COUNTY BRIDGE DECK REPLACEMENT WITH TRANSVERSE PRECAST CONCRETE PANELS

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Submitted to the Faculty of the
Graduate College of the
Oklahoma State University
in partial fulfillment of
the requirements for
the Degree of
DOCTOR OF PHILOSOPHY
May, 1991

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ACKNOWLEDGEMENTS

I wish to express my appreciation to Dr. Garold Oberlender for his support, encouragement, and advice throughout this research project and for serving as chairman of my advisory committee. I also express appreciation to Dr. Samir Ahmed, Dr. Robert Hughes, Dr. James Shamblin, and Dr. Farrel Zwerneman for their advice and for serving as members of my advisory committee.

I thank the Oklahoma Department of Transportation for their participation in the project by providing bridge data, reviewing design calculations, assisting in field work, and for providing partial funding. I thank the Center for Local Government Technology at Oklahoma State University for providing partial funding. I express sincere gratitude to Mr. Laverne "Buck" Rogers, Noble County District 3 Commissioner for providing the material, equipment, and labor to construct a demonstration bridge.

I wish to express my sincere appreciation to my parents, Frank and Evelyn for unending support, encouragement, and guidance. Finally, I would like to express sincere appreciation and many thanks to Linda for her support, patience, humor, meals, typing, editing, and advice throughout the course of this research project.

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INTRODUCTION

Background

Counties in Oklahoma have an increasing economic problem of maintaining, repairing, and replacing over 15,000 county bridges, many of which are in an advanced state of deterioration. Heavy trucks with excessive loads cause structural damage and wear to some bridges, however all bridges deteriorate due to the effects of weather and surface wear from vehicles. Bridge decks incur more deterioration from weather and wear than any other component of a bridge.

Counties often operate on limited budgets, therefore they have need of a self-supported program of bridge maintenance, repair, and replacement. This research project developed a system of precast concrete panels emphasizing efficient design and construction methods for use in deck replacement of county bridges. The system is designed to be administered and implemented primarily by county work crews.

Scope and Purpose

This report contains the results of a research study to develop a precast concrete deck replacement system for county bridges. The system was developed to provide an alternate method of deck replacement for counties to utilize on bridges on the county road system. The panels were designed to act independently after placement on the bridge without requirements of any panel-to-panel connections for load transfer. Construction is greatly simplified by requiring no on-site casting of concrete or use of grout. Counties can precast and stockpile the panels at county facilities during the winter months, and then place them on bridges during months of warm weather.

An analysis and design of the deck panels were performed in accordance with American Association of State Highway and Transportation Officials (AASHTO) standards. Design drawings and a construction procedure were developed for implementation by county work forces. An actual test of the proposed system was performed by the construction of a demonstration bridge which utilized material, equipment, and personnel from a county. Success of the proposed system was demonstrated by the construction, transport, and placement of the bridge deck panels with the county personnel having no major problems.

Chapter II presents the selection of bridges used for data analysis in this study; Chapter III presents the configuration, analysis and design of the deck panels. The construction of the demonstration bridge is presented in Chapter IV. Comparison of the precast concrete panel deck and a cast-in-place deck is given in Chapter V. Chapter VI presents the compilation and analysis of a mail survey of the counties in Oklahoma to ascertain the abilities of county work crews to implement this deck replacement method. Chapter VII presents a summary, conclusions, and recommendations for further research. A complete set of structural calculations for the panels is presented in Appendix A. Deck panel drawings, with details, are shown in Appendix B. The survey packet mailed to each of the seventy-seven Boards of County Commissioners is shown in Appendix C.

This research project is limited to an evaluation and study of the bridge decking and the interface of the bridge decking with the support beams. Since replacement of the deck on a bridge can affect the load carrying capacity of the beams, substructure, and foundation, a registered professional engineer should perform a thorough site investigation and engineering analysis of each specific bridge before deck replacement. This research project does not address the substructure and foundation.

Previous Work

In the late 1960's, precast concrete bridge decks were used as an approach to reduce the problem of deterioration of concrete bridge decks in Indiana [12]. The concept was that higher quality concrete could be produced by precasting bridge deck panels in a controlled environment (rather than casting bridge decks in the field), and that this higher quality concrete would result in less deterioration.

One of the early bridge deck replacement projects utilizing precast panels was in 1970 on a bridge near Bloomington, Indiana [12]. The precast deck consisted of panels with a minimum thickness of 6 in., at least 4 ft long (parallel to traffic), and a width of the transverse direction of the bridge. The panels were prestressed in their long direction to maintain compressive stresses under full design load. The panels were placed on steel beams with their long direction transverse to the bridge. After placement and alignment of all panels, the entire deck was post-tensioned in the longitudinal direction of the bridge to provide load transfer between panels. The deck was then connected to the beams using spring clips and bolts screwed into preset anchors located in the bottom of the panel. The joints between panels were tongue and groove with a thin neoprene sheet to seal the joint and to help minimize stress concentrations due to the post-tensioning. At the time of construction, the irregularities of the tongue and groove joint created stress concentrations at some locations which resulted in spalling at the joint. Also, water was able to penetrate the joints at some locations. However, after eleven years of performance these minor problems had not progressed and the deck was performing very well.

Precast concrete panels were used for deck replacement on a 1,627 ft long bridge on the Pennsylvania Turnpike [13]. The use of precast panels increased project safety by minimizing the number of personnel required on the 140 ft high structure on which traffic was maintained. The panels were cast off-site and were 7 ft 6 in. long (parallel to traffic), $6^3/_4$ in. thick, and 28 ft 8 in. wide. The panels were set directly into position on a grout bed of epoxy mortar on the top flange of the beam and connected with bolted spring clips. The joints consisted of female keyways on each panel which were filled with epoxy mortar to connect the panels together. A wearing surface was applied to the panels which consisted of $1^1/_4$ in. latex modified concrete.

During the mid and late 1970's, the New York Thruway Authority developed and implemented a system of precast concrete panels which were placed transversely to the bridge with composite action between the panels and supporting steel beams [15]. The panels were 4 to 5 ft in length (parallel to traffic), $7^1/_2$ or 8 in. thick, with width as required up to 40 ft. The panels were positioned on a bed of epoxy mortar placed on top of the beam flanges. The panels were connected together on the bridge by female keyways on the transverse edges which were filled with epoxy mortar. Composite action was achieved with full depth blockouts in the panels which were located directly above the beams on 15 in. centers. After the panels were in position on the bridge, two shear stud connectors were welded to the steel beams through the blockout. The blockouts were then filled with epoxy mortar. Moisture protection and vertical roadway profile were attained by application of a waterproof membrane and asphalt concrete pavement on the precast panels. Difficulty was experienced in controlling voids which appeared in the epoxy mortar bedding material placed on the beam flanges. Neoprene edge strips were later recommended to support the panels and contain the epoxy mortar. Cold weather hampered scheduling the placement of the epoxy mortar.

Research by the University of Virginia Civil Engineering Department [14] presented a system of placing precast panels on steel beams. The panels were 6 to 8 ft in length (parallel to traffic) with width as needed. The panels were placed 12 to 18 in. apart with reinforcing steel protruding between adjacent panels. A site-cast concrete closure was required to fill the spaces and tie the panels together.

The Thompkins County Public Works Garage [5] in Ithica, New York, produced transverse concrete panels for replacement of decks on existing bridges. The panels were designed for non-composite action with the supporting beams. The panels were cast on a concrete slab inside the garage. The dimensions were 4 ft in length (parallel to traffic) by 28 ft in width with a varying thickness from 6 in. at the 4 ft edge to 9 in. at the centerline. The varying thickness produced a 3 in. crown in the bridge deck. The panels were connected together by a tongue and groove joint requiring on-site use of grout. The panels were welded to the steel beams by an embedded steel plate that was cast into the bottom side of the panel. Epoxy-coated reinforcing steel was used for corrosion protection. A waterproof membrane was applied to the top surface of the concrete panels after placement on the bridge. A $1^1/_2$ in. asphalt overlay provided the wearing surface. The traffic rail posts were connected to bolts embedded in the panels. The panels were designed for AASHTO HS20 loading, Grade 60 reinforcing steel, and 4,000 psi concrete.

Grady County [6], Oklahoma, produced similar panels in 1987 to replace the timber deck of an existing bridge. The panels were cast on soil inside a county arena. The dimensions of the panels were 4 ft in length (parallel to traffic) with a 26 ft width and a 7 in. thickness. No crown was provided in the bridge deck. The panels had a 7 in. steel channel embedded on the edges of the 4 ft dimension. The panels were placed on the bridge with a space of 1 in. between them. They were connected together by welding a steel plate across the space to the steel channels of adjacent panels. The entire deck was then connected to the steel beams by welding the top flange of the beams to an embedded plate cast into the bottom side of the panel. The traffic rail posts were welded to the 7 in. channel and to the exterior steel beam. There was no specific structural design for these panels. The non-composite steel beam section of the Oklahoma Department of Transportation's (ODOT) <u>County Bridge Standards</u> [8] was used to establish panel thickness and reinforcing requirements. These standards were developed for the HS20 design loading.

CHAPTER II

SELECTION OF BRIDGES FOR STUDY

Database of Oklahoma Bridges

The criteria for selection of bridges in this research study were developed jointly between the author and personnel from the Bridge Division of the Oklahoma Department of Transportation (ODOT). The particular bridges that were identified for this research were selected from the 15,666 county bridges that were contained in ODOT's Structural Inventory and Appraisal Records in 1989 for bridges in Oklahoma. These records were stored on ODOT's mainframe computer in Oklahoma City. The 15,666 records of bridges were downloaded from the ODOT mainframe computer in June, 1989 into an IBM PC on the Oklahoma State University campus for analysis purposes.

Selection Criteria

The design of the precast concrete deck panels contained in this report was developed based upon data that identified the geometry, materials, and structural capacity of selected study bridges used in this research project. For each bridge in the ODOT Structural Inventory and Appraisal Record system there are 90 items that identify

TABLE I

SELECTION CRITERIA FOR BRIDGES IN STUDY[a]

INVENTORY ITEM NUMBER	ITEM DESCRIPTION	VALUE
21	Custodian	3
24	Highway System	7 and 11
42	Type Service on Bridge	1
43	Structure Type	3 and 4
43a	Type of Design	02
59	Superstructure Condition	≥ 5
60	Substructure Condition	≥ 5
62	Culverts and Retaining Walls	N
66	Inventory Rating	≥ 10

[a]Reference [10]

information related to structural capacity, bridge geometry, condition, and usage. Nine of the 90 items were used to identify the study bridges that were used in this research. All county bridges studied in this research project meet the criteria shown in Table I as defined in the <u>Bridge Inspectors Guide for the Recording and Coding of Oklahoma's Bridges</u> [10]. The criteria shown identified 1,752 study bridges (11.2%) of the 15,666 county bridges which were used for existing county bridge data analysis. The following paragraphs present the criteria that were used to select the 1,752 study bridges from the 15,666 in the database.

The selected study bridges are maintained by the counties for use by motor vehicles. Culverts, retaining walls, and railroad, pedestrian, and utility bridges were excluded. This research was intended for the study of bridges on low volume roads; therefore, roads classified as either federal aid secondary roads under local jurisdiction or local rural roads were selected. Items 21, 24, 42, and 62 were used from the ODOT inventory system to identify the criteria in this paragraph.

The selection criteria considered only steel beam bridges having either simple or continuous spans and a multiple beam configuration for support of the deck. Bridges with timber, steel trusses, or older cast-in-place concrete beams were excluded since they are typically too structurally deficient to support a new concrete deck. Items 43 and 43a were used from the ODOT inventory system to identify criteria in this paragraph.

The inventory system defines the superstructure as all structural members, bearing devices, and any drainage systems. Based upon discussions with ODOT Bridge Division personnel, only bridges having a superstructure condition rating of 5 or greater on a scale of 0 to 9 were selected for study. Condition 5 describes the superstructure as "in generally fair condition - potential exists for minor rehabilitation" [10].

The substructure of a bridge includes all piers, abutments, piles, and footings. Bridges with a substructure condition rating of 5 or greater were selected for this study. This criterion was selected by ODOT Bridge Division personnel to match the condition of the superstructure. Condition 5 for the substructure is defined the same as Condition 5 for the superstructure. Items 59 and 60 were used from the ODOT inventory system to identify the criteria for condition ratings.

Load rating is an important criterion in the selection of the study bridges. In order for federal or state funds to be allocated for use in the reconstruction of roads with bridges that are to remain in place, the bridges must meet the criteria set forth in the American Association of State Highway and Transportation Officials (AASHTO) <u>A Policy on Geometric Design of Highways and Streets</u> [1]. The criteria require a minimum design capacity of H10 loading for Average Daily Traffic (ADT) values of 0-50. For all other ADT conditions the minimum is H15. The minimum design load rating for school buses has not been established in Oklahoma, however, a 10 ton load rating is often discussed. This research considered bridges that have an inventory rating of 10 tons or greater as a criterion for selection of study bridges. Item 66 of the ODOT inventory system was used to identify this criterion.

Figure 1 shows the inventory ratings for the 1,752 existing bridges selected for this study. Distribution of the 1,752 bridges by the eight divisions of ODOT is shown in Figure 2.

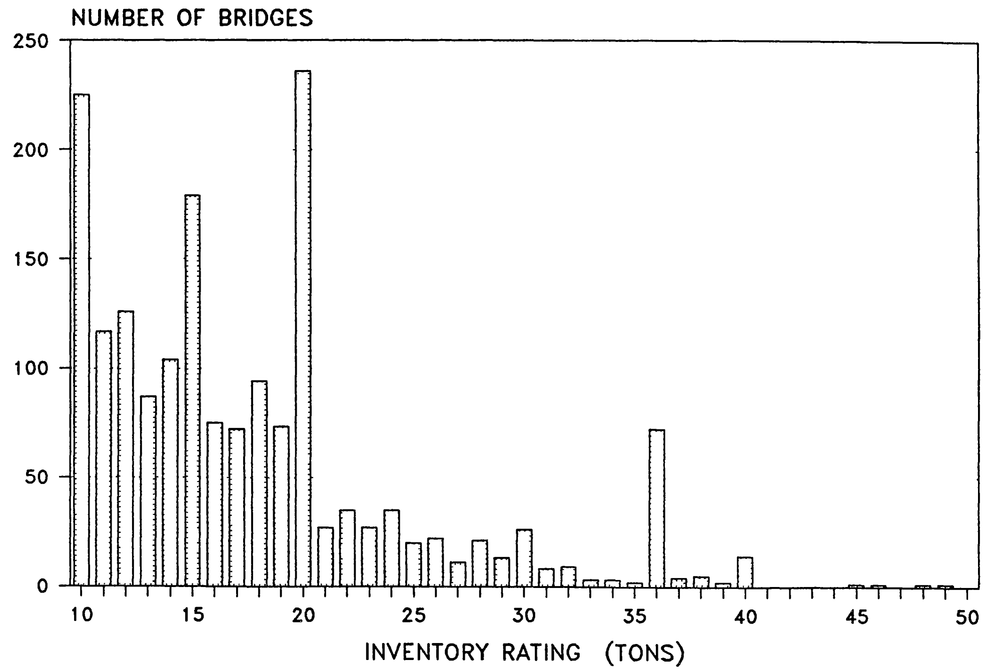


Figure 1. Inventory Ratings for the 1,752 Existing Bridges Selected for Study

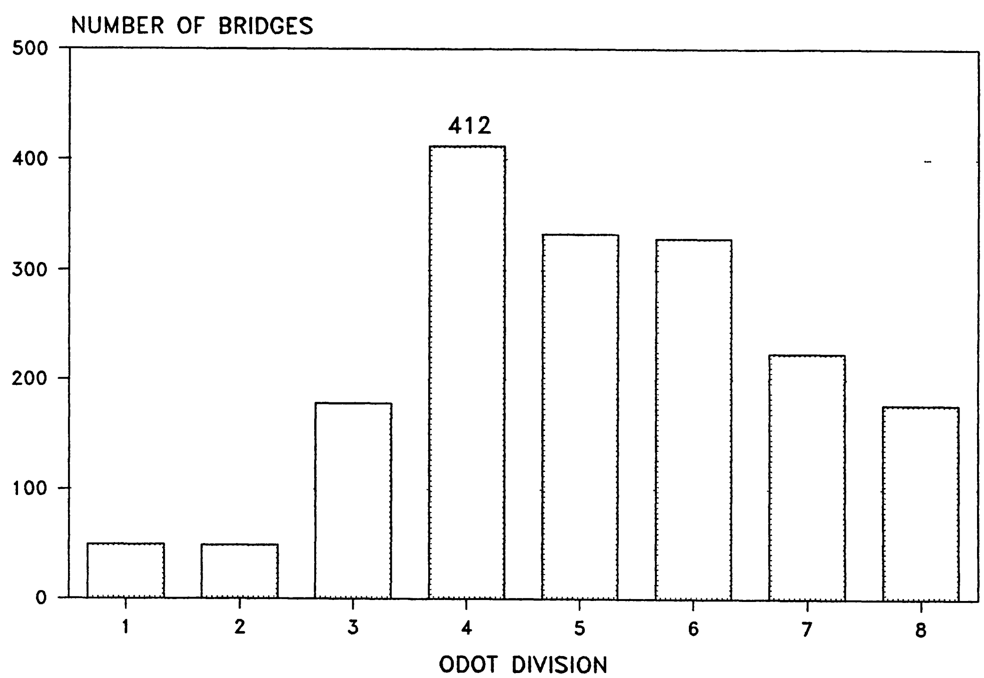


Figure 2. Distribution of the 1,752 Existing Study Bridges by the Eight Divisions of the Oklahoma Department of Transportation

CHAPTER III

DECK PANEL ANALYSIS AND DESIGN

Data From Existing Bridges Used in Design

Beam size and spacing are important factors for determining the load capacity of a bridge. In addition, the beam flange widths and spacings are required to design a concrete deck. These factors are not included in the data of the ODOT Structural Inventory and Appraisal Records. These data were collected at the ODOT Division 4 office in Perry, Oklahoma. Each bridge in an ODOT division office has a hard copy file which includes the beam size and spacing. Division 4 has 412 (24%) of the 1,752 study bridges used in this research project. From discussions with county commissioners and ODOT officials, it was concluded that the flange widths and beam spacings from the bridges in Division 4 would be representative of bridges across Oklahoma and would be used in the structural design of the precast concrete panels in this research project.

The distribution of flange widths for the selected 412 bridges in Division 4 is shown in Figures 3 and 4. From this distribution, a flange width of 6 in. was selected for use in the structural design of the panels. The

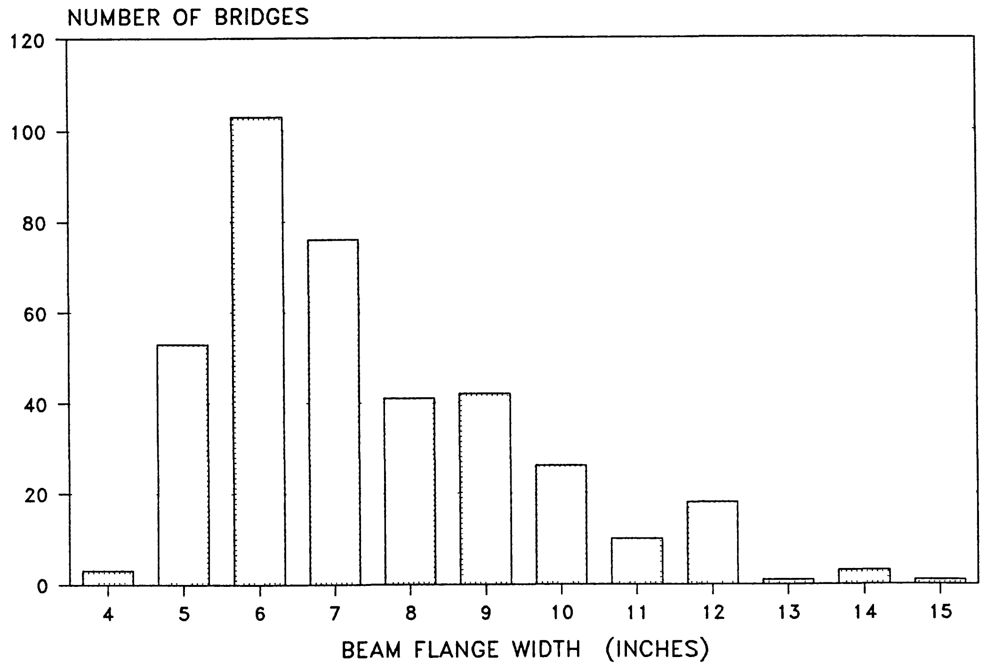


Figure 3. Distribution of Flange Widths for the 412 Study Bridges in ODOT Division 4 Rounded Up to the Nearest Inch

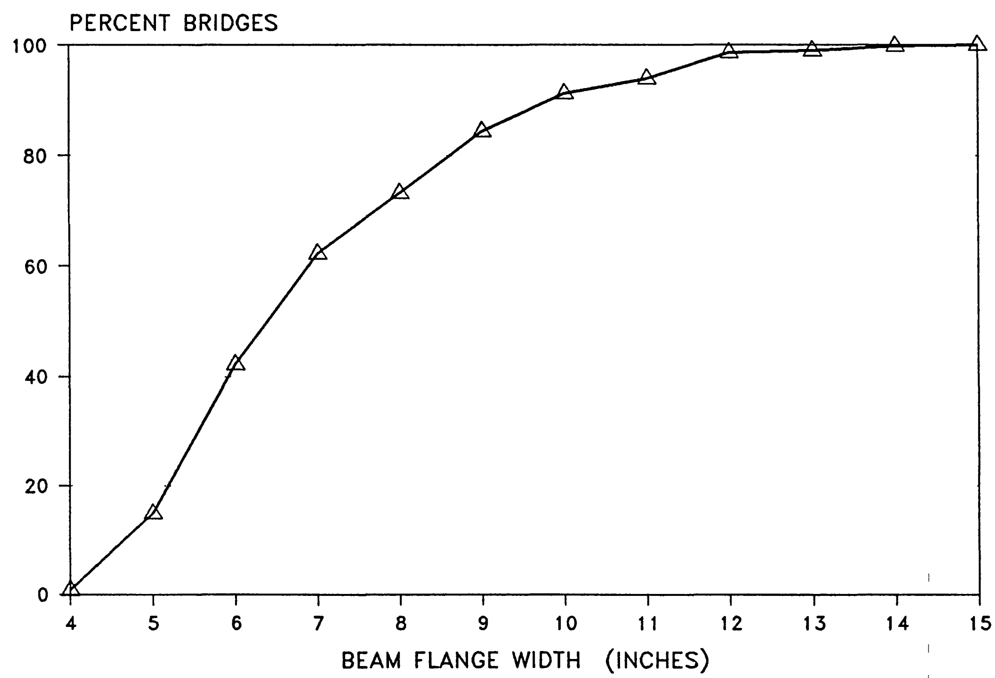


Figure 4. Cumulative Percentage of the 412 Study Bridges vs. Beam Flange Width

distribution of beam spacings is shown in Figures 5 and 6. It is shown that 98% of the 412 study bridges in Division 4 have a beam spacing of 5 ft 6 in. or less and 80% have spacings of 4 ft 0 in. or less.

The distribution of deck material for the 412 bridges of Division 4, reference Figure 7, shows that 75% have concrete decks and 25% have timber decks. A distribution of thicknesses of the concrete decks is shown in Figure 8. This figure shows that 80% of the concrete decks have thicknesses of 6 or 7 in. Deck replacement using the precast panels on existing county bridges with concrete decks which meet the selection criteria would have little or no additional dead load applied.

Panel Configuration

Panel length is defined as the dimension parallel to the direction of traffic and panel width is the dimension transverse to traffic. Clear roadway width of existing bridges is the governing factor for design widths of the panels. Figures 9 and 10 show clear roadway width rounded up to the nearest 2 ft for the 1,752 study bridges. It is shown that 97% of the bridges have clear roadway widths from 16 to 28 ft. A family of panels was established and designed to provide clear roadway widths of 16 ft 0 in. to 28 ft 0 in. on 2 ft 0 in. increments.

A fixed 4 ft 0 in. panel length was selected for all panel widths. For lengths greater than 4 ft 0 in., problems

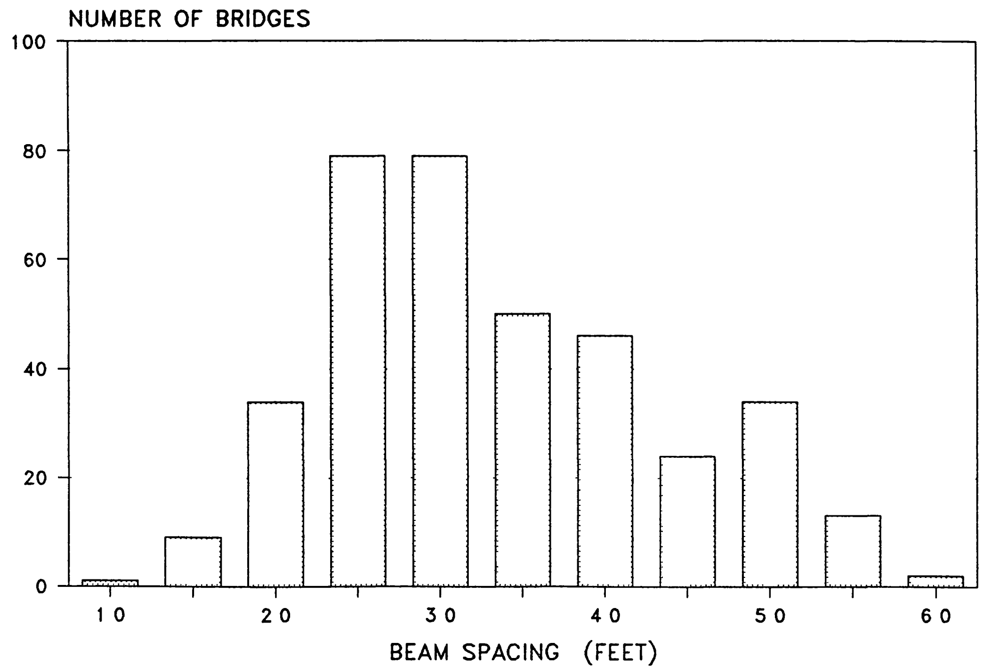


Figure 5. Distribution of Beam Spacings for the 412 Study Bridges in ODOT Division 4 Rounded Up to the Nearest Half Foot

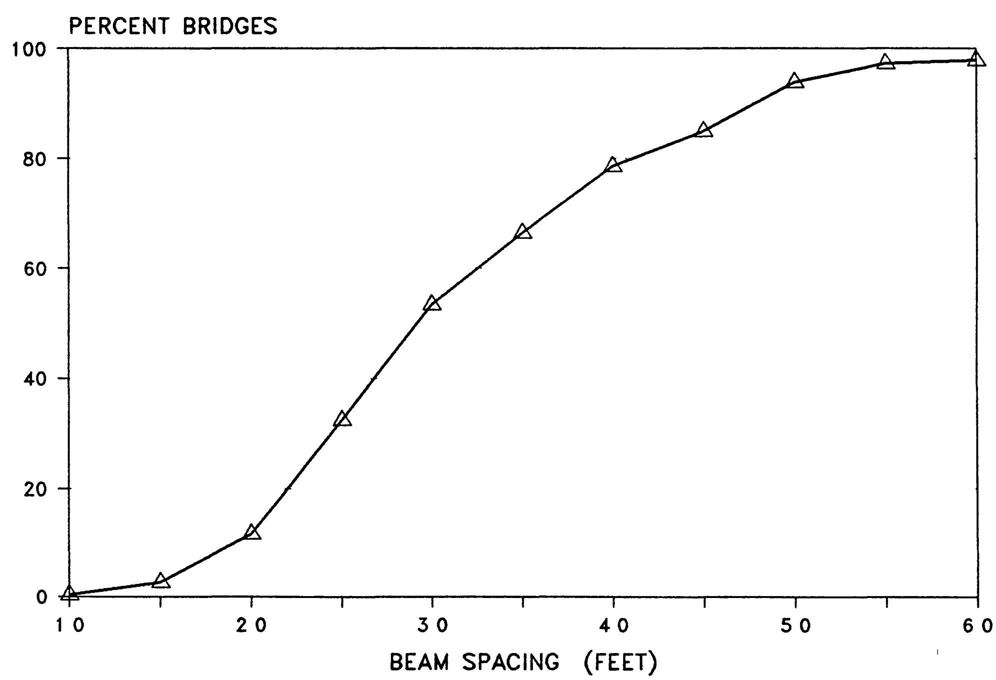


Figure 6. Cumulative Percentage of the 412 Study Bridges in ODOT Division 4 vs. Beam Spacing

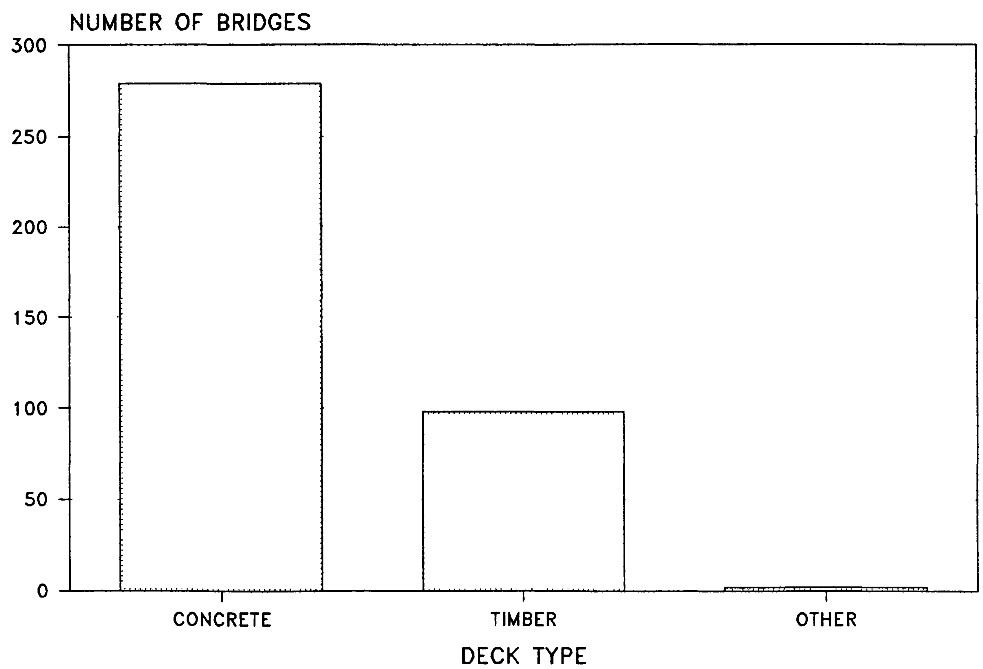


Figure 7. Bridge Deck Material for the 412 Study Bridges in ODOT Division 4

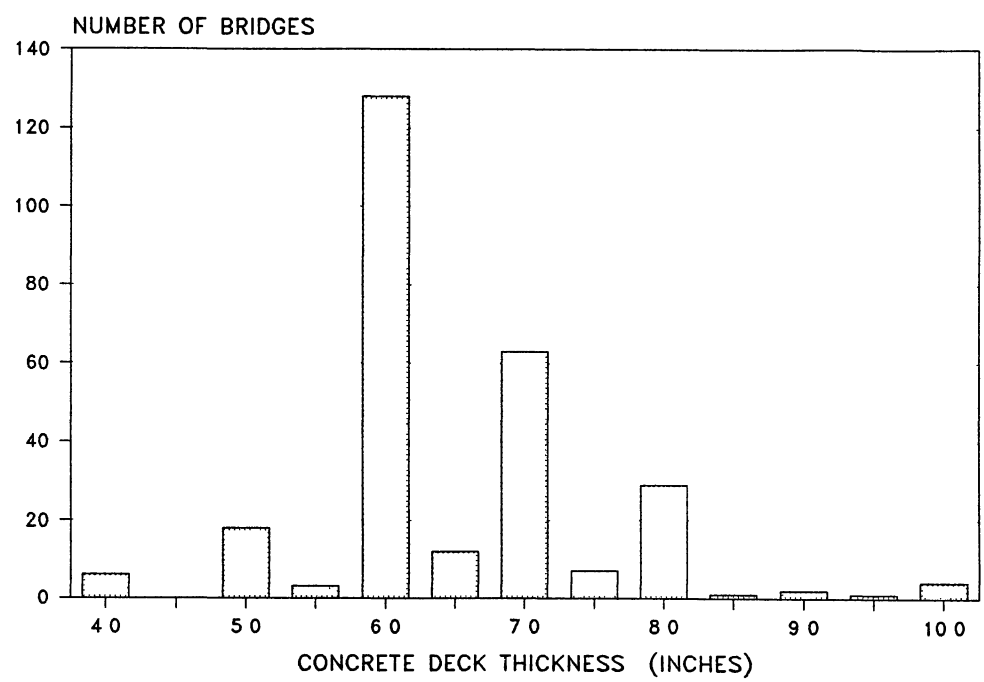


Figure 8. Concrete Deck Thickness for the 279 Study Bridges in ODOT Division 4 with Concrete Decks

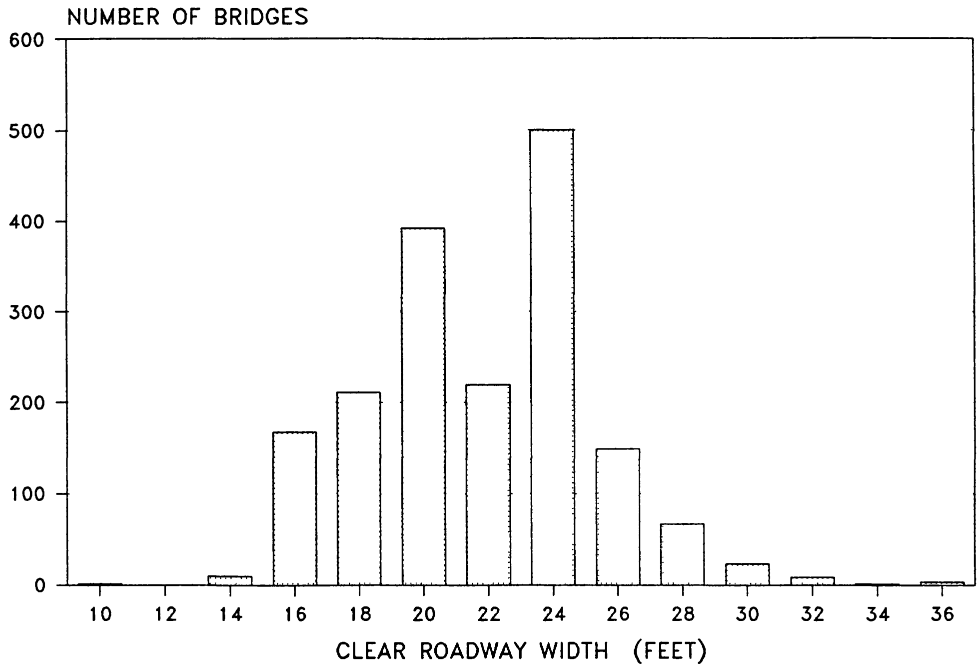


Figure 9. Clear Roadway Widths for the 1,752 Existing Bridges in Study

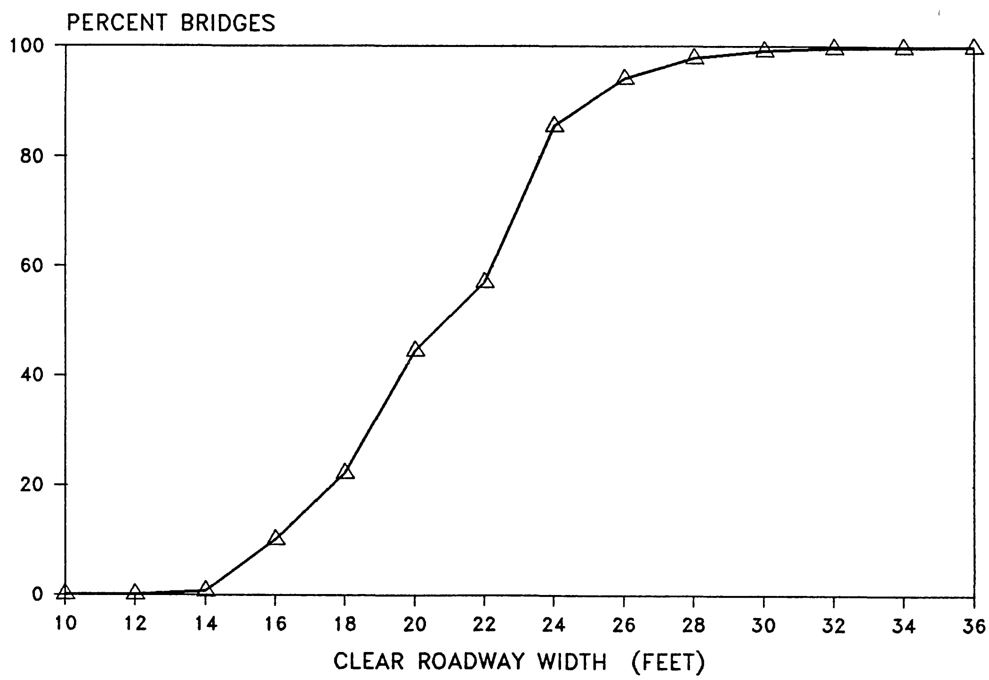


Figure 10. Cumulative Percentage of the 1,752 Existing Bridges in Study vs. Clear Roadway Width

in lifting and handling may develop due to increased weight. It is not economically efficient in terms of labor and material to construct panels less than 4 ft 0 in. in length.

Two thicknesses were established for the family of panels. A 6 in. thick panel was designed for use on bridges with a maximum beam spacing of 4 ft 0 in. A 7 in. panel was designed for bridges with maximum beam spacings of 5 ft 6 in. Both thicknesses allow for two mats of reinforcing steel. The 6 in. panel has a $1^1/_2$ in. clear cover of concrete over the top layer of reinforcing steel whereas the 7 in. panel has a 2 in. clear cover. The use of salt for deicing is not anticipated on most county bridges, therefore the $1^1/_2$ in. cover for the 6 in. panel is in compliance with AASHTO <u>Standard Specifications for Highway Bridges</u> [2]. If deicing salt is anticipated, the 7 in. panel should be used which provides a 2 in. cover.

For ease of construction the panels were designed to act independently, thus no connections are required between panels for load transfer after the panels are placed on the bridge. For compliance with AASHTO [2], support for the deck panel edge must be provided along the transverse joints. To satisfy this requirement a structural steel channel with a depth equivalent to the panel thickness was placed along each edge running the full width of the panel. The steel channels were designed to act compositely with an effective area of concrete to provide edge support for the panel. The channels were connected to the concrete with a series of $^3/_4$ in. dia. × 4 in. long headed studs.

The panels were designed for a $^3/_8$ in. gap between adjacent panels to allow weldment of a connection plate to tie the panels to the supporting beams. This $^3/_8$ in. gap allows drainage of surface water from the bridge deck.

Analysis of Panels

The panels were analyzed for bending moment using equations from AASHTO [2] and computerized structural analysis. Loads included the HS20 design truck and a 20 psf future wearing surface. The panels were analyzed in a three step process. The first step was analysis of the panel using the equations in AASHTO which provide bending moments per ft width for the panel. The second step involved a more exact analysis of the panel using tire contact area placed at locations along the edges of the panel to produce maximum positive and negative bending moments. The resulting moments were used to design the steel channel and effective portions of concrete along the edges of the panel. The third step considered the panel in a shimmed condition. If shims are required to provide bearing between the panel and beam, they are to be placed under the structural steel channels. The shims used should be 2 in. × 6 in. × required thickness. The bending moment due to a wheel load located in the center of the panel is resisted by a slab design in the 4 ft 0 in. direction of the panel. The three steps are discussed in

Appendix A. The maximum moment at any location of the panel produced by any of the steps was used in design.

Design of Panels

The concrete specified for the panels is class AA concrete as defined in the Oklahoma Department of Transportation's <u>Standard Specifications for Highway Construction</u> [9]. Class AA concrete has a specified minimum ultimate strength of 3,500 psi. Maximum aggregate size specified for the panels is 1 in. diameter. The reinforcing steel specified for the panels is plain reinforcing steel with a minimum yield strength of 60,000 psi (Grade 60). As previously stated, the use of deicing salts is not anticipated, but epoxy-coated reinforcing steel could be used if necessary. The structural steel channel is specified as A36 steel and should be painted with two coats of zinc rich paint to prevent corrosion.

The concrete portion of the panel was designed using Load Factor Design as described in AASHTO [2]. The steel channel was designed using Allowable Stress Design as described in AASHTO [2].

The design of the panel considered a 20 psf design load to allow for a future wearing surface. Before any wearing surface is applied, the $^3/_8$ in. gap between panels should be sealed to prevent cracking and spalling of the wearing surface.

The panels have been designed to interface with the TR-2 traffic rail as detailed in the ODOT County Bridge Standards [8]. Utilizing the TR-2 traffic rail allows complete construction of the rail from the deck surface. This saves labor and time by not requiring any scaffolding for traffic rail work.

Complete sets of structural calculations for the 6 in. and 7 in. panels are provided in Appendix A. A complete set of drawings for the 6 in. and 7 in. panels and accompanying detail sheets is shown in Appendix B.

CHAPTER IV

DEMONSTRATION BRIDGE

Agencies Involved

In the spring of 1990, a demonstration bridge was constructed to evaluate the constructability of the proposed deck replacement system developed in this research project. Several agencies were involved in the construction of the demonstration bridge. The bridge is located in Noble County, Oklahoma, on a county road over North Stillwater Creek where it flows into Lake McMurtry, reference Figure 11. The bridge is owned and maintained by Noble County. The Noble County District 3 commissioner supplied the material, equipment, labor, and general supervision for construction of the bridge. ODOT personnel were responsible for quality control and testing of the materials used in the deck panels. The author provided the plans for construction of the deck panels, details for the panel to steel beam connections, and general consultation during fabrication and installation of the panels.

Configuration of Existing Bridge

The existing bridge was originally constructed in 1919 as a one lane, three span bridge with a total length of

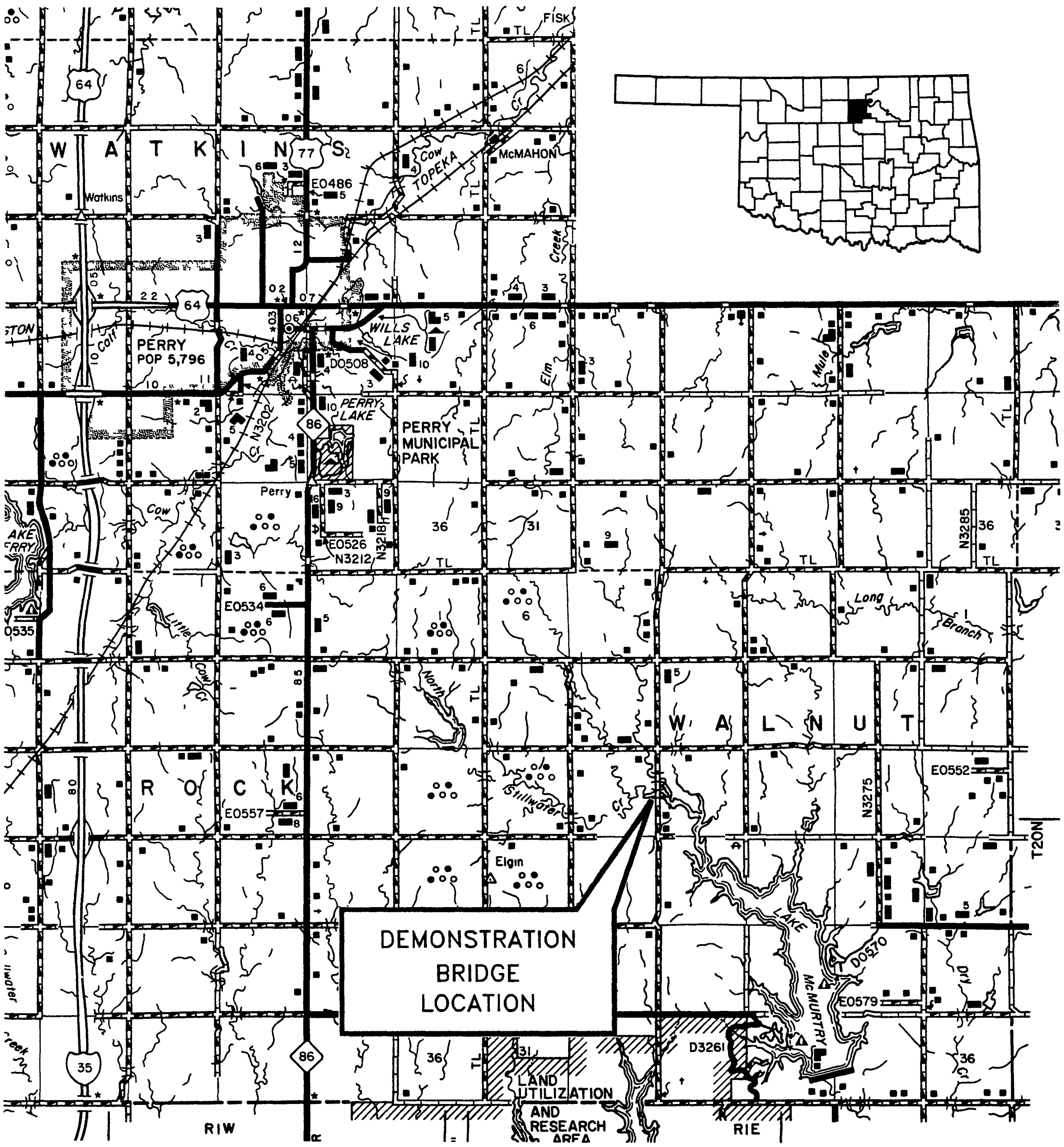


Figure 11. Location of Demonstration Bridge

44 ft 0 in., reference Figure 12. The deck had 3 × 12 in. timber planking with significant weathering and wear on the surface as shown in Figure 13. The age of the timber deck was not known. The bridge had no traffic rails. The two end spans utilized timber beams and the center span utilized steel beams. The bridge had timber abutments and piers founded on timber piles. The north abutment showed signs of buckling. The bridge was skewed to the road, crossing the channel at a right angle, creating a dangerous approach curve for night traffic or for those unfamiliar with the road. The road was approximately 20 ft wide with a dirt surface. The bridge was in very poor condition with a posted rating of 4 tons.

Configuration of New Bridge

The new demonstration bridge, which replaced the existing bridge, is a 28 ft 0 in. long single span steel beam bridge. The roadway was straightened and crosses the creek at a slight skew. The new bridge is shorter than the old bridge because of a change in hydraulic conditions due to the construction of Lake McMurtry. The new deck was raised 24 in. and the adjacent roadway was elevated by fill. The steel I-section beams were salvaged from a bridge replaced at another location. They are 24 in. deep with a 7 in. wide flange and are spaced on 4 ft 2 in. centers to support the precast deck panels. At each abutment, the steel beams were welded to a 10 in. deep cap beam supported

Figure 12. Existing Bridge Viewed from the South

Figure 13. Timber Decking of Existing Bridge Showing Significant Weathering and Wear

by HP10 × 42 steel foundation piling. Steel sheet piling retains the roadway fill. Piling for the abutment wingwalls are the 12 in. deep steel beams that were salvaged from the center span of the existing bridge.

The precast concrete deck panels each measure 3 ft $11^{5}/_{8}$ in. (parallel to traffic) × 28 ft 5 in. and are 7 in. thick. The clear roadway width was designed to be 26 ft 0 in. using the TR-2 traffic rail as detailed in the ODOT County Bridge Standards [8]. The custodian of the bridge, Noble County, chose to construct an alternate guardrail which provided a clear roadway width of 27 ft 10 in. An as-built cross section of the demonstration bridge is shown in Figure 14.

The precast panel bridge deck replacement system was developed for use on an in-place steel beam support structure. The demonstration bridge was constructed using steel beams from a salvaged bridge. Since used steel beams were utilized, the demonstration bridge simulates the condition of the proposed deck replacement of an existing bridge. This condition was verified by the fact that the beams on the demonstration bridge did not create a uniform level surface for connection of the precast concrete panels. Placement of shims was required to provide bearing for all panels on all beams. Shimming would most likely be required on a deck replacement project for an existing bridge.

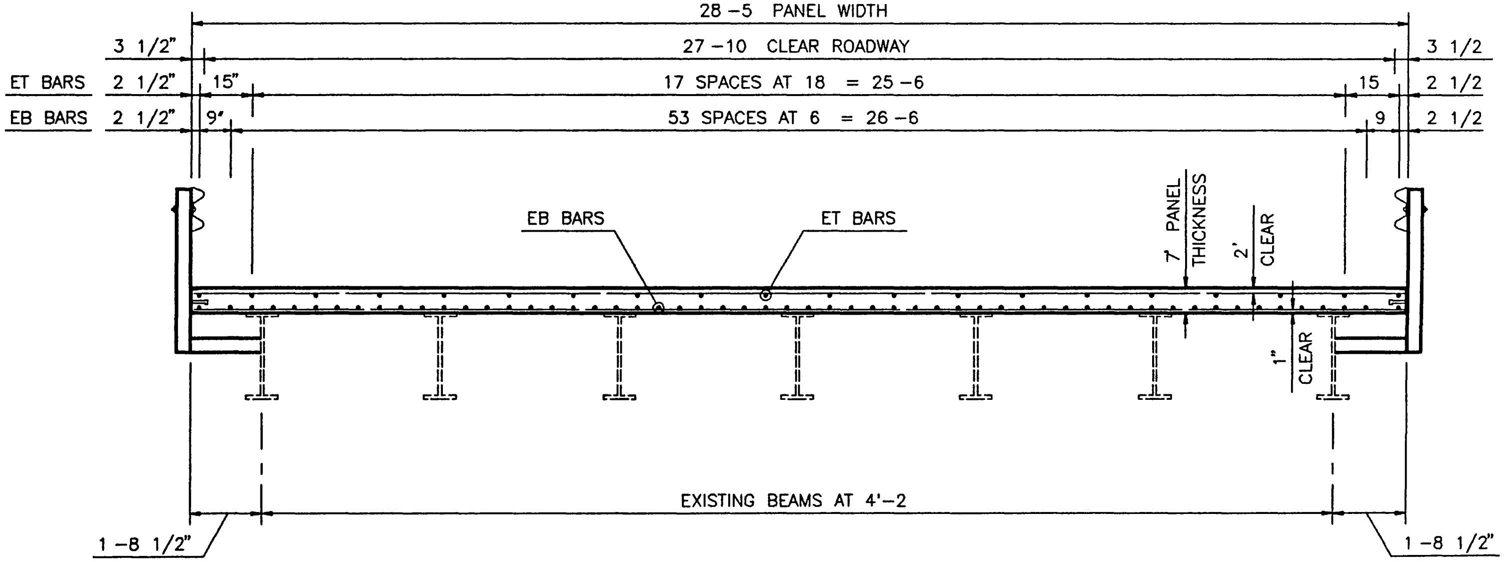


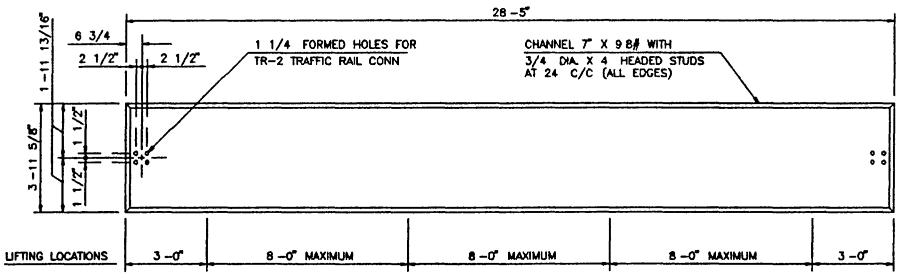
Figure 14. As-Built Cross Section of Demonstration Bridge

Construction of Deck Panels

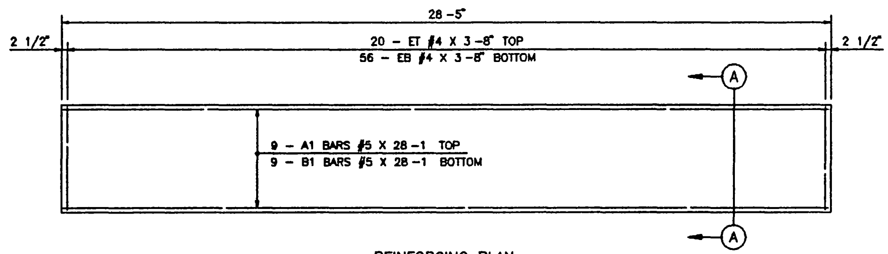
Preconstruction planning for the panels involved the selection and preparation of a suitable casting bed, orientation and placement of the panel frames for casting, and selection of the surface texture for the traffic surface of the panels. The panels for the demonstration bridge were constructed from the plans shown in Figure 15.

Three options were considered for the casting bed surface: concrete slab, wooden surface, and a soil surface. Existing concrete slabs are available at many county facilities, reference Chapter VI, however most are probably not suitable for a concrete casting bed because of irregularities in their surface. Although a wooden casting surface could easily be constructed, wooden forms are not conducive to a large amount of reuse. Casting on soil is available to all counties, therefore the seven panels of the demonstration bridge were cast on soil.

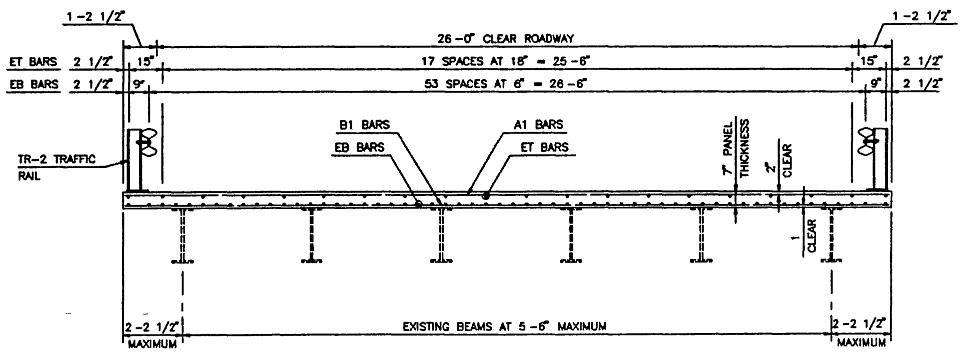
Since the panels were cast against soil, the casting orientation of the panels was traffic-side-down. This enabled finishing control of the beam contact surface of the panel which minimized the need for shimming the panels. There was concern that casting the panels traffic-side-up on a soil base would require an increased amount of shimming due to the uneven bottom surface. This concern was verified after the completed panels were inverted to a traffic-side-up orientation. The panel surface cast against the ground had several raised areas that would have significantly



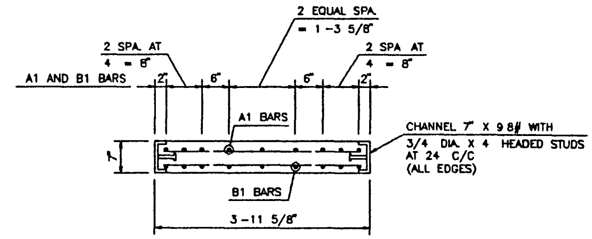
PANEL PLAN



REINFORCING PLAN



CROSS SECTION



SECTION A–A

PANEL BAR LIST

BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	28 -1
B1	#5	9	STR	28 -1
ET	#4	20	STR	3 -8"
EB	#4	56	STR	3 -8"

PANEL QUANTITIES

ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2.4
REINFORCING STEEL (GRADE 60)	LB	713
STRUCTURAL STEEL A36	LB	580

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO – 1989

	WSD	LFD
CONCRETE CLASS AA	———	F c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS.

7" UNIFORM PANEL THICKNESS 26 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P.S.I HAS BEEN REACHED

LIFTING NOTES

TOTAL PANEL WEIGHT = 11 380 LBS

PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 2 500 P.S.I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 26'–0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MARCH 1990		

Figure 15. Construction Plans Used for Demonstration Bridge Deck Panels

increased the amount of required shimming if the panels had been cast in a traffic-side-up orientation.

There are several materials that can be used for producing a texture on the driving surface of the panel when cast in a traffic-side-down orientation. Rock quarry screenings, coarse sand, and pea gravel were considered. A broom finish was considered if the panels had been constructed with the traffic surface as the upper side of the concrete panel.

The Noble County District 3 yard had an outside soil area large enough to cast the seven panels without stack casting. A grader was used to prepare a level soil surface. Rock quarry screenings were selected to produce traffic surface texture and a thin layer, approximately 2 in. thick, was placed on the level soil surface. The screenings were smoothed and compacted with a grader and roller.

The structural steel channels were welded by county welders into a rectangular frame which was used for two purposes; as a structural member for panel edge support and as a form for casting the concrete. Each rectangular panel frame was constructed from two 28 ft 5 in. long and two 3 ft $11^5/_8$ in. long steel channels. The installation of the headed studs at the fabrication shop caused a warpage of the steel channels that were delivered to Noble County. Extra effort was required by the county to straighten the channels while welding the frame together. Straightening was accomplished by welding a steel bar across the 3 ft $11^5/_8$ in.

dimension at the quarter points of the frame, reference Figure 16. After weldment into a frame, the channels were painted with a zinc rich paint.

The reinforcing steel was delivered prefabricated to length. Both top (traffic side) and bottom (beam side) mats of reinforcing steel were tied outside the channel frame, then inserted and secured in place. Since the panels were cast in a traffic-side-down orientation, the frames were held on edge while the top mat of reinforcing was placed on the ground. The frame was lowered and the mat was raised and tied in place to the crossbars and headed studs. The bottom mat of steel was placed into the frame and tied to support members placed across the top of the steel channels as shown in Figure 17.

The materials used in the panels for the demonstration bridge were those specified by design: Class AA concrete with a minimum ultimate compressive strength of 3,500 psi, Grade 60 plain reinforcing, and A36 structural steel. Concrete for the panels was obtained from a local ready-mix supplier. Three truck loads of concrete were used to cast the seven panels. Truck 1 provided concrete for panels 1 and 2; truck 2 for panels 3, 4, and 5; and truck 3 for panels 6 and 7, as shown in Figure 18. The concrete was delivered to each panel directly from the chute on the truck and vibrated as shown in Figure 19. Panels 1 and 2 were screeded and finished with a 2 × 4 in. board as shown in Figure 20. The remaining panels were screeded with a

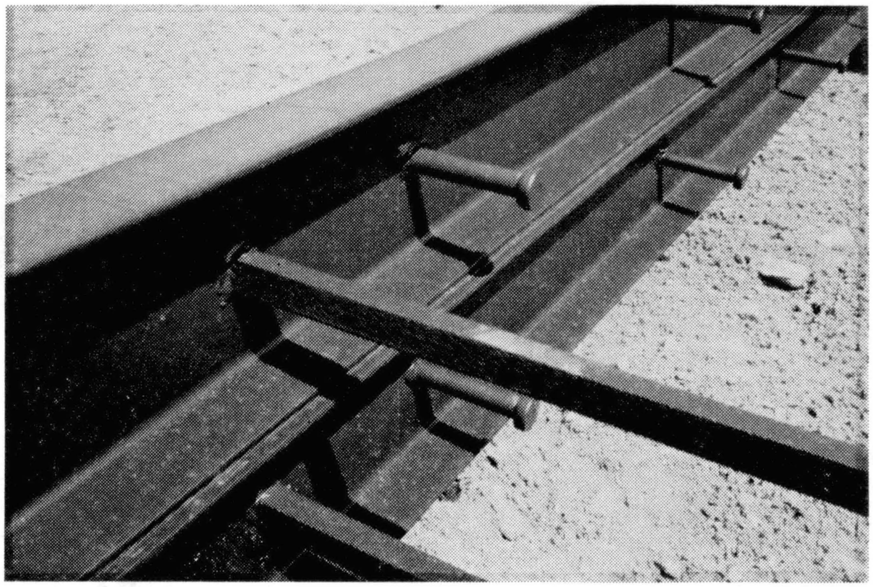


Figure 16. Structural Steel Channel Frames for Two Panels Showing Steel Bars and Headed Studs

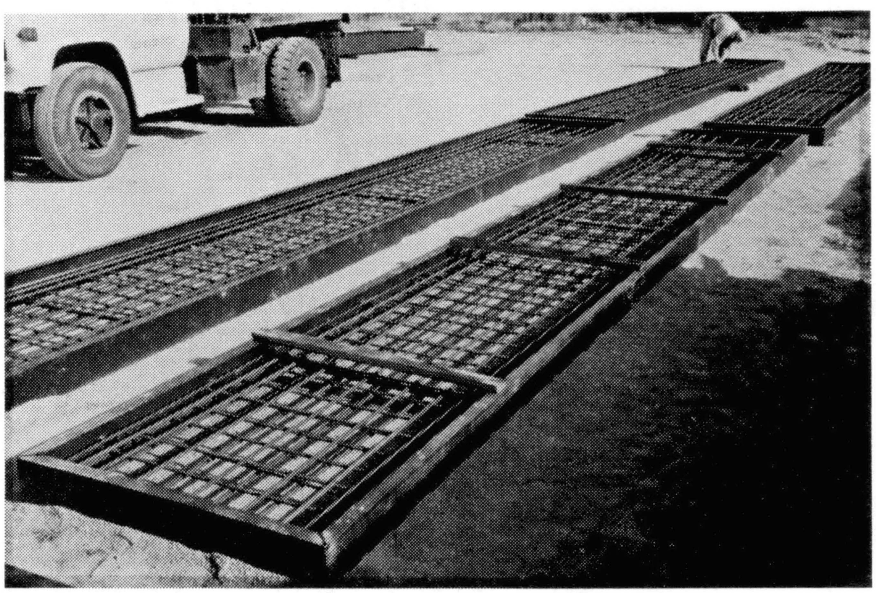


Figure 17. Support for Upper Layer of Reinforcing Steel

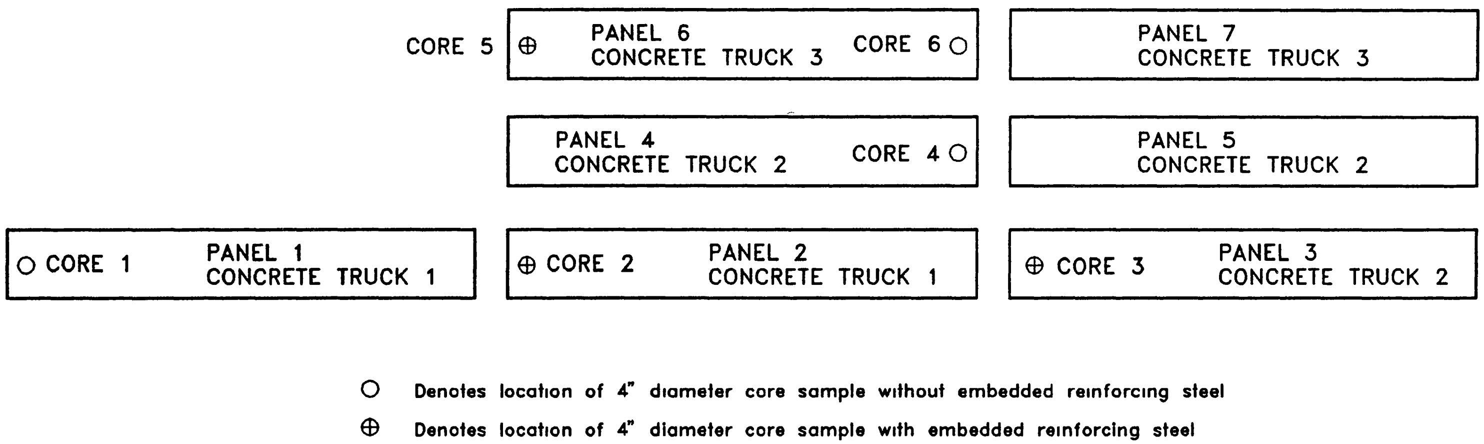


Figure 18. Deck Panel Layout on Casting Bed with Corresponding Concrete Truck Numbers and 4 in. Dia. Core Locations

Figure 19. Placement of Concrete into the Structural Steel Channel Frames

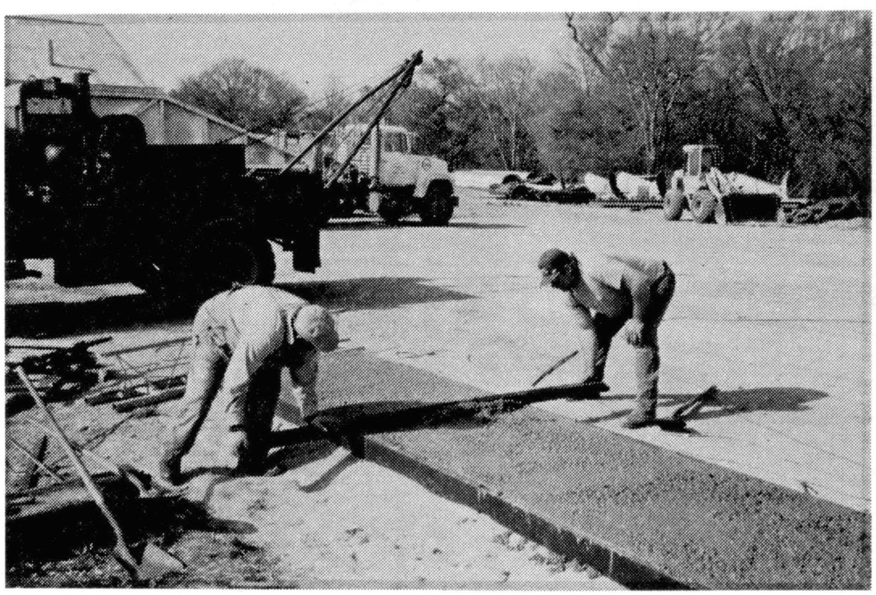


Figure 20. Screeding and Finishing with a 2 × 4 in. Board

motorized vibrating screed. A commercial curing compound was sprayed on the panels to facilitate the curing process after the initial set of the concrete. A completed panel is shown in Figure 21.

No slump tests were taken from any of the three trucks. The slumps were estimated by ODOT personnel and the author to be 3 in., 5 in., and 8 in. for trucks 1, 2, and 3, respectively. Four 6 in. dia. × 12 in. cylinders were prepared for compressive strength testing from truck 1, reference Figure 22. The 7 and 28 day test results for these cylinders are shown in Table II. Because no cylinders were made from trucks 2 and 3, six 4 in. dia. cores, two from each of the three truck loads of concrete, were taken from the panels for compressive strength testing. ODOT research personnel performed the coring operation in which cores were taken from the locations shown in Figure 18 at a concrete age of 43 days. Three of the six cores had embedded reinforcing even though a reinforcing steel locator was used to locate the mat of reinforcing nearest the top surface of the panels. To be consistent throughout the testing of the cores, all cores were sawed to a ratio of length to diameter (l/d) of approximately 1:1. This removed the embedded steel and made all cores geometrically similar. The cores were sawed at a concrete age of 85 days and then soaked in lime water for 123 hours prior to compressive strength testing at a concrete age of 90 days. The results of core testing, reference Table III, verified the ultimate compressive

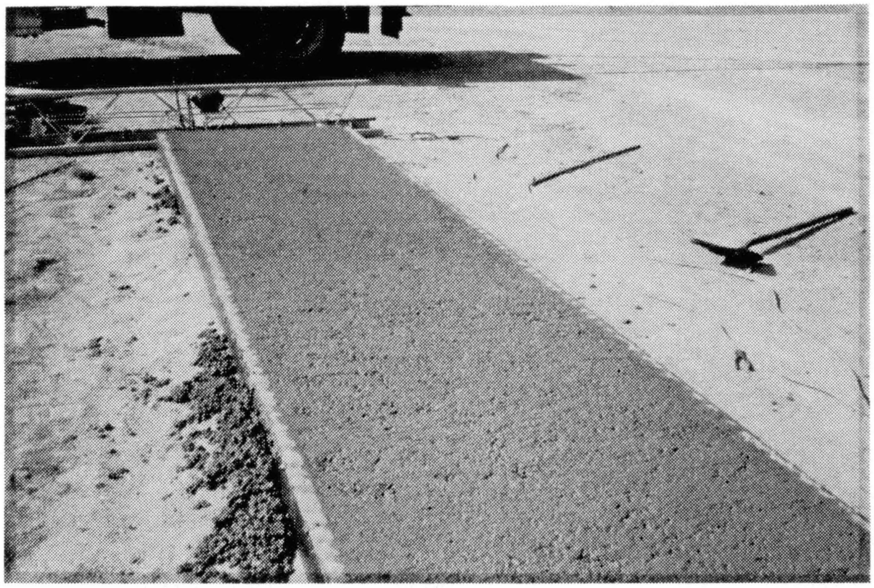


Figure 21. Completed Panel

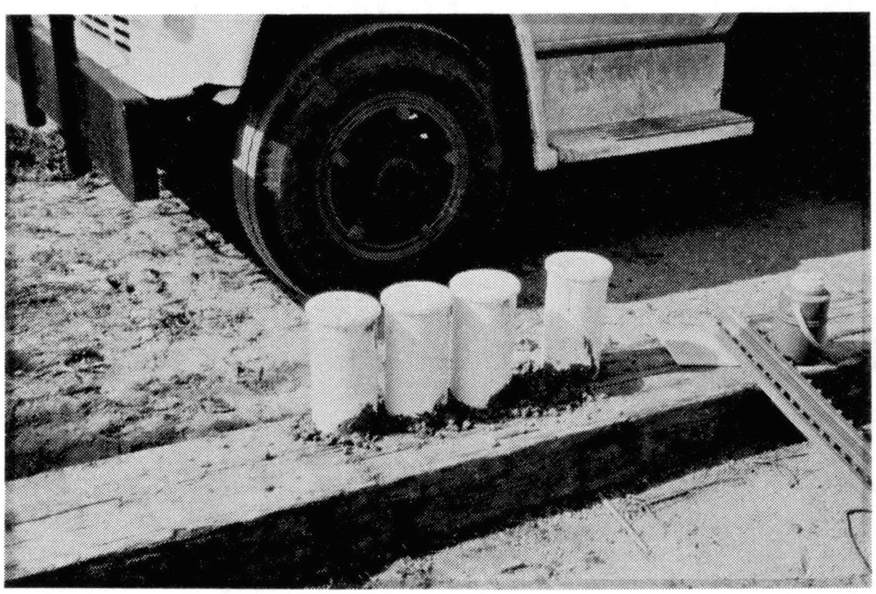


Figure 22. Test Cylinders Made from Concrete Truck 1

TABLE II

COMPRESSIVE STRENGTH TEST RESULTS FOR THE 6 IN. DIA. × 12 IN. CYLINDERS CAST FROM CONCRETE TRUCK NO. 1

CYLINDER NUMBER	AGE (days)	BREAK LOAD (lbs.)	AREA (in^2)	PSI	AVERAGE PSI
1	7	135,000	28.27	4,770	4,770
2	28	160,000	28.27	5,660	
3	28	165,000	28.27	5,840	5,720
4	28	160,000	28.27	5,660	

TABLE III

COMPRESSIVE STRENGTH TEST RESULTS FOR THE 4 IN. DIA. CORES AT CONCRETE AGE OF 90 DAYS

CORE NO.	DECK PANEL NO.	CONCRETE TRUCK NO.	L/D RATIO	AREA (in^2)	CORRECTION FACTOR	PSI	AVERAGE PSI
1	1	1	1.09	12.37	0.89	5,000	5,610
2	2	1	1.00	12.57	0.87	6,220	
3	3	2	0.96	12.47	0.85	5,970	5,910
4	4	2	1.05	12.37	0.88	5,850	
5	6	3	1.10	12.47	0.89	6,520	6,430
6	6	3	1.09	12.57	0.89	6,340	

strength of the concrete in the panels met the required specifications. Sampling and testing of the cores was done in accordance with the American Society of Testing and Materials (ASTM) [3].

The panels cured in place, reference Figure 23, for 49 days before being loaded for transport to the bridge site.

Loading and Transporting Deck Panels

The panels were a challenge to lift from the casting bed because no inserts or lifting loops were specified on the drawings. Inserts were intentionally omitted from the design to allow casting of the panels with the traffic-side-up or traffic-side-down orientation and to allow stack casting of panels. Commercial type inserts were also not considered so rural counties would not be required to procure special equipment or materials. The panels were designed for 8 ft 0 in. maximum spacing of lift points for panel support during lifting and handling.

Noble County personnel developed a lifting and inverting procedure for convenient and efficient handling of the panels. An edge of the panel was lifted approximately 18 in. with a front end loader by placing the bottom edge of the bucket underneath the center portion of the width of the panel (28 ft 5 in.) as shown in Figure 24. Four 4 × 4 in. timbers were then placed underneath the panel at the support locations shown on the plans. The bucket of the loader was

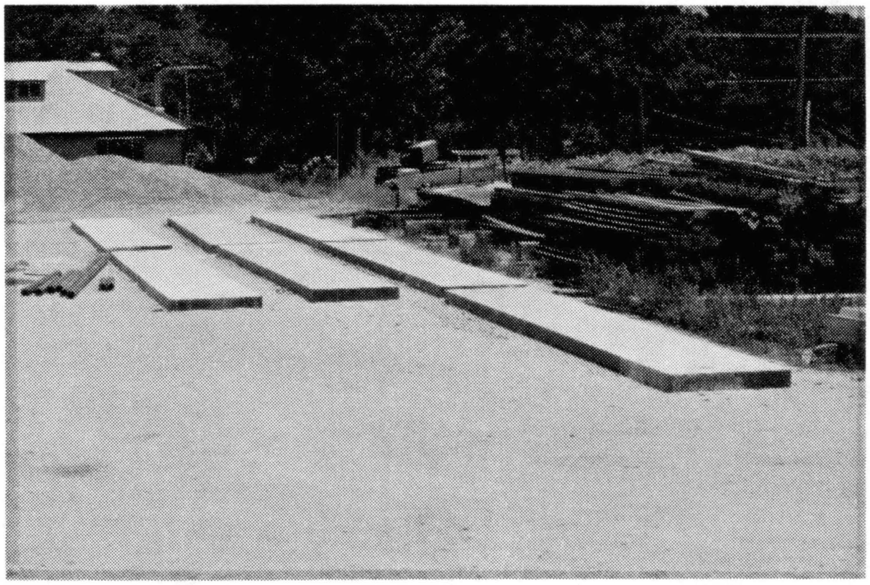


Figure 23. Panels Cured for 49 Days

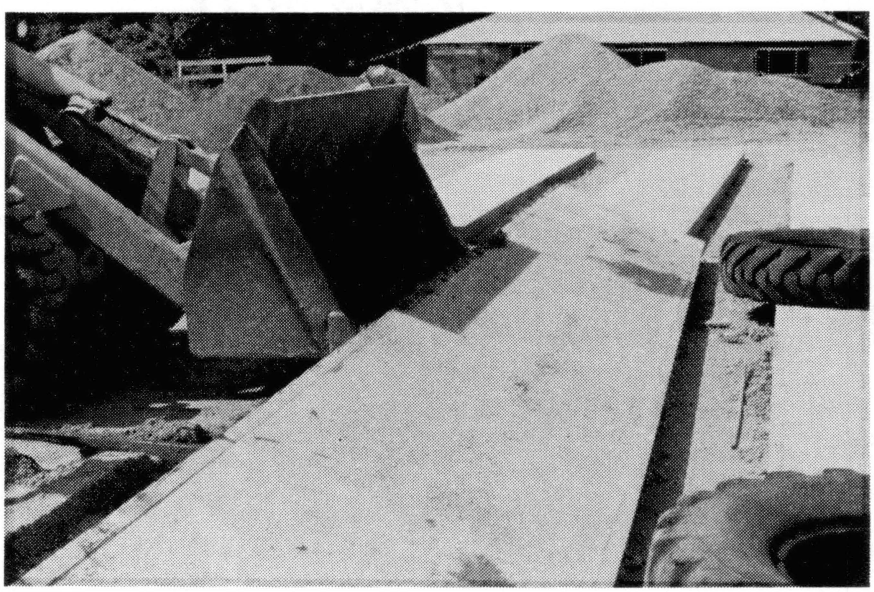


Figure 24. Initial Lift of Panel with the Bucket of a Front End Loader

then lowered which allowed the panels to rotate downward and rest on the 4 in. timbers. The panel at this point was still in a traffic-side-down position. Two chains were wrapped around the panel on each side of the bucket of the loader, approximately 8 ft apart. The chains were secured at the edge of the panel and to each edge of the bucket so when the bucket was raised the panel was suspended from the long edge, reference Figure 25. Approximately 1 to $2^1/_2$ in. of rock screenings were attached to the traffic side of the panel and were removed by wetting and scraping with shovels, reference Figure 26. The panels were then rolled over to a traffic-side-up position and gently lowered onto two used grader tires to cushion and hold the panel off the ground to maneuver the chains. At this stage another front end loader was used and four chains (2 chains for each loader) were wrapped around the panel at the lifting locations shown on the plans. This allowed lifting of the panel in a horizontal position as shown in Figure 27. Each chain was connected to the edge of a bucket and tensioned equally. The two loaders simultaneously raised the panel high enough to allow a flatbed trailer to back under the panel. The panel was then lowered onto the bed of the trailer as shown in Figure 28.

Four panels were transported during the first trip, reference Figure 29, and three panels during the second trip. The trip from the county yard to the bridge site required approximately 30 minutes.

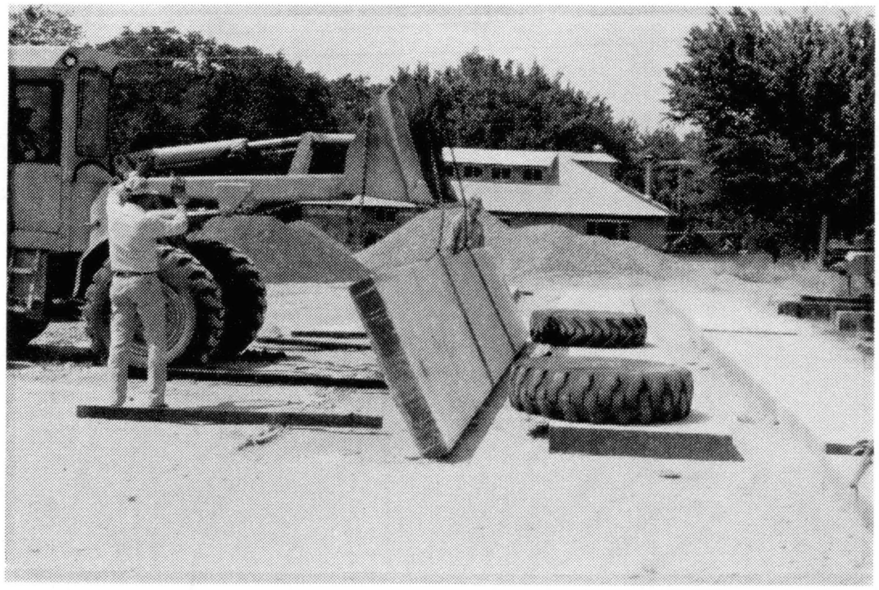


Figure 25. Lifting Panel on Edge for Roll Over to Traffic-Side-Up Position

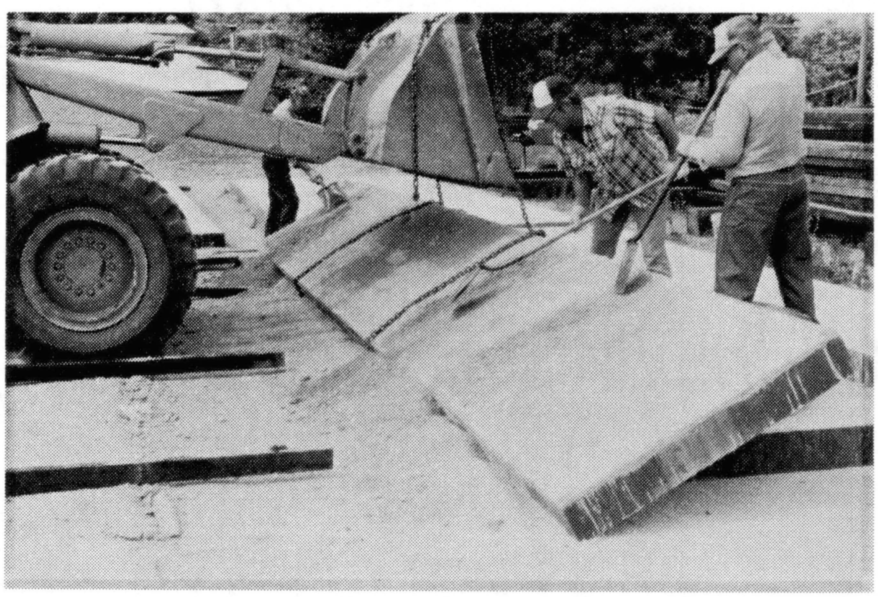


Figure 26. Removing Rock Screenings by Scraping with Shovels

Figure 27. Two Front End Loaders Lifting Panel in a Horizontal Position

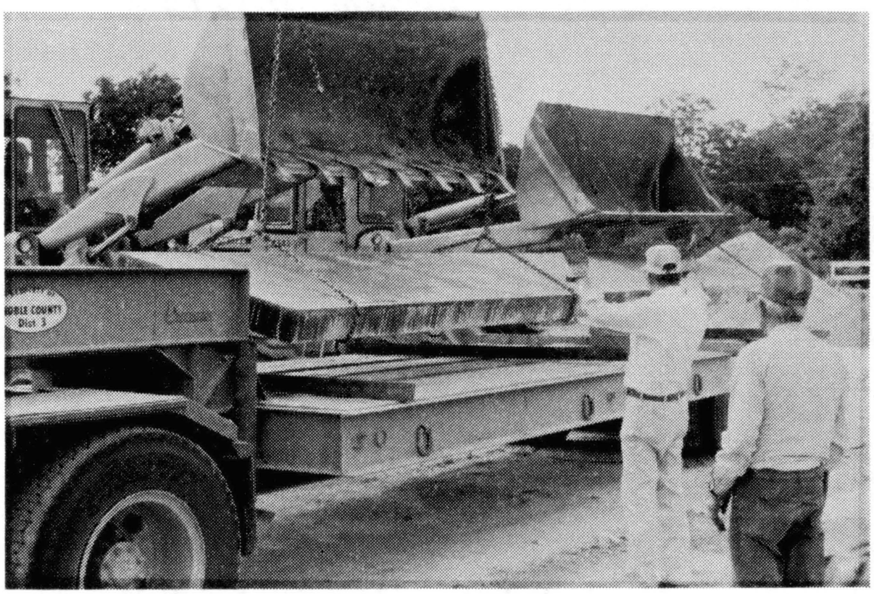


Figure 28. Lowering the Panel onto a Flatbed Trailer

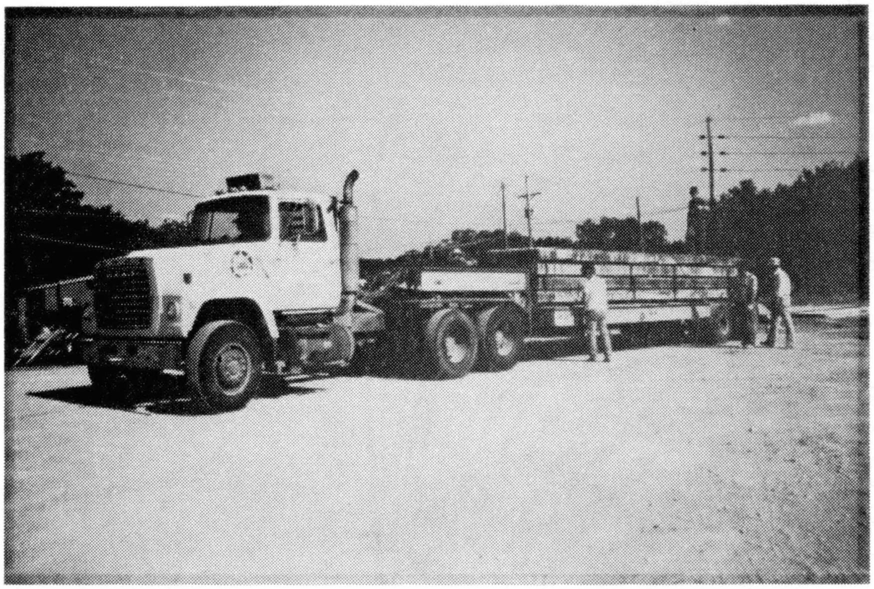


Figure 29. Four Panels on Flatbed Trailer Ready for Transport

Placement of Deck Panels on Bridge

The used steel beams employed in the demonstration bridge are shown ready for panel placement in Figure 30. Full depth end diaphragms were fabricated from the same section as the beams. As previously stated the new bridge deck elevation was raised and the adjacent roadway was re-worked. Therefore, there was no direct roadway access to place the panels using front end loaders. If this had been a bridge undergoing only a deck replacement with no adjacent roadway work, the panels could be utilized to drive on for subsequent panel placement as soon as they were connected.

For construction of the deck on the demonstration bridge, a crane was used to lift the panels from the flatbed

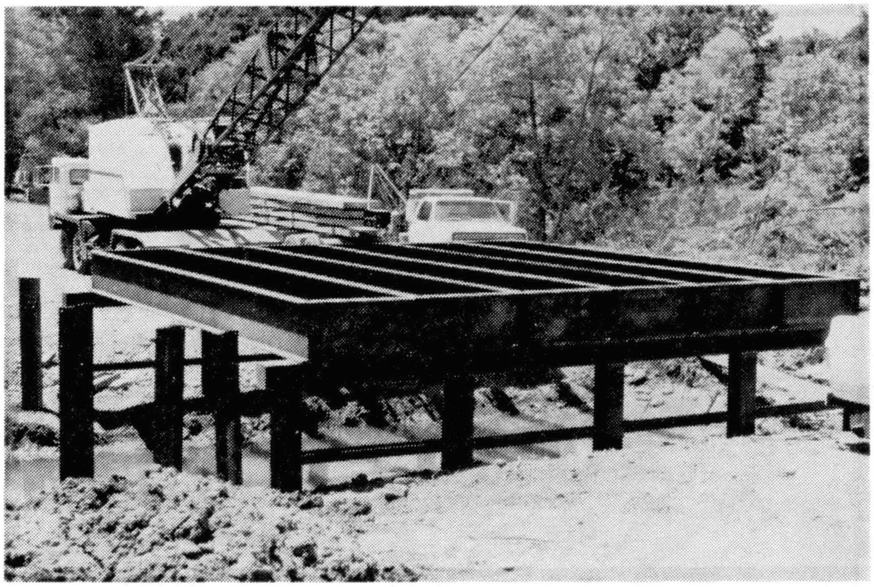


Figure 30. Used Steel Beams of Demonstration Ready for Panel Placement

trailer and place them on the bridge. Each of the panels on the demonstration bridge weighed 11,380 lbs. and were lifted using four cable slings at the positions specified on the plans, reference Figure 31. A spreader beam was used to equalize the load and to keep the cables from sliding. The panels could not be placed precisely in their final position because space was required between panels for removal of the cables, placement of connection plates, and placement of shims. This created the need for a method to align the panels into final position by sliding the panels along the top of the beams. The first panel placed at the end of the bridge could not be placed into final position because the full depth end diaphragms interfered with removal of the cables as shown in Figure 32. After the four panels from

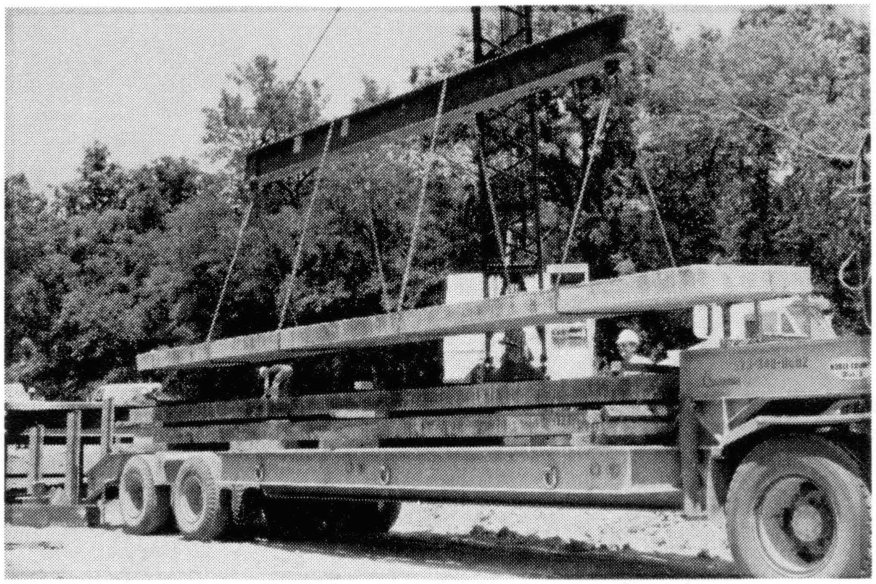


Figure 31. Lifting a Panel from the
Flatbed Trailer

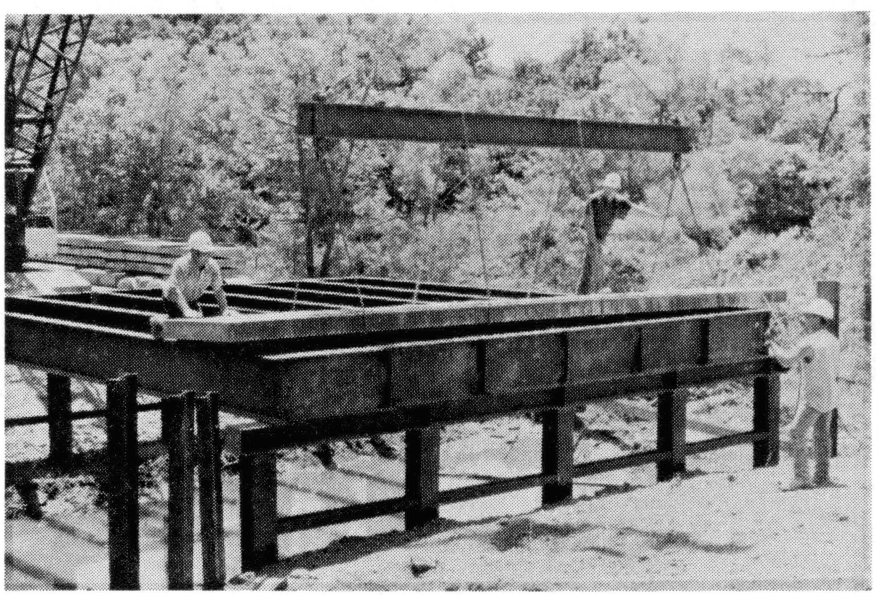


Figure 32. Placing the First Panel on the
Demonstration Bridge

the first load were placed on the bridge the alignment and connection operation began. The first panel was positioned using the bucket of a backhoe. This method did not work well because some areas of the diaphragm flanges protruded slightly above the beams which created difficulty in sliding the panel into place. The second panel was positioned with a hand jack. This method required considerable physical effort and was disliked by the workers. The third panel was aligned using a hook as shown in Figure 33. The hook was fabricated from a short section of HP10 × 42 piling and was placed over the edge of the panel. A chain was used to pull the panel into final position, reference Figure 34, using the bucket of a backhoe located off the bridge. This method was the most efficient and was utilized for alignment of the remaining panels.

The shimming and connection operation was performed as a part of the alignment of panels. Each of the six transverse joints on the demonstration bridge consisted of two edges (of adjacent panels) and a series of connection plates. For discussion purposes, the leading edge of a panel is defined as the second edge to be aligned, or completion edge, of the joint. The trailing edge is the first panel edge of a joint to be aligned into final position, i.e. the first half of a joint. Figure 35 shows the need for shimming along the trailing edge of a panel which has been aligned into final position. A 2 × 6 in. shim of the required thickness was placed underneath the structural

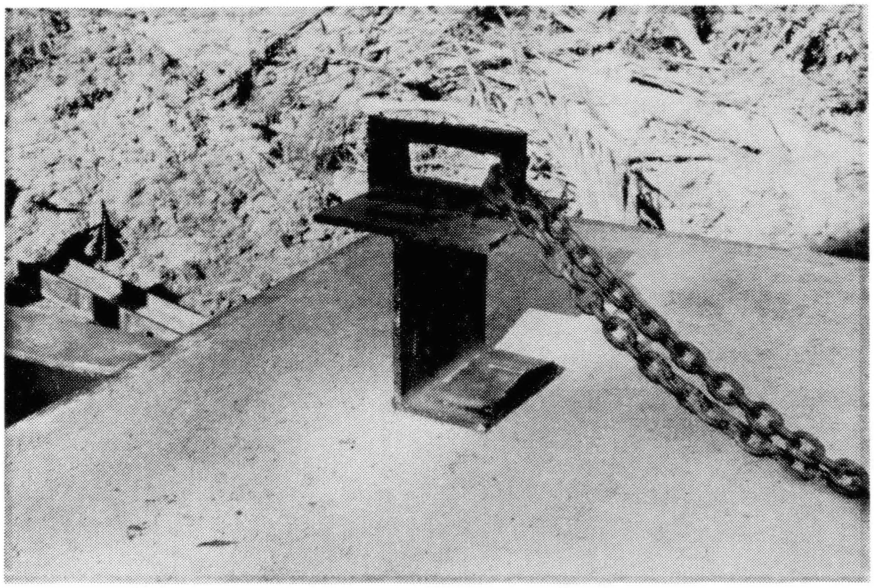


Figure 33. Hook Fabricated for Use in Panel Alignment

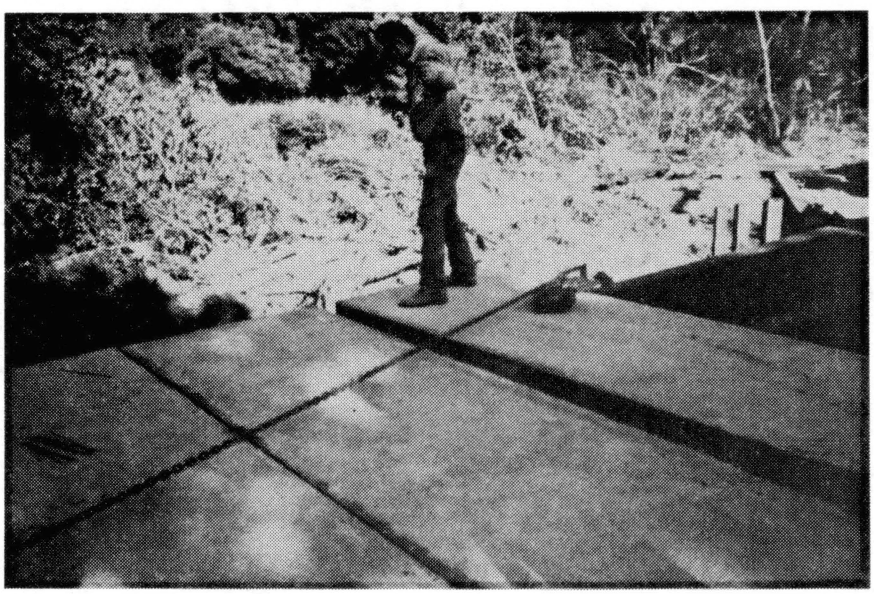


Figure 34. Panel Alignment Utilizing Hook with Chain

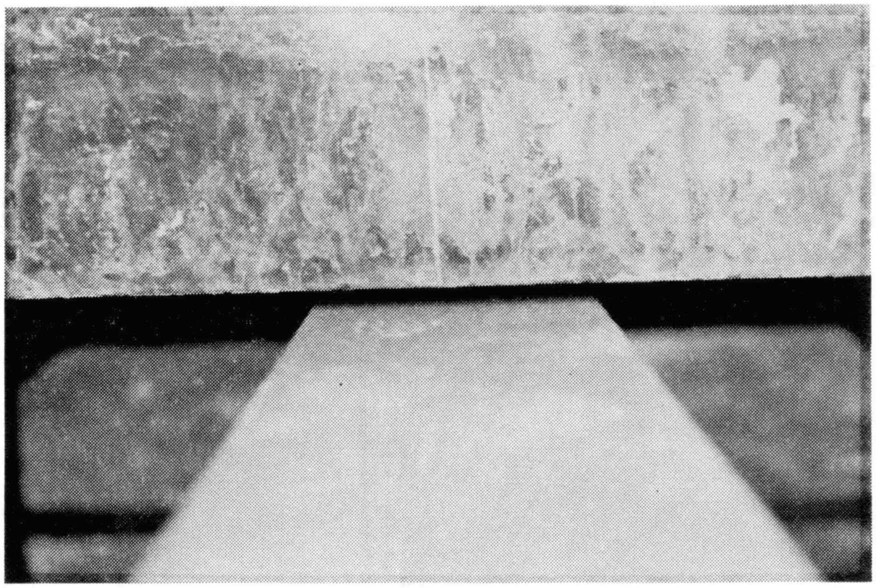


Figure 35. Need for Shims to Provide Bearing of Panel on Beam

steel channel and held in place by a tack weld on the channel. This was performed at all locations needing shims along the trailing edge of the panel. After shimming was complete along the trailing edge, a $^{3}/_{8}$ in. thick connection plate was positioned tightly against the flange of each beam and welded to the channel on the trailing edge of the panel as shown in Figure 36. No welds to the existing beam flanges were made.

Before the next panel was aligned into final position, an assessment was made whether shims were needed along it's leading edge. If required, shims were placed and tack welded to the channel. The panel was then aligned by sliding the panel (and shims along the leading edge) into final position. A second weld was made to secure the connection

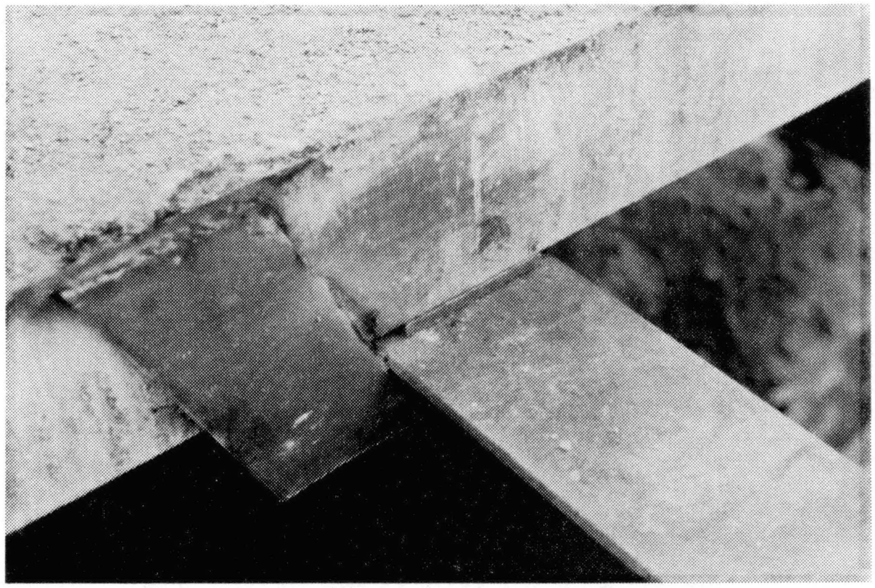


Figure 36. Connection of Panel to Beam with Shim in Place

plate to the leading edge of the panel and to connect the panel to the beams. After all welds were made, the joint was complete as shown in Figures 37 through 40. Figure 39 shows the alternating direction of the connection plates between the exterior and first interior beams. The connection plates are detailed on the panel plans shown in Appendix B.

The shimming procedure described above was repeated as panels were aligned and connected. To insure full bearing of the leading edge of a panel to the beams after final alignment, the shimming operation should be performed while the panel is as close to its final position as possible. A plan of the required shimming for the demonstration bridge is shown in Figure 41. A connection plate was placed at

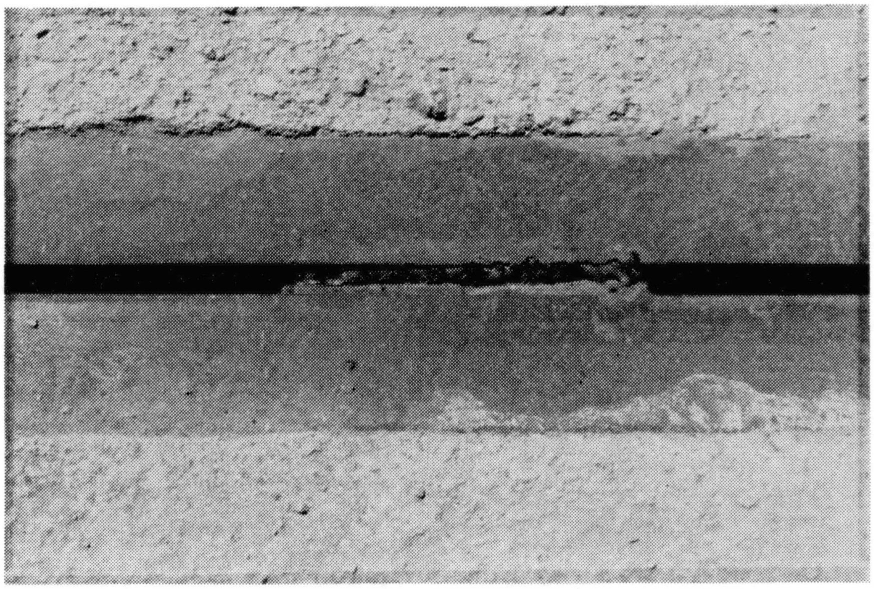


Figure 37. Connection Plate as Viewed from Above Showing the Welding of Plate to Structural Steel Channel on Each Side

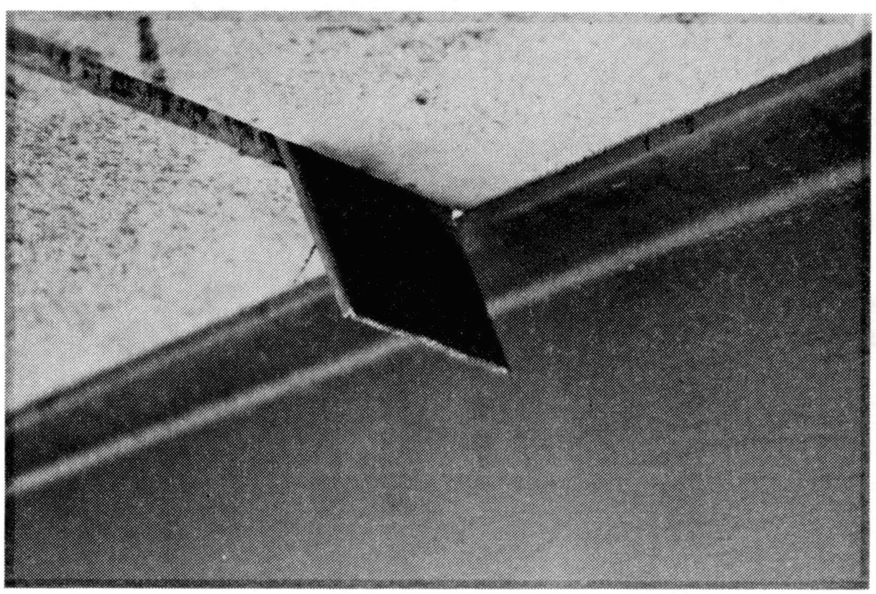


Figure 38. Connection Plate as Viewed from Below Deck

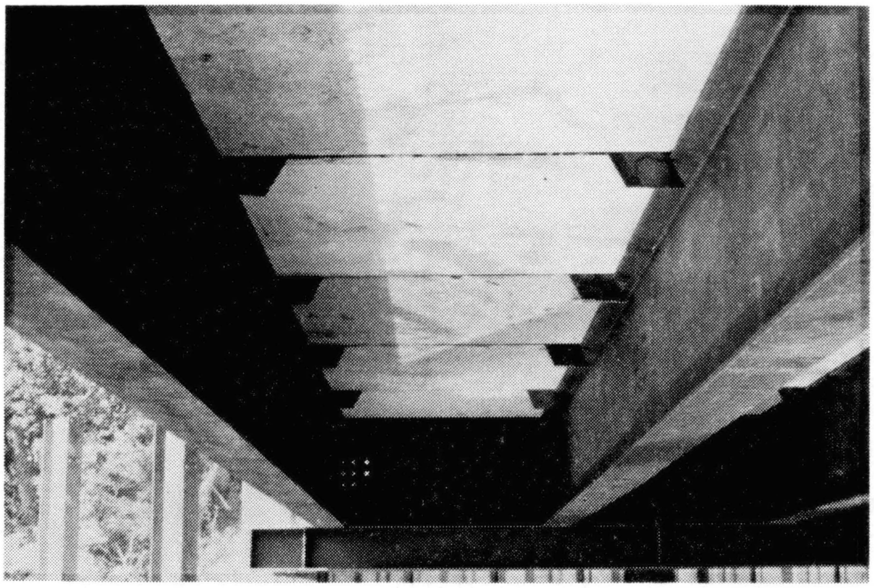


Figure 39. Arrangement of Connection Plates Between Exterior and First Interior Beams

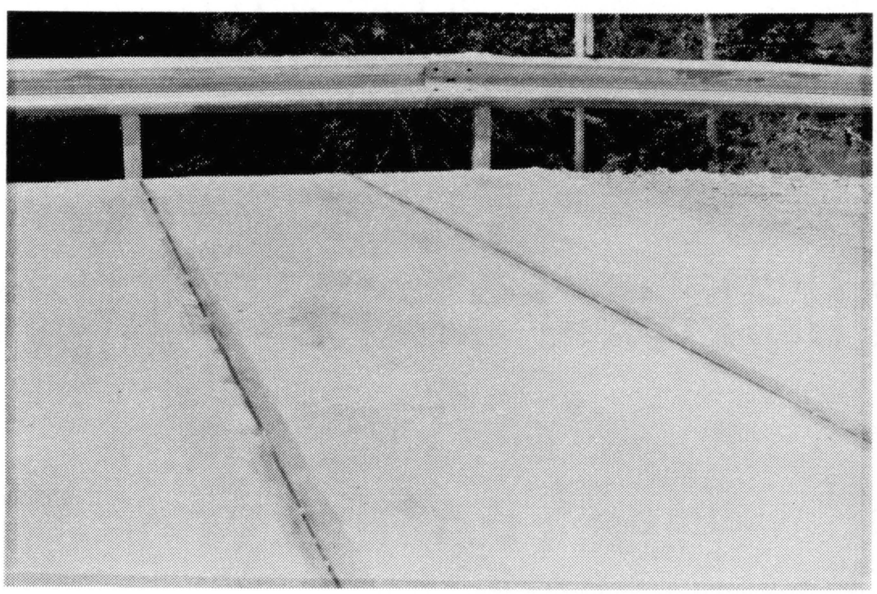


Figure 40. Surface of Deck Showing Joints

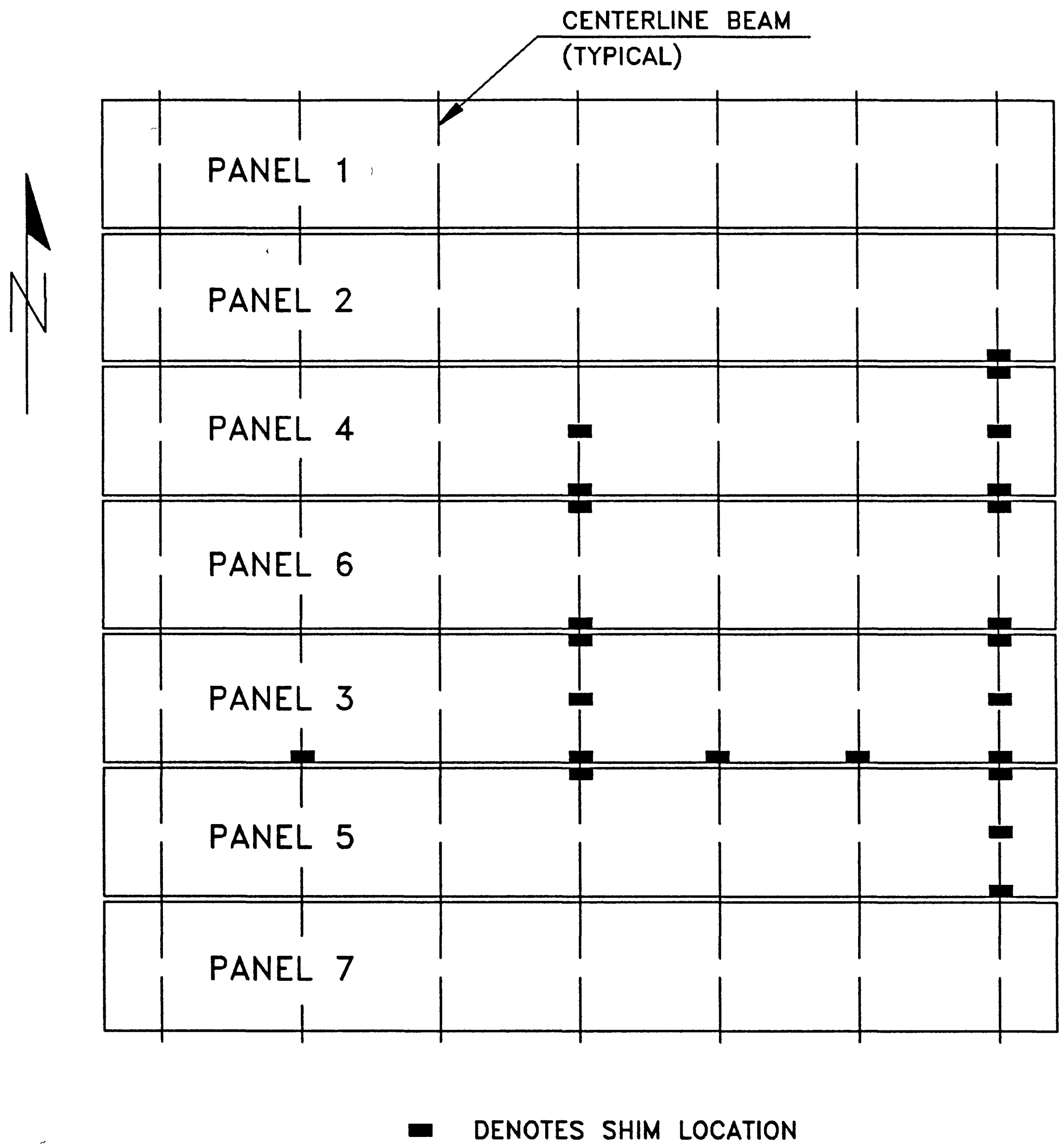


Figure 41. Plan Location of Required Shims and Panel Arrangement on Demonstration Bridge

each intersection of a joint and beam. This required 42 connection plates to secure the panels to the steel beams.

Total construction time for the demonstration bridge was thirteen ten-hour work days using a four man crew. Total construction included demolition of the existing bridge and salvaging the center span steel beams, driving of pile foundations, construction of abutments, placement of steel beams, placement of precast concrete deck panels, and installation of traffic rails. The seven panels were placed, aligned, and connected in eight hours. The demonstration bridge was opened to traffic after twelve days of construction, reference Figure 42. The traffic rails were connected as shown in Figure 43 on the thirteenth day of construction. As previously stated, Noble County chose not to use the TR-2 traffic rail. The rail constructed on the demonstration bridge required the use of scaffolding which increased the complexity and time of construction. The completed demonstration bridge is shown in Figures 44 and 45.

Figure 42. Demonstration Bridge After Twelve Days of Construction

Figure 43. Connection of Traffic Rails

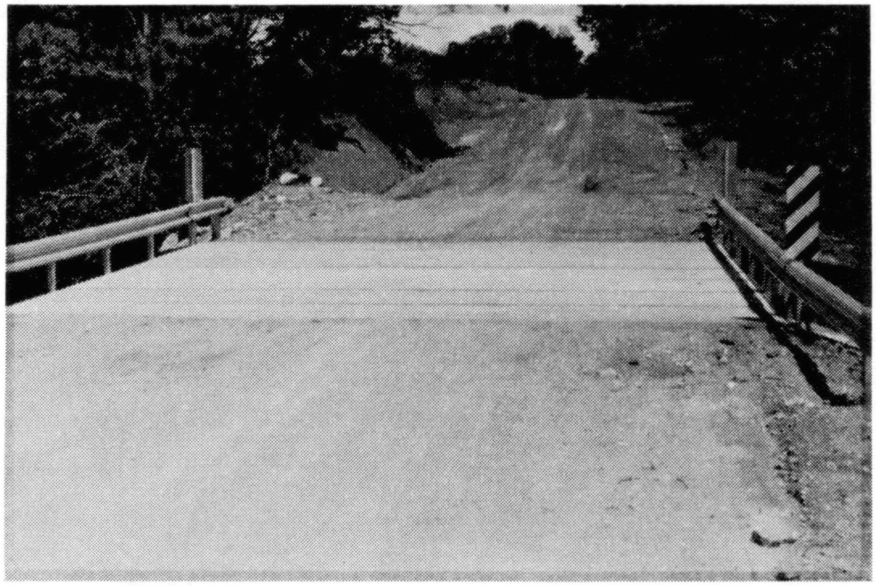


Figure 44. Demonstration Bridge as Viewed from the South

Figure 45. Demonstration Bridge as Viewed from the Northeast

CHAPTER V

COMPARISON TO AN ALTERNATE DECK REPLACEMENT METHOD

This chapter presents a comparison of the construction requirements of the precast concrete panel deck of the demonstration bridge to a traditional cast-in-place concrete deck. Material and labor requirements of the two methods are discussed.

Noble County District 3 personnel constructed four conventional cast-in-place bridge decks during the three years prior to construction of the demonstration bridge. The same crew performed all of the construction for the precast concrete panel deck of the demonstration bridge. The comparison of the two construction methods is performed on a square foot basis using overall deck dimensions. The precast concrete panel deck has total dimensions of 28 ft 0 in. long by 28 ft 5 in. wide (796 ft^2 of deck area). The cast-in-place deck has total dimensions of 21 ft 0 in. long by 26 ft 6 in. wide (556 ft^2 of deck area).

Noble County personnel and the author kept logs of the quantities of material, labor, and equipment required to construct the precast panel deck, reference Table IV. The labor and equipment hours in Table IV represent the actual

TABLE IV

MATERIAL, LABOR, AND EQUIPMENT REQUIREMENTS FOR THE PRECAST CONCRETE PANEL DECK OF THE DEMONSTRATION BRIDGE

MATERIAL:		
Concrete	17.1	c.y.
Reinforcing Steel	4,991	lbs.
Structural Steel	4,844	lbs.
LABOR:		
Preparation of Casting Bed:		
Foreman	1	hr.
Equipment operator	3	hr.
Fabrication of Seven Panels:		
Foreman	19	hr.
Welder	12	hr.
Helper	84	hr.
Loading and Transport of Seven Panels:		
Foreman	4	hr.
Front End Loader Operator	8	hr.
Truck Driver	5	hr.
Placement of Seven Panels on Bridge:		
Foreman	8	hr.
Welder	16	hr.
Helper	8	hr.
EQUIPMENT[a]:		
Preparation of Casting Bed:		
Grader	2	hr.
Self Propelled Steel Roller	1	hr.
Fabrication of Seven Panels:		
Front End Loader	4	hr.
Welding Machine	12	hr.
Concrete Vibrator	7	hr.
Loading and Transport of Seven Panels:		
Front End Loaders	8	hr.
Truck and Flat Bed Trailer	5	hr.
Placement of Seven Panels on Bridge:		
Crane	8	hr.
Truck and Flat Bed Trailer	7	hr.
Backhoe	8	hr.
Portable Welding Machines	16	hr.

[a]Equipment hours represent actual time to perform work.

time required to fabricate and erect the seven precast panels on a first time basis. Since this method of bridge deck construction was new to the crew, they were required to perform unfamiliar tasks. Examples are preparing a casting bed, hanging reinforcing steel from the top of the steel channel forms, lifting and maneuvering the panels, alignment of the concrete panels on the bridge beams, etc.

Method of Comparison

Since the precast concrete panel deck system was constructed for the first time by this crew, the hours shown in Table IV should be adjusted by an experience curve technique to obtain a valid comparison to a conventional cast-in-place system. This is because the cast-in-place system is familiar to the crew since they have performed this method of construction on a repetitive basis.

Studies have shown that any person who has performed the same task on a repetitive basis will require a shorter time to perform the task the second time than was required for the first time. The reduction in time required for successive tasks is due to greater familiarity of required tasks, better coordination of workers, and more effective use of tools and methods. Gates and Scarpa [4] presented a technique for experience curve adjustment for the time required to perform repetitive work.

Cumulative experience curves are based on the rate at which an individual or crew gains experience. If the repetitive tasks being performed are typical and common to the workers, the rate at which experience is gained is low and time reduction between tasks is less. If the repetitive tasks are not typical to the workers, the rate at which experience is gained is high and the time reduction between tasks is greater. Cumulative experience curves range from limits of 100% to 50%. For example, for two units a 100% curve shows no reduction in time to construct the second unit because:

$$100\% = [100\%(\text{first unit}) + 100\%(\text{second unit})]/2.$$

The 50% curve is the theoretical low because for two units:

$$50\% = [100\%(\text{first unit}) + 0\%(\text{second unit})]/2.$$

Construction related tasks typically are in the experience curve range of 70% to 90% [7]. A cumulative experience curve of 90% was selected for use in the adjustment of hours shown in Table IV.

The Gates and Scarpa adjustment used in the comparison is based on a cumulative average time (CAT) required for construction of the first n units. The CAT is expressed as a percentage of the time required to construct the first unit. As efficiency of constructing repetitive units increases, the time required to construct subsequent units decreases.

To be consistent with the number of cast-in-place decks constructed, the adjustment to the labor hours for the precast concrete deck panel system was based on the construction of four precast decks. For n = 4 the CAT to construct four similar precast decks is (Gates and Scarpa):

$$CAT = n^{-0.1521} \times 100\%$$

$$CAT = 4^{-0\ 1521} \times 100\%$$

$$CAT = 81\%$$

The average time per deck to construct four precast panel bridge decks is 81% of the time required to construct the first deck.

Table V shows an adjustment of the hours shown in Table IV using the Gates and Scarpa 90% cumulative experience curve technique. In addition to the experience curve adjustment, the hours shown in Table V do not include the time required to field straighten and weld the steel channels into seven panel frames, reference Chapter IV. This time included 3 hours of foreman, 12 hours of welder, and 10 hours of helper. To improve efficiency for repetitive work, this field work on the panel frames should be done as shop welding by the steel fabricator. Thus, the panel frames should be completely fabricated and delivered to the casting site fully assembled.

Data was acquired from the fourth cast-in-place bridge deck constructed in the series of four. Table VI shows the material and total labor hours required to construct the 21 ft 0 in. long by 26 ft 6 in. wide (556 ft^2) cast-in-place deck used in the comparison. Data for the cast-in-place

TABLE V

ADJUSTED MATERIAL AND LABOR REQUIREMENTS FOR THE PRECAST CONCRETE PANEL DECK OF THE DEMONSTRATION BRIDGE BASED ON A 90% CUMULATIVE EXPERIENCE CURVE[a]

MATERIAL:		
Concrete	17.1	c.y.
Reinforcing Steel	4,991	lbs.
Structural Steel	4,844	lbs.
LABOR:		
Preparation of Casting Bed:		
Foreman	0.81	hr.
Equipment operator	2.43	hr.
Fabrication of Seven Panels[b]:		
Foreman	12.96	hr.
Welder	0.00	hr.
Helper	59.94	hr.
Loading and Transport of Seven Panels:		
Foreman	3.24	hr.
Front End Loader Operator	6.48	hr.
Truck Driver	4.05	hr.
Placement of Seven Panels on Bridge:		
Foreman	6.48	hr.
Welder	12.96	hr.
Helper	6.48	hr.
Total Labor:	115.83	hr.

[a]Adjustment factor is 0.81.

[b]Hours for field straightening and welding of the seven panel frames are not included: 3 hours of foreman, 12 hours of welder, and 10 hours of helper.

TABLE VI

MATERIAL AND LABOR REQUIREMENTS FOR THE CAST-IN-PLACE BRIDGE DECK[a]

MATERIAL[b]:		
Concrete	12.0	c.y.
Reinforcing Steel	2,328	lbs.
Structural Steel	1,024	lbs.
Steel Form Deck	62	s.y.
LABOR[c]:		
Construction of Bridge Deck:		
Total Labor:	88	hr.

[a]Equipment utilization was not available.

[b]Material quantities are calculated. The reinforcing steel quantity was calculated using the reinforcing steel pattern in the ODOT County Bridge standards for non-composite steel beams on a 4 ft $1\frac{1}{2}$ in. spacing.

[c]From Noble County District 3 bridge files.

deck was acquired from the files of the Noble County District 3 office. Labor hours for the cast-in-place deck did not have a breakdown of construction activities, only total labor hours were available. For a valid comparison using CAT, the time required to construct the fourth deck (t_4) in the series must be calculated as a percentage of the time required to construct the first cast-in-place deck (t_1). From Gates and Scarpa:

$$t_1 = (i^{(1 - 0\ 1521)} - (i - 1)^{(1 + 0.1521)}) \times 100\%$$

$$t_4 = (4^{(0.8479)} - 3^{(0.8479)}) \times 100\%$$

$$t_4 = 70\%$$

The time required to construct the fourth cast-in-place deck is 70% of the time required to cast the first. For a direct comparison to the precast deck, the time required to construct the fourth cast-in-place deck must be adjusted to reflect a CAT of 81%. Therefore, the labor hours shown in Table VI should be adjusted for CAT by a factor of (81%) ÷ (70%), or 1.157%.

In addition to the experience curve adjustment, the labor hours shown in Table VI should be adjusted to account for the square foot size difference between the two comparison decks. The ratio of square footage of the precast panel deck (796 ft^2) to the square footage of the cast-in-place deck (556 ft^2) is 1.432. The time required for the same construction activities of bridge decks of different sizes is not linear. Activities such as preparing the site and equipment, some elements of formwork, and clean-up could be considered to require equal time for construction of cast-in-place bridge decks of two sizes. Time for activities such as placement of reinforcing steel, placement of concrete, and finishing concrete could be considered linear. For this comparison, 15% of the time required to construct the cast-in-place deck was considered constant with 85% considered as linear based on the 1.432 ratio. This yields a size adjustment factor to be applied to the labor hours of the cast-in-place bridge deck as follows:

$$0.15 + 1.432(0.85) = 1.367$$

Table VII shows the quantities of material and labor after adjustment for experience curve and deck size.

Comparison

The comparison of material and labor requirements was performed on a square foot basis between Table V for the precast panel deck and Table VII for the cast-in-place deck. Comparison summaries are shown in Table VIII.

The reinforcing steel and structural steel material quantities of the precast deck panel system are greater than those for the cast-in-place deck. It should be noted that the precast panels were constructed from a "standard" plan for use on bridge beam spacings up to 5 ft 6 in. They were placed on a 4 ft 2 in. beam spacing on the demonstration bridge. The quantity of reinforcing steel for the

TABLE VII

ADJUSTED MATERIAL AND LABOR REQUIREMENTS FOR THE CAST-IN-PLACE BRIDGE DECK BASED ON DECK SIZE

MATERIAL[a]:			
Concrete		17.2	c.y.
Reinforcing Steel		3,280	lbs.
Structural Steel		1,216	lbs.
Steel Form Deck		62	s.y.
LABOR[b]:			
Construction of Bridge Deck:			
Total Labor	88 hrs. × 1.582 =	139	hr.

[a]Material quantities are calculated. The reinforcing steel quantity was calculated using the reinforcing steel pattern in the ODOT County Bridge standards for non-composite steel beams on a 4 ft $1^1/_2$ in. spacing.

[b]Adjustment factor is 1.157 × 1.367 = 1.582.

TABLE VIII

COMPARISON OF THE PRECAST PANEL DECK TO THE CAST-IN-PLACE DECK ON A SQUARE FOOT BASIS[a]

	Type of Deck	
	Precast Panel (per ft^2)	Cast-In-Place (per ft^2)
Material:		
Concrete	0.021 c.y.	0.022 c.y.
Reinforcing Steel	6.27 lbs.	4.12 lbs.
Structural Steel	6.09 lbs.	1.29 lbs.
Steel Form Deck		yes[b]
Labor:	0.146 man-hours	0.175 man-hours

[a]Based on 796 ft^2

[b]Steel form deck was used for the cast-in-place deck in this comparison. Form deck increases material costs and reduces labor costs. Conventional wood forms reduces material costs and increases labor costs.

cast-in-place deck was determined for a 4 ft $1^{1}/_{2}$ in. beam spacing given in the ODOT County Bridge Standards [8]. Structural steel quantities for the precast system are greater due to the panel frames. Total labor hours required for construction of the precast deck were 83% of the labor hours required for the cast-in-place deck.

Due to the recent construction of the precast panel deck, maintenance and repair data is not yet available to include in the comparison. Life cycle costing for the precast deck system cannot be performed until data becomes available.

The cost/Benefit ratio is also an important factor for consideration. Demolition of the existing wooden deck and placement of the seven panels and traffic rails required only two days. Thus, the cost to the driving public due to detouring is greatly reduced for the precast system compared to the cast-in-place deck which took two days to form and place concrete, plus 28 days for the concrete to cure. At project locations where closure of the road due to replacement of a bridge deck is determined to be costly or inconvenient to the driving public, it is recommended that the precast system be considered for use.

At locations where project safety is threatened by many workers and pieces of equipment located in a congested area, the precast system should be strongly considered because placement of the panels on the bridge requires minimal personnel and equipment.

CHAPTER VI

SURVEY OF OKLAHOMA COUNTIES FOR CASTING FACILITIES, EQUIPMENT, AND PERSONNEL

A survey was conducted by mail to evaluate the ability of counties in Oklahoma to implement the construction and placement of the precast concrete deck panels using county facilities, equipment, and labor. Each of the seventy-seven Boards of County Commissioners was mailed a packet which included a cover letter, survey form, and detail sheets explaining the precast deck replacement system, reference Appendix C. Figure 46 shows the twenty-one counties (a 27% return rate) that responded to the survey packets. A review of Figure 46 shows a response from counties that are geographically distributed throughout the State. The rate of response and geographic distribution appears to provide a sampling that is representative of all counties in Oklahoma.

Casting Facilities

Counties were surveyed to determine their existing facilities for casting the concrete panels. The panels can be easily cast on either a soil surface or a concrete casting bed. It was concluded by the author that all counties would have an outside unpaved area for the casting of

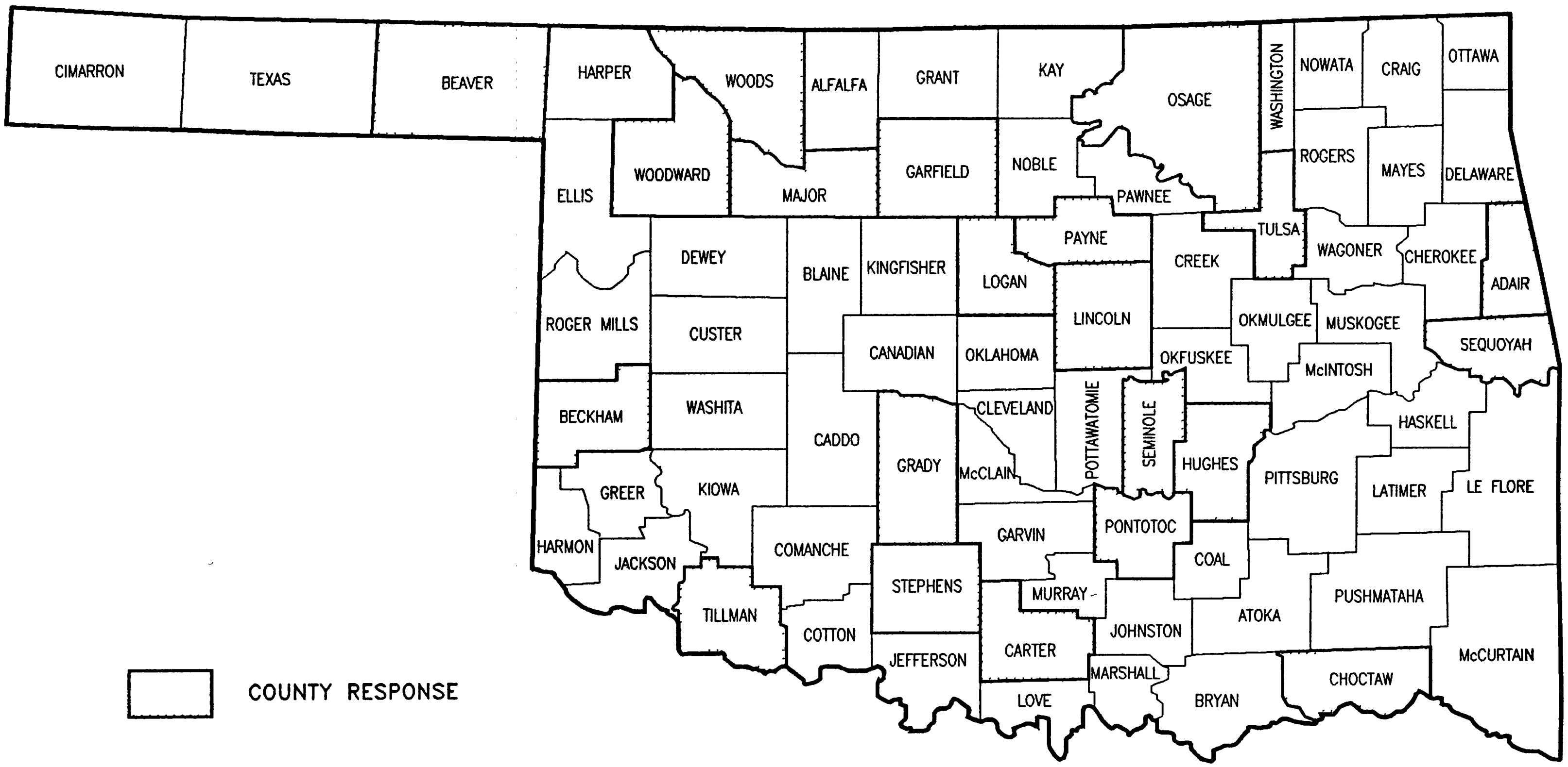


Figure 46. Location of Counties Responding to Survey

panels. Counties were asked about the availability of existing flat concrete slabs which could be used as a casting bed for the panels, either outside or inside a building. Table IX shows a compilation of the county responses. Of the 21 responding counties, five (24%) have outside flat concrete slabs. Of these five counties, four have slabs which range in size from 240 ft^2 to 8,000 ft^2 with an average of 3,860 ft^2. Eight of the 21 counties (38%) responded that they have a flat slab area inside a building. The inside slabs ranged in size from 200 ft^2 to 12,000 ft^2 with an average of 2,490 ft^2. Five of the eight responded that the building was heated which would allow casting during the winter months.

Although several counties have flat concrete slabs available for casting panels, the surface condition of the slabs would probably be inadequately level for casting in a

TABLE IX

CONCRETE SLABS AVAILABLE FOR CASTING DECK PANELS

LOCATION OF FLAT CONCRETE SLAB	COUNTIES WITH SLABS		SLAB SIZE (ft^2)		
	NUMBER	% OF TOTAL RESPONSE	MAX.	MIN.	AVE.
Outside	5	24%	8,000	240	3,860
Inside Building[a]	8	38%	12,000	200	2,490

[a]Of the eight counties which have a flat slab area inside a building, five responded that the building is heated.

traffic-side-up orientation without an extensive requirement of shimming.

County Equipment

Table X shows the county response to the availability of equipment to load, transport, and place panels on a bridge. None of the responding counties own a crawler mounted crane. However, eight (38%) own truck mounted cranes with lifting capacities ranging from 5,000 to 60,000 lbs with an average lifting capacity of 28,500 lbs. Two counties (10% of responding counties) can lease truck mounted cranes. It is probable that most counties can obtain truck mounted cranes through leasing. Nineteen counties (90% of responses) have ownership of wheel tractors with front loaders averaging a lifting capacity of 12,400 lbs. For safe handling of the larger panels, two (2) "wheel tractors with front loaders" should be used per panel as was demonstrated with the panels of the demonstration bridge. Many of the counties have winch trucks which could be used for lifting and placing the panels. Forty-three percent of the responding counties have winch trucks with lifting capacities of 5,000 to 10,000 lbs. Twenty-four percent have winch trucks with lifting capacities of greater than 10,000 lbs. Eighteen counties (86% of responses) have a truck which can safely transport one or more of the larger panels from the casting area to the bridge site.

TABLE X

COUNTY EQUIPMENT AVAILABLE FOR HANDLING DECK PANELS

EQUIPMENT TYPE	COUNTY RESPONSE								
	OWN			BORROW			LEASE		
	NUMBER	% OF TOTAL RESPONSE	AVERAGE LIFT. CAPACITY (LBS.)	NUMBER	% OF TOTAL RESPONSE	AVERAGE LIFT. CAPACITY (LBS.)	NUMBER	% OF TOTAL RESPONSE	AVERAGE LIFT. CAPACITY (LBS)
Crawler Mounted Crane (fixed boom)	-	-	-	-	-	-	1	5%	20,000
Truck Mounted Crane (fixed boom)	8	38%	28,500	-	-	-	2	10%	45,000
Self Propelled Crane (telescopic boom)	2	10%	20,000	1	5%	30,000	-	-	-
Wheel Tractor with Front loader	19	90%	12,400	-	-	-	-	-	-
Winch Trucks:									
< 5,000 lbs	3	14%	-	-	-	-	-	-	-
5,000 - 10,000 lbs	9	43%	-	-	-	-	-	-	-
10,001 - 15,000 lbs	3	14%	-	-	-	-	-	-	-
> 15,000 lbs	2	10%	-	-	-	-	1	5%	-
Other[a]									
Transport Trucks	18	86%	-	-	-	-	-	-	-

[a]One county reported owning a Gradall with a lifting capacity of 12,000 lbs.

County Personnel

Counties were surveyed to evaluate the experience level of county labor forces for concrete and bridge maintenance. The experience categories and compilation of responses is shown in Table XI. The responding counties reported that 52% have personnel with concrete and bridge maintenance experience between 1 and 5 years, and 57% with greater than 5 years. Seventy-six percent of responding counties have personnel with 1 year or more of experience. Five counties (24% of responding counties) reported having no personnel with concrete or bridge experience.

Based upon the survey of counties, interviews with county commissioners, and the results of the demonstration bridge, it is concluded that counties can successfully implement the deck replacement system developed in this research project.

TABLE XI

COUNTY PERSONNEL WITH CONCRETE AND BRIDGE MAINTENANCE EXPERIENCE

YEARS OF EXPERIENCE[a]	COUNTIES WITH PERSONNEL	
	NUMBER	% OF TOTAL RESPONSE
Less than 1	4	19%
1 to 5	11	52%
Greater than 5	12	57%

[a]Five, or 24% of the responding counties have no personnel with concrete and bridge experience.

It is recommended that if counties pursue this method of bridge deck replacement, a concrete casting bed be constructed with a surface as smooth and in-plane as possible to allow the panels to be cast in a traffic-side-up orientation. The most likely problem of implementing this system would be in placing the panels on the existing bridge. Typically, two front end loaders acting in parallel can handle the panels, however some bridge sites may have limited space which would hinder the process. Based on interviews with county commissioners, several responded that they would prefer leasing a crane for placing the panels on the bridge to ease construction and improve safety for the labor crews. However, evaluation of the survey data shows that many counties have front end loaders and winch trucks which would make the leasing of a crane unnecessary.

CHAPTER VII

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FURTHER RESEARCH

Summary

The purpose of this research project was to develop a system of precast deck panels which the counties of Oklahoma can implement with their own labor and equipment. The development of the system included data analysis of 1,752 county bridges selected for study. Based on the analysis, a family of panels was configured and designed for application to bridges meeting the study criteria.

All panels were designed for the AASHTO HS20 loading. The family of panels consisted of fourteen panels with two thicknesses, seven widths, and a standard 4 ft 0 in. length (parallel to traffic). The panels were designed to provide seven clear roadway widths as determined by the analysis of the study bridges. The clear roadway widths are 16 ft 0 in. to 28 ft 0 in. on 2 ft 0 in. increments. Two thicknesses of panels were designed to accommodate the wide variety of beam spacings on the existing study bridges. A 6 in. thick deck panel was designed for bridges with beam spacings of 4 ft 0 in. maximum and a 7 in. thick panel for beam spacings of 5 ft 6 in. maximum. The 6 in. panel can be used on 79% of the 1,752 study bridges whereas the 7 in. panel can be used on 97% of the bridges.

A demonstration bridge was constructed by Noble County, Oklahoma personnel to evaluate the constructability of the deck panel system. The bridge has a single span of 28 ft 0 in. and a clear roadway width of 27 ft 10 in. The clear roadway was designed as 26 ft 0 in. with the use of the TR-2 traffic rail, but Noble County selected a different type of rail. Seven 7 in. thick deck panels were used on the bridge with a beam spacing of 4 ft 2 in. The seven panels were cast at the county yard on rock quarry screenings in a traffic-side-down orientation. After 49 days the panels were loaded and transported to the bridge site. A crane was utilized to place the panels onto the steel beams of the demonstration bridge. The seven panels were placed, aligned, and connected in eight hours.

The construction requirements of the precast panel bridge deck were compared to a cast-in-place bridge deck, considering material and labor requirements. Material, labor, and equipment requirements were documented for the demonstration bridge. The same crew constructed a similar cast-in-place deck less than a year prior to the construction of the demonstration bridge. The labor required for the cast-in-place bridge deck was obtained from the county bridge file. Adjustments were made to the material and labor requirements for deck size and an experience curve was used to provide a valid comparison. The precast panel deck required more reinforcing steel and structural steel, but required only 83% of the labor hours required to construct the cast-in-place deck. Deck replacement time for the precast panel deck was two days compared to approximately one month for the cast-in-place deck.

A survey was conducted by mail to evaluate the ability of counties in Oklahoma to implement the construction and placement of the precast deck panels using county facilities, equipment and labor. Each of the seventy-seven counties was mailed a survey packet. Twenty-one counties responded resulting in a 27% return rate. The responding counties are geographically well distributed throughout the State of Oklahoma and provide a representative sample of all counties.

Conclusions

The panels of the demonstration bridge were cast with the traffic side down so the upper side of the panel could be finished to a smooth surface for bearing on the beams and to minimize the amount of required shimming. Since the traffic side was cast against the screenings there was no control of this surface. The concrete flowed underneath the flange along portions of the steel channels and subsequently chipped away causing an approximate $^5/_8$ in. variation in elevation. This variation causes a roughness when driving over the demonstration bridge, although the roughness does not exceed the roughness of the adjacent dirt road. This roughness can be eliminated on future panels if the panels are cast in the traffic side up orientation. However, if the panel is cast with the traffic side up, it is recommended that a concrete casting bed be constructed so that a planar bottom surface of the panel will be produced which will provide bearing of the panel on the beams and to minimize shimming. A more desirable driving texture and smoother driving panels can also be obtained. A concrete casting bed is recommended for counties interested in pursuing this method of deck replacement.

The seven panels used on the demonstration bridge were designed to produce a total bridge length of 27 ft $11^5/_8$ in. This length is the sum of seven 3 ft $11^5/_8$ in. panels and six $^3/_8$ in. joints. The as-built length of the bridge is 28 ft $2^1/_2$ in. The additional $2^7/_8$ in. are due to slightly out-of-square panels caused by a combination of welding, handling, and casting concrete into the steel channel frames. At some connection plate locations filler plates were needed to fill the joint for connection of adjacent panels to the bridge. To minimize the problem of the out-of-square panels, it is recommended that the panel frames be completely fabricated at the steel fabrication plant before delivery to the county. A steel fabricator has facilities for alignment of the channels for squareness. Based upon experience with the demonstration bridge plans, the original design of the channel frame has been modified to include cross-bracing to brace the frame during handling and transporting. Also, cross-bracing will keep the frame square during concrete placement. This process should improve panel quality and minimize the potential for growth in bridge deck length due to out-of-square panels.

The original deck panel design for the demonstration bridge had the top longitudinal reinforcing bars (parallel to traffic) different from the bottom longitudinal bars. This is typical of most bridge slabs. Since the field crew has the option of casting a panel in a traffic-side-up or traffic-side-down orientation, the potential exists for workers to inadvertently reverse the upper and lower longitudinal reinforcing steel. This error occurred on panels 1 through 5 of the demonstration bridge. Although the longitudinal reinforcing steel was reversed on panels 1 through 5, the concrete cover over the top mat (traffic side) and bottom mat were correct. The decision was made to place the panels as originally planned with the traffic surface cast against the rock screenings, thus providing a concrete cover over the top mat of steel of 2 in. instead of 1 in. Panels 1 through 5 do not meet the AASHTO [2] section which addresses distribution of reinforcement. This discrepancy is minimal because the panels were designed for a maximum beam spacing of 5 ft 6 in. and distribution reinforcement is a percentage of the required positive moment steel. Also, the panels were designed to span the 3 ft $11^5/_8$ in. dimension in a shimmed condition. With the reversed longitudinal reinforcing steel, panels 1 through 5 are inadequate to span the 3 ft $11^5/_8$ in. dimension in a shimmed condition because the required tension steel is located in the top of the panel. This discrepancy was corrected by placing a shim at the center of the panel. Based upon the experience with the demonstration bridge the design plans were modified to include identical steel for the top and bottom longitudinal reinforcing steel.

Recommendations for Further Research

Laboratory load testing of the panels designed in this research should be performed to provide a comparison of laboratory produced stresses to the calculated design stresses. The panel edges were designed using an effective width philosophy and moment distribution based on the moment of inertia of the steel channel with an assumed effective width of concrete, reference Chapter III. An actual effective width and moment distribution could be evaluated more precisely through laboratory testing. The results of such testing could lead to the modification of reinforcing in the panels.

The design of a connection which would consider composite action with the beams without increasing construction difficulty should be investigated. This may also improve the load ratings of some existing bridges.

Additional research utilizing lightweight aggregates and concretes should be performed to evaluate weight reduction of the panels. The concrete mix should be evaluated for weight reduction, strength characteristics, durability, and other factors.

Admixtures should be evaluated for use in the concrete for these panels. Such admixtures could possibly create a more dense wearing surface which would be resistant to the scouring effects of sand and gravel, which is typical for rural traffic conditions. The use of high early strength concrete could improve the turnaround time if a casting bed was utilized that could handle only a minimum number of panels, or if stack casting is necessary or desirable.

Consideration should be given to a co-operative effort between counties to construct and utilize a simple pre-stressing facility with a casting bed and bulkheads for constructing prestressed panels. The equipment required to perform the jacking operations could be purchased or leased.

Post-tensioning is an alternate method for fabrication of the panels that should be evaluated. The simplest method would be the use of threaded bars. This method could be performed at the facilities of individual counties. A portion of the post tensioning may be performed after the panels are on the bridge to tie the panels together.

The panel to beam connection used in the demonstration bridge was designed for ease of construction so workers would not be required to go underneath the deck to complete the connection. The design of the connection requires a $^3/_8$ in. space between panels to accommodate a $^3/_8$ in. connector plate. This joint provides drainage of surface water from the bridge but allows it to drain on the beams below. This creates a potential problem of corrosion of the beams. Alternate connection methods and joint details including sealants could be evaluated.

Most county bridges are narrow structures. Several counties in Oklahoma have policies that require a minimum of two lanes for all replacement bridges. With these policies in effect, methods for widening existing bridges should be developed to utilize existing structurally adequate members of the bridge. Such methods should include widening of abutments and wings, widening of piers, and the addition of beams.

One of the primary problems with bridges on the county road system is the age and poor condition of the bridge substructures which have low condition ratings. Many are founded on timber piling of unknown conditions. Many have timber abutments and piers in poor condition. Additional research is needed to develop methods for improving these elements of county bridges to a satisfactory condition at minimal expense to the counties.

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APPENDIX A

DECK PANEL STRUCTURAL CALCULATIONS

APPENDIX A NOMENCLATURE

A	effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars
A_s	area of tension reinforcement
a	depth of equivalent rectangular stress block
b	width of compression face of member
d	distance from extreme compression fiber to centroid of tension reinforcement
d_c	thickness of concrete cover measured from extreme tension fiber to center of bar located closest thereto
E	effective width of concrete section
F_y	specified minimum yield stress of steel
f_c'	specified compressive strength of concrete
f_r	modulus of rupture of concrete
f_s	tensile stress in reinforcement at service loads
I	impact fraction
I_{cr}	moment of inertia of cracked section transformed to concrete
I_{eff}	Effective moment of inertia for computation of stiffness and deflection
I_g	moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
L	span length
M_a	maximum moment in member at stage for which deflection is being computed
M_{cr}	cracking moment

M_D	moment due to dead load
M_{LL}	moment due to live load
M_U	factored moment
n	modular ratio of elasticity
P_{20}	16,000 pound wheel load
S	section modulus
S_{eff}	effective span length
w	load per unit length or area
X	distance in feet from load to support
y_t	distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension
z	quantity limiting distribution of flexural reinforcement
δ	deflection
ϕ	strength reduction factor

6 INCH THICK PANEL DESIGN

DESIGN:

AASHTO 1989 - 14th ed.

LOADING:

HS20-44 with 20 psf future wearing surface.

MATERIALS:

Concrete - ODOT Class AA, f_c' = 3,500 psi minimum.
Steel Reinforcing - Grade 60
Structural Steel - A36

TYPE OF DESIGN:

Load Factor Design for reinforced concrete.
Service Load Design for structural steel.

APPLICATION:

4 ft 0 in. maximum beam spacing.

DESIGN PROCEDURE:

The deck panels were designed to provide clear roadway widths of 16 ft 0 in. to 28 ft 0 in. (2 ft 0 in. increments) with 2 ft 5 in. of panel width occupied by the TR-2 traffic rail. They were designed to act independently thus requiring no connections to provide load transfer from panel to panel. Figure A.1 shows the deck panel plan. The panels were designed using a three step process. In Step 1 the panels were analysed based on the equations in AASHTO section 3.24. Step 2 considers a detailed analysis of loading an edge of the panel. Step 3 considers the effects of shimming the panel to provide full bearing. The maximum effect in an element of the panel produced by the three steps is used in design. The final panel design is a combination of the three steps.

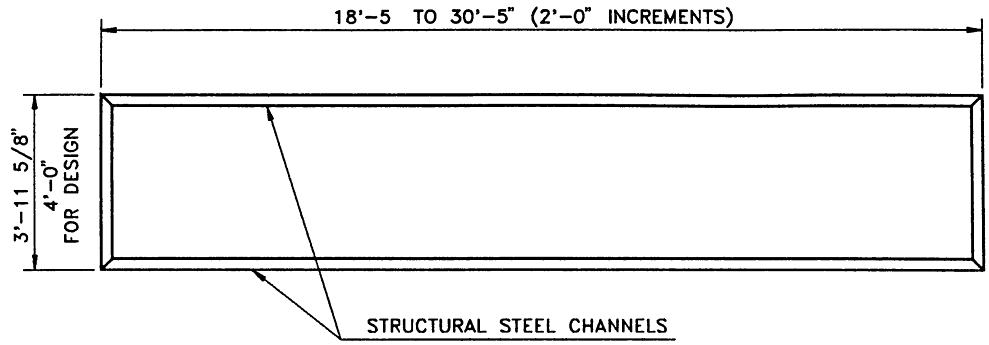


Figure A.1. Panel Plan

STEP 1) DESIGN PER AASHTO SECTION 3.24:

Figure 3.1 in Chapter 3 shows beam flange widths for the bridges considered in this study. A flange width of 6 in. was selected for design purposes.

Calculation of Moments:

$M_D = .8[wS_{eff}^2/8]$ (continuity factor = .8)

$w = 95$ psf (slab @ 150 pcf + future wearing surface)

$S_{eff} = 4.0$ ft - ((6 in.)/12)/2 = 3.75 ft (AASHTO 3.24.1.2)

$M_D = 0.134$ ft-kips/ft

$M_{LL+I} = .8(I)[(S_{eff} + 2)/32](P_{20})$ (AASHTO 3.24.3.1)

$I = 1.3$

$S_{eff} = 3.75$ ft

$P_{20} = 16$ kips

$M_{LL+I} = 2.990$ ft-kips/ft

$M_U = 1.3[M_D + 1.67(M_{LL+I})]$ (AASHTO 3.22)

$M_U =$ <u>6.67 ft-kips/ft</u>

Design for Bending Moments:

Try #4 bars @ 6-1/2 in.

Concrete cover for Top Bars = 1-1/2 in.

Concrete cover for Bottom Bars = 1 in.

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$

$\phi = 0.9$

$A_s = 0.37\ in^2/ft$

$d_{pos\ moment} = 4.75$ in.

$d_{neg\ moment} = 4.25$ in. (controls)

$a = (A_s F_y)/(.85 f_c'(b)) = 0.62$ in. (b = 12 in.)

$M_{U(provided)} =$ <u>6.56 ft-kips/ft</u> (1.7% overstress - OK)

Distribution Reinforcement (AASHTO 3.24.10):

Percentage $= 220/(S_{eff})^{0.5} \leq 67\%$

$= 114$ USE 67%

$A_s = 0.67(0.37) = 0.25\ in^2/ft$

Results of Step 1):

<u>#4 bars @ 6-1/2 in. Top and Bottom Transverse</u>

<u>#4 bars @ 9 in. Bottom Longitudinal</u>

<u>#4 bars @ 18 in. Top Longitudinal</u>

<u>STEP 2) DESIGN OF PANELS FOR EDGE LOADING</u>:

AASHTO section 3.24.9 states that a transverse edge of a bridge deck slab shall not be unsupported. This requirement is satisfied by utilizing a steel channel connected to the edge of the deck panel acting compositely with an effective width of concrete. A more exact analysis is performed in this step using the tire contact area of a 16 kip wheel, reference Figure A.2, placed along the edges of the deck panel. Figure A.3 illustrates the deflected shape of a deck panel with a 16 kip wheel load placed at the edge of the panel between support beams.

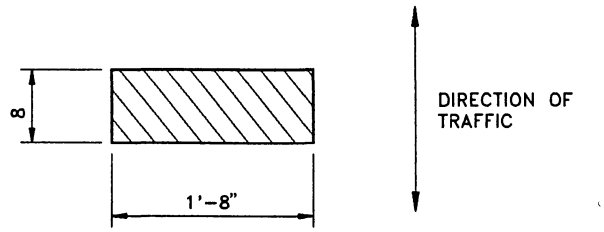


Figure A.2. Tire Contact Area of a 16 kip Wheel Load (AASHTO 3.30)

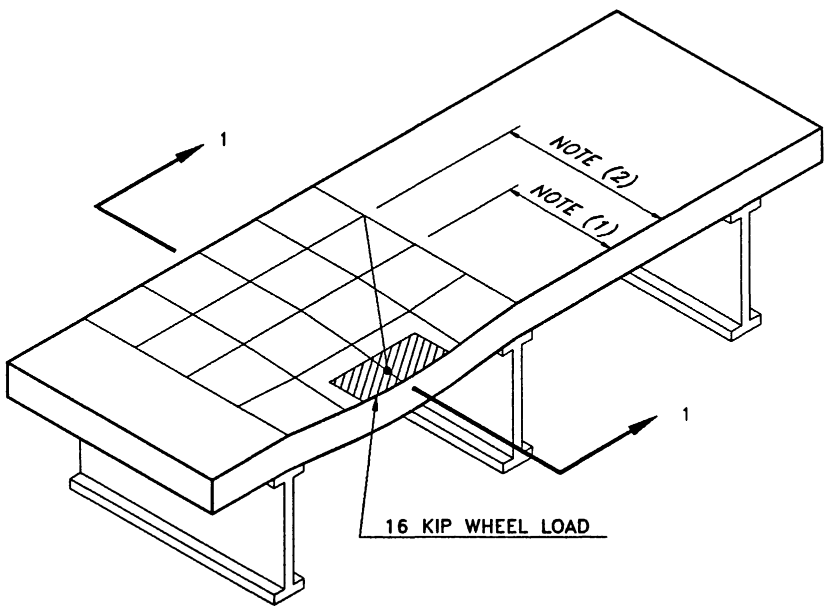


Figure A.3. Deflected Shape of Panel Due to Edge Loading

Note (1): Effective portion of panel width for positive moment resistance is assumed to be 2 ft 0 in. This assumption is based on edge loading small scale wood models of the panel and observing the deflection pattern.

Note (2): Effective portion of panel width for negative moment resistance is assumed to be half that provided by AASHTO 3.24.5.1.1 plus 4 in. (center of tire contact area to edge of slab).

E = [(0.8(X) + 3.75 ft)/2] + (4 in.)/12

E = [(0.8(2.0 ft) + 3.75 ft)/2] + .333 ft

= 3.0 ft

Calculation of Bending Moments:

Calculation of both maximum positive and negative moments in the deck panel due to edge loading was performed using computer aided structural analysis. The dimensions shown in Figures A.4 and A.5 are for both the actual bridge goemetry and the computer model used in analysis. The dimensions in parentheses are those used in the computer model to account for the use of an effective slab span of 3.75 ft as allowed by AASHTO. The uniform load shown will provide a 16 kip wheel loading to the structure in both cases.

Positive Moment Calculation:

Figure A.4 illustrates the loading condition for maximum positive bending moment due to edge loading. Maximum positive moment occurs at 0.4(length of span) of span A-B.

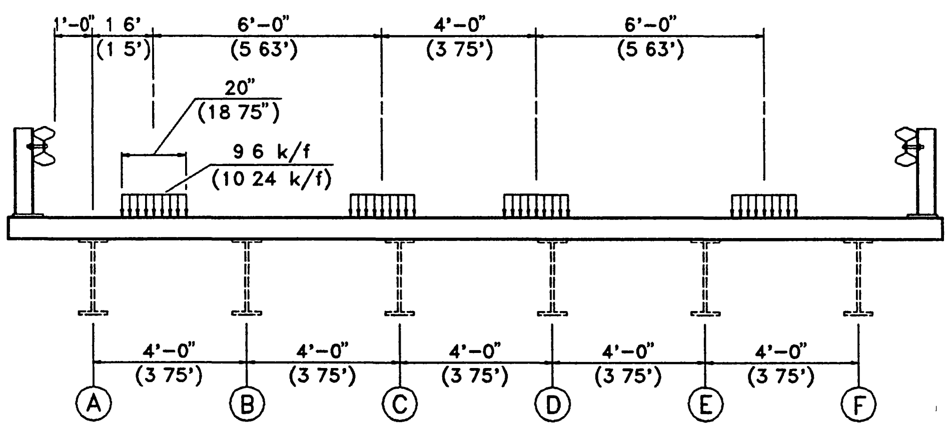


Figure A.4. Load Condition for Maximum Positive Moment

$M_{D(pos)}$ = (0.134 ft-kips)(2 ft) = 0.268 ft-kips

$M_{LL(pos)}$ = 8.99 ft-kips

$M_{LL+I(pos)}$ = 1.3(8.99) = 11.69 ft-kips

$M_{U(pos)}$ = 1.3(0.268 + 1.67(11.69)) = <u>25.73 ft-kips</u>

Negative Moment Calculation:

Figure A.5 illustrates the loading condition for maximum negative bending moment due to edge loading. Maximum negative moment occurs at support C.

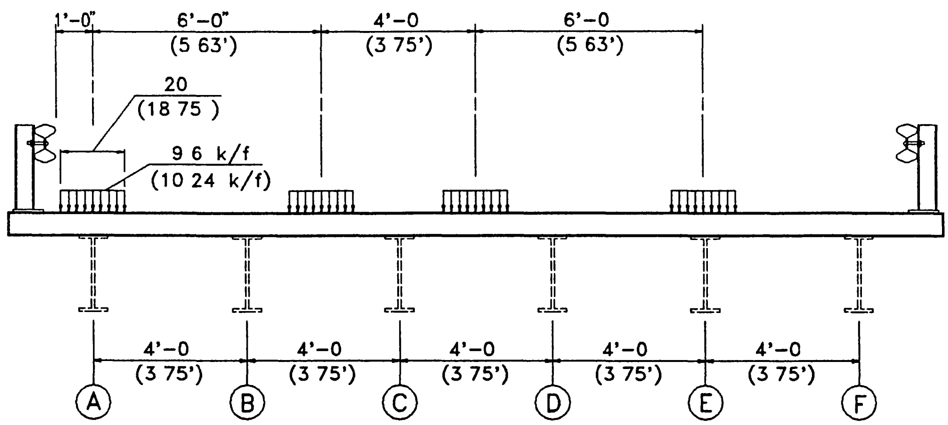


Figure A.5. Load Condition for Maximum Negative Moment

$M_{D(neg)}$ = (-0.134 ft-kips)(3 ft) = -0.402 ft-kips

$M_{LL(neg)}$ = -8.99

$M_{LL+I(neg)}$ = 1.3(-8.99) = -11.69 ft-kips

$M_{U(neg)}$ = 1.3(-0.402 + 1.67(-11.69)) = <u>-25.90 ft-kips</u>

Design For Positive Moment From Edge Loading:

As shown in Figure A.3, the effective concrete for resisting positive moment due to edge loading is 2 ft 0 in. Figure A.6 part a) shows Section 1-1 from Figure A.3 and

Figure A.6 part b) shows the model of Section 1-1 used in the panel design calculations. Positive moment is resisted by the steel channel (Ch), concrete section (1), and concrete section (2). Total positive moment is distributed to each of the three sections using stiffness and deflection compatibility. The steel channel is transformed into an equivalent section of concrete for calculations.

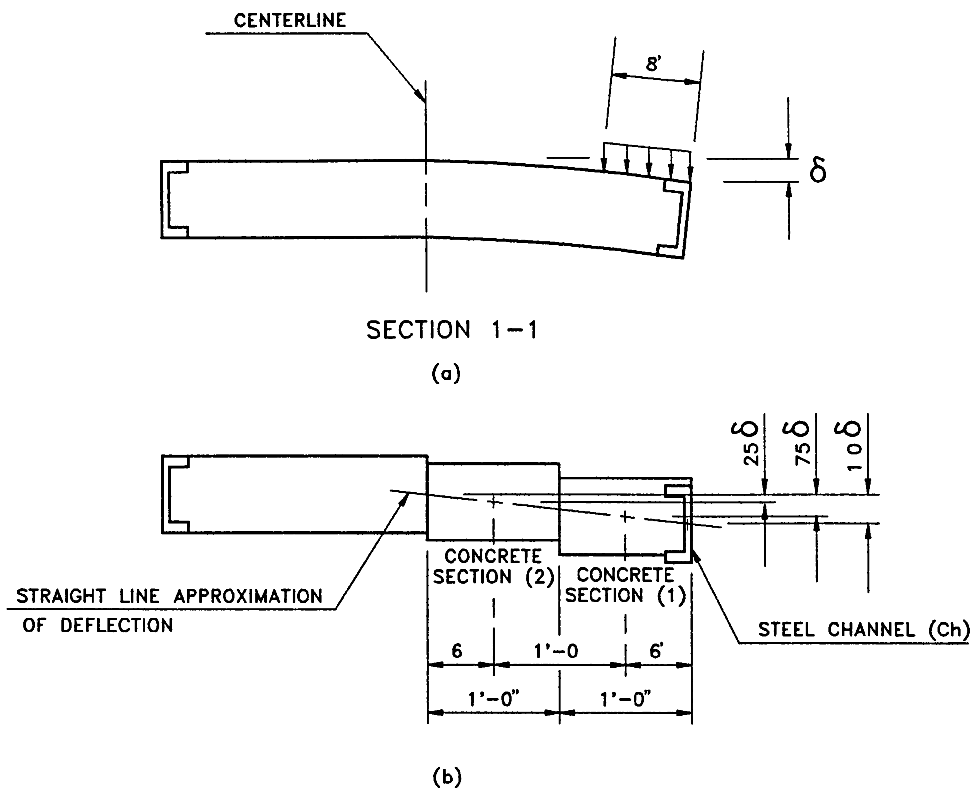


Figure A.6. a) Section 1-1 of Figure A.3 Showing Deflection of Panel Due to Edge Loading
b) Straight Line Approximation Used in Design

Effective Moments Of Inertia:

Steel Channel (Ch) - C6 X 8.2:

$I_{eff(Ch)} = I_g(n)$

$I_g = 13.1 \text{ in}^4$

$n = 9$

$I_{eff(Ch)} = 118 \text{ in}^4$

For calculation of I_{eff} in concrete sections (1) and (2), try the reinforcing steel pattern shown in Figure A.7.

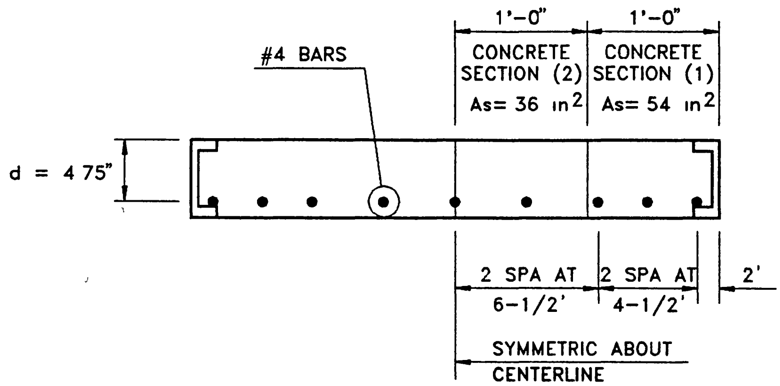


Figure A.7. Reinforcing Steel Pattern for Calculation of I_{eff} for Concrete Sections (1) and (2)

Concrete Section (1):

$$I_{eff(1)} = (M_{cr}/M_{a(1)})^3(I_g) + [1 - (M_{cr}/M_{a(1)})^3]I_{cr} \leq I_g$$

$M_{cr} = f_r(I_g)/y_t$

$f_r = 7.5 \quad f'c = 444$ psi

$I_g = 216$ in^4

$y_t = 3$ in

$M_{cr} = 2.66$ ft-kips

$I_{cr} = b(a^3)/3 + n(A_s)(d - a)^2$

$a = ((2(d)(B) + 1)^{0.5} - 1)/(B)$ (Reference [11])

$B = b/(nA_s) = 2.47$

$a = 1.60$ in.

$I_{cr} = 65$ in^4

$$I_{eff(1)} = (2.66/M_{a(1)})^3(216) + [1-(2.66/M_{a(1)})^3](65) \leq 216 \qquad \textbf{[EQ1]}$$

Concrete Section (2):

$$I_{eff(2)} = (M_{cr}/M_{a(2)})^3(I_g) + [1 - (M_{cr}/M_{a(2)})^3]I_{cr} \leq I_g$$

$I_g = 216$ in^4

$M_{cr} = 2.66$ ft-kips

$I_{cr} = 47$ in^4

$$I_{eff(2)} = (2.66/M_{a(2)})^3(216) + [1-(2.66/M_{a(2)})^3](47) \leq 216 \qquad \textbf{[EQ2]}$$

Distribution of Positive Moment:

From Figure A.6 part b) it can be seen that:

$\delta_{(C)} = 1.333\delta_{(1)} = 4\delta_{(2)}$

Therefore, with stiffness and deflection compatibility:

$M_{a(Ch)}/I_{eff(Ch)} = 1.333M_{a(1)}/I_{eff(1)} = 4M_{a(2)}/I_{eff(2)}$ **[EQ3]**

$M_{U(pos)} = 25.73$ ft-kips

$M_{a(Ch)} + M_{a(1)} + M_{a(2)} = 25.73$ ft-kips

Try: $M_{a(Ch)} = 15.07$ ft-kips
$M_{a(1)} = 7.03$ ft-kips
$M_{a(2)} = 3.63$ ft-kips

Substitution of $M_{a(1)}$ and $M_{a(2)}$ into **[EQ1]** and **[EQ2]** yeilds:

$I_{eff(1)} = 73.2 \text{ in}^4$
$I_{eff(2)} = 113.5 \text{ in}^4$

From Equation **[EQ3]**:

$15.07/118 = 1.333(7.03)/73.2 = 4(3.63)/113.5$

$0.128 = 0.128 = 0.128$ (equality)

Therefore, for the Steel Channel (Ch): M_U = <u>15.07 ft-kips</u>
Concrete Section (1): M_U = <u>7.03 ft-kips</u>
Concrete Section (2): M_U = <u>3.63 ft-kips</u>

Check Capacity:

Check Stress in Steel Channel:

Find service moment in channel

$M_{(channel)} = (15.07/25.73)(11.69 + 0.268) = 7.00$ ft-kips

Stress = M/S = (7.00 ft-kips)(12)/(4.38 in^3) = <u>19.18 ksi</u>

Allowable stress = $0.55(F_y) = 0.55(36) = 20$ ksi (OK)

Check Ultimate Moment Capacity of Concrete Section (1):

M_U = <u>7.03 ft-kips</u>

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$

$\phi = 0.9$

$A_s = 0.54\ in^2$

$d = 4.75$ in.

$a = (A_s F_y)/(.85 f_c'(b)) = 0.91$ in (b = 12 in.)

$M_U = .9(.54)(60)(4.75 - (0.91/2)) =$ 10.44 ft-kips (OK)

Check Crack Control in Concrete Section (1):

$z = f_s(d_c(A))^{0.33}$

$f_s = .6(60) = 36$ ksi

$d_c = 1.25$ inches

$A = 2(1.25)(12)/(2.70\ bars/ft) = 11.11$

$z =$ 87 < 130 (severe exposure) (OK)

Check Ultimate Moment Capacity of Concrete Section (2):

$M_U =$ 3.63 ft-kips

$M_{U(provided)} =$ 7.20 ft-kips

Check Crack Control in Concrete Section (2):

$z =$ 99 < 130 (severe exposure) (OK)

Design for Negative Moment from Edge Loading:

From Figure A.3, the effective width of concrete section (3) used in design for resisting negative bending from edge loading is 3 ft 0 in. Assume no deflection of panel at bridge beam support.

Effective Moments of Inertia:

Steel Channel (ch) - C6 × 8.2:

$I_{eff(Ch)} = 118\ in^4$

For calculation of I_{eff} in concrete section (3), try the reinforcing steel pattern shown in Figure A.8.

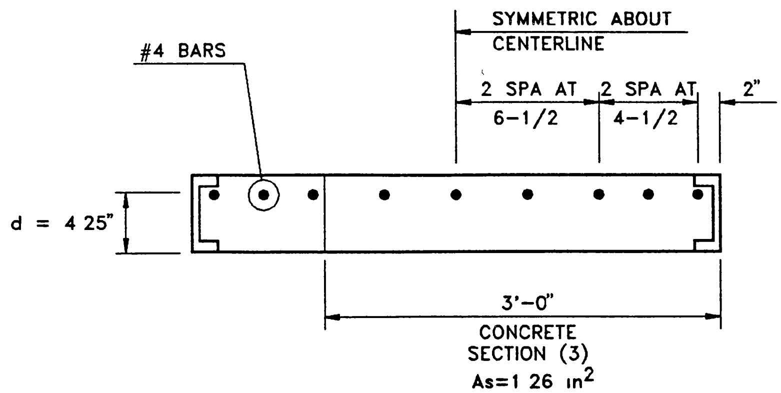


Figure A.8. Reinforcing Steel Pattern for Calculation of I_{eff} for Concrete Section (3)

Concrete Section (3):

$$I_{eff(3)} = (M_{cr}/M_{a(3)})^3(I_g) + [1 - (M_{cr}/M_{a(3)})^3]I_{cr} \leq I_g$$

$I_g = 648 \text{ in}^4$

$M_{cr} = 7.99$ ft-kips

$I_{cr} = 125 \text{ in}^4$

$$I_{eff(3)} = (7.99/M_{a(3)})^3(648) + [1-(7.99/M_{a(3)})^3](125) \leq 648 \quad \textbf{[EQ4]}$$

Distribution Of Negative Moment:

$M_{U(neg)} = -25.90$ ft-kips

$M_{a(Ch)} + M_{a(3)} = -25.90$ ft-kips

With $\delta_{(Ch)} = \delta_{(3)}$

$$M_{a(Ch)}/I_{eff(ch)} = M_{a(3)}/I_{eff(3)} \quad \textbf{[EQ5]}$$

Try: $M_{a(Ch)} = -9.86$ ft-kips

$M_{a(3)} = -16.04$ ft-kips

Substitution of $M_{a(3)}$ into equation **[EQ4]** yields:

$I_{eff(3)} = 190 \text{ in}^4$

From Equation **[EQ5]**:

$-9.86/118 = -16.04/190$

$-0.084 = -0.084$ (equality)

Therefore, for the Steel Channel (Ch): M_U = <u>-9.86 ft-kips</u>
Concrete Section (3): M_U = <u>-16.04 ft-kips</u>

Check Capacity:

Check Stress in Steel Channel:

Negative moment is less than positive moment. (OK)

Check Ultimate Moment Capacity of Concrete Section (3):

M_U = <u>-16.04 ft-kips</u>

$M_{U(provided)}$ = <u>-22.10 ft-kips</u> (OK)

Check Crack Control of Concrete Section (3):

z = <u>118</u> < 130 (severe exposure) (OK)

Results of Step 2):

<u>#4 bars spaced as shown in Figures A.8 and A.9</u>

<u>C6 × 8.2 Steel Channel</u>

<u>STEP 3) DESIGN FOR A SHIMMED CONDITION</u>:

In many instances of deck replacement on existing county bridges with steel beams, the top flange of the beams will not provide an in-plane table top surface to insure full bearing of the concrete deck panel on all beams. Placing shims between the steel channels of the deck panels and bridge beams may be required to provide full bearing. Figure A.9 part a) shows the arrangement of shims and the wheel location which produces maximum bending moment in a shimmed condition.

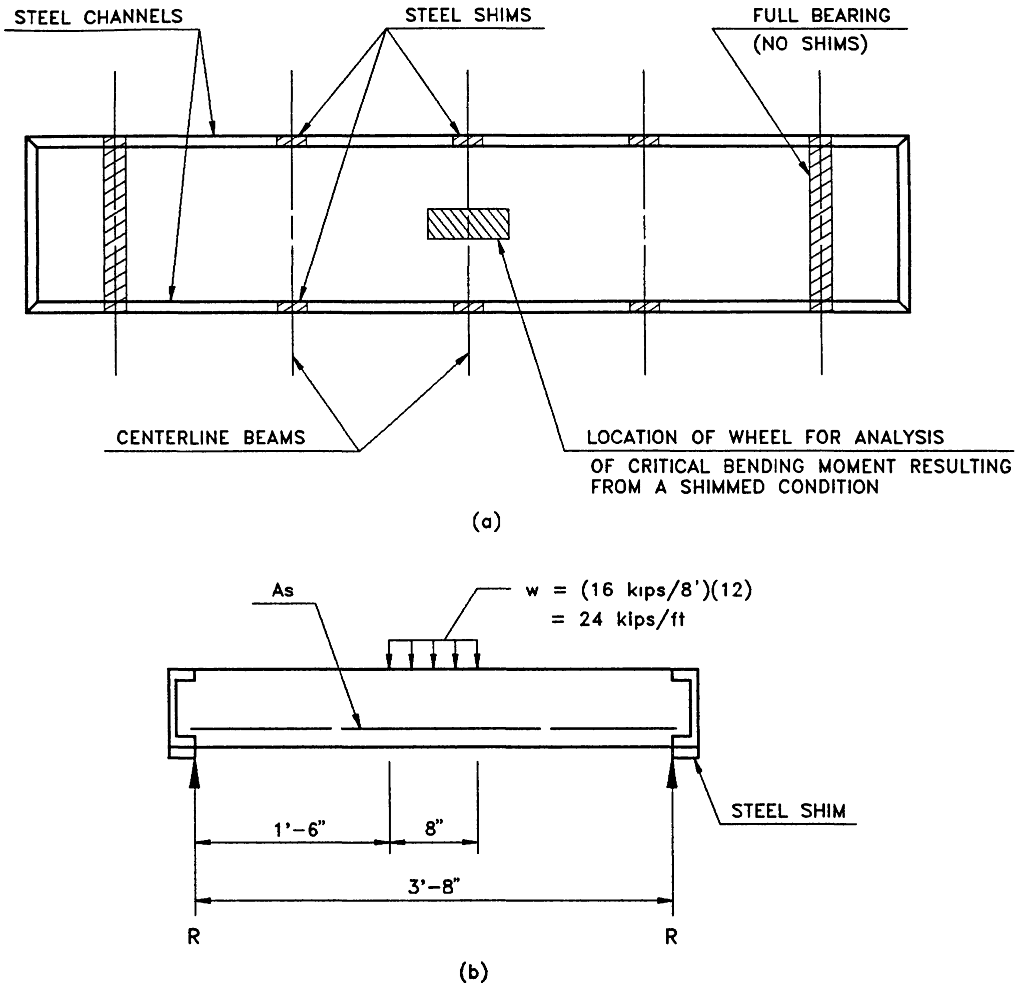


Figure A.9. a) Arrangement of Shims and Wheel Load Which Produces Maximum Bending Moment in a Shimmed Condition
b) Section Through Panel Showing Wheel Load

Calculation of Bending Moment:

Distribution Width (AASHTO 3.24.3.2)

$(4 + 0.06(3.67\text{ ft})) = 4.22\text{ ft}$

Moment Calculation:

$M_D = 0.160$ ft-kips/ft

$M_{LL+I} = 17.33$ ft-kips/4.22 ft = 4.11 ft-kips/ft

$M_U = 1.3[M_D + 1.67(M_{LL+I})]$
$M_U = \underline{9.13 \text{ ft-kips/ft}}$

Design for Bending Moment:

Try #5 bars @ 7 in.

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$
$\phi = 0.9$
$A_s = 0.54 \text{ in}^2/\text{ft}$
$d = 4.19 \text{ in}$
$a = (A_s F_y)/(.85f'c(b))$
$= 0.53(60)/(.85)(3.5)(12) = 0.89 \text{ in}$
$M_{U(provided)} = \underline{8.93 \text{ ft-kips/ft}}$ (2.2% overstress - OK)

Check Crack Control:

$z = \underline{129} < 130$ (severe exposure) (OK)

Results of Step 3):

<u>#5 bars at 7 in. Bottom Longitudinal</u>

<u>6 INCH THICK PANEL FINAL DESIGN</u>:

The final design detail for the 6 in. thick panel is shown in Figure A.10.

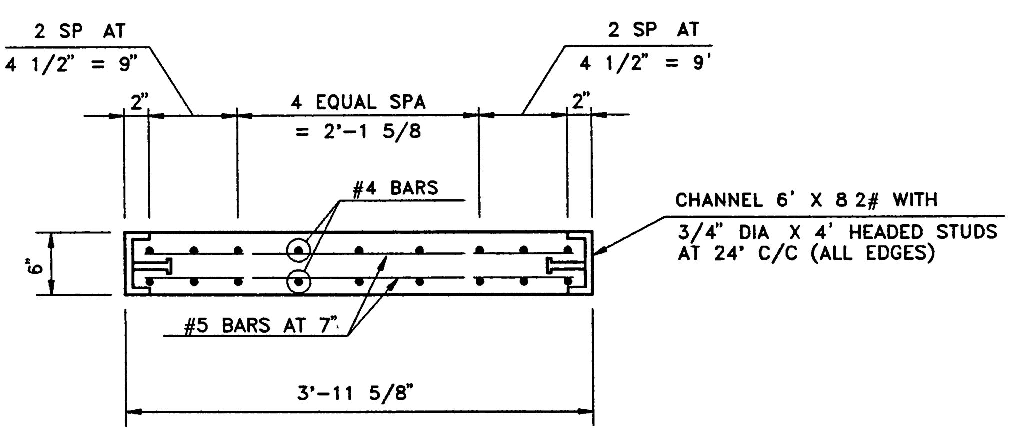


Figure A.10. 6 Inch Thick Panel Detail

7 INCH THICK PANEL DESIGN

DESIGN:

AASHTO 1989 - 14th ed.

LOADING:

HS20-44 with 20 psf future wearing surface.

MATERIALS:

Concrete - ODOT Class AA, f_c' = 3,500 psi minimum.
Steel Reinforcing - Grade 60
Structural Steel - A36

TYPE OF DESIGN:

Load Factor Design for reinforced concrete.
Service Load Design for structural steel.

APPLICATION:

5 ft 6 in. maximum beam spacing.

DESIGN PROCEDURE:

The deck panels were designed to provide clear roadway widths of 16 ft 0 in. to 28 ft 0 in. (2 ft 0 in. increments) with 2 ft 5 in. of panel width occupied by the TR-2 traffic rail. They were designed to act independently thus requiring no connections to provide load transfer from panel to panel. Figure A.11 shows the deck panel plan. The panels were designed using a three step process. In Step 1 the panels were analysed based on the equations in AASHTO section 3.24. Step 2 considers a detailed analysis of loading an edge of the panel. Step 3 considers the effects of shimming the panel to provide full bearing. The maximum effect in an element of the panel produced by the three steps is used in design. The final panel design is a combination of the three steps.

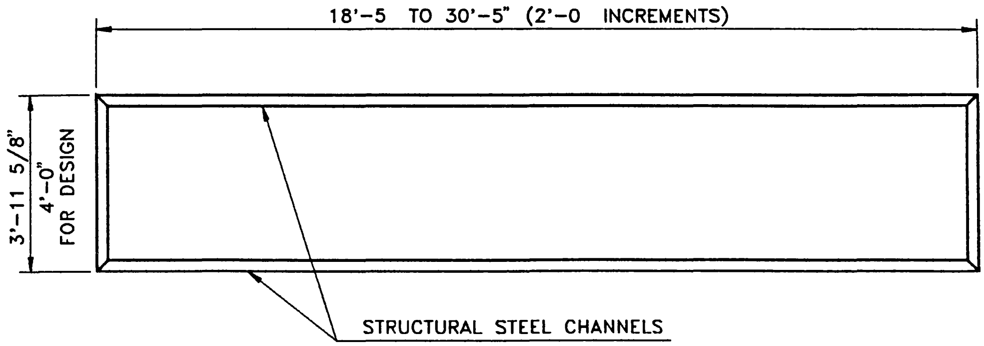


Figure A.11. Panel Plan

STEP 1) DESIGN PER AASHTO SECTION 3.24:

Figure 3.1 in Chapter 3 shows beam flange widths for the bridges considered in this study. A flange width of 6 in. was selected for design purposes.

Calculation of Moments:

$M_D = .8[wS_{eff}^2/8]$ (continuity factor = .8)

$w = 108$ psf (slab @ 150 pcf + future wearing surface)

$S_{eff} = 5.5 \text{ ft} - ((6 \text{ in.})/12)/2 = 5.25$ ft (AASHTO 3.24.1.2)

$M_D = 0.298$ ft-kips/ft

$M_{LL+I} = .8(I)[(S_{eff} + 2)/32](P_{20})$ (AASHTO 3.24.3.1)

$I = 1.3$

$S_{eff} = 5.25$ ft

$P_{20} = 16$ kips

$M_{LL+I} = 3.770$ ft-kips/ft

$M_U = 1.3[M_D + 1.67(M_{LL+I})]$ (AASHTO 3.22)

$M_U = \underline{8.57 \text{ ft-kips/ft}}$

Design for Bending Moments:

Try #5 bars @ 8 in.

Concrete cover for Top Bars = 2 in.

Concrete cover for Bottom Bars = 1 in.

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$

$\phi = 0.9$

$A_s = 0.47\ in^2/ft$

$d_{pos\ moment} = 5.69$ in.

$d_{neg\ moment} = 4.69$ in. (controls)

$a = (A_s F_y)/(.85f_c'(b)) = 0.79$ in.

$M_{U(provided)} =$ 9.08 ft-kips/ft (OK)

Distribution Reinforcement (AASHTO 3.24.10):

Percentage $= 220/(S_{eff})^{0.5} \leq 67\%$

$= 96$ USE 67%

$A_s = 0.67(0.47) = 0.31\ in^2/ft$

Results of Step 1):

#5 bars @ 8 in. Top and Bottom Transverse

#4 bars @ 6 in. Bottom Longitudinal

#4 bars @ 18 in. Top Longitudinal

STEP 2) DESIGN OF PANELS FOR EDGE LOADING:

AASHTO section 3.24.9 states that a transverse edge of a bridge deck slab shall not be unsupported. This requirement is satisfied by utilizing a steel channel connected to the edge of the deck panel acting compositely with an effective width of concrete. A more exact analysis is performed in this step using the tire contact area of a 16 kip wheel, reference Figure A.12, placed along the edges of the deck panel. Figure A.13 illustrates the deflected shape of a deck panel with a 16 kip wheel load placed at the edge of the panel between support beams.

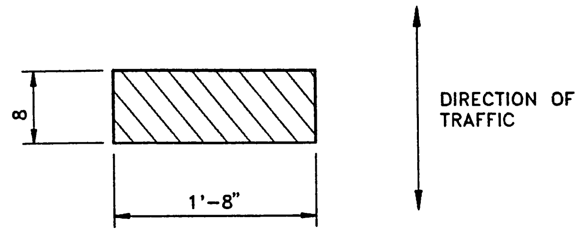


Figure A.12. Tire Contact Area of a 16 kip Wheel Load (AASHTO 3.30)

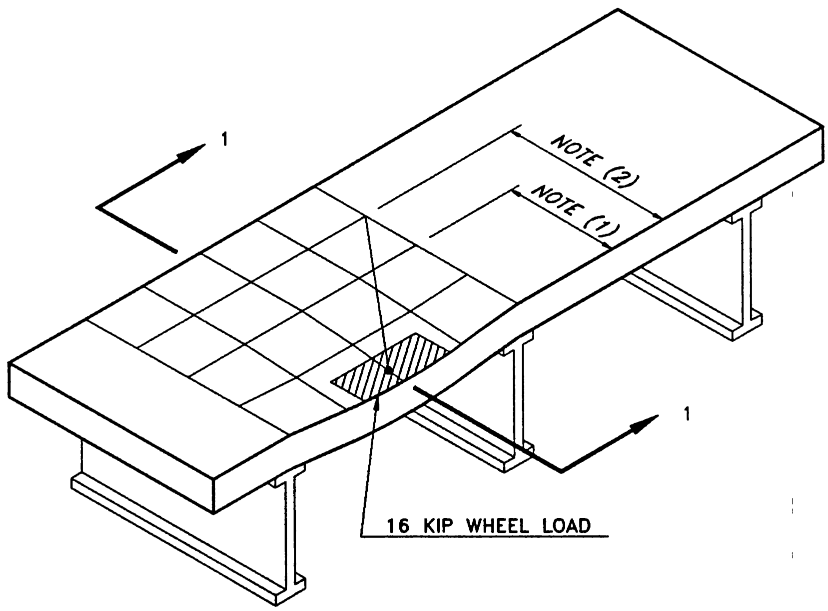


Figure A.13. Deflected Shape of Panel Due to Edge Loading

Note (1): Effective portion of panel width for positive moment resistance is assumed to be 2 ft 0 in. This assumption is based on edge loading small scall wood models of the panel and observing the deflection pattern.

Note (2): Effective portion of panel width for negative moment resistance is assumed to be half that provided by AASHTO 3.24.5.1.1 plus 4 in. (center of tire contact area to edge of slab).

$$E = [(0.8(X) + 3.75\ \text{ft})/2] + (4\ \text{in.})/12$$

$$E = [(0.8(2.75\ \text{ft}) + 3.75\ \text{ft})/2] + .333\ \text{ft}$$

$$= 3.31\ \text{ft} \quad \text{Use } 3.0\ \text{ft for design}$$

Calculation of Bending Moments:

Calculation of both maximum positive and negative moments in the deck panel due to edge loading was performed using computer aided structural analysis. The dimensions shown in Figures A.14 and A.15 are for both the actual bridge goemetry and the computer model used in analysis. The dimensions in parentheses are those used in the computer model to account for the use of an effective slab span of 5.25 ft as allowed by AASHTO. The uniform load shown will provide a 16 kip wheel loading to the structure in both cases.

Positive Moment Calculation:

Figure A.14 illustrates the loading condition for maximum positive bending moment due to edge loading. Maximum positive moment occurs at 0.4 × (length of span) of span A-B.

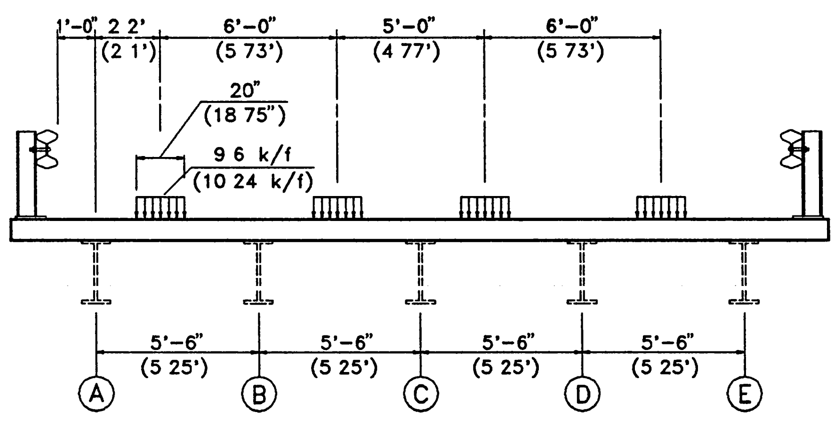


Figure A.14. Load Condition for Maximum Positive Moment

$M_{D(pos)}$ = (0.298 ft-kips)(2 ft) = 0.596 ft-kips

$M_{LL(pos)}$ = 12.14 ft-kips

$M_{LL+I(pos)}$ = 1.3(12.14) = 15.78 ft-kips

$M_{U(pos)}$ = 1.3(0.596 + 1.67(15.78)) = <u>35.03 ft-kips</u>

Negative Moment Calculation:

Figure A.15 illustrates the loading condition for maximum negative bending moment due to edge loading. Maximum negative moment occurs at support B.

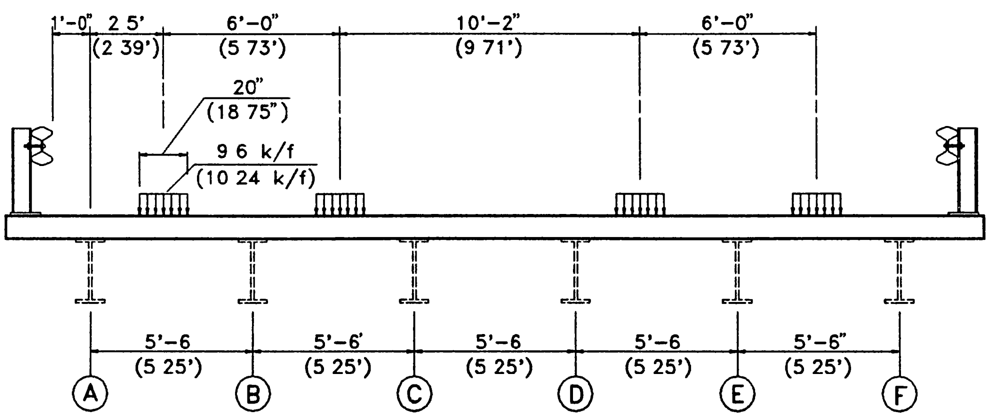


Figure A.15. Load Condition for Maximum Negative Moment

$M_{D(neg)}$ = (-0.298 ft-kips)(3 ft) = -0.894 ft-kips

$M_{LL(neg)}$ = -13.72

$M_{LL+I(neg)}$ = 1.3(-13.72) = -17.84 ft-kips

$M_{U(neg)}$ = 1.3(-0.894 + 1.67(-17.84)) = <u>-39.89 ft-kips</u>

Design For Positive Moment From Edge Loading:

As shown in Figure A.13, the effective concrete for resisting positive moment due to edge loading is 2 ft 0 in. Figure A.16 part a) shows Section 1-1 from Figure A.13 and

Figure A.16 part b) shows the model of Section 1-1 used in the panel design calculations. Positive moment is resisted by the steel channel (Ch), concrete section (1), and concrete section (2). Total positive moment is distributed to each of the three sections using stiffness and deflection compatibility. The steel channel is transformed into an equivalent section of concrete for calculations.

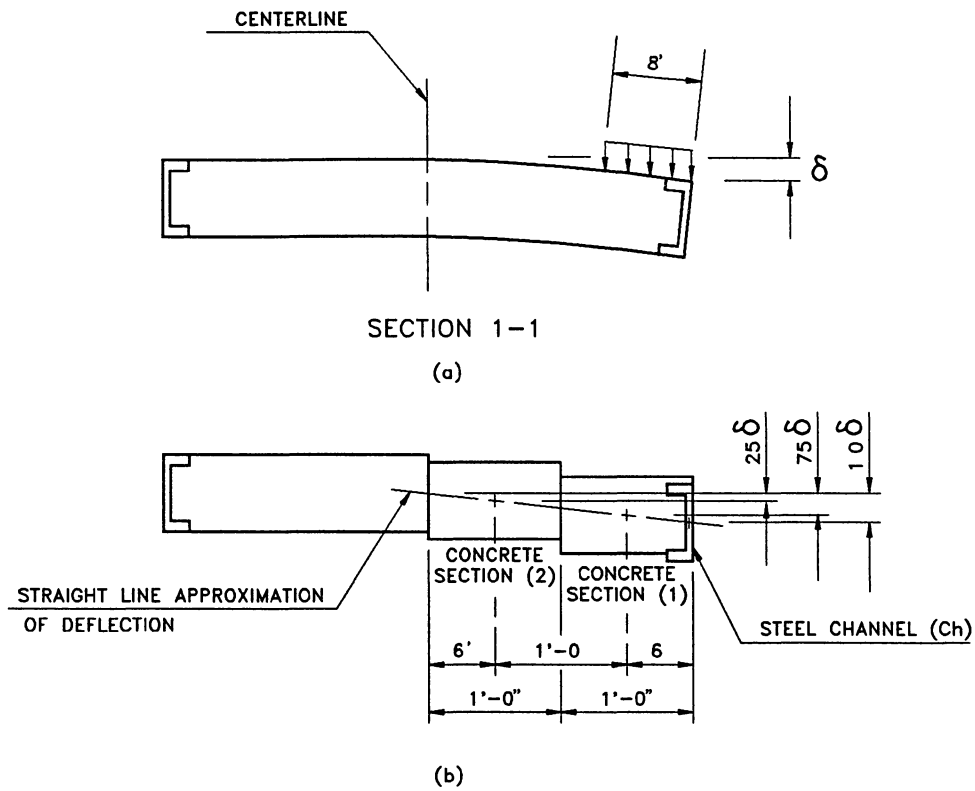


Figure A.16. a) Section 1-1 of Figure A.3 Showing Deflection of Panel Due to Edge Loading
b) Straight Line Approximation Used in Design

Effective Moments Of Inertia:

Steel Channel (Ch) - C7 X 9.8:

$I_{eff(Ch)} = I_g(n)$

$I_g = 21.3 \text{ in}^4$

$n = 9$

$I_{eff(Ch)} = 192 \text{ in}^4$

For calculation of I_{eff} in concrete sections (1) and (2), try the reinforcing steel pattern shown in Figure A.17.

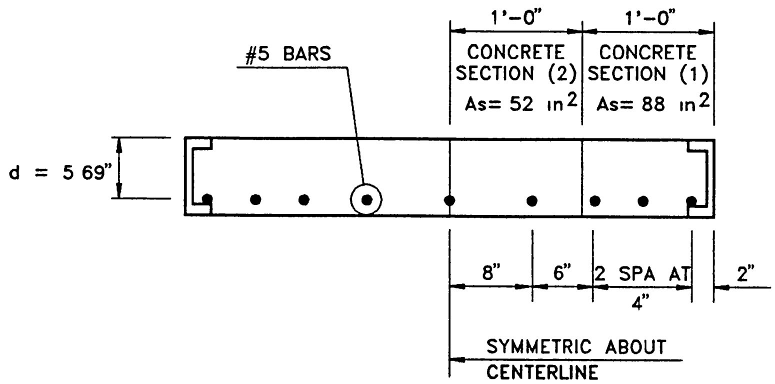


Figure A.17. Reinforcing Steel Pattern for Calculation of I_{eff} for Concrete Sections (1) and (2)

Concrete Section (1):

$$I_{eff(1)} = (M_{cr}/M_{a(1)})^3(I_g) + [1 - (M_{cr}/M_{a(1)})^3]I_{cr} \leq I_g$$

$M_{cr} = f_r(I_g)/y_t$

$f_r = 7.5 \sqrt{f'c} = 444$ psi

$I_g = 343\ in^4$

$y_t = 3.5$ in.

$M_{cr} = 3.63$ ft-kips

$I_{cr} = b(a^3)/3 + n(A_s)(d - a)^2$

$a = ((2(d)(B) + 1)^{0.5} - 1)/(B)$ (Reference [11])

$B = b/(nA_s) = 1.52$

$a = 2.16$ in.

$I_{cr} = 139\ in^4$

$$I_{eff(1)} = (3.63/M_{a(1)})^3(343) + [1-(3.63/M_{a(1)})^3](139) \leq 343 \quad \textbf{[EQ1]}$$

Concrete Section (2):

$$I_{eff(2)} = (M_{cr}/M_{a(2)})^3(I_g) + [1 - (M_{cr}/M_{a(2)})^3]I_{cr} \leq I_g$$

$I_g = 343\ in^4$

$M_{cr} = 3.63$ ft-kips

$I_{cr} = 94\ in^4$

$$I_{eff(2)} = (3.63/M_{a(2)})^3(343) + [1-(3.63/M_{a(2)})^3](94) \leq 343 \quad \textbf{[EQ2]}$$

Distribution of Positive Moment:

From Figure A.16 part b) it can be seen that:

$\delta_{(C)} = 1.333\delta_{(1)} = 4\delta_{(2)}$

Therefore, with stiffness and deflection compatibility:

$M_{a(Ch)}/I_{eff(Ch)} = 1.333M_{a(1)}/I_{eff(1)} = 4M_{a(2)}/I_{eff(2)}$ **[EQ3]**

$M_{U(pos)} = 35.03$ ft-kips

$M_{a(Ch)} + M_{a(1)} + M_{a(2)} = 35.03$ ft-kips

Try: $M_{a(Ch)} = 19.14$ ft-kips
$M_{a(1)} = 11.00$ ft-kips
$M_{a(2)} = 4.89$ ft-kips

Substitution of $M_{a(1)}$ and $M_{a(2)}$ into **[EQ1]** and **[EQ2]** yeilds:

$I_{eff(1)} = 146.3\ in^4$
$I_{eff(2)} = 196.5\ in^4$

From Equation **[EQ3]**:

$19.14/192 = 1.333(11.00)/146.3 = 4(4.89)/196.5$

$0.100 = 0.100 = 0.100$ (equality)

Therefore, for the Steel Channel (Ch): M_U = <u>19.14 ft-kips</u>
Concrete Section (1): M_U = <u>11.00 ft-kips</u>
Concrete Section (2): M_U = <u>4.89 ft-kips</u>

Check Capacity:

Check Stress in Steel Channel:

Find service moment in channel

$M_{(channel)} = (19.14/35.03)(15.78 + 0.596) = 8.95$ ft-kips

Stress = M/S = (8.95 ft-kips)(12)/(6.08 in^3) = <u>17.66 ksi</u>

Allowable stress = $0.55(F_y) = 0.55(36) = 20$ ksi (OK)

Check Ultimate Moment Capacity of Concrete Section (1):

M_U = <u>11.00 ft-kips</u>

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$

$\phi = 0.9$

$A_s = 0.88\ in^2$

$d = 5.69$ in.

$a = (A_s F_y)/(.85 f_c'(b)) = 1.48$ in. (b = 12 in.)

$M_U = .9(.88)(60)(5.69 - (1.48/2)) = \underline{19.60\ ft\text{-}kips}$ (OK)

Check Crack Control in Concrete Section (1):

$z = f_s(d_c(A))^{0.33}$

$f_s = .6(60) = 36$ ksi

$d_c = 1.31$ in.

$A = 2(1.31)(12)/(2.84\ bars/ft) = 11.07$

$z = \underline{88} < 130$ (severe exposure) (OK)

Check Ultimate Moment Capacity of Concrete Section (2):

$M_U = \underline{4.89\ ft\text{-}kips}$

$M_{U(provided)} = \underline{12.29\ ft\text{-}kips}$ (OK)

Check Crack Control in Concrete Section (2):

$z = \underline{105} < 130$ (severe exposure) (OK)

Design for Negative Moment from Edge Loading:

From Figure A.13, the effective width of concrete section (3) used in design for resisting negative bending from edge loading is 3 ft 0 in. Assume no deflection of panel at bridge beam support.

Effective Moments of Inertia:

Steel Channel (ch) - C7 × 9.8:

$I_{eff(Ch)} = 192\ in^4$

For calculation of I_{eff} in concrete section (3), try the reinforcing steel pattern shown in Figure A.18.

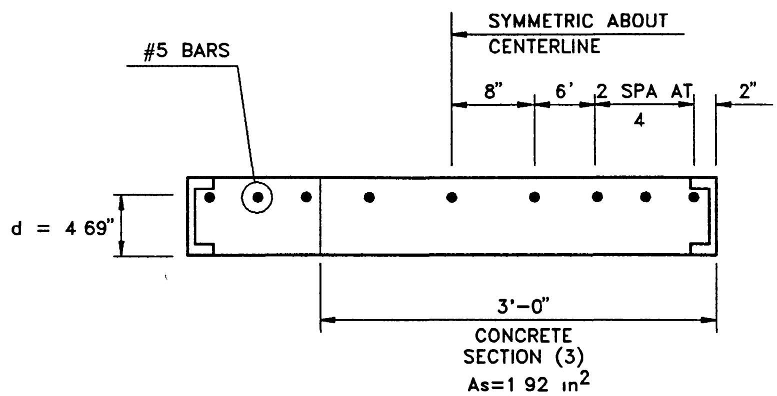


Figure A.18. Reinforcing Steel Pattern for Calculation of I_{eff} for Concrete Section (3)

Concrete Section (3):

$I_{eff(3)} = (M_{cr}/M_{a(3)})^3(I_g) + [1 - (M_{cr}/M_{a(3)})^3]I_{cr} \leq I_g$

$I_g = 1029 \text{ in}^4$

$M_{cr} = 10.9$ ft-kips

$I_{cr} = 213 \text{ in}^4$

$I_{eff(3)} = (10.9/M_{a(3)})^3(1029) + [1-(10.9/M_{a(3)})^3](213) \leq 1029$ **[EQ4]**

Distribution Of Negative Moment:

$M_{U(neg)} = -39.89$ ft-kips

$M_{a(Ch)} + M_{a(3)} = -39.89$ ft-kips

With $\delta_{(Ch)} = \delta_{(3)}$

$M_{a(Ch)}/I_{eff(ch)} = M_{a(3)}/I_{eff(3)}$ **[EQ5]**

Try: $M_{a(Ch)} = -15.90$ ft-kips

$M_{a(3)} = -23.99$ ft-kips

Substitution of $M_{a(3)}$ into equation **[EQ4]** yields:

$I_{eff(3)} = 289.1 \text{ in}^4$

From Equation [EQ5]:

$-15.90/192 = -23.99/289.1$

$-0.083 = -0.083$ (equality)

Therefore, for the Steel Channel (Ch): M_U = <u>-15.90 ft-kips</u>
Concrete Section (3): M_U = <u>-23.99 ft-kips</u>

Check Capacity:

Check Stress in Steel Channel:

Negative moment is less than positive moment. (OK)

Check Ultimate Moment Capacity of Concrete Section (3):

M_U = <u>-23.99 ft-kips</u>

$M_{U(provided)}$ = <u>-35.88 ft-kips</u> (OK)

Check Crack Control of Concrete Section (3):

z = <u>142</u> $\nless$ 130 but < 170 (moderate exposure) (Say OK)

Results of Step 2):

<u>#5 bars spaced as shown in Figures A.8 and A.9</u>

<u>C7 × 9.8 Steel Channel</u>

<u>STEP 3) DESIGN FOR A SHIMMED CONDITION</u>:

In many instances of deck replacement on existing county bridges with steel beams, the top flange of the beams will not provide an in-plane table top surface to insure full bearing of the concrete deck panel on all beams. Placing shims between the steel channels of the deck panels and bridge beams may be required to provide full bearing. Figure A.19 part a) shows the arrangement of shims and the wheel location which produces maximum bending moment in a shimmed condition.

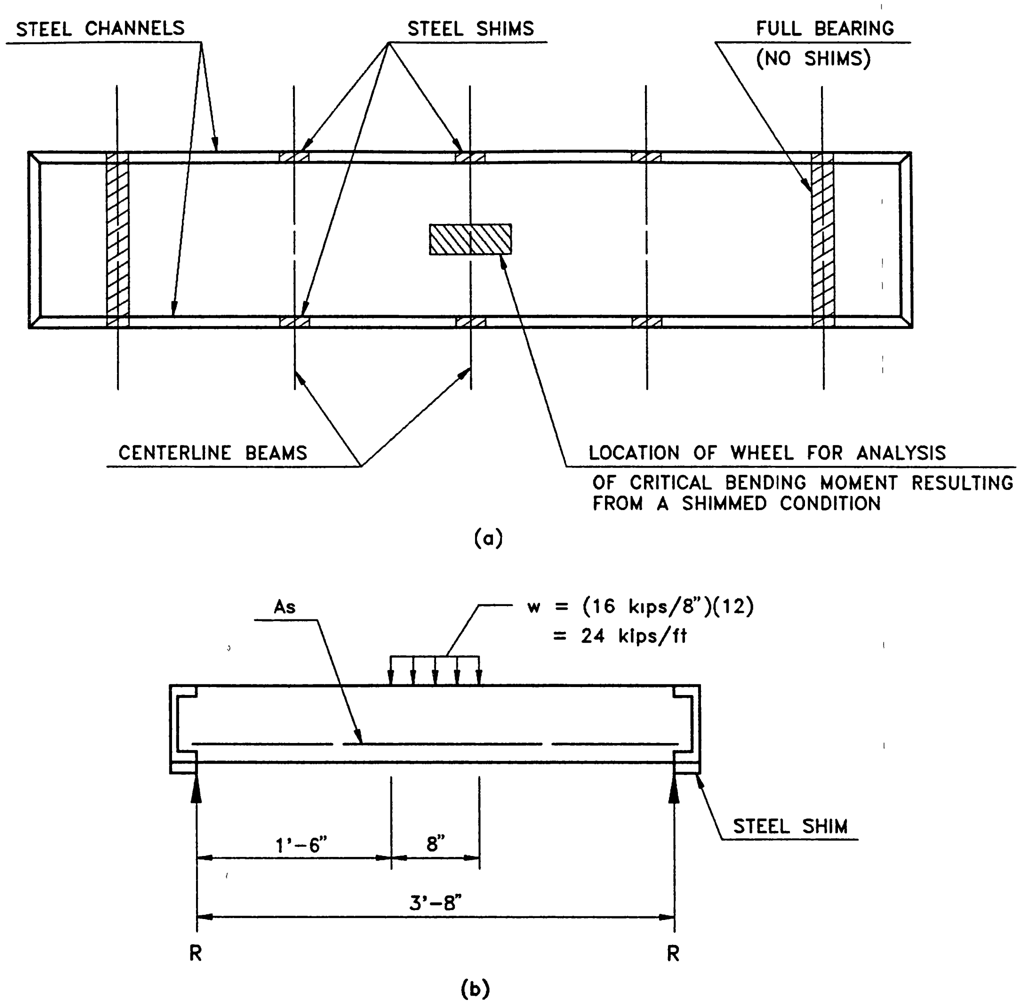


Figure A.19. a) Arrangement of Shims and Wheel Load Which Produces Maximum Bending Moment in a Shimmed Condition
b) Section Through Panel Showing Wheel Load

Calculation of Bending Moment:

Distribution Width (AASHTO 3.24.3.2)

$$(4 + 0.06(3.67\text{ ft})) = 4.22\text{ ft}$$

Moment Calculation:

M_D = 0.182 ft-kips/ft

M_{LL+I} = 17.33 ft-kips/4.22 ft = 4.11 ft-kips/ft

$M_U = 1.3[M_D + 1.67(M_{LL+I})]$

$M_U = \underline{9.16 \text{ ft-kips/ft}}$

Design for Bending Moment:

Try #4 bars @ 6 in.

$M_{U(provided)} = \phi A_s F_y(d - (a/2))$

$\phi = 0.9$

$A_s = 0.40 \text{ in}^2/\text{ft}$

$d = 5.13$ in.

$a = (A_s F_y)/(.85f'c(b))$

$= 0.40(60)/(.85)(3.5)(12) = 0.67$ in.

$M_{U(provided)} = \underline{8.63 \text{ ft-kips/ft}}$ (6.0% overstress - Say OK)

Check Crack Control:

$z = \underline{126} < 130$ (severe exposure) (OK)

Results of Step 3):

<u>#4 bars at 6 in. Bottom Longitudinal</u>

<u>7 INCH THICK PANEL FINAL DESIGN</u>:

The final design detail for the 7 in. thick panel is shown in Figure A.20.

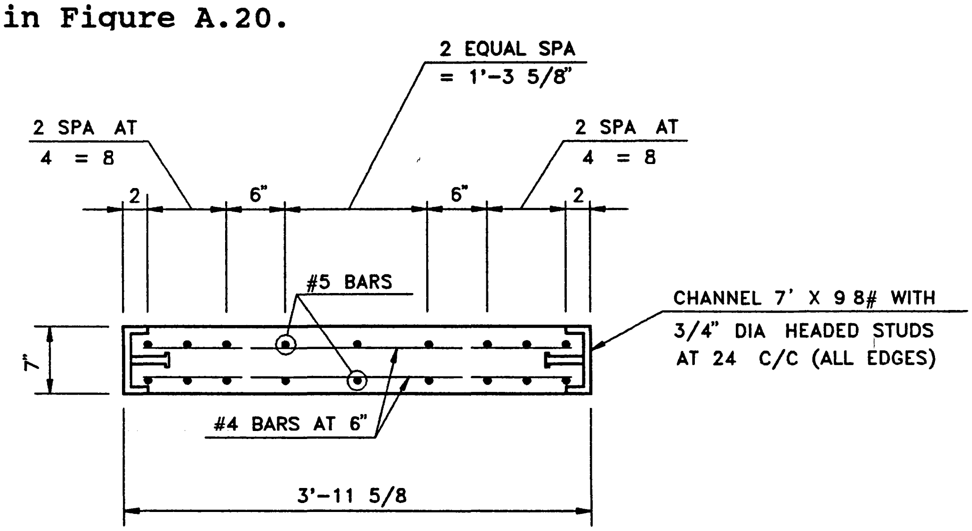
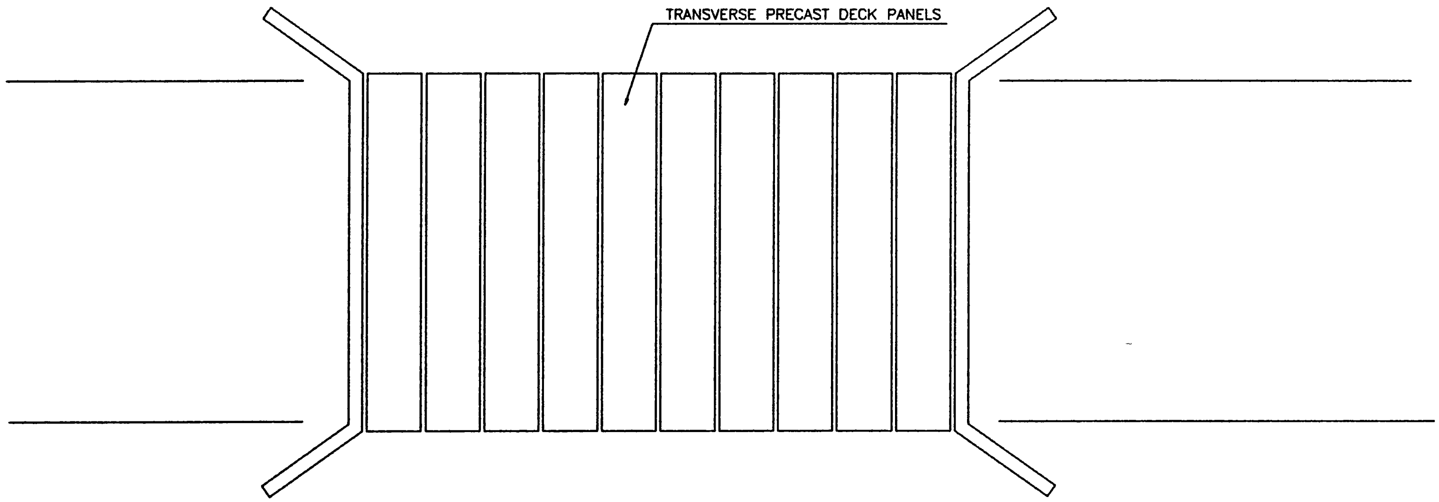


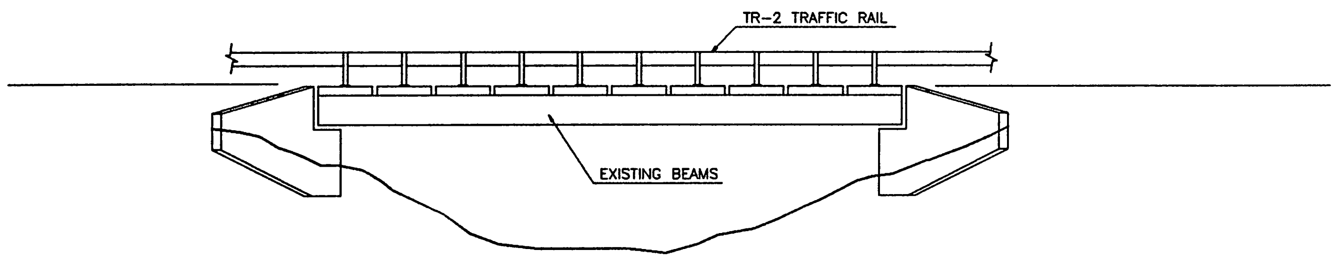
Figure A.20. 7 Inch Thick Panel Detail

APPENDIX B

DECK PANEL DRAWINGS

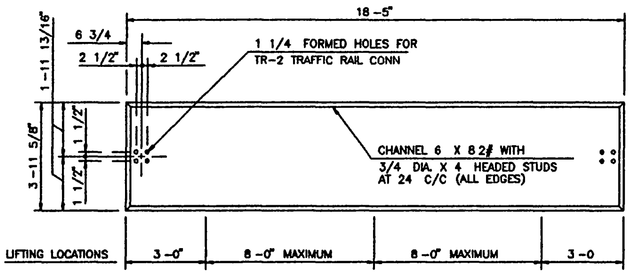


PLAN

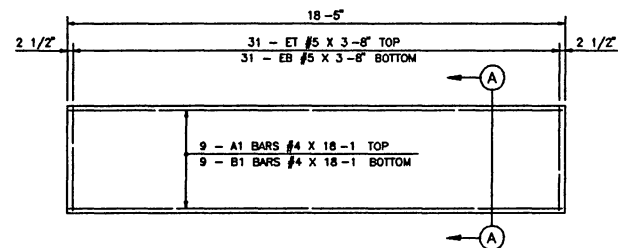


ELEVATION

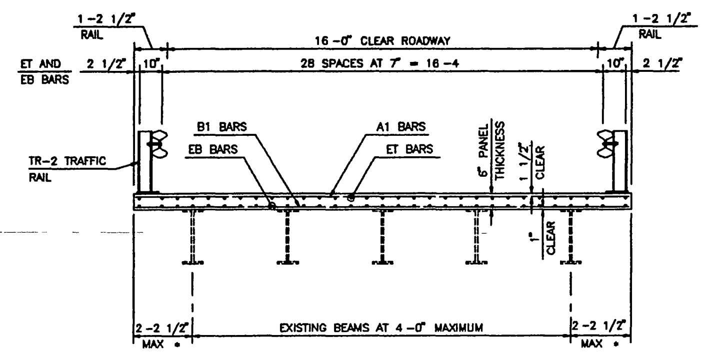
DESIGNED	GLF	PLAN AND ELEVATION
DRAWN	GLF	PRECAST DECK PANEL BRIDGE
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

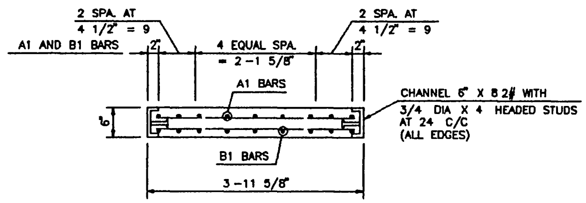


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	18 -1
B1	#4	9	STR	18 -1
ET	#5	31	STR	3 -8
EB	#5	31	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1.35
REINFORCING STEEL (GRADE 60)	LB	455
STRUCTURAL STEEL A36	LB	405

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 16 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL.
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

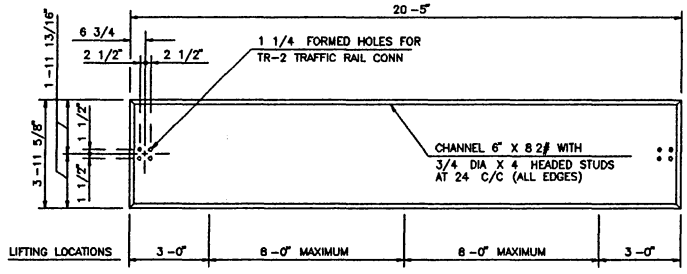
LIFTING NOTES

TOTAL PANEL WEIGHT = 6 330 LBS

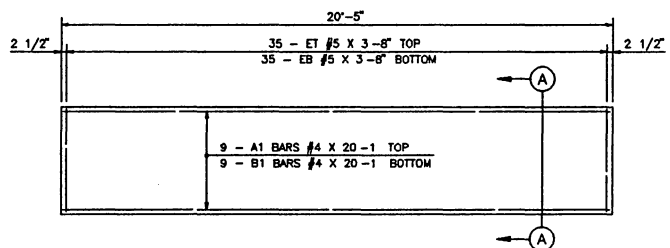
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

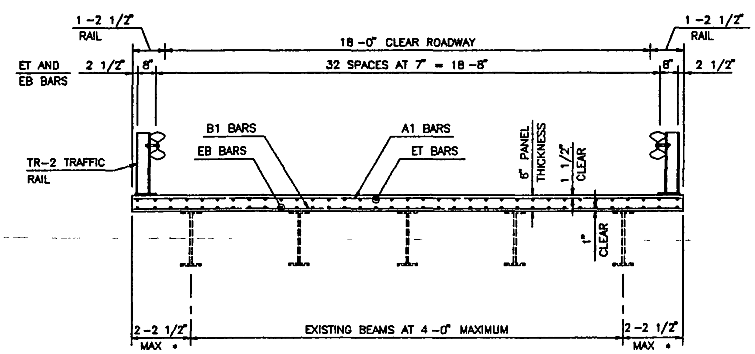
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 16 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

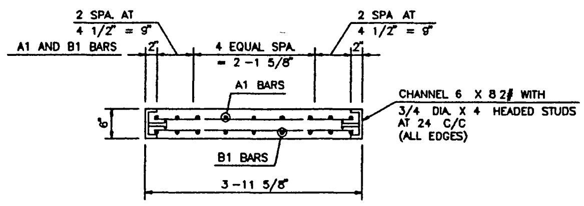


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	20 -1
B1	#4	9	STR	20 -1
ET	#5	35	STR	3 -8
EB	#5	35	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1 50
REINFORCING STEEL (GRADE 60)	LB	509
STRUCTURAL STEEL A36	LB	437

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 18 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

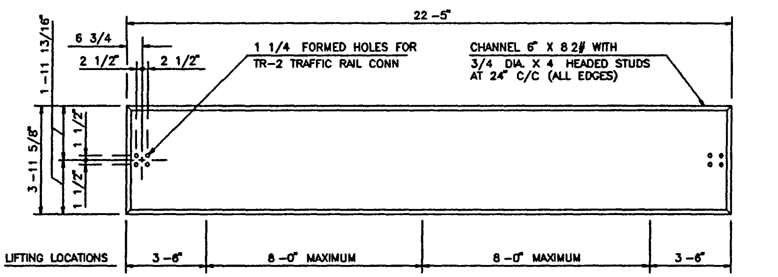
LIFTING NOTES

TOTAL PANEL WEIGHT = 7 020 LBS

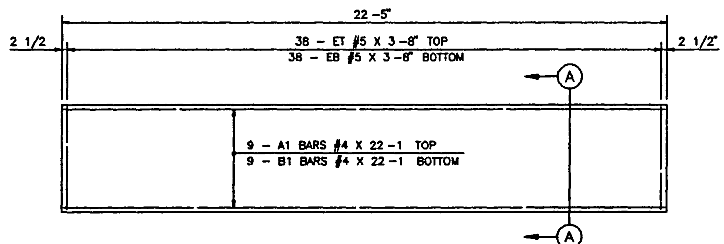
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

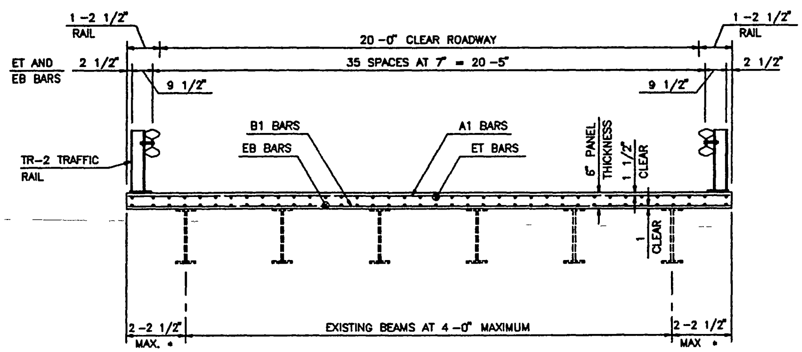
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 18 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

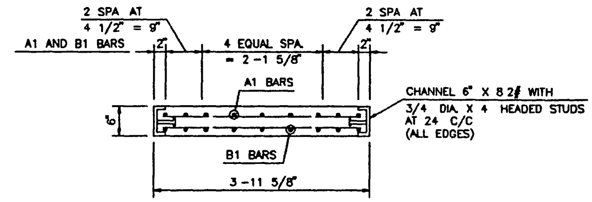


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	22 -1"
B1	#4	9	STR	22 -1
ET	#5	38	STR	3 -8
EB	#5	38	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1 65
REINFORCING STEEL (GRADE 60)	LB	556
STRUCTURAL STEEL A36	LB	488

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 20 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL.
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

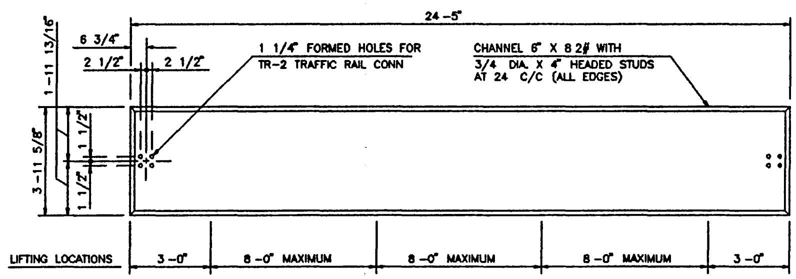
LIFTING NOTES

TOTAL PANEL WEIGHT = 7 730 LBS

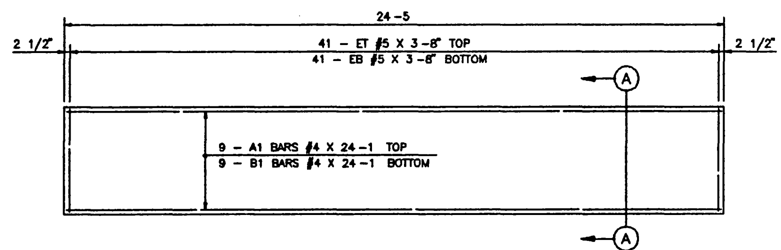
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

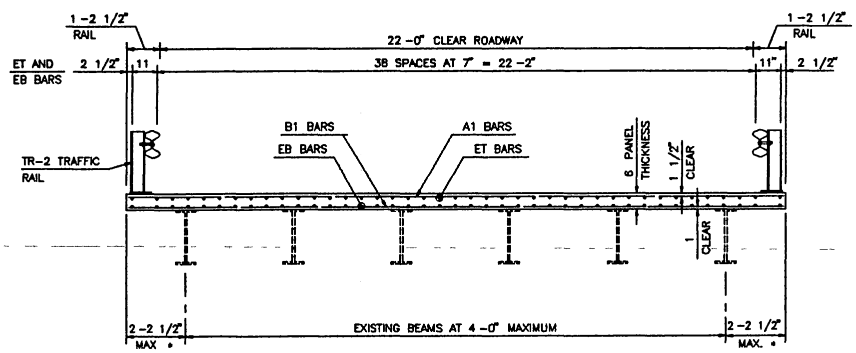
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 20 -0 CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

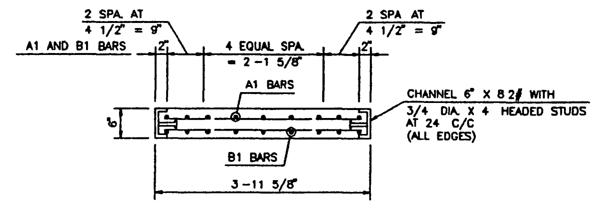


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	24 -1
B1	#4	9	STR	24 -1
ET	#5	41	STR	3 -8"
EB	#5	41	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1 79
REINFORCING STEEL (GRADE 60)	LB	603
STRUCTURAL STEEL A36	LB	523

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 22 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

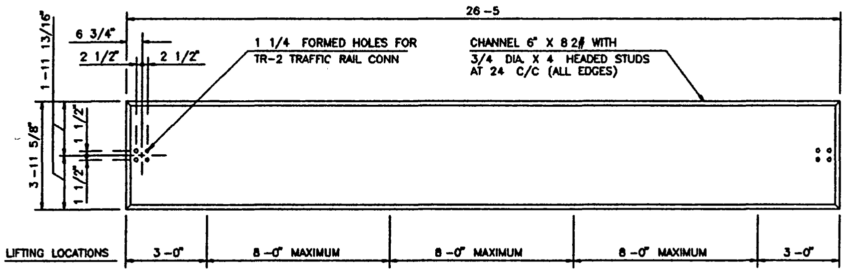
LIFTING NOTES

TOTAL PANEL WEIGHT = 8 380 LBS

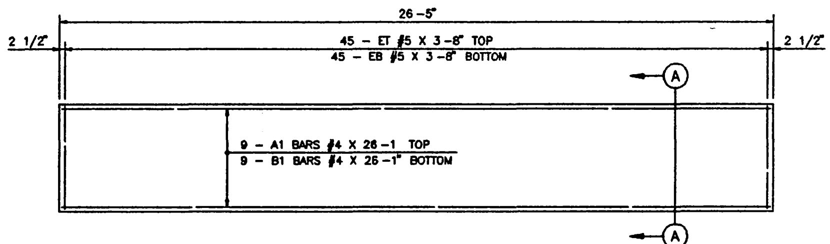
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

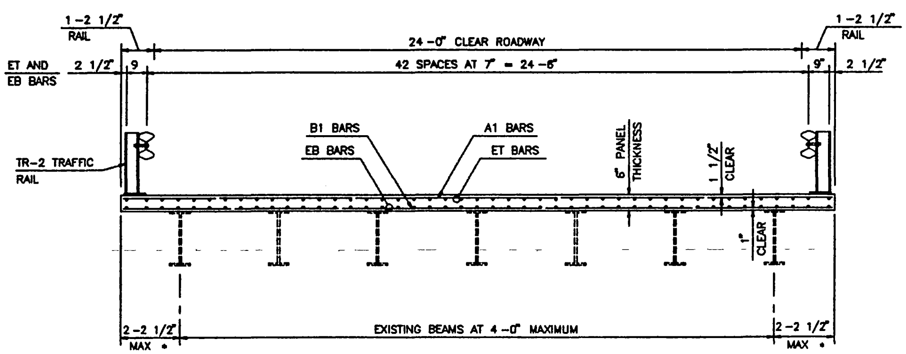
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 22 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

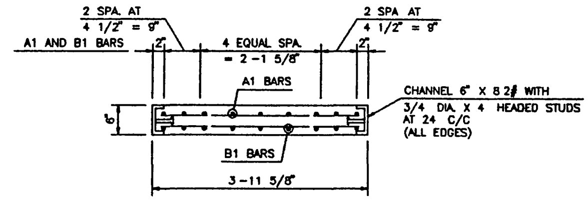


REINFORCING PLAN



• CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	26 -1
B1	#4	9	STR	26 -1
ET	#5	45	STR	3 -8"
EB	#5	45	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1 94
REINFORCING STEEL (GRADE 60)	LB	658
STRUCTURAL STEEL A36	LB	556

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 24 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA.
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S.I HAS BEEN REACHED

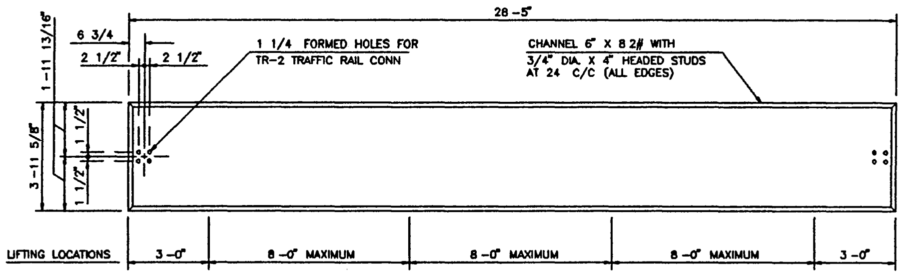
LIFTING NOTES

TOTAL PANEL WEIGHT = 9 070 LBS

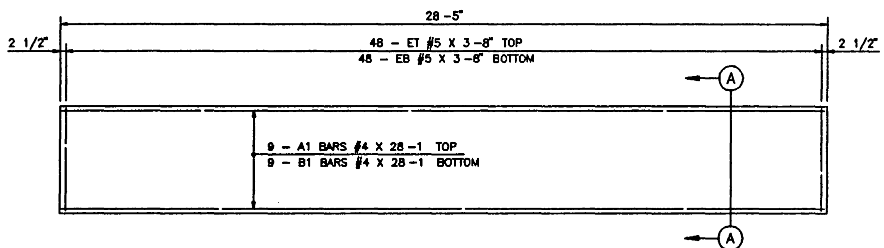
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

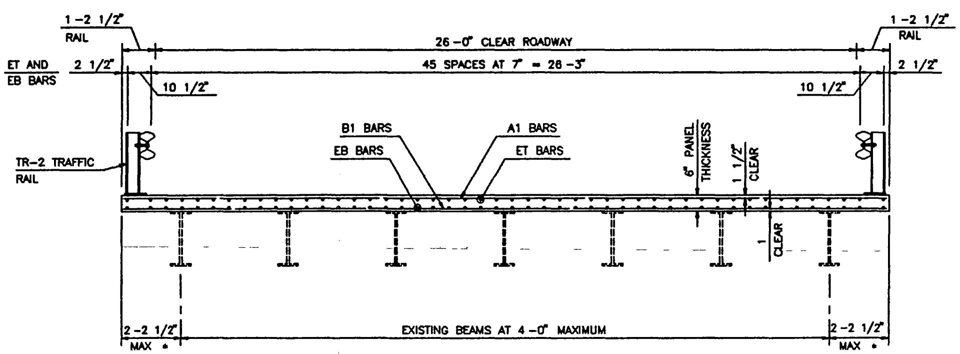
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 24 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

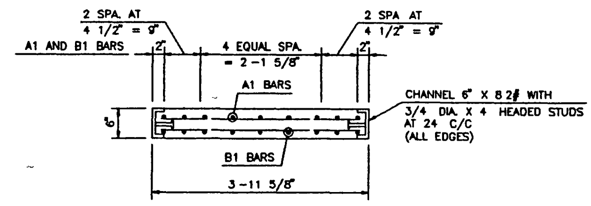


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	28 -1
B1	#4	9	STR	28 -1
ET	#5	48	STR	3 -8"
EB	#5	48	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 09
REINFORCING STEEL (GRADE 60)	LB	705
STRUCTURAL STEEL A36	LB	589

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS.

6" UNIFORM PANEL THICKNESS 26 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

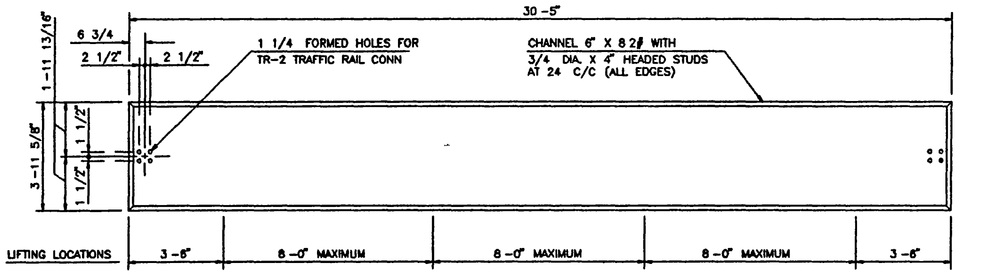
LIFTING NOTES

TOTAL PANEL WEIGHT = 9 760 LBS

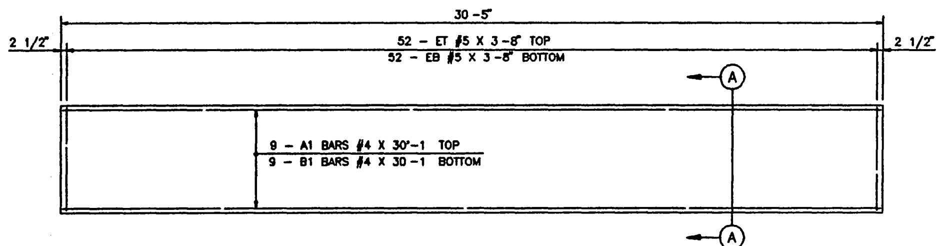
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

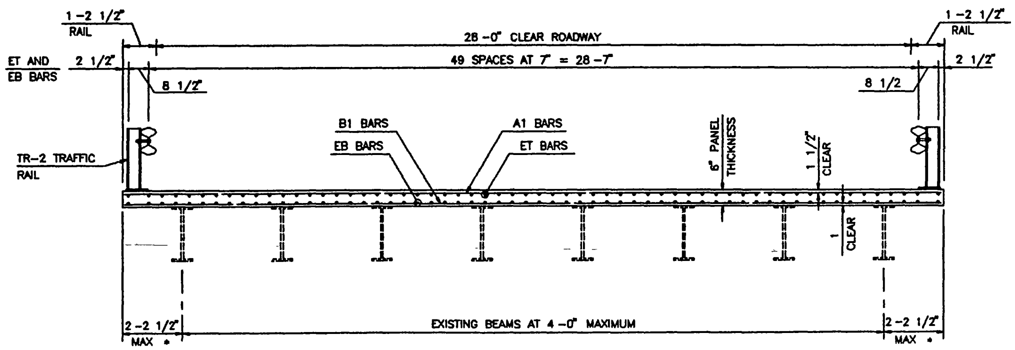
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 26 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

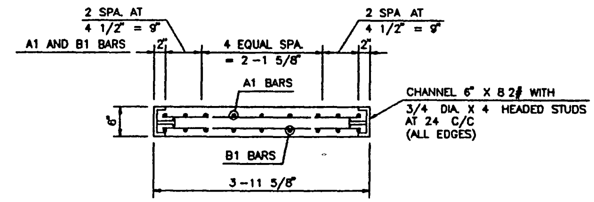


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#4	9	STR	30 -1
B1	#4	9	STR	30 -1
ET	#5	52	STR	3 -8"
EB	#5	52	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 24
REINFORCING STEEL (GRADE 60)	LB	760
STRUCTURAL STEEL A36	LB	637

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (4 -0" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

6" UNIFORM PANEL THICKNESS 28 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL.
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA.
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

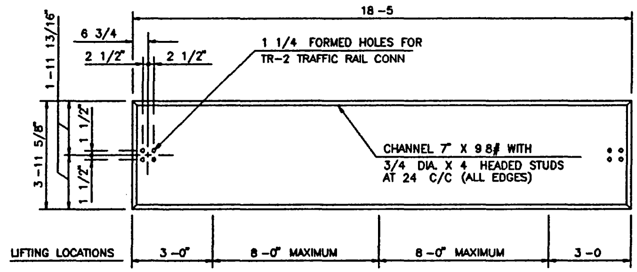
LIFTING NOTES

TOTAL PANEL WEIGHT = 10 470 LBS

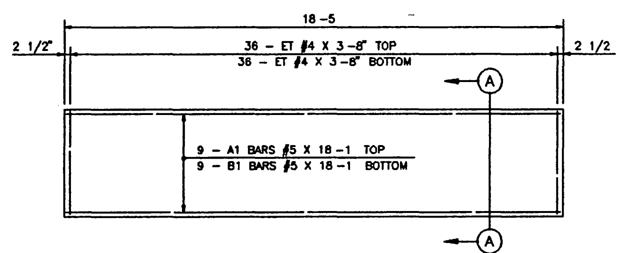
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

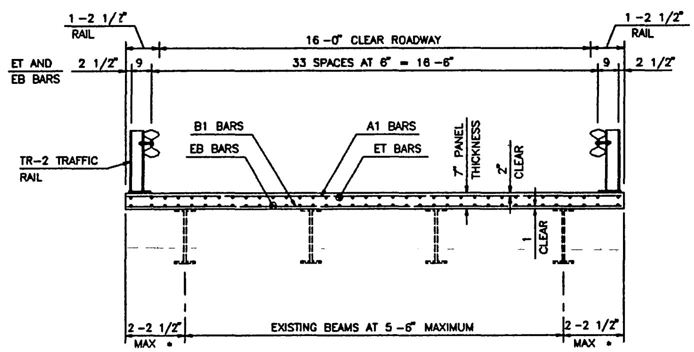
DESIGNED	GLF	6" CONCRETE DECK PANEL FOR 28 -0" CLEAR ROADWAY (4 -0" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

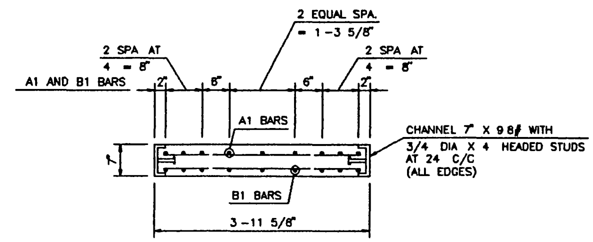


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	18-1
B1	#5	9	STR	18-1
ET	#4	36	STR	3-8
EB	#4	36	STR	3-8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 4
REINFORCING STEEL (GRADE 60)	LB	713
STRUCTURAL STEEL A36	LB	477

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5-6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 16-0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4-0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

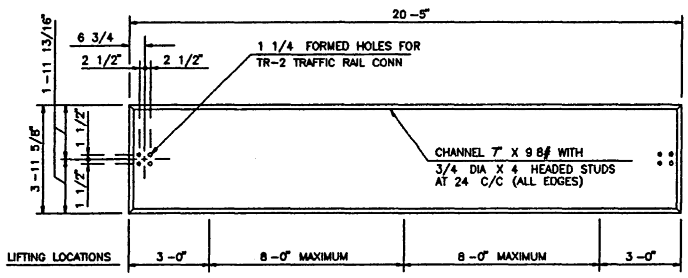
LIFTING NOTES

TOTAL PANEL WEIGHT = 7 400 LBS

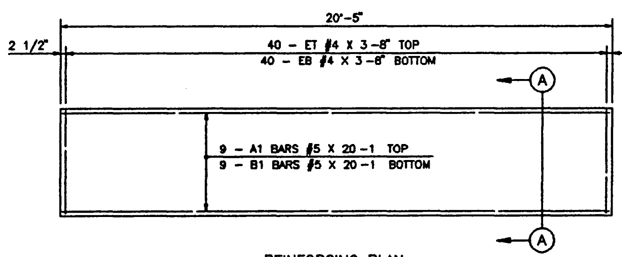
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

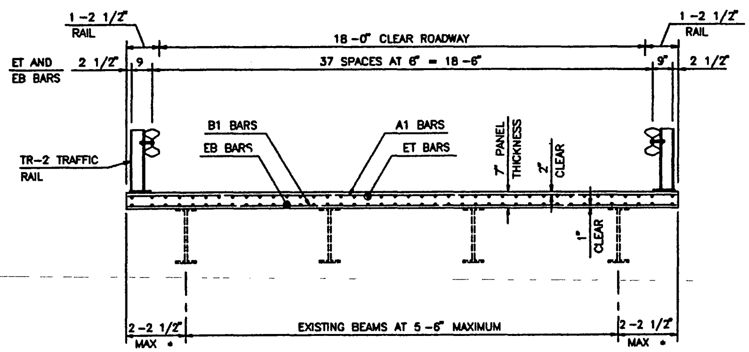
DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 16-0" CLEAR ROADWAY (5-6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

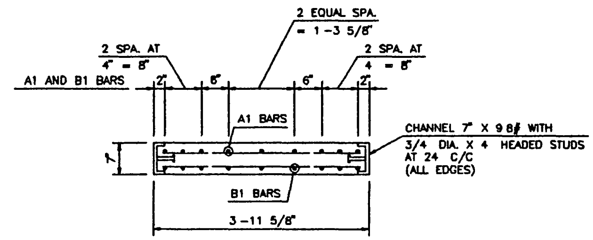


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	20 -1
B1	#5	9	STR	20 -1"
ET	#4	40	STR	3 -8
EB	#4	40	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1 75
REINFORCING STEEL (GRADE 60)	LB	573
STRUCTURAL STEEL A36	LB	515

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 18 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL.
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

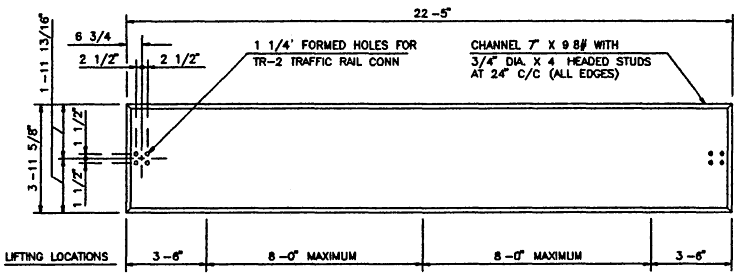
LIFTING NOTES

TOTAL PANEL WEIGHT = 8 180 LBS

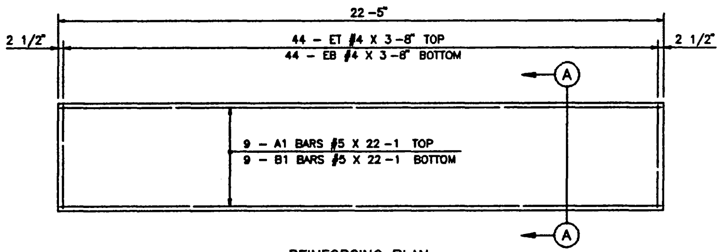
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

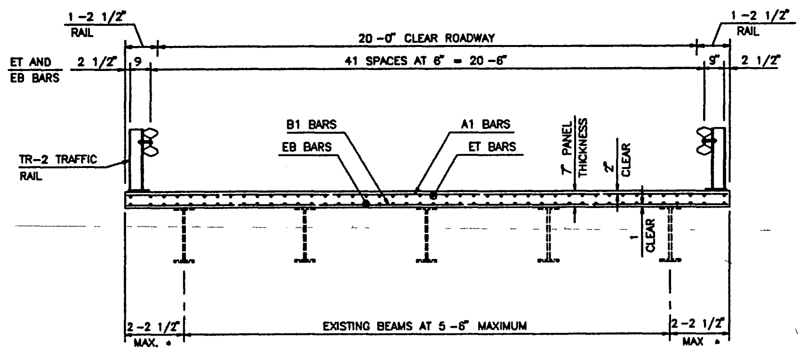
DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 18 -0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

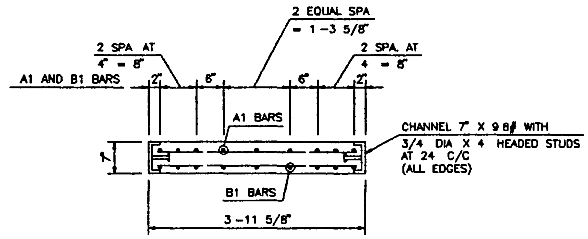


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	22 -1
B1	#5	9	STR	22 -1
ET	#4	44	STR	3 -8
EB	#4	44	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	1.92
REINFORCING STEEL (GRADE 60)	LB	830
STRUCTURAL STEEL A36	LB	573

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 20 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL.
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

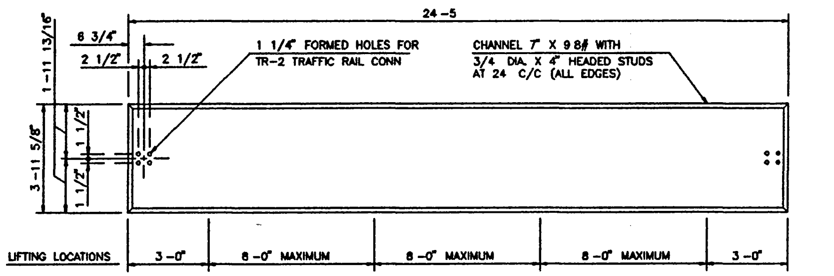
LIFTING NOTES

TOTAL PANEL WEIGHT = 8 980 LBS

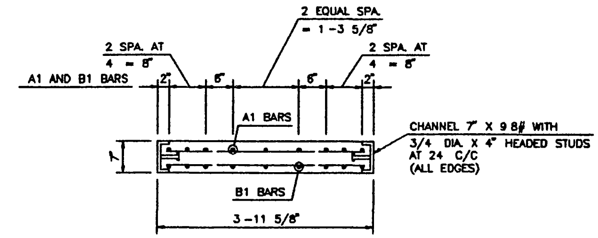
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 20 -0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



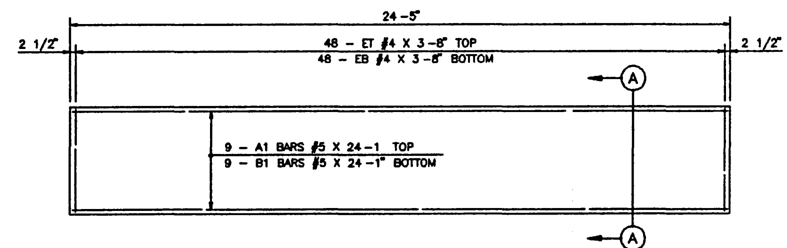
PANEL PLAN



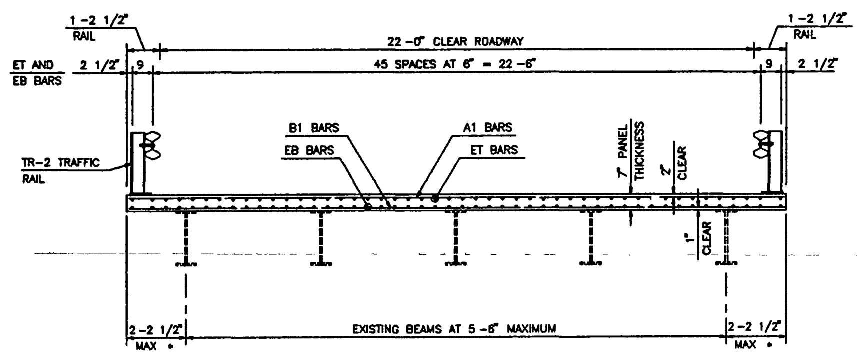
SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	24 -1
B1	#5	9	STR	24 -1"
ET	#4	48	STR	3 -8"
EB	#4	48	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 09
REINFORCING STEEL (GRADE 60)	LB	687
STRUCTURAL STEEL A36	LB	614



REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 22 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA.
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S.I HAS BEEN REACHED

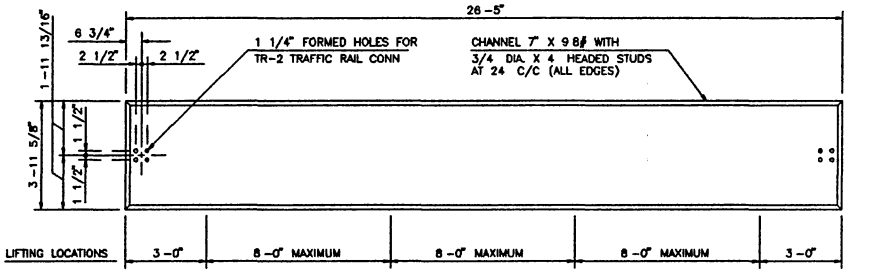
LIFTING NOTES

TOTAL PANEL WEIGHT = 9 770 LBS

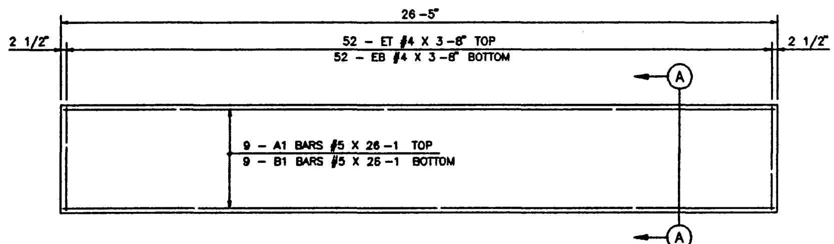
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

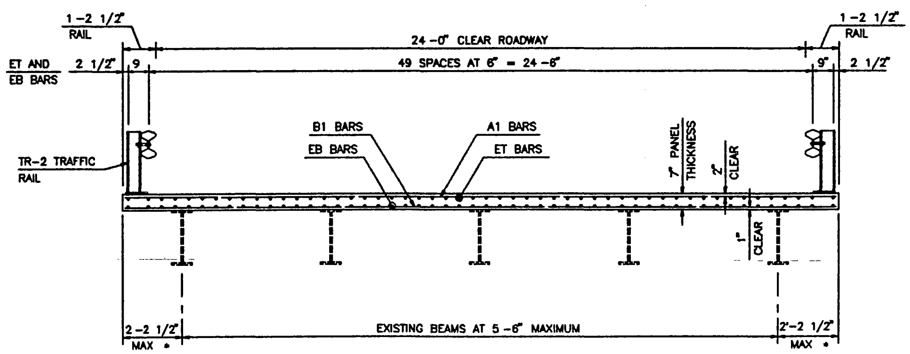
DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 22 -0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

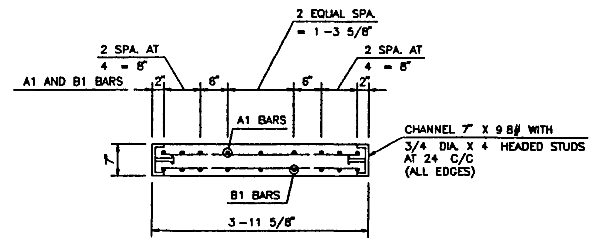


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	26 -1
B1	#5	9	STR	26 -1
ET	#4	52	STR	3 -8"
EB	#4	52	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	CY	2.27
REINFORCING STEEL (GRADE 60)	LB.	744
STRUCTURAL STEEL A36	LB	653

PANEL DESIGN DATA

LOADING HS20 WITH 20 PSF FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F'c = 3 500 PSI
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 PSI
STRUCTURAL STEEL A36	Fs = 20 000 PSI	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 24 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA.
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE.
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE.
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P.S.I HAS BEEN REACHED

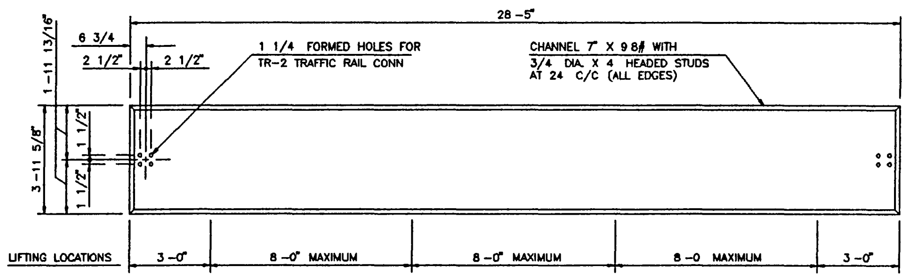
LIFTING NOTES

TOTAL PANEL WEIGHT = 10 590 LBS

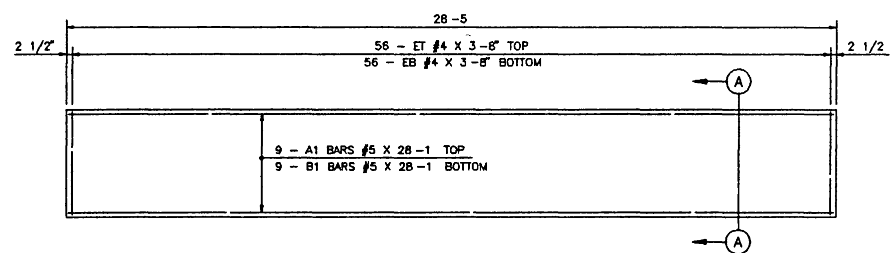
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 PSI HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS.

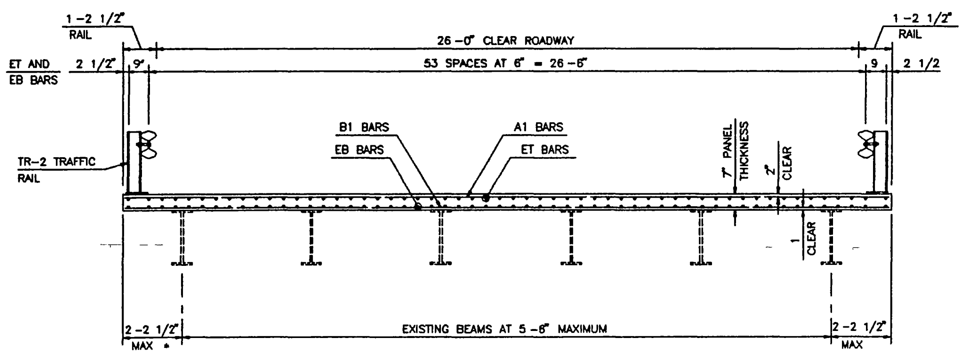
DESIGNED	GLF	7" CONCRETE DECK PANEL FOR 24 -0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		



PANEL PLAN

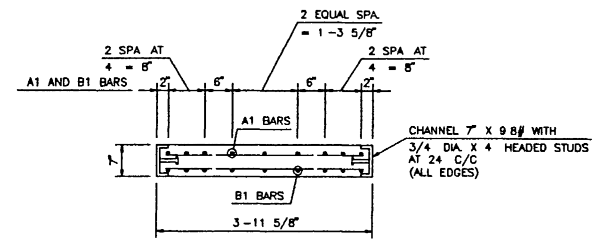


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	28 -1
B1	#5	9	STR	28 -1
ET	#4	56	STR	3 -8"
EB	#4	56	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 44
REINFORCING STEEL (GRADE 60)	LB	802
STRUCTURAL STEEL A36	LB	692

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO - 1989

	WSD	LFD
CONCRETE CLASS AA	———	F c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 26 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

LIFTING NOTES

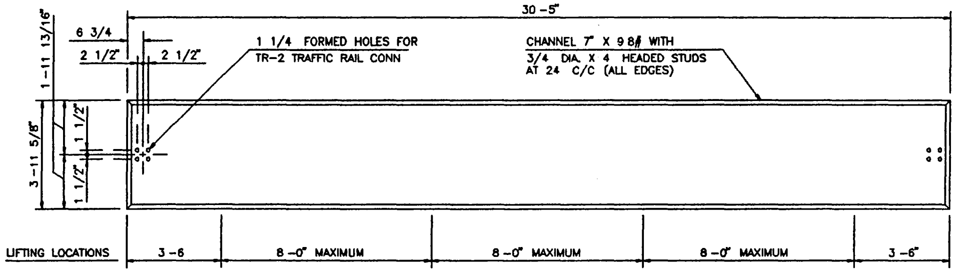
TOTAL PANEL WEIGHT = 11 380 LBS

PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

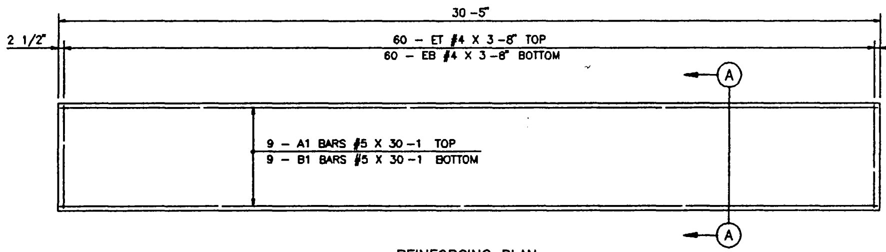
THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

DESIGNED	GLF
DRAWN	GLF
CHECKED	GLF
DATE MAY 1991	

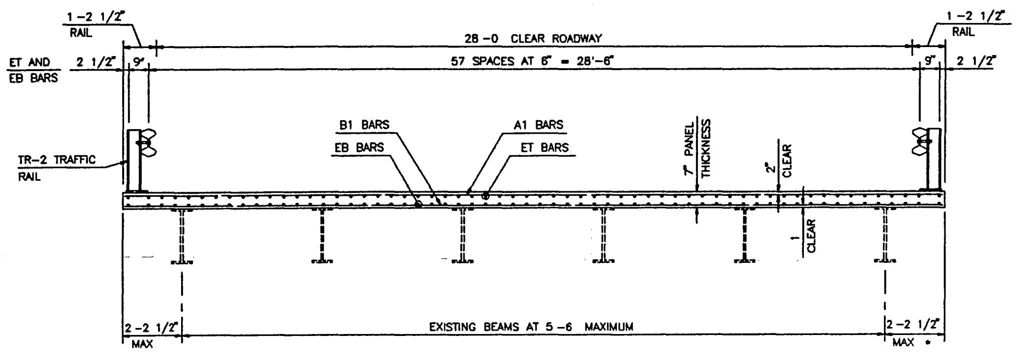
7" CONCRETE DECK PANEL FOR 26 -0 CLEAR ROADWAY
(5 -6" MAXIMUM BEAM SPACING)



PANEL PLAN

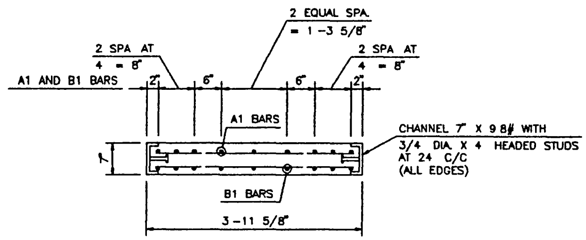


REINFORCING PLAN



* CLEARANCE SHALL BE MAINTAINED FOR CONNECTION OF TR-2 TRAFFIC RAIL

CROSS SECTION



SECTION A-A

PANEL BAR LIST				
BAR MARK	SIZE	NO	FORM	LENGTH
A1	#5	9	STR	30 -1
B1	#5	9	STR	30 -1
ET	#4	60	STR	3 -8"
EB	#4	60	STR	3 -8"

PANEL QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS AA CONCRETE	C Y	2 61
REINFORCING STEEL (GRADE 60)	LB	859
STRUCTURAL STEEL A36	LB	747

PANEL DESIGN DATA

LOADING HS20 WITH 20 P S F FUTURE WEARING SURFACE (5 -6" MAXIMUM BEAM SPACING)

DESIGN AASHTO – 1989

	WSD	LFD
CONCRETE CLASS AA	———	F c = 3 500 P S I
REINFORCING STEEL (GRADE 60)	———	Fs = 60 000 P S I
STRUCTURAL STEEL A36	Fs = 20 000 P S I	———

GENERAL NOTES

ALL CONSTRUCTION AND MATERIALS SHALL BE IN ACCORDANCE WITH THE 1988 OKLAHOMA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND SUPPLEMENTAL SPECIFICATIONS

7" UNIFORM PANEL THICKNESS 28 -0" CLEAR ROADWAY WITH TR-2 METAL TRAFFIC RAIL
CLASS AA CONCRETE WITH MAXIMUM AGGREGATE 1 DIA
REINFORCING SHALL BEGIN AND END A MAXIMUM OF 2" FROM FACE OF CONCRETE
NO WELDING OR TACK WELDING OF REINFORCING BARS SHALL BE PERMITTED
NO WELDING OR TACK WELDING TO THE FLANGES OF THE STEEL CHANNELS SHALL BE PERMITTED
FOR PANELS CAST IN A TRAFFIC SIDE DOWN POSITION THE REINFORCING BARS SHALL BE SUPPORTED FROM ABOVE
FOR PANELS CAST IN A TRAFFIC SIDE UP POSITION THE REINFORCING BARS SHALL BE SUPPORTED ON METAL CHAIRS SPACED AT 4 -0" MAXIMUM
CONCRETE IN EACH PANEL SHALL BE POURED IN ONE CONTINUOUS POUR

STEEL CHANNELS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

PANELS SHALL NOT BE PLACED ON BRIDGE UNTIL A MINIMUM CONCRETE STRENGTH OF 3 500 P S I HAS BEEN REACHED

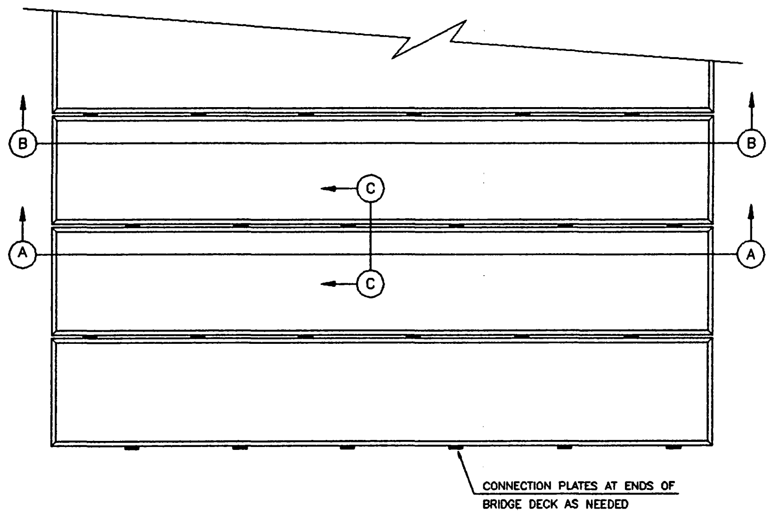
LIFTING NOTES

TOTAL PANEL WEIGHT = 12 180 LBS

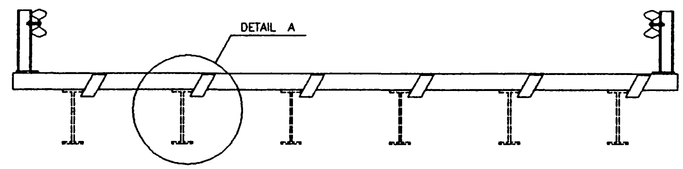
PANELS SHALL NOT BE LIFTED AND HANDLED UNTIL A MINIMUM CONCRETE STRENGTH OF 3 000 P S I HAS BEEN REACHED

THE PANEL SHALL BE SUPPORTED AS A MINIMUM AT THE LIFTING LOCATIONS SHOWN ON THE PANEL PLAN
THE LIFTING SYSTEM USED TO LIFT AND HANDLE THE PANEL SHALL PRODUCE EQUAL LOAD AT ALL LIFTING LOCATIONS

DESIGNED	GLF	7 CONCRETE DECK PANEL FOR 28 -0" CLEAR ROADWAY (5 -6" MAXIMUM BEAM SPACING)
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		

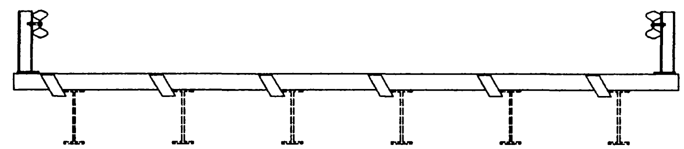


BRIDGE DECK PLAN



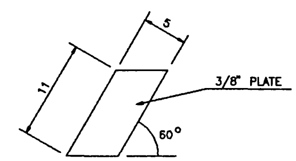
SECTION A–A

ALTERNATE WITH SECTION B–B

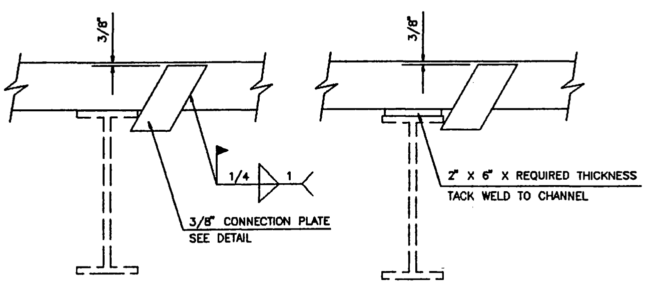


SECTION B–B

ALTERNATE WITH SECTION A–A



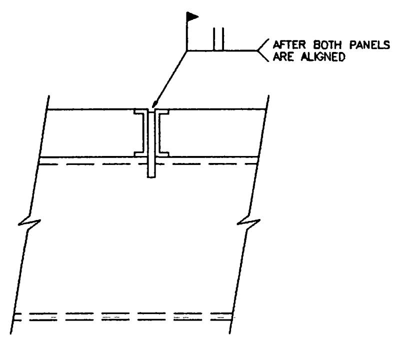
CONNECTION PLATE DETAIL



SHIM NOT REQUIRED

SHIM REQUIRED

DETAIL "A



SECTION C–C

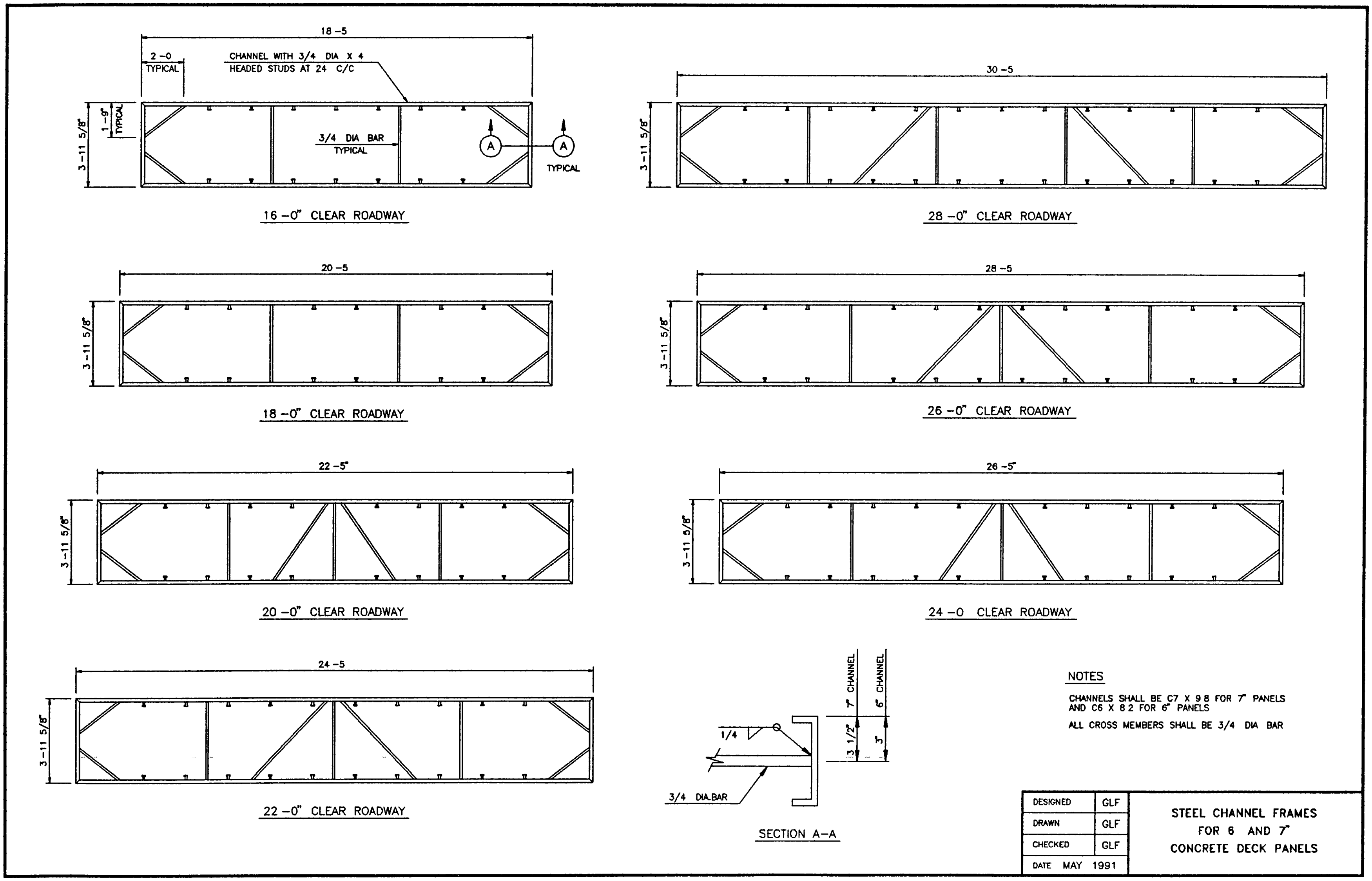
NOTES

THE CONNECTION PLATES AND SHIMS SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

THE SEQUENCE OF ALIGNMENT AND CONNECTION OF THE PANELS TO THE BRIDGE BEAMS IS AS FOLLOWS

ALIGN PANEL TIGHT FIT THE CONNECTION PLATE AGAINST BEAM AND WELD AS SHOWN IN DETAIL A ALIGN NEXT PANEL AND WELD AS SHOWN IN SECTION C–C THE GROOVE WELD SHALL BE PAINTED WITH TWO COATS OF ZINC RICH PAINT

DESIGNED	GLF	CONNECTION DETAILS FOR 6 AND 7 CONCRETE DECK PANELS
DRAWN	GLF	
CHECKED	GLF	
DATE MAY 1991		


18 -5
2 -0
TYPICAL
CHANNEL WITH 3/4 DIA X 4
HEADED STUDS AT 24 C/C
3-11 5/8"
1-9"
TYPICAL
3/4 DIA BAR
TYPICAL
A
A
TYPICAL
16 -0" CLEAR ROADWAY
30 -5
3-11 5/8"
28 -0" CLEAR ROADWAY
20 -5
3-11 5/8"
18 -0" CLEAR ROADWAY
28 -5
3-11 5/8"
26 -0" CLEAR ROADWAY
22 -5"
3-11 5/8"
20 -0" CLEAR ROADWAY
26 -5"
3-11 5/8"
24 -0 CLEAR ROADWAY
24 -5
3-11 5/8"
22 -0" CLEAR ROADWAY
7" CHANNEL
6" CHANNEL
1/4
3 1/2"
3"
3/4 DIA.BAR
SECTION A-A
NOTES
CHANNELS SHALL BE C7 X 9.8 FOR 7" PANELS
AND C6 X 8.2 FOR 6" PANELS
ALL CROSS MEMBERS SHALL BE 3/4 DIA BAR
DESIGNED GLF
DRAWN GLF
CHECKED GLF
DATE MAY 1991
STEEL CHANNEL FRAMES
FOR 6 AND 7"
CONCRETE DECK PANELS

APPENDIX C

COUNTY SURVEY PACKET

LEE CHEW President
Washington County Courthouse
420 S Johnstone Room 108
Bartlesville OK 74003
Office (918) 336-0330
Barn. (918) 536-3315

HANK YORK, Vice President
McClain County Courthouse
P O Box 629
Purcell OK 73080
Office (405) 527 3117
Barn. (405) 485-3388

WENDELL VENCL, Secretary/Treasurer
Garfield County Courthouse
Room 101
Enid, OK 73701
Office (405) 237-0227
Barn. (405) 863-2275

Association of County Commissioners of Oklahoma

1140 N W 63rd Suite 103
Oklahoma City Oklahoma 73116
(405) 840-9582

June 22, 1990

Board of County Commissioners
Noble County
Box 409
Perry OK 73077

Dear Commissioners:

The Department of Civil Engineering at Oklahoma State University in association with the Oklahoma Department of Transportation and Center for Local Government Technology is conducting a research project aimed at the replacement of decks on county bridges. The project involves the development of a precast concrete panel system for bridge deck replacement and a comparison of this system to methods presently being used for deck replacement. The system is primarily aimed toward utilizing county forces in the construction and placement of the precast panels. The bridges being targeted for study have steel beams, a substructure in reasonably good condition, and an inventory load rating of 10 tons or greater.

A series of precast concrete panels is being developed to encompass the variety of existing bridge widths and beam spacings presently found on the county road system. These concrete panels provide clear roadway widths of 16'-0" to 28-0" on 2'-0" increments. See the attached sketches for information concerning the panels and placement of the panels on the bridge.

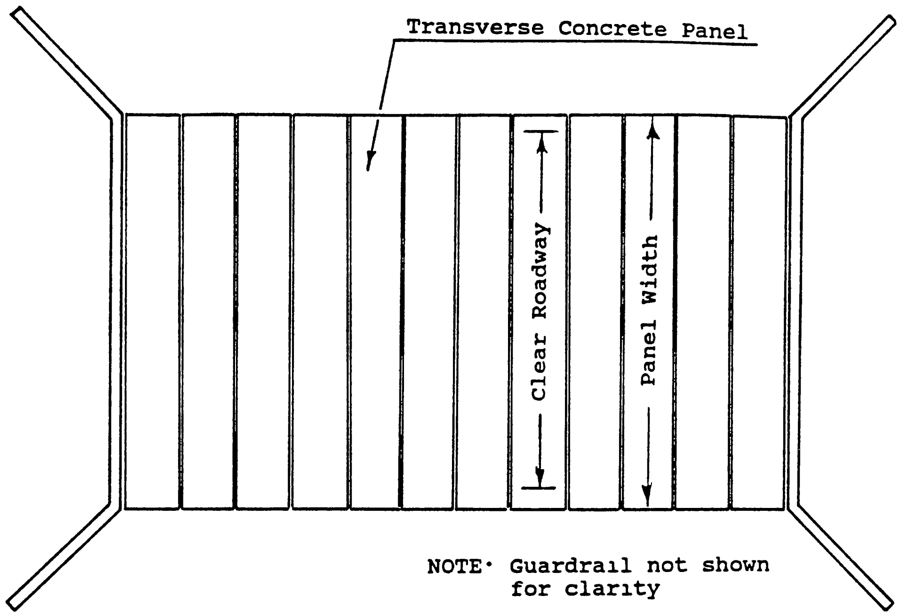
Collection of data pertaining to the ability of county forces to construct the precast concrete panels at county facilities, transport the panels to the bridge site, and place the panels on the bridge is vital to the research project. Please complete the enclosed questionnaire for your entire county (not districts) and return using the enclosed envelope. The accuracy of the information received will greatly effect the results of the study. Your time and cooperation are greatly appreciated on this project.

Sincerely,

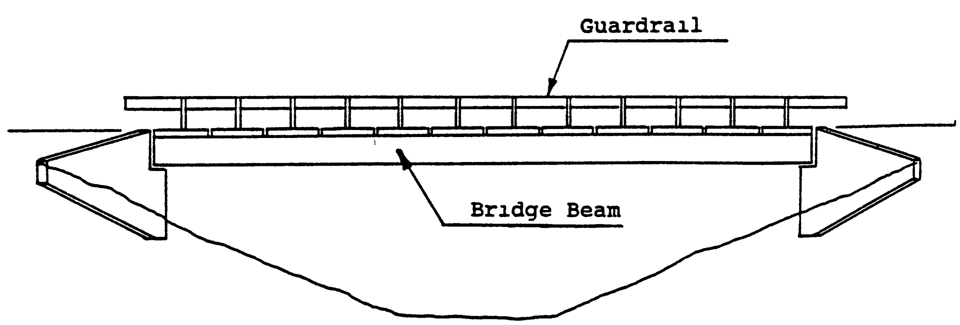
Lee Chew

Lee Chew

Attachments

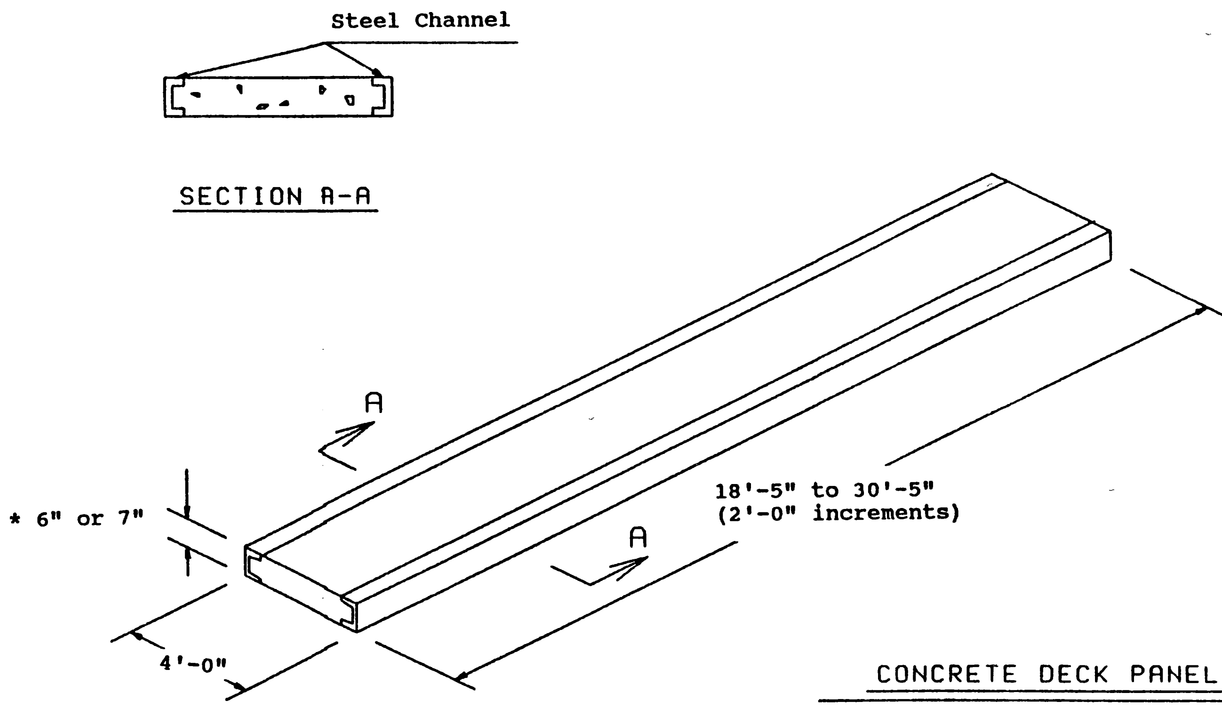


PLAN



ELEVATION

CONCRETE PANEL BRIDGE DECK REPLACEMENT SYSTEM



CONCRETE DECK PANEL

DESIGN DATA:

HS20-44 Loading
Grade 60 Reinforcing
F'c = 3500 psi

* 6" panel for beam spacings up to 4'-0"
7" panel for beam spacings up to 5'-6"

COUNTY. ________________ Person Completing Form. ____________________

Phone Number: ______________ Date: __________

A. CASTING FACILITIES:

1. Do you have an outside flat concrete slab area that could be used as a casting bed? yes____ no____

   If yes, what size? _______ ft. long x _______ ft. wide

2 Do you have a flat slab area in a building that could be used as a casting bed? yes____ no____

   If yes, what size? _______ ft. long x _______ ft. wide.

   If yes, is the building heated? yes____ no____

B. EQUIPMENT:

1. Check the equipment your county presently owns and give the safe lifting capacity for a bridge deck replacement project using these panels. Check the "borrow" and/or "lease" column if you have access to equipment. (See attached equipment pictorial sheet.)

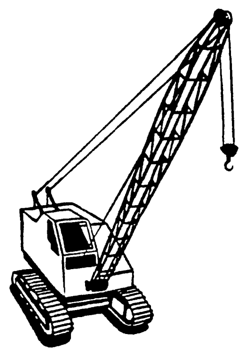
	Lifting Cap (lbs)	Own (Quant)	Borrow (X)	Lease (X)
Crawler mounted crane (fixed boom)	________	_____	_____	_____
Truck mounted crane (fixed boom)	________	_____	_____	_____
Self propelled crane with telescopic boom	________	_____	_____	_____
Wheel tractor with front loader	________	_____	_____	_____
Winch trucks -				
- less than 5,000 lbs. cap.	________	_____	_____	_____
- 5,000 to 10,000 lbs. cap.	________	_____	_____	_____
- 10,000 to 15,000 lbs. cap.	________	_____	_____	_____
- greater than 15,000 lbs. cap.	________	_____	_____	_____
Other _________________________	________	_____	_____	_____

2. Do you have a truck which could safely transport one or more 4' x 7" x 30'-5" concrete panels (10,700 lbs. each) from the casting area to the bridge site? yes____ no____

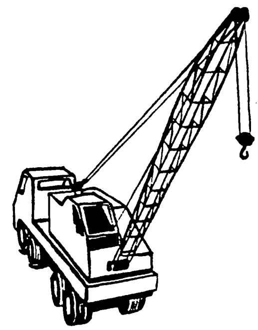
C. PERSONNEL:

1. Number of county personnel with concrete and bridge maintenance experience.

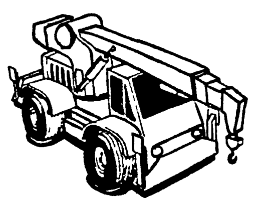
Experience	Number of Personnel	Range of hourly wages Min. $/hr.	Max. $/hr.
less than 1 year	__________	_________	_________
1 to 5 years	__________	_________	_________
more than 5 years	__________	_________	_________



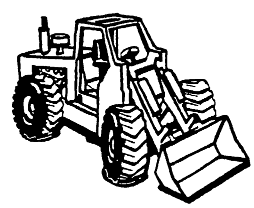
Crawler Mounted Crane
(fixed boom)



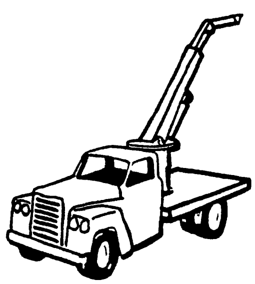
Truck Mounted Crane
(fixed boom)



Self Propelled Crane
(telescoping boom)



Wheel Tractor with
Front End Loader



Winch Truck

VITA

Gregory Lynn Fitter

Candidate for the Degree of

Doctor of Philosophy

Thesis: COUNTY BRIDGE DECK REPLACEMENT WITH TRANSVERSE PRECAST CONCRETE PANELS

Major Field: Civil Engineering

Biographical:

Education: Received Bachelor of Science in Civil Engineering from the University of Oklahoma in May, 1981; received Master of Science from Oklahoma State University in August, 1987; completed requirements for the Doctor of Philosophy at Oklahoma State University in May, 1991.

Registrations and Organizations: Registered Professional Engineer, Oklahoma; American Concrete Institute; Chi Epsilon, Tau Beta Pi.

Professional Experience: Research Associate and Teaching Assistant, Department of Civil Engineering, Oklahoma State University. August 1988 to January 1990.

Bridge Engineer, White Engineering Company, Oklahoma City. May 1981 to June 1984, May 1986 to January 1988. Responsibilities included the structural design of bridges, calculation of bridge geometrics and quantities, and the supervision of required drafting. Scope of bridge projects ranged from single-span county bridge to multi-span highway bridges. Designs utilized prestressed concrete beams, steel plate girders, and reinforced concrete box beams.