FACTORS AFFECTING GIRDER DEFLECTIONS DURING BRIDGE DECK CONSTRUCTION

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Problems involved in obtaining the desired thickness of bridge decks were investigated. The study, which was limited to decks longitudinally screeded during construction, included (a) field measurements of the girder deflections during construction and (b) theoretical frame analysis of the girder deflections under the field-loading conditions. Two simply supported steel-plate girder spans were investigated. When full-span length longitudinal screeding is used, the finished grade elevations are set on the screeding edge of the machine and remain independent of the bridge girder deflections, and thus the forming elevations, will in turn have a bearing on the final thickness of a bridge deck. In addition, all factors that in effect cause the deck forming to be too high at the time the concrete is screeded to grade have the potential of causing an inadequate deck thickness. The most significant factors were found to be (a) plan dead-load deflection values that are in error, (b) differential temperatures existing between the top and bottom flanges of the girders during concrete placement as opposed to those that may have existed when the forming elevations were established, and (c) the transverse position of the concrete dead loading at the time a final screeding pass is made over a given point on a span.

•AS bridge design trends have tended toward longer, more flexible spans and as construction techniques have become more sophisticated, the design thickness of bridge decks is often more difficult to obtain during construction. When deficient deck thicknesses occur, there are virtually no reliable corrective measures for restoring lost structural strength; where insufficient cover over the reinforcing steel results, permanent maintenance problems may develop.

During bridge deck construction there are two basic methods for screeding the concrete deck to grade: the transverse and the longitudinal (by nature of the screeding machine's orientation to the alignment of the bridge). This study was concerned only with the longitudinal placement and screeding technique, which is used by many contractors.

Longitudinal screeding machines, such as the one shown in Figure 1, are most often used on simple spans 100 ft or less in length. The transverse screed rails supporting the machine are normally set to the finished grade at each end of the span. The finished grade of intermediate points on the deck are set on the longitudinal strike-off edge of the screeding machine. If the structure of the machine is stable, then the elevations remain fixed and are independent of the girder deflections occurring during concrete placement. Consequently, the final thickness of the bridge deck will be dependent on the deflections of the girders at the time the concrete deck is struck off the grade. Accordingly, all factors influencing the girder deflections during construction have a direct bearing on the final thickness of a bridge deck.

When longitudinal screeding is used, one factor of concern involves the effect on deflections of interconnecting diaphragms between the bridge girders. Conventional

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procedures for computing plan dead-load deflection values normally assume that diaphragm connections are hinged, i.e., that each girder is free to deflect independently under the dead load of that portion of the concrete deck it would carry. When concrete is placed down one side of a bridge span, as is the case when a deck is to be longitudinally screeded, the deflections of girders directly under the load will be partially restrained by the interconnecting diaphragm action with the unloaded girders. Thus, if the concrete deck is struck off to grade over one girder before concrete is placed over the remaining girders, then the deflection of this girder will not be as much as calculated, and the deck will not be adequate. An earlier theoretical analysis (1) and field investigation (2) indicated that deficient deck thicknesses could result where longitudinal screeding follows too closely behind concrete placement. These studies, however, assumed full rigidity at all diaphragm connections. For bolted connections, the assumption of a rigid joint may not be applicable under the variable loading conditions that exist during deck placement. This paper presents the results of field measurements and a theoretical analysis of semirigidly connected simple-span bridge girders that were used to investigate the actual versus the theoretical deflections occurring during bridge deck construction.

Other factors that could have a bearing on girder deflections during construction were investigated. These included thermal effects such as the heat of hydration of the concrete during deck placement and solar heating of the top flanges of the steel girders prior to concrete placement. To determine the order of magnitude of the influence of the thermal factors, we measured temperatures on the steel girders during the field investigations.

PURPOSE AND SCOPE

The following were the main objectives of the study:

1. To investigate the girder deflections at progressive stages of concrete deck placement and to evaluate the adequacy of the conventional method of computing plan dead-load deflections for a bridge deck that is to be placed and screeded longitudinally over the full-span length;
2. To estimate—by use of a comparison of the theoretical and field data—the degree of diaphragm connection rigidity on the particular spans studied;
3. To investigate the theoretical effects of diaphragm connection rigidity on the deflections of a girder system and to compare the results with field deflection data obtained during progressive stages of deck placement; and
4. To obtain field data on the differential thermal conditions between the upper and lower flanges of steel girders due to solar heating prior to deck placement and the subsequent hydration heat of concrete.

The general scope of the study was limited to simple-span steel-girder bridges with bolted diaphragm connection type designs. The study was limited to bridge decks constructed by use of longitudinal placement and screeding of the concrete.

STRUCTURES STUDIED

One span on each of 2 bridges was instrumented for field study. These were span 3 of the Route 607 and span 4 of the Route 15 bridges over Interstate 64.

The Route 607 span was composed of 6 parallel girders; the Route 15 span was composed of 7. Figure 2 shows the dimensions and the locations of the test instrumentation for the Route 607 span.

INSTRUMENTATION, TESTS, AND PROCEDURES

The data collection techniques and measurement devices were selected to be of minimum obstruction and delay to the contractors during construction of the bridge and roadway grading. For obvious reasons, concrete placement operations could not be delayed for long periods of time. Thus, the number of measurements taken during each delay in deck placement was limited to that which could be handled in approximately 10 to 15 minutes.

Instrumentation

A high-precision modified Wild "N-III" level, capable of direct readings to 0.001 in., was used for measuring the girder deflections. The level was mounted on a trivet that in turn was set in stationary lugs on the top of a pier cap at one end of the test span. In addition, the level was centered on the cap directly above one of the circular pier columns.

Special design rod and scale units (Fig. 3) were attached to the lower flanges of each girder at the quarter-span and midspan length points. Engineer's scales with ½-in. major graduations were attached to the rods, and a reference scale was mounted to the pier cap at the end of the span opposite the position of the level instrument.

A 24-channel Honeywell thermocouple recorder powered by a portable generator was used to collect temperature data on the steel girders. Thermocouples were placed on the top and bottom flanges of the girders at the quarter-span and midspan length points. The top flanges were emphasized, however, because it was expected that temperature variations would be greatest on the top side due to the concrete placement operation and other factors affecting the sun's radiation. A complete cycle of 24 thermocouples was made every 12 minutes during operation of the recorder.

Both of the structures instrumented were designed with neoprene bearing pads located at the expansion ends of the spans. The dead-load deflections of these pads were measured by using dial gauges that were set as close to the centerline of bearing of each girder as was possible. In addition, deflection measurements were taken at a fixed-end steel bearing point to determine the order of magnitude of the vertical movement at these types of assemblies.

Strain measurements, which will not be discussed in this paper, were taken on some of the diaphragm members as indicated in Figure 2.

Test on the Plastic Concrete

Tests made on the plastic concrete were restricted to the measurement of those properties that would have the most direct influence on the bridge girder deflections during deck placement. The following tests and measurements were made on both spans.

1. The time of initial and final set (ASTM C403-68) was run on 3 representative batches of the deck concrete.
2. Unit weight determinations (ASTM C138-63) were made at intervals selected to be generally representative of the concrete placed in each area between the girders.
3. The temperature of the concrete was measured at discharge from the mixer trucks, and the ambient air temperature was recorded continuously during the placement operations.

Field Study Procedures

Initial readings were taken on all systems just prior to the beginning of deck placement operations. Subsequent measurements were taken by delaying placement operations when the concrete deck load was, as nearly as practical, midway between adjacent girders (with the exception that the first delay for measurements was made between the second and third girders from the beginning side of the span). Final measurements were taken when all the concrete was in place with the exception of the thermal data, which were collected for several hours after completion of the decks.

Temperature data were recorded automatically throughout the placement operations. In addition, temperature and deflection measurements were taken on the Route 607 span several days prior to concrete placement to investigate the independent effects on girder elevations of differential temperatures resulting from solar radiation.

During deck concreting, a record of the time and sequence of events was maintained. With the exception of the placement delays for measurements, the contractor's normal procedures were used during construction.

The deck concrete on each of the 2 spans was placed during warm and sunny weather. The air temperature during decking operations ranged from 66 to 89 F and from 64 to 92 F respectively for the Route 15 and Route 607 spans.

Figure 1. Longitudinal bridge deck screeding machine.

Figure 2. Locations and dimensions of test instrumentation for Route 607 span.

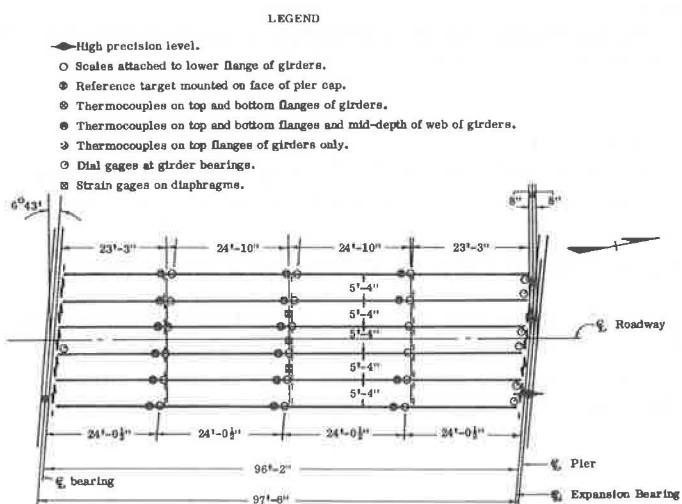
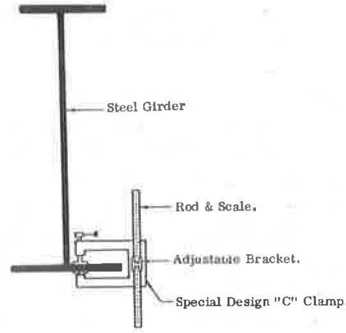


Figure 3. Typical rod and scale unit attached to the lower flange of a bridge girder.



ROUTE 607 SPAN RESULTS

Solar Radiation and Thermal Differentials

The Route 607 span runs in a general north-south direction. Accordingly, the morning sun falls on the east side of the superstructure and gradually passes over to the west side in the afternoon. During several sunny days in June and July, differential thermal and deflection readings were taken on the girders while only the deck forming was in place. With the deck forming in place, of course, the lower flanges of the interior girders were shielded from the sun. The exterior girder on the east side was exposed to the sun in the morning, and the exterior girder on the west side was exposed to the afternoon sun. In addition, the vertical forming on each side of the span tended to shield the top of the east girder in the morning and the top of the west girder late in the afternoon. A transverse section of the steel framing of this span is shown at the top of Figures 4a and 4b, which show respectively the average differential temperatures recorded between the top and bottom flanges of the girders and the resulting upward midspan deflections of the girders.

At 7:00 a.m. on July 1, the temperature differential between the top and bottom flanges was virtually neutral (Fig. 4a), and the corresponding girder elevations at midspan were recorded at that time and used as a reference (Fig. 4b). Comparisons of the temperature differentials at 10:00 a.m., 1:15 p.m., and 3:45 p.m. with the corresponding midspan deflections generally show that the upward deflection of the steel girders increases with increasing temperature differentials. In addition, a transfer of the thermal loading between girders via the diaphragm connections is indicated by the smooth transverse deflection pattern. Upward midspan deflections of 0.43 in. were recorded on girders 5 and 6 at 3:45 p.m. All the girders reached an upward deflection level of approximately $^3/_8$ in. above the reference level during the early afternoon. As will be discussed in more detail later, thermal deflections of this order of magnitude could have a significant bearing on bridge deck thicknesses.

It can also be noted that the differential temperatures varied transversely across the span width because of its orientation to the angle of the sun. Thus, not only did the midspan girder elevations vary significantly in magnitude, but also the slope of the transverse pattern of upward deflections reversed during the course of the day. This transverse "warping" effect, due to the sun moving toward the west, is shown in Figure 5, where the midspan girder elevations for 2 days are referenced to the elevations existing at 12:00 noon. A difference in the relative elevation of girders 6 and 1 on the order of $^1/_4$ in. occurred between 12:00 noon and 3:45 p.m. on June 30. It can also be noted from Figure 5 that, during days of similar climatic conditions and at nearly the same time of day, the differential temperatures and thus the upward deflections of the girders are quite similar. For the 2 comparative days illustrated, the maximum difference in elevation was $^1/_{32}$ in. at girder 6. It might be concluded from these data that, for 2 days having similar weather, temperature, and solar conditions, the elevations of the girders will be close to identical at approximately the same time of day. It is apparent, however, that exact girder elevations cannot be established when any degree of solar radiation is present.

During the thermal studies, temperatures on the order of 120 F were measured on the top flanges of the girders, but at middepth of the web the temperatures recorded were about the same as those on the lower flanges. It is likely that some of the heat from the top flanges is conducted down into the web but becomes insignificant before reaching the middepth level.

Although the maximum temperature differentials recorded between the upper and lower flanges in this study were on the order of 25 F, it is possible to experience differentials of a higher order of magnitude. In a study of the thermal behavior of a box section type bridge in the London area, for example, Capps (3) has reported extreme temperature differentials on the order of 50 F.

It is important to note that solar radiation can cause changes in the elevations of bridge girders during daytime deck placement operations. When girder elevation changes are considered relative to the initial elevations measured for calculation of forming elevations, significant deck thickness can be lost if the span is longitudinally

Figure 4. Upward midspan girder deflections due to differential temperatures between the top and bottom flanges of each girder.

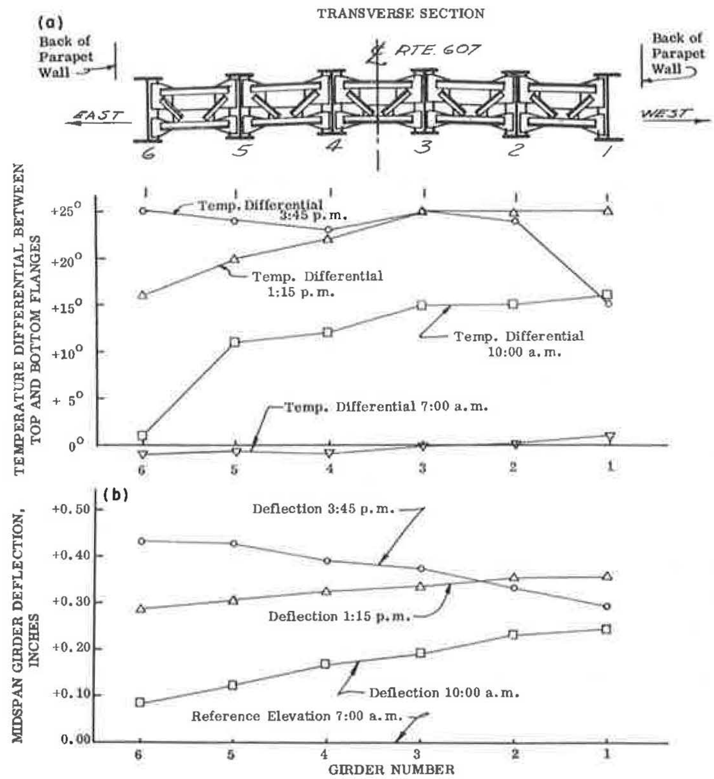
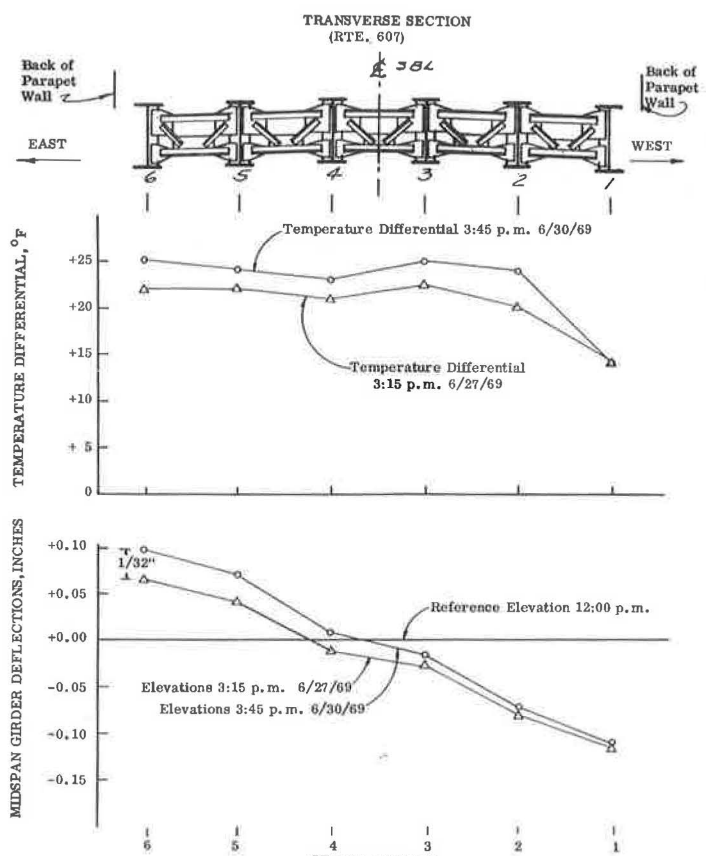


Figure 5. Midspan girder deflections and temperature differentials.



screeded (Fig. 6). If no temperature differential exists between the top and bottom flanges of a simply supported bridge girder, it is in a thermally neutral position (Fig. 6a). Under conditions of solar radiation, differential temperatures will generate an expansive force, F, in the upper flange, which is resisted by an opposing force in the lower flange to create a bending moment, M. The resulting effect is an upward deflection of the girder (Fig. 6b). If the deck forms are established to grades complying with the neutral position of the girder (but the concrete deck is screeded to grade under differential thermal conditions), the thickness of the deck will be decreased by an amount Δ (Fig. 6c).

The effects of solar radiation can be minimized by doing the following: Deck forming elevations should be established when the thermal conditions on the girders will approximate those anticipated at the time of concrete placement; and/or the deck forms should be adjusted vertically at a time when the thermal condition of the girders will approximate the condition expected to prevail at the time the concrete is screeded to grade. The latter precaution is important because the in-place forming will shield the lower portion of the girders from solar radiation and thus cause high differential temperatures on hot, sunny days. Solar thermal effects can be virtually negated, of course, by very early or very late deck placement operations.

In addition to solar radiation, other general factors such as the temperatures of the plastic concrete and the ambient air can cause thermal differentials during deck placement. Typical data showing the net effect of all these factors are discussed later.

Plan Girder Deflections

Deflections given on the bridge plans for simply supported spans are usually calculated by assuming each girder to be free to deflect as an individual unit. Thus, plan dead-load deflections are calculated by assuming that each interior girder, for example, will carry an equal portion of the concrete deck. By using this method, we found that the midspan deflections for the Route 607 interior girders were equal to 1.0 in. The plans, however, gave the value as $1^5/_8$ in., or 0.63 in. too high. Had the plan value been used, the forms would have been set too high; and with the longitudinal screeding the deck thickness would have been deficient by 0.63 in. (assuming that the correct conventionally calculated deflection represents the true situation and that all thermal factors are neglected). However, an inadequate deck thickness had resulted earlier on another bridge deck, and the contractor had made adjustments in the forming elevations to avert a similar occurrence on the test spans.

Plan deflection errors on the high side, as other studies (2) have shown, are a major cause of inadequate deck thickness when longitudinal screeding is used and would have caused a deficient deck on the study spans if adjustments had not been made during construction.

Field Versus Theoretical Girder Deflections

The field deflection measurements taken during the deck placement operation naturally incorporate the existing thermal conditions on the girders. Accordingly, the actual midspan deflections of the girders for each deck loading increment are shown in Figure 7. Additional measurements taken approximately 3 hours after completion of the deck finishing (2:55 p.m. data, Fig. 7) clearly show that continued heating of the top girder flanges results in an upward deflection of the span. Viewed as a proportion of the total dead-load deflection at 11:40 a.m., this average 18 percent "thermal uplift" demonstrates the remarkable forces generated by thermal differentials.

The general transverse pattern of the midspan girder deflections for all loading intervals shows that the structural steel framing is acting as a unit due to the diaphragm connections between the girders. Thus, the basic questions are as follows:

1. How do the actual deflections for each loading increment compare to those conventionally calculated?
2. How do the actual deflections compare with those computed by assuming rigid or semirigid connections between all the girders?

To study the latter question, we used a theoretical analysis of deflections of semirigidly connected girders. This analysis, which was developed by Lisle (4), utilizes a modified stiffness matrix and has been programmed in Extended ALGOL 60 for solution on a Burroughs B5500 computer. The program can be used for computing deflections of bridge girder systems with any degree of end fixity at the diaphragm connections. Thus, an end fixity factor of 1 would represent a rigid connection, and 0 would represent a pinned connection. Any value between 0 and 1 would represent a semirigid connection.

In using the program, the structural framing of a span is considered as a series of segments—each segment usually terminating at a connection. The moments of inertia of each segment are calculated by using conventional procedures. The differential thermal conditions existing on the girders at each loading increment can be accounted for in the program by applying calculated moments at the girder ends (as shown in Fig. 6) and at changes in the sectional dimensions of the girders.

For the loading on the frame, the actual unit weights of the concrete were used. The total weight of the concrete was determined from the unit weights and the volumes placed in each loading increment. The total weight was proportioned to each girder according to the plan dimensions. Because much of the concrete on the span had not been screeded to grade at each loading increment, this procedure was considered to be reasonable. Thus, the programmed loading corresponded as nearly as was practical to that existing during the field deflection measurements.

Diaphragm Connection Rigidity

A thorough theoretical analysis of a wide range of end fixity factors (EFF) was made for each loading increment shown in Figure 7. In general, very little difference was found between the deflections obtained by assuming EFF's ranging from 0.10 to 1.0. An EFF of 0.20, however, appeared to match the actual deflection patterns the closest. Consequently, the theoretical comparisons presented in this paper are based on semirigid diaphragm connections having an EFF of 0.20. For most practical purposes, this could be assumed to be virtually a rigid connection.

Actual and Theoretical Deflection Comparisons

Figures 8 through 11 compare the actual and the computed midspan deflections for each loading increment. The computed deflection values are shown both excluding and including the superimposed differential thermal conditions on the girders. The conventional deflections are based on a unit weight of 150 lb/ft^3 for concrete, which is commonly used for calculating plan deflections. The following observations are made from the data presented in the figures for several of the loading conditions.

1. In general, the deflections by the frame analysis including thermal conditions are in very good agreement with the field deflections.
2. The frame analysis excluding thermal conditions shows that the girder deflections would be considerably greater if differential thermal conditions did not exist.
3. Both the field and the frame analysis results were markedly different from the conventionally calculated deflections.
4. With reference to girder 2 during the third loading increment (Fig. 8), it can be observed that the concrete placement was 2 to $2\frac{1}{2}$ bays ahead of the final pass of the screeding machine over girder 2. (A bay is defined as the distance between adjacent girders.) Excluding thermal effects, the frame analysis deflection value is very nearly equal to the conventional deflection. With reference to the same point on Figure 9, which would constitute a 3- to $3\frac{1}{2}$-bay lead, the deflection value for girder 2 is almost identical to that observed under the loading condition shown in Figure 8. Thus, the greater is the lag of the final screeding pass, the greater is the chance of the actual deflections being the same as the conventional plan deflections and the less the chance of the deck thickness being inadequate. For the span in question, a final screeding pass lag of 3 bays behind concrete placement appears to be ideal when conventionally calculated deflections are used for establishing forming elevations.
5. General observations from the data presented indicate that the final screeding pass over the concrete averages a 2- to 3-bay lag behind concrete placement. Quite often, however, only a 2-bay lag was noted over some areas.

Figure 6. Possible effects of solar radiation on bridge deck thickness when longitudinal screeding is used.

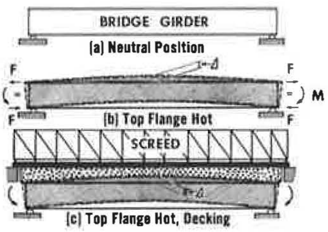


Figure 7. Midspan girder deflections due to concrete deck placement (Route 607, span 3).

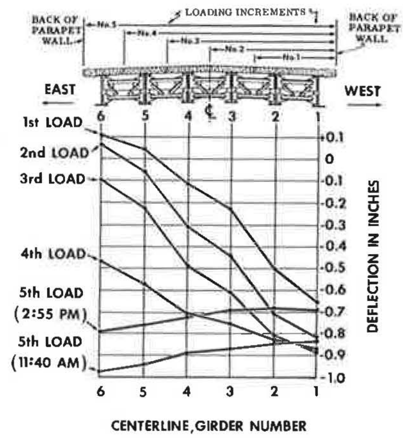


Figure 8. A comparison of actual field midspan deflections with computed deflections (loading interval 3, 9:15 a.m., Route 607, span 3).

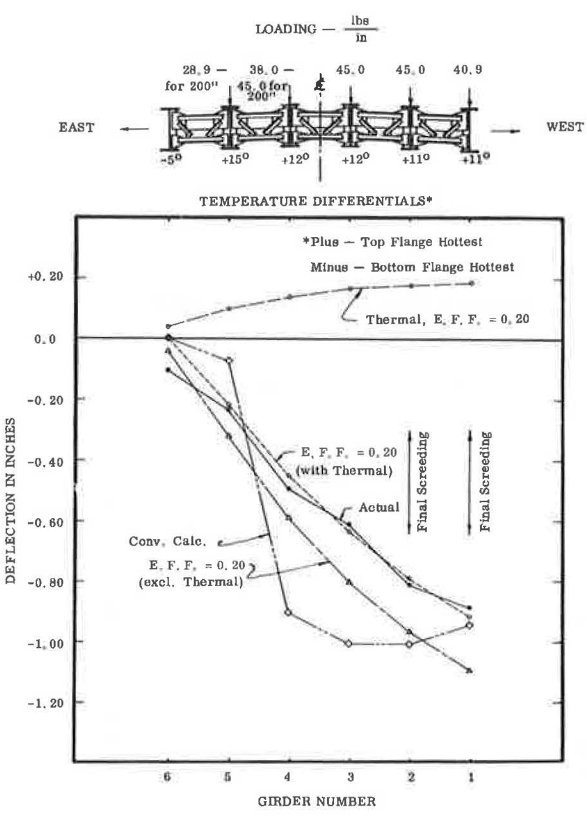


Figure 9. A comparison of actual field midspan deflections with computed deflections (loading interval 4, 9:45 a.m., Route 607, span 3).

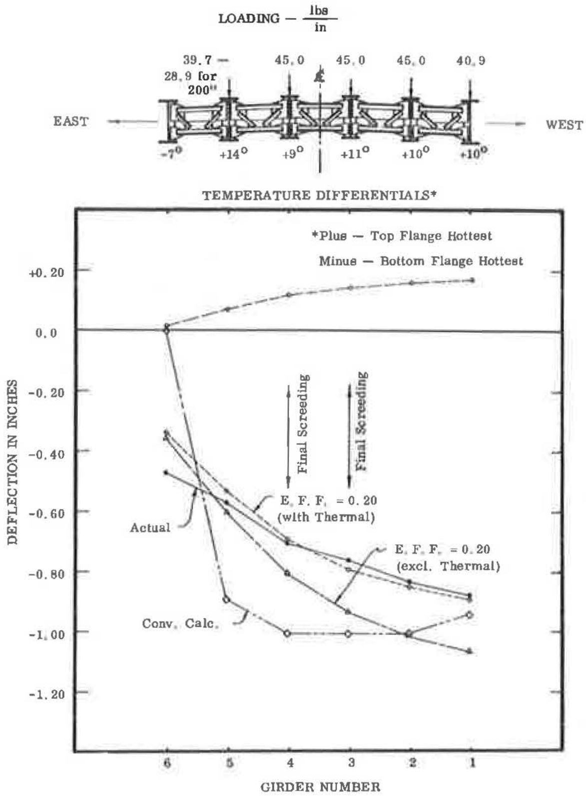


Figure 10. A comparison of actual field midspan deflections with computed deflections (loading interval 5, 2:50 p.m., Route 607, span 3).

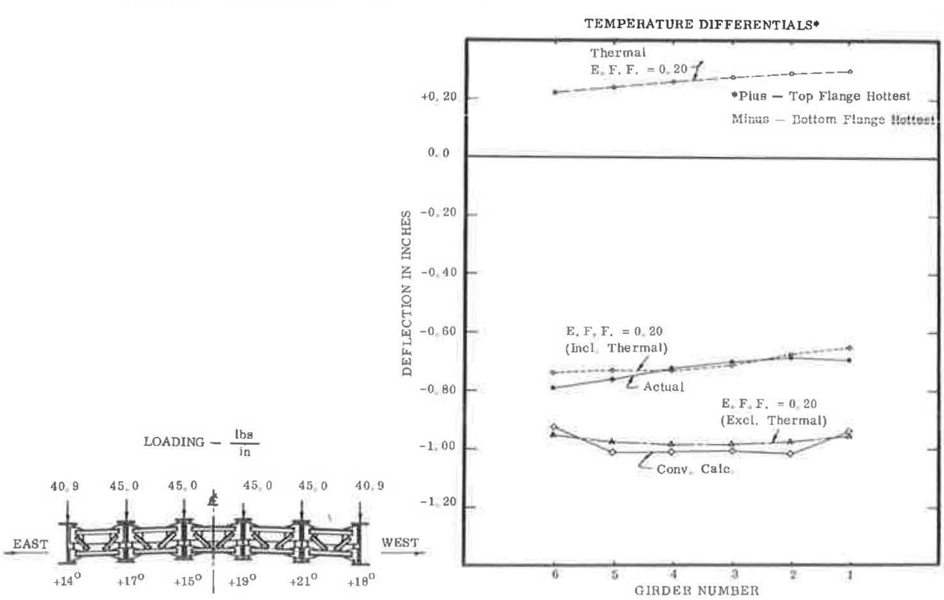
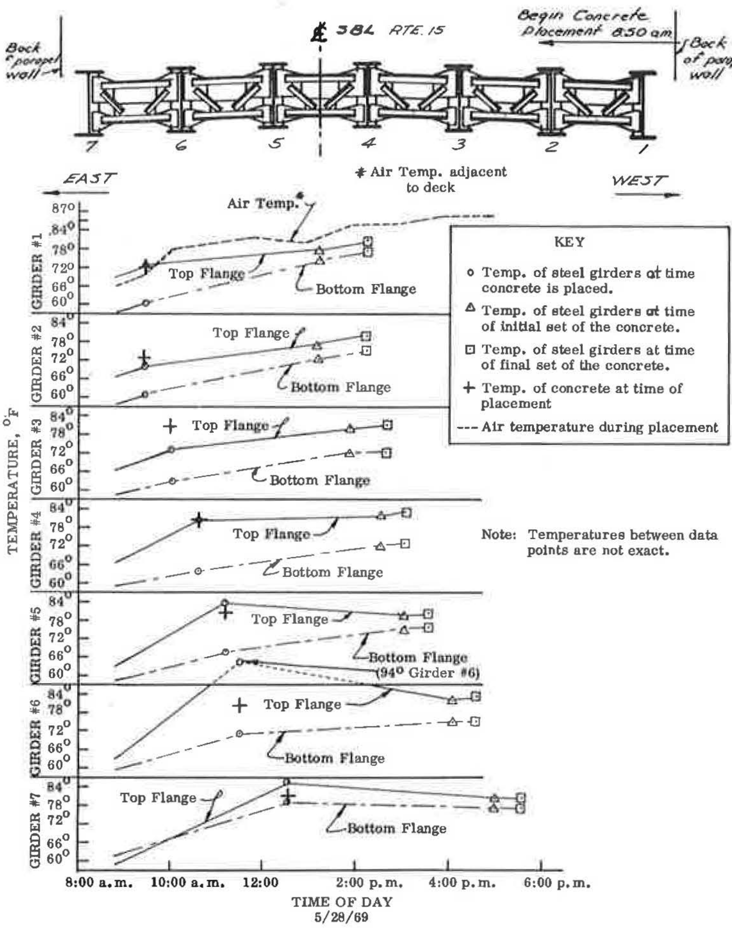


Figure 11. Temperature conditions during bridge deck placement (Route 15, span 4).



6. In one instance, the final pass of the screed was made with only a 1-bay lag. The difference between the field and conventional deflections at the final pass over girder 4 is 0.30 in. (Fig. 9). The deck thickness could possibly have been deficient by 0.30 in. in that vicinity if the forms had been set utilizing conventional deflections and if the initial girder elevations (taken for bolster calculations) had been measured when the girders were in a thermally neutral condition.

7. If we consider a hypothetical situation that would include the same conditions listed in the preceding situation, an average 0.40-in. deficient deck thickness could occur between girders 3 and 4 during the third loading increment (Fig. 8) if only a 1-bay lag were used in screeding the deck to grade.

8. The thermal uplift of the span, which occurred in a 3-hour period subsequent to completion of the deck finishing, was verified by the frame analysis results, which check very closely with the field deflections at that time (Fig. 10). The final midspan deflection of the span as calculated from the frame analysis is in excellent agreement with the conventionally calculated deflection (Fig. 10).

Bearing Pad Deflections

The results of measurements made at the neoprene expansion bearing pads during placement of the concrete on the Route 607 span indicated only slight vertical movements. The greatest pad deflection, 0.035 in., occurred under the first girder loaded with the plastic concrete. In general, the first pads that were loaded compressed the most, and the last several pads that were loaded compressed to a lesser degree. The average pad compression, which was on the order of 0.02 in., would have an additive but insignificant effect on deck thickness and does not warrant consideration in design or field calculations.

Measurements taken at a steel bearing assembly on the fixed end of the span indicated very slight vertical movements. The maximum compression measured was 0.01 in.—considerably less than the average at the neoprene bearings.

ROUTE 15 SPAN RESULTS

The results of the field measurements and data analysis on the Route 15 span were much the same as those presented for the Route 607 span. Because the latter results have been discussed in considerable detail, treatment of the Route 15 data will be confined to observations of the thermal conditions during deck placement.

Thermal data taken during placement of the Route 15 span are shown in Figure 11, and the following observations are presented.

1. The data clearly indicate that temperature differentials exist on the steel girders during summer daytime deck placement operations.

2. The temperature on the top flanges of the girders increased rapidly because of solar radiation until the concrete was placed over them.

3. After the concrete had been placed, the average rate of rise of the temperature on the top flanges was usually less than that of the ambient air temperature. On girders 5, 6, and 7, where the concrete was placed after 10:30 a.m., a net cooling of the top flanges prior to final set (4,000-psi penetration resistance) resulted.

4. The general rate of temperature rise on the bottom flanges was slightly less than the average rate of increase in the ambient air temperature (with the exception of girder 6 on the east side, which was exposed to morning solar radiation). The temperature on the bottom flanges of the girders was always less than the ambient air temperature adjacent to the deck.

Although the heat of hydration of the concrete prior to final set may have had some effect on the increased temperature of the top flanges of girders 1, 2, and 3, the evidence suggests that solar radiation, increasing ambient air temperature, and initial temperature of the plastic concrete were responsible for the increase.

DISCUSSION OF RESULTS

Although there was usually a reasonable lag between concrete placement and the final longitudinal screeding pass over a given area, occasionally there was only a 1-

or 2-bay lag. There would appear to be a need to compensate for such instances when conventional plan deflections are used to establish forming elevations. If one considers the hypothetical situation discussed in the Route 607 results, a 40 percent reduction in the plan girder deflections would have been needed to avert a 0.40-in. deficient deck thickness. On the other hand, if deck forming elevations were established to minimize the potential thermal differentials, only a 26 percent reduction in conventional plan deflections would be needed. Clearly, a routine reduction in conventional plan deflection values of at least 25 percent appears warranted where a sufficient lag between concrete placement and screeding cannot be ensured.

The results indicated that the bolted diaphragm connections act in a semirigid fashion on the 2 spans tested, but for most practical calculations they could be assumed to be rigid. Use of conventionally calculated dead-load deflections appears to be adequate as long as final longitudinal screeding follows concrete placement by approximately 3-bay lengths. This result, however, must be qualified to structures similar to those tested. Bridges on heavy skews, for example, would represent a different situation, and the use of conventional plan dead-load deflection values and longitudinal screeding could be quite risky.

CONCLUSIONS

The following conclusions are based on the results of the field and analytical study and pertain only to bridge decks constructed by use of the full-span longitudinal concrete placement and screeding technique.

1. Differential temperatures between the top and bottom flanges of steel girders can be quite high—due to solar radiation—when the deck forms are in place. The resulting effect is an upward deflection of the girders. If bridge deck forms are established to grades complying essentially with a thermally neutral condition on the girders, but the concrete deck is screeded to grade during differential thermal conditions, a deficient deck thickness could result. Upward midspan deflections on the order of 0.40 in. due to solar radiation were measured on a 96-ft 2-in. long steel girder span.
2. It is apparent that exact steel girder elevations cannot be established when any degree of solar radiation is present. On different days having similar weather, temperature, and solar conditions, however, the elevations of the girders will be almost identical at approximately the same time of day.
3. The heat of hydration of plastic concrete prior to initial set would have an insignificant effect on girder deflections for warm-weather deck placement conditions. The evidence suggests that solar radiation, changes in air temperature, and initial temperature of the plastic concrete influence girder temperatures more than does the heat of hydration. In this respect, it should be noted that differential girder temperatures could develop during cold-weather concrete placement as well as during warm-weather placement.
4. The average compression of the neoprene bearing pads (due to the dead load of the concrete deck) was on the order of 0.02 in., which does not warrant consideration in the calculation of dead-load deflections.
5. This study and others (2) indicate that there is a tendency for plan dead-load deflections to be in error on the high side. Thus, the deck forms would be set too high, and with full-span longitudinal screeding a deficiency in deck thickness would result. Plan deflection errors are believed to be due to the inclusion of the dead weights of all superstructure components in the calculations rather than the use of concrete deck weights only.
6. The field deflection measurements show that the structural steel framing of each of the 2 spans tested acted as a unit because of the diaphragm connections between the girders.
7. A comparison of the field deflection data with a theoretical analysis of deflections of semirigidly connected girders suggests that the bolted diaphragm connections on the 2 study spans act in a semirigid fashion. It was estimated from the comparison that the connections have an end fixity factor of approximately 0.20, which in effect is not greatly different from a rigid connection with an end fixity factor of 1.0.

8. For the 2 spans tested, the conventionally calculated dead-load deflection values were found to check very close to the actual field deflections when concrete placement was $2\frac{1}{2}$ to 3 bays beyond the girder in question. Thus, if the final screeding pass had lagged behind concrete placement by at least 3 bays, the conventionally calculated dead-load deflections would have been acceptable for both study spans. This result, however, must be qualified to structures similar to the 2 study spans. Bridges with high skew angles, for example, would likely present a different situation.

9. At a point where roughly three-quarters of the deck concrete had been placed, however, there was a tendency on both study spans for the final pass of the longitudinal screeding machine to follow too closely behind concrete placement. It is concluded that the plan dead-load deflection values, after being checked to ensure correctness, should be reduced by 25 percent to compensate for such occurrences on simple-span structures.

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