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STATE OF THE ART: PERMANENT BRIDGE DECK FORMS

subject areas
27 bridge design
32 cement and concrete
33 construction
34 general materials
40 general maintenance

FOREWORD

This report was prepared to summarize the research that has been conducted and the experience that has been gained relative to the use of two types of permanent forming for bridge decks. Part I of the report deals with the use of prestressed panel subdecks and Part II with permanent steel forms. Both techniques represent progressive approaches to bridge deck construction and both offer attractive advantages to the bridge construction industry. New techniques, however, often create new problems that must be recognized and understood by potential users; only then can they be applied to maximum advantage. The report represents a state-of-the-art review conducted to assemble the currently available information on these two techniques so that those responsible for selecting or approving them can have a knowledgeable basis for decisions.

INTRODUCTION

A number of factors, both economical 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 -- most notably, 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. Permanent steel forms have been used in several northeastern states. Another relatively recent approach has been the use of precast prestressed panel subdecks which 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 some states to date and others have structures in the planning and design stage which will utilize precast prestressed panel subdecks.

Highway bridge contractors have become increasingly interested in permanent type forms for several reasons. First, the uncertain economic conditions in the early seventies and strong foreign demand for wood and plywood tended to cause wide fluctuations in lumber prices, thus, at times, making other materials more competitive. Occasionally, certain types of lumber have 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 permanent type forms could reduce the amount of time and labor required to form a bridge deck.

One of the primary ways of containing 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 reduction in the number of production operations required and/or by increased automation. In bridge construction the use of permanent 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 permanent forms and weigh those against the advantages before embarking upon widespread use of the technique.

At this writing, only one state transportation department is known to be making routine use of prestressed panel subdecks, and, as mentioned previously, several states are now routinely using permanent steel forms. On the other hand, some states have reservations regarding the use of either of these two techniques for the construction of bridge decks. In the case of prestressed panel subdecks, some organizations are not fully aware of the research and the actual experience that has been gained in the use of this technique. In the case of permanent steel forms there is concern about the long term effects the forms might have on the durability of concrete bridge decks and concern regarding future corrosion problems that might develop in conjunction with the use of forms. The purpose of this review, however, is to present some of the available information on each of these two techniques and to evaluate the current state of the art.

PART I

PRESTRESSED PANEL SUBDECKS

Considerable laboratory and field work to investigate the use of prestressed panel forms for bridge decks (Figure 1) has been conducted in Texas. Further laboratory and full scale tests have been conducted in Pennsylvania and Florida. In view of the amount of research that has been completed, plus the in-service experience gained in several areas of the country, this state-of-the-art review is an effort to synthesize this information. 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, some problems that have been identified to be peculiar to the use of precast prestressed panel type forms are discussed.

Texas Studies

The Texas Department of Highways and Public Transportation has used the precast prestressed concrete panel technique for the construction of bridge decks for a number of years. Three overpass structures that were constructed by this method 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 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 a bridge completed in 1973 between Corpus Christi and Padre Island, for example, the panels were used on all 36 spans approaching the main cantilevered box girder spans.[5] Texas' largest bridge project, which consists of one hundred forty-seven 80 ft. (24.4 m) prestressed girder spans also utilizes the panels. At this time more than 1,500,000 sq. ft. (139,400 m^2) of subdeck panels are in service on Texas bridges..

Studies of Three In-Service Bridges

The first of the three in-service bridges investigated consisted of two end spans of 45 ft. (13.7 m) length and two interior spans of 60 ft. (18.3 m) length. Each span consisted of four simply supported prestressed girders with a lateral spacing of 6 ft.-8 in. (2 m) on center. The 3 in. (76 mm) thick prestressed panel subdecks were 6 ft.-2 in. (1.9 m) long and 4 ft. (1.2 m) wide. (Details of the Texas standard prestressed panel design are given in Appendix A.) The remaining two bridges were twin structures -- each consisting of two 40 ft. (13.7 m) and three 50 ft. (15.2 m) simply supported prestressed girder spans. The six girders in each span were spaced at 7 ft.-3 in. (2.2 m) on center and the 3 in. (76 mm) thick subdeck panels were 6 ft.-9 in. (2.1 m) wide and varied from 1 ft.-5 in. (43 cm) to 5 ft.-2 in. (1.57 m) in length. A typical panel layout on a 50 ft. (15.2 m) span is shown in Figure 2. All of the panels on the three bridges had 3/8 in. (10 mm) diameter 7-wire strands prestressed at 14 kips (6.35 Mg) per strand. The strands were spaced at 4-1/2 in. 114 mm) on center at mid-depth of the panels. Running transverse to the prestressed strands, number 2 plain reinforcing steel was spaced at 6 in. (152 mm) on center.

The cast-in-place concrete portion of the decks was 3 in. (76 mm) thick and reinforced with number 5 bars spaced at 15 in. (381 mm) 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. (381 mm) 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 2. 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 (1.97 MP_a).

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 slips 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 (32.6 Mg) 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 span of the panel subdecks is a factor to consider. Tests were conducted on 3-1/4 in. (83 mm) thick panels to determine the development length of the strands both initially and after a repeated loading. For 3/8 in. (10 mm) diameter 7-wire strands tensioned with a force of 13.75 kips (6.24 Mg) an average development length of 22 in. (55.9 cm) was required. For 1/2 in. (13 mm) diameter strands tensioned with a force of 27.50 kips (12.5 Mg) each, an average 34 in. (86.4 cm) 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 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. (1.74 m), the full prestress force could be developed in each case. When the 1/2 in. (13 mm) diameter strands were used in the 68 in. (1.74 m) 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 1400 psi (9.64 MPa) of compression and zero tension in the prestressed panels since 700 psi (4.83 MPa) of compression was induced by the prestressing. The results of these tests showed that the cyclic loading had a neglible 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, Toprac and Jones[3] on a full scale 23 ft. (7 m) wide, 50 ft. (15.2 m) simply supported span composed of prestressed concrete girders, 3-1/4 in.(83 mm) thick prestressed subdeck, and a 3-1/2 in. (89 mm) thick cast-in-place deck. The experimental structure was designed for an HS20-44 loading. 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. 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 variable, 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 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. 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. (.05 mm) 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 provide transverse support for the prestressed panels. For shot 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 tests 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 3-1/2 x 22 x 92 in. (8.9 cm x 55.9 cm x 2.34 m) panels were fabricated without and three with the "Z" bars spaced at 18 in. (45.7 cm) on center. In the test program failure was assumed to be a condition of panel inserviceability or 1/4 in. (6 mm) 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 writers at this time. A continuous 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

Figure 1. Prestressed panel subdecks in place on a bridge deck in Texas.

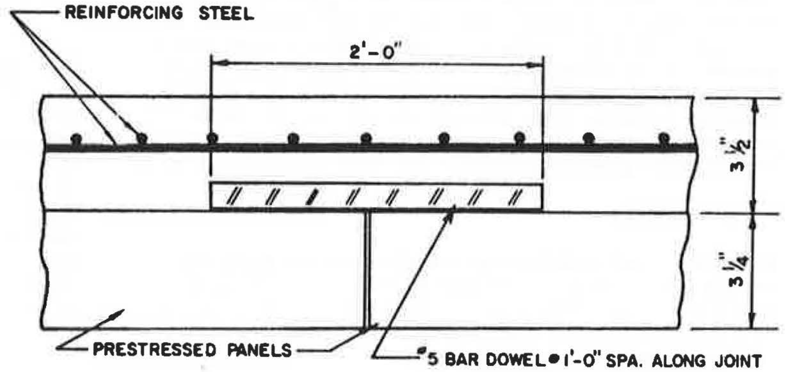


Figure 4. Dowel bars used at selected panel butt joints (3).

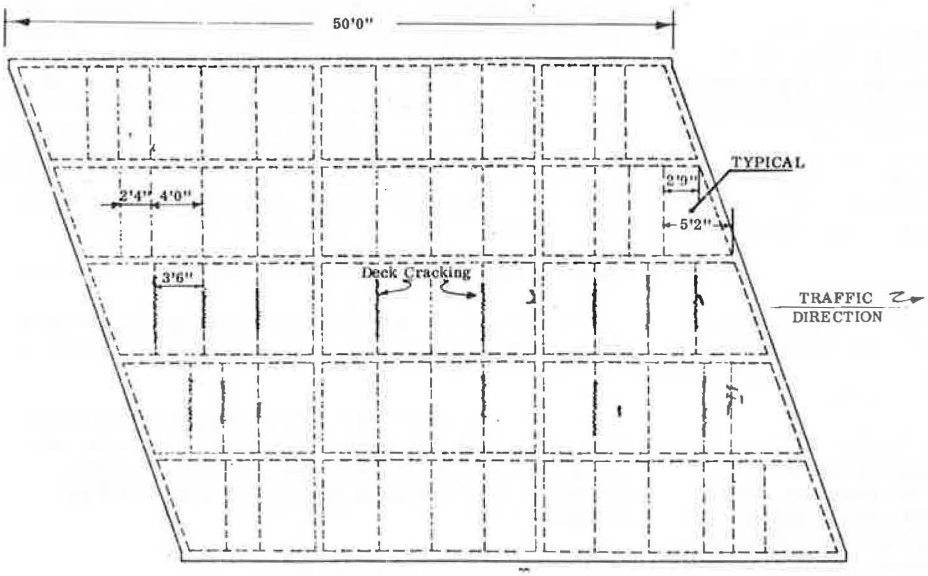


Figure 2. Typical panel layout and cracking on deck surface above subdeck joints (1).

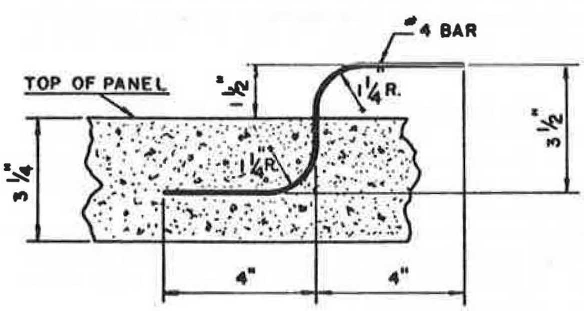


Figure 3. Z-bars used in selected panels to aid in providing structural connection between panel and cast-in-place deck (3).

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 span units. Each continous unit contained four 85 ft. (25.9 m) 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.

Pennsylvania Studies

Studies similar to those conducted in Texas have been conducted at Penn State University. Barnoff and Orndorff[8] have reported on the construction and testing of an experimental bridge composed of two 60 ft. (18.3 m) prestressed concrete I-beam spans. One span was constructed by using a 3 in. (76 mm) thick prestressed panel subdeck with a 4-1/4 in. (114 mm) cast-in-place slab. The second span was constructed using steel stay-in-place forms and regular wood forms -- each covering half the span length.

The structure has been tested with a semi-trailer truck moving at crawl speeds, a five axle vehicle traveling at 45 mph (72 km/hr.) (which produced the equivalence of 1,100,000 repetitions of an 18 kip (8.16 Mg) axle load), an HS20 truck, and finally with progressively increasing overloads to cause structural damage to the bridge. Overloads were applied with a tandem axle semi-trailer which was constructed to permit extremely large wheel loads to be developed.

It should be noted that there were some significant differences between the panels used in Pennsylvania and those used in Texas. Details of the panels used at Penn State are shown in Figures 8 and 9. There were no ties to develop composite action between the panels and the cast-in-place topping. Full composite action was obtained by using a roughened surface on the top of the panels. Prestressing strands projected 6 in. (152 mm) from the ends of the panels to provide a connection between the panels and the cast-in-place topping, and to add to the development length of the strand. The strand was used as the bottom reinforcement for the slab in the transverse direction. All prestressed panels were set on cement grout. On one-half the span the grout had a nominal thickness of 1/4 in. (6 mm) and the depth of the cast-in-place topping varied. For the other half of the span the grout thickness varied and the topping was maintained at 4-1/2 in. (114 mm).

In addition to varying the depth of the grout, other construction options were investigated during the construction of the bridge. On one portion of the span the panels were cantilevered over the fascia beam and served as the bottom form for construction of the parapet and curb. This detail eliminated the need for elaborate supports required to hold the parapet forms during construction. Shear connectors cast in the fascia beam at this location projected through preformed holes cast into the panels. A similar exterior panel design was used by Buth, et al.[3] in their full scale tests.

Interior diaphragms were not used on the experimental bridge at Penn State, but full depth diaphragms were used at the fixed supports. At one location the panels were detailed to cover the area of a full depth diaphragm, and holes were provided to place the concrete in the diaphragm. This detail was costly and time consuming for the contractor, and difficulty was experienced in placing and vibrating the concrete.

After construction was completed the contractor was questioned on his opinion of the relative cost of the deck constructed with precast panels versus the conventional deck. Although no monetary values were given, the contractor did indicate that considerable labor was saved in the construction of the deck with precast panels. This labor savings was the result of the following factors:

1. Panels were placed rapidly with only minimum fitting and no fastening required.

2. Only one layer of deck reinforcement had to be placed.

3. In the area where the panels cantilevered over the fascia beam, bottom forms were not required for the parapet and curb concrete.

4. A smaller quantity of concrete was placed for the deck with panels versus the conventional deck.

Laboratory tests conducted by Barnoff and Rainey[9] appear to have yielded results generally similar to those obtained by Furr et al.[1, 2, 3] In a general sense the results concerning bond, load transfer across the panel joints, and load capacity of the panels is 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. (11 mm) diameter strand length with an initial tensile force of 21.7 kips (9.84 Mg) per strand applied.

One of the objectives of the laboratory tests conducted by Barnoff and Rainey was to compare different types of joints between the panels. Three different joint types were studied: a butt joint, a beveled joint, and a U-joint. There were no significant differences in the structural performance of panels made with these different joints. It should also be noted that simulated wheel loads of 80 kips (36.3 Mg) applied directly over the joint were required to fail the panels. Failure occurred when yield lines formed, and delamination between the panels and cast-in-place topping did not occur even though shear connectors were not used.

The researchers also made observations during fabrication of the prestressed panels. No difficulties were encountered in fabrication except for longitudinal cracking along the center strand on a large number of panels. This cracking was observed immediately after release of the prestressing force, and was caused by the nonuniform spacing of the prestressing strand as shown on Figure 8. Larger strains were produced along the edges of the panels than in the center, and tensile stresses caused cracking along the center strand. This cracking did not affect the structural strength of the panels.

Results

The load tests on the Penn State bridge did not

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

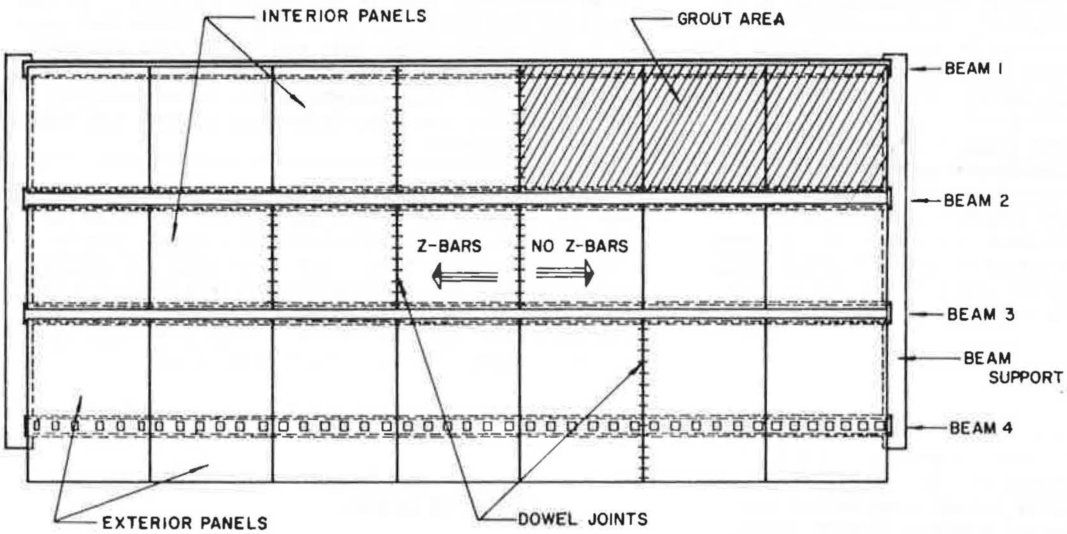


Figure 6. Variations in fiberboard thickness on continuous steel girders utilizing prestressed subdeck panels.

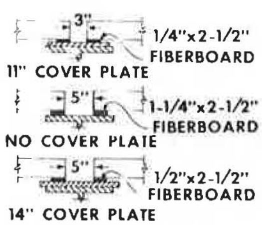


Figure 7. Shear connector details for continuous steel girder utilizing prestressed subdeck panels.

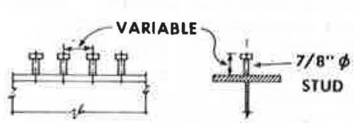
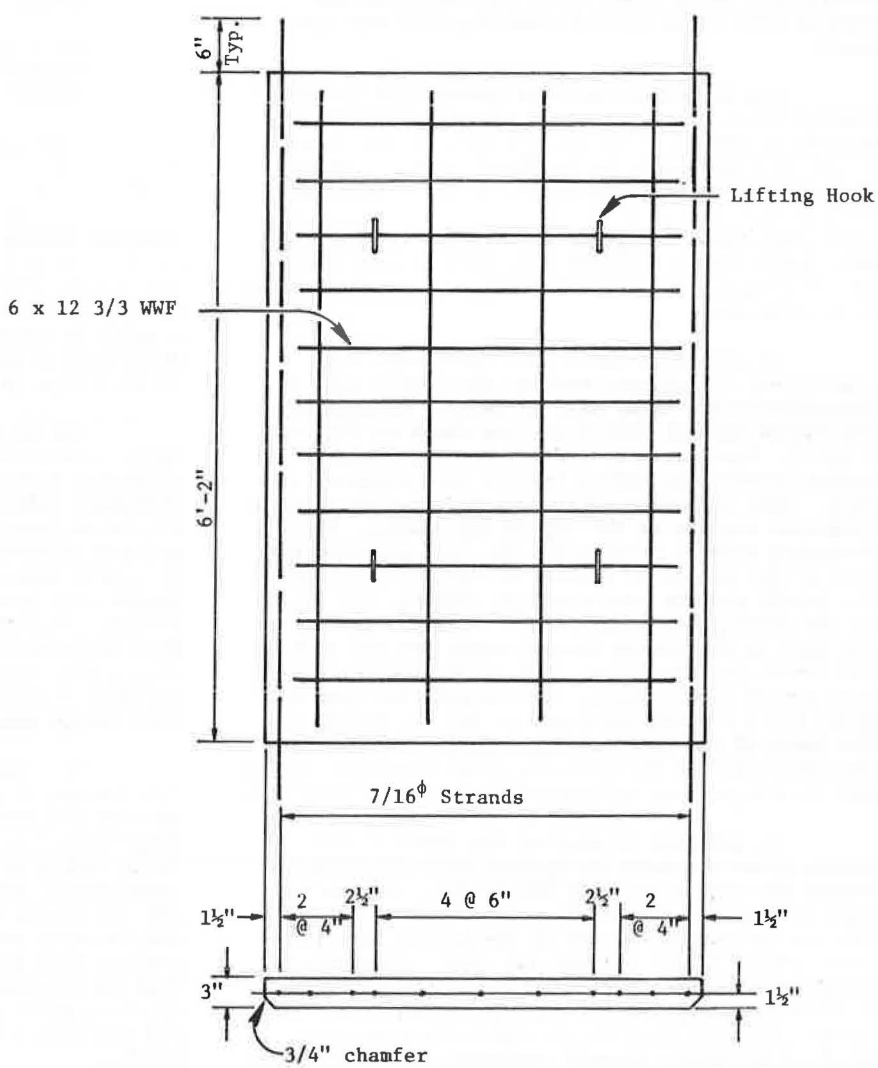


Figure 8. Form panel dimensions and details (Penn State experimental bridge).



reveal any deficiencies in the deck constructed with precast prestressed panels. None of the loading caused structural failure in the panel subdeck. Instrumentation was provided to measure strains in the deck and girders, displacement of the girders, slip between the panels and girders in the longitudinal direction, torsional rotation of the girders, and strains in the deck reinforcement. Few of the measurements showed any correlation with computed stresses using standard design assumptions. Without exception, measured values were considerably lower than computed stresses. For example, at axle loads of 100 kips (45.4 Mg) the maximum measured reinforcing steel stress was approximately 16,000 psi (110 MPa) as opposed to a calculated stress of 20 ksi (138 MPa) for HS20 loading. Other measurements and observations showed similar discrepancies between design and measured values. Flexural cracking of the concrete decks was not detected until axle loads of 40 kips (18.1 Mg) were applied to the decks.

Normal loading on the bridge did not show any deficiencies in the deck constructed with prestressed panels. No difference in behavior of the deck was observed after 1,100,000 cycles of an equivalent 18 kip (8.16 Mg) axle load and 1,000 cycles of an HS20-44 truck. Separation between the bottom of the panel and the top of the grout was observed after 500,000 cycles of the equivalent 18 kip (8.16 Mg) axle loads. Cores taken from these areas showed that the separation extended only to the ends of the panel. Inspection of the cores also indicated that there was no delamination between the panels and the cast-in-place concrete. Composite action between the girder and deck was not affected. In general, the deck constructed with prestressed panels performed as well as the conventional deck during the two-year testing period.

Florida Studies

Extensive studies of the precast panel subdecks have been conducted by Sawyer and Kluge[9] at the University of Florida. The purpose of these studies was to determine the extent of interaction between a cast-in-place concrete deck and a precast panel subdeck without shear connectors and to develop design criteria for this type bridge deck system. All of the 57 test panels used in the study were 3 in. (76 mm) thick and had 7/17 in. (11 mm) diameter, 270 ksi (1862 MPa) strands spaced at 9 in. (229 mm) on center. The study was limited to laboratory testing which was performed in a manner to simulate loading on an actual bridge.

Results

The results of the study showed that prestressed panel subdecks with a cast-in-place deck behave the same as monolithic concrete decks under static and repeated loads as long as no foreign material is allowed on the panel -- overlay interface during construction. The deleterious effect of punching shear and the notch effect at panel joints was found to be insignificant and it was concluded that design for shear would not be necessary. Because the results of their studies indicated that strand development length in the panels is usually less than the AASHTO requirements, Sawyer and Kluge[9] recommended the use of transverse reinforcement no less than no. 3 bars at 12 in. (305 mm) on center and special precautions during fabrication to keep foreign substances off the prestressing strands. In addition, minimum embedment lengths including the free strand ends should not be less than 62 in. (1.57 m) for 3/8 in. (9.5 mm) strands and 80 in. (2.03 m) for 7/16 in. (11.1 mm) strands.

Other Applications of Prestressed Panels

The prestressed "plank" for deck forms was first used in 1957 on the Illinois Toll Highway and the bridge decks constructed by this technique have performed well.[10] The prestressed subdeck panels used were 2-1/2 in. (63.5 mm) thick and utilized 3/8 in. (9.5 mm) diameter strands stressed with 16.1 kips (7.3 Mg) of force for each strand. These panels, 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. (305 mm) on center spacings. The same type construction was used on extensions of the Illinois Toll Highway.

The state of Missouri has also constructed a bridge which utilized prestressed panel subdecks.[11] The design of the panels for this bridge drew upon the experience and research developed in Texas. Louisiana has used 3-1/2 in. (39 mm) thick subdeck panels on at least one bridge and the state of Virginia plans to construct its first bridge by this technique during 1976. Additional states not known to the writers may also have used, or plan to use, this technique in the future.

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.

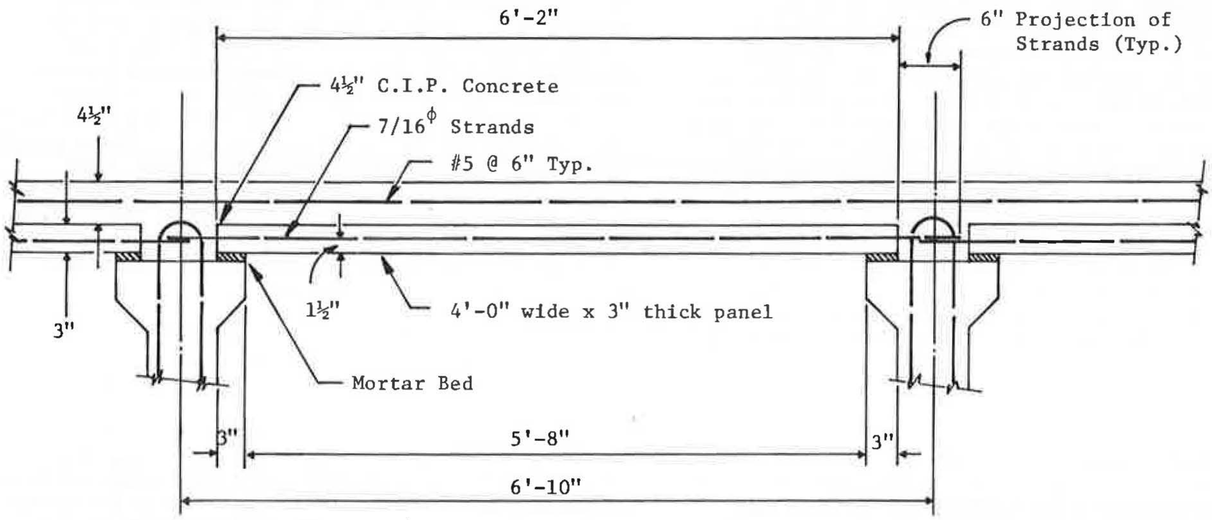
In the positive moment region between girders, the gross section (panel plus slab) is designed to resist the live load moments which are superimposed on the dead load panel moments without creating tension in the prestressed concrete panel. The usual AASHTO distribution formula for slabs on girders is 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 (34.5 MPa) and release strength of 4,000 psi (27.6 MPa). However, the panels used on the Penn State bridge were designed for a release strength of 3,000 psi (20.7 MPa) and an ultimate 28-day strength of 4,000 psi (27.6 MPa). The panels usually bear on grout or on 1 in. (25.4 mm) 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" (9.5 mm) diameter prestressing strands are tensioned to 16.1 kips (7.3 Mg) per strand, and the 7/16" (11.1 mm) strands are tensioned to 21.7 kips (9.8 Mg). The strands normally used are 270 kip grade.

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

Details of a recent Texas standard design are given in Figures 10, 11 and 12.

Figure 9. Typical details of precast, prestressed concrete form panel and cast-in-place topping for Penn State experimental bridge.



Design Notes: 7/16ᶲ - 270 k strands.
Initial force = 21.7 kips/strand
Conc. for panel, f'_{ci} = 3000 psi, f'_c = 4000 psi

Figure 10.

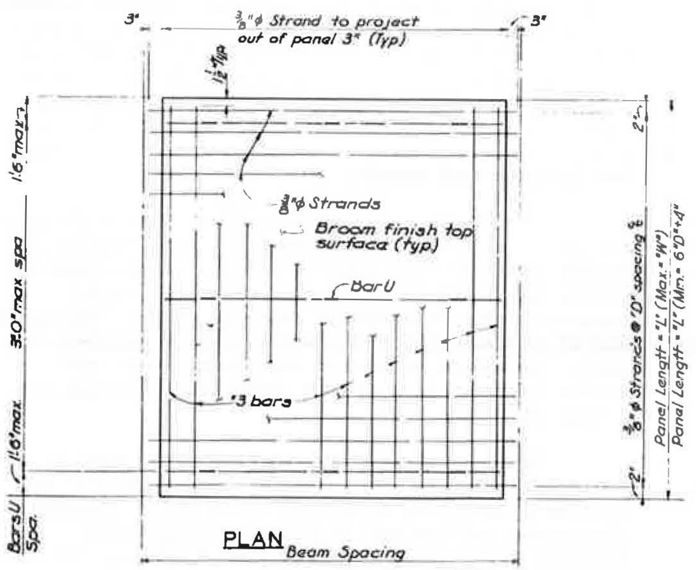


Figure 12.

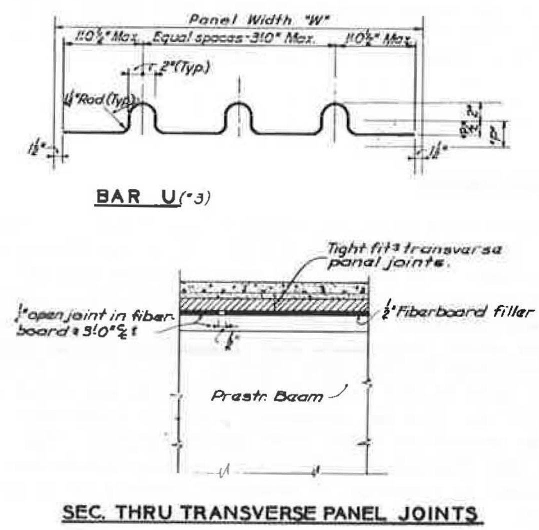
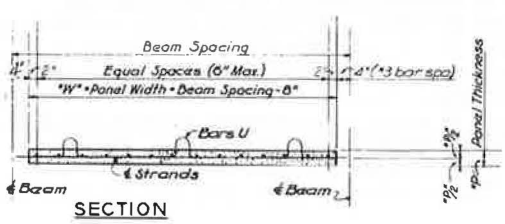


Figure 11.



DESIGN DIMENSIONS

Beam Spacing	"C"	"P"	"D"	"E"
5.00' thru 5.72'	3¼"	3¼"	6½"	6¾"
5.73' " 6.26'	3¾"	3¼"	6¼"	6¾"
6.27' " 6.94'	4"	3¼"	5½"	6½"
6.95' " 7.63'	4"	3½"	5"	6¼"
7.64' " 8.35'	4¼"	3½"	4½"	6"
8.36' " 9.07'	4"	4"	4"	5¾"
9.08' " 10.00'	4¼"	4"	3½"	5½"

Summary

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 or no 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.

Of the different types of joints between panels that were investigated the simplest butt joint was found to be as good as any other. Composite action between the subdeck panels and cast-in-place deck and load transfer across the joints between adjacent panels has not been found to be a problem. Performance of the bridge decks that have been constructed with the subdeck panels have been satisfactory to date.

Conclusions

Based on the research and experience to date the prestressed panel subdeck appears to be a suitable technique for constructing bridge decks. Furthermore, the prestressed panel subdecks now used by Texas have had the benefit of considerable research and field experience and the experience in Illinois has been favorable. It is therefore concluded that the standard panel details used by Texas, for example, and included in Appendix A, could be adopted by any highway and transportation organization with any necessary modifications being applied to make the design compatible with their specifications and/or standards for use on prestressed concrete girders. An additional advantage of the panels as opposed to permanent steel forms is that the potential for future corrosion problems is greatly reduced.

Adopting the prestressed panel forming technique to a particular steel girder type bridge should be given consideration during the design of the bridge. There appears to be no reason why the precast panel technique could not be used on steel as well as on prestressed girder type structures.

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PART II

PERMANENT STEEL FORMS

While prestressed concrete panel subdecks have been used by a number of states, permanent steel forms (Figure 1) have probably been used on a larger number of bridges throughout the country. Many states, however, do not allow the use of these type forms at all, whereas others allow contractors to use them on an optional basis. Still others allow the use of permanent steel forms only on special request or in special situations where their use is deemed to be advantageous.

Whether representing a state that allows or disallows use of the steel forms, most highway engineers and administrators appear to have reservations concerning their use. These reservations primarily revolve around the long-term durability of the bridge deck concrete as it might be affected by the permanent forms and the long-term corrosion resistance of the forms themselves. In the former case, it is feared that entrapment of moisture and/or deicing salts between the forms and the deck might cause or accelerate deterioration of the reinforced concrete deck. In the latter case, it is feared that corrosion of the forms themselves may lead to future maintenance problems and/or the necessity of ultimate form removal. Either of these possibilities would require expensive corrective operations and are just cause for concern.

From a construction viewpoint, however, the use of permanent forms is a logical step and the pressure for their use is likely to increase. Undoubtedly, there are potential risks involved in using permanent steel forms as well as a number of disadvantages which sometimes may tend to offset initial construction advantages. This state-of-the-art review is an attempt to assemble much of the available information regarding the experience that many of the states have had as well as to examine certain research results that may be relevant to the use of permanent steel forms on bridge decks. Specifically, the review was undertaken for the following purposes:

1. To summarize the advantages and disadvantages that have been noted from the experiences many of the states have had with the use of permanent steel forms
2. to review and summarize the pertinent research results that are available in the literature
3. to evaluate the potential long-term detrimental effects of the forms as related to concrete deterioration and steel corrosion, and
4. to determine some precautions that can be taken to minimize the disadvantages that are related to the use of permanent steel forms.

The scope of the material contained in this review is limited to the use of permanent steel forming on bridge decks. Consideration is given to both the use of permanent forms as a construction material and to an analysis of the potential long-term effects the forms might have on the future integrity of a structure.

Experience Survey

Permanent steel forms have been used for the construction of bridge decks for a number of years. Although not extensively used during the early years of their availability, permanent steel forms have been installed on a considerable number of bridges in the last 10 to 15 years. At the outset of the survey it was suspected that a few states in the northern regions might have installations on the order of 20 years old. It was known, however, that several of the northeastern states had made the most extensive use of steel forms. Accordingly, all of the northeastern and a large sample of the remaining states were queried with regard to their experiences in the use of permanent steel forms on bridge decks. A letter of inquiry was mailed to the construction engineers in 36 states and direct communication was made with the bridge engineers of 2 additional states. Specifically, these engineers were requested to supply general information regarding the problems that their state might have had with the use of steel forming. In addition, each state was asked whether they had observed concrete deck deterioration that could be associated with the use of steel forms, or if they had observed any corrosion of the forms on their older installations.

The majority of those states that were not surveyed are located in the western and southwestern regions of the country. Only a random sampling of these states was taken since preliminary information had indicated that they had built few structures using steel forms. The results from those western states surveyed tend to support that observation.

Replies were received from all but 3 of the total 38 states contacted. Of the 35 responding states, only 8 had generally permitted the contractor the use of permanent steel forms as an alternate (Table 1). One of these would not allow their use over salt water. As also shown in Table 1, however, 13 additional states have allowed the use of permanent forms on some projects or on projects under contract upon special request by the contractor. An additional 6 states have a general policy of not permitting the use of permanent forms except in special situations. Some typical special situations were generally deemed to be high bridges over stream and rail crossings or in instances where the use of permanent forms might save time on contracts having tight completion schedules.

Twenty-one of the 35 responding states have allowed the use of permanent forms on an alternate basis or they appear to have no definite policy of avoiding their use. At the time of the survey, however, 9 of the 21 states had used the forms on fewer than 5 bridges. Eight states have not permitted permanent steel forms at all, and half of these states cited the fear of future maintenance problems as the reason for avoiding their use. The 4 remaining states simply had not used the forms.

Advantages and Disadvantages of Permanent Steel Forms

The advantages of permanent forms are well recognized and generally are related to the savings of construction time and labor and to the reduced safety hazards that their use affords. Bid prices for bridge deck concrete have sometimes been significantly reduced when steel forms are used. A number of the diadvantages associated with the use of permanent forms, however, were pointed out by the respondents to the survey. Some of the disadvantages, or problems related to the use of the forms have been actual observed situations whereas others are potential problems that construction and maintenance engineers are concerned about. Nonetheless, all these problem areas are logical concerns which should be recognized by all those who have the responsibility of approving the use of steel forming for certain bridge construction projects. Therefore, a summary of those actual or suspected potential problem areas mentioned by the respondents to the survey follows.

1. Future bridge widening and/or reconstruction can be more difficult and costly if permanent steel forms need to be removed prior to construction. As a result, in some isolated instances, the initial advantages of steel forms can be lost in cases where widening and reconstruction are required.
2. When permanent forms are removed for bridge widening the concrete in the fluted area of the form's surface has sheared off in some instances and exposed portions of the lower reinforcing steel in the deck.
3. Safety hazards are increased if permanent forms need to be removed prior to widening an existing bridge deck.
4. The forms are usually attached to the bridge girders by angles welded to the girder flanges. Consequently, the need for field welding is sometimes considered a disadvantage and, once set to fixed elevations, vertical adjustments of the forms prior to concrete placement is difficult.
5. It was reported by one state that, in some instances, the upstanding vertical leg of the form support angles can "perform" continuous

longitudinal cracks in the underside of the deck slab. (It should be noted, however, that this problem can be handled by not allowing the legs of angles to protrude into the deck slab.)

6. As opposed to wooden forms, the use of reinforcing steel tie downs on steel forms can be more difficult if they are to be used.

7. The use of insulation on the underside of steel forms for cold weather concreting is a difficult problem since the insulation cannot be tacked to the forms as it can to the wood forming.

8. One state reported that some rusting of the steel forms occurred during storage on the job site. Insufficient zinc coating on the steel was cited as the cause of the problem. (It should be noted that this is a quality control problem that can be common to all similar type materials.)

9. Where permanent steel forms are used near longitudinal joints and drainage scuppers, some form rusting has been reported after only two to three years' service. The rusting in these areas has been attributed to water and salt solutions filtering through the longitudinal joints and drainage openings to make contact with the metal forms.

10. In locations where moisture does seep through cracks in the concrete deck, it can be directed to the supporting structural steel members and cause corrosion of the top flanges. Slight steel corrosion resulting from this cause was suspected by one state. (It should be noted that moisture can seep through any deck that has previously cracked and/or deteriorated.)

11. Permanent steel forms are believed to cause excess moisture content in concrete bridge decks by preventing or slowing moisture evaporation. Excess moisture content in the concrete has caused bonding problems with the application of waterproof membrane overlays on some bridge decks in one of the responding states.

12. The possibility of form corrosion causes some concern that future painting of the underside of the forms might be required. In a related manner, the possibility of rust staining could have an effect on the overall appearance of a structure. (It should be noted that neither of these two possibilities was reported as having occurred as yet.)

13. The size of structural members may need to be increased to carry the additional weight of the forms and the concrete that occupies the flutes in the forms.

14. Welding of form support angles to the flanges of steel girders is considered to be undesirable -- particularly welding in zones where tensile stresses will occur.

15. Forms of too thin a gage may have been used on some earlier installations and resulted in greater than desirable deflections under the weight of the concrete deck.

16. The underside of concrete decks cannot be visually inspected after initial construction or at later dates for maintenance purposes.

17. Inspection of the concrete deck by "sounding" on the underside of the steel forms by random tapping with a hammer or other object has not always been found to be a reliable indication of soundness. One state reported that more than 30% of the steel panel emitted a hollow sound when tapped but the concrete was sound when the forms were removed for visual inspection. This observation indicates that the forms do not always bond to the concrete as expected.

18. In one unusual case, it was reported that a vehicular fire beneath an overpass structure damaged the permanent steel forms, caused considerable distortion and loss of galvanizing, and generally created a visibly undesirable situation.

A number of these disadvantages and potential problem areas are discussed in more detail later.

Concrete Deck Durability

As stated earlier, one of the primary concerns among potential users of permanent steel forms is the possibility of the forms contributing to the deterioration of concrete bridge decks. Referring to Table 2, only 1 of the 24 states that commented on the effect of permanent form installations could associate concrete deck deterioration with the presence of steel forming. Since bridge deck deterioration problems have been a national concern for a number of years, it is not surprising that visual inspections would not indicate a noticeable difference between bridge decks with or without permanent steel forming. The underside of the decks is the area of concern, however, since moisture and deicing salts could be entrapped at the interface between the concrete and the forms. With the intent of permanent forms being to leave them in place, removal for inspection purposes is a difficult operation. Accordingly, there have been few attempts to remove the forms. Only 3 states indicated that some of the forms had been removed -- 2 of these for the purpose of widening the bridges involved. One state, which widened three 10 to 13 year old structures, removed the forms from the fascia bays of the superstructure and reported no material difference between the appearance of the concrete on these structures as opposed to others without the forms. A second state (which removed some damaged forms from a bridge after a vehicular fire underneath) found no deterioration of the deck. A third state widened 2 bridges (approximately 15 years old) and found shallow surface deterioration on 1 structure and unsound concrete in the curb zone of the other. The unsound concrete was found to extend approximately 2 feet from the curb. Beyond this zone only minor surface deterioration was noted. It was not clear whether the deterioration could be directly attributed to the presence of the steel forming. Nevertheless these conflicting observations tend to suggest that factors such as the relative location of permanent forms with respect to joints, drainage scuppers, etc., may have a significant bearing on the potential for entrapment of moisture and salts between the forms and the deck. Adequate drainage of the deck as well as the initial quality of the concrete would also appear to have a significant bearing on the durability of decks having permanent forms. These last named factors, of course, are important for any bridge deck concrete.

The experience of one state (Table 2) indicated that the presence of permanent steel forms prevented the concrete from drying out and resulted in excess moisture in the decks. The excess moisture was noted as causing some problems when placing bituminous waterproofing overlays on the decks. Tests by Cady,[11] however, do not verify this observation.

Two states routinely use waterproof membranes on the bridge deck surface and, as a result, cannot inspect either side of their decks when permanent forms are used. It should be emphasized, however, that inspection during construction when the concrete is placed is of primary importance.

In general, the response to the inquiry did not clearly indicate that permanent steel forms would or would not reduce the long-term durability of bridge decks. Obviously complicating the problem of evaluating the effect of the forms are several factors. First, durability has been a problem on many bridge decks both with and without permanent steel forms, so it has been virtually impossible to detect significant differences in deck performance to date. Secondly, the difficulty involved in removing the panels has tended to discourage inspection of the underside of the decks. In cases where moisture has been found under the forms during removal, it has been difficult to determine whether it resulted from, caused, or has contributed to a concrete durability problem.

Permanent Form Durability

In addition to the concern about possible concrete durability problems, the possibility of corrosion of the forms themselves is an area of concern. Many of the states that were surveyed had only a few relatively new permanent steel form installations and, consequently, could not evaluate form durability. Only 16 of the 35 responding states commented on form corrosion, with half of these reporting that no corrosion had been observed (Table 3). Three states reported that corrosion had been observed in areas where the forms were adjacent to longitudinal joints, deck drains, and expansion joints. One of the 3 states reported that some corrosion had been observed in the vicinity of scuppers and longitudinal joints after the forms had been in service for only 2 or 3 years. Corrosion at some of the welds which join the form support angles to the bridge girders was observed by 3 states, and 1 state reported some minor rusting at the juncture area between the forms and the girders.

Two states reported rusting on some panels which were removed from several bridge decks. One of the installations was from a 10 year old bridge; the other was removed from two 15 year old bridges which were being widened. In the former instance, 2 panels of the forming were removed for inspection. One panel had some rusting whereas the other did not. In the latter case, the corrosion was observed on the forming in areas adjacent to openings provided for deck drains.

Although the information obtained on form corrosion is limited, it is apparent that permanent steel forms will corrode if exposed to water and detrimental quantities of deicing salts. The greatest potential for corrosion, however, appears to be strongly related to the location of the forms with respect to joints, drainage scuppers or other points where water can gain access to the forms. Judicious use of the forming or bridge design modifications to avoid the placement of the forms adjacent to joints and drains could reduce the possibility of corrosive conditions.

Results of Prior Studies

General Steel Form Surveys

One of the earliest surveys of permanent steel formed bridge decks was reported by Benjamin and Walsh,[1] who inspected 21 steel formed and 10 wood formed bridges in Illinois. The bridges were from 6 months to 5 years old at the time and were located in 3 geographical areas of the state. Three of the bridges had both metal and wood formed areas.

The quality of the concrete in the deck was determined by use of a Swiss hammer and the data were statistically analyzed. By use of this technique the investigators concluded that the quality of the concrete was independent of the method of forming. Additional sounding tests indicated that, in general, the forming had good bond to the deck. Corrosion of the forms was reported on only 1 bridge in an area adjacent to a floor drain. The general condition of the forms was described as good.

Deuterman et al.[2] reported on some of the earlier use of steel for the forming of bridge decks and cited several structures that were built approximately 60 years ago and were still in good condition. The steel used on these very early installations was not designed specifically for use as forming but was an adaptation of material designed for other uses. One such case cited by the authors involved the use of corrugated barn siding on a structure in Texas. Over the years some of these panels have broken loose from the underside of the deck since adequate permanent attachments to the structure were not used.

A number of the early (prior to 1966) installations of permanent forms that had had failures were jointly investigated by the Bureau of Public Roads and the American Iron and Steel Institute.[2] Most of the problems found were of the type that could be avoided by proper deck placement techniques and by certain modifications in the design of the structure, the forms, and in the specifications. Rusting of the forms was found beneath longitudinal construction joints and longitudinal median joints. Calcium chloride, used as a retarder in the concrete, was suspected as contributing to the rusting of the forms on 1 bridge deck. Bending and distortion in the forms on one structure were attributed to dropping the concrete from excessive height during construction and to installing certain steel panels upside down. Other problems were found to be related to poor consolidation of the concrete, a too large aggregate and poor attachment of the forms to the girders. The results of the investigation led to the development of specifications for the use of steel forms.[2] Many of these provisions appear to be incorporated in the specifications now employed by many state highway and transportation departments.

Composite Action with Concrete Decks

The amount of composite action that can be expected from the use of permanent corrugated steel forms was investigated by Barnoff et al in a study of 2 parallel continuous girder bridged located in Pennsylvania.[3] One of the twin structures was constructed using permanent steel forms and the other utilized conventional wood forms that were removed after construction. The bridge without the permanent forms had significantly more transverse cracking than the one with the permanent forms. Other tests and analyses indicated that the steel forms tend to act as shear connectors and develop considerable composite action between the concrete deck and steel girders. Composite action was found to be greater when the slab and beam were subjected to positive moment than to negative moment stresses.

In further studies for composite action, Barnoff and Jones[4] conducted push-out tests on specimens having steel forms supported by several types of beam to form connections. The general type connection most widely used by contractors (Figure 2) for attaching the steel forms to bridge beams was found to produce a high percentage of composite action. Although full-scale tests were not conducted, the tests' results indicated that the number of shear connectors used on a girder theoretically could be substantially reduced when using steel forms. It should be noted that this finding was only meant to indicate the degree of composite action obtained from corrugated forms. It was not recommended that the number of shear connectors be reduced for design purposes without full-scale study.

Deck Durability with Permanent Steel Forms

In a study of 249 4-year old bridge decks in Pennsylvania, Cady et al[5, 6] related the extent and severity of bridge deck deterioration to those factors or combination of factors which could cause deterioration. The lengths of cracks, areas of spalls, fracture planes, and mortar deterioration were measured on each deck and the data analyzed using a computerized statistical technique which related the interaction of causes with the deterioration noted on the bridge decks. It was generally found that variations in construction practices was a major cause of the differences in deck deterioration observed. In addition, the analysis indicated that the use of permanent steel forms was related to reduced deck cracking and could also be associated with slight increases in surface mortar deterioration on the decks. Many decks with steel forming and high traffic counts, however, had little surface mortar deterioration, which suggested that construction practices and concrete quality are, as always, important factors and may assumed added significance when permanent forms are used.

Spalling of the concrete decks was not found to be related to permanent forming. Fifty-three of 55 decks having spalling were found to have insufficient cover over the reinforcing steel.

The deterioration of 7 concrete bridge decks was observed by Cady et al.[5] for periods of at least 5 years subsequent to construction. Three of these decks were built with conventional removable forming, whereas 4 were built with permanent steel forms. The decks built with removable forms exhibited higher rates of cracking but lower rates of surface mortar deterioration than the decks built with permanent steel forms. The average rate of cracking on the bridge decks is shown in Figure 3, which indicates that after 5 or 6 years of service, the conventionally formed decks exhibited roughly 3 times more cracking than did the permanently formed decks. The authors suggested that the reduced cracking on the latter decks could be attributed to two factors. First, as indicated earlier in the work conducted by Barnoff et al.[3] permanent forms produce decks that are stiffer than conventionally formed decks and thus their use might reduce cracking precipitated by dynamic loads. Secondly, the steel forms would aid moisture retention for longer periods of time and thus reduce shrinkage cracking resulting from early moisture loss in the newly placed concrete.

Figure 4 shows the relationship between the form type used and the mean rates of surface mortar deterioration (SMD) found on the bridge decks. As indicated in Figure 4, the SMD on the permanently formed decks after 1 year was roughly equal to that developed after 3 years on the conventionally formed decks. The trend of the conventional form curve, however, suggests that the SMD may only be lagging by several years the deterioration noted on the permanently formed decks.

In a later report,[11] Cady cautions that permanent forms cannot be assumed to be the primary cause of SMD, but in combination with the contractor involved and calcium chloride usage they were found to have influence on the problem. It was suggested that the cause of the greater SMD on the permanently formed decks might be due to longer moisture retention, causing the decks to be more susceptible to frost action and perhaps to traffic wear. Further studies by Cady and Carrier[7] involved the measurement of moisture content and distribution in the concrete of 2 bridge decks in Pennsylvania. One was constructed with removable forms and the other with permanent forms. Measurements were averaged over an 11-month period and the moisture distribution compared with that of a concrete pavement. As shown in Figure 5, the permanent steel formed deck and the concrete pavement had similar moisture distribution characteristics, with the higher moisture content at the center and bottom of the sample cores. The exposed surfaces (top and bottom) of the conventionally formed bridge deck sample were drier than those for the pavement and permanently formed deck. Thus, it was concluded that bridge decks built with permanent metal forms have generally higher moisture contents than decks built with removable forms. It should be noted, however, that this result might have varying significance in different geographic regions having climatic conditions different from those of Pennsylvania.

The results of studies conducted by Chamberlin et al.[8] involving 716 bridges in New York State were generally similar to those from the work conducted in Pennsylvania. Scaling and spalling were found to be roughly equivalent on bridges formed by either procedure in New York, whereas spalling was less extensive on steel formed decks in Pennsylvania.[11] Transverse cracking, however, was found to be considerably less on permanently formed decks in both states. The deck cracking results of the New York and Pennsylvania studies were compared by Chamberlin et al.[9] and found to be quite similar, as shown in Table 4. The relationship between surface mortar deterioration and permanent steel forms was not found in the New York study.

Additional work in New York, reported by Allison,[10] involved removing several permanent form panels from some 10 to 12 year old bridge decks. All of these decks were covered with an asphalt wearing surface and were believed to represent the worst permanently formed decks available. In a few instances some deteriorated concrete was found on the underside of the deck but no determination of the basic cause or severity of the distress was made. In all cases the concrete was found to be damp when the panels were removed but no rusting of the steel was noted.

Freeze-Thaw Durability

Since permanently formed decks were generally found to have higher moisture contents than conventionally formed ones, Cady investigated the possibility that

Figure 1. Permanent steel forms being installed on a steel girder bridge.

Table 1. State policies concerning use of permanent steel forms (as of 1974).

Total states surveyed	States responding	States permitting as an alternate	States that have permitted on some contracts	States permitting only in special situations	States that have not permitted
38	35	8	13	6	8

Table 2. Effect of permanent steel forms on deck durability.

States commenting on deck durability	States indicating no unusual deck deterioration	States indicating deck problems related to the forms	States routinely employing deck overlays	States that have removed some forms from older decks
24	21	1	2	3

Figure 2. Typical form support and sheet metal screw connection, used in push-out tests conducted by Barnott and Jones (4).

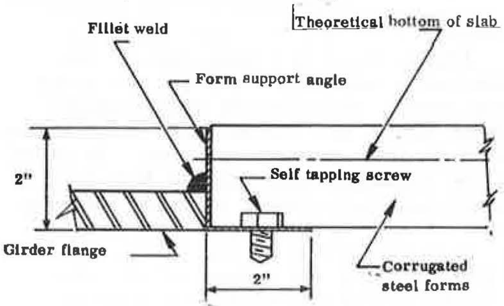

Table 3. Corrosion of permanent steel forms.

States commenting on form corrosion	States reporting no corrosion	States reporting corrosion at deck joints, drains, etc.	States noting corrosion at welds attaching form supports	States noting corrosion at juncture of forms and girders	States noting corrosion on forms removed
16	8	3	3	1	2

Figure 3. Relationship of rate of deck cracking to type of deck forming used (5).

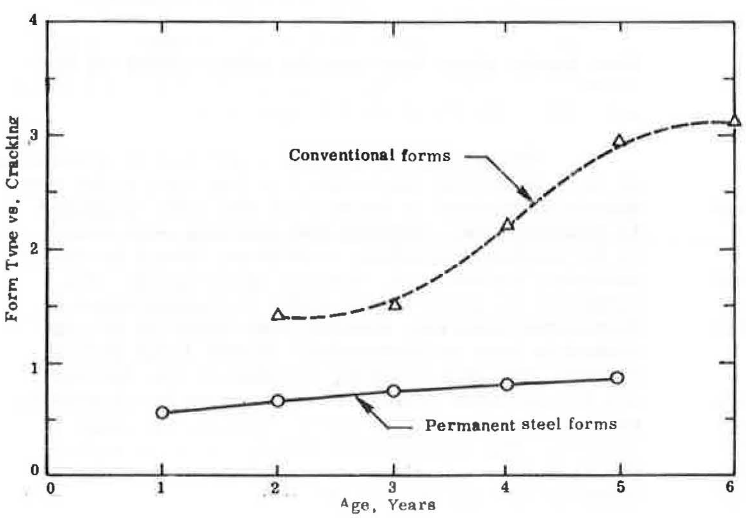

Figure 5. Moisture distribution in a pavement and in bridge decks (7).

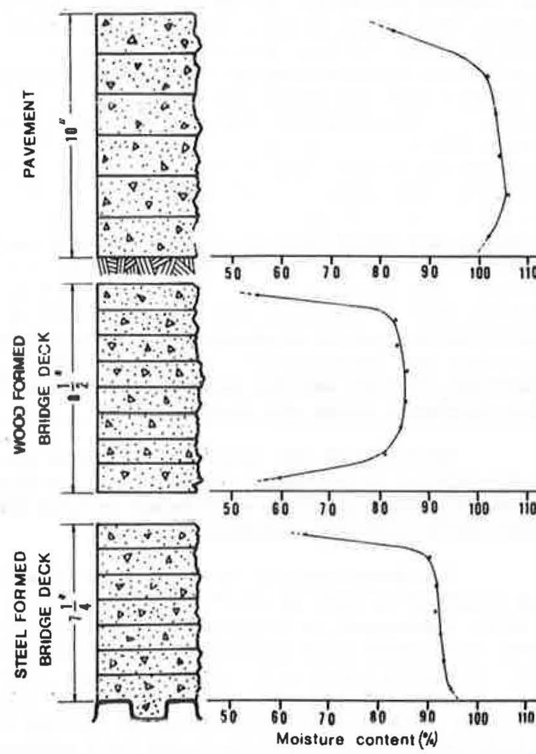

Figure 4. Relationship of rate of deck SMD to type of deck forming used (5).

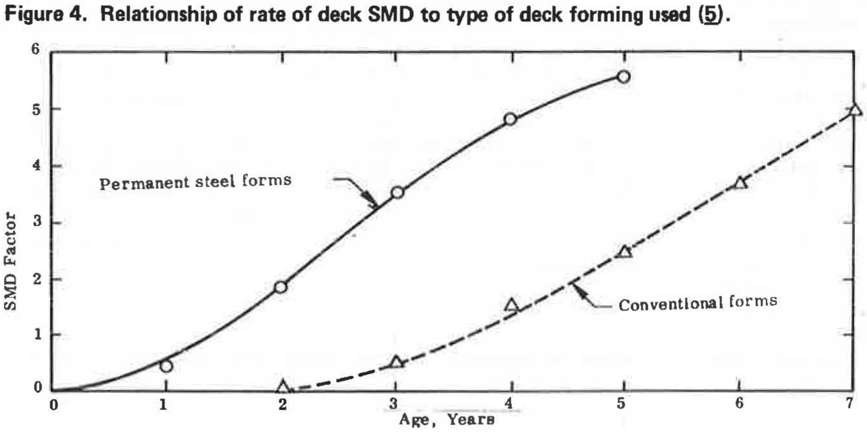

Table 4. Linear feet of transverse cracks per 100 ft^2 of bridge deck (9).

Source of Data	Forming Method SIP	Conventional
Pennsylvania	0.46	1.50
New York	0.55	1.27

these decks would be more susceptible to freeze-thaw damage.[11] Both laboratory and field exposure tests were conducted on test slabs with and without permanent forms and with and without several types of surface overlay treatments. In the laboratory, the slabs were maintained in a saturated condition and subjected to 75 freeze-thaw cycles. Subsequently, the slabs were exposed to one winter of natural conditions. Pulse velocity tests and measurements of expansion during freezing (dilation) indicated no evidence of freeze-thaw damage.

Additional inspections and tests were conducted on 26 bridges -- 13 each in New York State and on the New Jersey Turnpike. One of the New York bridges did not have permanent forms, but all were 11 to 13 years old at the time of the survey. The New Jersey structures ranged in age from 1 to 11 years old. With the one exception noted, all the decks had permanent forms and all were sealed with some type of waterproofing material. These decks were chosen to investigate the possibility of increased freeze-thaw susceptibility when the concrete is sealed on the top and bottom. Pulse velocity tests revealed that 2 of the New York decks probably contained deteriorated concrete. One of these was the deck without permanent forms. Tests on the remaining bridges did not indicate freeze-thaw damage.

Form panels from 12 bridges and the concrete on the underside of the decks was found to be in sound condition. The results of these inspections are summarized in Table 5.

Thus, the results suggest that the freeze-thaw durability of covered concrete decks with permanent forms is not different from those without the forms.

Testing for Incomplete Consolidation

In support of the state survey discussed earlier, Cady et al.[11] found that hammer soundings on the underside of steel formed decks did not necessarily relate to incomplete consolidation of the concrete. In addition to locating areas of incomplete consolidation, hollow soundings were found to indicate separation of the forms from the decks. The extent of hollow soundings was also found to be related to deck age and thus to progressive separation of the forms from the concrete. The authors concluded, however, that separation of the forms from the concrete has no effect on either the durability of the deck or of the forms.

Interpretation

An interpretation of the studies and field inspections that have been reported to date do not implicate permanent forms as singularly causing deck durability problems. The preponderance of the evidence suggests that given deck design details which will minimize the possibility of deck drainage gaining contact with the forms, plus the construction of an initially durable concrete, the forms should not have a detrimental effect on deck durability. In some respects -- such as the case with cracking and spalling -- the forms appear to aid deck durability. While surface mortar deterioration was found to correlate with the use of permanent forms in 1 state, the correlation was only valid when the use of the forms was taken in combination with several other variables.

Corrosion Resistance of Permanent Steel Forms

General visual inspections of the in-place forms during the New York study[8] and the earlier Pennsylvania studies[6] revealed little or no evidence of form corrosion. Removing some panels from what was described as the worst permanently formed decks available in New York, however, Allison found some corrosion. On several panels, approximately 5% of the total area was described as completely rusted through. These forms had been installed for 10 to 12 years prior to their removal but no indication of the possible causes of the distress was given.

In the latest study by Cady et al.[11] form panels were removed from 5 bridges in New York State and 6 on the New Jersey Turnpike. The New York bridges were of the same age group as those involved in Allison's work, and all the decks, including those in New Jersey, were covered with some type of waterproof membrane and/or asphalt. In general, corrosion on the inside of the removed panels was described as virtually nonexistent. Two panels had a few spots of light rusting and 1 some slight corrosion where wood and sawdust (see Table 5) had been left on the form prior to concrete placement.

General visual inspection of the forms indicated that they were in good condition. Most of the corrosion that was observed was located along the fascia girders and at the ends of spans and could be traced to the drainage of salts from the deck surface. Drop inlets on bridge decks were particularly related to adjacent form corrosion. Therefore, deck design features which permitted water to come in contact with the forms were related to the majority of the form corrosion. A summary of the observations on form corrosion is given in Table 6. As indicated, extensive corrosion was concentrated at span ends and along the fascia girders with very little being found in other areas of the decks. The authors concluded from their inspections that corrosion is generally not a problem where deck design provides for drainage which will not make contact with the forms.

Time to Corrosion

Field studies have indicated that galvanized steel forming will corrode under adverse conditions. Under exposure to water and salt solutions, for example, form rusting has been observed after roughly 8 to 12 years of service. On the other hand, most (or all) of the forming is protected from some of the normal atmospheric weathering conditions such as precipitation and direct sunlight. Thus, it is difficult to predict how long the forming might survive corrosion at any given location. Data are available,[12, 13] however, concerning the time to various stages of corrosion of galvanized test sheets exposed to atmospheric conditions at a rural setting in Pennsylvania. These particular samples, representing various manufacturers, have been continuously exposed since 1926, and data are still being collected. Although the test samples are exposed to the weather whereas permanent forms normally are not, these data do afford a basis for judgment. Some of the corrosion data, showing the years to first rusting, 100% rusting, and first perforation for selected weights of zinc coating, are given in Figure 6. All of the data shown are for NO. 22 gage material, which is usually the lightest gage sheeting used for forming. The curve showing first rusting represents the average of the data for 18 samples. The 100% rusting curve is an average of all data available up to the 1.5 oz./ft.^{2} (.46 kg/m^2) coating. The remainder

Table 5. Condition of concrete where forms were removed (11).

Bridge	Excellent	Incomplete Consolidation	Deteriorated	Foreign Materials Found Under Forms
NYDOT-2	X			Sawdust Wood Chips
NYDOT-5	X			
NYDOT-7	X			
NYDOT-10	X			
NYDOT-13	X			Sand
NJTP-2	X			
NJTP-3	X			
NJTP-5	X			
NJTP-6	X	X		
NJTP-7	X			
NJTP-8	X			
NJTP-10*	X			
TOTALS	12	1	0	

*Removed by Others

Figure 6. Average time to corrosion of 26 by 30-in. No. 22 gage corrugated galvanized sheets exposed to atmospheric corrosion at State College, Pa., in 1926. (Data developed from references 12 and 13).

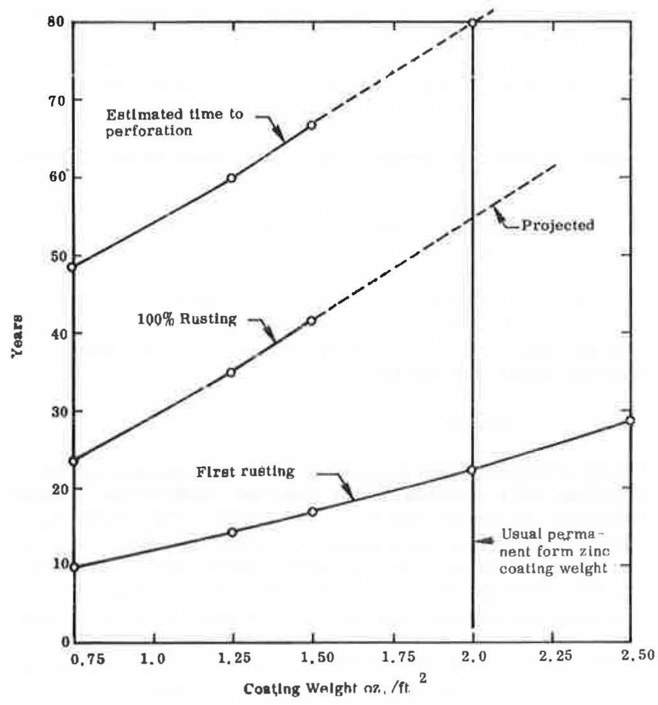

Figure 7. Effect of environmental conditions on the atmospheric corrosion of 28 by 36-in. corrugated galvanized sheets exposed at different locations. (Data developed from references 12 and 13).

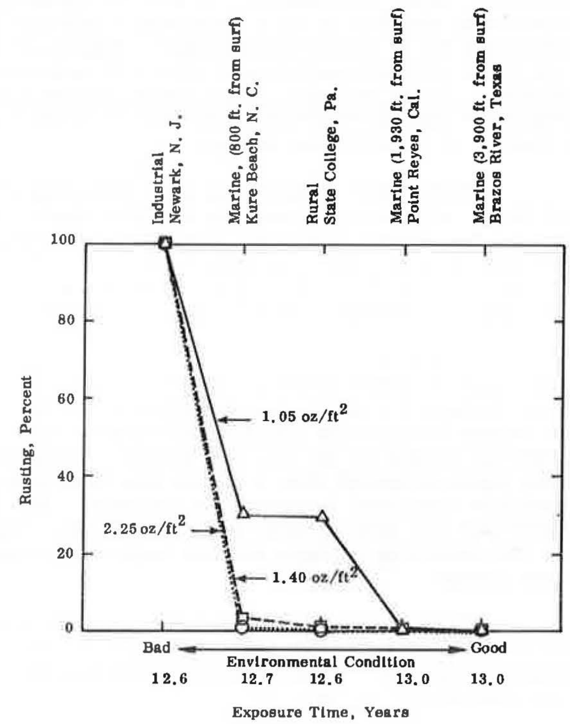

Table 6. Summary of observation of form corrosion (11).

Bridge	Age Yrs	Form Corrosion								
		Along Fascia Girders			Span Ends (Expan. Joints)			Random		
		Extensive	Slight	Negligible	Extensive	Slight	Negligible	Extensive	Slight	Negligible
NYDOT-1	12	X			X					X
-2	12	X			X					X
-3	12	X			X				X	
-4	12	X			X			X		
-5	11			X			X			X
-6	11			X	X					X
-7	12			X	X					X
-8	12			X		X				X
-10	13			X			X			X
-11	13			X	X					X
-12	12			X	X					X
-13	12			X		X				X
NJTP -1	9			X		X			X	
-2	9			X			X			X
-3	9	---Not Applicable*---					X			X
-4	9	---Not Applicable*---			---Not Applicable*---				X	
-5	9	---Not Applicable*---					X			X
-6	8			X		X			X	
-7	11		X				X			X
-8	2			X			X			X
-9	1			X			X			X
-10	1			X			X			X
-11	1			X			X			X
-12	1			X			X			X
-13	1			X			X			X
TOTALS		4	1	17	8	4	12	1	4	20

*Sections of the deck having steel forms were not adjacent to fascia girders and/or abutments.

of the curve is projected from the established trend. The time to perforation curve was conservatively estimated by adding to the former curve the lower of the time to perforation data for the 22 gage uncoated metal samples.

For a zinc coating weight of 2.0 oz./ft.^{2} (0.61 kg/m^2) which is usually used for forming, the average first rusting, 100% rusting, and first perforation would be, respectively, 22.5, 55, and 80 years. Therefore, if a 22 gage forming were to corrode at a rate comparable to that experienced under atmospheric conditions at a rural setting in Pennsylvania, it would survive an average of 80 years before a perforation would occur. Assuming protection from water and deicing salts, the life of a galvanized forming would generally appear to be as long as that normally experienced with bridge decks. One might expect, however, that a deteriorating bridge deck could precipitate deterioration of the forming.

The geographic location of a forming installation could also have a bearing on the corrosion factor, as indicated by data developed from other tests.[13] Samples exposed in 1960 to atmospheric corrosion at 5 sites demonstrate that heavy industrial and salt water environments can be severe. As shown in Figure 7, all test samples exposed at an industrial site at Newark, N. J., showed 100% rust after 12.6 years. On the other hand, Cady et al.[11] found little corrosion of the forms on a New Jersey turnpike bridge after 8 years exposure adjacent to an oil refinery. It is likely that the presence of SO_2 and SO_3 in the air in some industrial regions increase the corrosion rate due to the acidity of condensed moisture.[14] It is interesting to note that the rusting of forming reported at joints and drainage areas on some bridges appears to be on a time scale similar to that of the corrosion observed on the industrial area test samples.

The next most severe of the 5 sites was a marine environment located 800 ft. (244 m) from the surf. At 2 other marine sites located, respectively, at 1,930 ft. (588 m) and 3,900 ft. (1189 m) from the surf, zinc corrosion products were noted but no bare metal rusting was found. Although it has been suggested that protective films of zinc carbonate are sometimes formed in marine locations, and help retard corrosion,[14] the data suggest avoiding areas close to salt water.

One might conclude from the data that even on the underside of bridge decks, galvanized forming might corrode faster in industrial and at, or near, salt water areas. In rural areas the principal corroding agent appears to be water, so galvanized panels should corrode slowly in areas with dry climates. Regardless of the location, the corrosion rate of galvanized sheeting will vary from the averages given in Figures 6 and 7. This is due mainly to the fact that zinc coatings are never uniformly distributed, and base steel at the edges of some panels could accelerate corrosion.[14]

Chloride Penetration

Chloride penetration to the depth of the forming has also been a matter of concern in some quarters. Studies on 3 bridges,[11] however, indicated that after 7 years' service the chloride content was negligible below 1.5 in. (38 mm) from the deck surface. Data developed by Clear[15] indicates that chloride contents at 3 in. (76 mm) depth are usually below the corrosion threshold for concrete having water-cement ratios of 0.5. Since chlorides would have to travel through on the order of 6 in. of concrete to reach the lower reinforcing steel and usually 8 in. (203 mm) or more to reach the forming, corrosion of the lower steel and the forming as a result of chloride penetration appears unlikely. Joints, drainage areas, and deep cracks or deteriorated areas of concrete would appear to be the most likely channels for penetration.

Interpretation

There is little doubt that permanent forming will corrode in only a few years if exposed to water and, particularly, salt solutions. With the use of adequate joint and drainage designs and forming installations designed to minimize possible contact with corrosive agents, however, permanent form durability should equal that of the bridge deck.

Summary of Conclusions

1. The state survey revealed a wide difference of opinion concerning the use of permanent forms. Some states do not allow the use of forms, fearing future maintenance problems, whereas others feel their advantages outweigh their disadvantages.

2. A composite view of the state survey and the prior research studies indicate that permanent steel forms do not singularly affect the durability of concrete bridge decks. Removal of a number of forming panels in a recent study suggests very few instances where moisture has been entrapped between the deck and the forming, except in instances where the moisture could be traced back to joints and drainage areas on the deck.

3. Evidence suggests that the durability of the concrete deck may have a more significant effect on the durability of the forms than the forms will have on the durability of the deck.

4. Permanent steel formed decks generally have less transverse cracking and less spalling, and improve the composite action between the deck and the girders.

5. Permanent formed concrete decks have higher moisture contents and, in combination with other factors, have been found in one state to have generally more surface mortar deterioration than conventionally formed decks.

6. Hammer soundings taken on the forming to check for incomplete consolidation of the concrete are not reliable. "Hollow" sounds often reveal a lack of bond between the deck and the forming rather than poor consolidation of the concrete.

7. Corrosion of the permanent forms can be a significant problem if bridge and forming installation designs allow ready access to the forms through joints or drainage features. Therefore, permanent forms should not be used on those areas on the underside of bridge decks that are readily accessible to drainage moisture.

8. Data on the atmospheric corrosion of galvanized sheeting suggest that given adequate protection as described above, the forming should have a life expectancy equal to or greater than that normally experienced with bridge decks.

9. Data on the atmospheric corrosion of galvanized sheeting exposed close to salt water (marine) environments and in heavy industrial areas indicates the possiblity of a higher than normal rate of corrosion.

10. Studies indicate that penetration of chlorides through the concrete deck to gain contact with permanent forming is unlikely. The most likely access channels for chlorides to gain contact with the forming would be through joints, drainage features such as drop inlets, deep cracking or deteriorated deck concrete.

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