FINAL REPORT

CONSTRUCTION OF PRESTRESSED CONCRETE SINGLE-TEE BRIDGE SUPERSTRUCTURES

by

Michael M. Sprinkel
Research Engineer

Virginia Highway and Transportation Research Council
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SUMMARY

This report discusses in detail the construction of the first five precast, prestressed concrete, single-tee beam bridge superstructures to be let to contract in Virginia. The data suggest that this single-tee beam enables efficient construction of the superstructures of bridges in the short-span range, because the contractor can move from the bridge seat stage of construction to the forms-in-place stage by erecting tee beams mass produced at a fabricating plant. Using a 45-ton (4×10^4 kg) crane, several men can erect, connect, and overlay the tee beams at the rate of about one 42 ft. x 44 ft. (12.6 m x 13.2 m) span per week. With the addition of precast parapets, superstructure form work at the bridge site is almost eliminated. On-site construction time for the single-tee superstructure is controlled primarily by the time required for the site-cast concrete used in the diaphragms, overlays, and backwalls to attain the design strength. Single-tee bridge construction should be continued in a manner consistent with the conclusions of this report. Some additional research is recommended.

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INTRODUCTION

Precast, prestressed concrete single-tee bridge construction has been implemented in Virginia after some 6 years of joint efforts by many individuals, organizations, and committees. The research advisory committee for industrialized construction, a committee made up of individuals from the Department of Highways & Transportation, the Virginia Highway & Transportation Research Council, and private industry, sponsored the basic concepts and recommended action by the Virginia Department of Highways & Transportation. Through the coordination of the Portland Cement Association, the Virginia Prestress Concrete Association cooperated with bridge contractors in Virginia to provide the Department with designs of bridge systems, such as the single-tee, which could be competitively fabricated and erected in Virginia.(1) The Bridge Division of the Department provided input to the systems design concept and supplied the final design details for the structures built to date.

The prestressed, precast concrete tee beam provides for efficient bridge construction for numerous reasons. The shape of the member, shown in Figure 1, is particularly suitable for use in systems construction in that by maintaining a constant stem width of 1 foot (0.3 m) and a flange width of about 4 feet (1.2 m), only the bottom pallet or sides and end bulkheads of the casting form have to be adjusted to provide economical beams for spans between 30 feet (9 m) and 66 feet (19.8 m) long. The end bulkheads can also be skewed in the forms to accommodate most bridge configurations. Since the flange of the tee serves as the lower half of the bridge deck, in one step the contractor can advance from the bridge seat stage of construction to the forms-in-place stage in a few hours. The need to remove major deck forms is eliminated. Because of the reduced volume of cast-in-place concrete required, more extensive use of high quality concrete mixes which enhance durability may be reasonably justified for the 4-inch (10 cm) composite overlay.

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DESIGN DEPTHS

SPAN RANGE	DEPTH d
30'-35'	2'-0"
36'-40'	2'-2"
41'-43'	2'-4"
44'-46'	2'-6"
47'-50'	2'-8"
51'-53'	2'-10"
54'-56'	3'-0"
57'-60'	3'-2"
61'-63'	3'-4"
64'-66'	3'-6"

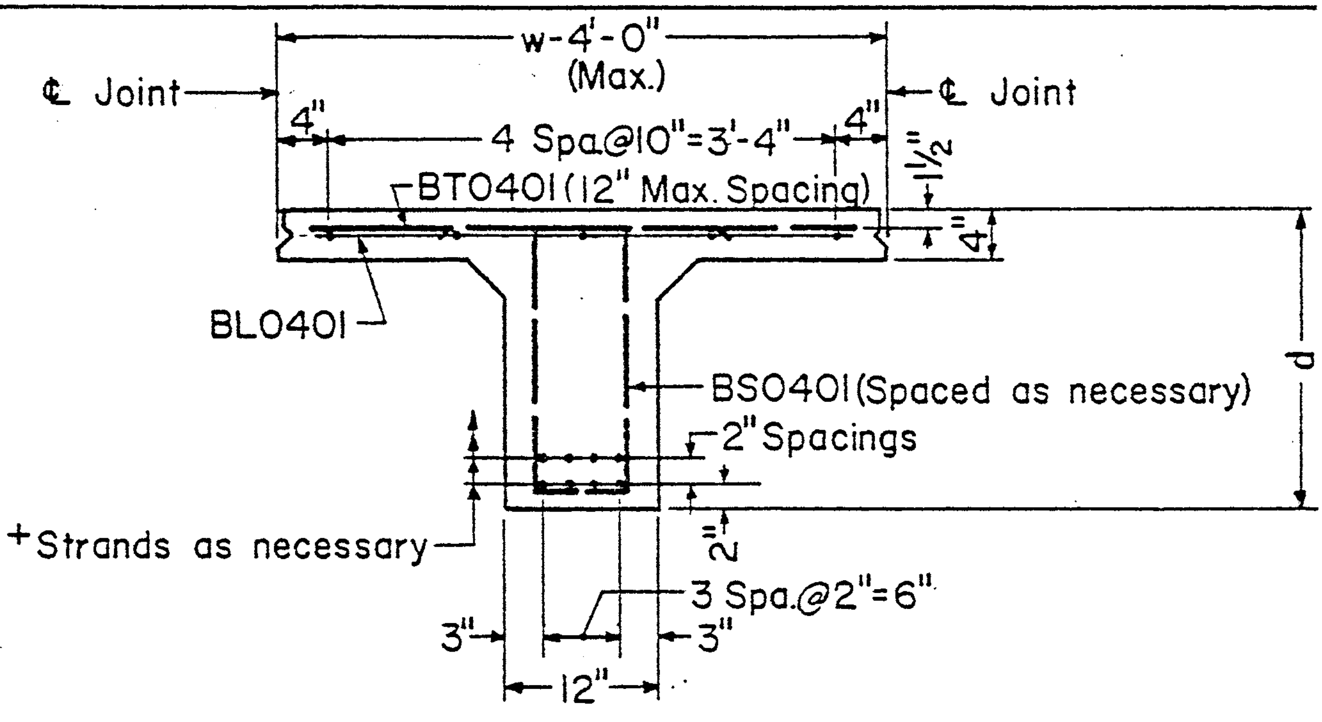
CRITERIA FOR USING PRESTRESSED SINGLE T-BEAMS:

1) Geometric Criteria
   - Span lengths: 30' to 66'
   - Skew angle: 0° to 45°
   - Tangent Alignment
   - Any desired roadway width
2) Use 4" (Min.) CIP Conc. slab overlay on tees. (Place longitudinal steel under transverse steel to provide 2½" [Min.] clear cover from top of slab.)
3) Abut. and Pier seats should be nearly parallel to Bridge Deck.

Tee depths were based on 4'-0" flange width (effective width used was 3'-10").

Design moments used were based on a reduction of span length by 1'-6".

Reduction of ¼" in deck slab thickness was used in computing composite section properties.



INTERIOR

+Design Tees for ½"φ Grade 270 Str'ds. with $\frac{7}{16}$"φ Str'ds. as a substitute.

Figure 1. Systems tee-beam shape. (From Reference 2). (1 ft. = 30 cm)

In this report the prestressed concrete single-tee bridge design is evaluated based on observations of the fabrication and construction of the first five single-tee bridges to be let to contract in Virginia.(3) The report also discusses the fabrication and installation of precast parapets which were specified on each of the single-tee bridges. In addition, the report discusses the use of a superplasticizing admixture in the concrete specified for the 4-inch (10 cm) composite overlay on two of the bridges. Data for the report were collected in the concrete and petrography laboratories at the Research Council, in the casting yard at Phoenix Concrete Products, and at the bridge sites in the city of Norton, Dickenson County, and Floyd County. The data were gathered after giving consideration to the suitability of a particular span or structure and availability of manpower and equipment.

ON-SITE CONSTRUCTION TIME

A four-span, precast, prestressed single-tee concrete bridge structure carrying Walnut Street over the McClure River in Dickenson County was opened to traffic on October 6, 1976, less than 9 weeks after construction on the superstructure commenced. Working with a bridge crew of two to four men, the Crowder Construction Company successfully erected, connected, and overlaid the tee beams in the four spans in roughly 4 weeks (see Figure 2). A 45-ton (4×10^4 kg) truck crane and four men erected the eight 35-foot (10.5 m) tee beams for each span in several hours. Another 3 days per span were required to form and cast the diaphragms (see Figure 3) and to grout the keyways between the tees (see Figure 4). The side forms were prepared, the reinforcing steel positioned, and the concrete placed to provide a 4-inch (10 cm) overlay in another day and a half. While the concrete in the overlays was gaining strength, the crew formed and placed the backwalls and the terminal walls, erected and connected the precast parapets (see Figure 5), and back filled and graded the approach lanes. Construction sequence data for the Dickenson County bridge are shown in Figure 6.

Using equipment and manpower similar to that used on the Dickenson County bridge, Edwin O'dell Construction Company opened to traffic a three-span single-tee bridge structure carrying Rte. 639 over the Little River in Floyd County on November 26, 1976, also within 9 weeks after the first tee beam was placed (see Figure 7). With the exception of a 2-week delay caused by a crane failure, the Floyd bridge was constructed at about the same pace as the Dickenson bridge.

It is believed that the Dickenson and Floyd County bridges are representative of the amount of on-site construction time required to construct a single-tee bridge superstructure as currently designed. The Dickenson and Floyd superstructures were constructed at the rate of about 500 ft.^{2} (46.5 m^2) and 450 ft.^{2} (41.8 m^2) per week, respecitvely. The rate should be somewhat greater for a bridge with longer spans, more than four spans, or with high early strength concrete in the overlay.

The amount of site time required to construct a concrete bridge superstructure is the cumulative sum of the times required to prepare the formwork, place the steel and concrete, strip the form work, and obtain design concrete cylinder strengths. None of these four activities can be completely eliminated unless all site-cast concrete is completely eliminated. The precast, prestressed single-tee design reduces site time by eliminating most of the forming and form removal usually required for conventional site-cast bridge decks and beams. But when the diaphragms are formed and cast at the site, an overlay cannot be placed until the diaphragm concrete has attained 75% of its design strength, which usually takes 3 to 7 days. Additional spans cannot be overlaid until the concrete in the adjacent overlays has reached 50% of its design strength, which takes from 2 to 3 days. If the backwall is used to support the screed, it also must have attained 50% of its design strength. A bridge cannot be opened to traffic until all the concrete in the superstructure has attained its 28-day design strength.

In Table 1 the construction sequence data obtained from the Dickenson County bridge are applied to a hypothetical one-span tee beam bridge to determine the site time required before it can be opened to traffic. Based on the Dickenson County data, it is apparent that a one-span tee beam superstructure could be constructed in 27 to 48 days. The opening of the structure to traffic would be controlled by the cylinder strength of the backwall or of the overlay. Mix designs providing high early strength could be used in the diaphragms, backwalls, and overlays to reduce the delay in opening the structure to traffic. But regardless of how well the construction operations are organized, a one-span single-tee bridge as currently designed and constructed and incorporating conventional concrete mix designs cannot be opened to traffic sooner than about 1 month after the first tee beam is placed. The current tee beam design reduces site labor considerably, but reduces site time only marginally when compared with more conventional types of construction.

Site time for a one-span single-tee bridge could be reduced to less than 1 month only if more precast concrete or high early strength concrete is used in the structure. For example, precast concrete or steel diaphragms would reduce labor time by 3 days and strength development time by 3 to 7 days and total site time by 6 to 10 days. Forming for the overlay could begin the same day that the tees are placed and the diaphragms connected. Precast backwalls would allow the contractor to screed off the backwall and to begin grading operations immediately after the backwall is positioned. Conceivably all the precast pieces could be placed and connected in 1 day, the overlay could be placed in another day, and grading operations could be completed on a third day. However, until the overlay develops 85% of its design strength the parapets cannot be placed, and until the overlay develops 100% of its design strength the bridge cannot be opened to traffic. By precasting all the superstructure components except the overlay the contractor could complete the structure in about 5 work days, but the structure could not be opened to traffic for at least 14 to 28 days. Design strengths were obtained for the superplasticized overlays used in Norton in about 1 week. Only when the single-tee flange is designed to provide the full thickness can one hope to open a single-tee superstructure to traffic after 1 work day. A single-tee bridge requiring no site-cast concrete may be obtainable in the foreseeable future, but it will likely require post-tensioning and a bituminous overlay.

Figure 2. Dickenson County bridge prior to placement of 4-inch (10 cm) concrete overlay on the first of four spans.

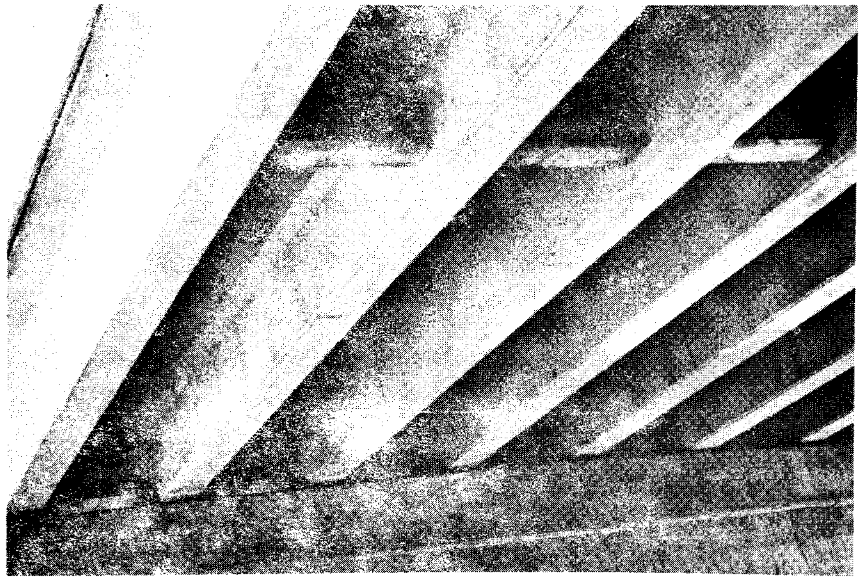


Figure 3. Site-cast concrete diaphragms between the precast tee beams.

Figure 4. Nonshrinking cement paste placed between adjacent tee beams.

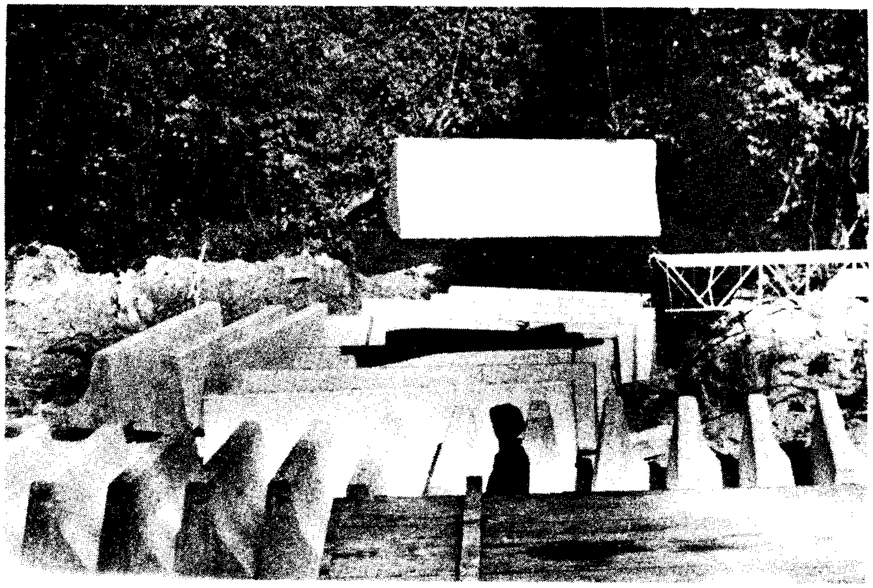


Figure 5. Precast parapet sections are unloaded at the bridge site.

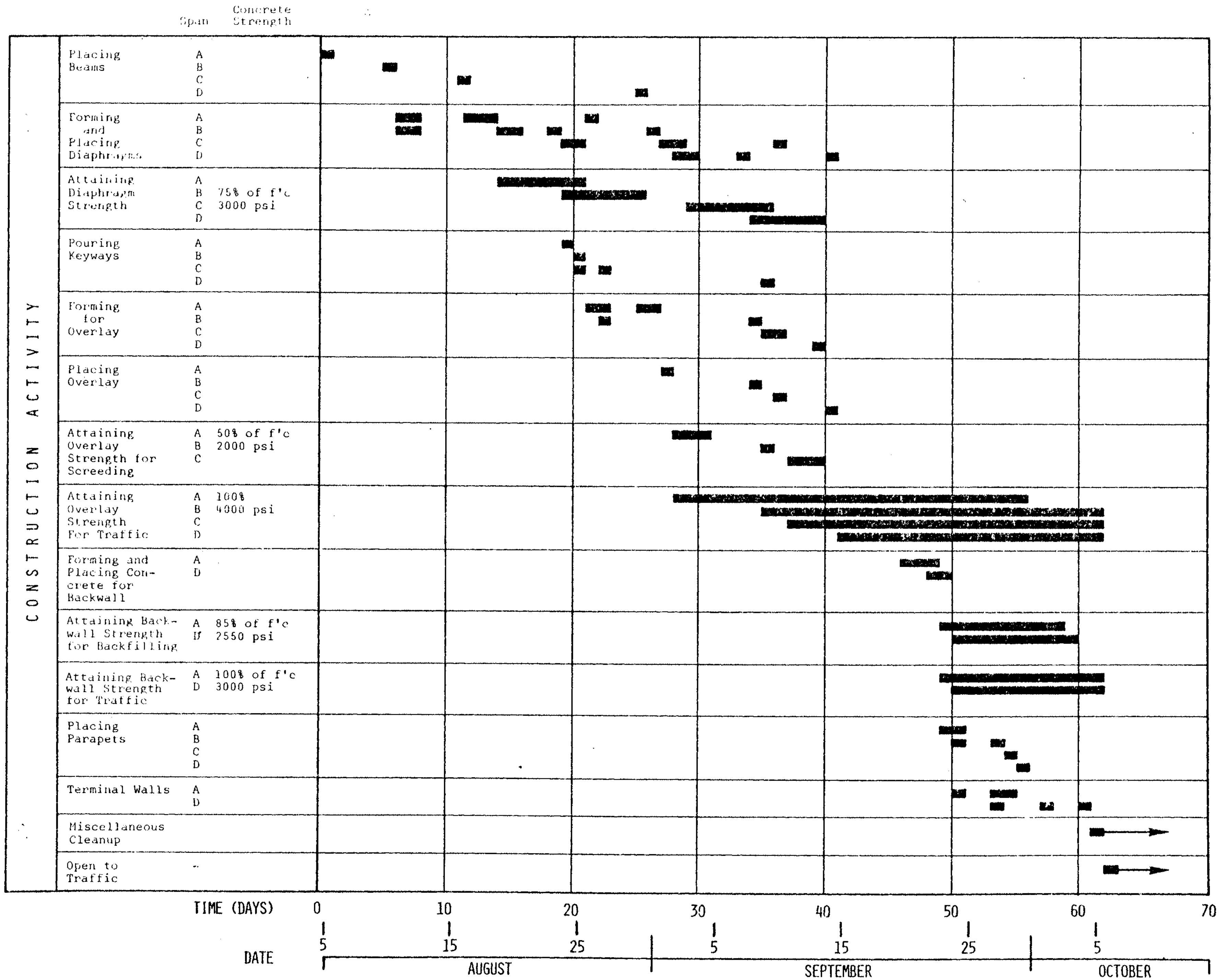


Figure 6. Construction sequence data for Dickenson County bridge. (1 psi = 6.894 Pa)

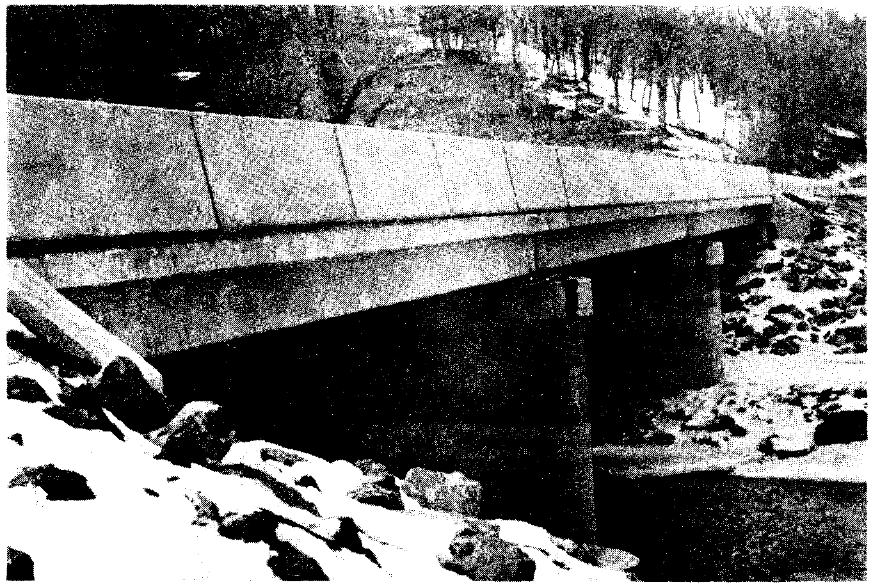


Figure 7. The Floyd County single-tee bridge.

Table 1

Cumulative Construction Time for a One-Span Single-Tee Bridge

	Activity	Time* (Days)	Cumulative Time (Days)
1.	Place tee beams	0.5	0.5
2.	Form and place diaphragm concrete and grout keyways	3	3 - 5
3.	Develop diaphragm strength (.75 f'c) before placing overlay	3 - 7	6.5 - 10.5
4.	Form for overlay	1	4.5
5.	Place overlay	0.5	7 - 11
6.	Develop overlay strength (.60 f'c) before removing forms	3 - 6	10 - 17
7.	Develop overlay strength (0.85 f'c) before placing parapets (To place crane on deck need f'c)	7 - 14	14 - 25
8.	Develop overlay strength (f'c) before opening to traffic	14 - 28	21 - 39
9.	Forming and placing concrete in backwalls	3	13 - 20
10.	Develop backwall strength (.85 f'c) before back filling	7 - 14	20 - 34
11.	Develop backwall strengh (f'c) before opening to traffic	14 - 28	27 - 48
12.	Place and connect precast parapets	3	17 - 28
13.	Forming and placing concrete for terminal walls	4	21 - 32
14.	Grade approach roadway	1	21 - 35

*The strength development times are for typical conventional concrete. The time could be reduced by specifying high early strength concrete.

FABRICATION OF TEE BEAMS

Quality control and efficiency at the fabrication plant are probably the most essential ingredients for the successful construction of a modular or prefabricated structure. Precast components will fit together satisfactorily in the field only if they are cast to close tolerances. Since the major portion of a modular construction project takes place in the factory, the major portion of the supervision and inspection also must take place there. Fabrication errors that are not detected at the plant can be very costly and time-consuming to remedy in the field. Precast components cast in a good set of forms and under close supervision will fit together quickly and securely in the field, and will provide a structure far more economical and superior to that which can be obtained with conventional construction techniques.

By adjusting the bottom pallet, the flange side supports, and the end bulkheads, Phoenix Concrete Products of Salem fabricated the 155 tee beams for the five bridge projects considered in this report. Personnel from the Research Council were present at the fabricating plant during the load test of a 42-foot (12.6 m) tee beam for the Norton project and for the fabrication of twenty-one 35-foot (10.5 m) tee beams for the Dickenson County project. The information reported here is based on observations made during the visits and on records made by the Department inspectors assigned to the plant.

Casting Bed

The casting bed shown in Figure 8 is 320 feet (96 m) long. The metal form was specially prepared for the Department's tee beam design. The bottom pallet may be raised or lowered in the 1 foot (0.3 m) wide stem to accommodate beam depths up to 30 inches (0.75 m). The end bulkheads, strand hold-downs, and insert clamps may be adjusted to accommodate various span lengths, but the span length is limited to 45 feet (13.5 m) because of the 30-inch (0.75 m) maximum depth. The form is best suited for casting seven beams in one placement. Forms owned by other prestressors in Virginia can accommodate longer spans. Spans between 30 feet (9 m) and 66 feet (19.8 m) are considered to be economical in Virginia.

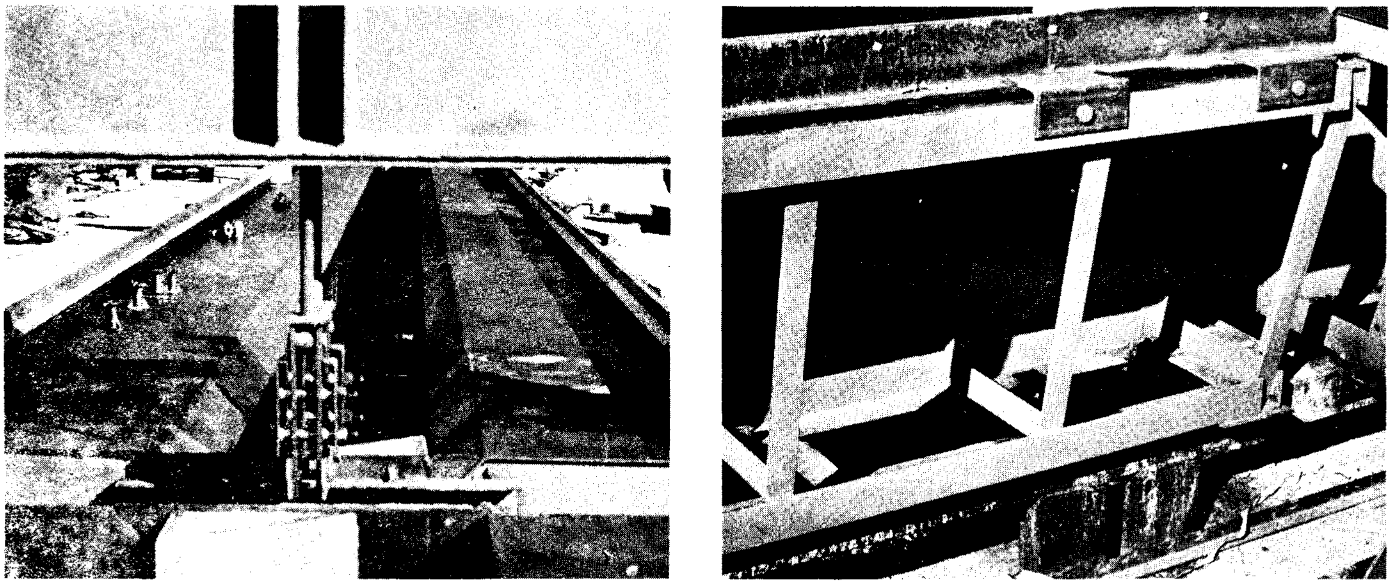


Figure 8. Casting bed at Phoenix Concrete Products.

Casting Operations

Seven tee beams can be produced every 3 days using the casting bed described above, but in the bridges considered here seven beams were typically produced every week. For a 3-day cycle the form is oiled, the reinforcing steel, strands and inserts positioned, and the strands tensioned on the first day; the concrete is placed on the second day and steamed overnight; and the strands are cut and the beams removed on the third day. Strands are tensioned individually from one end of the bed approximately 24 hours before the concrete is placed. Approximately 3 hours is required to place and consolidate the concrete, to finish the surface of the tee, and to cover the bed for steaming. Internal vibration consolidates the concrete in the stem and a vibrating screed consolidates and levels the concrete in the flange (see Figure 9). After the sheen disappears from the surface and prior to initial set the top of the tee is grooved by passing the metal disc shown in Figure 10 across the surface. Once the concrete has reached final set as determined by ASTM C403-70, the beams are steamed at about 150°F until a cylinder strength of 4,000 psi (27.6 x 10^6 Pa) is obtained. Once the beams have cooled the cover is removed, the flange side supports are slid back, the hold-downs are removed, and the strands are cut with a torch, starting with the top strands at each end and working toward the center of the bed. A crane removes the beams from the form and sets them on timbers next to the bed. Inserts for the diaphragm steel are removed from the beams at this time (see Figure 11). The plant inspectors check the beams for length, width, height, camber, sweep, and general condition. The beams are hand rubbed and patched where necessary and a number is placed on each one. Once the concrete in the tee beams has attained a 5,000 psi (34.5 x 10^6 Pa) strength the beams are hauled to a storage area (see Figure 12) where they are placed on timbers until the contractor is prepared to receive them at the bridge site. As the beams are loaded for shipment they are again checked for length and camber. Department inspectors are present at the plant at all times to ensure that all operations are conducted in accordance with the Department's Road and Bridge Specifications.

Figure 9. Concrete is placed at Phoenix Concrete Products.

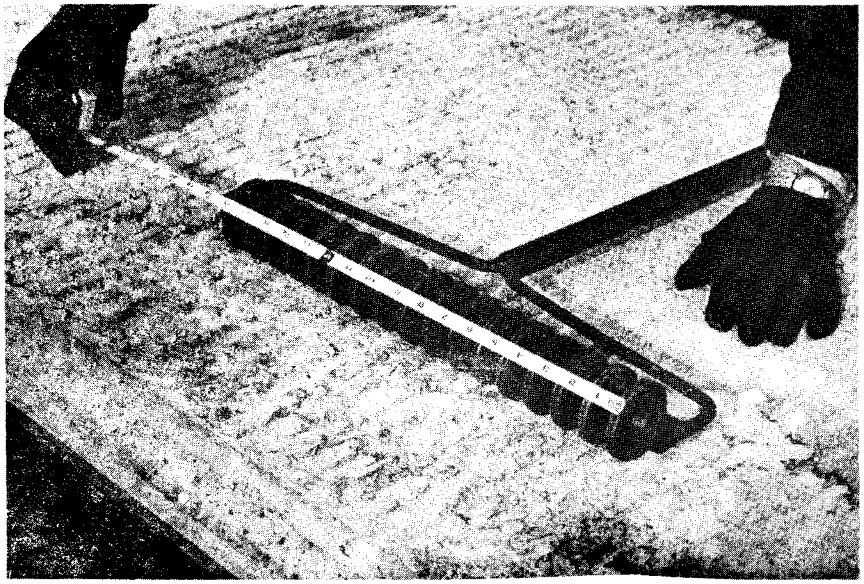


Figure 10. Metal disc used to place grooves in surface of tee beams.

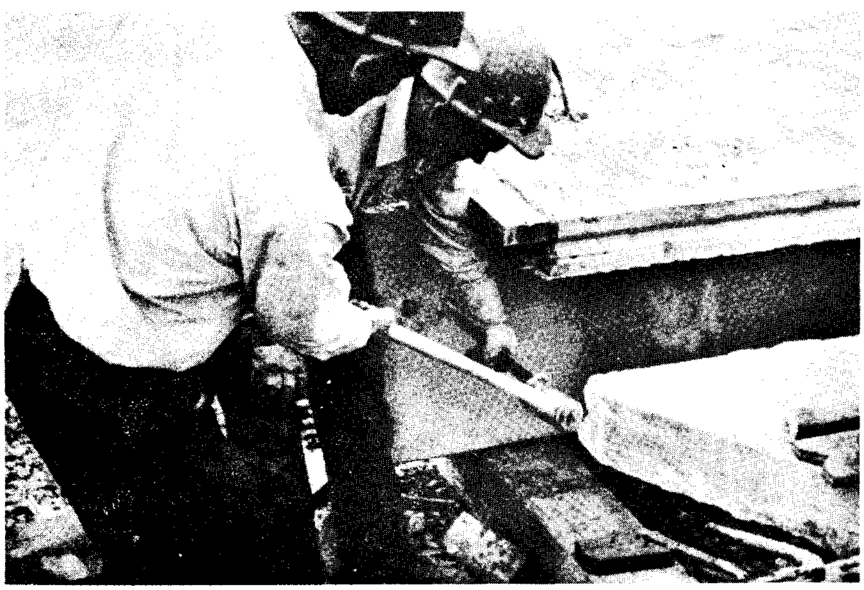


Figure 11. Inserts that provide voids for the diaphragm steel are removed.

Figure 12. Tee beams are stored at fabrication plant.

Load Test

One beam from the first concrete placement at Phoenix Concrete Products was load tested (see Figure 13) in accordance with Section 219.14(c) of the Department's specifications following the procedure set forth in Virginia Test Methods (VTM-20— July 1, 1970). The observed deflection at midspan for the 42-foot (12.6 m) interior tee beam was 75% of the calculated deflection. No cracks were visible after the maximum load was maintained for 5 minutes. After the load was removed the rebound was 100%. The test confirmed that the tee beam produced by Phoenix Concrete Products was performing satisfactorily.

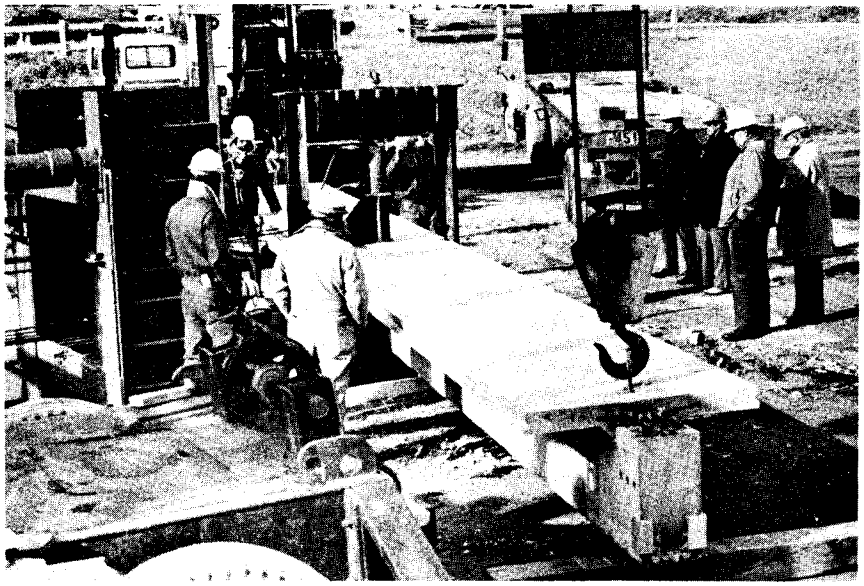


Figure 13. Tee beam is load tested.

Special Items

Although an effort was made to standardize the shape of the tee beam for bridges, the exterior beam (see Figure 14) as designed required special time and attention during fabrication. An extra 4 inches (10.2 cm) of concrete is required for the outside 2 feet (0.6 m) of the flange. Threaded inserts which provide connections for the parapets and the diaphragm steel must also be cast into the beam. The special shape of the exterior beam requires special treatment in that the top surface has to be hand finished, the face of the flange has to be properly roughened to ensure bond with the overlay along a vertical plane, and the unstable shape of the beam means that it has to be supported against overturning until it is properly secured in the bridge. Shaping the flange of the exterior beam like the interior beam would facilitate mass production and reduce costs. Also, a vertical bond plane and stability problems would be eliminated. A bridge incorporating these features is under contract.

Metal inserts always require special attention, increase costs, and increase the chances for mistakes. The current tee beam design requires that metal tubing be positioned in the exterior beam forms at each end and at midspan to provide openings through which to place diaphragm steel.

Threaded inserts are required for the inside face of the stem of the exterior beams. The location of the tubing and inserts is a function of the depth and length of the beam and must be adjusted for each bridge project. The tubing was not properly positioned in several of the beams for the Norton job and as a result the problem had to be corrected at the bridge site by constructing a nonstandard, extra wide diaphragm.

Another insert required by the tee beam design is a flat metal plate containing four studs which must be positioned on the bottom of the form and at the bearing areas of the beam. The slighest inperfection between the bottom of the form and the bottom of the plate allows cement paste to flow between the two surfaces. The excess paste and concrete must be removed from the bottom of the beam to allow for proper bearing with the soleplate. An insert as currently specified which is wider than the soleplate and properly secured makes it unnecessary to grind the bottom of the beam adjacent to the insert plate. Cement paste must be removed from the plate before attaching the soleplate.

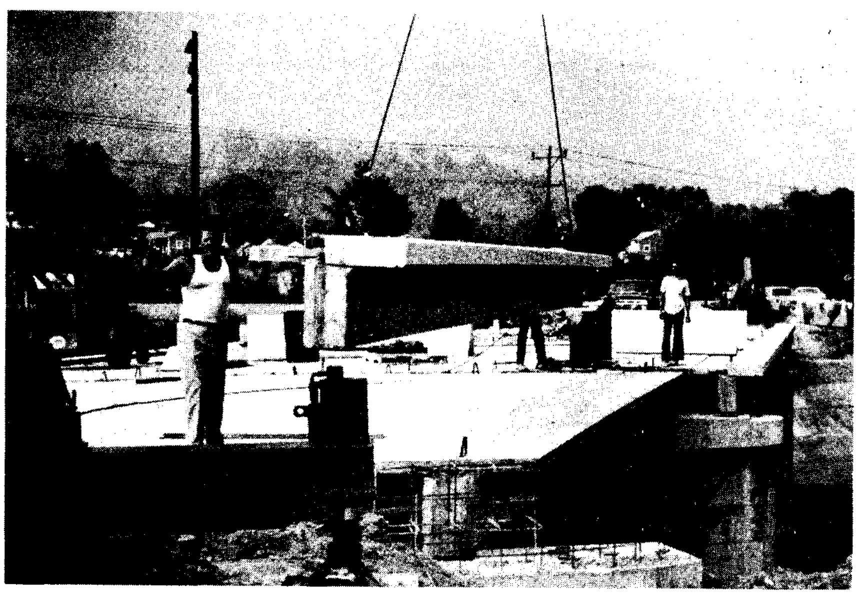


Figure 14. Exterior tee beam is placed into position on span C B604, Norton.

Tolerance

A precast member takes the shape of the form in which it is cast. A form must be constructed and maintained to close tolerances to provide tee beams which fit close to close tolerances. Construction activities can distort the shape of the form and, therefore, it should be checked periodically with a level, taut line, and square. A warped tee beam can delay operations as the contractor tries to obtain a reasonable fit for the member. Since the flange of the tee serves as the deck form the contractor cannot adjust the form without adjusting the tee, and this can cause bearing problems.

Particular attention should be devoted to the portion of the casting bed which supports the bearing area of the beam. The depth of the beam must be correct at the bearing area for the subdeck elevation to be correct in the field. An insert plate which floats upward or twists during the fabrication of the beam will cause bearing problems in the field.

Proper quality control at the fabricating plant is essential to prevent large variations in the camber in the beams. Since the deck forms cannot be adjusted in the field other than by adjusting the entire tee at the bearing area, the tee beam which sets the highest will control the finish grade of the bridge. The quantity of concrete required for the 4-inch (10.2 cm) composite overlay is determined by the camber and the relative fit of the beams, both of which are controlled in fabrication. Although a beam with high camber can be loaded during storage, such an operation is often not practical. Camber is a time-dependent variable but the additional overlay required because of camber due to prolonged storage appears to be insignificant for a tee beam span. The need for additional overlay concrete is more a function of the relative depth of the beams as measured from the top of the soleplate to the top of the flange.

TRANSPORTING, ERECTING, AND CONNECTING THE TEE BEAMS

Transporting Tee Beams

The tee beams were usually loaded on a lowboy at the Phoenix Concrete Plant on the day before they were scheduled to arrive at the bridge site. Two beams were loaded side by side on each low boy. No special hauling permits were required because the total flange width of two beams was 8 feet (2.4 m) and the weight of two beams was from 16 to 21 tons (14.4×10^3 kg to 18.9×10^3 kg) depending upon the length of the tees. The beams were properly braced and secured so that flexure of the trailer bed was not transferred to them and trailer movements would not cause them to shift. A wooden frame located near the bearing areas was used to support the beams. Beams were transported to the bridge site in the order in which they were to be placed, and deliveries were scheduled so that they could be placed as soon as possible after they arrived. Satisfactory communication between the prestressor and the contractor was essential to ensure that the beams were placed in a manner that eliminated delays and extra handling of the beams. Of the 80 or so shipments of tee beams required for the five study bridges, only five shipments encountered problems. Four of the shipments for the Floyd County bridge required extra handling because the 45-ton (40.5×10^3 kg) crane failed to operate on the day the center span beams arrived (see Figure 15). The beams were unloaded with a smaller crane and stored at the site until the larger crane could be repaired. A shipment destined for B604 in Norton was damaged when the beams were dumped from the low boy as it was being backed into position at the bridge site. The driver removed the chains which secured the tee beams prior to positioning the truck for unloading. Construction was delayed until Phoenix Concrete Products could fabricate one replacement beam and repair the other beam.

Figure 15. Tee beams ready to be placed on the center span of Floyd County bridge.

Erecting the Tee Beams

Men and equipment were usually ready at the bridge site when the beams arrived. The crane was positioned in an appropriate, predetermined location. When possible the crane was located so that it would not interfere with traffic and would have to be moved as few times as possible.

For the Floyd and Dickenson County bridges a considerable amount of time and effort was required to get the crane to the site and to the most appropriate location. The structures are located in remote areas and the closest detour required several miles of travel. For the Floyd County bridge the crane had to be dismantled before it could be moved across the Little River, which was necessary to complete the placement of the beams for all three spans. The Norton project, on the other hand, involved the construction of three bridges simultaneously and the use of adjacent temporary bridges which permitted a large crane to be readily available.

A 45-ton (40.5 x 10^3 kg) crane should be satisfactory for handling most beams for most site conditions. Although a smaller crane can handle smaller beams, it's better to have a crane which is too large than one which is too small. The boom distance, weight of the crane, weight of the beams, and crane cost should be taken into account when selecting a crane for a particular job.

Bearing areas should be properly prepared before the beams arrive. Once a beam is placed it is examined for fit. Beams which are fabricated accurately will fit together in the field quickly and easily. Beams which have excessive sweep or different depths and don't bear properly will require additional time and attention in the field. The best procedure is to try to achieve a properly prepared bridge seat and an accurately fabricated tee beam and to be ready at the site to apply some suitable corrective measure. Tee beams can be lifted from a low boy and put into place in a few minutes. On-site construction time is primarily a function of the time required to apply the necessary corrective measures for poor fitting tee beams. Beams for a typical 42 ft. x 44 ft. (12.6 m x 13.2 m) span can be unloaded and set in position in about 2 hours. The beams are fabricated with a width reduction of up to 0.38 inch (0.95 cm) to facilitate placement.

Bearing Areas

There were no significant bearing problems with the tee beams fabricated for the Dickenson and Floyd County bridges. The beams were set and the soleplates welded at the bridge site. Adjacent tee beams fit together very well. Only the surface of the exterior beams appeared to be significantly lower (0. 75 inch [1.9 cm]) than the adjacent beams. Evidently the tolerance of the casting bed used to fabricate the exterior beams caused the difference in height.

Considerable time was required to properly fit some of the tee beams for the Norton bridges. The soleplates were welded at the fabricating plant before the beams were delivered and they could not be adjusted in the field. Several of them did not match up with the anchor bolts permanently secured in the bridge seats (see Figure 16), and some of them appeared to be out of plane. For example, a soleplate on one end of the beam would bear uniformly on the bridge seat whereas that on the other end would not. A tee beam which is warped or an insert plate which is cast out of plane would cause poor bearing. The fact that the soleplate was bearing partly on concrete and partly on the metal insert plate also contributed to the poor bearing. Time-consuming corrective measures applied at the bridge site included grinding the bridge seat, placing expoxy on the bridge seat, and enlarging the anchor bolt holes in the soleplate. The corrective measures were very time-consuming because the beams had to be continually jacked and braced in and out of position until a satisfactory bearing was achieved. Current tee beam design specifies an insert plate which is wider than the soleplate and thus prevents the soleplate from bearing on concrete.

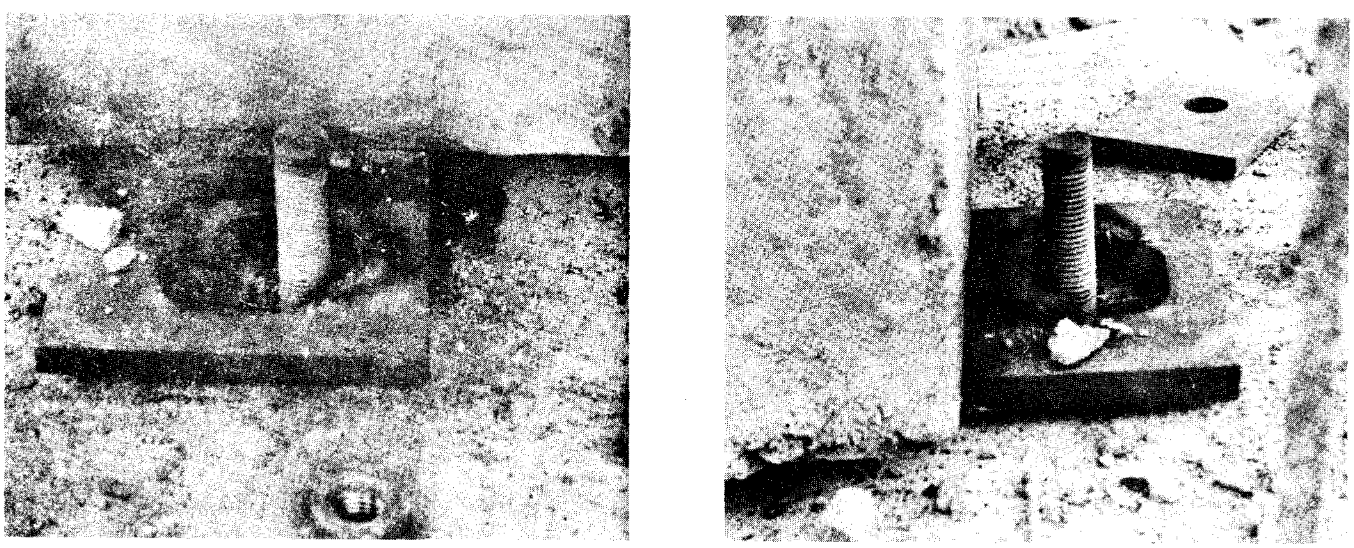


Figure 16. Corrective measures necessary when a soleplate attached at the fabricating plant will not align with the anchor bolts positioned in the bridge seat.

Diaphragms

After the tee beams were properly seated the diaphragms were constructed. The contractors prepared and anchored the forms for the diaphragms in various ways and quite often the same set of forms was used on several spans (see Figures 17 and 18). Approximately 3 days per span were required to form and place the concrete for the diaphragms on the Dickenson and Floyd County bridges. Forms for the midspan diaphragms for the Floyd County bridge were prepared before the tee beams arrived and they were quickly secured into position. If one set of forms is used for each span, many small concrete pours are required; if forms are built for each span, then concrete can be placed in all the diaphragms at the same time. The reinforcing steel for the diaphragms is positioned as the tee beams are placed.

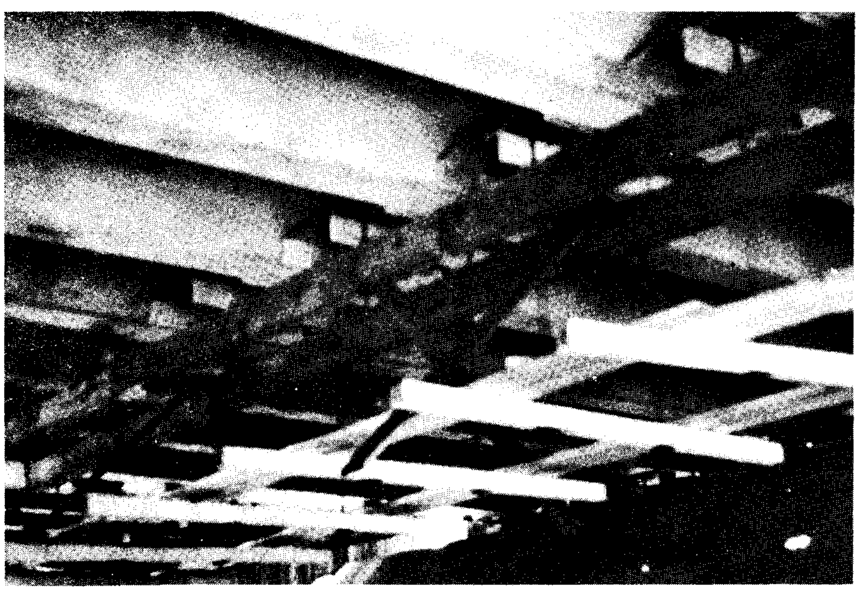


Figure 17. Forms for the midspan diaphragms.

Figure 18. Concrete placed in the end span diaphragm forms.

Keyways

The keyways between adjacent tee beams were filled with a neat Durcal gypsum cement paste having enough water to provide a workable consistency. An approved sealer was used to seal the bottom of the keyways on the Dickenson County bridge before placing the paste. No sealer was used on the Norton and the Floyd County structures since in many cases the bottoms of the tee flanges were close enough together to prevent the paste from running out. The center keyway on the Dickenson County bridge was wider than anticipated [about 2 inches (5.08 cm)] because the tee beams were positioned on the anchor bolts which were dimensioned from the center of the bridge without consideration of the transverse slope of the deck. Current plans provide dimensions along the top and bottom of the transverse section of the tee beams.

Laboratory studies to be discussed later indicated that when the surface of the keyways is properly prepared and the nonshrink paste is properly cured the paste will bond well. A core removed in one piece from the keyway of B602 in Norton exhibited questionable bond. Very little site time is required to fill the keyways with paste but the nonshrink cement is expensive. To save money a mortar consisting of one part Durcal gypsum cement and one part sand was mixed at the site and used to fill the keyways of B602 in Norton.

TEE BEAM CAMBER

Department inspectors assigned to the prestressing plant used a taut line stretched between the storage supports to determine the camber in the tee beams at the time they were removed from the forms and at the time they were loaded for shipment to the bridge site. The data as reported by the plant inspector are summarized in Table 2.

It is apparent from Table 2 that the measured cambers are influenced by the bed position in which the beams are cast, since there is more variability in camber between the beams from different bed positions than between beams cast in the same bed position. For initial cambers one standard deviation for a particular bed position is ± 1/16 — ± 1/8 inch (1.6 mm - 3.2 mm) whereas one standard deviation for all seven bed positions is ± 3/16 inch (4.8 mm). Since the bottom pallet of the form can be adjusted to achieve the desired beam depth, the accuracy to which the bottom pallet can be positioned can be responsible for some of the differences in camber between the bed positions. There is slightly more variability in camber among the beams at the time of delivery. One standard deviation for the beams is ± 1/16 to ± 5/16 inch (1.6 mm - 7.9 mm) for the same bed position and from ± 3/16 to ± 1/4 inch (4.8 mm - 6.4 mm) for the entire form. Since the camber in the beams increases with time and since the beams were delivered anywhere from 0 to 40 weeks after fabrication, more variation in camber would be expected at delivery than just after casting. One standard deviation for the average camber at delivery is between ± 1/8 and ± 1/4 inch (3.2 mm - 6.4 mm) for most of the beams, and therefore 68% of the beams satisfy the current Department specification[4] for I beams, which allows 1/2 inch (12.7 mm) differential camber between adjacent beams at the time of erection. Also, there is a high probability that many of the remaining 32% of the beams satisfy the specification, because two standard deviations for the average camber at delivery is only ± 1/4 to ± 1/2 inch (6.4 mm - 12.7 mm). On the average only the beams for B639 in Floyd County satisfy the Department specification for I beams which allows a ± 50% camber differential from the computed camber at the time of erection. The Floyd County beams were delivered 2-4 weeks after they were fabricated.

Table 2

Beam Camber for Different Bed Positions
(Inches x 16)

Bridge	Norton - B602, 603, 604						Floyd County - B639						Dickenson County - B616					
Number Beams	99						24						32					
Design Camber	11						10						6					
	Release		Delivery		Difference		Release		Delivery		Difference		Release		Delivery		Difference	
Bed Position*	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	θ	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	θ	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	θ
1	10	2	18	2	8	1.8	9	1	13	4	4	1.4	5	1	10	1	5	2.0
2	8	1	17	2	9	2.1	10	1	14	1	4	1.4	7	1	9	3	2	1.3
3	12	2	23	3	11	1.9	13	2	16	5	3	1.2	10	1	16	2	6	2.3
4	9	1	17	2	8	1.9	11	1	11	1	0	1	5	1	9	2	4	1.8
5	14	1	24	2	10	1.7	15	2	19	1	4	1.3	10	2	15	1	5	1.5
6	15	1	23	2	8	1.5	16	2	20	2	4	1.3	14	2	17	2	3	1.2
7	10	2	19	2	9	1.9	11	2	12	2	1	1.1	5	1	9	2	4	1.8
All Positions	·11	3	20	3	9	1.83	12	3	15	4	3	1.24	8	3	12	4	4	1.7
Theoretical Camber	11		17-28		6-17	1.5 - 2.5	10		18-30		8-20	1.5-2.5	6		12-20		6-14	1.5-2.5

$\bar{x}$ = average
s = 1 standard deviation (plus or minus)
θ = camber at delivery/camber at release

1/16 inch = 1.6 mm

*Steel strands tensioned adjacent to bed position 7.

It is reasonable to expect some variation in camber between beams because of variations in concrete strength, imperfections in the bottom of the beams, variations in the distance between storage supports, thermal loadings at the time the cambers were measured, and differences in persons measuring the cambers. Camber data for selected bridge spans are shown in Table 3. Additional camber data are presented with the depth of cover data to be discussed later.

According to the Prestressed Concrete Institute the camber in the beams at the time of erection should be 150%-250% of the camber at release. The study beams had a camber at delivery of 100%-230% of the camber at release, and on the average the cambers at delivery were 124% for the Floyd County beams, 170% for the Dickenson County beams, and 183% for the Norton beams. The values are consistent with good quality control and typical of the behavior to be expected since the camber in the beams increases with time.

The average values for camber at release, at delivery, and, in most cases, at the time the overlay was placed are plotted on log paper in Figure 19 for each of the 16 bridge spans. It is apparent from Figure 19 that the camber in the beams increases linearly with the log of time. From the data the following equation was formulated.

$$\text{Camber}_{(t)} = \text{camber}_{(t\,=\,1)} + \varnothing \log t$$

where t is time in days and Ø is .20, .14, and .13 for the Norton and Floyd County and Dickenson County bridges, respectively. One would expect Ø to vary with the depth of the tee beam, the strand pattern, and the span length. The data for B602 at Norton appear to be the most variable, but the author cannot explain why. The B602 beams were the last to be fabricated and were shipped in August. Most of the data fall within a band width of 1/8 inch (3.2 mm), which is the limit of the accuracy of any of the data because of thermal loadings, differences in personnel, and the manner in which the camber was measured. A spot check of the camber in the Norton beams some 8 to 10 weeks after fabrication revealed average cambers of about 1 1/16 inch (27 mm), which plots linearly with the camber data recorded at release, at delivery, and prior to placement of the overlay. In general the beams appeared to behave in a desirable fashion, which is reflective of the quality control that is required by the Department.

Because the camber increases linearly with the log of time a typical Norton beam with an initial camber of .69 inch (18 mm) would have a camber of .92 inch (23 mm) in 2 weeks; 1.08 inch (27 mm) in 3 months; 1.20 inch (30 mm) in 1 year; and 1.44 inch (37 mm) in 15 years. Since a tee beam will seldom be overlaid in less than 2 weeks after fabrication, very little additional overlay concrete will be required because of camber if the beams are overlaid 3 months or 1 year after fabrication. Also, beams could be stockpiled

TABLE 3

Camber Data for Selected Bridge Spans

Bridge: B604 Norton, Span: A

		1	2	3	4	5	6	7	8	9	10	11	x̄	s
	Bed position	1	2	2	3	3	3	4	7	5	5	1		
Camber (16th of inch)	At release*	11	9	7	10	16	11	8	9	13	14	9	11	3
	At delivery*	18	19	14	19	18	22	17	9	22	22	18	18	4
	At overlay**	13	20	14	12	17	18	16	5	27	26	7	16	7
Time (weeks)	Release to delivery	24	24	27	27	24	22	25	0	30	24	16		
	Release to overlay	30	30	33	33	30	28	31	3	36	30	22		

Fab. date: January -.February 1976
Delivery date: July 27 & 29, 1976
Overlay date: September 11, 1976

Bridge: B604 Norton, Span: B

		1	2	3	4	5	6	7	8	9	10	11	x̄	s
	Bed position	1	2	2	3	3	4	4	4	5	5	1		
Camber (16th of inch)	At release*	13	10	8	12	16	9	9	10	12	15	9	11	3
	At delivery*	15	15	18	18	19	16	15	18	20	25	16	18	3
	At overlay**	7	18	17	14	19	16	23	19	18	24	13	17	5
Time (weeks)	Release to delivery	20	20	22	22	20	23	21	18	21	19	22		
	Release to overlay	28	28	30	30	28	31	29	26	29	27	30		

Fab. date: January - February 1976
Delivery date: June 30 & July 1, 1976
Overlay date: August 30, 1976

Bridge: B603 Norton, Span: B

		1	2	3	4	5	6	7	8	9	10	11	x̄	s
	Bed position	1	2	2	3	6	7	6	7	5	6	3		
Camber (16th of inch)	At release*	9	9	8	12	15	10	15	9	14	16	11	12	3
	At delivery*	16	18	15	24	20	17	25	19	25	25	24	21	4
	At overlay**	13	17	15	21	21	15	17	14	23	19	18	18	3
Time (weeks)	Release to delivery	34	31	34	33	38	38	36	36	33	33	31		
	Release to overlay	54	51	54	53	58	58	56	56	53	53	51		

Fab. date: February - March 1976
Delivery date: October 28, 29, 30, 1977
Overlay date: March 16, 1977

Bridge: B639 Floyd, Span: B

		1	2	3	4	5	6	7	8	x̄	s
	Bed position	3	7	6	5	4	2	7	1		
Camber (16th of inch)	At release*	14	9	15	13	9	9	11	10	11	2
	At delivery*	18	11	18	20	11	13	14	11	15	4
	At overlay**	14	13	17	19	16	15	16		16	2
Time (weeks)	Release to delivery	2	2	2	2	2	2	2	2		
	Release to overlay	7	7	7	7	7	7	7	7		

Fab. date: September 1976
Delivery date: September 22, 1976
Overlay date: November 1, 1976

Bridge: B639 Floyd, Span: C

		1	2	3	4	5	6	7	8	x̄	s
	Bed position	3	3	7	6	5	4	2	1		
Camber (16th of inch)	At release*	14	14	13	16	17	12	10	8	13	3
	At delivery*	18	20	12	20	18	10	13	10	15	4
	At overlay**	13	15	13	16	17	8	16	10	14	3
Time (weeks)	Release to delivery	3	2	2	2	2	2	2	3		
	Release to overlay	7	6	6	6	6	6	6	7		

Fab. date: September 1976
Delivery date: October 2, 1976
Overlay date: October 28, 1976

*Data reported by plant inspectors (string line bottom of tee beams).

**String line of tops of tees before placement of overlay.

1/16 inch = 1.6 mm

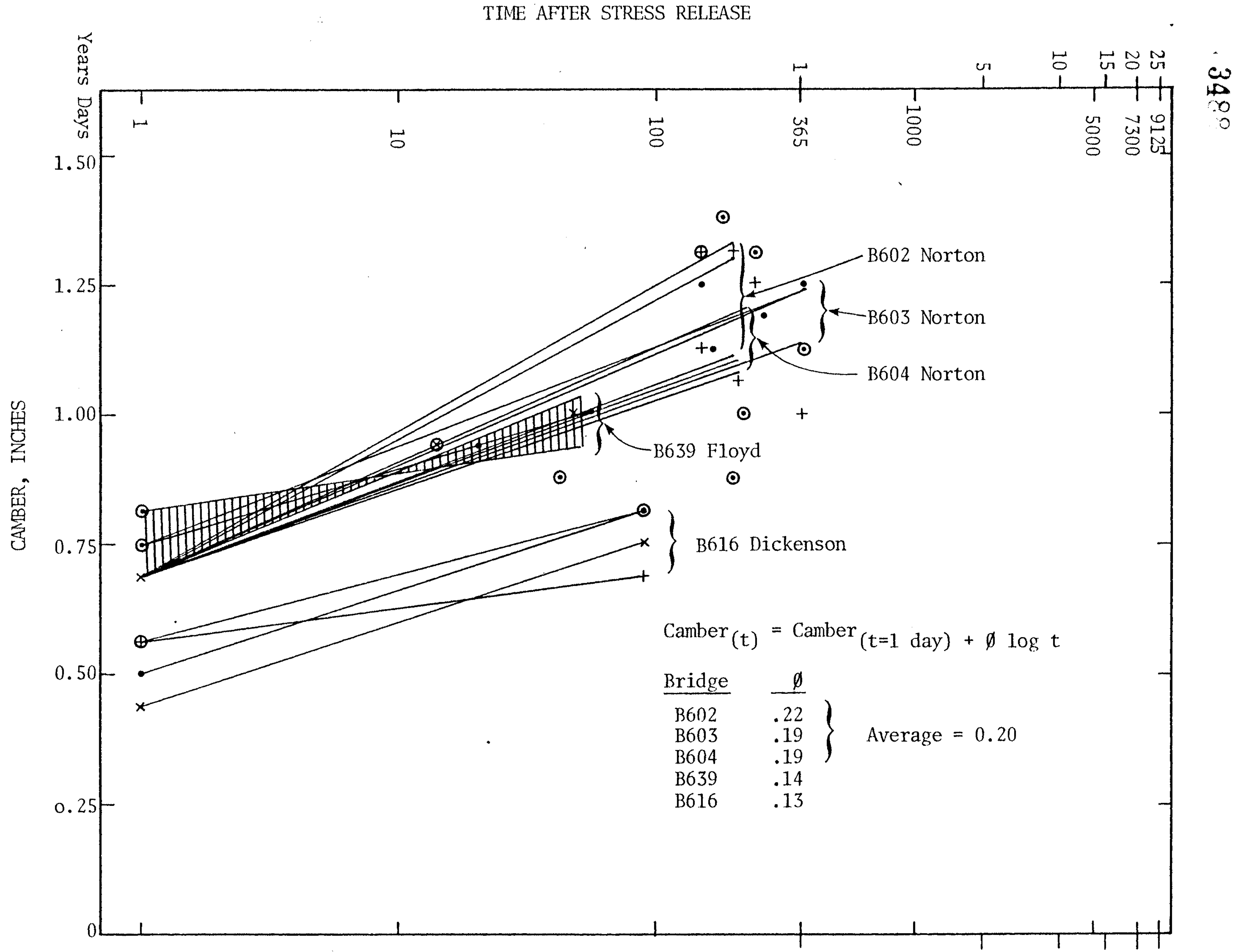


Figure 19. Tee beam camber plotted as a function of time after prestress release. (1 inch = 2.54 cm)

for emergency purposes since the increase in camber is a function of the log of time. Theory indicates that deflections due to the dead load of the overlay on a 42-foot (12.6 m) span is 0.12 inch (3.1 mm) initially and an additional 0.12 inch (3.1 mm) due to long-term creep. The difference between the tee beam camber at 1 year and 15 years is 0.24 inch (6.1 mm). So, if the overlay is placed 1 year after fabrication, the dead load deflection will compensate for the increase in camber. If the tee beams were overlaid several weeks after fabrication, the increase in camber could cause an undesirable upward curvature in the riding surface of each span of the completed structure.

Section 405.11 of the Department specifications should be upgraded to include the precast, prestressed, single-tee concrete beam.

CONSTRUCTING THE COMPOSITE OVERLAY

Formwork

Several men can construct the formwork, place the layer of reinforcing steel, and set the screed for the overlay for one span in about 1 work day. To maintain a minimum overlay thickness of 4 inches (10.2 cm), the formwork cannot be prepared until the tee beams are in position, the diaphragms constructed, and the keyways grouted. Since the backwall is not constructed until after the overlay concrete is placed it cannot be used to support the screed. Also, forms must be attached along the exterior tee beams since the average thickness of the overlay is greater than the 4 inches (10.2 cm) of concrete precast on the outside, top edge of the exterior tee beams. The holes through which the midspan diaphragm concrete is placed must also be plugged before the overlay is placed. One inch (2.5 cm) metal chairs are used to support the reinforcing steel 1 inch (2.5 cm) or more above the top of the tee beams.

Surface Preparation

Prior to placement of the reinforcing steel for the overlay, compressed air is used to remove dirt, debris, and laitence from the surface of the tee beams (see Figure 20). Water is applied to the beams to ensure that the surface is damp as the overlay is placed. The water should be sprayed on the surface for best results, because other application techniques tend to provide excess water which collects in low areas such as in the holes required to place the diaphragm concrete, along the exterior beams, and in the lower areas of the corrugated tee beam surface. Ideally, the beams should be soaked with water for 24 hours and the excess water removed with compressed air just prior to placing the overlay.

The corrugated surface rolled into the top of the tee beams during fabrication tends to double the effective surface area to which the overlay can bond. Although sandblasting was not specified for the study bridges, it will be used on future bridges because it is believed that air pressure is not sufficient to remove the scale, laitence, and weak concrete caused by roughening the surface of the beams during fabrication. The sandblasting will be done just prior to placement of the reinforcing steel.

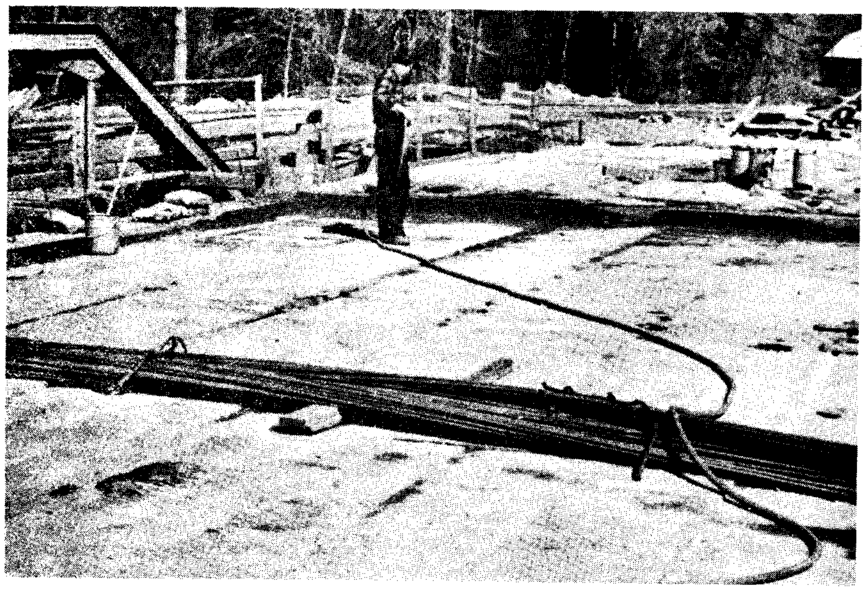


Figure 20. Compressed air is used to remove debris from top of beams prior to placement of reinforcing steel for overlay.

Placing Concrete

About 6 to 8 men were involved in placing the overlays for the Norton bridges. They used a crane and 1-cubic yard (0.76 m^3) bucket, a longitudinal screed, several internal vibrators and shovels, floats, etc. to place the overlays (see Figure 21).

The construction activities for four of the overlay installations were monitored with emphasis being placed on the time and sequence of the activities. The time data for spans A and B of B604 in Norton are given in Table 4. Conventional A4 concrete having a w/c of 0.43 was used on both spans. The average time interval between batching the first truck and completing the screeding activities on the overlay was 4 hours, which agrees well with the 4.25 hours required to place a 6-1/2 in. (16.5 cm) thick base layer for the two-course construction done in Berryville.[5] Another 45 minutes were required to complete the hand finishing operations and apply the curing compound. The screed was passed over the surface of the overlay three to four times to provide the desired finish. The screed tended to "pull" the concrete at times and it was necessary to apply confilm to maintain desirable finishing characteristics.

Time data for spans A and B of B602 in Norton are also given in Table 4. A superplasticizer was used in the A4 concrete to provide a workable mixture with a w/c of 0.34. More time was required to deposit the superplasticized concrete than the conventional concrete because the superplasticizer was added at the site (see Figure 22). Additional plasticizer and air entraining admixture was added as the concrete lost slump and air. The screed was passed over the concrete five to six times, but, by necessity, less time was required than for B604 because the concrete lost its workability and finishability faster than did the conventional concrete. More time was required to hand finish, texture, and apply the curing compound to B602 because more hand finishing was required, more bleed water was present, and the finishability of the deck varied significantly (see Figure 23). It was impossible to apply a satisfactory screed finish to a superplasticized deck because the concrete under part of the screed pulled while that under other portions bled excessively and flowed. The time between batching and the completion of the screeding activities was about the same for both bridges but the total time was 25% greater for B602 than for B604. The superplasticized concrete is discussed in more detail in another section of this report.

Table 4

Time Required for Various Construction Activities

Bridge	W/C	Average Time (minutes) for Each Batch				Interval between first batch time and completion of screeding	Interval between first batch time and completion of applying curing compound
		Travel to site	Deposit concrete	Screed concrete	Finish, texture, apply curing compound		
604	.43	21	23	87	66	4.0 hours	4.75 hours
602	.34	19	39	66	115	4.3 hours	6.0 hours

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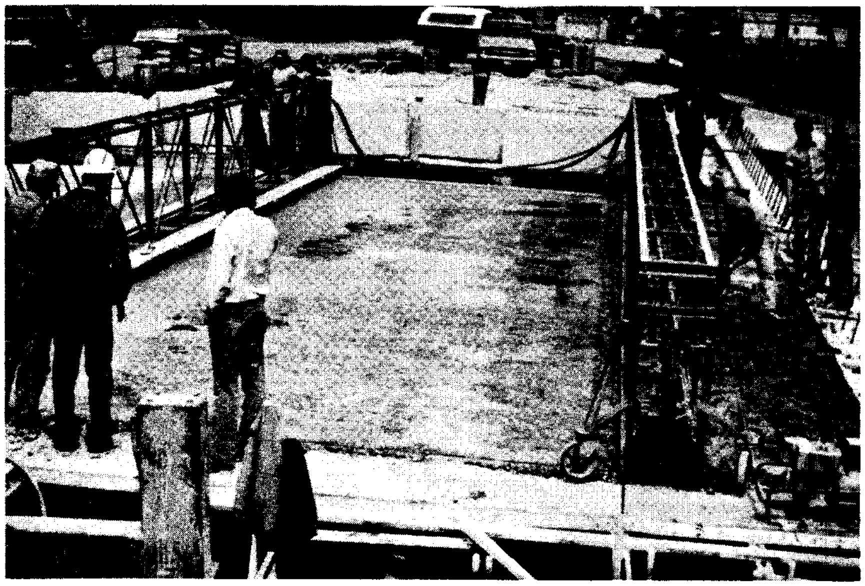

Figure 21. Composite overlay is placed on Span A of B603 in Norton.

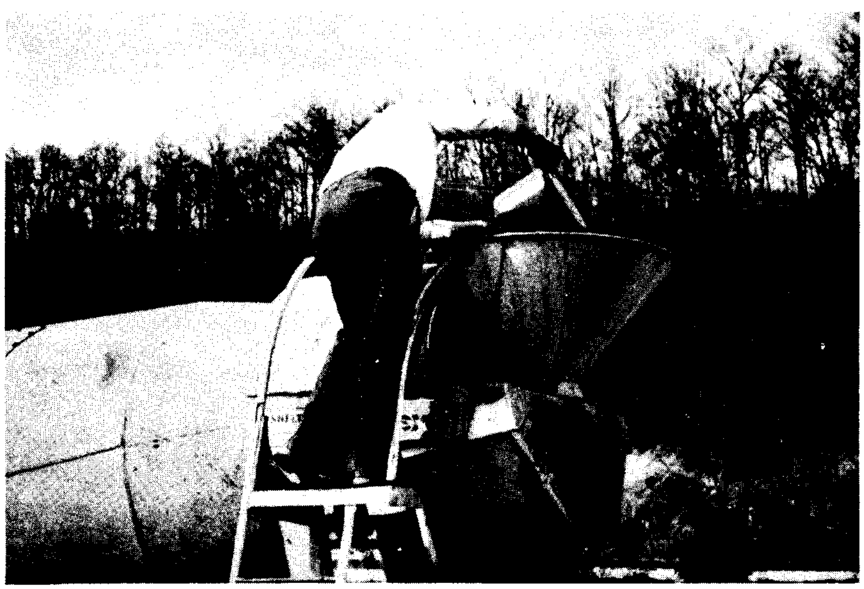

Figure 22. Superplasticizer is added to mix truck at the bridge site.

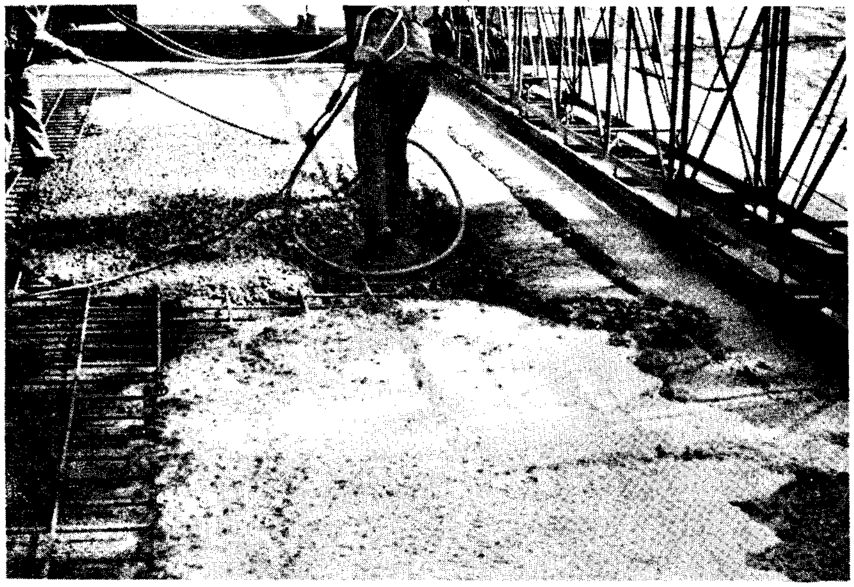

Figure 23. Superplasticized concrete is placed on span A B602 in Norton.

Tee Beam Deflections

Scales were attached at midspan of the tee beams in spans A and C of the Floyd County bridge to monitor beam movements as the composite overlay was placed (see Figure 24). A Wild N-III high precision level was mounted on the edge of the abutment adjacent to the span being overlaid. A construction level was also used to monitor the deflections in span C as the overlay was placed and to run levels on the deck of all three spans about 1 month after the overlays were placed. A construction level was also used to check the elevations at midspan of the bottom of the beams in span C of B602 in Norton. The elevations were run before the overlay was placed and again 5.5 months later. To identify thermal loadings, thermocouple wires were installed at various locations in two beams in span A of B604 in Norton. The temperature distribution in the beams was monitored as the overlay was placed.

It is apparent from Figures 25 and 26 that the beams deflected both up and down over a 24-hour period due to thermal loadings in the deck and beams. Table 5 shows the temperature distribution in two typical tee beams at various times. Although the data are limited there are enough to establish the presence of a temperature differential throughout the depth of the beams. A beam will move to equalize the stresses caused by differential temperatures.

Figure 24. Scales attached at midspan for measuring beam deflections as the overlay was placed.

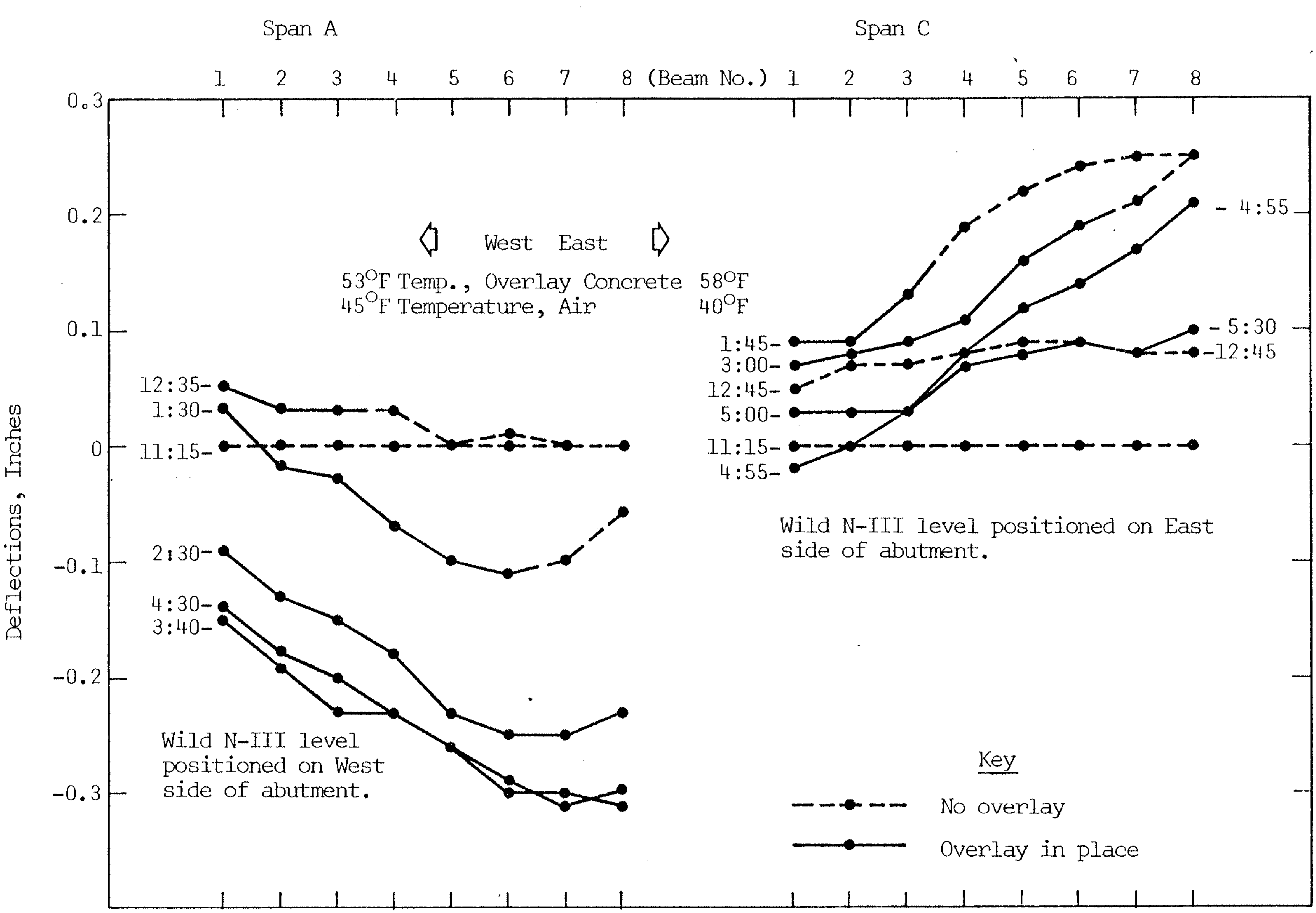

Figure 25. Midspan beam deflection for spans A and C of B639 based on high precision level. (1 inch = 2.54 cm; $^{o}C = [^{o}F - 32]\ 5/9$)

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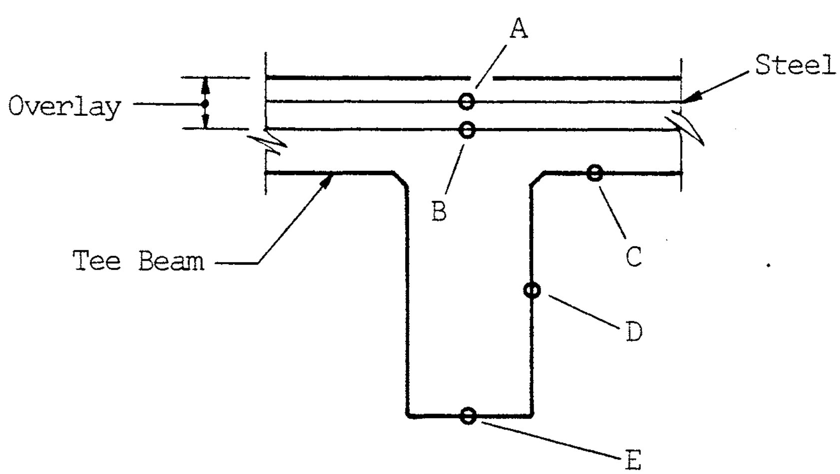
Table 5

Temperature Distribution in Two Tee Beams in Span A of B604 Norton

Date	Time	Air Temp.	Conc. Temp.	Thermocouple Number* 4A	4B	4C	4D	4E	9A	9B	9C	9D	9E
				Thermocouple Temperatures $^{\circ}F$									
9/11/1976	8:45	48		48	48	48	54	54					
9/11/1976	9:15**	50	68	62	56	50	56	56					
9/11/1976	10:30			62	59	56	56	56					
9/11/1976	10:40								56	56	52	52	53
9/11/1976	10:45**	60	69						61	60	52	52	53
9/11/1976	11:15								68	64	56	56	56
9/11/1976	12:30 p.m.								70	65	61	56	56
9/11/1976	1:10								70	65	61	56	56
9/11/1976	1:15	72	71	71	66	60	57	57					
11/17/1976	5:00 p.m.	47							46	44	43	38	40
11/18/1976	9:45 a.m.	45							36	35	36	37	39

$^{\circ}C = (^{\circ}F - 32)\ 5/9$



* Tee beam instrumentation at midspan with thermocouples A - E.

**Overlay just placed on top of instrumented tee beam.

Figure 25 shows the movement of the tee beams in spans A and C of B639 caused by a combination of dead load from the overlay and thermal load. For span A the zero datum was established 15 minutes before the first part of the overlay was placed. Most of the beam movement was downward because of the dead load of the concrete and the gradual loss in thermal gradient which occurred in the afternoon hours. For span C the zero datum was established 2 hours before the first part of the overlay was placed. During this 2-hour period the beams continued to rise due to the thermal loading. As the overlay was placed, the beams began to deflect downward due to the dead load and to the loss in thermal gradient. It is believed that the net movement of the beams in Figure 25 was upward in span C as opposed to downward in span A because the reference datum coincided with the start of the overlay placement in span A, whereas it preceded the placement by 2 hours for span C. The data for Figure 25 are supported by Figure 26, which has the reference datum for span C coinciding with the start of the overlay placement. The thermal movements appear to be greater on the east side than the west side of the spans because the east side was the first to be influenced by the sun and the last to be overlaid.

From the data it can be concluded that on the average the tee beams in a 40-foot (12 m) span deflect downward about 0.1 inch (2.5 mm) due to the dead load of the overlay. The tee beams move upward about 0.1 inch (2.5 mm) due to thermal loading. Deflections as much as 0.25 inch (6.4 mm) upward and downward can be expected as an overlay is placed, depending upon the thermal loading on the tee beams. Design calculations indicate that the beams should deflect downward 0.10 inch (2.5 mm) due to the dead load of the overlay.

Figure 27 shows the relative elevations of the beams in span C of B602 in Norton 5.5 months after the overlay was placed. On the average the beams had deflected about 0.1 inch (2.5 mm), which is equivalent to the dead load deflection at the time the overlay was placed on B639 in Floyd County. The data in Figure 27 are also supported by data taken for the Floyd County bridge about 1 month after the overlays were placed. Elevations that were run on the top of the deck at 12:00 noon showed that on the average the beams exhibited a midspan deflection of 0.1 inch (2.5 mm).

For practical construction purposes the deflection due to the overlay on a 40-foot (12 m) span is negligible at the time the overlay is placed and at 5.5 months later. The accuracy of a construction level is 0.24 inch (6.1 mm),which is greater than or equivalent to the dead load and thermal load deflections. But it should be remembered that camber measurements on 40-foot (2.5 mm) beams may be 0.2 inch (5.1 mm) greater at 4:00 p.m. on a clear summer day than at 8:00 a.m. on the same day. Also, it would be possible to construct an overlay which is less than 4 inches (10.2 cm) thick if the screed were set at 8:00 a.m. and the overlay were placed at 3:00 p.m. However, the amount less than 4 inches(10. 2 cm) would be negligible and within the limits of accuracy of a construction level.

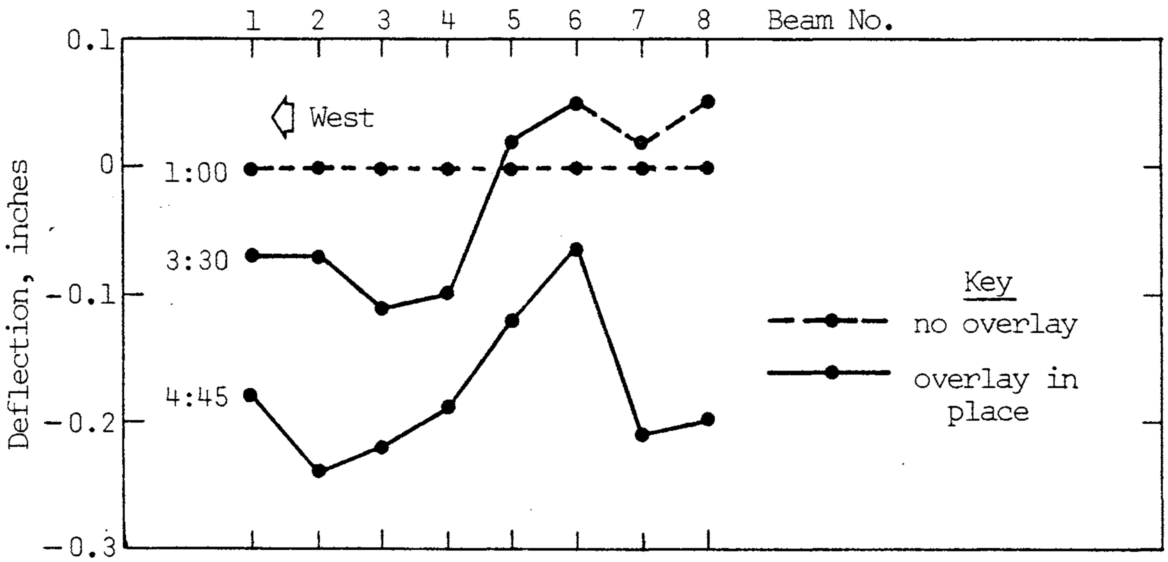


Figure 26. Midspan beam deflections, span C of B639, Floyd County. Construction level on west side of span. (1 inch = 2.54 cm)

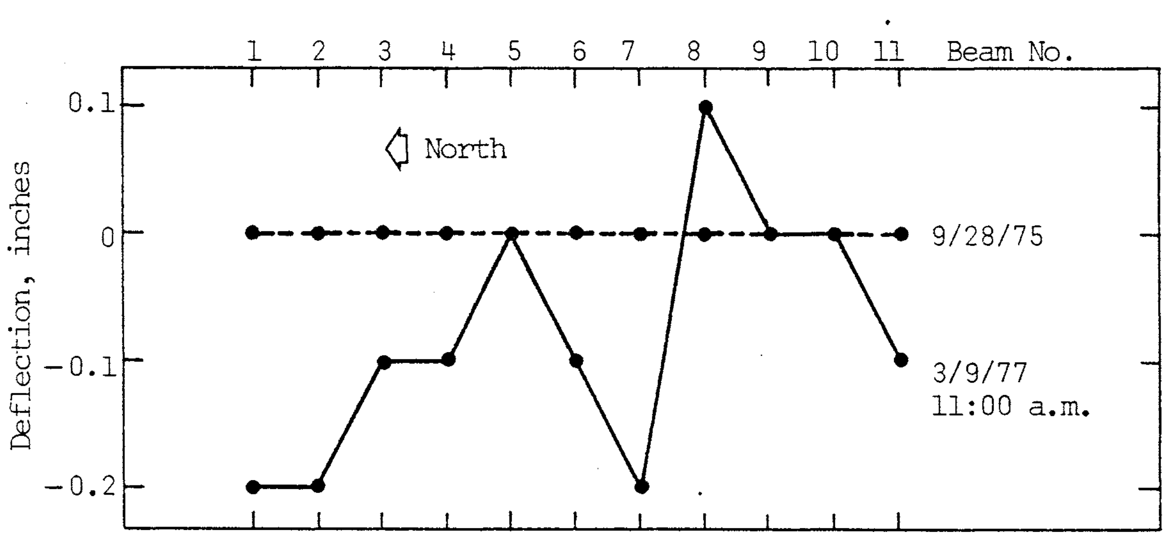


Figure 27. Midspan beam deflections 5-1/2 months after overlay placed, span C of B602, Norton. Construction level on south side of span. (1 inch = 2.54 cm)

Depth of Cover

The single-tee bridge design requires a concrete overlay 4 inches (10.2 cm) or more thick. The actual depth of the cover is a function of the camber in the beams, the relative depths of the beams, the relative elevations of the bridge seats, and the fit between the bridge seat, soleplate and insert plate.

Selected spans on the Norton and Floyd County bridges were examined for depth of cover by various means. These included probing the fresh concrete, measuring the distance between the top of the deck and the bottom of the screed before the overlay was placed, and stretching a string line over the screed supports and measuring the depth before the overlay was placed. The data are presented in Table 6. From these data the following can be seen.

1. Measuring the distance between the bottom of the screed and the top of the tee beams is the most accurate way of determining the depth of cover.

2. Measuring the distance between a string line and the top of the tees is about as accurate as using the screed as a reference point, but care must be taken to make sure that neither the line nor the screed has sag. When the deck is in a vertical curve, the string line values must be corrected for the curvature in the screed.

3. Probing the concrete for depth of cover is the least accurate and most undesirable because the hole must be patched. Although probing takes into account the thermal and dead load deflections in the beams, data previously presented indicate that for beams up to 42 feet (12.6 m) long the form movements due to dead load and solar interaction are 0.25 inch (6.4 mm) or less. The texture of the top of the tee beams can influence depth probe measurements approximately 0.25 inch (6.4 mm).

4. The camber at the time the overlay is placed is 1.1 to 1.9 times the camber at release, depending upon such variables as the time interval between fabrication of the beams and placing the overlay (most of the change occurs in the first month), the temperature at the time camber measurements are taken (about 0.1 inch (3 mm) influence on camber), and the manner in which the measurements are taken (0.1 inch [3 mm] or more influence on camber).

5. The average depth of the overlays is 5.12 inches (13.0 cm), which is equivalent to the 4-inch (10.2 cm) minimum cover plus the average camber in the beams at the time the overlays were placed. On the average 0.46 inch (1.2 cm) (40%) of the extra overlay is caused by camber, and the remaining 0.66 inch (1.7 cm) (60%) is caused by the relative depth of the beams, the relative elevations of the bridge seat, the fit between the bridge seat, soleplate, and insert plate, the accuracy to which the finish grade is set to ensure the 4-inch (10.2 cm) minimum thickness, and the relative elevation of the point used to establish the finish grade.

6. The "plan thickness" of the overlay should take the camber of the beams into account. For example, the plan thickness should be 4 inches (10.2 cm) at midspan, 4 inches (10.2 cm) + c/3 at the quarter points, and 4 inches (10.2 cm) + c at the ends of the span, where c is the camber at the time the tee beams are overlaid.

Table 6

Depth of Cover Data for Selected Bridge Spans (Inches)

Bridge -Span	Measurement Method	Depth Locations ℓ = span length										Depth		Camber (Inches x 16)								Theoretical Overlay Depth**	Average Depth Minus Theoretical Depth
		oℓ		ℓ/4		ℓ/2		3ℓ/4		ℓ				Initial		Delivery		Before Overlay		After Overlay			
		x̄	s	x̄	s	x̄	s	x̄	s	x̄	s	Average*	Minimum	x̄	s	x̄	s	x̄	s	x̄	s		
B604-A	Probes	5.0	.7	4.6	.4	4.4	.3	4.8	.2	5.3	.3	4.80	4.08	11	3	18	4	16	7	14	4	4.44	0.36
B604-B	Probes	5.7	.4	4.9	.6	4.7	.5	5.0	.3	5.3	.5	5.06	4.08	11	3	18	3	17	5	—	—	4.44	0.62
B604-C	Probes	5.5	.4	5.3	.3	5.0	.5	5.2	.9	5.6	.4	5.28	4.20	11	3	22	4	—	—	14	4	4.57	0.71
B604-A	String-line	5.5	.4	4.9	.4	4.6	.4	5.1	.3	5.8	.4	5.08	4.25	—	—	—	—	—	—	—	—	4.44	0.66
B604-B	String-line	5.9	.4	5.2	.3	4.7	.4	5.2	.3	5.9	.4	5.23	4.38	—	—	—	—	—	—	—	—	4.44	0.79
B602-A	Screed	5.9	.1	4.9	.3	4.5	.2	4.8	.2	5.7	.3	4.98	4.32	11	3	20	3	21	6	—	—	4.55	0.43
B602-B	Screed	5.5	.2	4.7	.2	4.5	.3	5.1	.3	6.0	.3	5.02	4.08	11	3	21	3	21	5	—	—	4.55	0.47
B602-B***	String-line	5.7	.3	4.6	.2	4.2	.2	4.9	.3	5.9	.2	4.87	4.01	—	—	—	—	—	—	—	—	4.55	0.32
B602-C	String-line	5.5	.4	4.9	.4	4.4	.3	4.9	.4	5.4	.3	4.89	4.08	11	3	21	3	14	6	—	—	4.36	0.53
B603-A	Screed	6.0	.6	5.0	.6	5.1	.2	5.3	.3	5.6	.3	5.30	4.08	11	3	19	3	20	3	—	—	4.52	0.78
B603-B	—	—	—	—	—	—	—	—	—	—	—	—	—	12	3	21	4	18	3	—	—	4.47	—
B603-C	Screed	5.7	.2	5.2	.3	4.8	.3	4.9	.2	5.2	.3	5.08	4.56	11	3	20	3	16	3	—	—	4.42	0.66
B639-A	Probes	5.6	.4	5.3	.3	5.2	.3	5.2	.2	5.7	.4	5.33	4.75	—	—	—	—	—	—	—	—	4.42	0.91
B639-C	Probes	5.8	.3	5.4	.1	5.0	.2	4.9	.3	5.5	.5	5.22	4.75	—	—	—	—	—	—	—	—	4.36	0.80
B639-A	String-line	5.8	.2	—	—	4.8	.6	—	—	5.5	.2	5.22	4.25	12	3	15	4	16	6	14	1	4.42	0.80
B639-B	String-line	5.9	.3	—	—	4.9	.2	—	—	5.7	.2	5.35	4.50	11	2	15	4	16	2	—	—	4.42	0.93
B639-C	String-line	5.8	.2	—	—	4.8	.3	—	—	5.3	.3	5.19	4.38	13	3	15	4	14	3	13	1	4.36	0.83
Average	—	5.67	.3	4.99	.3	4.72	.3	5.03	.2	5.59	.3	5.12	4.30	11	1	19	3	17	3	—	—	4.46	0.66
Average S	—	—	.35	—	.33	—	.30	—	.30	—	.33	—	—	—	3	—	3	—	4	—	—		

*Average Depth = $1/8 \left[\bar{x}_{o\ell} + \bar{x}_{\ell} + 2 (\bar{x}_{\ell/4} + \bar{x}_{\ell/2} + \bar{x}_{3\ell/4}) \right]$

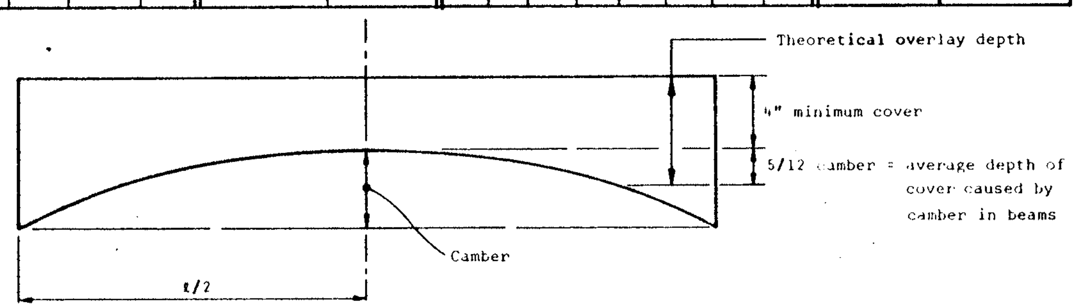
**Theoretical Depth = 4" + 5/12 [camber]

***String-line values for B602-B were adjusted for the vertical curve in the span.

1 inch = 2.54 cm

$\bar{x}$ = average

S = 1 standard deviation



PRECAST PARAPETS

General Observations

The parapet lends itself ideally to a systems concept as it has a constant shape suitable for mass duplication and there is sufficient demand for it statewide to allow economical production. Since forming for conventional cast-in-place concrete parapets can be a costly and time-consuming job, precast parapets were specified on all of the tee beam bridges built in Virginia. The parapets were precast upside down (see Figure 28) to eliminate the honeycombing typically associated with site-cast parapets. The standard 8-foot (2.4 m) long precast parapets were set in cement mortar spread on top of the exterior beam. The parapets may be anchored to the tee beam in several ways, but thus far all the contractors have chosen to make the connection with threaded metal rods which screw into inserts in the top of the tee and extend upward through voids cast into the parapet (see Figure 2). Nonshrink cement paste is used to grout the voids and anchor the parapet. With the aid of a light truck crane, three men can place and connect the 2-ton (1.8×10^3 kg) parapet sections on a three-span structure in 2 or 3 days (see Figure 29).

Alignment

Considerable effort on the part of the contractor was required to obtain an aesthetically pleasing alignment with the precast sections. Satisfactory alignment was difficult to achieve for the following reasons.

1. The tolerance to which the sections were precast caused slight dimensional differences.

2. The camber in the exterior tee beam produced an undesirable, curving surface upon which to place the precast sections.

3. The thickness of the overlay exceeded 4 inches (10.2 cm).

Most of the contractors used metal shims to support the precast sections. The shims were positioned before the mortar was spread on the exterior tee. After the shims were positioned, the parapets were removed with a crane and mortar was spread on top of the exterior beam and finished to the elevation of the shims. The parapet section was then lowered into the plastic mortar. At times it was necessary to raise the parapet and adjust the mortar bed several times to obtain the desired fit between the precast section and the mortar bed and the adjacent precast sections.

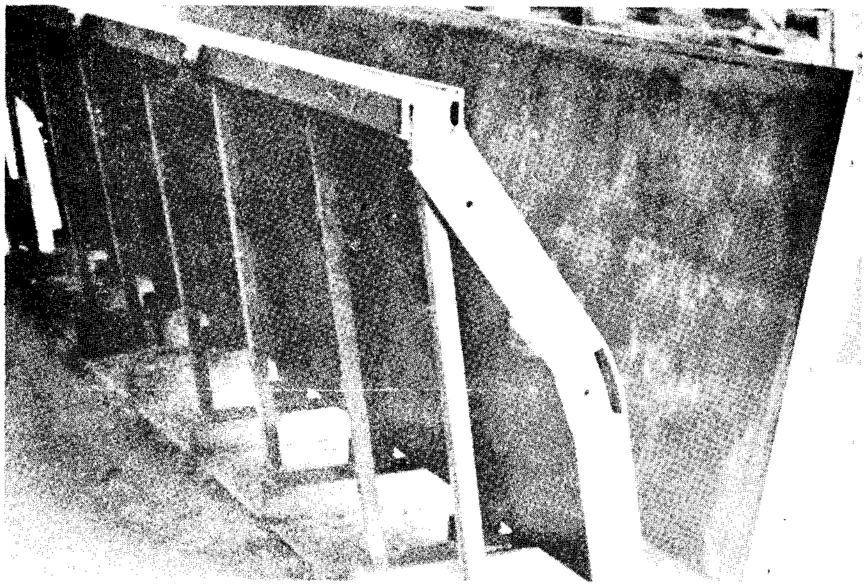


Figure 28. Metal form used to precast the parapets in inverted position.

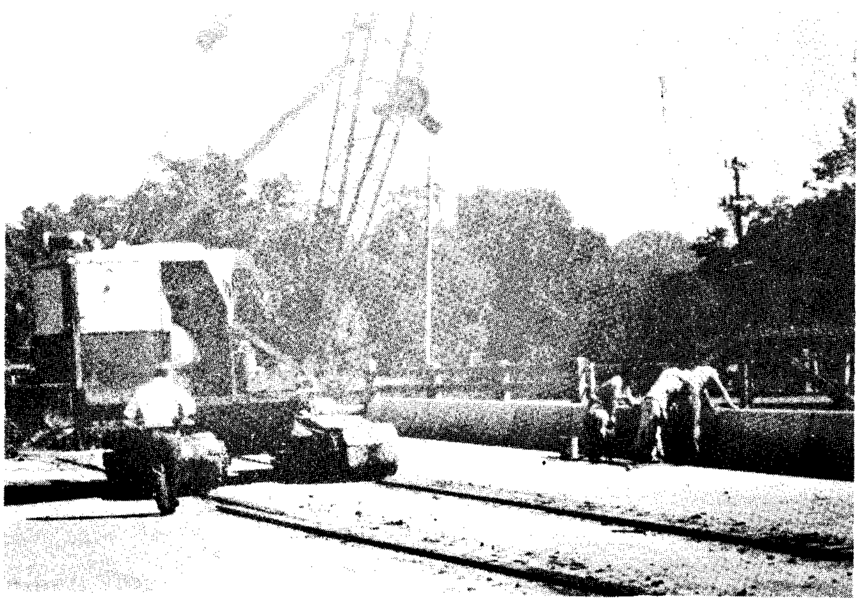


Figure 29. Precast parapet sections are adjusted for alignment on B604 in Norton.

Bond and Anchorage

Satisfactory bond between the precast parapet and the mortar bed was almost impossible to achieve (see Figure 30). Water draining from the roadway, under the parapets, and down the outside of the bridge in Figure 31 is evidence of the poor bond. The prolonged infiltration of water and deicing chemicals will certainly weaken the connection between the parapet and the exterior tee and produce an aesthetically unpleasing structure.

To provide the necessary structural anchorage between the parapet and the deck, nonshrink grout was poured around the threaded inserts extending through the parapets. A neat mix with enough water to obtain a workable consistency was specified on the plans but the parapets on B602 in Norton were placed with a mix consisting of one part sand and one part cement.

Several of the parapet sections on B602 in Norton developed structural cracks and began to spall in the vicinity of the threaded insert (see Figure 32). It is believed that the cement mortar was not properly mixed and therefore provided an absorbent passage through which water could infiltrate the parapet. Also temperatures dropped to about 25°F* for four consecutive nights after the cement mortar was placed, even though daytime temperatures were as high as about 50°F. Continued freezing and thawing of the water in the highly absorbent mortar likely caused the structural cracks and spalling in the parapets.

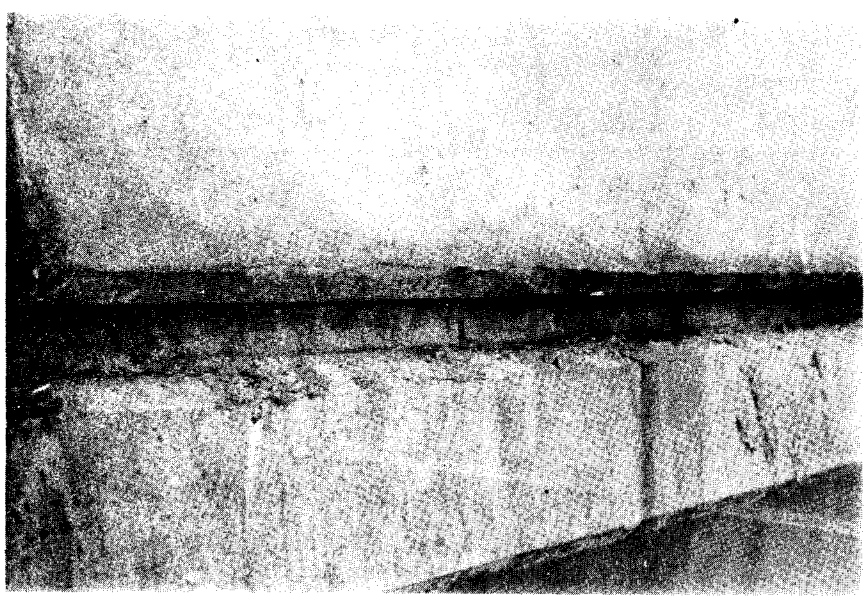


Figure 30. Bond between the precast parapet and mortar bed.

* °C = (°F -32) 5/9

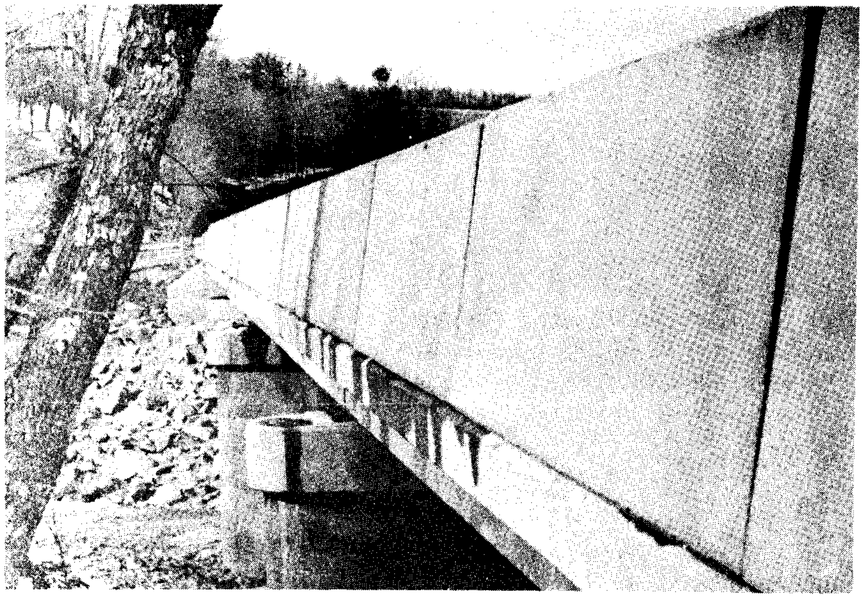


Figure 31. Water draining between precast parapet and exterior tee beam is evidence of poor bond.

Figure 32. Structural cracking and spalling of precast parapets in vicinity of mortar-filled void.

Quality Control

Since the cement mortar and cement paste used to anchor the parapets were mixed in small batches at the bridge site (see Figure 33) with little quality control, it is believed that the problem will continue to occur unless the contractor is required to submit a mix design for the mortar and paste and an inspector is present at all times to ensure compliance with the mix design. A portable mixer should be used to mix the mortar and hand mixing of the mortar in a wheelbarrow should not be allowed. Furthermore, nonshrink cements require more water for proper curing than do conventional cements. "There is no point in using shrinkage-compensating cement unless protection from early evaporation is provided within an hour after casting."(6) The plans for the Norton bridges did not require any type of curing for the paste-filled parapet voids. A specification must be prepared to ensure that the paste and mortar are properly batched, placed, and cured. Concrete for a site-cast parapet would be batched under strict quality control and therefore, the paste and mortar used to secure the parapet should receive the same amount of attention. For economy, it may be desirable to cap the voided area in the parapet with a waterproof epoxy and permit the use of conventional cement mortar for grouting around the threaded insert. Filling the entire void with a neat mix of nonshrink cement is not economical. Furthermore, absorption tests conducted at the Research Council have indicated that a neat Durcal mix is 400% - 700% more absorbent than the A5 concrete used in the parapet. Mixing one part sand to one part Durcal reduces the absorption to between 200% and 300% of the A5 concrete. The mortar-and paste-filled voids and the mortar bed are likely to control the durability of the precast parapets.

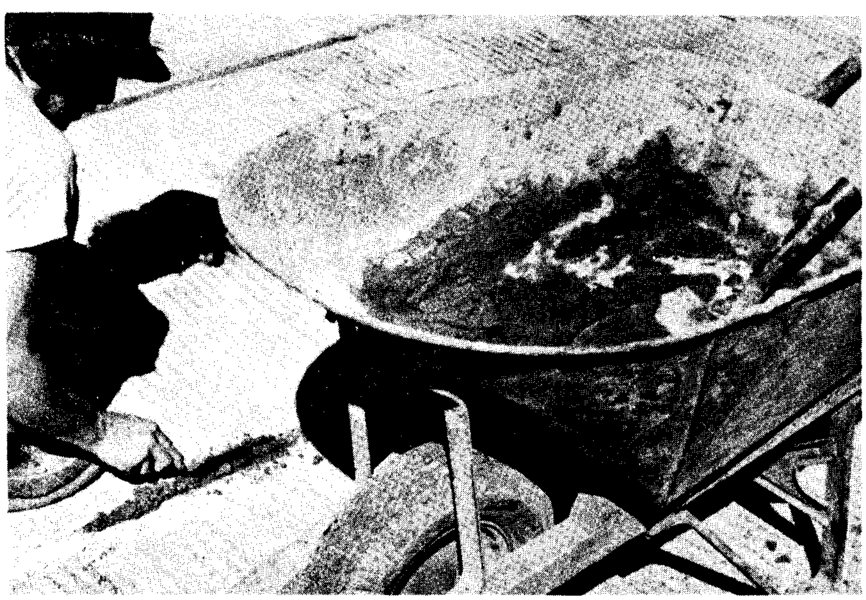


Figure 33. Nonshrink cement paste used to fill voids in precast parapets and the keyways between the tee beams was hand mixed at the bridge site.

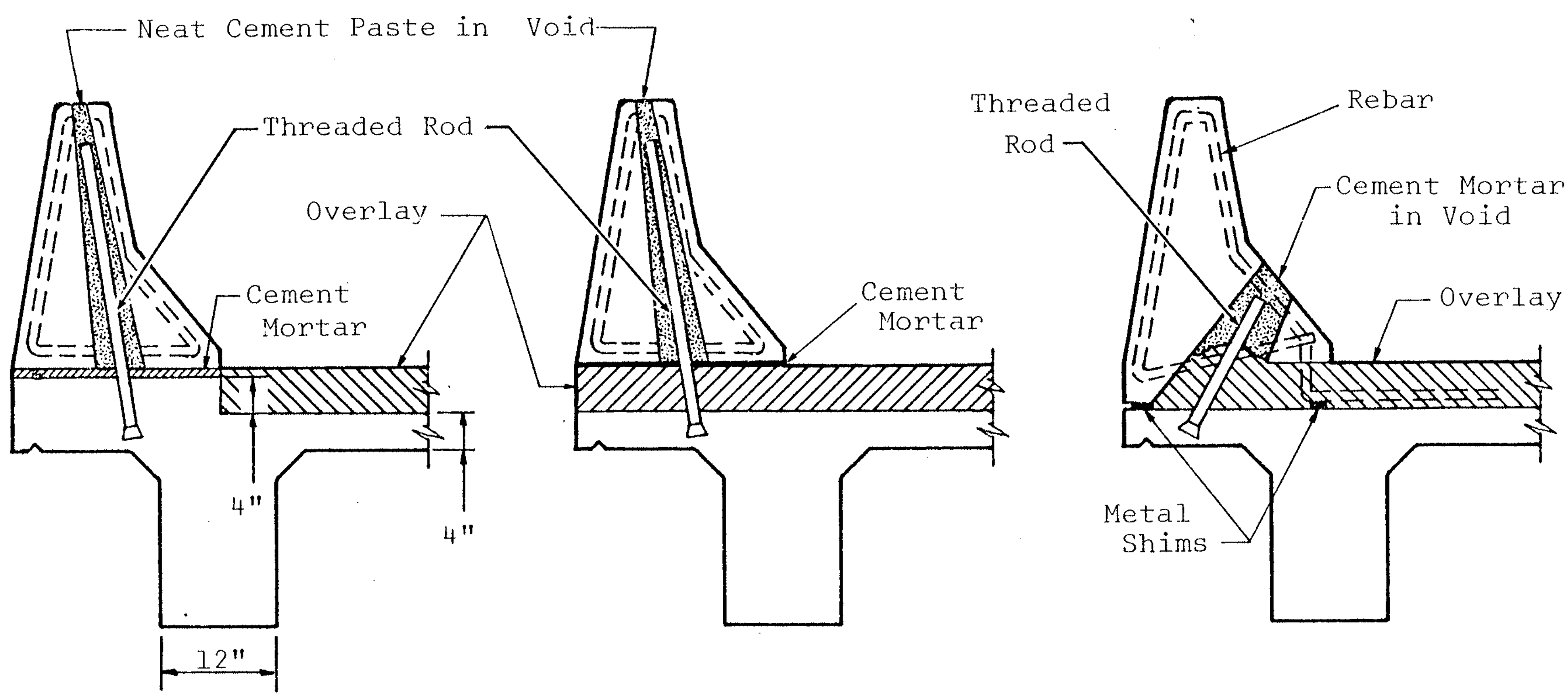
Design Changes

It is believed that the bond between the parapet and the mortar bed can be improved by designing the exterior tee beam so that the overlay covers it entirely as shown in plan B of Figure 34. The surface upon which the parapet is placed would have the same curvature as the bridge deck. If the contractor finishes the surface properly, it may be possible to use an epoxy or a thin layer of mortar to bond the parapets to the deck. The specifications should indicate that the bottom surface of the parapet sections should be roughened to improve the bond. A disadvantage of plan B is that the parapet cannot be placed until the concrete in the overlay has attained 85% - 100% of the design strength.

A bridge has been let to contract which has the exterior tee as shown in plan B. It will be necessary for the contractor to form the overlay, but this has also been necessary with the old exterior tee beam design as shown in plan A of Figure 34. With the new shaped exterior beam, a vertical cold joint is eliminated and the cost of the exterior tee is reduced.

Although the new exterior beam shape should improve the fit between the deck and the parapet, it is believed that the most successful detail will be similar to plan C of Figure 34. It is believed that with plan C 1) the parapet can be connected to the deck with conventional reinforcing steel and concrete, and a limited number of threaded inserts, and 2) forming for the overlay slab will be eliminated. Under plan C, the contractor can position the parapet one time with metal shims and then place the overlay around the parapet, an operation which is similar to that followed in conventional cast-in-place parapet and deck construction. When the overlay has obtained the desired strength, the contractor can fill the voids in the precast parapet.

Although plan B is a tremendous improvement over plan A, it is believed that the contractor will never achieve a satisfactory bond with either design because the parapet must be set in mortar. A satisfactory bond can be achieved only if the mortar or concrete is placed around a previously positioned parapet similar to that shown in plan C or the parapets are posttensioned to the exterior beam with a design similar to that shown in plan B. If possible, it would be desirable to eliminate the voids in the precast parapets. Further research is needed on the precast parapet.



A— ORIGINAL DESIGN

B— MODIFIED DESIGN

C— PROPOSED DESIGN

Figure 34. Exterior beam and parapet sections. (1 inch = 2.54 cm)

ATTACHING THE SOLEPLATES

Laboratory Investigation

A laboratory investigation was conducted to determine how the temperature due to field welding a soleplate to the metal insert cast into a prestressed single tee affects the elastomeric bearing pad, soleplate epoxy and grit coatings, and the concrete in the prestressed single tee.

A model of the bearing assembly was prepared in the laboratory at the Research Council in December 1975. A plywood form was constructed and thermocouple wires were secured at various locations throughout the form. Class A5 concrete made with Type III cement was mixed and placed in the form. A table vibrator provided consolidation. The model was steam cured for 21 hours at 150°F. Cylinder strengths of 4,100 psi (28.3 x 10^6 Pa) were obtained after 14 1/2 hours of steaming. Following the completion of the steam curing, additional thermocouple wires were attached to the model.

Twenty-eight days after the concrete was cast, the bearing assembly was placed in a hydraulic press and loaded to 11,000 lb. (5,000 kg) to simulate the dead load of a 42-foot (12.6 m) single tee beam and to provide lateral support during the welding process (see Figure 35). Copper-constantan thermocouple wires were connected to a Honeywell thermocouple recorder to provide data on temperatures between 0° and 225° F. Iron-constantan thermocouple wires were connected to a Thermo Electric Multimite instrument to measure temperatures up to 600°F. Tempilstik marks were placed on the soleplate to determine areas where temperatures exceeded 600° and 800° F.

The shielded metal-arc welding process was used to weld the soleplate to the metal insert. The electric arc, varying from 175-225 amps at 23 to 24 volts, and several 5/32-inch (4 mm) E6010 electrodes were used in making the weld. The welding was begun by preheating the weld area to approximately 150° F with an oxyacetylene flame. Following the preheating, three passes of the electrode were required to provide the required 3/8-inch (9.5 mm) groove weld. To fill the groove completely, a total of five passes were made on one side of the model with an electric arc varying from 170 to 200 amps. Four passes were made on the other side with an electric arc of 200 to 225 amps. The approximate time required for each 6-inch (15 cm) pass was 1 minute and the average cooling interval between passes was 2 minutes. The first side was welded between 3:00 and 3:15 p.m. on January 7, 1976, and the second side between 10:10 and 10:23 a.m. the following morning. Following the completion of the welding, test cylinders were broken which indicated that the concrete compressive strength at the time of the welding was 6,000 psi (41 x 10^6 Pa).

Results

The most significant and immediately apparent effects due to the welding were (1) the warping of the soleplate, (2) the delamination between the concrete and the metal insert plate directly above the weld (see Figure 36), (3) the discoloration of the concrete in the area of the weld, and (4) the loss of bond between the epoxy and grit coatings on some areas of the soleplate. An exaggerated version of the warpage is shown in Figure 37. A 1-inch (2.5 cm) thick soleplate should warp about half as much as the 0.75 inch (1.9 cm) thick soleplate.

The distribution of the maximum temperatures in x, y, and x', y planes through the model caused by welding is shown in Figure 38. Maximum temperatures ranged from 95° F at the bottom of the bearing pad to 400°F at the top. The maximum temperature reached by the epoxy and grit coatings on the bottom of the soleplate was 400° F. Temperatures in the area of fusion between the soleplate and metal insert probably reached 2,700° F.[2] The concrete in the immediate vicinity of the weld reached a temperature in excess of 600° F, but concrete approximately 8 inches (20 cm) from the weld area remained at room temperature. Because steel transmits heat better than concrete does, the maximum temperature in the concrete in the vicinity of the studs (x', y plane, Figure 38) was somewhat higher than in the concrete between the studs (x, y plane).

Figure 39 shows the distribution of the maximum temperature in the y', z' plane through the weld caused by preheat, one pass, three passes, and five passes. It is apparent that temperatures can be held to a minimum if the bearing assembly is allowed to cool to the preheat temperature before starting a pass.

Figure 40 shows an estimate of the time interval after welding for a point at a given distance from the weld to reach a maximum temperature. The curves are based on heat transmission solely through the specified material except for an initial 0.75-inch (1.9 cm) transmission through steel. Heat spreads through the steel much faster than through concrete or elastomer. For example, the temperature in the bottom of the bearing pad reached a maximum of 95° F approximately 18 minutes after the five welding passes had been completed. Approximately 18 minutes were required for concrete 7 inches (18 cm) from the weld area to reach a peak temperature. A point in the concrete or elastomer which is influenced by the rate of heat transmission in the steel can be expected to reach a maximum temperature at a time which falls within the areas bounded by the curves in Figures 40 for steel and for concrete or elastomer, respectively. For example, a point at the top of the bearing pad and 1.5 inch (3.8 cm) from the weld would reach a maximum temperature about 1 minute after welding was complete (see curve for steel), whereas a point at the bottom of the bearing pad and 1.5 inch (3.8 cm) from the weld would reach a maximum temperature about 18 minutes after welding was complete (see curve for bearing pad).

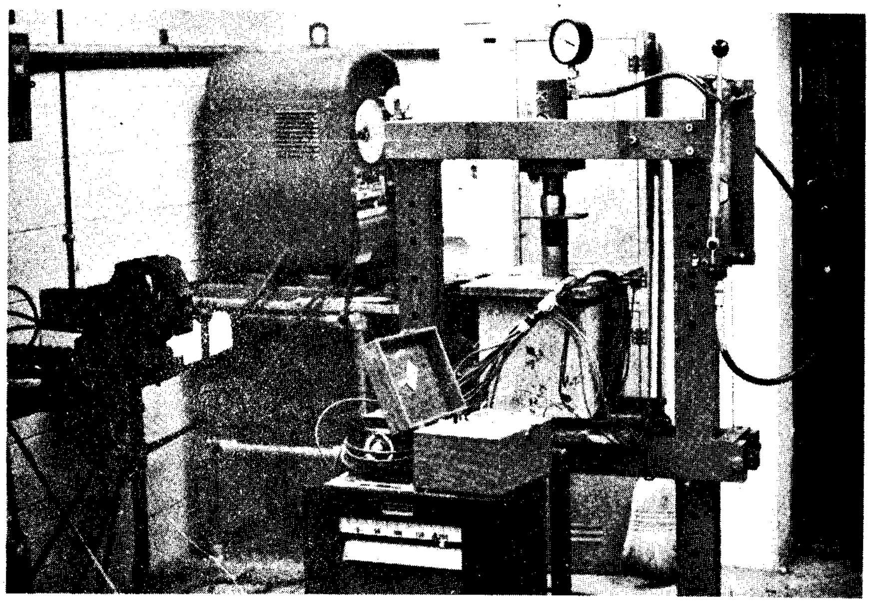


Figure 35. Model of bearing assembly prepared for welding.

Figure 36. Delamination between the concrete and metal insert and discoloration of concrete caused by welding.

3512

Soleplate thickness (inches)	3/4*	1**
ΔA (inches)	0.025	0.01
ΔB (inches)	0.01	0.00
ΔC (inches)	0.10	0.04
ΔD (inches)	0.01	>0.01

* actual values
** estimated

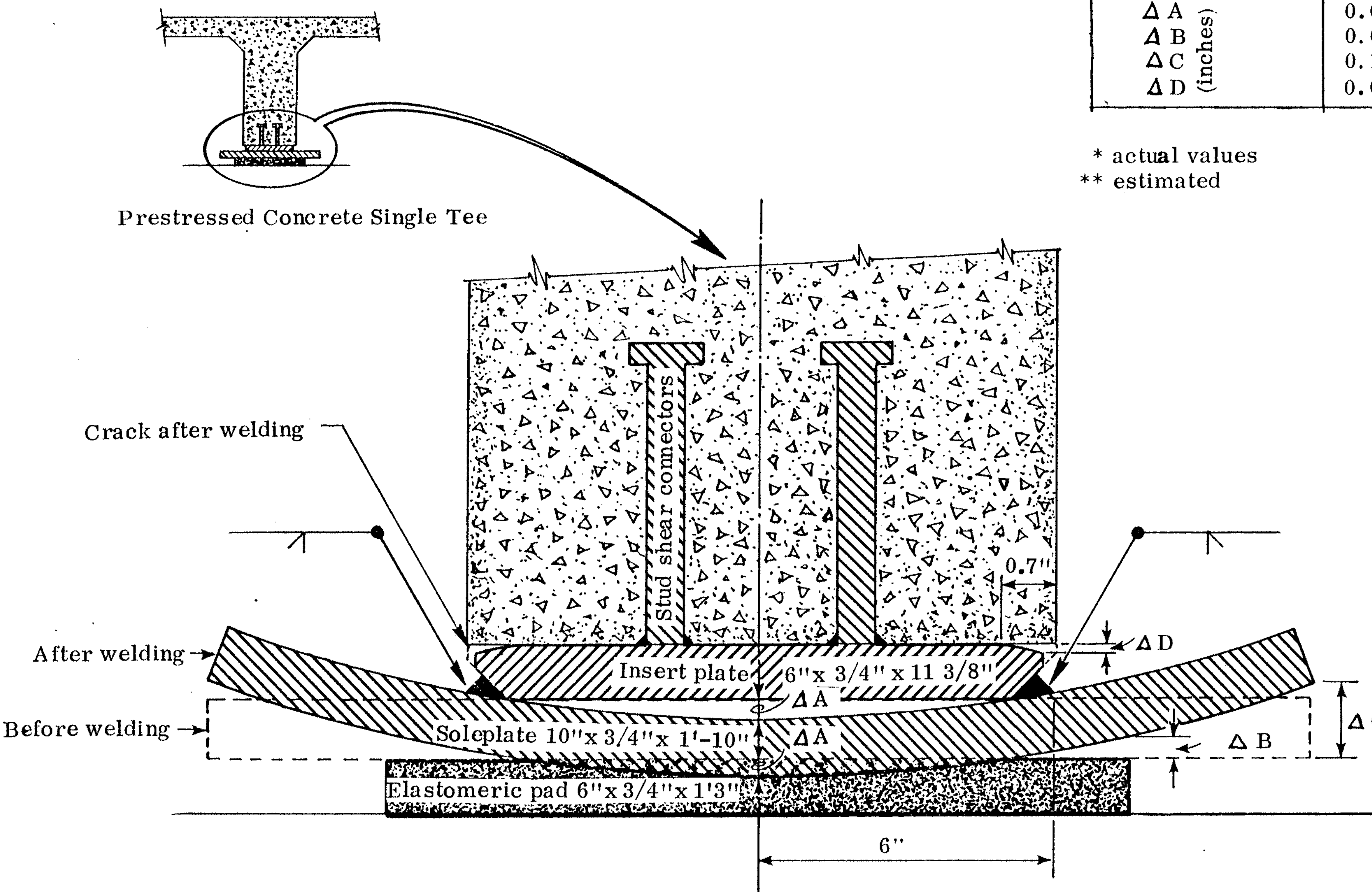


Figure 37. Distortion caused by welding (not to scale). (1 in. = 2.54 cm)

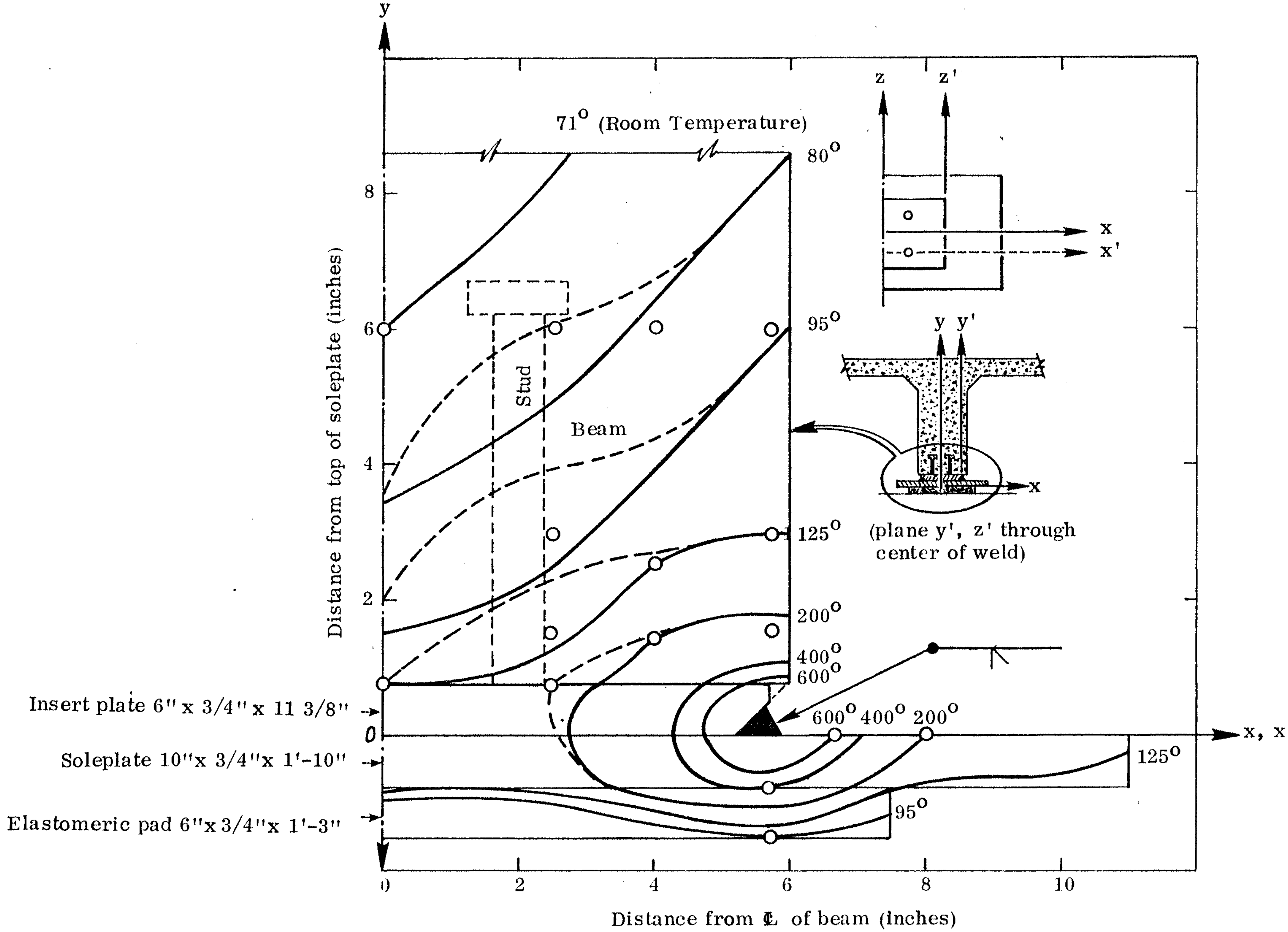


Figure 38. Distribution of maximum temperatures in x, y, and x', y planes through model due to welding. (1 in. = 2.54 cm)

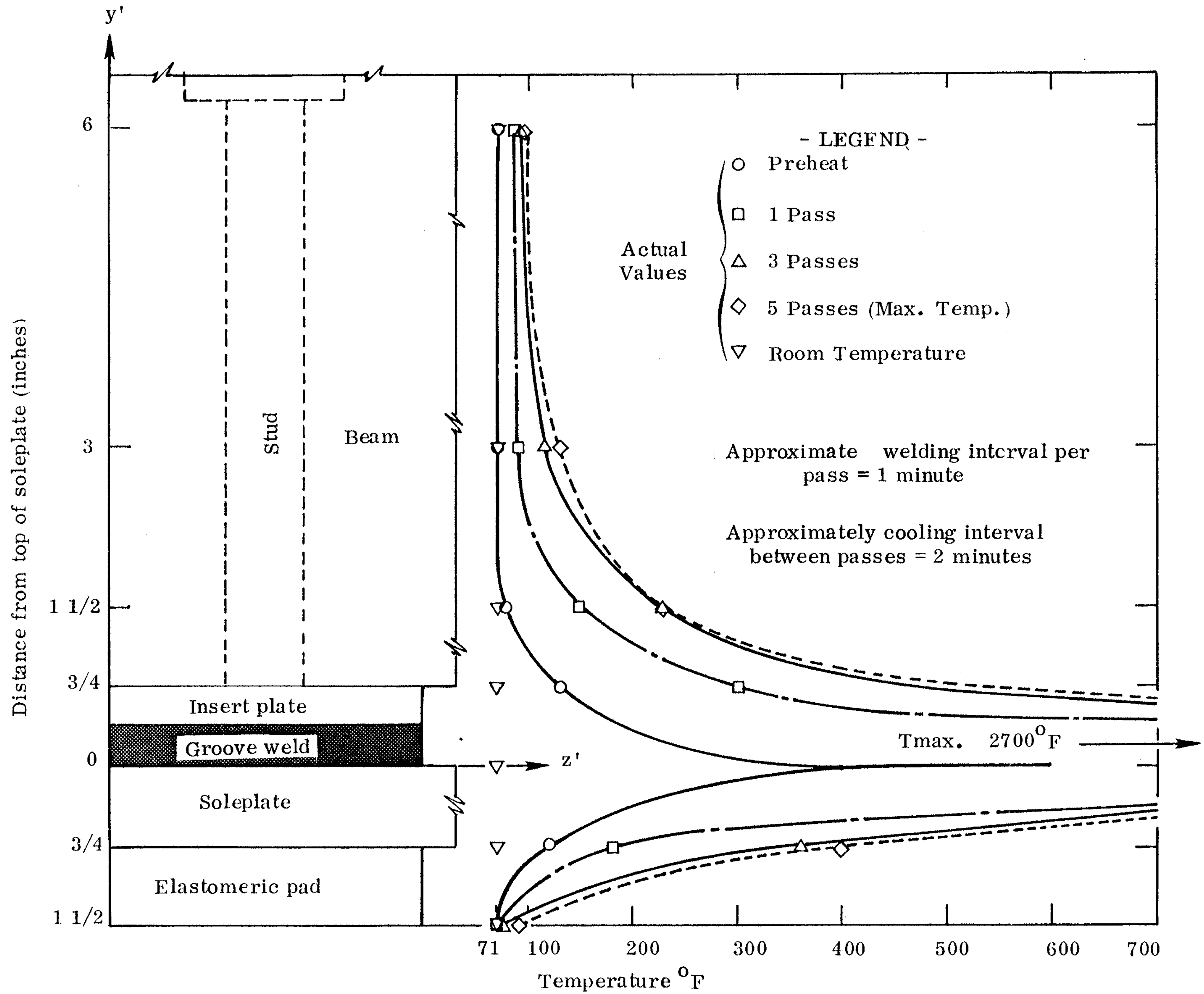


Figure 39. Maximum temperatures °F in y', z' plane(see Figure 37)through weld (1 inch = 2.54 cm)

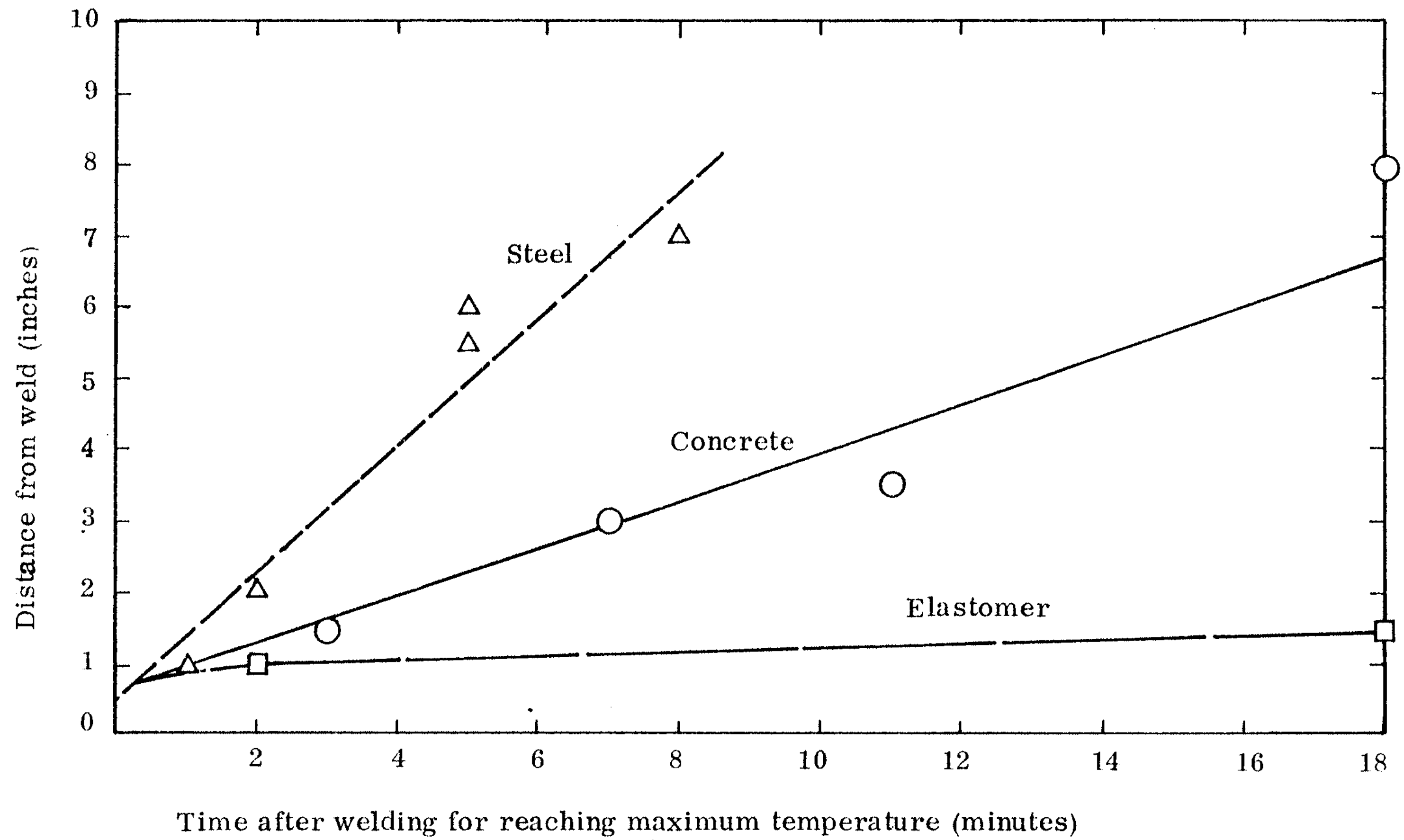


Figure 40. Time after welding vs. distance from weld for reaching maximum temperatures in specified material (heat conductivity through specified material with the exception of an initial 0.75 inch (1.9 cm) transmission through steel). (1 in. = 2.54 cm)

Discussion

The warped soleplate produced by this laboratory investigation will not comply with Section 411.08 of the Department's Road and Bridge Specifications, which requires "a uniform bearing over the whole area ".(4) The warpage can be decreased by decreasing the number of passes or by increasing the interpass temperature.(7) However, the use of higher interpass temperatures is not recommended since higher temperatures may prove harmful to the bearing assembly and concrete. Warpage also may be decreased by adequately bracing the parts or by using other than continuous welding techniques, each of which may be economically impractical. Warpage may be best improved or corrected in this situation by reducing the weld size and by increasing the thickness of the soleplate and insert plate, or by using an alternative bearing assembly.

The warpage of a 1 inch (2.5 cm) thick soleplate should be about one half as much as that of the 0.75 inch (1.9 cm) thick soleplate used in the laboratory model, but the delamination between the concrete and the top of the insert plate above the weld may increase when a 1-inch (2.5 cm) soleplate is used unless steps are taken to reduce warpage. The delamination provides a bad appearance and should allow corrosion of the metal insert plate. Where aesthetics is a matter of concern the discoloration of the concrete above the weld area can be eliminated by shielding the concrete from the welding. As can be seen from Figure 36, the application of duct tape to the concrete can prevent discoloration.

The loss of grit was confined to the area of the soleplate in contact with the bearing pad, but was not extensive enough to alter the behavior of the pad. With a 1-inch (2.5 cm) soleplate the loss of grit should be negligible. However, if temperatures on the bottom of the soleplate were to exceed 400° F, a significant breakdown in the bond between the epoxy and grit may occur.

All of the problems suggested by the laboratory investigation of the bearing assembly specified for the Norton bridges could be eliminated by removing the field weld, the soleplate, and the anchor bolts from the assembly. The metal insert could rest on the elastomeric pad and concrete steps could be cast into the tops of the abutments and pier cap beams to provide support in the transverse direction.(8) Unfortunately, AASHTO requires that a superstructure be anchored to the substructure if the bridge spans a body of water and Virginia uses anchor bolts in all structures to accommodate thermal and settlement movements. Countersunk thread bolts could be used instead of the weld to anchor the soleplate to the insert plate. Nevertheless, the warpage in the soleplate should not be detrimental to the performance of the elastomeric bearing pad. According to a recent report "bearing pads can be subjected to normal loading inclined up to 2 degrees without permanent damage occurring".(9) The angle of loading caused by the warpage of a 0.75 inch (1.9 cm) thick soleplate is 0.26 degrees. The angle of loading caused by camber in a 42 ft. (12.6 m) span is between 0.29 - 0.58 degree, which is greater than that caused by warpage but which is also less than 2 degrees.

Conclusions

The following conclusions are based on the assumption that the welding process, equipment, and procedures used in the field are comparable to those used in the laboratory investigations.

1. The elastomeric bearing pad will not be damaged by field welding with the pad and beam in place.

2. The epoxy and grit coatings on the bottom of the soleplate will not be significantly damaged, although some loss in bond may occur.

3. Delamination between the concrete and the top of the insert plate can be expected.

4. Discoloration of the concrete can be expected in the vicinity of the welding.

5. Slight warpage of the soleplate can be anticipated, and therefore, strictly speaking, it will not comply with Section 411.08 of the Department's Road and Bridge Specifications. However, for all practical purposes the bearing should be satisfactory if a groove weld is applied to both sides of the soleplate at approximately the same time the single-tee is braced to prevent asymmetrical warping, and the thickness of the soleplate is 1 inch (2.5 cm) or more.

6. The camber in the tee beams and the relative fit between the soleplate, insert plate, and bridge seat will usually have a greater effect on the uniformity of bearing than will the warpage caused by welding the soleplate to the insert plate. Neither the camber or the warpage should cause premature failure of the bearing pads.

7. Field welding the soleplate to the insert plate is preferable to plant welding, because a plant welded bearing assembly cannot be adjusted in the field to obtain the most desirable bearing.

STRUCTURAL BEHAVIOR OF THE TEE BEAM BRIDGE DECK

The tee beam bridge deck is different from the conventional bridge deck in the following aspects.

1. The bottom half of the deck is cast monolithically with the bridge stringers.
2. The bottom half of the deck is prestressed longitudinally.
3. Grouted keyways spaced at 4-foot (1.2 m) intervals connect the bottom precast sections of the deck.
4. The bottom layer of reinforcement is not continuous through the keyways.
5. The tee beams are spaced at 4-foot (1.2 m) intervals rather than at 7-or 8-foot (2.1 m - 2.4 m) intervals as is common with steel stringer concrete deck construction.
6. The top half of the deck is cast at the bridge site 2 months or more after the bottom portion has been precast at a fabricating plant.

Because of the uniqueness of the tee beam design, some limited laboratory studies were conducted to examine the structural behavior of the deck.

Laboratory Deck Models

Ten half-scale models [concrete slabs 1 foot x 4 feet x 4 inches (30.5 cm x 122 cm x 10.2 cm)] of a portion of the tee beam bridge deck were constructed in the laboratory at the Research Council. The base slabs [two slabs 1 foot x 2 feet x 2 inches (30 cm x 61 cm x 5.1 cm)] for each model were batched in groups of four. Different surface textures were applied to the base slabs and all the base slabs were steam cured for about 18 hours. Approximately 1 week after the base slabs of the deck were prepared, two base slabs were placed in each of five forms and the keyways were grouted with a neat mix of Durcal (w/c = 0.3). The keyways were moist cured for about 2 days, at which time a 2 inch (5.1 cm) thick overlay was applied. Welded wire fabric was used to reinforce eight of the base slabs and overlays required for the models. Two models were constructed without reinforcement. Metal studs were used to connect the overlay to the base slabs in one of the reinforced models, and the bottom reinforcement was extended through the keyway in another model. In addition to the 10 half-scale models, 2 full-depth control slab models were constructed. Reinforcement was used in the top and bottom of the control models. All 12 models were statically loaded in various ways to examine the structural integrity of the tee deck design with respect to a conventional full-depth deck design.

Flexure Tests Under Positive Moment Static Loading

The models were loaded at midspan as shown in Figure 41. In 9 of the 12 models failure was initiated between the Durcal keyway and one of the lower slabs, and propagated upward through the center of the keyway and eventually through the overlay. The 1 model containing the reinforcement extending through the keyway failed at two points, each coinciding with the termination of the overlay of the reinforcement through the keyway. Failure in each of the full-depth models was initiated at one or two points several inches to the left and/or right of midspan.

The average yield strength and ultimate strength for the 9 tee beam models were 47% and 36%, respectively, of the strength of the 2 full-depth models. The yield and ultimate strengths for the model containing the reinforcement through the keyway were 59% and 58%, respectively, of the strength of the 2 full-depth models. It is obvious that the tee deck design is much weaker in flexure produced by positive moment loading than is the conventional design, which would be expected as the tensile steel was not continuous through the keyway. A welded connection could be used to improve the flexural strength. However, it should be noted that the tee beam models did not fail at static loadings less than the AASHTO design load. Evidently there is more economy in spacing the tee beams on 4-foot (1.2 m) centers and allowing the 4-inch (10.2 cm) overlay to support the flexural wheel loads than there is in spacing steel stringers on 8-foot (2.4 m) centers and using continuous tensile reinforcement in the bottom portion of a full-depth slab. It should be noted that most of the keyway failure was through the Durcal rather than along the bond area between the edge of the tee and the Durcal. The function of the grouted keyway is to transmit shear loads for which no reinforcement is required. However, tensile stresses caused by thermal loadings will be concentrated in the overlay directly above the grouted keyway since the bottom reinforcement is not continuous through the keyway.

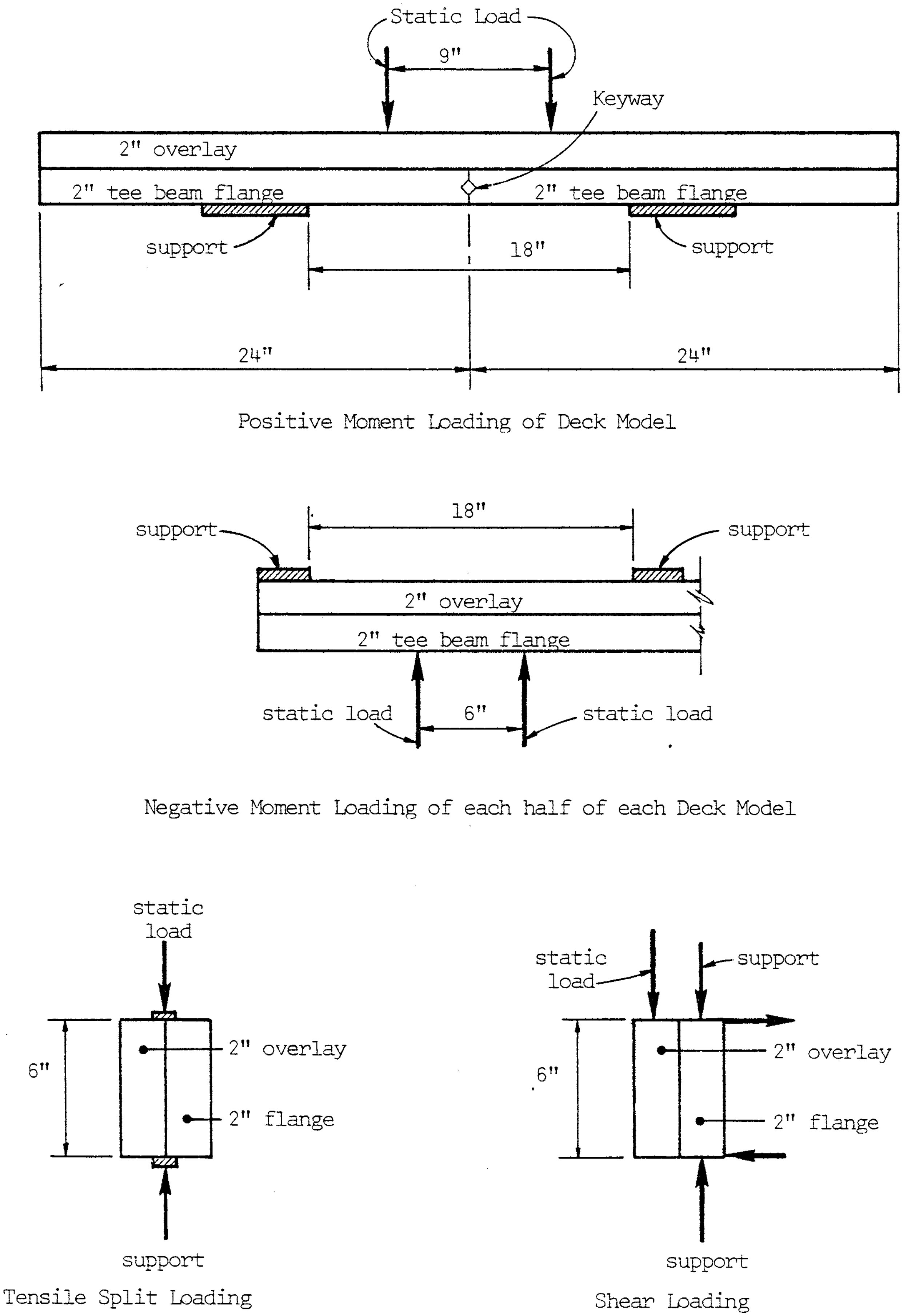


Figure 41. Geometry for static test loading of deck models.

Flexural Tests Under Negative Moment Loading

The 24 specimens remaining after completion of the load test for positive moment were load tested for flexural strength under a negative moment loading condition (see Figure 41). Since no keyways were subjected to tensile loadings under this loading condition, all 24 specimens yielded at about the same magnitude of loading and, disregarding the 2 unreinforced specimens, the ultimate strengths were about the same. It is safe to conclude that the design for negative bending moment in tee beam decks may be the same as that for conventional full-depth decks.

A metal trowel finish had been applied to the surface of the tee beams used in 2 of the 24 specimens. Partial bond failures between the overlay and the base slab accompanied the flexural failures of these 2 specimens. None of the other 22 specimens showed any signs of bond failure.

Shear and Tensile Tests for Bond Strength

Since only two cases of bond failure were noted when the 24 specimens were subjected to flexural loadings, 54 specimens 6 inches x 7 inches (15.2 cm x 17.8 cm) were cut from the deck model specimens and subjected to shear and tensile test loads (see Figure 41). The results of these tests are shown in Table 7. Some of the specimens subjected to the tensile and shear tests are shown in Figure 42. Note the irregular failures of the full-depth specimens (center of Figure 42) as compared to the planar and bond failures of the layered specimens.

From Table 7 one can conclude the following:

1. Concrete placed in two layers is weaker than concrete placed in one lift, when the bond is subjected to tensile and shear stresses which cause yielding.

2. Applying a cement slurry to the base slabs prior to placing the overlay appears to have no effect on bond strength.

3. The presence or absence of slab reinforcement appears to have little effect on shear and tensile tests for yielding.

4. A metal trowel finish leads to poor bond strength.

5. The grooved finish currently being applied to the surface of the tee beams provides for the best and most consistent bond strength that can

Table 7

Bond Strengths for Various Tee Beam Surface Conditions
(Percentage of Full-Depth Construction)

Tee Beam Surface Condition	Cement Slurry	Number Specimens	Tensile Test		Shear Test	
			Strength	Failure Mode[a]	Strength	Failure Mode[a]
Metal trowel finish	No	3	45	bond	23	bond
Metal trowel finish	Yes	2	37	bond	11	bond
Broom finish	No	3	53	bond	62	bond
Broom finish	Yes	2	53	bond	26	bond
Grooved finish, metal shear connectors	Yes	3	78	50% planar 50% irregular	80	50% planar 50% irregular
Longitudinally grooved finish(b)	No	5	43	bond	49	50% bond 50% irregular
Longitudinally grooved finish	Yes	5	53	70% bond 30% irregular	55	70% bond 30% irregular
Transversely grooved finish(c)	No	8	66	60% bond 20% irregular 20% planar	50	60% bond 20% irregular 20% planar
Transversely grooved finish	Yes	7	64	50% planar 50% irregular	55	50% planar 50% irregular
Transversely grooved finish (no rebar)	No	4	58	planar	51	50% planar 50% irregular
Transversely grooved finish (no rebar)	Yes	4	61	10% bond 90% planar	58	30% bond 70% planar
Full-depth slab deck	—	8	100	100% irregular	100	100% irregular

(a) Bond means failure along the bond interface, planar means failure in one plane, and irregular means random orientation of failure surface (rough surface).

(b) Load was applied parallel to the direction of the grooves.

(c) Load was applied perpendicular to the direction of the grooves. The transversely grooved finish is currently used.

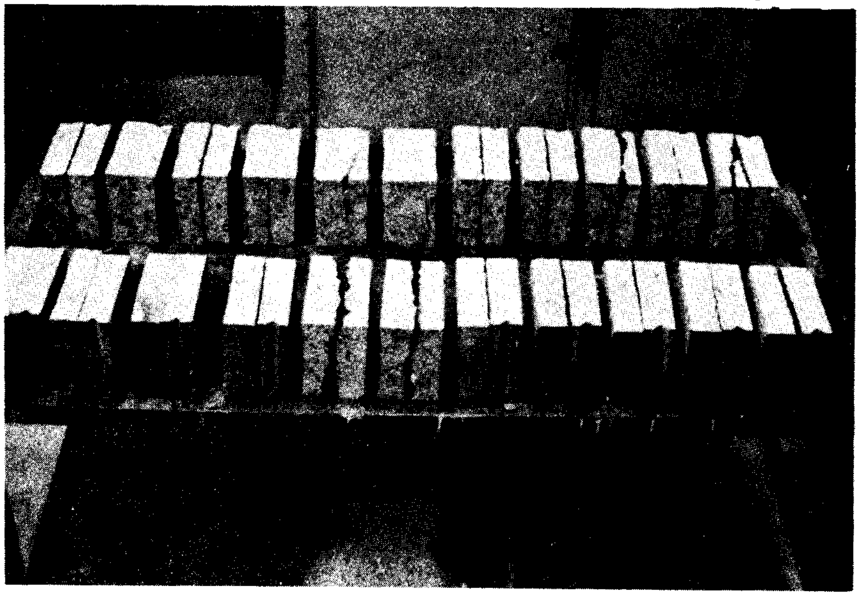


Figure 42. Some of the specimens subjected to shear and tensile tests loadings. Full-depth specimens are in center.

be achieved without adding metal shear connectors. However, this performance is for loads applied perpendicular to the direction of the grooves. The grooves are not as effective for loads applied parallel to this direction, probably because less keying action is developed. Since it would not be practical to groove the beams in more than one direction, it it believed that the transverse direction currently used is the best.

6. The greatest bond strength was obtained with the specimen containing the metal shear connectors, but only three specimens contained the connectors.

7. The bond strength of a full-depth slab is a function of the strength of the aggregate, the strength of the cement paste, and the bond between the paste and aggregate. The bond strength between a bottom slab and an overlay is a function of the strengths of the slab and the overlay and the interlocking strength between the two. For the mix design used

for the laboratory models, it appears that the first 33% of the full-depth bond strength is developed by properly preparing the surfaces upon which the overlays are cast, another 33% is developed by increasing the effective area of contact between the overlay and the bottom slabs, and the final 33% can be achieved only by means of mechanical shear connectors such as metal studs.

8. The failure mode data reveal that increases in bond strength may be associated with increases in the area of the surface over which failure occurs.

Discussion

The flexural tests revealed that the bond strength developed between the bottom slabs and the overlay is sufficient to maintain a flexural integrity equivalent to that of full-depth construction, when the bottom slab is properly cleaned and dampened before the overlay is placed. The tensile and shear tests revealed that bond strength between the bottom slab and overlay of sufficient magnitude to withstand the same tensile and shear stresses that can be handled by a full depth-slab can not be achieved, even by increasing the effective contact area and by adding shear connectors between the overlay and the base slabs. Assuming that fatigue and thermal loadings do not lead to bond failure, it can be concluded that for practical purposes adequate bond strength is developed by properly cleaning and wetting the surface of the bottom slab. Additional protection against bond failure is achieved by increasing the effective contact area and by adding metal shear connectors. Although the data seem to indicate that for extreme loading conditions the metal shear connectors could offer a marginal improvement in bond strength, it is questionable that the added cost can be justified on the basis of the improvement in bond strength.

It is apparent from the laboratory tests that a cold joint is a plane of weakness, but there is no evidence that the weakness will be of sufficient magnitude to control the durability of the bridge deck if the surface of the bottom slabs are properly cleaned and roughened. If bond failures occur in the tee beam bridges they will most likely be along the perimeter of the slab or near the grouted keyways where shear and tensile stresses are the largest. The chances for bond failure in the vicinity of the parapet would be decreased if the overlay was constructed continuously over the width of the exterior tee beam. Further research is needed to determine if the grouted keyway provides an adequate connection when the deck is subjected to thermal and fatigue loadings.

A rough surface has been applied to the tops of all the tee beams fabricated to date. The surface of the tee beams to be used on all bridges let to contract after September 1976 will be sandblasted to remove dirt and laitance Since it has been observed that a contractor will use the tee beam subdeck for the storage of equipment and materials, it is essential that the Department continue to require the sandblasting of the tee beam surfaces prior to the placement of the reinforcement for the overlay.

It is interesting to note that of the 12 cores removed from B604 and B602 in Norton 3 failed at the bond area; 3 failed through the reinforcing steel in the overlay, which is positioned about 1 inch (2.5 cm) above the bond area; and 6 were removed in one piece. Petrographic examinations revealed that there was no microcracking in the bond area of 6 of the 12 cores. One of the full-depth cores was taken through a grouted keyway and showed fair bond between the nonshrink cement mortar and the tee beams (see Figure 43). By sandblasting the tee beams there should be fewer bond failures when cores are taken.

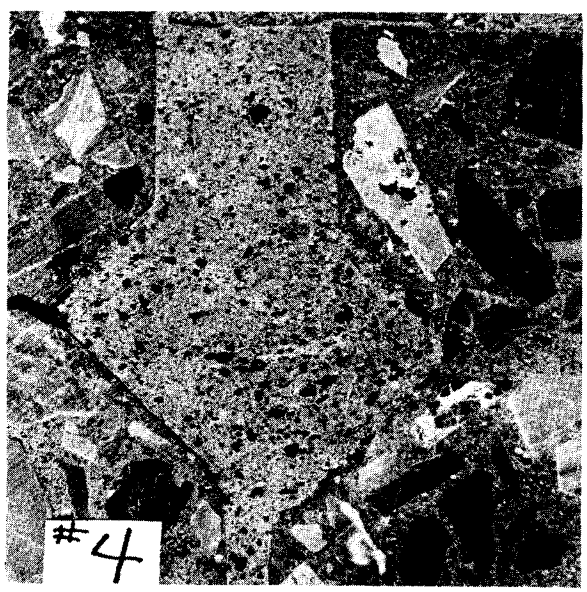


Figure 43. Part of full-depth core removed from span A of B602 in Norton shows mortar filled keyway between adjacent tee beams.

INITIAL EVALUATION OF THE SUPERSTRUCTURE CONCRETE

Mix Designs

As is shown in Table 8, seven concrete, paste, and mortar mixes were used in the construction of the single-tee bridge superstructures located in Norton.

Preliminary evaluations of the concretes used in the superstructures were based on a) field observations and data collected at the time the concretes were placed; b) laboratory tests of 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical compression test specimens and 3 inch x 4 inch x 16 inch (7.6 cm x 10.2 cm x 40.6 cm) prismatic test specimens made from random samples of the concretes; c) petrographic examinations of cores removed from B602 and B604; and d) petrographic examinations, cement content determinations, and chloride content determinations of portions of selected 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical specimens made from random samples of the concretes. Additional research on superplasticized concrete is planned, so a detailed discussion of some of the data collected in Norton will be presented at a later date. Mix design data for the superstructure concrete are shown in Table 9.

Table 8

Concrete, Paste, and Mortar Mixes
Used in Norton Superstructures

Concrete Mix	Superstructure Element
Class A5 portland cement concrete	Prestressed, precast single-tee beams, and precast parapets
Class A4 portland cement concrete	Diaphragms and overlay, B604
Class A4 portland cement concrete with superplasticizer	Overlay, B602
Class A4 portland cement concrete with superplasticizer	Overlay, B603
Neat water and Durcal gypsum cement paste	Keyways and parapet voids, B604
Durcal gypsum cement mortar	Keyways and parapet voids, B602
Portland cement mortar	Parapet-to-beam connection

Table 9

Mix Design Data

Superstructure Element	Type Concrete	Cement Content		Aggregate Content		Maximum Water Content		Maximum w/c
		bags/yd.^{3}	lb./yd.^{3}	Coarse	Fine	gal./yd.^{3}	lb./yd.^{3}	
Precast, prestressed tee beams	Class A5	7.5	705	1976	992	36	300	0.43
Diaphragms and overlay, B604	Class A4	6.75	635	1792	1136[a]	33.7	281	0.44
Overlay, B602	Class A4 with superplasticizer	6.75	635	1838	1256[a]	26.2	218	0.34
Overlay, B603	Class A4 with superplasticizer	6.75	635	1746	1350[a]	26.2	218	0.34

a) Becker Sand Cherau, S. C. F. M. = 2.8

Conversion Factors:

1 bag/yd.^{3} = 1.37 bags/m^3

1 lb./yd.^{3} = 0.593 kg/m^3

1 lb. = 0.45 kg.

1 gal./yd.^{3} = 5 l/m^3

Prior to construction of the tee beam bridges, a very limited amount of research had been done at the Research Council with the shrinkage compensating cement(10) used to grout the keyways and anchor the parapets. However, the emphasis of this work was directed to pavement repairs. Based on observations made during construction of the bridges and because of the high cost of a neat cement paste, it is believed that research needs to be directed toward determining the most satisfactory way to connect the parapets to the tee beams and to connect the flanges between adjacent tee beams.

Portland cement mortar containing 2 1/2 parts fine aggregate and 3% to 7% air was used to bond the parapets to the exterior tee beams. Neat Durcal cement pastes and Durcal cement mortars were used to grout the keyways and to fill the voids in the parapets. Water content data are not available for any of the pastes and mortars since current specifications do not require the water content to be measured. Water is added to obtain the desired consistency. The Durcal cement mortar was hand mixed at the bridge site, therefore the fine aggregate content was estimated to be equal to the cement content. No air entraining agent was used in the Durcal cement paste and mortar. The strength of the cement pastes and mortars is influenced by the mix design, mixing procedures, degree of consolidation, and curing procedures. It is believed that research should be conducted to develop an appropriate specification to cover batching, mixing, placing, and curing the pastes and mortars used to connect the precast members. The members are fabricated under strict quality control conditions and it is felt that they should be connected with strict quality control. Mix designs for mortar and paste should be submitted for approval, and an inspector should be present to ensure that the paste and mortar are batched, mixed, placed, and cured in accordance with specifications. The hand mixing of a mortar without a portable mixer should be prohibited. The mortars and pastes placed in the Norton bridges should not be used in any evaluation of the performance of tee beam structures since they were batched, mixed, placed, and cured with little quality control. A recent ACI publication indicates that nonshrink cements should be covered to prevent loss of moisture within one hour after they are placed, otherwise shrinkage comparable to that of portland cement concrete will occur.(6) Hand mixing a neat paste with a shovel and wheelbarrow may be satisfactory, but the water/cement ratio should be monitored by the inspector and retempering the paste should be prohibited. For adequate frost resistance the entrained air content of the pastes and mortars should exceed the 3% - 7% specified in section 220.03 of the Department's Road and Bridge Specifications.

Concrete Properties

As required by Department specifications, the fresh concrete was checked for air content and consistency prior to being placed. The ASTM C231 pressure method(11) was used to measure the air contents of all the study concretes shown in Table 10. The consistency of the concrete was determined in accordance with ASTM C143. The characteristics of the superplasticized concretes changed as the concrete was placed, therefore air content and consistency measurements were made several times as a batch was discharged. Also, at times no measurements were made on the superplasticized concrete since delays caused by checking the concrete properties inhibited the contractor's operations. In addition to the air content and consistency measurements, the unit weight of the concrete was determined using a unit weight bucket or the air meter bucket. The measured properties are shown in Table 10.

It is immediately apparent from Table 10 that a significant difference between superplasticized concrete and conventional concrete is the variability in the properties of the plastic concrete. The magnitudes of the standard deviations for the superplasticized concrete are from 1.5 to 4.5 times greater than for the conventional concretes. It is believed that the decrease in the magnitude of the standard deviations for the concrete used on B603, as compared to that on the earlier built B602, resulted from the increased fines content of the B603 mix and the experience in handling the superplasticized concrete gained from placing the concrete on B602.

All of the conventional concrete conformed to Department specifications whereas some of the superplasticized concrete did not, even though it was incorporated into the decks. The large variability in the measured properties of the superplasticized concrete was caused by the rapid change in the consistency of the concrete and the retempering efforts to achieve a more uniform consistency. It was common for the slump to decrease by 50% in 10 to 20 minutes. An effort was made to use high slump concretes on B602 because of the rapid loss in slump. Segregation occurred when the slump exceeded 8 inches (20.3 cm) for the mix design used on B602 (see Figure 44). Adequate consolidation was difficult to achieve when the superplasticized concrete was placed with a slump of 4 inches (10.2 cm) or less (see Figure 45), and over consolidation could have occurred when an internal vibrator was used to consolidate the high slump concrete (see Figure 46). It is believed that excessive bleed water (see Figure 47) may be an indication that over consolidation has occurred. A satisfactory screed finish was almost impossible to achieve because of the variability in the consistency of the concrete (see Figures 48 and 49).

Table 10

Properties of the Plastic Concrete

Type Concrete	Air Content, %		Slump, in.		Measured Unit Weight, pcf		Calculated Unit Weight, pcf
	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	
Class A5 concrete, tee beams	4.4	0.95	3.3	0.55	149.4	1.45	147.2
Class A4 concrete, overlay B604	6.8	0.85	2.8	0.55	145.2	1.46	142.4
Class A4 concrete superplasticized, overlay B602	5.4	1.9	5.7	2.8	146.9	4.40	146.2
Class A4 concrete superplasticized, overlay B603	6.1	1.4	4.8	2.3	—	—	146.3

Conversion Factors:

1 inch = 2.54 cm

1 pcf = 16.02 kg/m^3

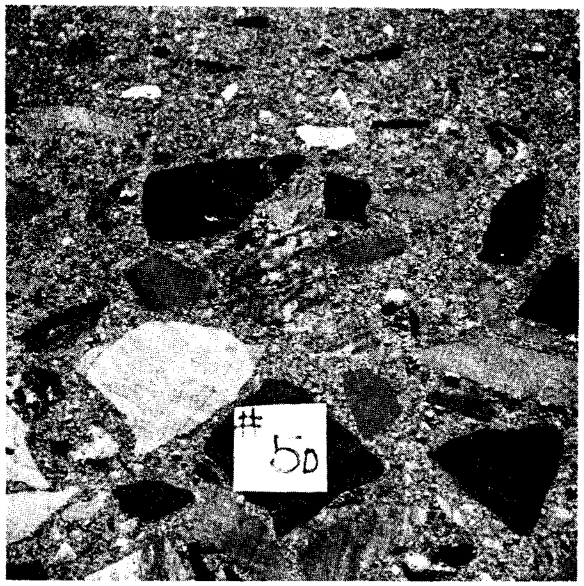


Figure 44. Core from span A of B602 shows **segregation of fluid** concrete.

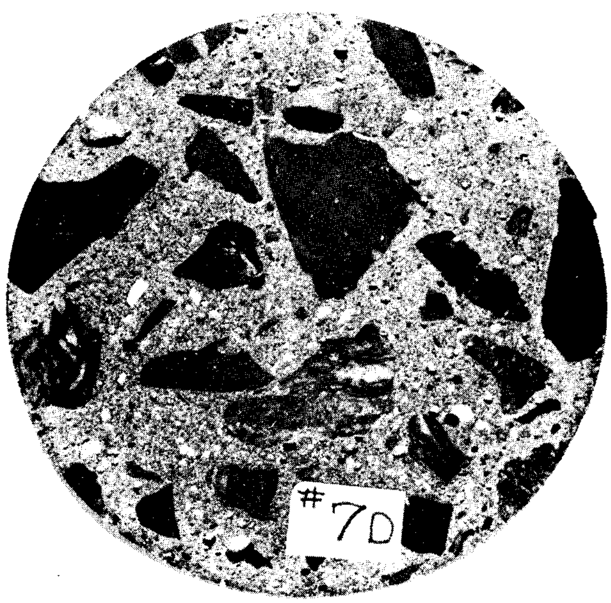


Figure 45. Core from span C of B602 shows **large voids caused by** poor consolidation of the **low slump concrete.**

Figure 46. Core from span B of B602 shows that much of the cement mortar and air has been lost.

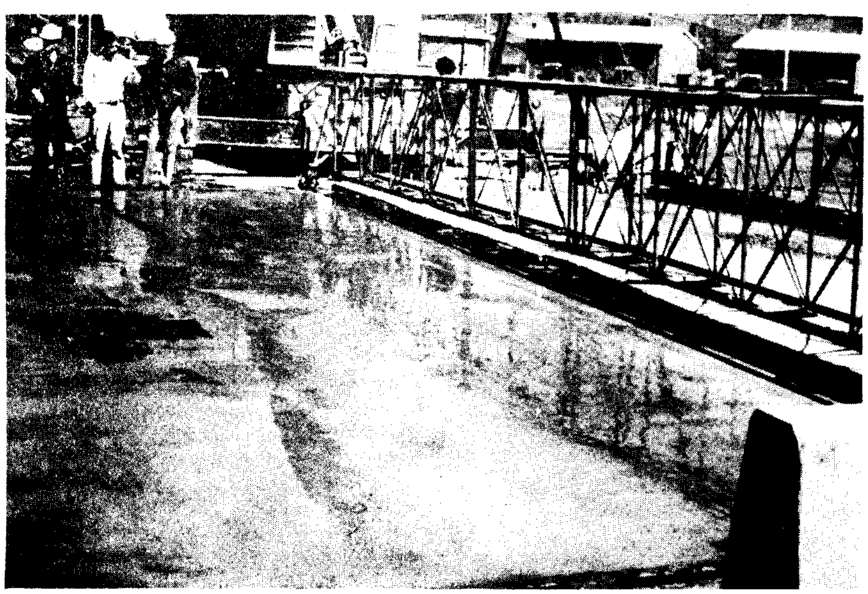


Figure 47. Excess bleed water covers most of span C of B603 as screeding and finishing operations are completed.

Figure 48. Variable consistency concrete pulls in lower part of figure and flows in upper part.

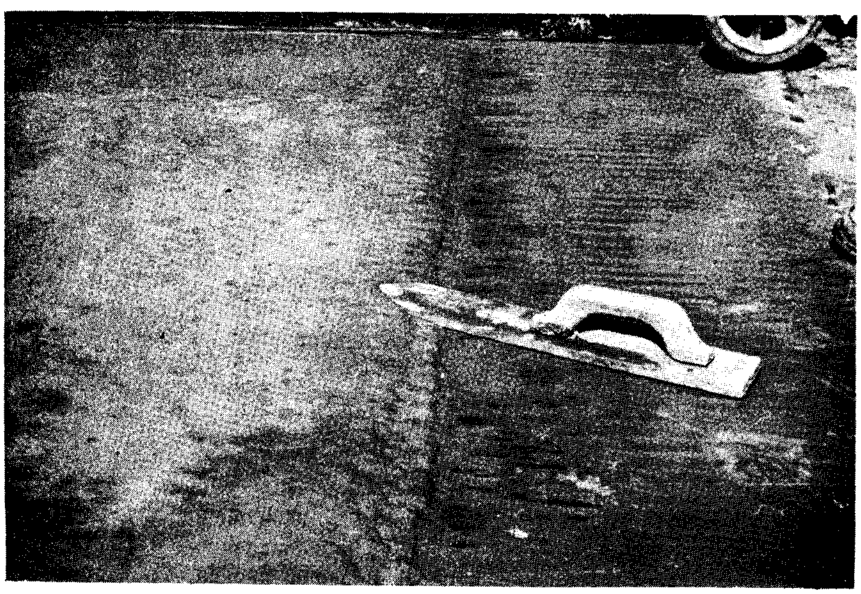


Figure 49. Fluid concrete continues to flow after fourth pass of screed on span A of B602. The deck slopes about 2% to the right and to the bottom of the figure.

Cylinder Strengths

Standard 6 inch x 12 inch (15.2 cm x 30.5 cm) compression test specimens were made from random samples of concrete used in the tee beams and the overlays constructed in Norton. The results of compression tests (ASTM C39) made on the moist cured cylinders (3-day cylinders were field cured) are reported in Table 11. No compression test data are available for the pastes and mortars used in the bridges. In most instances the cylinder strength of the concrete used in the tee beams exceeded 4,000 psi (27.6 x 10^6 Pa) with less than 24 hours of curing. For the overlay concretes, 28-day strengths significantly exceeded the 4,000 psi (27.6 x 10^6 Pa) design requirements, with the highest early and 28-day strengths being achieved with the superplasticized concrete. The variation in strength between cylinders was greater for the superplasticized concrete. Because of the high early strength achieved with the superplasticized concrete, it would be possible to open a bridge to traffic in less than 1 week after an overlay is placed.

Table 11

Cylinder Strengths, psi

Type Concrete	1 Day		3 Day (a)		14 Day		28 Day		Design
	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	
Class A5 concrete, tee beams	4660	338					5974	257	5000
Class A4 concrete, overlay, B604			2530	170	4210	156	5980	274	4000
Class A 4 concrete, superplasticized, overlay, B602			3455	163	5875	656	7000	829	4000
Class A4 concrete, superplasticized, overlay, B603			2229	657	8067	170	8891	401	4000

(a) Field cured specimens.

1 psi = 6.895 kPa

Freezing and Thawing Tests

Standard 3 inch x 4 inch x 16 inch (7.6cm x 10.2cm x 40.6cm) freeze-thaw test beams were made from random sanples of the concrete in the tee beams and the overlays. The condition of the field cured beams after being subjected to 300 cycles of freezing and thawing in accordance with ASTM C666 Procedure A, modified by using 2% $NaCl_2$ by weight in the water, are shown in Table 12. Prior to testing, the beams were field cured for 1 - 6 months rather than being moist cured for 14 days and laboratory cured for 7 days. The freeze-thaw specimens are evaluated periodically throughout the 300 cycle test with respect to surface appearance, weight loss, and the durability factor. Prior experience at the Research Council suggests that a surface rating less than three and a weight loss less than 7% are indications of good performance for 300 cycles. Low durability factors are an indication of internal cracking and values above 70 are an indication of satisfactory performance for 300 cycles.

From Table 12 it is apparent that, on the average, all of the field specimens performed satisfactorily with respect to surface rating and weight loss. The segregated superplasticized concrete specimens scaled severely, but the other superplasticized specimens performed better than the conventional concretes with respect to scaling and weight loss. The performance of the Class A5 concrete was as would be expected, since freeze-thaw durability is not a major consideration in designing the mix. The durability factors were the lowest for the superplasticized concrete, but an average value above 70 was obtained by eliminating the values for the segregated specimens from the data. Low durability factors for superplasticized concrete have been reported by others.(12) It is felt that the results reported in Table 12 are representative of the anticipated performance of the concretes in the study structures. Elimination of the values for the segregated specimens from the data indicates the anticipated performance, if slumps are less than 8 inches (20.3 cm) for the superplasticized concrete as designed for B602 in Norton. Additional freezing and thawing tests of superplasticized concrete are being conducted and will be reported as part of another study.

Table 12

Freezing and Thawing Performance, 300 cycles, For Field Cured Beams

Type Concrete	Number of Specimens	Surface Rating		Weight Loss		Durability Factor	
		Avg.	Std.Dev.	Avg.	Std.Dev.	Avg.	Std.Dev.
Class A5 concrete, tee beams	9	2.8	0.33	3.66	1.53	88	5.4
Class A4 concrete, overlay B604	6	1.7	0.33	1.32	0.39	86	4.4
Class A4 concrete, overlay B639(Floyd)	4	2.5	0.24	3.2	0.94	90	14
Class A 4 concrete superplasticized, overlay B602 (a)	10	2.1	1.1	1.76	1.81	68	29
Class A4 concrete superplasticized, overlay B602 (b)	8	1.6	0.47	0.99	0.71	80	8.3

(a) Test results of 10 specimens from spans A and B.

(b) Test results after eliminating two segregated specimens.

Petrographic Examinations

Petrographic examinations were conducted to determine the quantity, size, and spacing, of voids in 4-inch (10.2 cm) diameter cores removed from B604 and B602 in Norton and in 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical specimens made from random samples of the study concretes. The voids data are shown in Table 13.

Research has shown that concrete which is properly batched and consolidated will exhibit a void content in the hardened concrete which is approximately equal to the air content of the fresh concrete. When the void contents of 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical test specimens are higher than the measured air contents of the fresh concrete the difference may be attributed to water voids. When the void contents of cores removed from a structure are different from the void contents of 6 inch x 12 inch (15.2 cm x 30.5 cm) cylinderical test specimens the difference may be attributed to the degree of consolidation. Close agreement between the void contents of cores and the measured air contents of the fresh concrete is often an indication of adequate consolidation and a mimimum of water voids. However, over consolidation and high water content may also produce hardened concrete with a void content equal to the measured air content. Water voids and entrapped voids are >1mm in diameter and entrained air voids are ≤1mm in diameter. The size of the voids as well as the void content must be known before an accurate analysis of the concrete can be made.

From Table 13 it can be seen that there is close agreement between the measured air contents and the void contents of the 6 inch x 12 inch (15.2 cm x 30.5 cm) specimens made for the tee beam concrete and the conventional overlay concrete. The higher void content in the cores taken from the tee beams indicates that the consolidation of the concrete in the flanges of the tee beams was not as good as the consolidation in the specimens, which can be assumed to reflect excellent consolidation. The high variability in the data for the superplasticized concrete is again reflected in Table 13, which shows that although the average void content of the specimens was much greater than the average measured air content, the values agreed within one standard deviation. The high void content of the superplasticized specimens indicates that either the air content measurements were low or else there were a lot of fluid voids in the specimens. The low voids content of the cores taken from the superplasticized overlay as compared to the specimens indicates that over consolidation may have occurred. There is good agreement between the void contents of the cores and cylinders for voids >1mm, but the voids <1mm appear to have been lost from the superstructure concrete through either the agitating action of the superplasticizer or over vibration. The voids smaller than 1 mm are needed for freeze-thaw durability.

Air Void Spacing Factor

An air void spacing factor ($\bar{L}$) of 0.008 inch (0.2 mm) or less is needed for satisfactory freeze-thaw durability in conventional bridge deck concrete. Values of $\bar{L}$ were calculated for the study concretes and are reported in Table 14.

From Table 14 it can be seen that there is good agreement between the $\bar{L}$ values as determined from the cores and those as determined from the 6 inch x 12 inch (15.2 cm x 30.5 cm) specimens made of fresh concrete. The greatest difference is associated with the superplasticized concrete mix, with the higher values found in the cores probably reflecting a consolidation problem.

Table 13

Void Contents of Cores and Field Specimens

Type Concrete	Measured Air Content %		Voids in Cores, %						Voids in Specimens, %					
			> 1 mm		< 1 mm		Total		> 1 mm		< 1 mm		Total	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
Class A5 concrete, tee beams	4.4	.95	2.1	.60	3.4	.96	5.6	1.4	1.3	.17	2.6	.44	3.9	.47
Class A4 concrete, overlay B604	6.8	.85	2.8	.65	3.7	.25	6.5	.53	1.5	.53	4.6	1.5	6.1	1.4
Class A4 concrete superplasticized, overlay B602	5.4	1.9	2.5	1.2	2.9	1.6	5.3	2.2	2.2	.77	5.3	2.9	7.5	3.4

$\bar{x}$ = average

s = 1 standard deviation

Satisfactory spacing factors were obtained in the overlay concrete used on B604. Values of $\bar{L}$ for the superplasticized overlay were about twice as large on the average as for the conventional concrete. The Class A5 concrete was not designed primarily for freeze-thaw durability, so satisfactory values of $\bar{L}$ were not expected. It appears that a large value of $\bar{L}$ is associated with low air content. In general, the air content of the superplasticized concrete was lower than anticipated. Unfortunately, two freeze-thaw specimens of superplasticized concrete which had a satisfactory value of $\bar{L}$ but a very high air content failed the freeze-thaw test.

Table 14

Air Void Spacing Factors

Type Concrete	Type Specimen	Air Void Spacing (inches)	
		Average	Std. Dev.
Class A5 concrete, single tee beams	Cores	.0101	.0036
Class A5 concrete, single tee beams	6" x 12"	.0106	.0013
Class A4 concrete, overlay B604	Cores	.0067	.0006
Class A4 concrete, overlay B604	6" x 12"	.0069	.0021
Class A4 concrete superplasticized, overlay B602	Cores	.0133	.0040
Class A4 concrete, superplasticized, overlay B602	6" x 12"	.0099	.0032

1" = 25.4 mm

Chloride and Cement Contents

Prior experience at the Research Council has shown that concrete will exhibit a chloride content above zero when tested by the specific ion electrode titration method within 28 days after it is placed.[12] The chloride content caused by the infiltration of chloride from deicing chemicals can be determined only if the initial chloride content is known.

Also, since segregation was readily apparent in some of the superplasticized concrete used on B602 in Norton, selected 6 inch x 12 inch (15.2 cm x 30.5 cm) specimens were tested for cement and chloride contents. Samples were taken from the top and bottom portions of the specimens. Void content determinations were also made of the top and bottom portions of the 6 inch x 12 inch (15.2 cm x 30.5 cm) cylinder specimens. On the average, the cement content was the highest in the top 2 inches (5.1 cm) of the cylinder and the chloride content was the highest in the bottom 2 inches (5.1 cm). However, the difference was much less than the magnitude of one standard deviation for the data. The void contents, on the other hand, tended to be one standard deviation higher in the top than in the bottom portion of the cylinder. The cement, chloride, and void content data are shown in Table 15.

From Table 15 it is apparent that the design cement content was maintained throughout the batching, mixing, and placement operations and that in general segregation did not occur. The chloride content data will serve as initial values upon which to base the change in chloride content over the years due to the infiltration of chloride from deicing chemicals. Whereas cement and chloride content determinations can be made from samples taken from any portion of a 6 inch x 12 inch (15.2 cm x 30.5 cm) cylinder, it is believed that the most accurate value for void contents can be determined by sampling the center portion. As has been mentioned, the most significant variability in the data is associated with the superplasticized concrete.

Table 15

Cement, Chloride, and Void Content Data for Selected Specimens

Type Concrete	Cement Content By Percent Weight				Cloride Content By Percent Weight		Void Contents By Percent Volume					
	Design(d)	Measured		$\frac{(x-d)}{d} \times 100$			Top		Bottom		Both	
		$\bar{x}$	s		$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
Class A5, single tee beams	17.7	17.6	2.7	-0.7	.015	.0022	4.1	0.44	3.7	0.47	3.9	0.47
Class A4, B604 overlay	16.5	17.3	2.7	4.9	.008	.0021	6.9	1.6	5.3	0.74	6.1	1.4
Class A4 super-plasticized, B602 overlay	16.1	16.6	2.9	2.9	.006	.0022	8.7	4.5	6.3	1.4	7.5	3.4

$\bar{x}$ = Average

s = 1 standard deviation

Consolidation of the Overlay

It has been suggested that a vibrating screed would provide effective consolidation of an overlay of the type used on the single-tee bridges. It has been established that a vibrating screed provides adequate consolidation when a 2 inch (5.1 cm) thick unreinforced overlay is placed.(5) Internal vibration is used in conventional bridge deck construction. Based on the data in Table 13, it appears that internal vibration provides adequate consolidation for an overlay with a thickness greater than 4 inches (10.2 cm). Also, since the overlay used on the tee beams is reinforced, it more closely represents full-depth construction than the thin overlay construction used in Berryville. A core removed from span B B604 is representative of the degree of consolidation normally achieved with internal vibration (see Figure 50). It would be desirable to specify a vibrating screed for the consolidation of a tee beam overlay so that the controversy could be ended. It is believed that a vibrating screed would help consolidate the superplasticized concrete having a stiff consistency but it is likely that the same screed would over consolidate the high slump superplasticized concrete. It is questionable whether or not a vibrating screed would provide better bond between the overlay and tee beams when the overlay thickness exceeds 4 inches (10.2 cm). A vibrating screed was used to consolidate the concrete in the flange of the tee beams and according to Table 13, the degree of consolidation was less than was achieved by rodding a 6 inch x 12 inch (15.2 cm x 30.5 cm) concrete specimen.

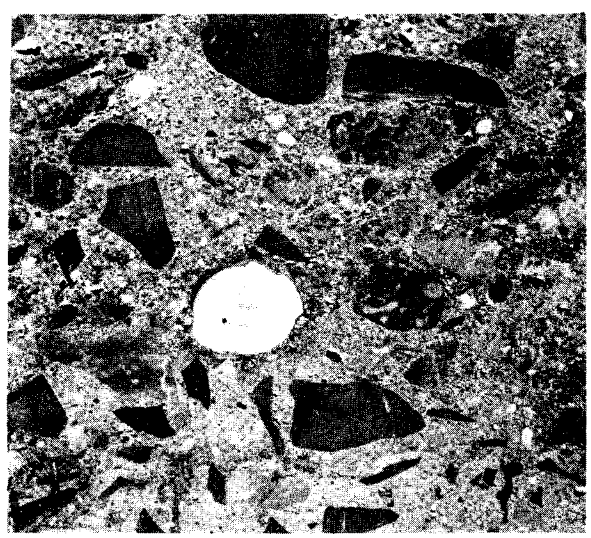


Figure 50. Core removed from span B B604 represents the degree of consolidation usually achieved with internal vibration.

Present Status of Superplasticized Concrete Overlays

Mighty 150 was used in a 4-inch (10.2 cm) composite overlay required for the two 3-span bridges located in Norton. Superplasticized concrete of conventional slump was placed on span C of B602 and the contractor had difficulty placing, consolidating, and finishing it before it lost its workability. Concrete with a higher initial slump and containing Mighty 150 was used on span B of B602 with a reasonable amount of success, but as before the contractor had difficulty finishing portions of the span. Part of span A was overlaid with what may be classified as flowing concreté. Again the contractor had difficulty finishing the span because of the abnormally nonuniform behavior of the overlay. High slump areas bled considerably and exhibited some degree of retardation, whereas concrete in the low slump areas lost it workability before finishing operations could be completed. Cores removed from high slump areas of the deck showed obvious signs of segregation. Similarly, 6 inch x 12 inch (15.2 cm x 30.5 cm) compression test cylinders made from high slump areas showed signs of segregation, but the 28-day strengths were well above the 4,000 psi (27.6×10^6 Pa) design requirement. Freeze-thaw specimens made from the high slump concrete exhibited extremely poor durability characteristics, but a majority of the other freeze-thaw specimens exhibited satisfactory durability behavior and 6 inch x 12 inch (15.2 cm x 30.5 cm) compression test cylinders produced strengths above 8,000 psi (55.2×10^6Pa) at 28 days. Three-day cylinder strengths were over 3,000 psi (20.7×10^6 Pa). All six overlays containing Mighty 150 had a w/c of 0.34, which is 22% below the w/c of 0.43 required for B604 overlayed by the same contractor using a conventional concrete of similar mix design. The air void spacing factor ($\bar{L}$) ranged from 0.006 inch to 0.007 inch (0.15 mm - 0.18 mm) for cores taken from B604, whereas it ranged from 0.006 inch to 0.02 inch (0.15 mm - 0.51 mm) for the cores taken from the overlay on B602 containing Mighty 150. With the exception of the segregated specimens, the higher $\bar{L}$ values for the superplasticized overlay corresponded to durability factors slightly lower than those obtained for the structure containing the conventional concrete. The segregated specimens exhibited extremently poor freeze-thaw behavior. The average $\bar{L}$ for the superplasticized overlay was 0.013 inch (0.33 mm) and the only core exhibiting an $\bar{L}$ less than the desired maximum value of 0.008 inch (0.20 mm) contained segregated concrete. The superplasticized specimens exhibited excellent freeze-thaw behavior based on surface rating and percentage weight loss. The fines content of the mix design was increased prior to placing the overlay on B603, but based on field observations alone only a marginal improvement over the B602 installation was achieved.

The experience in Virginia and elsewhere seems to indicate that superplasticized concrete has tremendous potential for use in construction once certain problems are overcome or are recognized as being insignificant.(13,14,15) Further research is urgently needed to gain a better understanding of superplasticized concrete. A superplasticizer is specified in the contracts for one new bridge deck overlay construction project in Roanoke, and another much larger overlay repair project near Gaston Lake. High early and long-term concrete strength is an asset in most construction but freezing and thawing durability as well as strength is of paramount importance in bridge deck construction. Guidelines must be developed to provide for effective field use of superplasticized concrete if a final product of superior and cost effective performance is to be consistently obtained.

CONCLUSIONS

The following conclusions are based on the information presented in this report on the construction of the first five single-tee bridges in Virginia.

1. Since the flange of the tee beam serves as the lower portion of the bridge deck, in one step the contractor can advance from the bridge seat stage of construction to the forms-in-place stage in a few hours. Costly deck forming at the bridge site is almost eliminated.

2. The current tee beam design reduces site labor considerably but reduces site time only marginally when compared with more conventional types of construction. The use of precast concrete or structural steel diaphragms, precast backwalls, and high early strength concrete overlays is needed to reduce site time significantly.

3. Precast components cast in a good set of forms and under close supervision can be put together quickly and securely in the field. Forms should be checked periodically for proper alignment and shape.

4. The standard shape of the interior tee beam allows efficient and economical fabrication. The shape of the exterior beam requires special time and attention during the fabrication and handling of the beam and the cost is higher than for the interior beam.

5. Inserts and special items increase costs and chances for fabrication and construction errors.

6. The tee beams are easy to transport and place at the bridge site. Satisfactory communications between the prestressor and contractor is essential to ensure that the beams are placed in a manner that eliminates delays and extra handling. Obtaining access to a bridge located in a remote area will occasionally present a problem.

7. To provide proper bearing and fast erection of the tee beams, soleplates should be attached at the bridge site rather than at the fabricating plant. The neoprene pad will not be damaged by field welding. The soleplate will

warp when welded, but the magnitude of the warpage will not cause bearing problems. The camber in the tee beams will cause some nonuniform bearing, but the magnitude of the nonuniformity should not cause premature failure of the bearing pad.

8. When properly mixed, placed, and cured a neat nonshrink cement paste provides an adequate shear key between the tee beams. Nonshrink cement is expensive and the use of a nonshrink cement mortar may be more cost effective than the use of a neat paste. Other types of tee beam connections may be more economical. A welded connection is needed to carry tensile stresses through the keyway.

9. The camber in the tee beams increases linearly with the logarithm of time. When the tee beams are fabricated under the Department's specifications, the rate of increase in camber appears to be influenced by the length and depth of the beam and the strand pattern. The tee beam does not completely satisfy the Department's specifications for the precast, prestressed I-beam; therefore, a tolerance specification should be prepared to cover the tee beam.

10. The tee beam may be stored in a satisfactory manner for many months, since most of the camber occurs in the first few weeks after fabrication.

11. Several men can construct the side formwork, prepare the surface of the tee beams, place the reinforcing steel, and set the screed for the overlay in less than 1 work day. In most cases, sandblasting is needed to properly clean the surface of the tee beams.

12. Concrete for the 4-inch (10.2 cm) overlays was placed at about the same rate as the concrete was placed in the 6-1/2 inch (16.5 cm) base layers in the two-course construction used in Berryville. Twenty-five percent more time was required to complete a superplasticized overlay than a conventional A4 concrete overlay for the study structures.

13. Tee beam movements at midspan are upward due to thermal loads and downward due to the dead load of the overlay. Net midspan movements of 0.25 inch (6.4 mm) can be expected for a 40-foot (12 m) beam. Therefore, for construction purposes it can be assumed that the forms

do not move as an overlay is placed on a 40-foot (12 m) span tee beam bridge. Depth of cover determinations for 40-foot (12 m) spans may be made by measuring the distance between the top of the tees and the bottom of the screed prior to placing the overlay.

14. The average thickness of the overlays on the study bridges was 5.12 inches (13 cm). On the average, 40% of the overlay concrete above the 4 inches (10.2 cm) minimum was caused by camber in the beams and 60% was caused by other variables.

15. The plan thickness of the overlay should be computed with consideration being given to the camber in the beams. The plan thickness should be 4 inches (10.2 cm) at midspan, 4 inches (10.2cm) + c/3 at the quarter point and 4 inches (10.2 cm) + c at the ends of the span, where c is the camber in the beams at the time the overlay is placed.

16. The parapet lends itself to a system concept as it has a constant shape suitable for mass duplication. With the aid of a light crane, 3 men can place and connect the precast parapets on 3 spans in 2 or 3 days. Most of the site time is associated with obtaining an aesthetically pleasing alignment.

17. Satisfactory bond between the precast parapet and the mortar bed is difficult to obtain. Eliminating the extra 4 inches (10.2 cm) of concrete in the outside flange of the exterior tee should improve the fit between the parapet and the mortar bed and reduce the cost of the tee.

18. The quality of the paste-filled voids, the mortar bed, and the bond between the paste or mortar and the precast parapet section will likely control the durability of the parapet.

19. A specification should be developed to ensure that these pastes and mortars are properly mixed, placed, and cured.

20. Further research on connection details for the precast parapets and the flanges between adjacent tee beams is needed. A keyway connection which can support tensile stresses may be desirable.

21. The flexural, shear, and tensile tests performed on models of the tee deck construction indicate that adequate composite action between the overlay and the flange of the tee is developed, but that concrete placed in two layers is 20% - 60% weaker than concrete placed in one lift when the bond area is subjected to tensile and shear stresses which cause yielding. There is no evidence that the weakness is of sufficient magnitude to control the durability of the tee beam bridge deck, if the surface of the tee beams are properly cleaned and roughened. Adequate quality control in the field and sandblasting of the tops of the tee beams is essential for the tee beam structures to perform as well as the laboratory models.

22. Concrete used in the tee beams, the parapets, and the conventional overlays was batched, placed, and cured in accordance with the Department's specifications. Some of the superplasticized concrete used in the overlays did not satisfy Department specifications on air content and consistency. The nonshrink pastes and mortars were mixed by hand at the bridge site under little or no quality control.

23. The measured properties, all of which include air content, slump, unit weight, cylinder strength, freezing and thawing performance, and void content, were approximately 1.5 to 4.5 times more variable for the superplasticized concrete than for the conventional A4 concrete used in the overlays.

24. The high early strengths attained with the superplasticized concrete provide for rapid construction.

25. With the exception of some of the superplasticized concrete specimens, specimens of all of the study concretes performed adequately when subjected to the freezing and thawing test. The performance was the best for the conventional A4 concrete used in B604. Low durability factors were reported for the superplasticized concrete used on B602, but performance with respect to surface rating and weight loss was generally satisfactory. The class A5

concrete experienced the greatest weight loss of the study concretes, as would be expected because it has a low design air content since the members are protected from harsh weather conditions.

26. When compared with the consolidation of standard 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical specimens made in the field, the degree of consolidation of the concrete in the cores removed from the study structures was similar for the A4 concrete overlay, slightly low for the flanges of the tee beams, and high for the superplasticized overlays in B602. The degree of consolidation was the most variable for the superplasticized concrete. The small percentage of voids < 1mm in diameter in the superplasticized concrete suggests poor freezing and thawing durability.

27. Satisfactory air void spacing factors were exhibited by the 4 inch (10.2 cm) diameter cores and 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical specimens of the conventional A4 concrete overlay. By design the air void spacing factor for the class A5 concrete was high, but the spacing factors for the superplasticized concrete were also high, which suggests poor freezing and thawing durability. A satisfactory bridge deck overlay must have excellent resistance to freezing and thawing actions.

28. Although segregation was obvious in some of the superplasticized concrete, cement and chloride content determinations made of the top and bottom portions of 6 inch x 12 inch (15.2 cm x 30.5 cm) cylindrical specimens indicated that, in general, the magnitude of the segregation was not significant. Void contents in the top portion of the specimens were about one standard deviation higher than in the bottom portion of the specimens for all the study concretes.

29. Further research is urgently needed to gain a better understanding of the characteristics of superplasticized concrete, especially those related to field handling and durability. Most of the data presented in this report are based on one mix design for superplasticized concrete.

RECOMMENDATIONS

1. The Department should continue to specify precast, prestressed, single-tee concrete beams for bridge construction in a manner consistent with the conclusions of this of this report.

2. Further research should immediately be directed to connection details for the precast parapets, since the parapets are now being used with more conventional bridge superstructures.

3. Consideration should be given to developing a precast concrete or steel diaphragm and precast concrete backwall.

4. Consideration should be given to developing a keyway which can support tensile stresses.

5. Further research should be directed to the effective field use of superplasticized concrete. The Department should proceed with caution when specifying superplasticized concrete because of its questionable freezing— thawing durability. Concrete of high durability is needed for the top portion of a bridge deck.

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REFERENCES

1. "New Approaches in Prestressed, Precast Concrete for Bridge Superstructure Construction in Virginia", Virginia Prestressed Concrete Association, August 1973.

2. "Prestressed Single T-Beam Details" Bridge Division, Virginia Department of Highways and Transportation, August 20, 1976.

3. Sprinkel, M. M., "Working Plan— Construction of Prestressed, Concrete Single-Tee Bridge Superstructures", Virginia Highway and Transportation Research Council, November 1975.

4. Road and Bridge Specifications, Virginia Department of Highways and Transportation, July 1974.

5. Tyson, S. S., and M. M. Sprinkel, "Two-Course Bonded Concrete Bridge Deck Construction— An Evaluation of the Technique Employed", VHTRC 76-R13, Virginia Highway and Transportation Research Council, November 1975.

6. Shalon, R., "Hot-Dry Climate Effect on Stress Development in Shrinkage Compensating Concrete", ACI Journal, March 1977, p. 112.

7. Udin, Harry E., Welding for Engineers, John Wiley & Sons, Inc., New York, 1954.

8. "Precast, Prestressed Concrete Short Span Bridges", Prestressed Concrete Institute, Chicago, 1975.

9. Price, A. R., "Abnormal and Eccentric Forces on Elastomeric Bridge Bearings", Transportation and Road Research Laboratory, Crowthorne, Berkshire, 1976, p. 8.

10. Newlon, H. H., Jr., and S. S. Tyson, File: 5.5.2.9.1 (Concrete Materials Evaluation).

11. Annual Book of ASTM Standards for Concrete and Mineral Aggregates, American Society for Testing and Materials, Philadelphia, 1973.

12. Tyson, S. S., "Two-Course Bonded Concrete Bridge Deck Construction— Concrete Properties and Deck Condition Prior to Opening to Traffic", VHTRC 77-R3, Virginia Highway and Transportation Research Council, July 1976.

13. Session 107, Transportation Research Board Meeting, Sheraton Park Hotel, Washington, D. C., January 1977.

14. "Superplasticizing Admixtures in Concrete", Cement and Concrete Association, Waxham Springs, Slough, January 1976.

15. Newlon, H. H., Jr., "Examination of Cores from Overlay of Barracks Road Bridge", memorandum dated September 3, 1976, File: 5.5.2.2.

APPENDIX

SUMMARY OF MODIFICATIONS AND CLARIFICATIONS TO ORIGINAL TEE-BEAM DETAILS

1. Abutments

lay '75) a. Note for backwalls to be cast after deck overlay is in place. (Reason: To allow for adjustments in bridge grade if required due to excessive camber.)

ept.'76) b. Eliminate 1/2" lip on face of backwall. (Reason: Difficult to form and odd section of parapet would take care of additional 1/2" length.)

2. Tee-Beam

ct.'74) a. Flange to be fabricated with a width reduction of up to 3/8" per member. (Reason: To facilitate placement of member.)

ept.'76) b. 0058-017-103, B604 (252-94) Rte. 58/Little Reed Island Creek. This project only: Detailing the 4" overlay the total width of the bridge. Exterior beam cast same as interior beam, except for reinf. steel and parapet anchors. (Reasons: Camber in beams cause an irregularity in grade of top of parapet. A vertical cold joint and stability problem are eliminated.)

3. Transverse Section

ept.'76) a. Note added: After erection, tops of tee-beams shall be lightly sandblasted and blown off with oil-free compressed air to remove all dirt, laitance, and other foreign objects prior to the placement of deck reinforcement.

ept.'76) b. Dimensions along slope at bottom of beam (specifically the first beam each side of centerline) as well as dimensions along top of beam shall be detailed on the Transverse Section. (Reason: To show the effect of the cross-slope in erection of the beams.) Also, beam spacing dimensions shown on abutments and piers shall be measured along slope and bottom of beams.

4. Bearings

(Mar. '76) a. Increased thickness of insert plate (3/4" to 1") and soleplate (3/4" to 1"). Reduced size of fillet and groove weld from 3/8" to 5/16". (Reason: Changes made to try to eliminate warping of soleplate and insert plate when welding.)

(Sept.'76) b. Made insert plate 1" wider than soleplate. (Reason: To eliminate concrete bearing on soleplate.)

(1 inch = 2.54 cm)