INVESTIGATION OF CONCRETE OVERRUNS ON I 69 TWO-SPAN CONTINUOUS-TYPE BRIDGES
Pearl Beach Road Over I 69 (S11 of 12033A)

E. L. Marvin, P. E.

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ABSTRACT: As a result of deck concrete overruns on various two-span, continuous-type bridges, detailed field measurements were taken on one of the first of this type built in Michigan. Concrete volume in place was determined and the elevation profile measured. The investigation showed that the deck was too thick, accounting for most of the overrun. Beams sagged under the first pour placed and rotated over the pier, primarily as a result of composite stiffness in the span under the first pour, which resisted the upward deflection caused by loading of the other span as the deck was poured there. Shrinkage-induced deflection also occurred, contributing to the total sag measured. Some corrective measures have already been applied, and recommendations are given for other modifications in design and construction of this type of bridge.

KEY WORDS: overruns, continuous structures, continuous beams, bridge decks, bridge spans, bridge/structures, bridge design, volume determination, shrinkage, composite beam bridge decks, screeds, sagging, slabs, thickness.

SYNOPSIS

Summary

During 1966, consistent and large concrete overruns occurred in construction of decks on new, two-span, continuous-type bridges on the I 69 freeway project near Coldwater. In investigating this problem, the following field studies were conducted:

1. Concrete was measured in place in two pours of a completed two-span continuous bridge deck, and the actual volume determined.

2. On the same bridge, the elevation profile of the top of the bridge deck was measured along the centerline and each gutter line.

In addition to the field investigations, the problem of overruns on two-span continuous bridges was discussed with responsible personnel in the Offices of Design and Construction. The information obtained was very helpful to this study. Finally, 50 concrete proportioning pour reports from various other bridges constructed on the I 69 freeway were reviewed.

The field investigations showed that:

1. The deck of the bridge measured was too thick for various reasons and this extra thickness accounted for most of the overrun reported.

2. The measured concrete volume in place was only about 3 percent less than the reported volume.

3. The measurements taken showed that beam sagging occurred in Span 1 under the first pour placed. Also, it was noted that the beams rotated over the pier. Both of these irregularities are believed to have been caused primarily by composite stiffness, which developed in Span 1 and resisted the upward deflection in that span caused by the loading of Pour B in Span 2.

4. The measurements taken also showed that the beams were low on the bearings at Reference Line A and Pier 1.

5. The extent of the Span 1 sag indicated that significant shrinkage-induced deflections probably occurred and contributed to the total sag measured.

Discussions with field and design personnel turned up several causes of overrun that had been discovered prior to this study. In those cases, corrective measures had been adopted. The review of pour reports showed that small overruns consistently occurred on most bridge projects checked.

Recommendations

1. Composite beam stiffness should be considered in the design of screeds for continuous-type bridges. Shoring the beams under Span 1 during the placement of Pour A, and then removing this shoring prior to placement of Pour B in Span 2, would provide a more predictable structural stiffness in the beams. This procedure would also reduce tension cracking in Pour A, which has become a problem on these continuous bridges.

2. Proper adjustment of slab haunch thickness over beams is essential to insure that uniform thickness will be obtained throughout the deck. It was found that considerable effort is expended by project engineers in making these adjustments. The adjustments required vary considerably throughout the deck. It is thought that more beam deflection information and beam profile elevation data should be made available for project engineers' use. It is understood that more information is now being provided on the bridge plans to aid in dimensioning these haunch thicknesses. If additional information is required during construction, it is noted that electronic computation of this type of information is provided by the Management Services Division in Lansing, and thus some method could probably be devised to provide this service to field engineers.

3. There is some evidence that the screed on the longitudinal screeding machine rides up on the concrete surface and causes the slab to be from 1/8 to 1/4 in. thicker than planned. To allow for this, some project engineers decrease the screed camber slightly. It is thought that this correction may be necessary, although this procedure of decreasing the camber ordinate should be carefully controlled to insure that no less than the minimum designed slab thickness is maintained throughout the deck.

4. The preponderance of small overruns indicated to be occurring on bridge projects could possibly be reduced by developing and adopting more accurate methods of estimating concrete wastage.

5. Concrete shrinkage-induced deflection is not a cause of concrete overrun. However, it is thought that concrete shrinkage does cause beam sag which results in low spots in the finished bridge deck where salt and water can collect and cause damage to the bridge deck riding surface. Theory necessary for predicting the behavior of this mechanism has been developed. It is believed that some further study of shrinkage-induced deflection, and development of modifications to existing bridge designing computer programs would lead to a simple means of predicting this type of deflection, which could be used in the design of bridges.

CONTENTS

INVESTIGATION OF CONCRETE OVERRUNS
ON I 69 TWO-SPAN CONTINUOUS-TYPE BRIDGES
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The subject of concrete overruns on bridge deck pours was discussed by W. W. McLaughlin at the August 27, 1966 Testing and Research staff meeting and was subsequently assigned to the Research Laboratory for investigation. It had been observed that consistently large overruns had been reported on nearly all I 69 bridge projects in the Coldwater area. Particularly large overruns were noted on deck pours of new two-span continuous-type bridges being constructed there. The purpose of this study was to determine the causes of these overruns and what corrective measures should be taken to prevent them on future projects.

Preliminary discussions were held with the Bridge Design Supervising Engineer responsible for the design, the I 69 project Resident Engineer, and several Project Engineers in the Coldwater area. It was learned that some causes of deck overruns had been determined and that corrective measures had been introduced on succeeding construction projects. One important corrected problem concerned beam deflections with respect to sequence of pouring, as discussed in Appendix A of this report.

Another cause of deck overruns was thought to be sagging of formwork between steel beams during placing of the concrete deck. On subsequent projects, the size of the wooden joists between beams was increased by contractors from 2 by 6 in. to 2 by 8 in. Still another problem concerned settlement of formwork at the beams. This was apparently caused by the steel wire hangers, which attach to the beams and support the formwork, cutting into the wooden joists during placement of deck concrete. It is understood that steel bearing clips will be used on future projects, which will bear between the wire hangers and the joists, thus preventing the wire from cutting into joists and causing settlement of the formwork.

It was determined, however, that smaller overruns continued to occur after making these initial adjustments of screed elevations and joist sizes. Particularly perplexing was the occurrence of overruns in Pour C of two-span continuous bridge decks. Pour C, as indicated in Figure 1, is almost directly over the pier, where deflections are quite small. No explanation could be given for overruns occurring there, and it thus became apparent

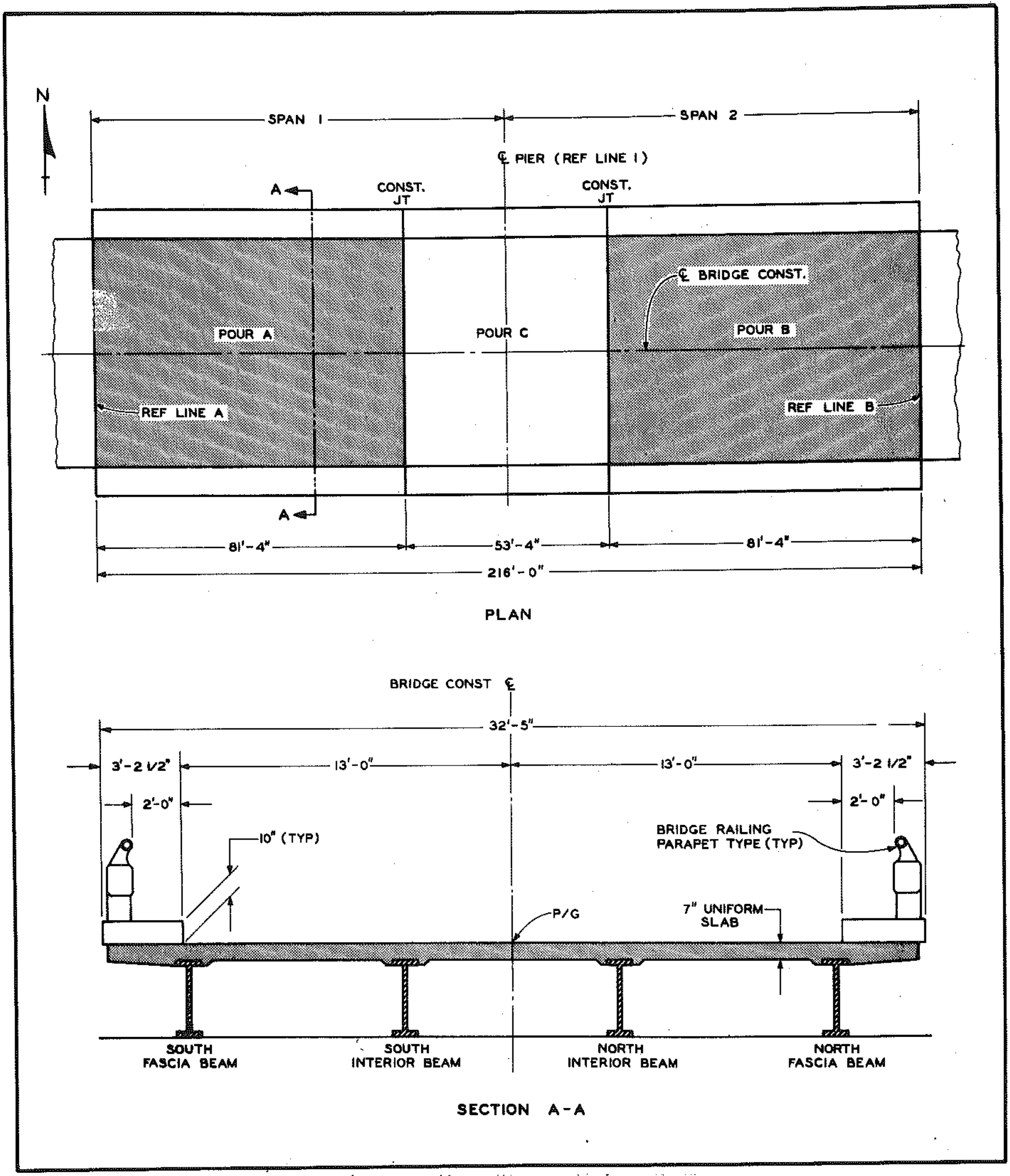


Figure 1. Bridge plan and cross-section, indicating concrete pour locations.

that overrun problems on two-span continuous bridges could be divided into two classes:

1. Explainable causes for which corrective measures had been taken or are being implemented.
2. Undefined causes.

It was decided to attempt to determine some of these undefined causes of overruns, and by direct measurement to check the volume of concrete in place on Pours A and C on one two-span continuous bridge.

FIELD MEASUREMENTS

Bridge Project S11 of 12033A (Pearl Beach Road over I 69) was selected for a detailed investigation. This bridge (Fig. 2) was selected both because it was one of the first of its type constructed in Michigan, and because large overruns occurred in the deck. It was thought likely that several causes of overrun, therefore, would be exhibited here. Bridge concrete proportioning pour reports (Appendix B) indicate that Pour A in Span 1 was overrun 22.5 percent or 14.4 cu yd, and Pour C over the pier was overrun 17.2 percent or 6.4 cu yd. It should be noted that refinements have been achieved through experience in subsequent construction of this type of bridge deck.

Field measurements were made on the Pearl Beach Road structure for the following purposes:

1. To determine, by direct measurement, the actual volume of concrete in place in Pours A and C of the deck.
2. To determine the actual longitudinal gutter line and centerline elevation profiles.
3. To determine how slab thickness varied in these pours.

The deck on the Pearl Beach Road bridge was cross-sectioned at nine stations in Pour A and five stations in Pour C, using special direct measuring equipment constructed in the Structures Unit machine shop. The method of measurement and equipment used are illustrated in Figure 3. First, aluminum tubes were suspended from rigid frames attached to the sidewalk at both sides of the deck. Two wires were attached to the tubes below the bridge beams, and each marked with 28 steel clips at positions where thickness measurements were to be taken under the deck. Two wires were used in order to keep the measuring probe plumb. Care was exercised in accurate placement of the posts at the same station and the tops of posts at the same elevation. Further caution was taken to insure that the metal clips on the two wires matched vertically, and that the wires were taut and level. Vertical measurements were made from the bottom wire to the bottom of the deck slab at the desired locations, using a lightweight probe as illustrated in Figure 3. These measurements were read accurately to the nearest 1/16 in. Next, a similar wire was placed over the top of the deck, with care taken to insure that the clips on this wire lined up vertically with clips on the wires below the deck. Measurements were made with the probe from the top of the slab to this wire as illustrated in Figure 3. In all, bridge deck thickness in Pours A and C was measured at 297 locations.

Figure 2. Pearl River Road structure over I 69.

BM 24, El. 990.40, P.K. nail and tag in east root of a 20-ft hickory stump, 250 ft left of Sta. 289+00 on I 69 was used as a reference elevation. A Ziess Ni 2 automatic level with an optical micrometer was used to differential level to the bridge at TP 2, indicated by black paint on the northwest corner of the north sidewalk. The elevation of TP 2 was found to be 1013.23 and was checked by reverse leveling to BM 24. Using the level, a profile was taken in each gutter line and along the centerline of the roadway. Theoretical elevations of these points were determined by use of a computer program and may be compared with the measured values in Figure 4.

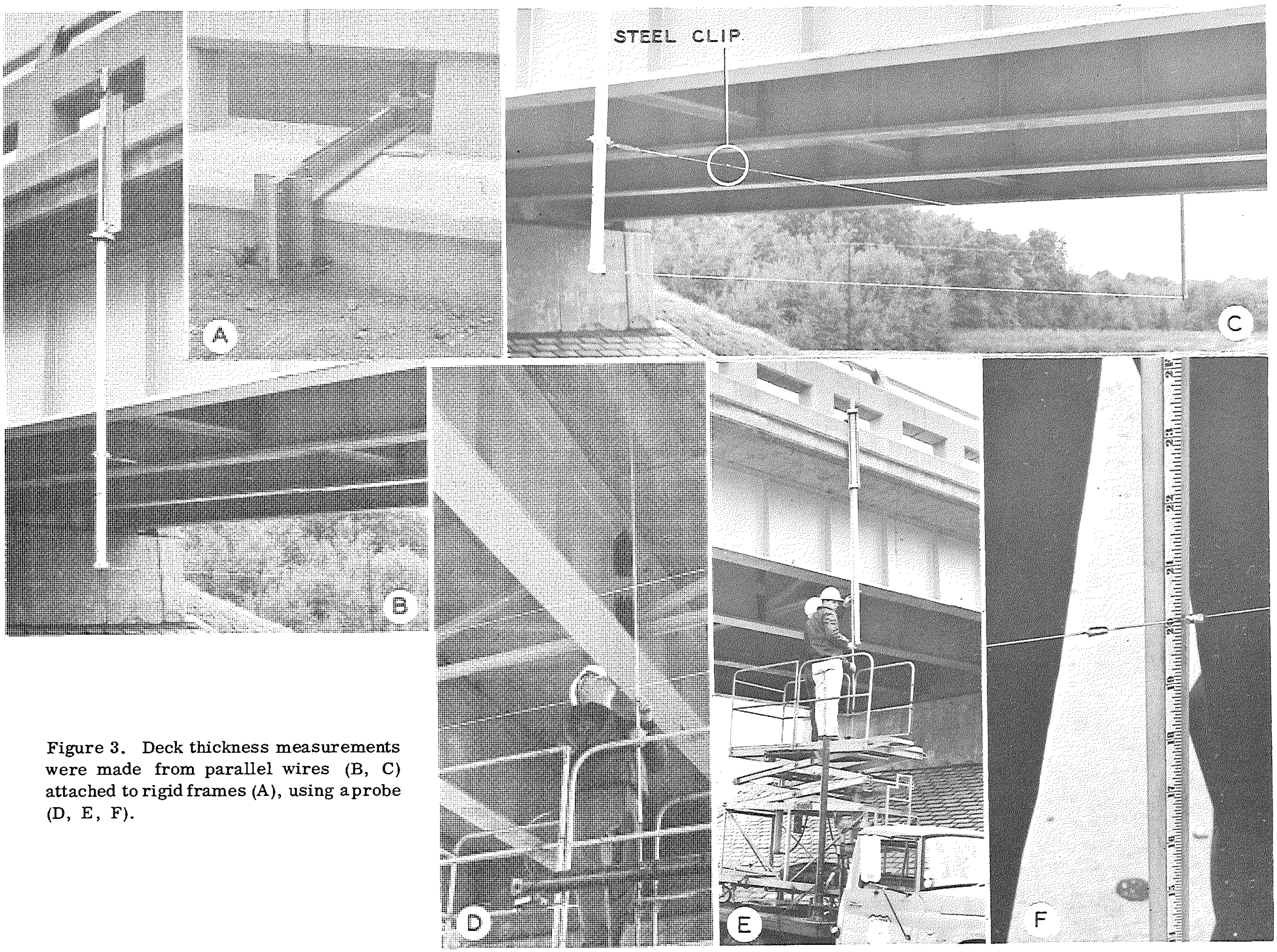
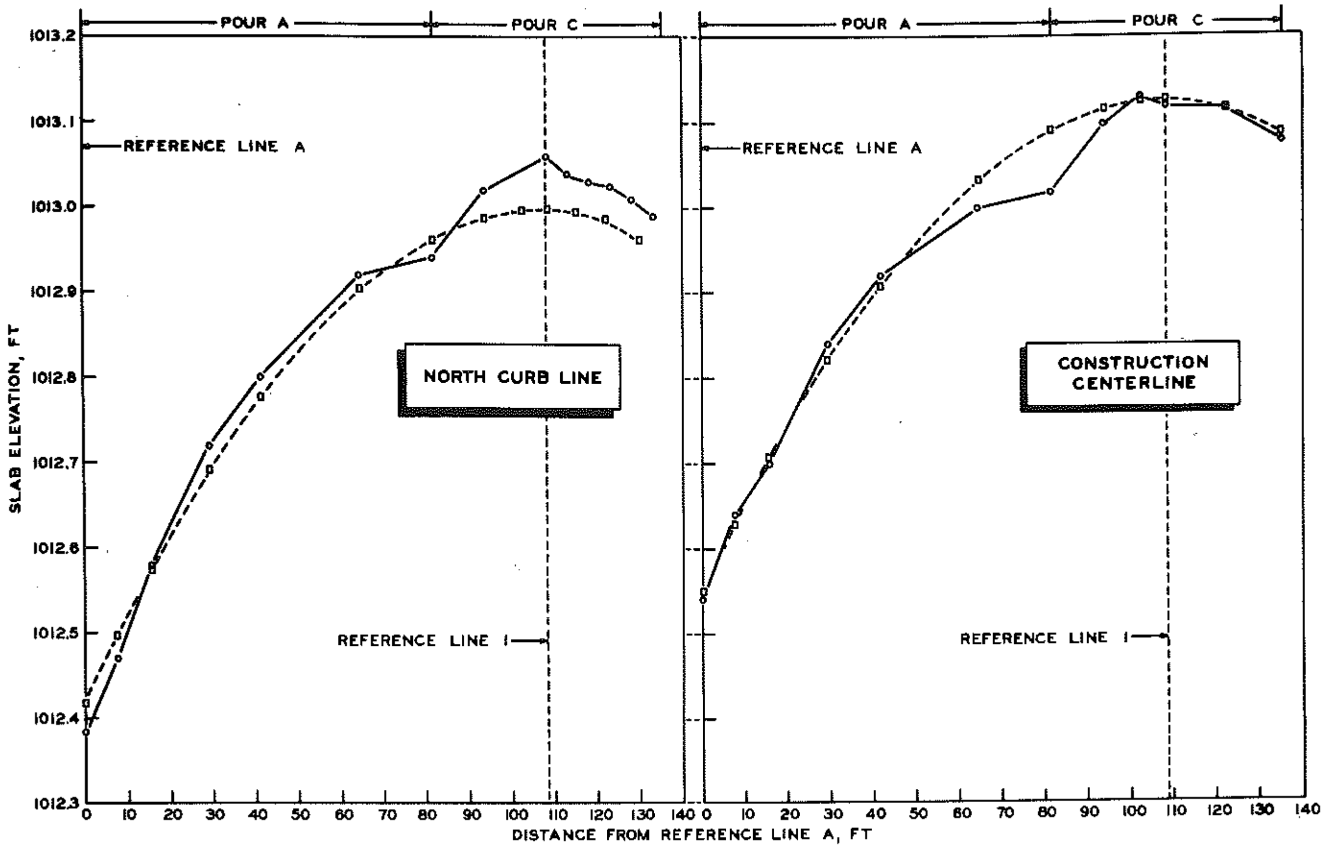
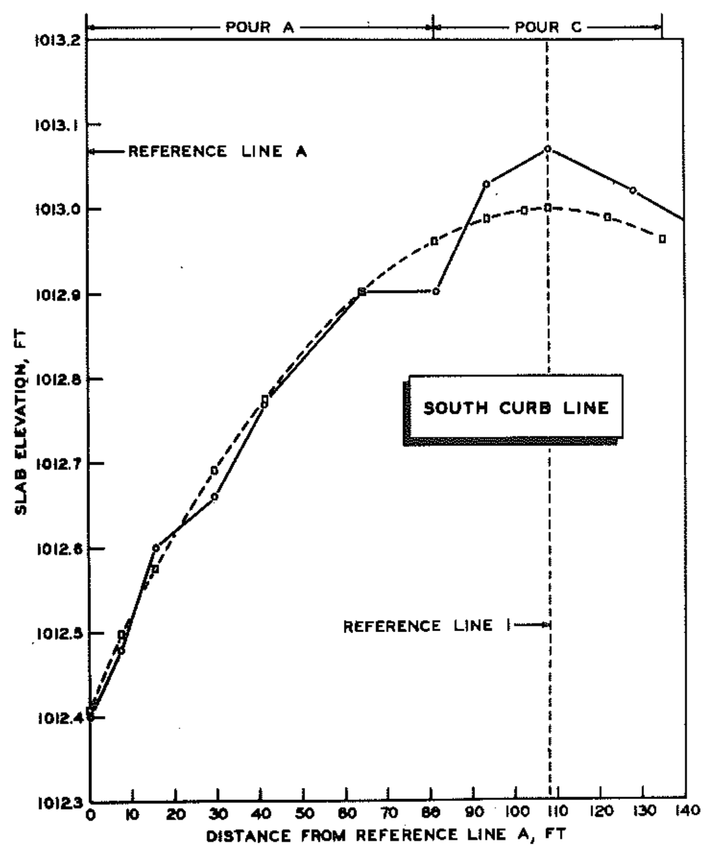


Figure 3. Deck thickness measurements were made from parallel wires (B, C) attached to rigid frames (A), using a probe (D, E, F).





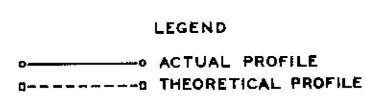


Figure 4. Profiles of top of slab. Locations of reference lines and concrete pours are shown in Figure 1.

MEASURED VOLUMES OF DECK CONCRETE

The slab thickness data thus obtained were compiled, and the actual volume of concrete in the deck was computed from them using a computer program based on the average end area method of computing irregular volumes. These electronic computations were independently checked by drawing end area sections to 1/4 scale and measuring the plotted end areas with a plainimeter. The location of end areas and the values determined are shown in Figure 5.

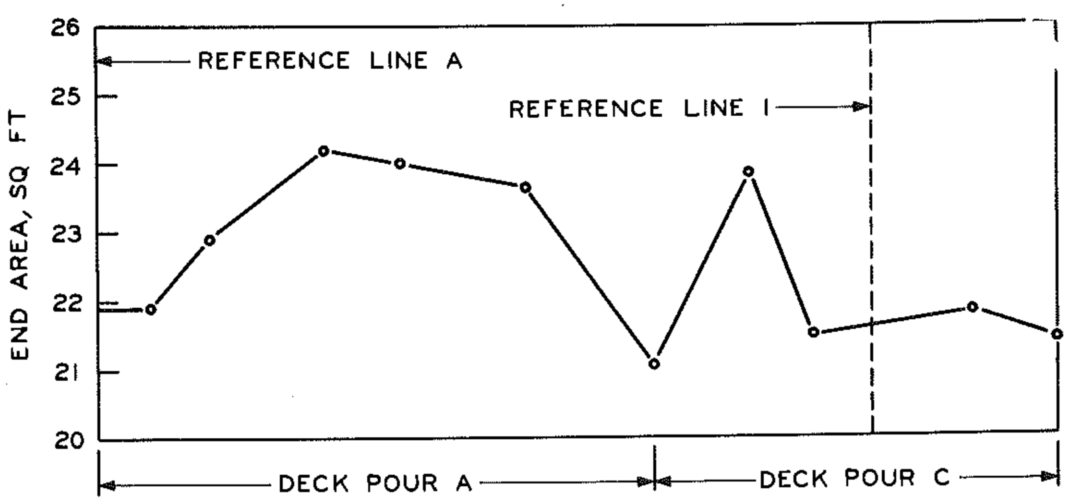


Figure 5. Measured bridge deck end areas.

The actual volume of concrete in the deck was found to be 76 cu yd in Pour A and 42 cu yd in Pour C. These volumes include allowances for variable factors, such as deck length and width variations, possible slab thickness variations under the sidewalks, and concrete shrinkage. The volume of the steel reinforcing bars was deducted from the gross volume measured. The bridge concrete proportioning reports (Appendix B) indicated that 78.5 cu yd of concrete was used in Pour A and 43.5 cu yd in Pour C.

Some of this variation between measured and reported volume used may have occurred in estimating the quantity of concrete wasted. During the preliminary investigation it was mentioned by some of the field personnel that it is difficult to estimate wastage since this is done visually by just looking inside the mixer. There also may possibly be some loss or wastage occurring during handling of the concrete after it is batched, and before and during placement in the deck.

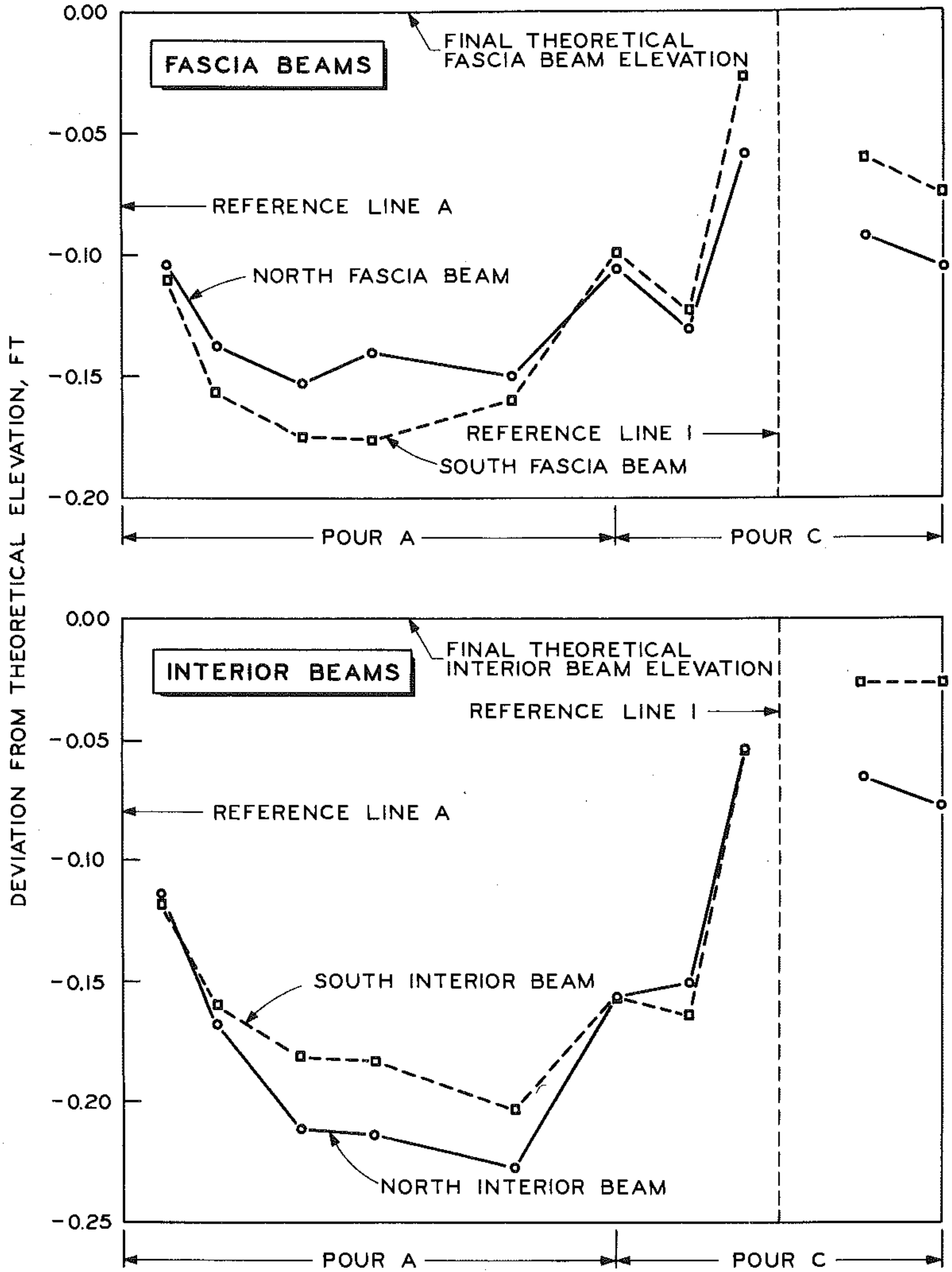


Figure 6. Variations of tops of beams from theoretical final positions.

DECK PROFILE AND SLAB THICKNESS VARIATIONS

Pour A

Figure 4 illustrates that in Pour A, the final roadway surface along the construction centerline matches the theoretical plan grade quite well, to within about 35 ft of the first construction joint where the actual slab dips below the theoretical curve. The low point of the depression occurs at the construction joint between Pours A and C. Actually, considering the mid-ordinates of the theoretical curve and the actual top-of-slab curve, in combination with the actual final beam elevations, it becomes apparent that a 5/8-in. "hump" occurred at the midpoint of Pour A. Presumably, this hump was caused primarily by the upward deflection that occurred in Pour A when Pour B was placed on the adjacent span (Appendix A). It is also thought that from 1/8 to 1/4 in. of this hump may have been caused by the tendency of the longitudinal screed to ride up on the slab surface. The reason that a dip appears on the graph is that the tops of the beams, under Pour A, are low.

Variation of the elevation of the top of each beam from that theoretically predicted is illustrated in Figure 6. The theoretical value is taken as zero variation and is plotted as a horizontal straight line. Slab thickness measurements were correlated with slab profile measurements to obtain this variation information. It was assumed for the purpose of computing this information that the bottom surfaces of the top beam flanges were flush with the bottom surfaces of the concrete haunches. This assumption introduces a possible error of about 0.01 ft. However, this is not significant because of the magnitude of the top of beam variations. Figure 6 indicates that the beams are low at both Abutment A and Pier 1. Furthermore, it is noticed that an average sag of about 3/4 in. occurred in the beams under Pour A. Some of this sag in the beams was caused by the extra dead load of the thickened slab. The extra slab thickness along the centerline of the bridge and the south curb line was found to be about 1 in., and at the north curb line to vary from 0 to 1-1/2 in. Figure 7 illustrates the measured values of slab thickness at the Pour A locations discussed here. The slab thickness shown on the bridge plans was 7 in. Based on a positive variation in slab thickness of 1-1/2 in., it is estimated that 1/4-in. sag should have occurred in the beams between Reference Line A and the construction joint. Therefore, it is surmised that most of the sag that occurred in the beams in Span 1 was caused by other factors.

In-place measurements indicate that the beams rotated over the pier and acted compositely with the slab in Pour A in Span 1 in resisting the

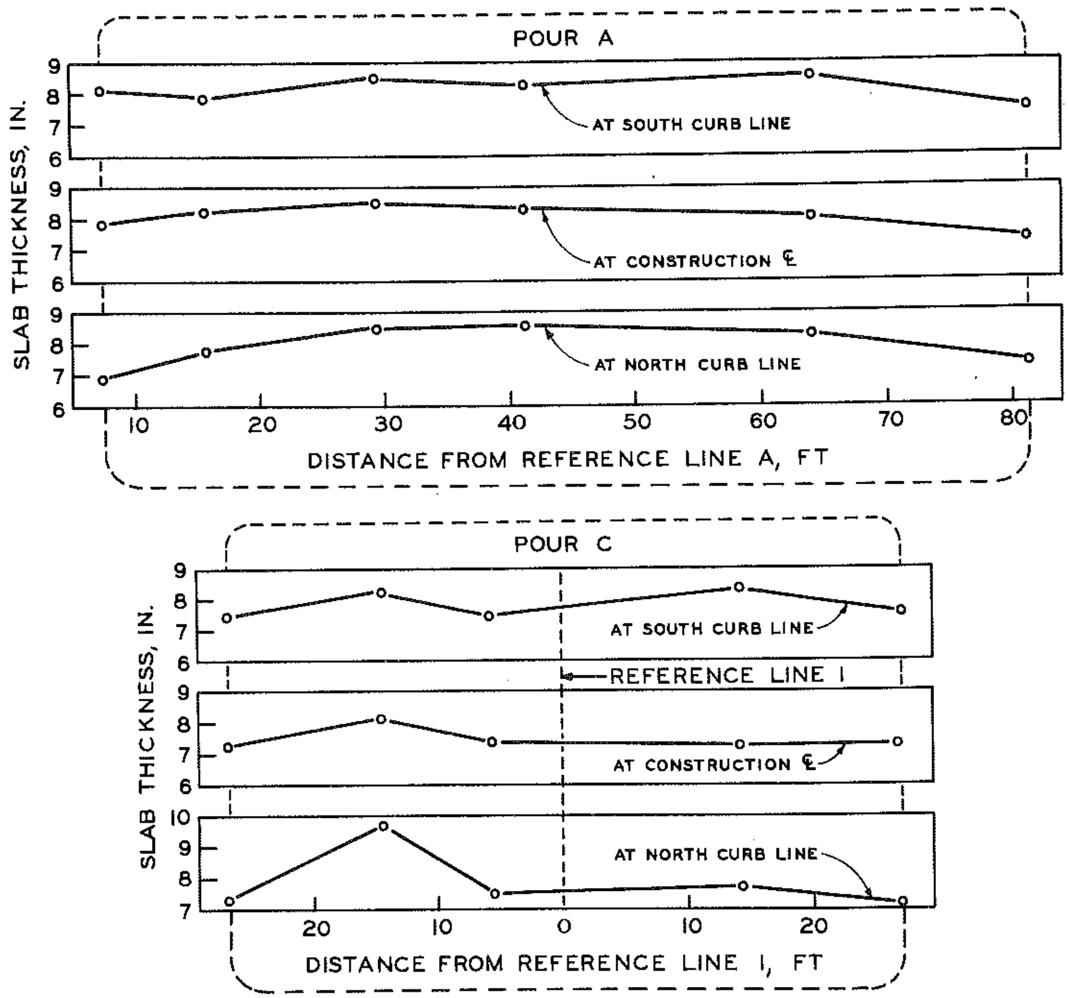


Figure 7. Measured slab thicknesses.

loads induced by placement of Pour B on Span 2. Figure 6 illustrates, for example, that the final elevation of the top of the north interior beam at the left construction joint of Pour C is about 0.075 ft lower than at the right construction joint. The Project Engineer's top-of-beam measurements prior to the placement of Pour A indicated that the left joint was then only 0.02 ft lower than the right joint. It is thought that most of this rotation resulted from a reduction in anticipated upward deflection of Span 1 under the Pour B slab loading in Span 2. This reduction in upward deflection is believed to have been caused primarily by the development of composite beam stiffness in Span 1. This composite response under Pour A probably caused most of the sag in the beams under Pour A.

It is also likely that shrinkage of concrete in Pour A contributed to the sag in Span 1. About 24 hr elapsed between placing Pour A and Pour B and it is thought that a shrinkage strain of 1×10^{-4} could occur during this time (1). Such shrinkage of the concrete would cause the beams to deflect downward under Pour A. It is also noted that after Pour B was placed, shrinkage-induced deflection would continue to occur in both Pours A and B. Accordingly, a sag also should have occurred in Pour B by the time the concrete in-place measurements were made for this study. The limited data obtained indicate, however, that only a slight sag occurred in Span 2.

Shrinkage-induced stresses in composite beams have been considered by William Zuk (2). Dr. Zuk presented a sample problem in which he considered a simply supported steel beam with a 7-1/2-in. composite, unreinforced concrete slab. The span length considered was 66 ft and the beam spacing 7 ft 6 in. A linear shrinkage strain of 3×10^{-4} was assumed. For the example given, his theory predicted a downward beam deflection of about 3/4 in. Although the two-span continuous bridge, with only Pour A in place, differs considerably from the example problem, it is apparent that shrinkage according to Dr. Zuk's theory probably caused some of the beam sag that occurred. It is noted that Dr. Zuk's assumed shrinkage coefficient appears reasonable if the slab's long-term shrinkage over several months is considered.

Pour C

The theoretical and measured longitudinal profiles in Pour C as depicted in Figure 4 show that a hump occurred in the top of the slab in each gutter, and that along the bridge centerline the slab rises from the low point at the joint adjacent to Pour A to the theoretical grade elevation at Reference Line 1 over the pier. The cause of these humps at the curb lines can be determined by considering the mid-ordinate of the profiles of the top of slab in Pour C. It can be seen that a mid-ordinate screed camber of 7/8 in. occurred in most of Pour C. This screed ordinate is 3/8 in. greater than the 1/2-in. screed camber called for on the bridge plans. This additional 3/8-in. screed camber was required to insure that the roadway centerline profile would match the theoretical profile at Reference Line 1. In other words, the 3/8 in. corrected for the dip at the construction joint. Now, studying the theoretical and actual profiles at the north curb line, shown in Figure 4, it is noticed that the dip at the first joint here is much less than at the centerline and that the top of the slab is high at the left end of Pour C. These two factors combined with the 3/8-in. extra camber to cause the hump that occurred along this curb line. Considering the south curb line, it was found that a 1-3/8-in. longitudinal camber occurred here. Primarily, this must be attributed to some irregularity occurring in the finishing of the concrete at this location, since the mid-ordinate of the surface profile here exceeds that found at other longitudinal sections in Pour C. It was mentioned during the preliminary discussions that the longitudinal screeding machine seemed to ride up on the concrete surface in Pour C in one location and caused the surface of the concrete to tear. This area where the surface was torn was finished with hand tools. It is possible that this is the cause of most of the hump at the south curb line.

SUMMARY

Beam Elevation Variations

It was determined that the beams were slightly low under Pours A and C. Although corrections were made to the haunch depths above the beams, overruns still resulted because of the extra concrete required in the thickened haunches, and because in some cases where the adjustments were not sufficient, resulting slab thickness above the haunches was greater than the plan dimension of 7 in.

Beam Sag

It was determined that beams sagged under Pour A. This was probably caused, to a small extent, by the extra dead load of the thickened slab and, to a large extent, by other factors. Deflection measurements by the Project Engineer during placing of Pour A showed that the beam over-deflected about 3/8 in., which agrees well with the theoretical increase in deflection that can be attributed to the extra slab dead load. Therefore, it is concluded that most of the sag discovered in the beams occurred after Pour A was constructed. The primary cause of this sag is thought to be the development of composite stiffness in Span 1, which prevented some of the predicted upward deflection in Span 1 under the loading of Pour B in Span 2. Another cause of beam sag is thought to be concrete shrinkage-induced deflection.

Top of Slab Variations

High spots were found along the curb lines in Pour C over Pier 1. These humps in the slab are attributed, at least in part, to excess camber in the longitudinal screed profile. If this was caused by adjusting the screeding machine to match the centerline grade of the reference line, it can of course be avoided on future projects. If this excess camber was caused by the screeding machine's riding up on the concrete surface over the pier, then it will probably be necessary to reduce screed camber to allow for this.

Haunch Depth Variation

It was found that haunch depth in the concrete slab over the beams was adjusted considerably throughout Pours A and C. The actual average haunch depth measured along each beam is shown in Figure 8. The haunch depth indicated on the bridge plans is 1-1/2-in. uniform thickness. This specified uniform haunch thickness is based on the assumption that the beams are precambered exactly as planned, that the tops of beams are at the exact elevations indicated over the pier and abutments, and that beams will deflect under slab loads as predicted. Haunch thickness along the beam is therefore adjusted as required to insure that the uniform specified slab thickness will be obtained.

Near Abutment A, haunches over the interior beams are 2-1/2 in. deep, or 1 in. in excess of the plan dimension. At the same location, haunch depth over the north fascia beam is 2-3/4 in. and at the south fascia beam, 2-1/4 in. It is also noted in Figure 7 that slab thickness above the top of the haunch at Abutment A is 8 in. at the south curb line and construction centerline, and 7 in. at the north curb line. It is apparent that the haunch adjustment here was adequate over the north fascia beam, but should have been about 1-in. greater over the interior beams and south fascia beam.

It can be seen now that a large adjustment in haunch thickness was required at the construction joint in Span 1 between Pours A and C where the low spot occurred. For example, it is estimated that a total haunch depth of 3-3/8 in. was required on the interior beams there. The measured haunch depth there was about 2-1/2 in., averaged transversely across the beams. The Project Engineer's records indicate that an average haunch depth of 2-3/8 in. was set there. Based on the top-of-beam elevations measured by the Project Engineer, and the predicted slab load deflection, assuming the beams acted in a non-composite manner, this haunch setting of 2-3/8 in. was correct. However, upon considering the Project Engineer's midspan measurements of deflection, it is apparent that the portion of the beam from Reference Line A to the end of Pour A acted as a composite beam when Pour B was placed in the adjacent span. This composite action of the beam reduced the upward deflection under Pour A from the value predicted by considering only the stiffness of the steel beams. It is estimated that the composite stiffness of the beam caused the joint to be 1/2-in.

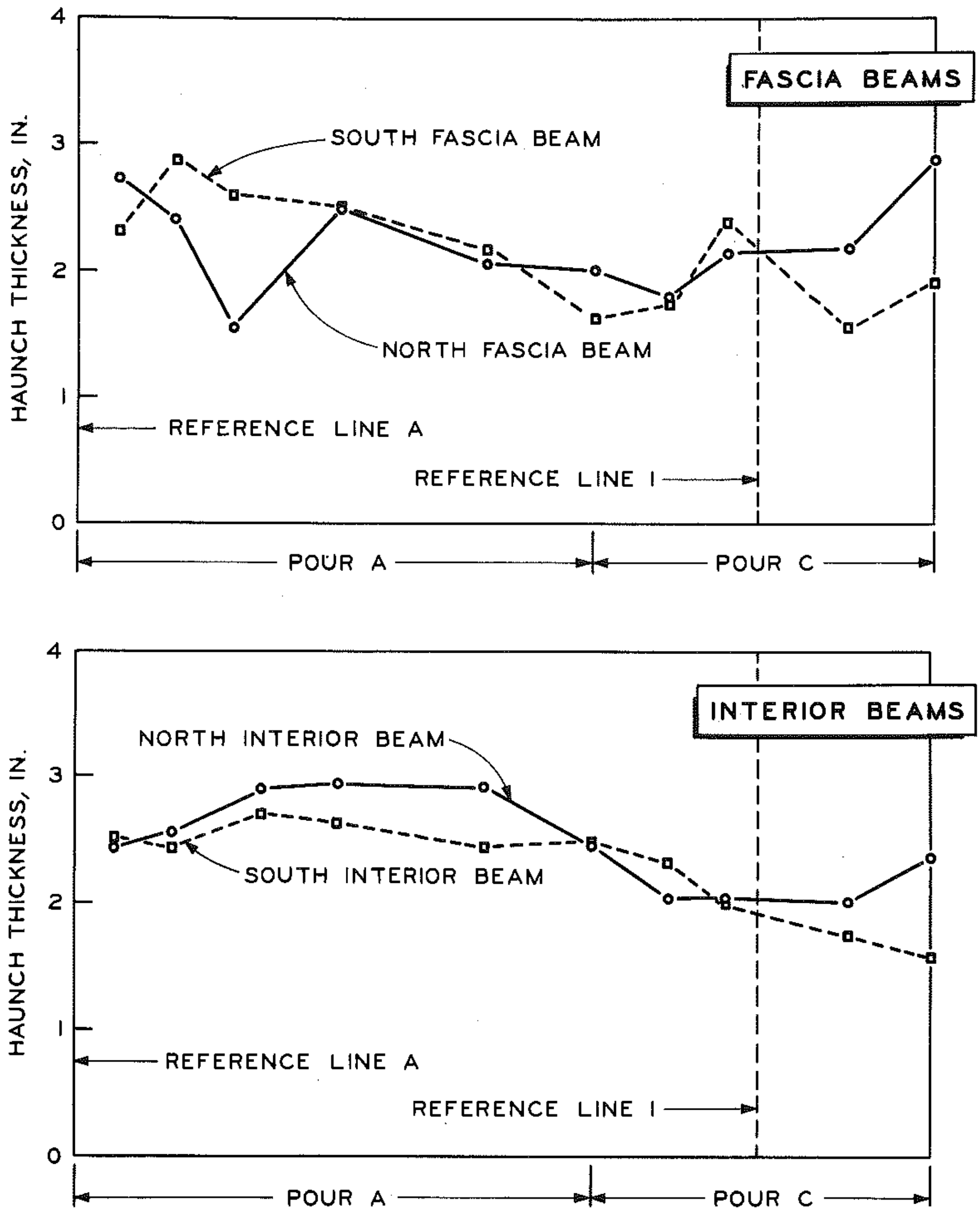


Figure 8. Longitudinal profiles of actual concrete haunch thicknesses.

lower than anticipated. An electronic computer program is currently being prepared by the Office of Design that will provide theoretical solutions to this type of deflection problem. When this program is available, a more accurate determination of this composite effect will be possible than has been given here. Assuming a 1/2-in. decrease in upward deflection at the joint, it would seem that a haunch thickness of about 2-7/8 in. was required. Actually, there is a possible problem of reference which makes it difficult to correlate the Project Engineer's measurements with those taken after the bridge was constructed. The Project Engineer's established reference elevation was defined to be 0.03 ft higher than the value used in making the bridge in-place measurements. If a correction of +0.03 ft for this variation in definition is added to the haunch depth, a required depth of 3-1/4 in. is indicated, which is just 3/8 in. less than the 3-5/8-in. haunch depth required. It is thought that the additional 3/8-in. depth requirement could have resulted from shrinkage-induced deflections.

Near the pier, slab haunch thicknesses over the beams were all about 2 in., or 1/2-in. deeper than shown on the bridge plans. The slab thickness there (Fig. 7) was about 7-1/2 in. It appears that an additional increase of 1/2-in. haunch depth was required there. By comparing Figures 4, 6, and 7, haunch adjustments that would have been necessary to insure a uniform slab throughout the deck could be determined. It appears that proper adjustment of haunch depth over the beams is the most important factor involved in obtaining a uniform slab thickness.

Sag and Settlement of Formwork

It was mentioned during the preliminary discussions that formwork seemed to sag between beams when deck concrete was placed on the Pearl Beach Road bridge. The size of formwork joists was increased on subsequent projects to correct this problem. To evaluate this, the bottom slab thickness measurements were plotted to a full vertical scale and the extent of the sag was measured at several different stations. The measured formwork deflections generally varied from 0 to 1/4 in., and a net overrun of 0.6 cu yd in Pour A and 0.2 cu yd in Pour C can be attributed to form sag. In one location at Station 99+79.95 in Pour C near the south interior beam, a sag of 5/8 in. was indicated.

It was also noticed by field personnel that the wire hangers that support joists on beam flanges appeared to cut into the joists and cause the formwork to settle when the concrete was placed. In making slab thickness measurements of the bridge deck in place, it was observed that the bottoms

TABLE 1
ANALYSIS OF BRIDGE CONCRETE PROPORTIONING REPORTS
I 69 Structures Listed South to North

Project No.	Date	Report No.	Overrun, cu yd	Underrun, cu yd	Percent Variation	Wasted, cu yd
S03 of 12033A, Part 2	5-25-66	3	0.3		0.5	0.2
S04 of 12033A, Part 2	3-24-66	3	0.3		1.0	0.0
	3-25-66	4	0.0		0.0	0.0
	3-28-66	5	0.0		0.0	0.0
	3-30-66	6	0.9		10.0	0.0
	4-4-66	8	0.4		4.4	0.5
	4-12-66	14	0.5		1.7	4.5
	4-13-66	15	0.6		2.1	0.3
	6-27-66	24		4.1	7.0	5.5
	6-28-66	25		1.3	2.2	0.0
	6-29-66	26	3.7		10.6	0.0
	6-30-66	27	4.3		12.1	0.0
S11 of 12033A, Part 3	4-18-66	2	0.0		0.0	0.0
	4-19-66	3	0.0		0.0	0.0
	4-22-66	3	0.5		5.0	0.0
S06 of 12033B, Part 4	4-20-66	11	0.6		5.4	0.2
S03 of 12034B, Part 5	8-25-66	5	0.8		7.6	0.2
	8-29-66	6	0.6		6.3	0.0
	9-26-66	13		1.4	1.6	1.7
	9-27-66	14		3.7	4.3	0.0
	9-28-66	15		0.7	1.4	0.0
S04 of 12034B, Part 6	11-19-65	1	0.6		7.0	0.5
	11-22-65	2	0.6		2.5	1.0
	11-24-65	3	0.6		6.5	0.0
	11-24-65	4	2.1		8.8	0.0
	12-7-65	7	0.0		0.0	0.3
	12-8-65	8	0.1		0.4	1.0
	12-9-65	9	0.2		2.2	0.45
	6-22-66	20	4.6		8.3	0.0
	6-23-66	21	2.6		4.7	0.5
	6-24-66	22	4.3		6.0	1.5
S05 of 12034B, Part 7	2-9-66	1	0.0		0.0	0.6
	2-10-66	2	0.4		0.8	0.5
	2-11-66	3	1.25		11.9	0.25
	7-21-66	12	14.4		10.5	0.5
	7-22-66	13	4.2		10.4	0.5
B02 of 12034B, Part 3	4-21-66	9	1.6		4.3	0.5
	4-22-66	10	0.0		0.0	0.1
	4-28-66	12	0.85		4.62	0.75
	5-20-66	13	0.8		1.4	0.0
	5-24-66	14	3.05		5.3	0.75
B04 of 12034B, Part 4	3-28-66	1	0.1		0.3	0.0
	3-30-66	2	0.0		0.0	0.0
	4-6-66	6	1.35		3.38	1.25
	5-16-66	14	1.3		2.3	0.0
S07 of 12034B, Part 2	5-17-66	1	0.8		14.0	0.0
	5-18-66	2	1.2		2.7	0.0
	5-19-66	3	0.7		6.6	0.3
	8-4-66	15	12.8		10.0	1.0
	8-5-66	16	2.65		7.1	0.5
Total			76.65	11.2		

76.65 cu yd overrun
-11.2 cu yd underrun
65.45 cu yd net overrun

5 underruns
8 exact
37 overruns
30 overruns greater than 2 percent

of the beam flanges were not exactly flush with the bottoms of the concrete haunches. Unfortunately, accurate measurements of this variation were not obtained. However, it is estimated that this variation was no greater than 1/8 in. and most likely averaged about 1/16 in. Based on an average variation of 1/16 in., this settlement would have caused an overrun of 0.5 cu yd in Pour A and 0.3 cu yd in Pour C.

Review of Other Concrete Proportioning Reports

Upon concluding the Pearl Beach Road bridge measurements, it was decided to review some pour reports from other bridge projects in the area. Fifty pour reports from 10 projects were reviewed and are summarized in Table 1. It was found that 37 reports indicated overruns, 5 indicated underruns, and 8 indicated no variation from plan quantities. The net overrun reported in these 50 reports covering a total of 1,851.5 cu yd, was 65.45 cu yd, with 21 of these overruns amounting to less than 1.0 cu yd. No wastage of concrete was reported on 22 of these reports. From the pour reports reviewed, it appears that small overruns are frequently reported.

RECOMMENDATIONS

1. Concerning slab haunch thickness corrections. Proper adjustment of haunch thickness appears to be the most promising method of correcting for the uncontrollable factors affecting slab thickness, and consequent overruns. The procedure used, as explained by construction personnel during the preliminary discussions, consists first of determining the top-of-beam elevations of beams in place on the bearings before deck concrete is placed. Then an estimate of theoretical slab dead load deflections is subtracted from these elevations and the plan slab dimension (7 in. in this case) is added to the value obtained, giving the final predicted top-of-slab elevation. The theoretical top-of-slab elevation is then computed and compared with the predicted elevations. Finally, the haunch depth is adjusted to correct for deviations between the theoretical and predicted elevations, to insure that the slab will be of proper uniform thickness. It was mentioned that this is a difficult job involving considerable manual calculations. It is thought that this procedure would be considerably refined if the available electronic computer programs, such as in this case the two-span continuous beam deflection program and the straight bridge elevations program,

could be utilized by project engineers. It is recommended that if the haunches are to be adjusted, theoretical deflections be determined exactly along the entire beam length and the prorating or estimating of deflections be avoided.

2. Concerning concrete shrinkage-induced deflection. Further study of the problem of concrete shrinkage-induced deflections should be undertaken. The first step in such a study would be to modify the existing computer program used for analyzing two-span continuous beam deflection problems so that the effect of the shrinkage-induced bending moment and horizontal shear forces on continuous beam deflection can be analyzed.

The magnitude of shrinkage that will occur in Pour A in one of these two-span continuous structures prior to placement of Pour B, is a function of time and, therefore, this problem could possibly be relieved by decreasing the time lag between placement of Pours A and B. However, it should be noted that placing Pour B soon after Pour A, before Pour A concrete has gained adequate strength, will magnify the problem of concrete cracking in Pour A, resulting from the negative bending moment induced in Span 1 by the placement of Pour B in Span 2. It may become possible in the future to predict the shrinkage deflection effect as a function of time. With such information, it would be possible to allow for a predictable shrinkage deflection in precambering of beams if the pouring schedule could be controlled to a reasonable extent. The ideal construction method for preventing unbalanced shrinkage deflection would be simultaneous placement of A and B.

3. Concerning composite beam stiffness. This study indicates that if a delay of 24 hr occurs between placement of Pours A and B, the beams and slab in Pour A will act compositely to resist the negative bending moment induced in Span 1 by placement of Pour B in Span 2. This loading places the top of the slab in Pour A in tension, and increases the probability of crack development in Span 1. Placing Pours A and B simultaneously would eliminate this problem, but this does not seem practicable. The modified computer program being prepared to facilitate analysis of this problem should provide the bridge designer with an adequate means of predicting these deflections.

It is understood that shoring may be used on future projects. If shoring were to be placed under Pour A and then removed after adequate concrete strength is obtained in Pour A, the beams would then act compositely in Span 1 under the loading of Pour A and compressive stresses would occur there in the top of the slab. Pour B could be placed and probably very little tension cracking would occur in Span 1, particularly at mid-span, because existing compressive stresses in the concrete would need to be relieved before any tension could occur. Furthermore, for purposes of predicting slab load deflections, the shoring procedure as described would enable the designer confidently to predict beam stiffness for different phases of slab construction.

4. Concerning predominance of small overruns. One important factor involved in this problem is the estimation of wasted concrete. If these small overruns are of concern, it is suggested that a more positive method of determining wastage be introduced.

5. Concerning screed machine riding up on slab surface. It was observed that on some subsequent projects, the screed ordinates were reduced from 1/8 to 1/4 in. to compensate for the screed's riding up on the concrete slab surface. On the Pearl Beach Road structure, this was done in Pour B in Span 2. The actual top-of-road profile measured there showed that no hump occurred in the slab along the centerline of the roadway. However, it was noted that high spots occurred along the curb lines. Based on these observations, it appears that this practice of slightly reducing the screed ordinate offers at least a partial solution to this problem. This practice should be carefully controlled, however, to insure that minimum concrete cover over the reinforcing bars will be maintained throughout the entire slab.

REFERENCES

1. Swayze, M. A. "Early Concrete Volume Changes and Their Control." ACI Journal, p. 425, April 1942.

2. Zuk, William. "Thermal and Shrinkage Stresses in Composite Beams." ACI Journal, p. 327, September 1961.

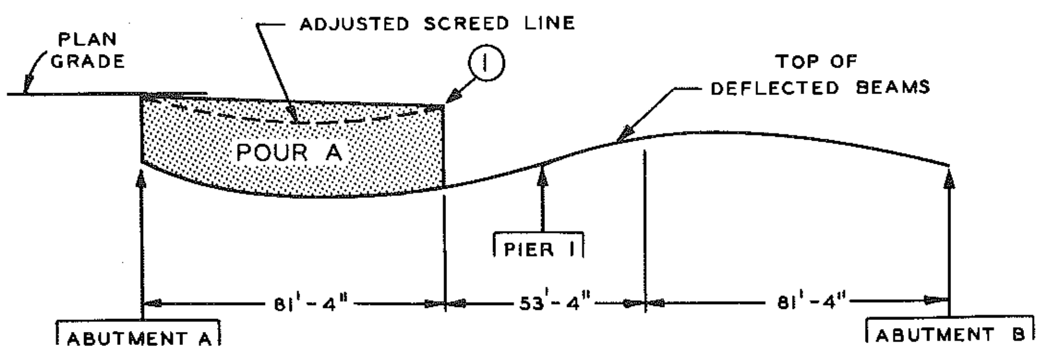
APPENDIX A

Screed Correction for Continuous Beam Rebound

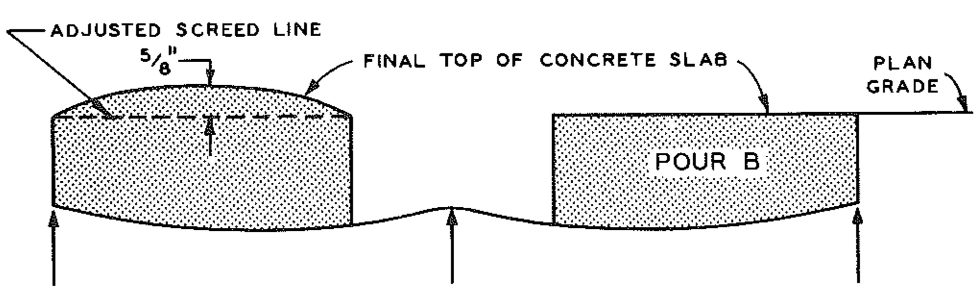
When a simultaneous and symmetrical pouring sequence is not utilized on a continuous structure, a correction is required in the longitudinal screed ordinates over Pour A to provide for the continuous action of the beams under the deck. This problem can be understood by considering the action of a hypothetical bridge deck, shown below, having a flat grade of 0 percent, and beams containing 0 in. precamber that are continuous from Abutment A over Pier 1 to Abutment B.

When Pour A concrete is placed, the top of the bridge deck slab is screeded to the final plan grade, neglecting for the purpose of this discussion the small deflection that occurs at Point 1. Then Pour B is placed, and because of the continuous action of the beams, the added slab load in Pour B causes the beams under Pour A to deflect the deck slab upward, causing the top of the deck slab in Pour A to rise above the planned final grade.

It was determined by the Office of Design that on actual two-span continuous bridges with 26-ft roadways this rebound under Pour A would cause an overrun of about 5 percent in that pour. To correct this problem the longitudinal screed over Pour A was adjusted on subsequent projects, as illustrated below by the dashed line.



BEAMS DEFLECT WHEN POUR A IS PLACED



BEAMS REBOUND UNDER POUR A WHEN POUR B IS PLACED

APPENDIX B

Bridge Concrete Proportioning Reports
Pearl River Road over I 69 (S11 of 12033A, C9)

Form 340
(Rev. 7/64)

MICHIGAN STATE HIGHWAY DEPARTMENT

OFFICE OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO. S11 of 12033 A, C9
ROUTE Pearl Beach CROSSING I-69
CONTRACTOR Canonie Construction Co.
REPORT NO 13 DATE 7-25-66

Name and Location of Concrete Proportioning Plant Certified Transit Mix Inc.
Source of Cement Medusa Silo # ____ Car # ____ Truck # ____
Source of Aggregate: Fine Stukey 12-31 6AA Am. Agg. 47-3 6B ____
Plan Volume of Pour 64.1 Cu. Yds. Grade AA Chart No. 66 MV 574 Consistency: ☒ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt. per cu. ft. (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	Slump—inches	Estimated sacks of cement per cu. yd. of concrete
6:30	104	188.5	328	45	4.4	1.0	6	1181	1988	200.5	*24.1	24	1.15	4"	6

Weather: A.M. Clear 80°
P.M. ____
Pour A Span 1
Time, from 8:00 A.M. to 12:00 P.M.
Number of Test Beams 2 Series 3
Date water gauge checked 6-1-66 Result ok
Date batch scale checked 6-1-66 Result ok
Date Auto. Controls checked 7-25-66 Result ok

(A) Actual Volume of Pour 64.1 cyds
(B) Actual Quantity Placed by Batch Count 78.5 cyds
(C) Difference between A & B 14.4 cyds*
(D) Overrun or Underrun (C/A) 22.5%
(E) Total Concrete Batched 80 cyds
(F) Excess (E minus B) 1.5 cyds*
(G) Total Cement Batched 120 bbls
*Explain under Remarks

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
6:00	70°	--	--	--	82°		
8:00	80°				Acme 7.0	Chace	10.0
8:15						Acme	7.3
8:40						Chace	8.3
8:45						Chace	7.9

REMARKS * Approx. 10% water cut per load as per manufacturer's recommendation when using Sika Plant. 4 oz. Sika Plant added per sack of cement. High over-run due to over deflection of Plate Girder Beams.
*(F) Wasted using white curing membrane.

Prepared by Clyde Gilbert (Inspector) Checked by R. Welke (Project Engineer)

One gallon of water is equivalent to 8.33 pounds.
Reports are to be filled out completely and mailed not later than the day following the cast.
Original to District Materials Supervisor, Copy to Br. Constr. Engr., Copy to Dist. Br. Engr., Retain one copy.

Form 340
(Rev. 7/64)

MICHIGAN STATE HIGHWAY DEPARTMENT

OFFICE OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO. S11 of 12033 A, C9

ROUTE Pearl Beach CROSSING I-69

CONTRACTOR Canonie Construction Co.

REPORT NO 14 DATE 7-26-66

Name and Location of Concrete Proportioning Plant Certified Transit Mix, Coldwater

Source of Cement Medusa Silo # Car # Truck #

Source of Aggregate: Fine Stukey 6A A Am. Agg. 6B

Plan Volume of Pour 64.1 Cu. Yds. Grade AA Chart No. 66 MV 574 Consistency: ☒ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt. per cu. ft. (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	Slump—inches	Estimated sacks of cement per cu. yd. of concrete
6:00 A.M.	104	188.5	328	45	4.4	1	6	1180.8	1987.7	200.5	24.1	24	1.15	4"	6

Weather: A.M. Clear 85°

P.M.

Pour B Span 2

Time, from 6:30 A.M. to 10:30 A.M.

Number of Test Beams 0 Series 0

Date water gauge checked 6-1-66 Result ok

Date batch scale checked 6-1-66 Result ok

Date Auto. Controls checked 7-25-66 Result ok

(A) Actual Volume of Pour 64.1 cyds

(B) Actual Quantity Placed by Batch Count 72.0 cyds

(C) Difference between A & B 7.9 cyds*

(D) Overrun or Underrun (C/A) 12.3 %

(E) Totol Concrete Batched 75 cyds

(F) Excess (E minus B) 3 cyds*

(G) Total Cement Batched 112.5 bbls

*Explain under Remarks

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
6:00	70°						
6:35	70°				84°	Acme	4.8
7:10						Acme	5.0
8:00						Acme	8.5
8:25					Chace 8.0	Acme	6.9

REMARKS Water content of concrete reduced approx. 10% as recommended by manufacturer of Sika. 4 oz. of Sika added per sack of cement.

*(F) Wasted High over-run due to over deflection of beams.

Prepared by Al Decker (Inspector) Checked by R. Welke (Project Engineer)

One gallon of water is equivalent to 8.33 pounds.

Reports are to be filled out completely and mailed not later than the day following the cast.

Original to District Materials Supervisor, Copy to Br. Constr. Engr., Copy to Dist. Br. Engr., Retain one copy.

Form 340
(Rev. 7/64)

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS

OFFICE OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO. S11 of 12033 A, C9
ROUTE Pearl Beach CROSSING I-69
CONTRACTOR Canonie
REPORT NO 15 DATE 8-28-66

Name and Location of Concrete Proportioning Plant Certified Transit Mix Inc.
Source of Cement Medusa Silo # ______ Car # ______ Truck # ______
Source of Aggregate: Fine Stukey 12-31 6AA American 47-3 6B ______
Plan Volume of Pour 37.1 Cu. Yds. Grade AA Chart No. 66MV 574 Consistency: ☒ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt. per cu. ft. (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart			Moisture Percent		Total Computed Mix				Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	Slump-inches	Estimated sacks of cement per cu. yd. of concrete
		Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Sand	Coarse Aggregate	Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds					
6:30	104	188.5	328	45	4.4	1.6	6.0	1181	2000	188.7	22.6	21.0	1.15	4"	6

Weather: A.M. Over cast 74
P.M. ______
Pour C-Spans 1 & 2
Time, from 8:20 to 10:20
Number of Test Beams ______ Series ______
Date water gauge checked 6-1-66 Result ok
Date batch scale checked 6-1-66 Result ok
Date Auto. Controls checked 7-25-66 Result ok

(A) Actual Volume of Pour 37.1 cyds
(B) Actual Quantity Placed by Batch Count 43.5 cyds
(C) Difference between A & B 6.4 cyds*
(D) Overrun or Underrun (C/A) 17.2 %
(E) Total Concrete Batched 45.0 cyds
(F) Excess (E minus B) 1.5 cyds*
(G) Total Cement Batched 67.5 bbls
*Explain under Remarks

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
6:45	72						
8:20	77				84	Acme	5.8
8:45						Acme	5.7
9:00						Chace	6.8
9:30						Chace	6.0

REMARKS 1/8 Darex added per sack of cement.
* 3 oz. Sika plant retarder added per sack of cement, water cut approx. 10% as per manufacturer's recommendation.
* (F) Wasted.
Can not explain over-run at this time.

Prepared by D. Wilson (Inspector) Checked by R. Welke (Project Engineer)

One gallon of water is equivalent to 8.33 pounds.
Reports are to be filled out completely and mailed not later than the day following the cast.
Original to District Materials Supervisor, Copy to Br. Constr. Engr., Copy to Dist. Br. Engr., Retain one copy.