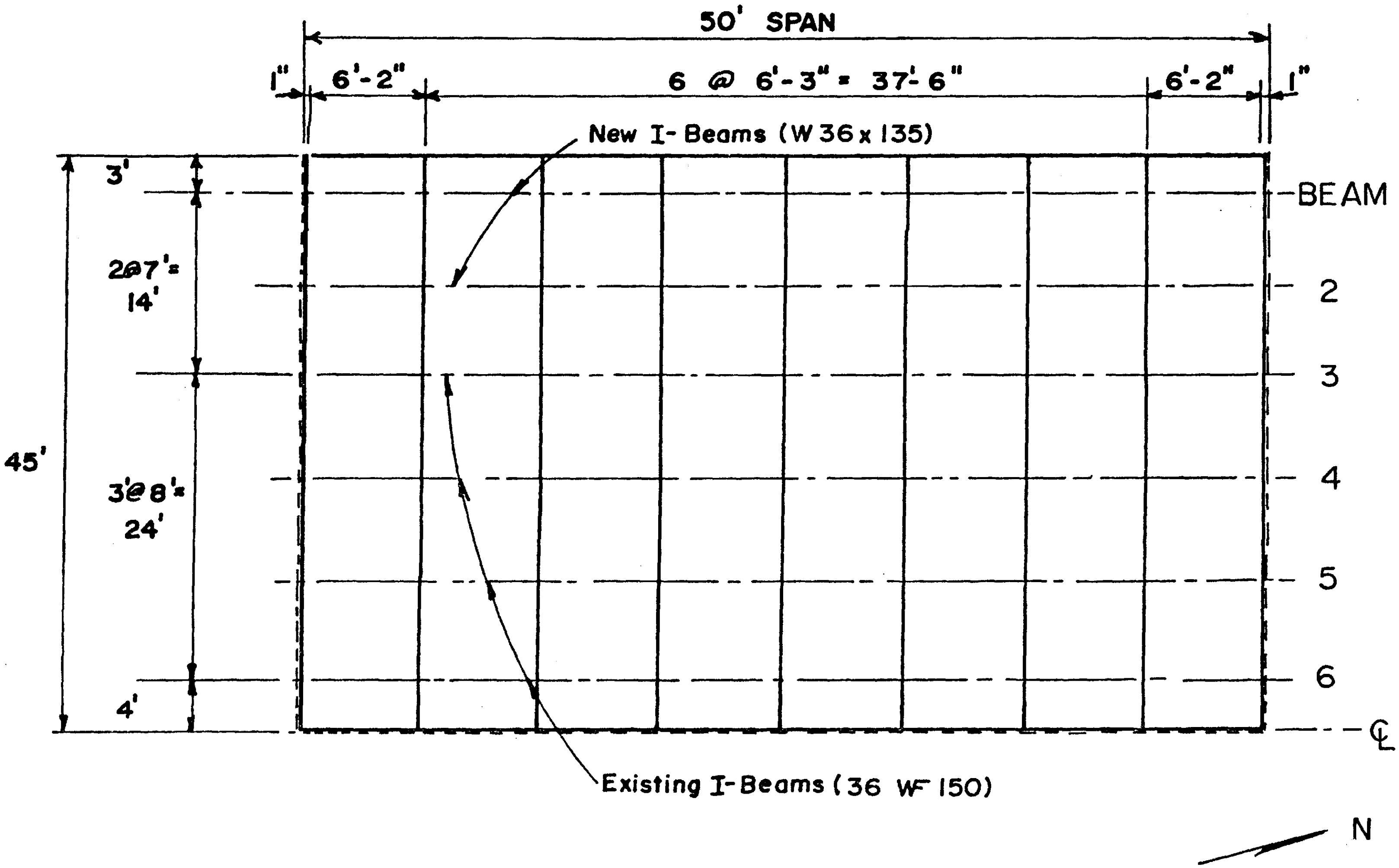


Figure 5. Plan View of Precast Decked Span of SPUR 326 Bridge.

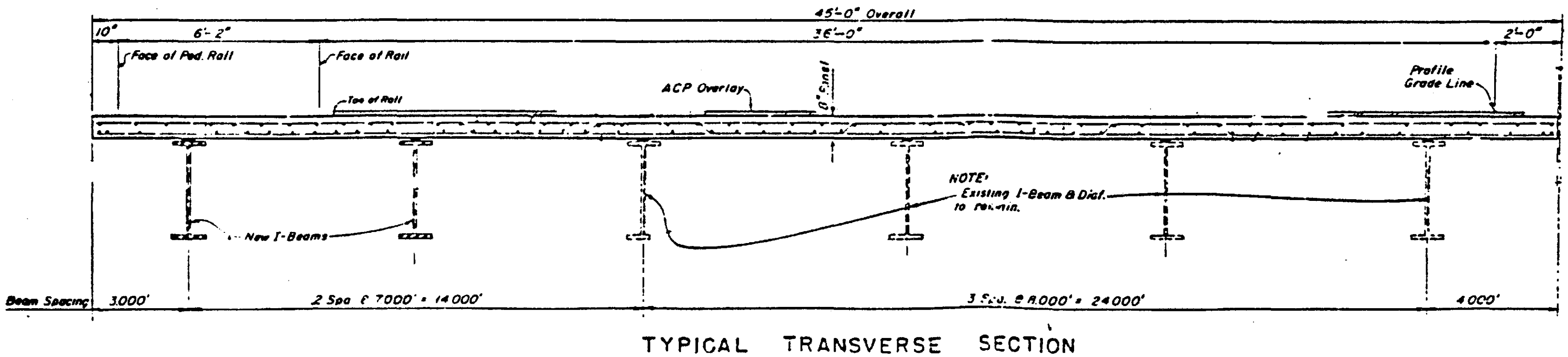


Figure 6. Typical Transverse Cross Section of Precast Decked Span.

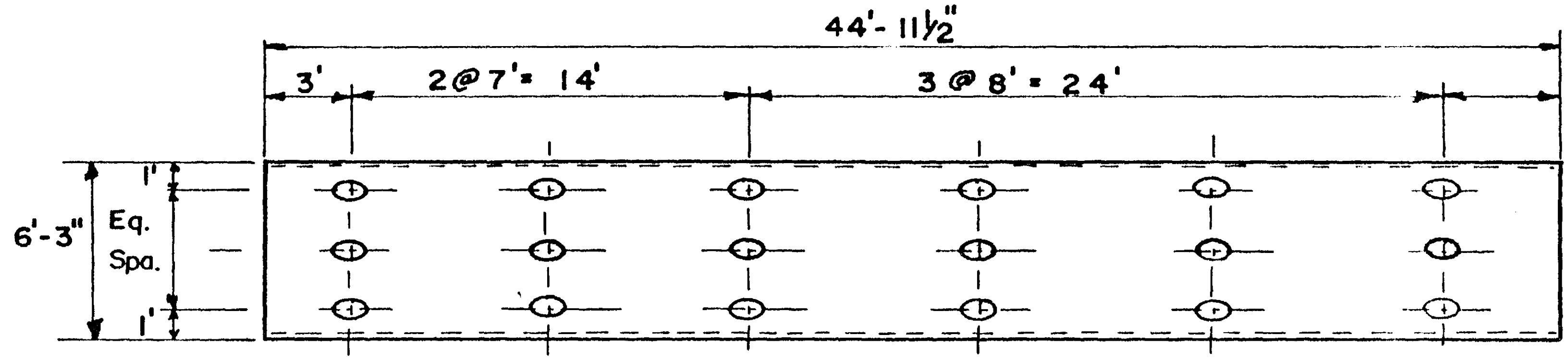
I-Beams

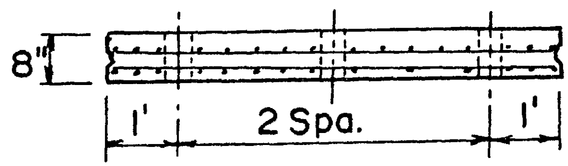
The existing I-beams consisted of old standard 36WF150 wide flange sections spaced 8 ft apart as seen in Figure 6. Beam 6 is located 4 ft from the centerline of the bridge. The new I-beams (Beams 1 and 2) are W 36x135 sections and were spaced 7 ft center-to-center from the outermost existing stringer. All stringers were supported using 11 by 10 by 2-1/2 in. neoprene bearing pads. The installation of new bracing at the supports and at midspan was also done.

Precast Panels

There were two types of panels. As shown in Figure 5 and 7a, one type was used for the panels that were placed at the ends; the other was used for the interior panels. Both types were 8 in. thick and 44 ft-11-1/2 in. wide. The end panels were 6 ft 2 in. long while the interior panels were 6 ft 3 in. long. The reinforcement was typical of that used by the Texas SDHPT in 8-in. decks. Three block-out holes were molded per panel per stringer to accommodate the shear connectors. The locations of the holes were designed to align with the I-beams; their longitudinal spacing is shown in Figures 7a and 7b. The dimensions of a typical block-out hole are shown in Figure 7c. Furthermore, the transverse sides of the panels were grooved so that the shear key way shown in Figure 8a could be formed. Note that the key way opening at the top is larger than that at the bottom. This was done to facilitate the pouring of the epoxy mortar from the top and the sealing of the key ways at the bottom.

Since it was expected that the supporting stringers were not going to be leveled, disuniformities in the bearing supports of the panels became a concern because of possible induced tensile stresses to the slabs. This problem was partially solved using the cushionable bearing pads shown in Figure 8b. Small 2 by 6 by 1 in. thick neoprene bearing pads placed on top of the steel flanges were each shared by the two adjacent panels. This detail created some installation problems which are discussed in the next chapter.





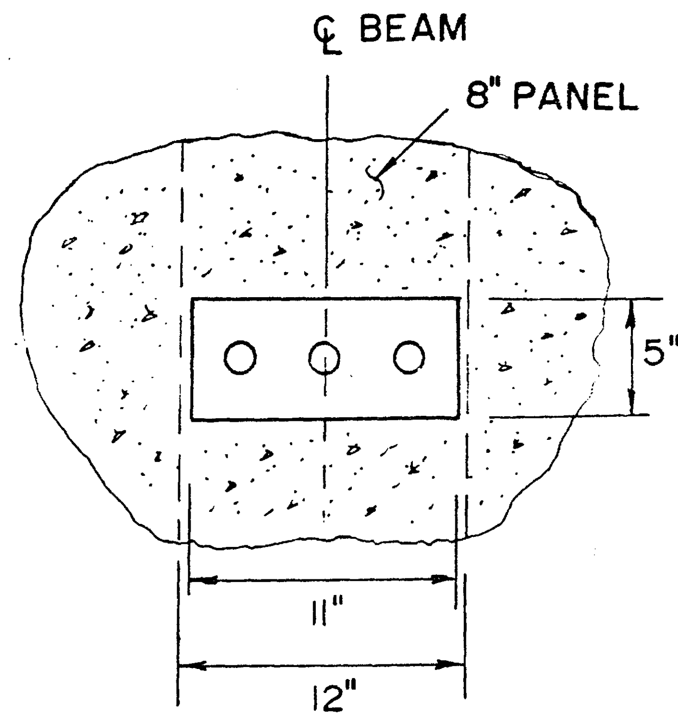


Figure 7. Precast Panel Details.

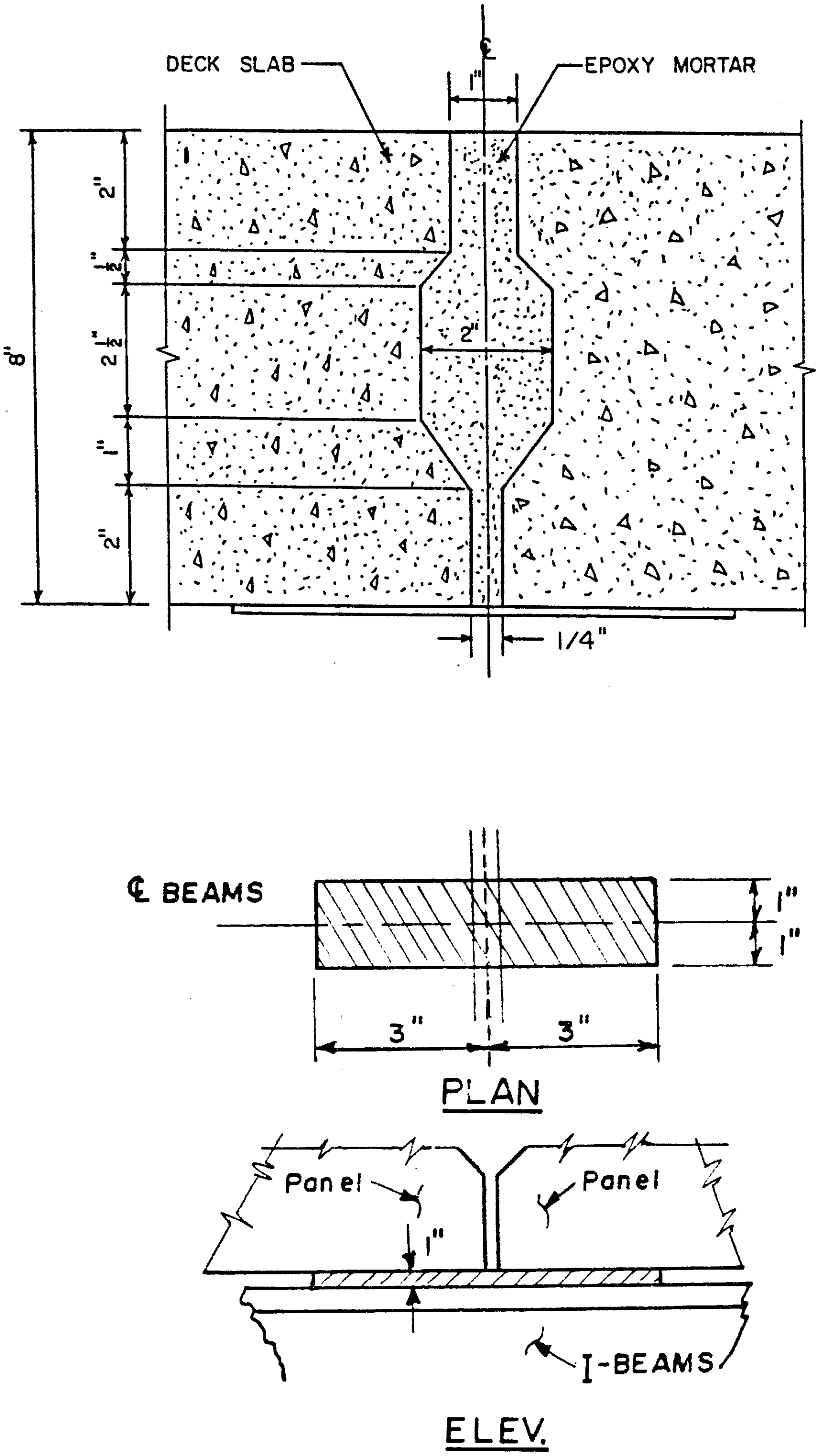


Figure 8. Typical Key Way and Bearing Details Between Adjacent Panels.

Shear Connectors and Deck-Stringer Interface

Three shear connectors, 7/8 in. diameter by 6 in. long, were designed to fit in each block-out hole as shown in Figure 9. This figure also illustrates the details of the sealing of the sides of the deck-stringer interfaces. The design called for the use of galvanized sheet metal angles tack welded to the I-beams and fastened to the concrete. The fastening was not done because of concrete spalling problems.

Epoxy Mortar

The most critical material of the replacement method is the epoxy mortar. All forces required to develop composite action are transferred by this material. The deck-stringer horizontal shear transfer is done by the mortar at the interface and the transfering of normal forces between adjacent panels is done through the grouted key ways. If the bond of the mortar at the interface fails, the horizontal shear transfer is then provided by the shear connectors bearing against the mortar material.

Before selecting the specific epoxy, manufacturers and highway officials were consulted for advice. Elements for the selection process included: a) all tests performed on the laboratory model involved B-102 epoxy binder, and b) personnel in the Texas State Department of Highways and Public Transportation (SDHPT) have had experience in using this epoxy as a patching material.

However, concerns were raised by the manufacturer about possible shrinkage problems of the grout at the block-out holes. It was suggested that a modified B-102 epoxy be employed to slow down the curing time. This suggestion had strength implications to the mortar but minimized shrinkage. Material samples of the proposed epoxy were tested for viscosity, set time, and tensile strength requirements by the Materials Division of the SDHPT. However, none of the required tests included any epoxy mixed with sand. Sample material was obtained by the authors and was mixed with a SDHPT Grade No. 1 sand at room temperature using a sand-epoxy ratio of 3. Compressive tests on cylindrical samples indicated that the strength of the

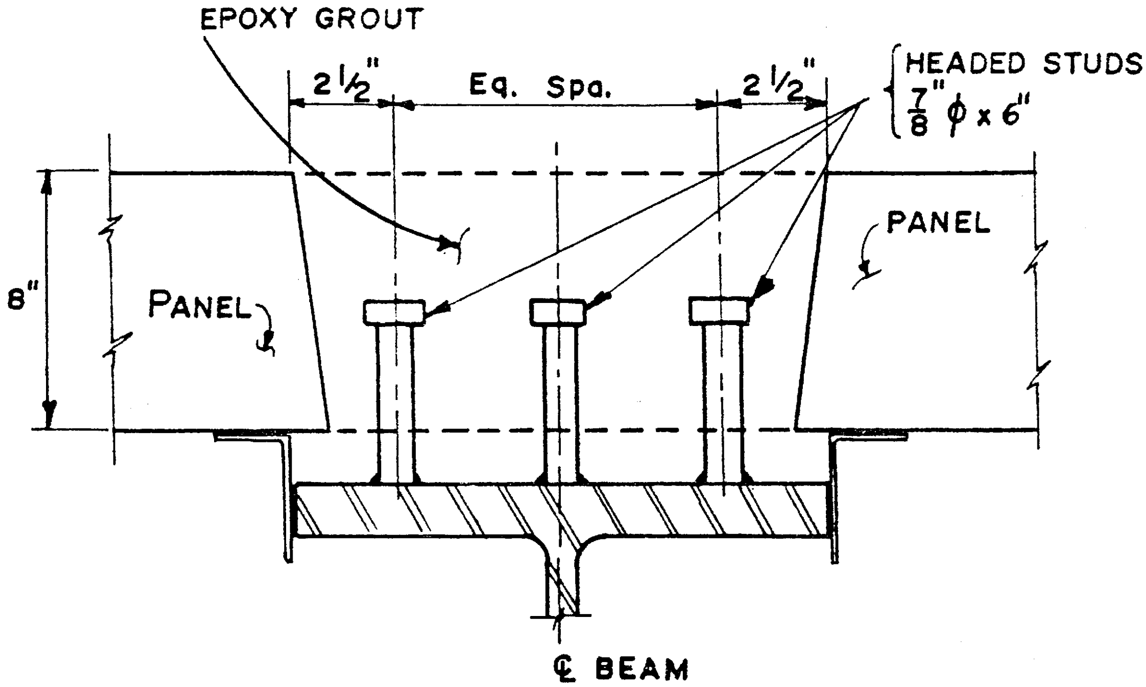


Figure 9. Typical Shear Connector Detail.

modified B-102 was not a concern. All samples exhibited strengths in excess of 5000 psi at 24 hours and over 7000 psi at 7 days. It was further realized that shrinkage might not be a problem because of the sand content. Nonetheless, it was conservatively decided to use the modified B-102 epoxy binder at a minimum sand-epoxy ratio of 3.

Summary

In this chapter the description of the prototype bridge was presented. All dimensions shown are nominal values assuming that there were no misalignments. The final design included significant features to minimize fitting problems during the installation. The panels were sized such that there were 1/4 in. tolerances between panels. Tolerances for the block-out holes were not a concern because the steel connectors were to be welded after the panels had been placed. Tolerances on the bearing points of the panels were a concern, but it was known that such bearing problems could be controlled by elevation surveys and the use of shim plates. The next chapter describes the installation procedures along with descriptions of some problems encountered.

CHAPTER III

DESCRIPTION OF CONSTRUCTION

General

In this chapter the construction procedures from the installation of the new I-beams to the grouting of the precast panels are described. This deck placement required significant planning; and because of all efforts involved by the SDHPT, the researchers and the contractor, the installation process went relatively smoothly but not free of problems. Minor problems included the bearing of the panels, roadway curvature, epoxy handling and epoxy leakages. These problems were corrected as they were identified and did not cause any serious delay in the assembly of the deck. Figures 10 through 16 show some of the construction and installation details.

Installation of New I-Beams

Because the roadway width was increased, the job required the addition of two new I-beams. For this, new abutments, a pier, and bents were constructed. However, the existing bent also required some rehabilitation. Consequently, after the removal of the deck, the existing I-beam floor system was totally removed from its supports.

Once the work of the bents and abutments was completed, the existing I-beam system was placed back on its supports and seated on 11 by 10 by 2-1/2 in. neoprene pads. The new W 36x135 I-beams were then placed. New bracing was welded at the supports and at midspan of the new beams. No modifications were made to the bracing of the initial I-beam floor system. During this process, all existing shear connectors were removed from the top flanges.

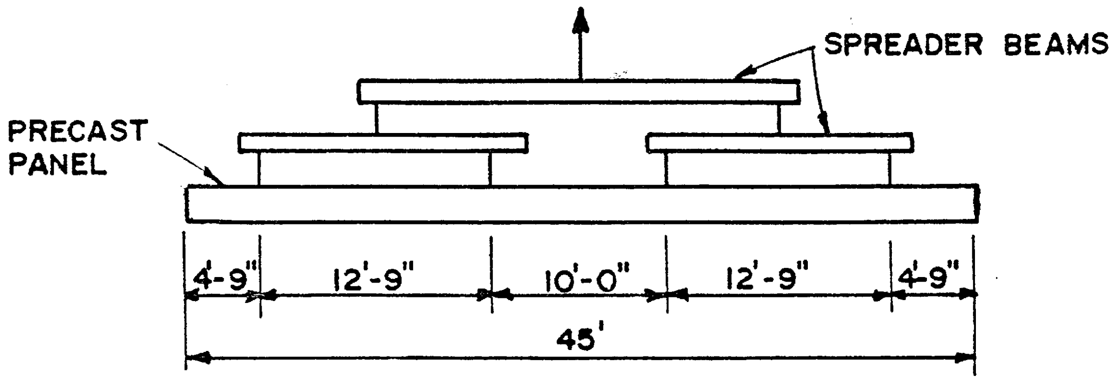


Figure 10a. Pick-up Spreader

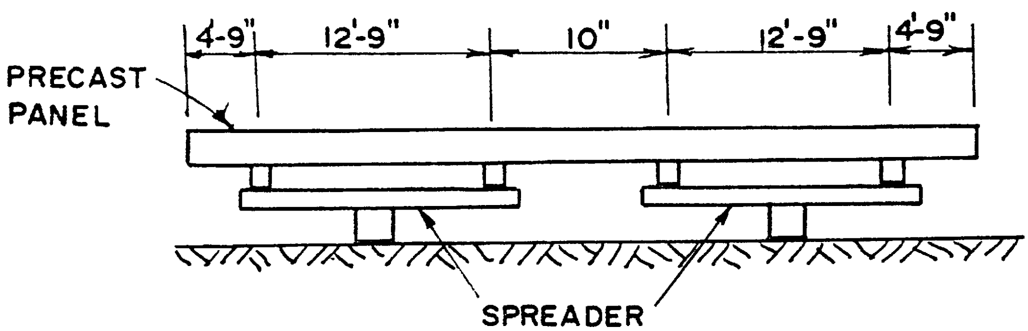


Figure 10b. Support Spreader

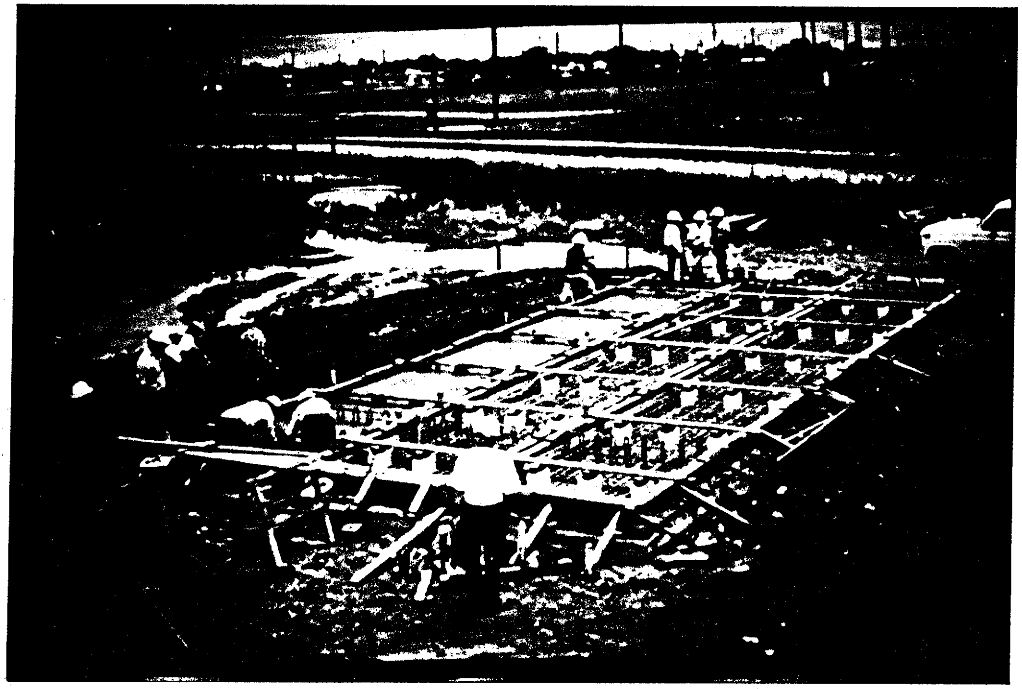


Figure 11. Casting Bed with One Panel in Place.

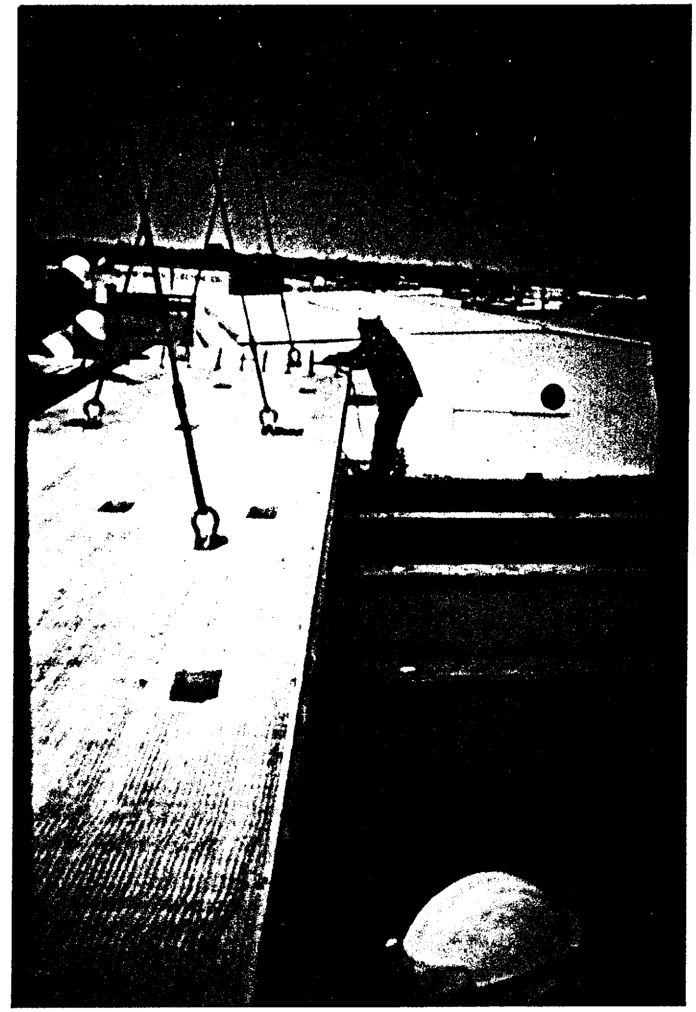


Figure 12. The Next to the Last Panel Being Positioned on the Bridge.

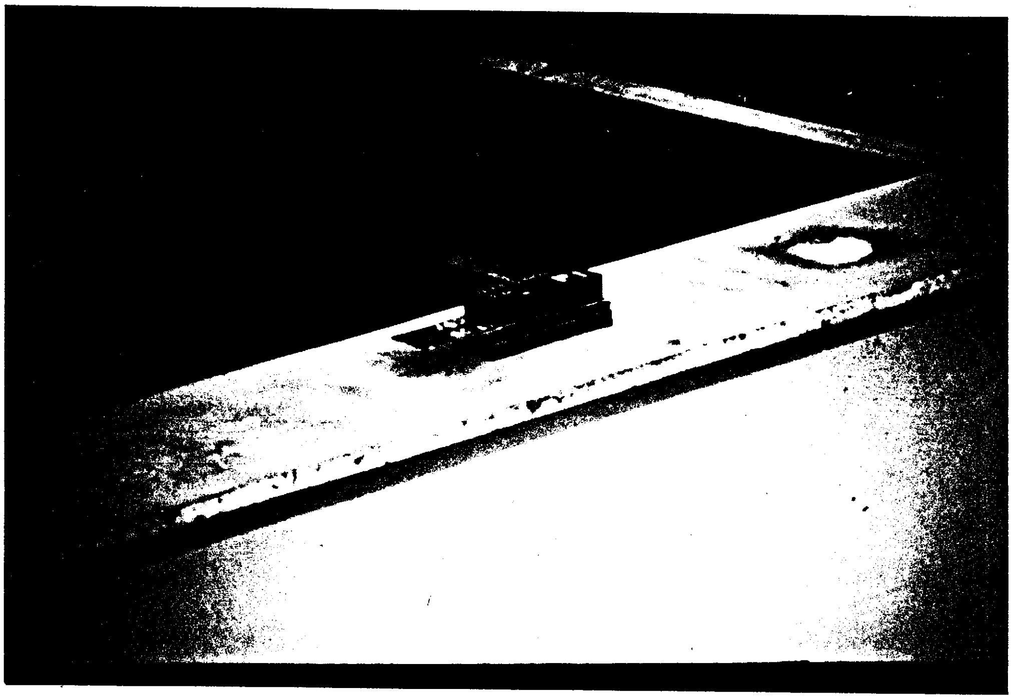


Figure 13. A Typical Bearing Pad to Be Shared By Two Panels.

Figure 14. Mixing the Epoxy Mortar.

Figure 15. Pouring the Mortar Through the Deck Openings.

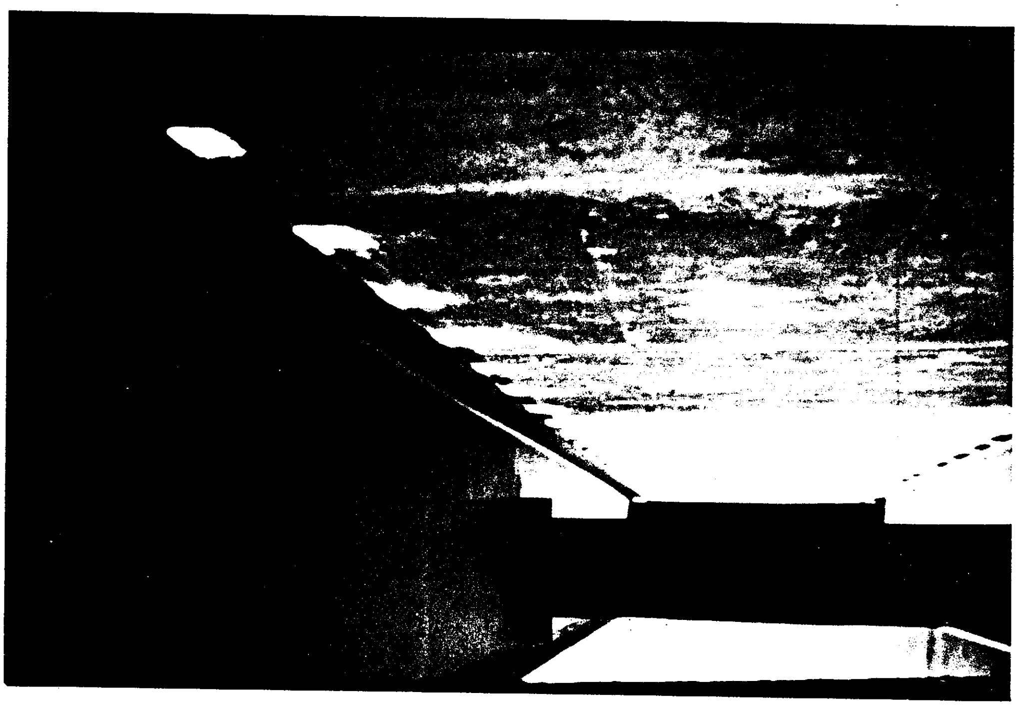


Figure 16. A Photograph From Under the Bridge Showing the Panels Prior to Grouting.

Casting of Precast Panels

When the contract was awarded, the contractor had the option to use either precast panels lifted with a four-point pickup, or precast panels with transverse posttensioning lifted with a two-point pick-up. The contractor selected the first option.

A casting bed was fabricated next to the bridge span. The bed was large enough to allow the simultaneous casting of three panels (see Figure 11). In the first pour, three panels were cast. The concrete was then allowed to cure for about ten days. Then, using a 60-ton movable crane the panels were lifted using the four-point pick-up shown in Figure 10a. The panels were then temporarily stored in a lot adjacent to the bridge and supported as shown in Figure 10b. The procedure was repeated two more times, such that a total of eight panels were cast. After the last cast, three panels were left in place . A record was kept of the location where each panel was cast.

Preparations for Placement of Panels

Two steps were made prior to the placing of the panels to ease installation procedures and to correct for elevation differences of the I-beams. One step was required to accomplish the curvature of the roadway following the placement of the slabs. The second step involved the correction for relative elevation differences at the respective bearing points between the I-beams and panels as cast in the bed.

To correct for curvature, a complete elevation survey was made prior to the placing of any panel. These results were used to determine the shim thicknesses required at each bearing point. The thicknesses were determined to within 1/16 in.

The second correction involved the determination of the elevations at all bearing points at the casting bed and of the bearing points at the I-beams to correct for relative elevation differences. Elevation differences within a panel amounted up to 1/2 in. These relative elevations were later compared to the relative elevations taken at the bearing points of the I-beams just prior to the placing of each panel. Additional shims plates were then placed so that the relative elevations between the I-beams and the panels be minimized. The primary purpose for this step was to minimize the distortion of the panels by uneven supports.

Placement of Panels

During the installation process, two cranes were used. One 120-ton stationary crane was used to lift the panels from the ground to the top of the I-beam floor system. The floor system was located about 25 ft from the ground elevation. This crane had the reach to the casting bed and to the bridge and was used because of its boom length, not because of its capacity. A lighter crane with a similar boom length could have been used. The second was a 60-ton mobile crane which was used to move five panels from the storage lot to within reach of the 120-ton crane.

The first panels that were installed were those left in the casting bed. Then, the other panels were moved from the storage lot to within reach of the 120-ton crane and were subsequently installed. The sequence of the placing of the panels was from south to north. A summary of the step-by-step installation procedure is as follows:

1) The shims required to correct for roadway curvature were placed at all bearing points of all panels.
2) The first panel was lifted and moved into position for placing.
3) The elevations of the bearing points at the I-beams were determined. The relative elevations were assessed and then compared to the relative elevations of the bearing points at the locations where the panel was cast.
4) The shims for the correction of relative elevations were then placed. Subsequently, the bearing pads were installed on top of the shims.

5) The first panel was then lowered and placed into its final position. The alignment of the panel was done with spacers.
6) The rest of the panels were placed using similar procedures as described in steps 3 to 5 (see Figure 12). The longitudinal spacing between the panels was controlled using 1/4 in. spacers to form the key ways.

Some of the problems found were associated with the 2 by 6 by 1 in. thick bearing pads being shared by two adjacent panels. The shims required for the correction of elevations were not necessarily the same for bearing points of the two panels that shared a single pad. Consequently different shim thicknesses were required to fit the two slabs. This was corrected by placing excess shim plates on one side only (see Figure 13). In some cases when the elevation difference was large, the neoprene pad was cut in half. It is suggested that if this detail is used, the panels must be match-cast. An even better solution is to use independent bearing pad supports for each panel.

Furthermore, in some places the shims added up to 2-1/2 in. and the resulting gap thickness added up to 3 in. The final dimensions of the gaps were not uniform and measured from 1 in. to 3 in. Despite these problems, the maximum elevation difference of the top surface of the deck between the panels was less than 1/2 in.

Preparations for Grouting

The preparations for the pouring of the epoxy mortar included the welding of the stud connectors, and the sealing of the deck-stringer interfaces and the key ways. Three 7/8 in. diameter by 6 in. long stud connectors were welded in each of the 5 by 11 in. block-out holes. No difficulties were reported during this task. Subsequently, all dust and ceramic debris were blown from the top flanges of the I-beams.

The next procedure consisted of the placing of the galvanized angles for the forming of the gap between the panels and stringers. The galvanized angles were tack-welded to the top flanges. The original specifications included the fastening of the angles to the concrete using powered concrete fasteners. This task was initiated but resulted in some concrete spalling, and it was decided to discontinue it. The gaps between the galvanized angles and the beams and concrete were subsequently sealed using a heavy duty tape to avoid leakages of the grout. This last detail did not work, and adjustments had to be made at the time of the grouting. This problem is reported in the following section.

The bottom face of the key way was also sealed by using 1/2 in. thick by 2 in. wide strips of wood. These strips were set in place from the bottom of the bridge and were tied from the top. To avoid leakage, gaps between these wood strips and the concrete panels were sealed with styrofoam and tape. No serious difficulties were reported from this sealing detail.

Grouting of Connections

In this section descriptions of the grout, mixing equipment, and the grouting procedures are provided. Some difficulties faced during the grouting process because of factors such as low temperatures and leakage of the grout are also included.

Description of Grout

The contract specifications for the grout consisted of using SDHPT Type VIII epoxy mixed with clean and dry Grade No. 1 sand at a minimum ratio of 3 parts sand to 1 part epoxy by weight. The slow curing B-102 Epoxy Binder met these requirements for Type VIII epoxy and was used. According to instructions provided by the epoxy manufacturer, this epoxy can only be used when the ambient temperature is at least 60^{o}F. Grouting was initiated when the ambient temperature was 55^{o}F, but air heaters were used to warm up the epoxy. The actual sand-epoxy ratio that was used always exceeded 3.

Mixing of Mortar

The mixing of the mortar was done with large drills with mixer paddles attached (see Figure 14). The epoxy was delivered in five-gallon containers filled with one gallon of epoxy resin and hardener. The containers were then used for the mixing and were placed in stands that prevented their rotation. Three stands were available, and two crew workers were used for each; one worker was mixing and the other pouring the preweighed sand. The Grade No. 1 sand was delivered preweighed to mix with one gallon of epoxy at a 3 to 1 ratio. This ratio was further increased when the epoxy mix was found to be too flowable. The actual sand-epoxy ratios ranged from 3 to 5.

Some problems encountered during the mixing were primarily associated with the ambient temperature. The grouting was done in the month of October, and the temperature on the early morning of the grouting date was about 47°F. The grouting was initiated when the temperature rose to 55°F. This represented a delay of three hours. To compensate for this low temperature, heaters were brought to the field to warm up the epoxy. Nonetheless, the low temperature had a significant effect on the consistency of the epoxy. At 55°F, the 3-to-1 sand-epoxy mortar was very stiff and difficult to mix in volumes of five gallons. However, it was still flowable. When the temperature increased during the day, higher proportions of sand were used to achieve a similar consistency of mortar with the same flowability. Samples were obtained for all typical mixtures and some of them tested at 7 days. All showed compressive strengths in excess of 7000 psi.

Placement of Grout

The grout was poured into place through the block-out and the key way openings (see Figure 15). The sequence of the pouring was as follows. First, grouting of the gaps and the block-out openings were done one stringer at a time. When this was done, the key ways were grouted.

The epoxy mortar was poured through the panel openings to fill the gaps. Since the bridge has a 3 percent upward slope to the north, the gaps were filled from south to north. The filling of the gaps was first completed using gravity only. Then, the filling of the block-out panel openings proceeded. During this process a significant leakage problem developed. The tape sealing the gaps between the galvanized angles and the concrete and steel was unable to contain the pressure of the mortar. Some locations experienced severe leakages. This was immediately corrected by pressing wood strips against the sides of the forming angles and by increasing the sand content in the mortar mix. No further problems then developed.

The epoxy mortar was then placed in the key ways. During this activity, no major problems developed. It was initially planned to use trapezoidal funnels for this activity; however, during the course of grouting, the mortar was simply poured into the key ways without the use of funnels. This clearly gave an indication that the 1 in. key way opening at the top was adequate.

Installation Time

In this section the installation time of the precast panels from the placing to the grouting is documented.

The placing of the panels involved two cranes with three operators, eight crewmen for the handling of the panels, one surveyor and a rodman, and a technical person to calculate the shims required for correction of relative elevations. The panel placing was initiated at 9:00 A.M. on October 9, 1987 and was completed at 2:00 P.M. of the same day.

The welding of the studs was conducted by a single person. A total of 432 studs, 7/8 in. diameter by 6 in. long, were welded in a lapse of four hours.

The installation of the galvanized angles along the edge of the flanges to form the gaps was performed by two workers with the help of a cherry picker. It was reported by the contractor that it took them 8 hours to install the angles. The attachment of the wood strips to form the bottom of the key ways took two workers about 3 hours.

The placing of the grout was the most laborious activity. It involved the use of six workers mixing the mortar, and another six placing it. This activity was completed in 12 hours. Remarks about this include the inefficiency of the mixing method, the leakage problems, and actions taken to correct them. The total volume of epoxy mortar that was installed was approximately 125 cubic feet. It is suggested that in future projects, better mixing equipment with larger volume capacity should be considered.

So it can be concluded that the installation time was relatively fast when compared to a conventional decking using cast-in-place concrete. The major time saving arises from the relatively fast installation of the panels and the fast setting properties of the mortar.

Summary of Deck Installation

In summary, the deck installation consisted of several steps associated with the supporting I-beam floor, the placement of the panels, the sealing of gaps, the placement of bearing shims, and the grouting of the connections. Figure 11 through 14 show some of the deck installation procedures as described in this chapter. The deteriorated deck and existing shear connectors were removed. Two new beams were then added. The modular panels were seated on the I-beam floor system through bearing pads. The pads were supported on shim plates that were positioned to correct for roadway curvature and differences in elevations. Stud shear connectors were then welded to the top flanges through the openings in the panels. The spaces between the panels and I-beams (see Figure 16) were then sealed and subsequently grouted with epoxy mortar. Finally, the molded openings and the key ways between adjacent panels were also grouted.

It is recommended that the detail involving the bearing pads shared by two adjacent panels not be used. Instead, the bearing of each panel must be independent. Furthermore, the detail for sealing the galvanized angles can be used if rubber sealants are used instead of tape. Another less laborious potential solution includes the use of readymade sealants, such as caulking rods or foam insulating strips that avoid the time-consuming installation and welding of the galvanized angles.

It is also recommended that if the volume of mortar to be placed is large, considerations be given to using commercially available epoxy mortar mixers. These mixers can be more efficient in mixing larger quantities, and the time savings could be significant.

CHAPTER IV

LOAD TESTS AND RESULTS

General

Full-scale static load tests were performed on the 50-ft precast deck simple span of the SPUR 326 bridge. The experiments were conducted about 45 days after the construction was completed. These tests had the primary objective of verifying the composite behavior of the bridge. In this chapter, these tests are described and include procedures, loading vehicles, instrumentation, and measured deformations.

The instrumentation for these experiments was designed to be simple and reliable by the avoidance of electronic equipment. The only variables that were measured were the beam deflections by means of mechanical dial gages. The deflection readings were made at different locations of the structure. A description of all phases of the test program proceeds.

Loading Vehicle

The primary loading vehicle consisted of a fully loaded Texas SDHPT gravel truck type Chevrolet V8 60 Dump Truck. This truck was placed at eight different locations on the bridge while deflections were recorded. Figure 17 shows the geometric wheel distribution and the measured axle loads. The vehicle axles were individually weighed on a scale belonging to a local asphalt paving company.

The truck had a gross weight of 24,940 lbs; 6,820 lbs were on the front axle and 19,120 lbs on the rear axle. The axle spacing was 12 ft 6 in. and the transverse wheel spacing was 6 ft 6 in. It was to assume that the axle loads were equally distributed to each of the wheels on that axle.

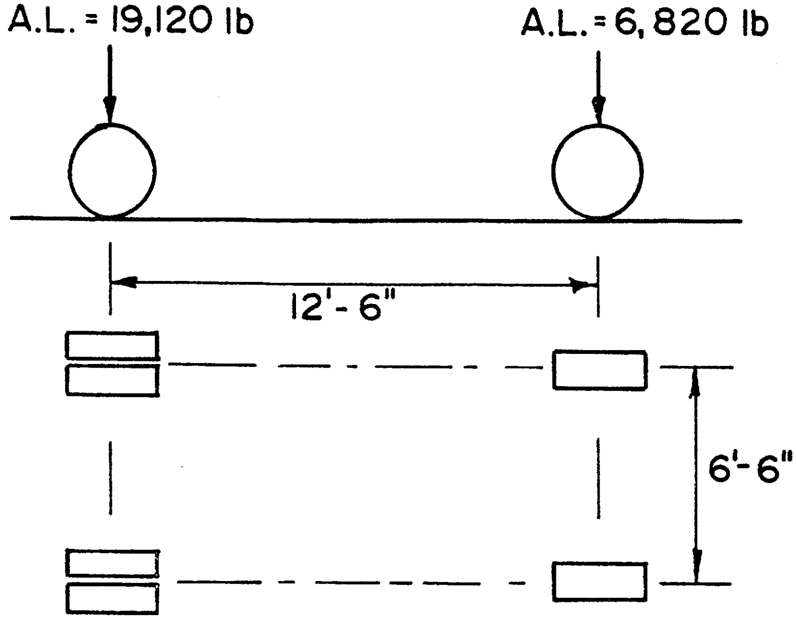


Figure 17. Loading Vehicle and Its Wheel and Axle Load Distribution.

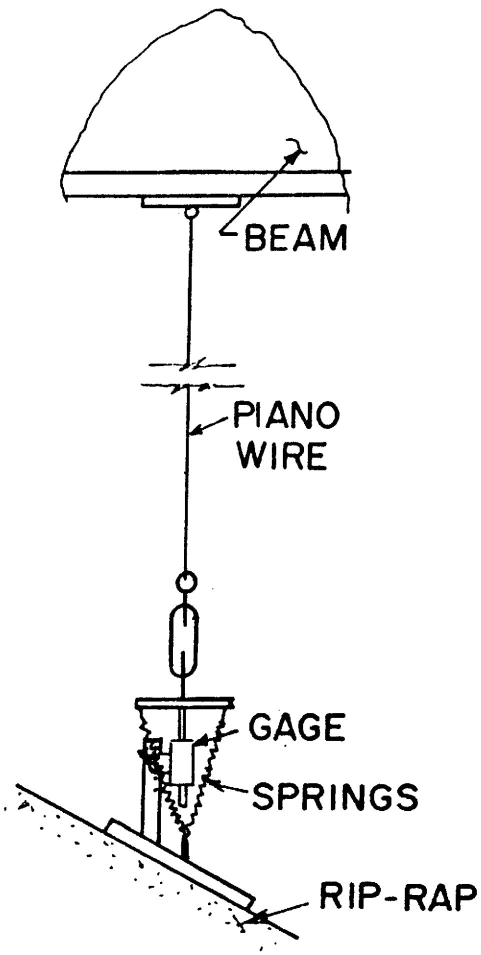
Description of Instrumentation

The instrumentation simply consisted of mechanical dial gages installed at different locations of the bridge. Devices were made such that the deflections could be taken with respect to the rip-rap directly below the points of measurements. Dial gages with resolutions of 0.001 in. and 0.0001 in. were used.

Figure 18 illustrates the deflection measurement device that was used. Each device consisted of steel plates, C-clamps, soft springs, stiff piano wire and a turnbuckle. A 5 by 12 in. plate was clamped to the bottom flange of the I-beams. A piano wire was then suspended from this plate to a turnbuckle, the bottom of which was connected to a 4-in. square plate with four corner holes. A bottom plate was then bonded to the rip-rap surface of the sloping ground, or anchored into the soil if the area underneath did not have a rip-rap surface. This plate had a steel strip that projected vertically to which the dial gage was attached, and two welded hooks. Four soft springs were then placed to connect the four corner holes of the middle plate to the hooks of the bottom plate. The turnbuckle was then used to pretension the springs and the piano wire.

The concept of this device is simple. Beam deflections would be reflected by vertical movements of the middle plate. In turn, movements of the middle plate are then indicated by the dial gage. If the system is elastic, the bottom readings are directly proportional to the beam deflections. Furthermore, if the piano wire is much stiffer than the modulus of the springs, the beam and the recorded deflections would be almost identical.

These concepts had earlier been confirmed using a similar assembly inside the laboratory. The length of the piano wire of the device was varied from 10 to 25 ft. Known deflections were induced at the top while displacement readings were made at the bottom. The maximum reading error was found when the piano wire was 25 ft in length. This error only amounted to 2% of the total displacement at the top of the wire. Therefore, it was decided that correction factors for the readings were impractical and were not needed.



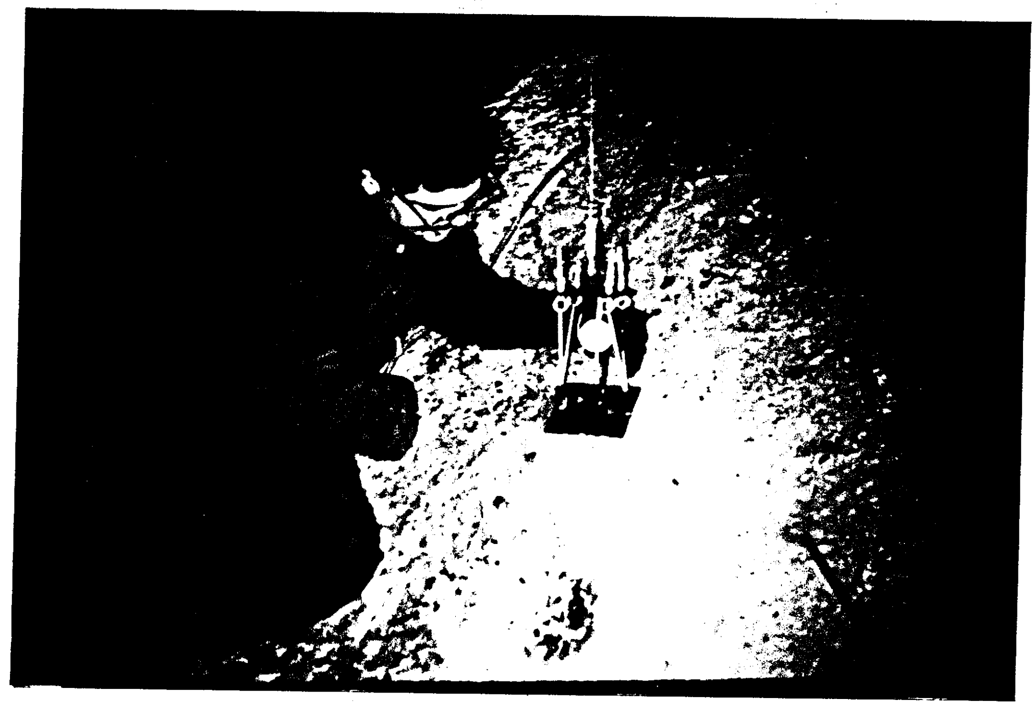


Figure 18. Deflection Measuring Device.

The described measuring devices were installed at eleven different locations of the span. These locations are shown in Figure 19. Two dial gages were also attached to two of the supports to assess the deformation of the 11-in. by 10-in. by 2-1/2-in. bearing pads. Note that only the south half of the bridge was instrumented. This was done because of bridge symmetry about the midspan, and to optimize the readings by reversing the position of the loading vehicles. For accounting purposes, the dial gages were labelled from 1 to 13.

Position of Loading Vehicle

The position of the loading vehicle was determined so that maximum moments were obtained in Beams 2, 4 and 6, and maximum reaction at the south support of Beam 4. There were a total of eight different loadings. The first two are referred as L0N and L0S and are illustrated in Figure 20. In loading L0N, the left front wheel of the truck was placed on top of the south support of Beam 4, as shown. In loading L0S, the right rear wheel was similarly placed over the same support. In both cases the wheel on the other side of the truck falls near Beam 5, as shown. The load positions were selected so that a spring modulus value for the neoprene bearing pads could be assessed.

The next six loadings maximized the longitudinal moments at the midspan locations of Beams 2, 4 and 6. These loadings are illustrated in Figures 21 and 22. The description of the nomenclature is described in Table 1. For example, in loading L2N the gravel truck faced North and was placed to maximize the bending moment in Beam 2; loading L2S also maximized the moment at Beam 2, but the truck faced South. By reversing the position of the trucks, the deflections of the north half of the span can be deduced because of the bridge symmetry about the midspan. This optimizes the information available from the deflection readings.

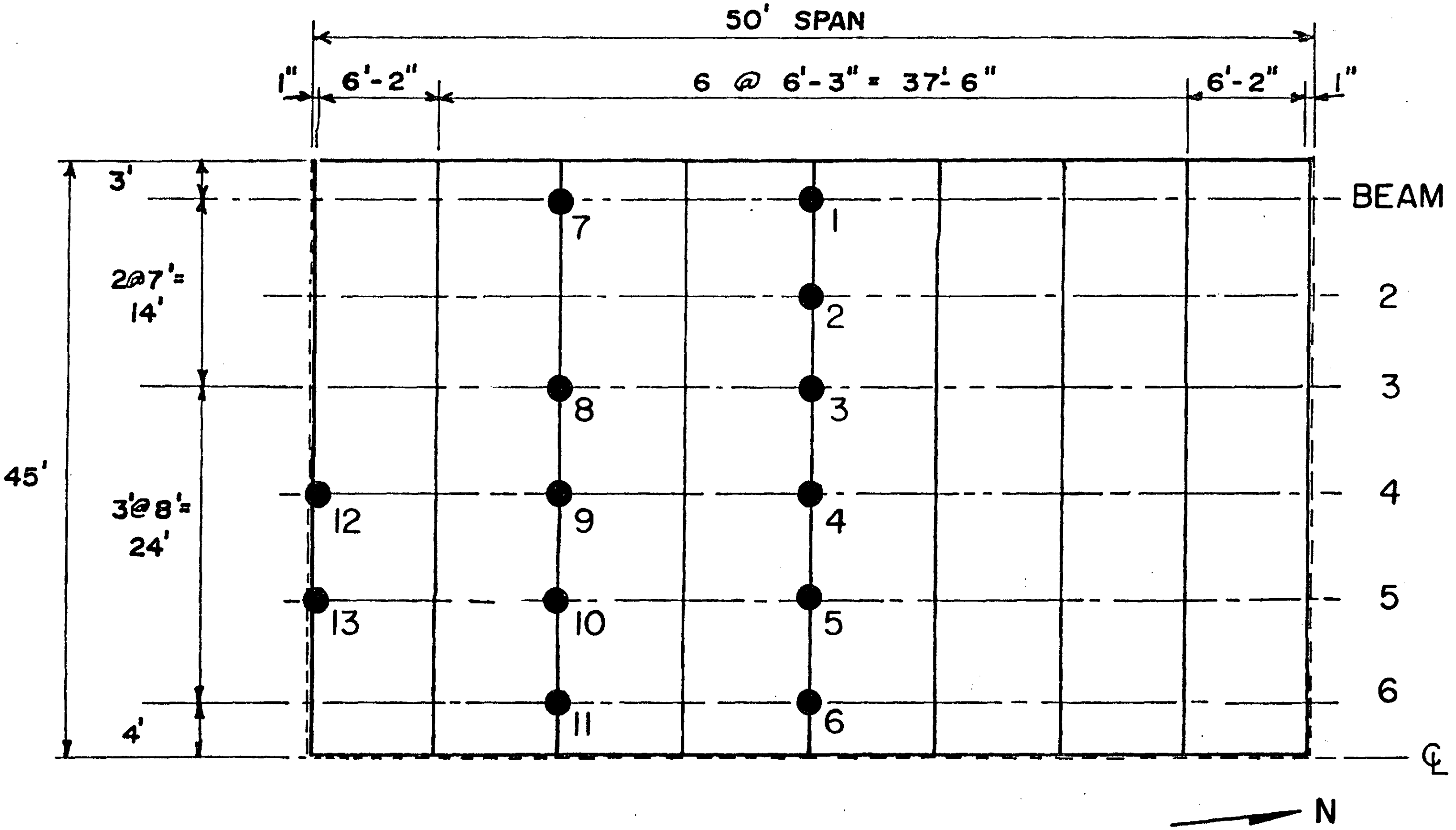


Figure 19. Location of Installed Dial Gages.

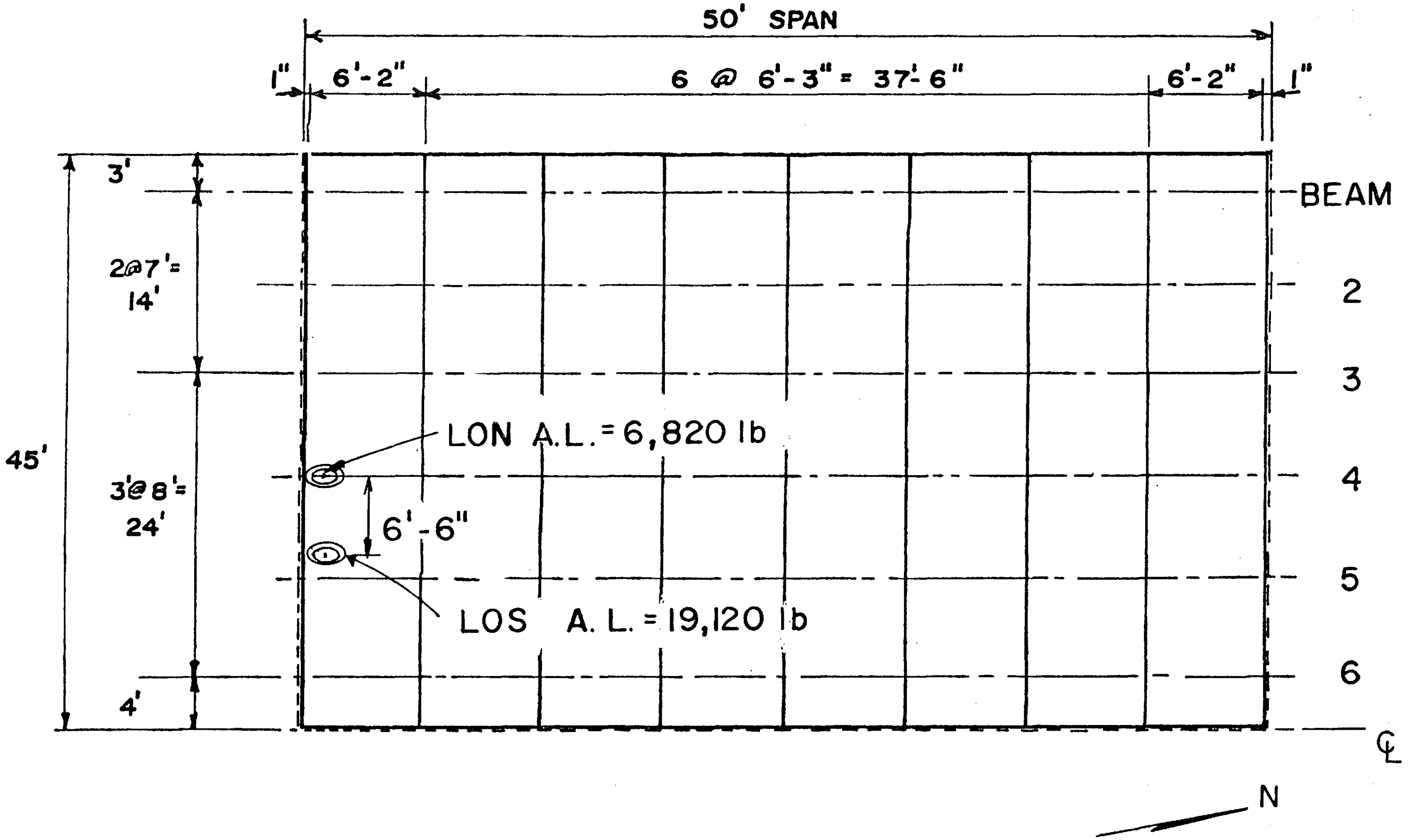


Figure 20. Position of Truck for Loadings LON and LOS.

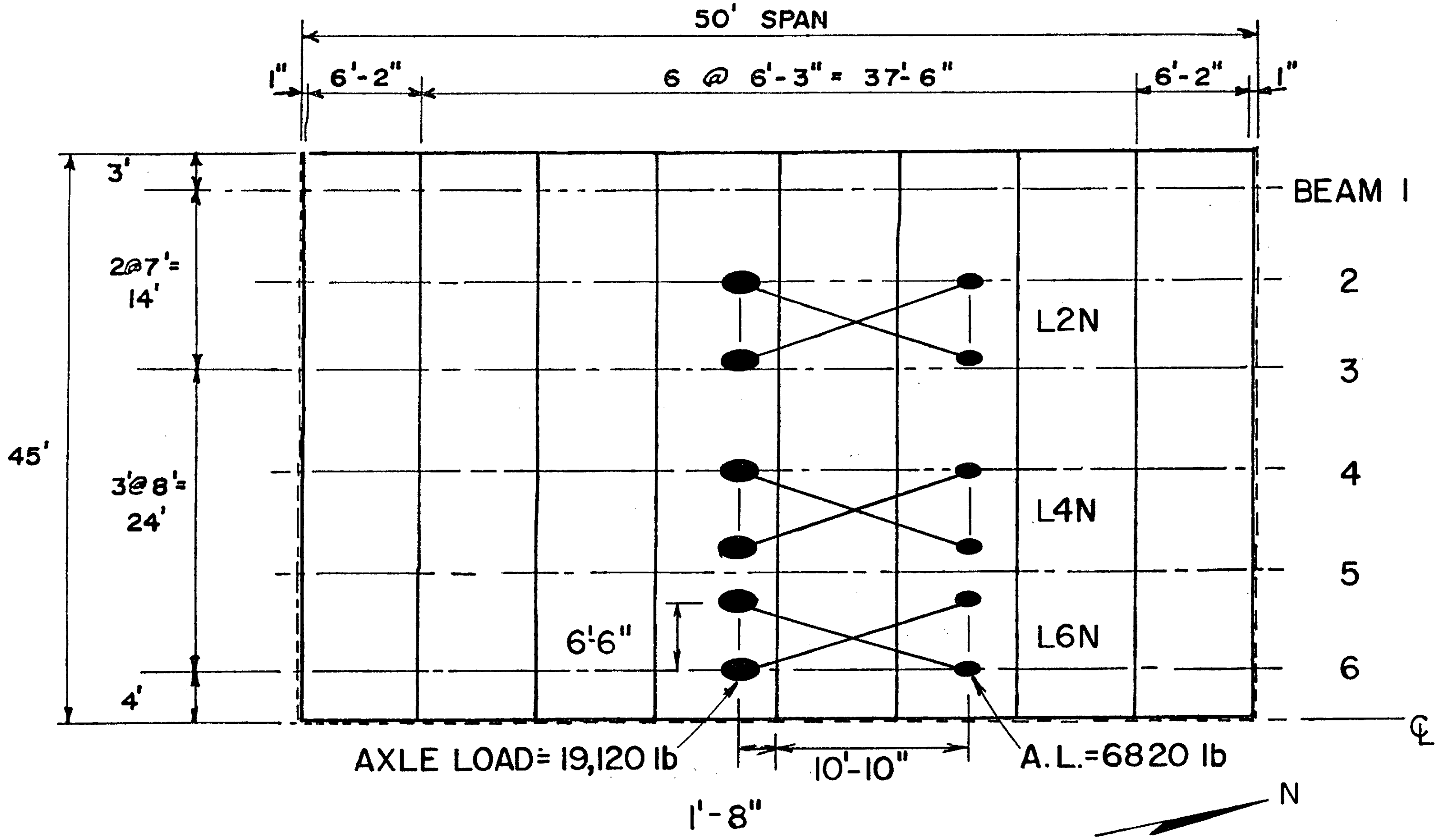


Figure 21. Position of Truck for Loadings L2N, L4N and L6N.

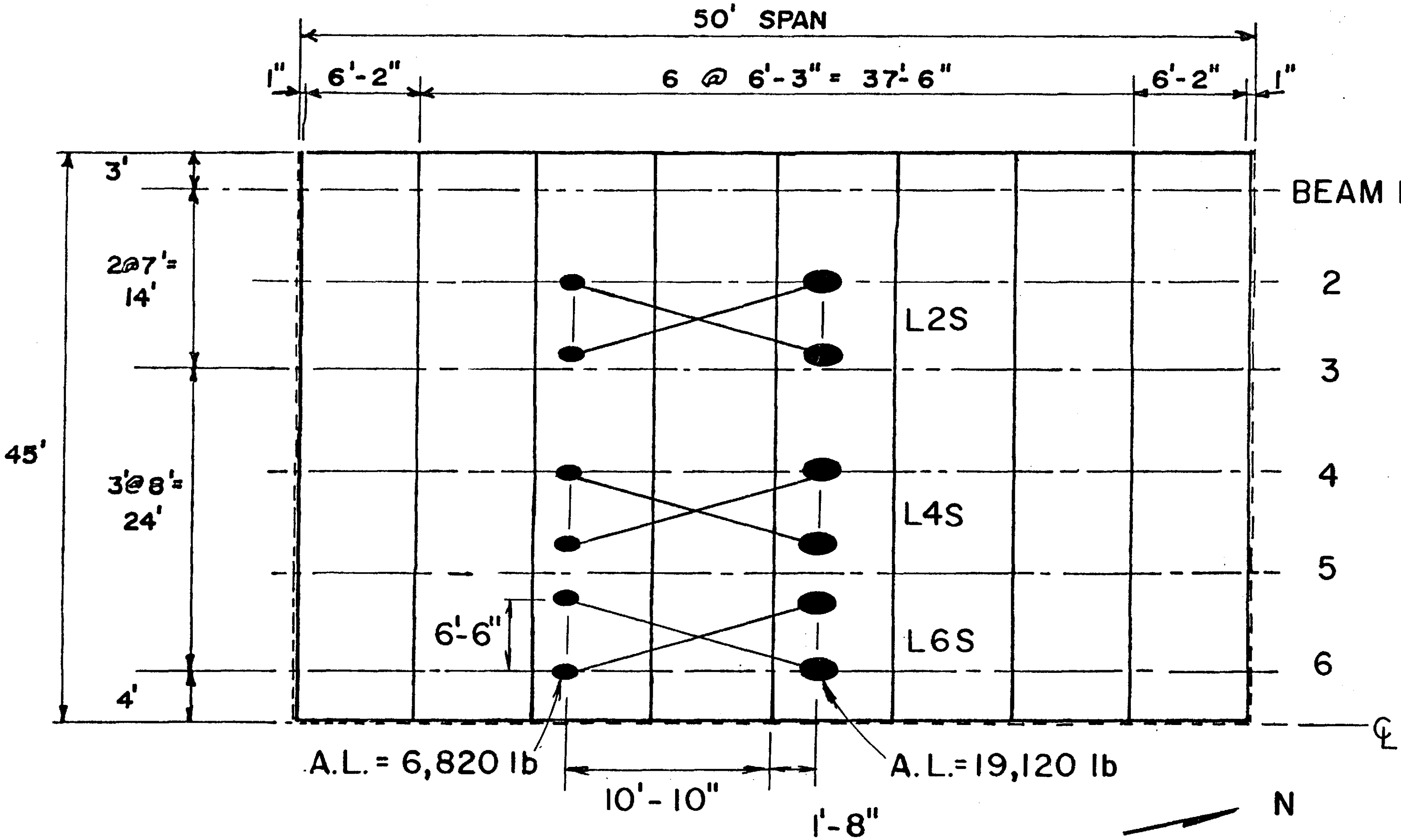


Figure 22. Position of Truck for Loadings L2S, L4S and L6S.

Table 1. Nomenclature for the Loading Vehicle Positioning.

Loading	Truck Direction	Description
L0N	North	Front left wheel over south support of Beam 4.
L0S	South	Rear right wheels over south support of Beam 4
L2N	North	Rear axle placed 1′ 8" south from midspan, left wheel over Beam 2
L2S	South	Rear axle placed 1′ 8" north from midspan, right wheel over Beam 2
L4N	North	Rear axle placed 1′ 8" south from midspan, left wheel over Beam 4
L4S	South	Rear axle placed 1′ 8" north from midspan, left wheel over Beam 4
L6N	North	Rear axle placed 1′ 8" south from midspan, right wheel over Beam 6
L6S	South	Rear axle placed 1′ 8" north from midspan, left wheel over Beam 6

Deflection Readings

The deflection readings were made using the previously described devices. These measuring devices were pretensioned to about a 1/2-in. extension of the soft springs. The recording procedures were as follows. First, the dial gages were zeroed-out. Then, the first loading was applied and the changes in deflections were recorded. The truck was then removed from the bridge. Just prior to its positioning for the next loading, the unloaded readings were taken again. Then, when the vehicle was back on the bridge, the dial gage readings were again recorded. Thereafter, the same sequences were repeated over and over. The measured deflections are summarized in Table 2.

It is also noted that the magnitude of all deflections was relatively small. The maximum deflection reading was recorded at the midspan of Beam 6 when the truck was placed on this beam. This deflection was only 0.073 in.

For the purpose of comparing the measured to the calculated deflections, it is convenient to infer the deflections at the north half of the span and to consider only half of the loadings. When the vehicles faced south, the readings at the south quarter points reflect the deflections of the north quarter points when the vehicles faced north. Therefore, the deflections can be assessed for the entire span assuming bridge symmetry about the midspan. These inferred deflections for loadings L2N, L4N, and L6N are listed in Table 3. The midspan deflections listed in this table are the average values.

Table 2. Deflection Readings.

Gage* Location	Deflection Readings (in.) Loading LON	LOS	L2N	L2S	L4N	L4S	L6N	L6S
1	.000	.002	.026	.025	.002	.000	.000	-.002
2	.000	.002	.044	.043	.010	.010	.000	-.001
3	.000	.002	.042	.042	.023	.024	.003	.002
4	.000	.003	.021	.020	.045	.045	.018	.016
5	.000	.003	.008	.006	.039	.040	.053	.049
6	.000	.003	-.002	-.003	.020	.021	.073	.073
7	.0000	.0007	.0169	.0169	.0011	-.0003	.0000	-.0005
8	.000	.002	.028	.030	.018	.019	.001	.003
9	.000	.004	.014	.015	.029	.031	.012	.012
10	.000	.001	.004	.004	.025	.027	.030	.030
11	.000	.002	-.001	-.002	.014	.014	.049	.051
12	.0008	.0019	.0012	.0017	.0020	.0033	.0003	.0010
13	.000	.001	.001	.000	.001	.001	.002	.002

* For gage location see Figure 19.

Table 3. Beam Deflections.

Location		Deflection (in.) L2N	L4N	L6N
Beam 1	SQP*	0.0169	0.0011	0.0000
	MSP	0.0255	0.0010	-0.0010
	NQP	0.0169	-0.0003	-0.0005
Beam 2	SQP	-	-	-
	MSP	0.0435	0.0100	-0.0005
	NQP	-	-	-
Beam 3	SQP	0.0280	0.0180	0.0010
	MSP	0.0420	0.0235	0.0025
	NQP	0.0300	0.0190	0.0030
Beam 4	SQP	0.0140	0.0290	0.0120
	MSP	0.0205	0.0450	0.0170
	NQP	0.0150	0.0310	0.0120
Beam 5	SQP	0.0040	0.0250	0.0300
	MSP	0.0070	0.0395	0.0510
	NQP	0.0040	0.0270	0.0300
Beam 6	SQP	-0.0010	0.0140	0.0490
	MSP	-0.0025	0.0205	0.0730
	NQP	-0.0020	0.0140	0.0510

* SQP = South Quarter Point
MSP = Midspan point
NQP = North Quarter Point

Summary

In this chapter, the test procedures were described and included the descriptions of the loading vehicle, the devices to measure deflections, and the positions of the vehicle on the bridge. The measured deflection readings were also reported. These deflections showed that the bridge behaved almost symmetrically. In the proceeding chapter, the deflection values are compared to deflections computed by a finite element model of the full-composite, 50-ft span bridge.

CHAPTER V

ANALYSIS OF TEST RESULTS

General

The results of an analysis to compare the theoretical deflections to the measured values are described in this chapter. Also reported are measured elastic properties of concrete and epoxy mortar samples and the computed properties of the composite sections. The finite element program SLAB49 [9] was employed for the determination of theoretical deflection profiles. These profiles were graphed and compared to the observed deflections for loadings L2N, L4N, and L6N. Good correlations were obtained in each instance. The results are very conclusive that the bridge is performing in a full composite manner.

Elastic Properties

The elastic properties of concrete and epoxy mortar cylindrical samples were measured. The elastic modulus of the concrete was determined from cylinder tests performed on seven 6 by 12 in. samples, three weeks after the load testing of the bridge. The age of the samples varied from 4 to 5 months. The load deformation curve was obtained by placing the samples in a loading machine while deformations of the samples were measured by an extensometer. The initial slope of the load-deformation curve was used to calculate the elastic modulus. The average modulus was found to be 3,900 ksi. The lowest and highest measured values were 3,650 and 4,100 ksi. This variation can be attributed to the different batches and ages of the concrete. The average value was taken as representative for the entire slab for the purpose of computing the properties of composite sections.

In a similar manner, the elastic modulus of the epoxy mortar was determined. Twelve 3 in. by 6 in. cylindrical samples were tested and the respective load-deformation curves obtained. Then, the elastic moduli of the samples were determined from the initial slope of the load-deformation curves. Variations of the modulus were found because of the different sand-epoxy ratios. The lowest and the highest modulus were 460 and 970 ksi and the average of all samples was 750 ksi. This average value was not used for the calculation of the theoretical deflections. This is because the calculations of the composite sectional properties and, in turn, deflections are insensitive to the epoxy mortar elastic properties.

Composite Geometric Properties

In this section the composite properties of the beams were computed using the transformed area method. These properties were calculated assuming nominal dimensions of the beams, a slab thickness of 8 in., a slab-to-beam gap dimension of 1.75 in., and a full effective slab width. Other assumptions included: the elastic modulus of steel and concrete were taken as 29,000 ksi and 3,900 ksi, respectively, and the epoxy material in the gap was ignored. Table 4 lists the composite moments of inertia of the transformed steel areas of all six beams.

Finite Element Program

The finite element program that was used for the deflection computations is SLAB49 and is described in Ref. [9]. This program was developed for the analysis of orthotropic slabs and bridge floor systems. Features of this program include the modelling of the following: a) the plate-like behavior of the slab, b) the stiffening effects of the floor system, c) the spring supports, and d) the diaphragm system.

The finite element grid is defined by specifying stations and their x and y increments. Figure 23 shows the corner stations that were used for the building of the finite element mesh. The mesh consisted of rectangular elements spaced every 1 ft in the transverse direction, and every 1.21875

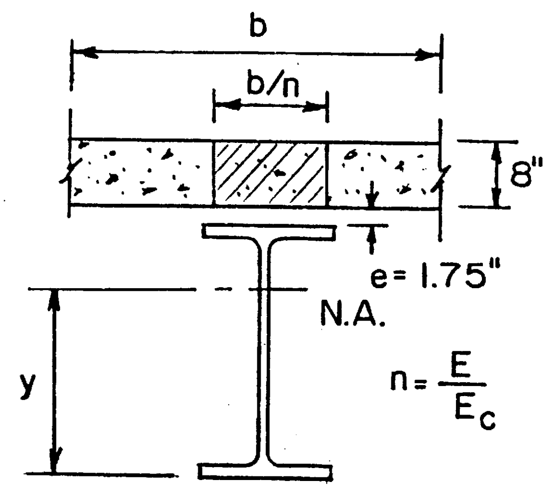
Table 4. Composite Sectional Properties of Beams.

Beam No.	Steel Beam	b (in.)	y (in.)	I_c *** (in^4)
1	W36x135*	78	33.7	23,200
2	W36x135	84	34.1	23,500
3	36WF150**	90	34.0	26,500
4	36WF150	96	34.4	26,900
5	36WF150	96	34.4	26,900
6	36WF150	96	34.4	26,900

* Ref.[3], I = 7800. in^4, A= 39.7 in^2 , d = 35.55 in.

** Ref.[4], I = 9012. in^4, A= 44.2 in^2 , d = 35.84 in.

*** Note: I^c transformed to steel area, E=29,000 ksi and E^c=3,900 ksi



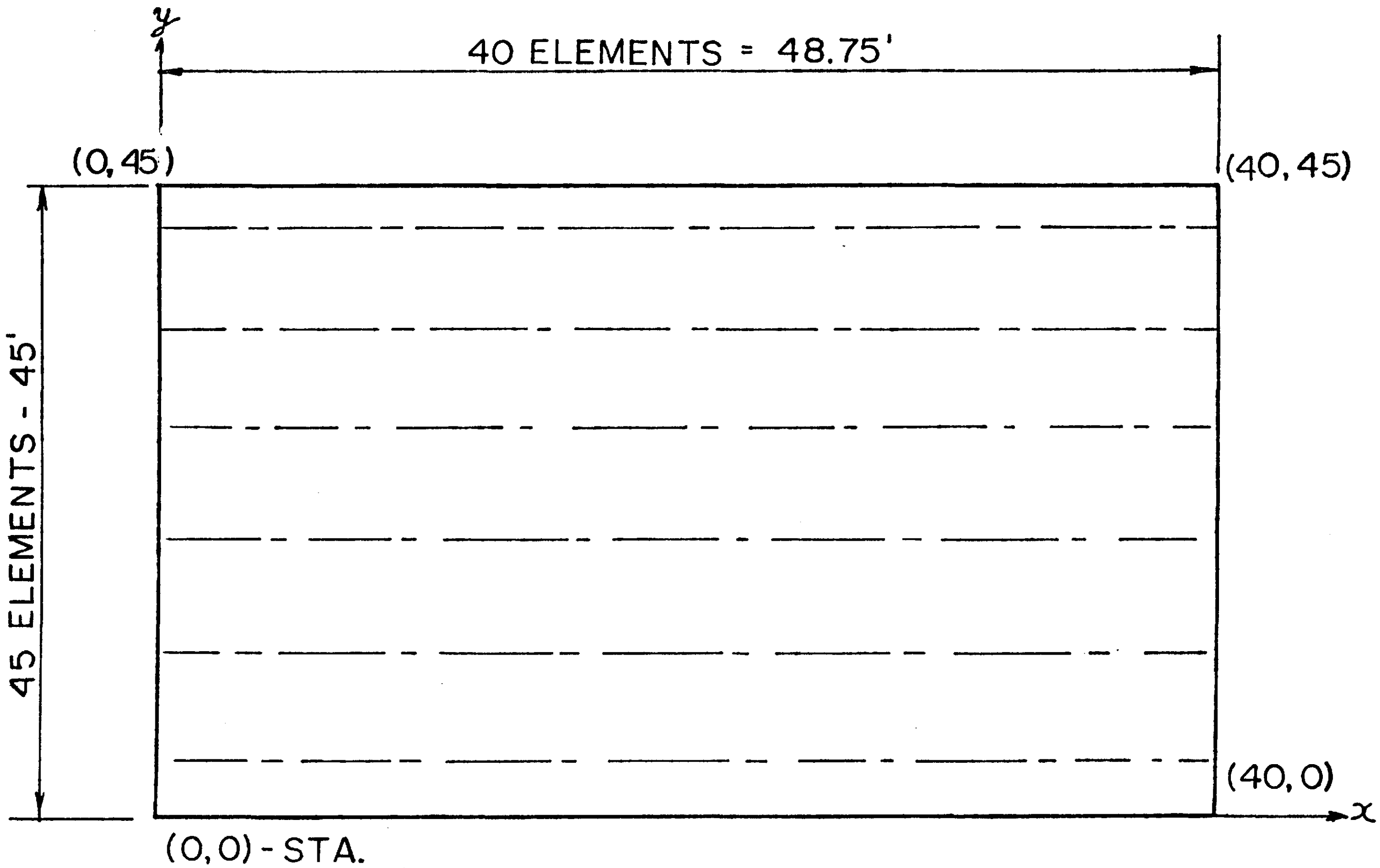


Figure 23. Finite Element Layout of Bridge Span.

ft in the longitudinal direction. The actual span length between the center of the bearing supports is 48 ft-9 in.

The input parameters, besides the usual geometry, connectivity and loadings, included the stiffnesses of the composite beams, bracing system, and the bending and twisting stiffnesses of the slab. All values were calculated assuming elastic moduli of 29,000 ksi for steel, and 3,900 ksi for concrete. The beam stiffnesses arising from the sectional properties shown in Table 4 were obtained by multiplying the composite moments of inertia by 29,000 ksi and subtracting the plate bending stiffness of the slab.

Determination of Support Spring Modulus

All I-beams were seated on bearing pads measuring 11 by 10 by 2-1/2 in. thick (see Figure 24). Because of these pads, it is very clear that, for modelling purposes, the supports cannot be assumed rigid. Therefore, an estimation of the spring moduli of the supports was done using the deflections measured at the south support of Beam 4 during loadings LON and LOS.

First, the readings taken from dial gage 12 (see Table 2) are considered. These reading are 0.0008 in. and 0.0019 in. for loadings LON and LOS, respectively.

Second, the reactions of the south supports during loading LON and LOS are estimated (see Figures 19 and 20). It is assumed that the reaction equals the wheel load right on top plus a simply-supported contribution of the other wheel load. These reactions are as follows:

Loading LON,

Reaction = Wheel Load X [1 + (1 - Wheel Spacing)/Beam Spacing)]

R = 3410 lbs (2 - 6.5/8) = 4,050 lbs

Loading LOS,

R = 9560 lbs (2 - 6.5/8) = 11,350 lbs

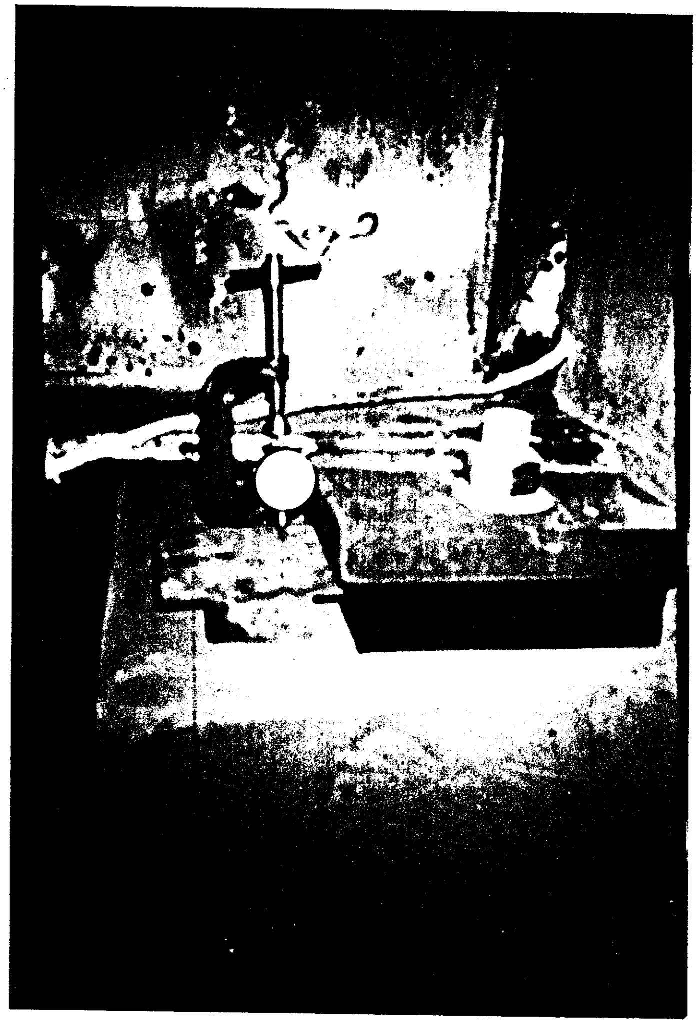


Figure 24. Neoprene Bearing Pad for Beam Supports.

Then, dividing the first reaction by 0.0008 in. and the second by 0.0019 in. and averaging the two values, a support spring modulus of 5,500 k/in. is calculated. This value is assumed representative of all beam supports.

Measured versus Computed Deflections

The measured deflections are compared to the deflection profiles of all six beams as computed from SLAB49 for loadings L2N, L4N, and L6N. These loadings are described in Table 1 and in Figures 20 through 22. The slab bending and twisting stiffness were calculated assuming an 8 in. nominal thickness, a Poisson's ratio of 0.2, and modulus of 3,900 ksi for the concrete. The beam bending stiffnesses were computed by multiplying the composite moments of inertia (see Table 4) times the steel modulus, and subtracting the longitudinal slab bending stiffness. The bending stiffnesses of the diaphragm beams were also considered and were obtained by multiplying the nominal moments of inertia times 29,000. All supports were assumed nonrigid with a spring modulus of 5,500 k/in. The station coordinates for the four loadings were then determined. All these parameters were entered in the program and the deflections computed.

The computed deflections were then graphed and compared to the deflection measurements shown in Table 3. Figure 25 illustrates the measured versus the computed deflections for loading L2N. In this loading, the left wheels of the gravel truck were placed on top of Beam 2. From this graph, it is seen that measured deflections are a little smaller than the computed profiles. In general a very good correlation was obtained for the deflected beams. Figure 26 shows the comparison for the deflections measured when the truck was placed on top of Beam 4. Again, the predicted and computed correlate well. The measured midspan deflection of Beam 4 is a little larger than the computed values. Figure 27 shows the correlation of deflections measured during loading L6N and the computed profiles. It is also seen that both computed and measured values are in good agreement. However, the measured midspan deflection at Beam 4 is, again, a little larger.

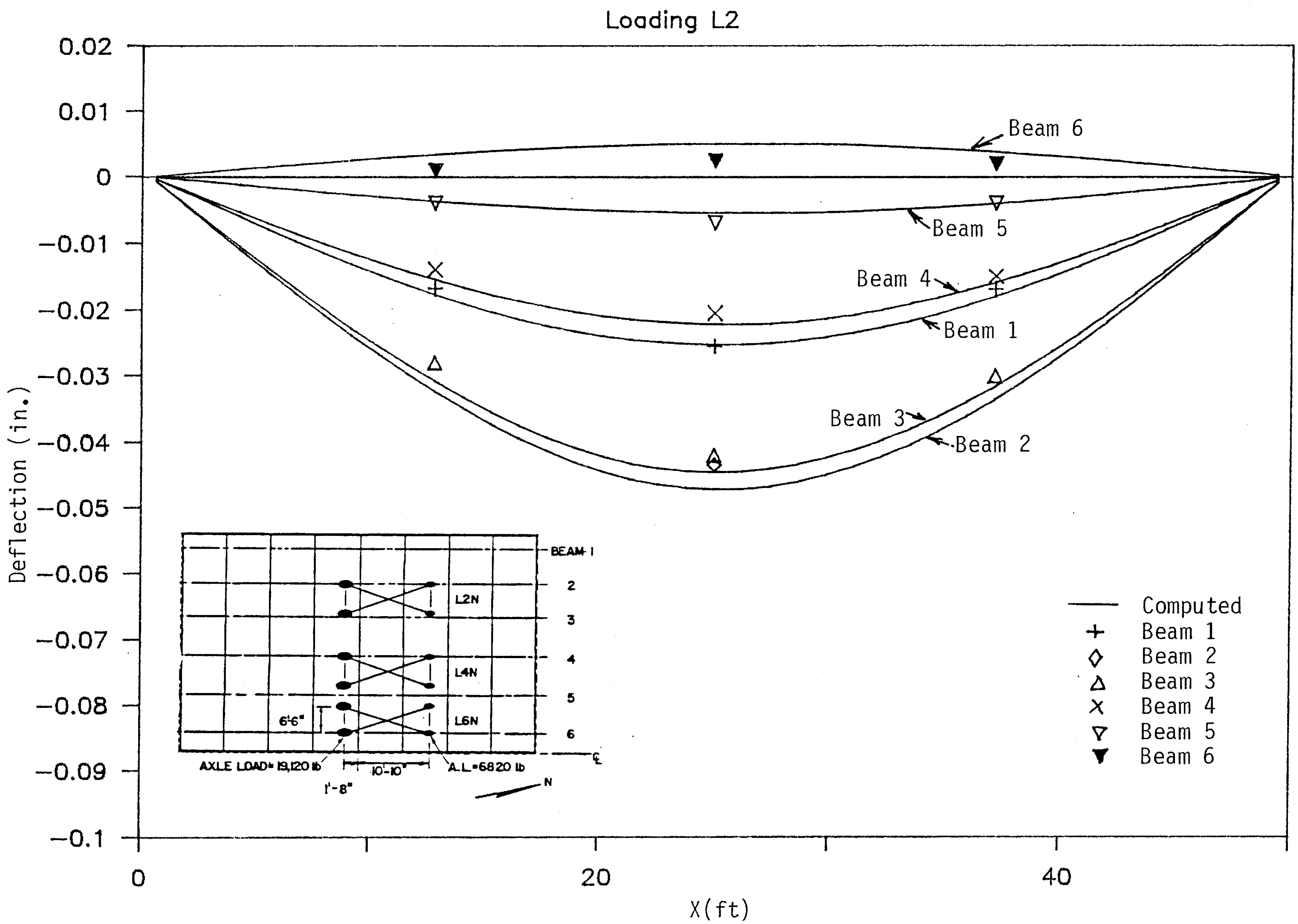


Figure 25. Measured Versus Computed Deflections for Loading L2N.

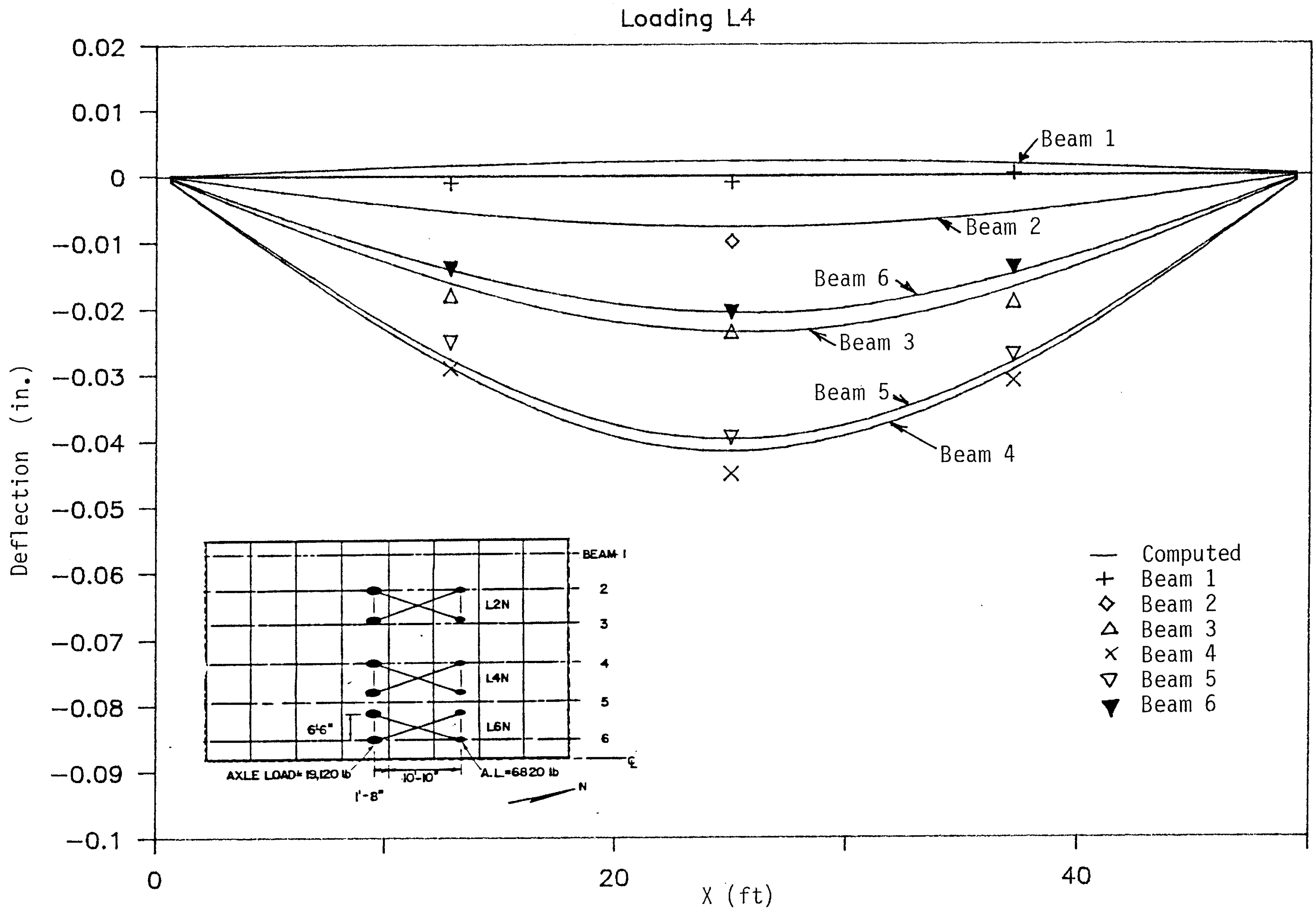


Figure 26. Measured Versus Computed Deflections for Loading L4N.

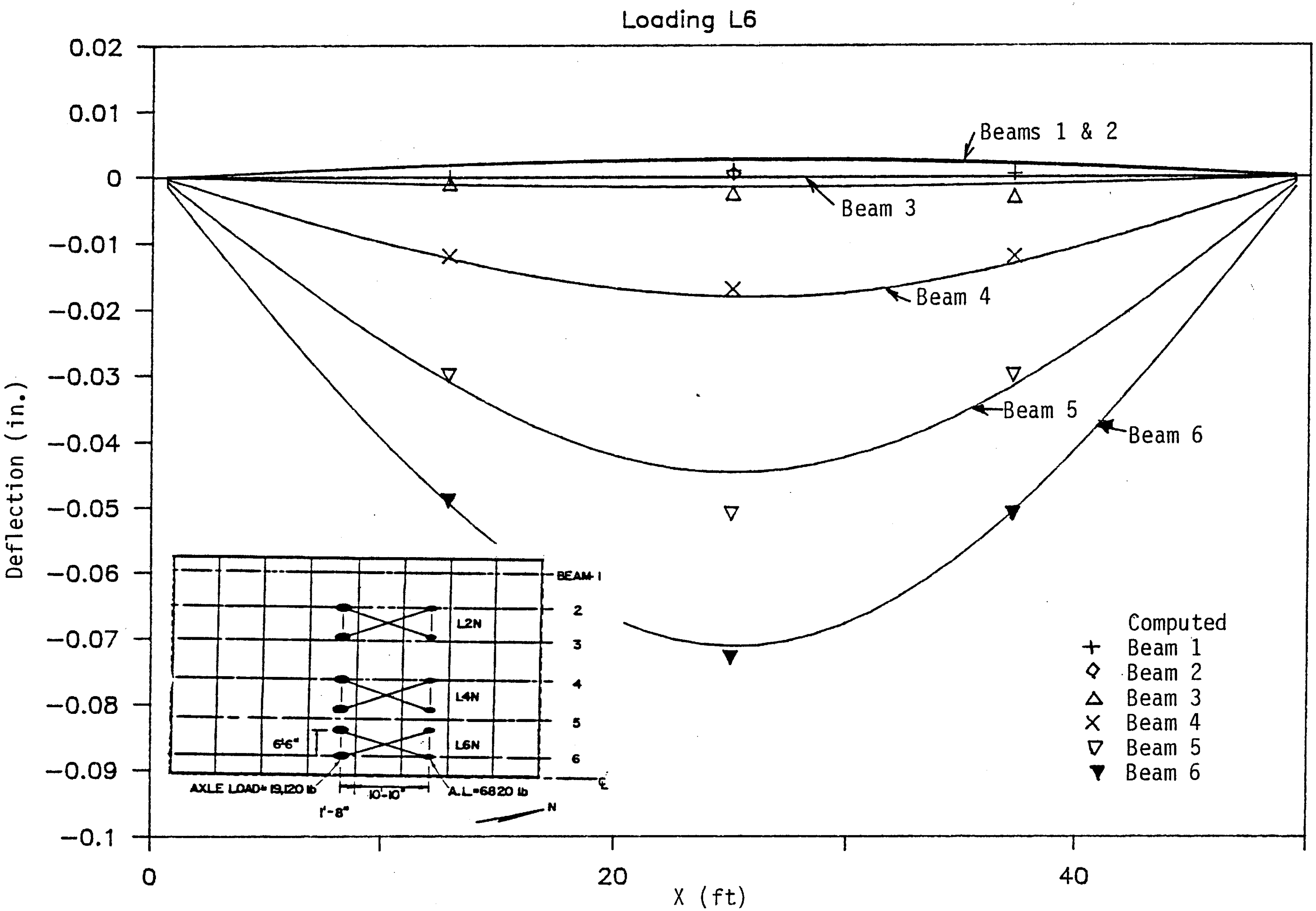


Figure 27. Measured Versus Computed Deflections for Loading L6N.

Table 5 illustrates a comparison of the maximum measured deflections with the maximum computed deflections of the full and non-composite models of the span. On the average, the analytical maximum non-composite deflections are about 140 percent larger than the maximum measured values. The maximum computed full-composite deflections are within 7 percent of the maximum measured deflections.

Table 5. Comparison of Maximum Measured and Computed Deflections.

Loading	Beam	Maximum Deflection (inches)		
		Measured	Computed	
			Full-Composite	Non-Composite
L2N	2	0.044	0.047	0.118
L4N	4	0.045	0.042	0.094
L6N	6	0.073	0.072	0.181

Summary

In this chapter, the measured deflections were compared to deflection profiles calculated with a finite element program. Good correlations were obtained for each of the loadings. Therefore, the agreement of the measured deflections with deflection profiles of the full-composite model and the comparison of the maximum measured values with those of the full- and non-composite models provide evidence that the span behaves in a full-composite manner.

CHAPTER VI

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

With the increasing problems of bridge deck deterioration, many concrete decks are being, or will be, replaced. One replacement method that has proven to be fast and to minimize traffic interference is the use of full-depth precast concrete panels. The panels are cast on the site or in a casting yard. The old deck is removed and the panels are placed and connected to the supporting stringers.

Full-depth precast concrete panels placed on simple span bridges can be connected to develop composite action. To assure the horizontal shear force transfer, the modular panels are connected to steel I-beams using epoxy mortar grout and standard shear stud connectors. Epoxy mortar-grouted key ways are employed for the transfer of compressive normal forces between adjacent panels.

Current construction methods used to develop composite action appear to be adequate for a fast reconstruction. First, the deteriorated deck and existing shear connectors are removed such that the top flanges of the I-beam stringers are left bare. The modular panels are then placed on the I-beam floor system with or without bearing pads or strips. Stud shear connectors are then welded to the top flanges through openings in the concrete panels. Gaps between the panels and stringers are sealed and subsequently grouted with epoxy or polymer mortar. Finally, the molded openings and the key ways between adjacent panels are also grouted. After the grout has cured, the bond at the panel-stringer interfaces, the shear connectors, and the key way connections make the steel I-beams and the modular precast deck act as an integral unit in resisting traffic loads.

This study represented the final phase of a research project for the implementation of full-depth precast concrete panels for the rapid replacement of deteriorated bridge decks. It involved the documentation of the reconstruction and load testing of a 50-ft simple bridge span redecked using full-depth precast concrete panels.

Conclusions

The conclusions of this study can be divided into two categories, those for the construction procedures and others for the full-scale load test results.

From the construction procedures it is concluded that this replacement method is fast and, if it is repeatedly implemented, it can be cost-effective. Practically, the major savings will be realized due to the shortened construction time. In the SPUR 326 bridge, eight panels were installed and grouted in a period of 25 nonconsecutive hours. If another 24 hours had been allowed for the complete curing of the epoxy mortar (depending on the set time of the material), this bridge would have been operational in about 2 days after the installation of the panels began. This time savings can be significant when compared to a cast-in-place replacement method. In this particular job, the total installation time could have been made even shorter had more effective sealing and grouting procedures been followed. Better equipment (for example a high-volume epoxy mortar mixer) and improved materials (for example an epoxy barrier that can be more conveniently installed) are certain to follow once the method is in more widespread use.

The bearing details of the panels using 2 by 6 in. neoprene pads proved to be adequate, but some difficulties were found because the pads were shared by two adjacent panels. It is suggested that bearing details in future projects include independent panel bearings. These pads were designed to assure equal distribution of bearing loads at selected points such that unwanted tensile stresses in the panels be minimized. Apparently, the pads worked well because no significant cracking was observed.

The main conclusion reached from the full-scale load tests is that the redecked SPUR 326 bridge behaves in a full-composite manner. The conclusion was reached by comparing the measured and theoretical deflection values. The theoretical deflections were obtained by using a finite element model of the bridge that contained the full-composite properties of the longitudinal stringers. The measured values are almost identical to the computed values.

Design Recommendations

From the work on experimental models as reported in Refs. [5,6,7,8], some design recommendations can be made. These are as follows:

1) The design loads can be assumed to be resisted by the properties of the transformed steel-concrete composite sections if the concrete slab is always subjected to compressive stresses and if shear connections are adequate.
2) It is recommended that composite sectional properties in negative moment regions not be considered, not even composite properties due to the tensile reinforcement because of discontinuities across adjacent panels.
3) The design of the shear connectors for the ultimate shear capacity can be made in accordance with the current AASHTO bridge specifications [1]. However, the AASHTO formula for the ultimate shear capacity of stud connectors must be as follows:

   $$V_c = 0.4\, d^2 \sqrt{E_e\, f_e}$$

   where,

   d = diameter of the stud connector,

   E_e = the modulus of the embedding material, and

   f_e = the ultimate compressive strength of the embedding material.

   In the formula above, the embedding material is referred as the epoxy mortar inmediately surrounding the shear connector. This formula proved to be conservative against actual capacity values obtained from push-out tests [8]. When the properties of the embedding material are not known, judgement must be employed by the designer in estimating these parameters.
4) The design of shear stud connectors for fatigue can be conservatively made in accordance with the current AASHTO

specifications[1]. However, the designer must be aware that the actual composite shear transfer is conducted by the adhesive bond of the adhesive interface material. The shear connector will not carry any significant load unless the bond fails. In fact, the presence of epoxy mortar bond at the deck-stringer interface prolongs the fatigue life of the shear connectors as reported in Ref. [7].

5) The block-out openings in the precast panels must be large enough to accommodate the welding of the shear connectors. A minimum width of 5 in. appears to be adequate for 7/8 in. studs. However, it is suggested that stud manufacturers be contacted for the specific studs to be used.

Inspection Recommendations

It is further recommended that the precast decked bridge span of the SPUR 326 be regularly inspected to investigate the long-term environmental effects on the connections of the panels to the stringers. Inspection procedures must include a visual inspection of the concrete deck to ascertain the severity of the cracking and compare it to similar cracking in the cast-in-place deck of the mirror bridge. Other critical areas that need to be routinely inspected include the vicinity of the grouted block-outs in the panels, and the bond of the epoxy grout between the beams and the precast deck. It is important to document if and when cracking or debonding of the epoxy mortar occurs.

Research Recommendations

One final recommendation that can be made is to suggest that load tests similar to those described in this report be made at regular intervals, say every five years, to monitor changes in the structural properties of the bridge due to time.

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