STAY-IN-PLACE BRIDGE DECK FORMS
(A State of the Art Review)

PART I: PRESTRESSED PANEL SUBDECKS

by

M. H. Hilton
Highway Research Engineer

Virginia Highway Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia
Department of Highways and the University of Virginia)

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INTRODUCTION

A number of factors, both economic and practical, have combined in recent years to cause the highway construction industry to search for more expeditious and versatile methods of approach to the construction of bridges. One particular area of concern has been related to the construction of bridge decks--and most notably to the forming of bridge decks. Except in a few areas of the country, wood and plywood have been the mainstay materials used for forming decks. Steel stay-in-place forms have been used in some instances in many states and widely used in several northeastern states. Another relatively recent approach has been the use of precast prestressed panel sub-decks that serve as the forming for the finished deck concrete while also becoming an integral part of the completed deck thickness. This technique has been used successfully in several states and several others have structures in the planning and design stage which will utilize the system.

Highway bridge contractors have become increasingly interested in stay-in-place forms for several reasons. First, the uncertain economic conditions during the last few years coupled with record construction activity and strong foreign demand for wood and plywood have forced lumber prices up, thus making other materials more competitive. Occasionally, lumber has been in short supply or unavailable when needed. Secondly, the stripping of lumber forms from bridge decks is a hazardous operation which has become more undesirable with the advent of more stringent federal safety requirements for the protection of workmen. Thirdly, the elimination of the form stripping operation can lead to considerable savings of both construction time and labor costs. Lastly, with proper planning and development the use of stay-in-place forms could reduce the amount of time and labor required to form a bridge deck.

One of the primary ways of restraining production costs in the manufacturing industry has been to reduce the time and labor required to produce the end product. This has normally been accomplished by reducing the number of production operations required and/or by increasing automation. In bridge construction the use of stay-in-place forms is a basic step in the direction of industrialized construction which can reduce time and labor costs and totally eliminate one operation— that of form stripping—from the construction process. Both highway engineers and contractors are attracted by these obvious advantages. As with most innovations, however, highway engineers must concern themselves with possible disadvantages associated with the use of stay-in-place forms and weigh those against the advantages before embarking upon wide-spread use of the technique.

PURPOSE AND SCOPE

During a meeting of the Virginia Department of Highways — Virginia Road Builders Association (VDH-VRBA) Joint Cooperative Committee, it was recommended that the Highway Research Council review the prestressed concrete panel type forming to determine its feasibility for use on bridge construction in Virginia. Later discussions with the Department's bridge and construction engineers revealed that they were also quite concerned about the long-term effects relating to the use of steel stay-in-place forms. As a result, it was decided to include both the steel and the prestressed concrete type stay-in-place forms in the overall review. This report, however, is limited only to a review of the prestressed concrete panel type forming.

Considerable laboratory and field work to investigate the use of prestressed panel forms for bridge decks has been conducted in Texas. Further laboratory and full-scale tests are under way in Pennsylvania. In view of the amount of research that has been completed or is now being conducted by others, this study was concerned only with a state of the art review of the material now available on the use of precast prestressed panels for constructing bridge decks. The pertinent results of all investigations that have been conducted and reported to date along with a review of the general designs that have been used on some actual structures are included. In addition, any problems that have been identified to be peculiar to the use of prestressed panel type forms are discussed.

TEXAS STUDIES

The Texas Highway Department has used the precast prestressed concrete panel technique for the construction of bridge decks for a number of years. Three overpass structures constructed by this technique, for example, were opened to traffic in August 1963 and have subsequently been the subject of a study conducted by Jones and Furr.[1] More recent studies concerning the development length of the panel prestressing strands plus additional full-scale laboratory studies have been conducted.[2,3,4]

Due to the favorable experience with the precast subdeck approach, Texas is now making considerable use of the technique. On the recently completed bridge between Corpus Christi and Padre Island, for example, the panels were used on all 36 spans approaching the main cantilevered box girder spans.[5]

Studies of Three In-Service Bridges

The first of the three in-service bridges investigated consisted of two end spans of 45 ft. length and two interior spans of 60 ft. length. Each span consisted of four simply supported prestressed girders with a lateral spacing of 6 ft.-8 in. on center. The 3 in. thick prestressed panel subdecks were 6 ft. -2 in. long and 4 ft. wide. Details of the most recent Texas standard prestressed panel design are given in the Appendix.) The remaining two bridges were twin structures — each consisting of two 40 ft. and three 50 ft. simply supported prestressed girder spans. The six girders in each span were spaced at 7 ft.-3 in. on center and the 3 in. thick subdeck panels were 6 ft. -9in. wide and varied from 1 ft. -5 in. to 5 ft. -2 in. in length. A typical panel layout on a 50 ft. span is shown in Figure 1. All of the panels on the three bridges utilized 3/8 in. diameter 7 -wire strands prestressed at 14 kips per strand. The strands were spaced at $4\frac{1}{2}$ in. on center at mid-depth of the panels. Running transverse to the prestressed strands, number 2 plain reinforcing steel was spaced at 6 in. on center.

The cast-in-place concrete portion of the decks was 3" thick and reinforced with number 5 bars spaced at 15 in. on center in both the longitudinal and lateral directions. Additional short lengths of steel were used in the lateral direction and placed over each girder at 15 in. on center.

Test Results

A field survey of the test bridges revealed hairline transverse cracking on the surface of the deck of the most heavily traveled twin bridges. The vast majority of the cracking occurred directly above the butt joints between the prestressed panels as shown in Figure 1. Core samples drilled from the deck through two of the cracked locations indicated that the cracks extended approximately half way through the upper cast-in-place slab. Interestingly, a survey of monolithically cast bridge decks in Texas revealed an average transverse crack frequency similar to that found on the study structures.[6] In the latter structures, however, the subdeck panels appear to control the crack location. The cores gave no indication of a bond failure or delamination between the subdeck and the upper deck. A direct shear test on one core yielded a substantial bond stress of 285 psi.

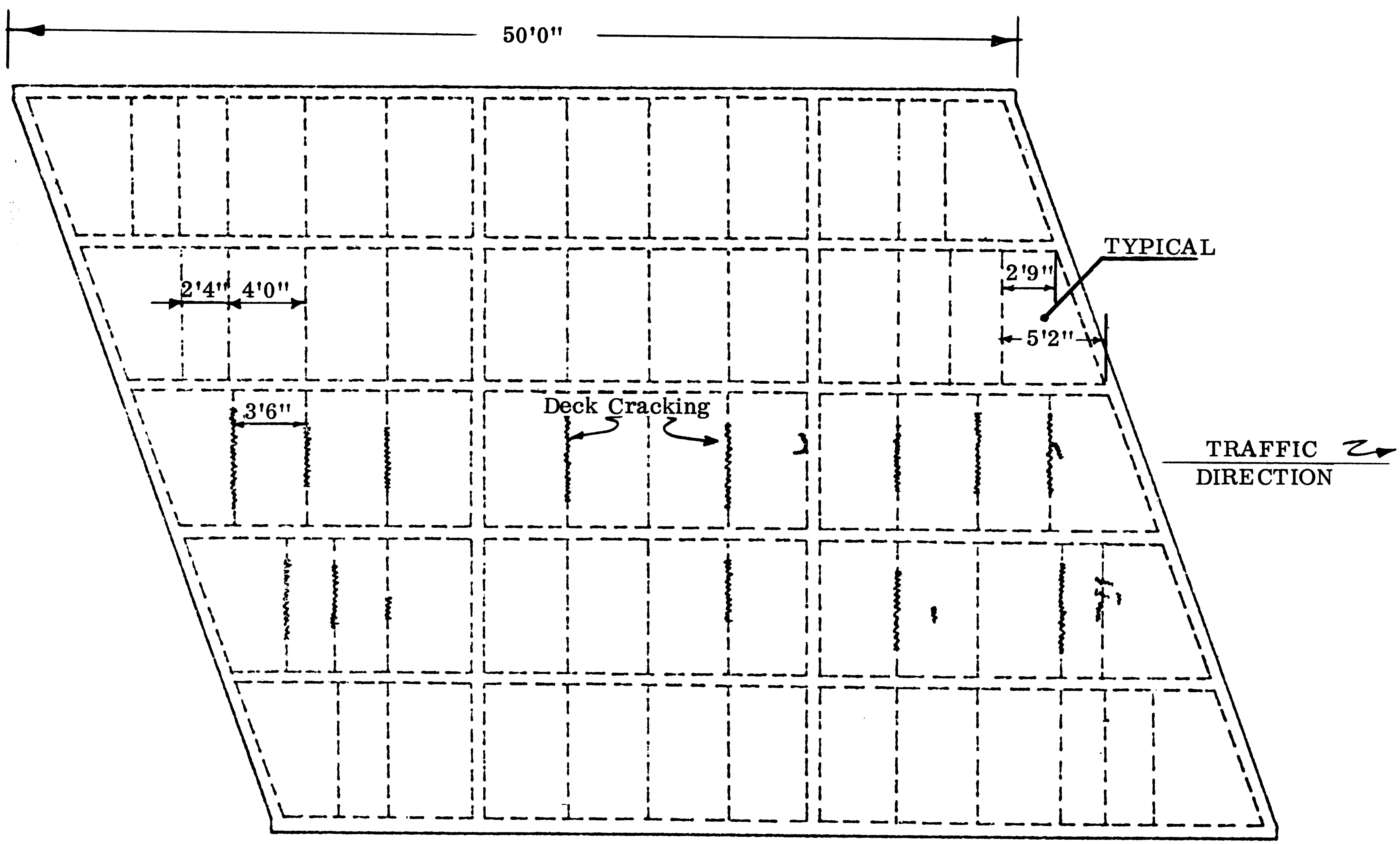

Figure 1. Typical panel layout and cracking on deck surface above subdeck joints. (From Reference 1.)

Soundings were also taken down the length of the right lane of one bridge with only two small areas of delamination believed to be found. These areas were not considered significant. No delamination was found in the vicinity of any of the transverse deck cracking.

Strain gages were employed on the top and bottom of the deck and within a core hole. Dial gages were also employed to measure possible slip between the panels and the girders and any relative movement between adjacent panels. No slippage was found in either case when the span was loaded at various positions with a 71.8 kip truck. Other strain readings indicated that there was a smooth transfer of load from panel to panel and no discontinuities were found between the panels and the slab.

No cracking or distress was found in the prestressed panels and the bridges were concluded to be in sound condition after approximately seven years under heavy traffic.

Strand Development Length

Additional studies by Jones and Furr [2] were concerned with the structural properties of the prestressed panels themselves. Since the prestressing strands must transfer stresses to the concrete over some finite distance from each end of a member, the relatively short width of the panel subdecks is a factor to consider. Tests were conducted on $3\frac{1}{4}$ in. thick panels to determine the development length of the strands both initially and after repeated loading. For 3/8 in. diameter 7-wire strands tensioned with a force of 13.75 kips an average development length of 22 in. was required. For $\frac{1}{2}$ in. diameter strands tensioned with a force of 27.50 kips each, an average 34 in. of development length was required. These development lengths were based on a gradual release of the jacking force used to stress the prestressing strands. (Flame cutting of the strands usually results in longer development lengths due to a sudden release of the prestressing force.) Since the length of the shortest test panels was 68 in., the full prestress force could be developed in each case. When the $\frac{1}{2}$ in. diameter strands were used in the 68 in. panels, however, it was concluded that only a few inches near midspan received the full prestress force.

Cyclic Loading

Fifteen of the twenty test panels studied were subjected to two million cycles of load. The load was selected to give bending stresses of 1,400 psi compression and zero tension in the prestressed panels since 700 psi of compression was induced by the prestressing. The results of these tests showed that the cyclic loading had a negligible effect on panel stiffness and on the development length of the prestressing strands.

Laboratory Tests of a Full-scale Bridge

Further tests were conducted by Buth, Furr, and Jones [3] on a full-scale 23 ft. wide, 50 ft. simply supported span composed of prestressed concrete girders, $3\frac{1}{4}$ in. thick prestressed subdeck, and a $3\frac{1}{2}$ in. thick cast-in-place deck. A cutaway view of the structure, which was designed for an HS20-44 loading, is shown in Figure 2. The basic purpose of this work was to investigate the ability of the structure to distribute wheel loads and to behave as a composite unit under loads.

The bridge was subjected to cyclic loading on either side of a panel butt joint to simulate a truck wheel load crossing the joint. In addition, other loading tests were conducted. The bridge was finally subjected to static failure loads applied adjacent to some of the panel butt joints.

In the design of this type bridge it is assumed that all elements bond together and act as a composite section in the transfer of stresses across the cast-in-place deck, prestressed panel, and at the slab to beam interfaces. Thus, as test variables, the researchers investigated three methods of bonding the cast-in-place concrete to the prestressed panels. One method employed Z-bars (see Figure 3) in selected portions of the bridge to provide both shear and tensile bond between the upper deck and lower panels. In another area of the bridge portland cement grout was brushed onto the prestressed panels just prior to placement of the upper deck. On the remainder of the bridge no bonding treatment at all was used.

An additional variable involved the use of dowel bars at some selected transverse butt joints. The dowels were placed on the surface of the panels, as shown in Figure 4, to determine what effect they would have on the transfer of wheel loads across the joints between panels. Additional tests were run on two panels constructed separate from the full-scale bridge. The locations of the dowel bars and of the variable bond treatment areas are shown in Figure 5.

Results

Two million applications of simulated design axle loads including impact were applied to the test structure at several locations and on opposite sides of a panel butt joint. No distress was caused by this cyclic loading with regard to either the bond at the interface (between the prestressed panels and the cast-in-place concrete) or the butt joints between panels. No indication of bond distress was noted after the load to failure tests.

The results of the tests further indicated that the Z-bars used for mechanical shear connectors, the surface grouting treatment, and the dowel bars used at some joints provided no measurable improvement in the performance of the bridge.

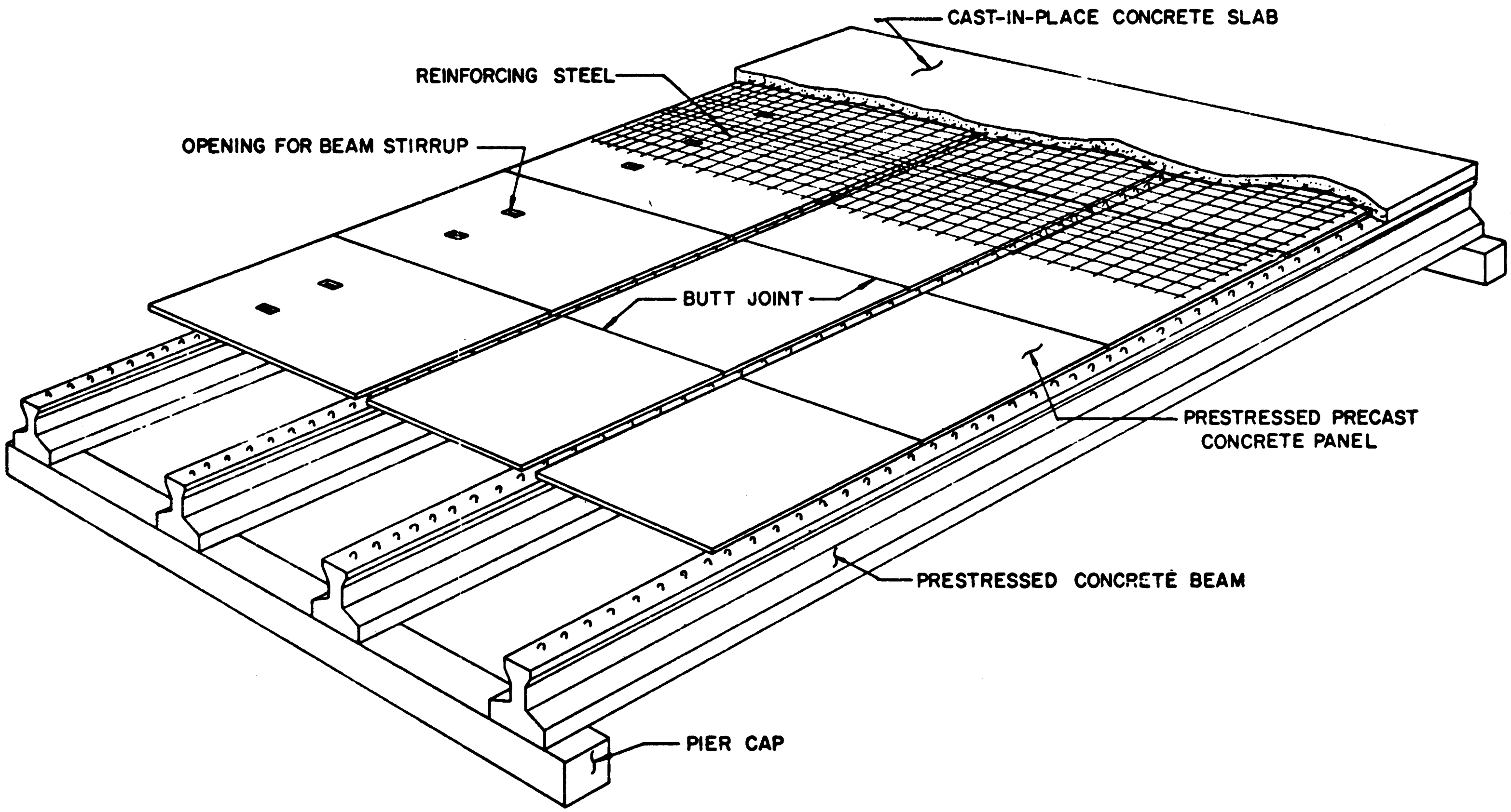

Figure 2. Cutaway view of prestressed panel type bridge investigated by Buth et al. (3)

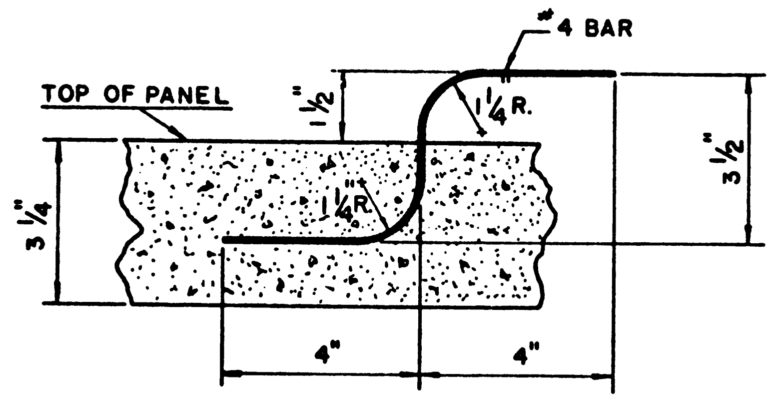

Figure 3. Z-bars used in selected panels to aid in providing structural connection between panel and cast-in-place deck. (From Reference 3.)

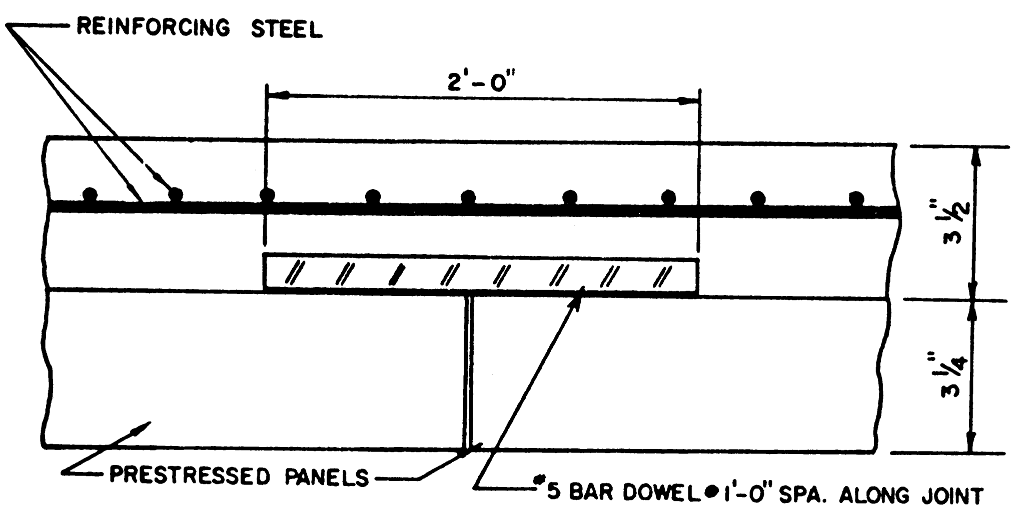

Figure 4. Dowel bars used at selected panel butt joints. (From Reference 3.)

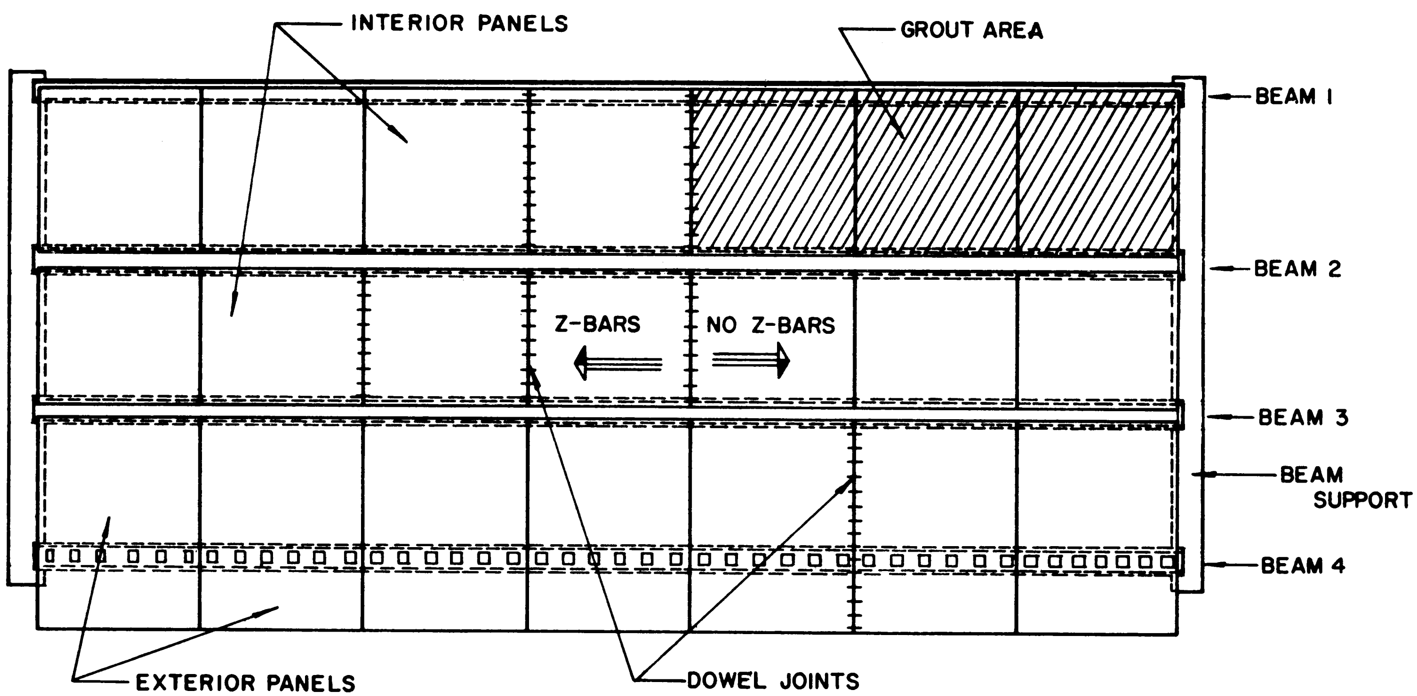

Figure 5. Location and identification of various structural details in full-scale bridge. (From Reference 3)

Actually, some of the highest failure loads were recorded in areas where no Z-bars, grout, or dowels were employed. The loads at which cracking first occurred and the modes of failure were similar in each case and it was concluded that the special treatments for bond and load transfer did not result in improved performance of the structure.

The lowest load at which cracking in the deck occurred was 3.8 times the design wheel load, including impact, and the lowest ultimate load was 12 times the design wheel load plus impact.

Similar to the in-service bridges investigated earlier, the full-scale test bridge cracked in some locations directly above the panel butt joints. The cracks, which were approximately 0.002 in. in width, occurred in both loaded and unloaded areas of the deck. Cores taken after the ultimate load tests indicated that the cracks stop at approximately half the depth of the cast-in-place slab. It was noted, however, that the diaphragms on bridges should be positioned so that they do not provede transverse support for the prestressed panels. For short spans and near the ends of longer spans the compressive stresses under heavy loads are sometimes not as large as the tensile stresses which may cause transverse cracks to form. It was noted that the cracking on the in-service bridges investigated was more frequent on the shorter spans and toward the ends of the longer spans.

It was concluded from the full-scale tests that the bridge performed satisfactorily under all test conditions and that the prestressed panel technique is a suitable method for constructing bridges.

Additional Cyclic Loading Tests

Furr and Ingram[4] conducted additional cyclic loading tests to investigate the Z-bar shear connectors. Four $3\frac{1}{2}$ x 22 x 92 in. were fabricated without and three with the Z-bars spaced at 18 in. on center. In the test program failure was assumed to be a condition of panel inserviceability or $\frac{1}{4}$ in. deflection.

By loading the test panels at 210% of the design loading the ones with the shear connectors took 11.9 million cycles of loading without failure. The panels without the shear connectors failed by deflection after 2.25 million cycles at 210% of design loading.

In static testing, both panels had the same stiffness up to approximately the design load. Beyond the design load the panel with shear connectors was stiffer.

PRESTRESSED SUBDECKS ON STEEL GIRDER BRIDGES

Prestressed panel subdecks have been used only on prestressed concrete girder bridges in all applications known to the writer at this time. A continous steel girder bridge in Texas that was to be redecked, however, was contracted with the option of using or not using prestressed panels. The contractor elected not to use the panel subdecks. Had the panels been used they would have been of the same design as is used on the Texas prestressed concrete girders, and they would have been erected in a similar manner. The particular steel structure in question was composed of three continuous units. Each unit consisted of four 85 ft. spans. The shear connector details and the panel bearing details at the top flange of the continuous girders are shown in Figures 6 and 7. It would appear from these details that prestressed panels could be used on steel girder bridges whenever the width of the upper flange is sufficient to provide both a bearing area for the panels and mounting space for a sufficient number of shear connectors to satisfy the design requirements.

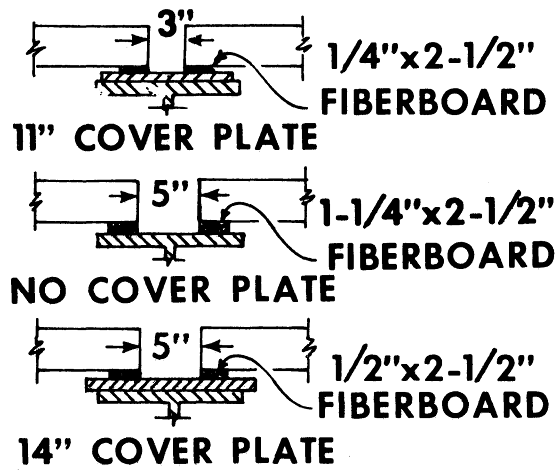

Figure 6. Variations in fiberboard thickness on continuous steel girders utilizing prestressed subdeck panels.

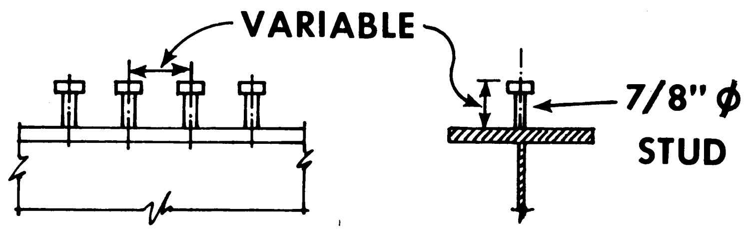

Figure 7. Shear connector details for continuous steel girder utilizing prestressed sub-deck panels.

PENNSYLVANIA STUDIES

Studies similar to those conducted in Texas are under way at Penn State University. Barnoff and Orndorff [8] have reported on the construction and testing of an experimental bridge. This bridge is composed of two 60 ft. prestressed concrete girder spans. One span is constructed by using a 3" thick prestressed panel subdeck with a $4\frac{1}{2}$ in. cast-in-place slab. The second span was constructed using steel stay-in-place forms and regular wood forms—each covering half the area of the deck.

Only static load tests on the structure have been reported to date. All results tend to support those discussed earlier concerning the studies in Texas. Full composite action was developed between the panels and deck without the use of shear connectors. Full composite action was also obtained between the deck slab, panel and girders. No significant separation of the joints between the panels occurred during static loading tests. Measured displacements of each span indicated that the prestressed panel span deflected slightly more than the conventional span. Future tests will investigate this finding further. One million cycles of an 18-kip axle load and overload tests to failure of the deck are to be conducted.

Laboratory tests conducted by Barnoff and Rainey [9] appear to have yielded results also similar to those obtained by Furr, et al. [1-3] In a general sense the results concerning bond, load transfer across the panel joints, and the load capacity of the panels are in agreement with the Texas results. The strand development lengths were also of the same general order of magnitude—noting that the studies by Barnoff utilized a 7/16 in. diameter strand length with an initial tensile force of 21.7 kips per strand applied, whereas the Texas studies utilized $\frac{3}{8}$ in. diameter strands under an initial tensile force of 13.75 kips per strand.

OTHER APPLICATIONS OF PRESTRESSED PANEL SUBDECKS

The prestressed "plank" for deck forms was first used in 1957 on the Illinois Toll Highway.[10] These structures have performed well. The prestressed planks used were $2\frac{1}{2}$ in. thick and utilized $\frac{3}{8}$ in. diameter strands stressed with 16.1 kips of force for each strand. These planks, like those used in the Pennsylvania study, were placed on a mortar bed on the top edges of the prestressed concrete girders. Four rows of shear ties were also used at 1 ft. on center spacings. The same approach to bridge construction was to be used on extensions of the Illinois Toll Highway.

The state of Missouri also plans to build a bridge which will utilize prestressed panel subdecks.[11] The design of the panels for this bridge will draw upon the experience and research on the technique developed in Texas. While other states may be planning to use the technique now or in the future, no other works are known to the writer at this time.

BRIDGE DESIGN USING PANEL SUBDECKS

In the design of prestressed panel subdecks, the panels are designed to carry the dead weight of the cast-in-place concrete slab and then they are assumed to form a composite section with the slab in resisting live load moments. In the negative moment region over the beams, a cracked section reinforced concrete design is made on the total depth of the slab and panel. The transverse reinforcing in the cast-in-place slab is designed to resist all of the tension due to live load moment.

In the positive moment region between girders, the gross section (panel plus slab) is designed to resist the live load moments which are superimposed on the dead load panel moments without creating tension in the prestressed concrete panel. The usual AASHO distribution formula for slabs on girders is used. Composite action of the panel, slab, and girder is assumed as is the case when designing for a full thickness deck.

The concrete for the panels should have a minimum 28-day strength of 5,000 psi and release strength of 4,000 psi. The panels usually bear on 1 in. wide bituminous fiberboard material, which is mounted between the panel and top flange of the girders. The bearing areas on the outside edges of the girders are troweled smooth and the top surface of the prestressed panels are broomed to enhance bonding to the deck slab. The 3/8" diameter prestressing strands are tensioned to 16.1 kips per strand.

While the results of the research studies have indicated no need for the shear ties between the panel and slab under design loadings Texas plans to continue using a nominal number of ties.[7]

Details of the most recent Texas standard design are given in the Appendix.

SUMMARY AND CONCLUSIONS

The experience and research results available to date indicate that the prestressed panel subdeck with cast-in-place concrete slab is a reliable and suitable method for constructing simple span prestressed concrete girder bridges. There appears to have been little application of this technique to steel girder bridges although one continuous girder bridge contracted in Texas offered this approach as an alternate. Texas highway officials feel that the technique could be used on steel as well as prestressed type girders.[7]

Cracking in the bridge deck surface directly above the butt joints between the prestressed panels can be expected at many of the joints. No method of preventing the cracking has been discovered, but the cracks have not been found to have any significant effect on the performance of the structures. Tests have indicated that the cracks terminate approximately half way through the cast-in-place slab.

RECOMMENDATION

Based on the material reviewed it is concluded that the use of the prestressed panel subdeck is a suitable technique for constructing bridges. Furthermore, the prestressed panels now used by Texas have had the benefit of considerable research and field experience. It is therefore recommended that the standard panel details used by Texas, and included in the Appendix, be adopted by Virginia, and any necessary modifications be applied to these details to make them compatible with Virginia specifications and/or standards for use on prestressed concrete girders.

Adopting the prestressed panel forming technique to a particular steel girder type bridge should be given further design consideration, since the technique should also be applicable to this type structure.

ACKNOWLEDGMENTS

Thanks are extended to Bob Barnoff, Howard Furr, and Wayne Henneberger, respectively of the Pennsylvania State University, Texas A & M University, and the Texas Highway Department, for providing the research reports and other information for the writer.

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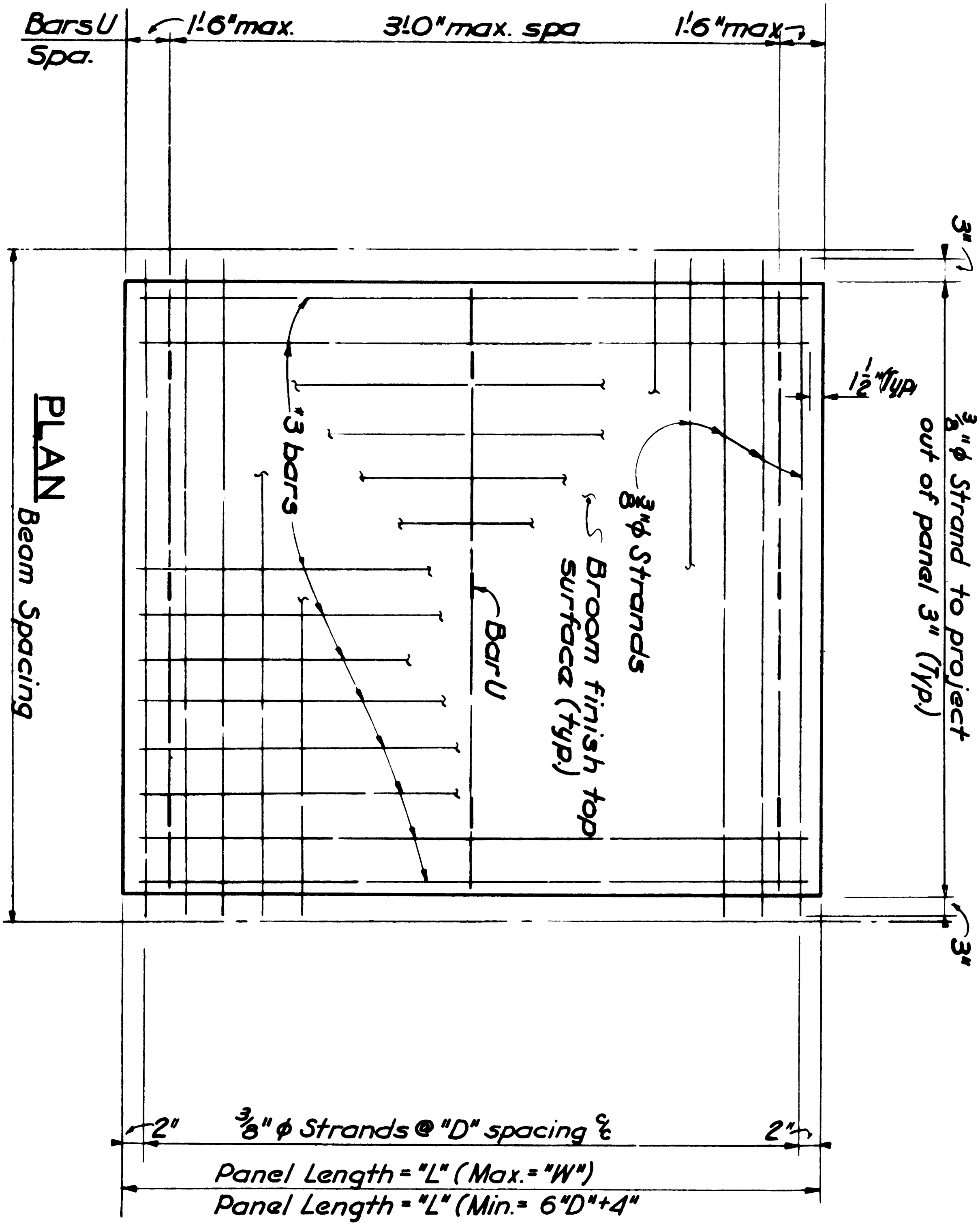
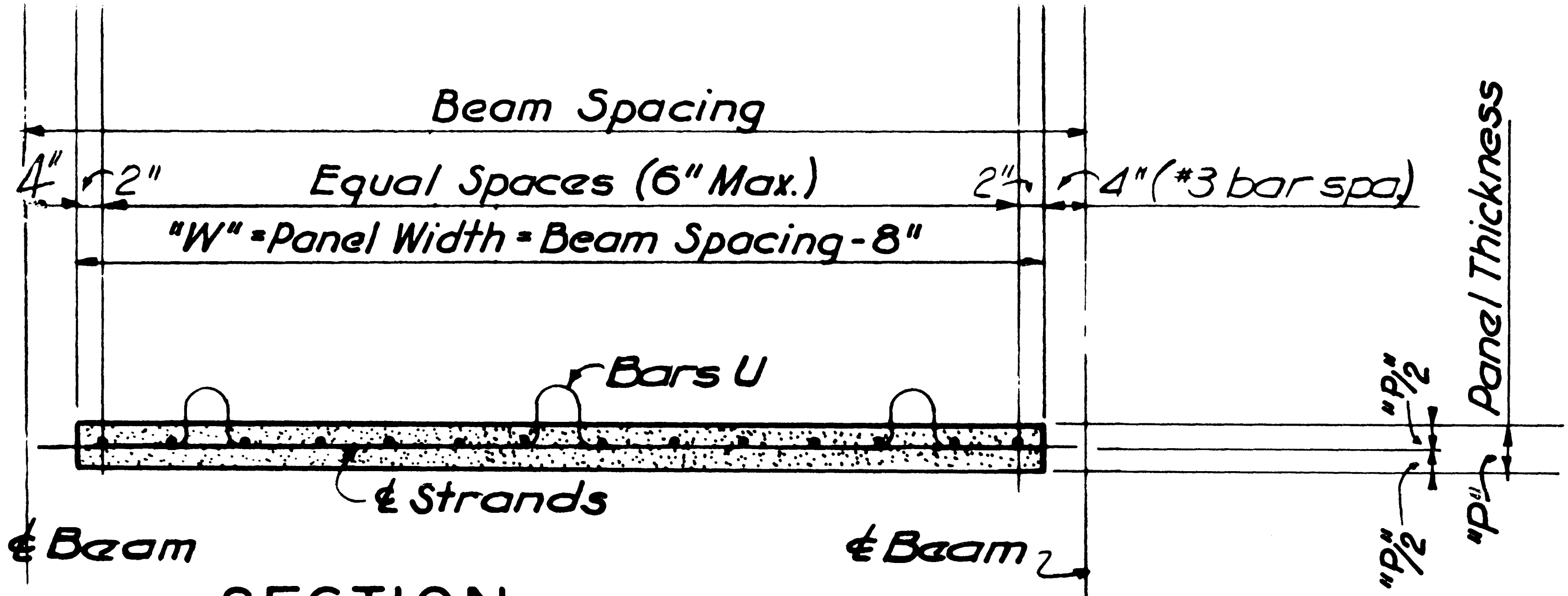
APPENDIX

Recent Standard Design Details For Prestressed Concrete Panel Subdeck Used in Texas.

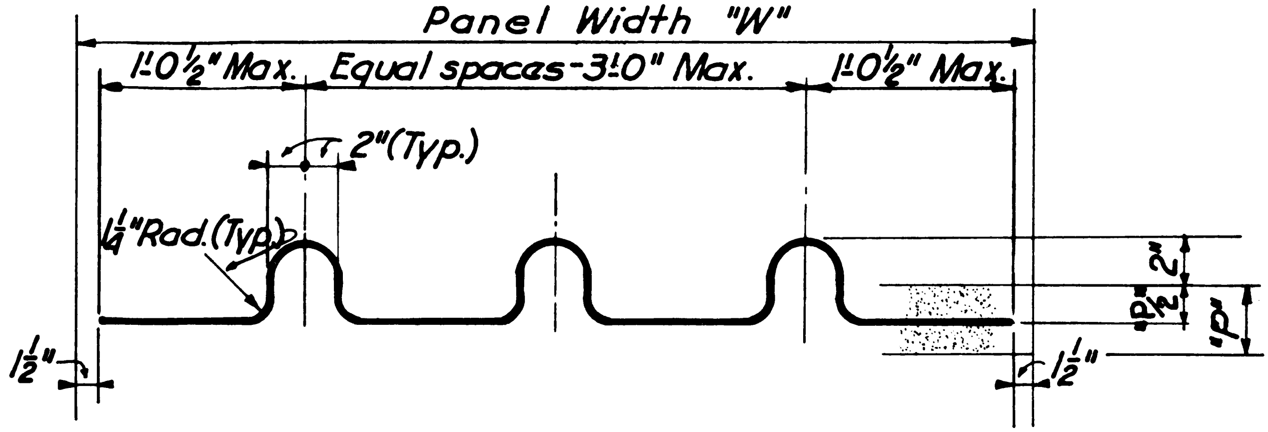


Bars U
Spa.
1'-6" max.
3'-0" max. spa
1'-6" max.
3"
3"
1½" Typ.
3/8" φ Strand to project out of panel 3" (Typ.)
PLAN
Beam Spacing
#3 bars
3/8" φ Strands
Broom finish top surface (typ.)
Bar U
2"
3/8" φ Strands @ "D" spacing c/c
2"
Panel Length = "L" (Max. = "W")
Panel Length = "L" (Min. = 6"D" + 4"

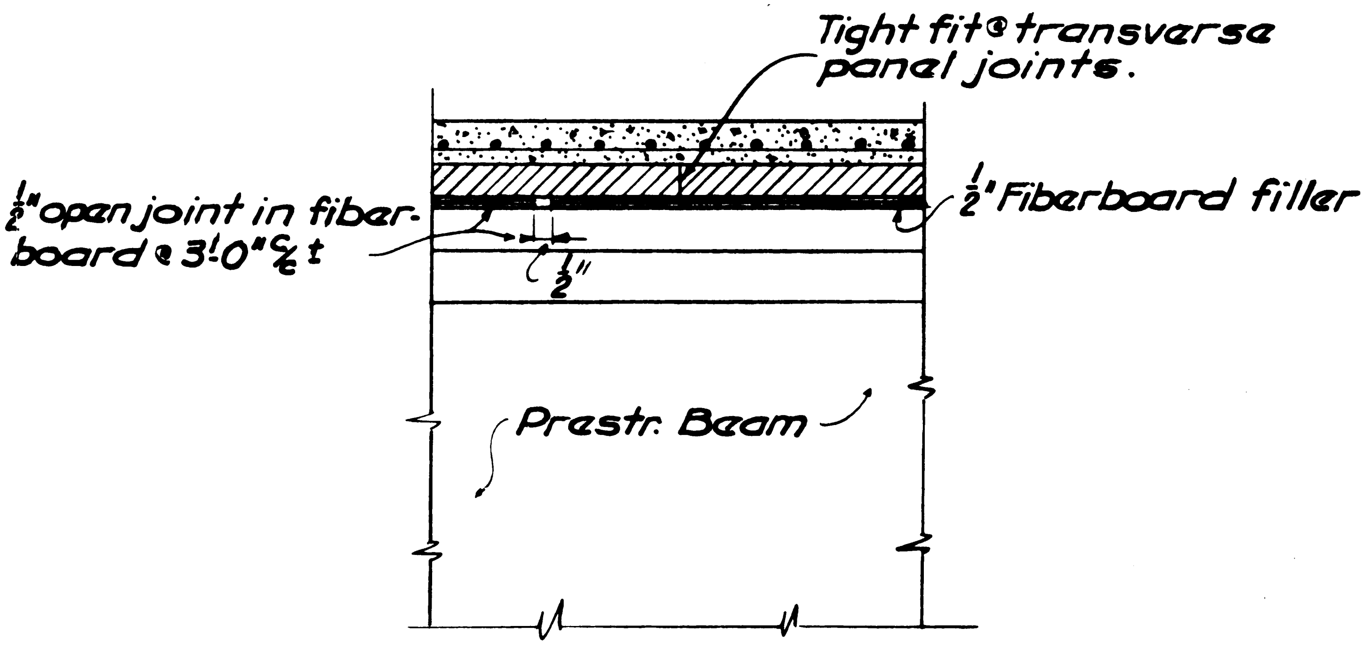


SECTION

DESIGN DIMENSIONS				
Beam Spacing	"C"	"P"	"D"	"E"
5.00' thru. 5.72'	$3\frac{1}{2}$"	$3\frac{1}{4}$"	$6\frac{1}{2}$"	$6\frac{3}{4}$"
5.73' " 6.26'	$3\frac{3}{4}$"	$3\frac{1}{4}$"	$6\frac{1}{4}$"	$6\frac{3}{4}$"
6.27' " 6.94'	4"	$3\frac{1}{4}$"	$5\frac{1}{2}$"	$6\frac{1}{2}$"
6.95' " 7.63'	4"	$3\frac{1}{2}$"	5"	$6\frac{1}{4}$"
7.64' " 8.35'	$4\frac{1}{4}$"	$3\frac{1}{2}$"	$4\frac{1}{2}$"	6"
8.36' " 9.07'	4"	4"	4"	$5\frac{3}{4}$"
9.08' " 10.00'	$4\frac{1}{4}$"	4"	$3\frac{1}{2}$"	$5\frac{1}{2}$"



BAR U(#3)



SEC. THRU TRANSVERSE PANEL JOINTS