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HEAT-CURVED GIRDERS: DEFLECTIONS AND CAMBER LOSS DURING AND SUBSEQUENT TO BRIDGE CONSTRUCTION

by

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(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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SUMMARY

The present AASHTO bridge design specifications require that additional camber be built into steel girders that are to be heat curved. The additional camber is provided to allow for subsequent losses due to the dissipation of residual stresses imposed by the heat-curving process during fabrication of the girders. Many bridge engineers and steel fabricators, however, question whether the additional camber is necessary.

To check the camber loss in a heat-curved girder bridge, a 140-ft., simply supported span was instrumented during construction of the bridge. The span was composed of four steel plate girders having radii of curvature varying from 802.51 ft. on the inside to 834.51 ft. on the outside of the alignment curvature. Girder deflection and camber loss were measured prior to and subsequent to the construction of the bridge deck.

The AASHTO specifications for highway bridges indicate that losses of camber in heat-curved girders will occur both during construction of the bridge and subsequently under service loading. It is suggested that 50% of the camber loss will occur during construction and an additional 50% after the bridge has been subjected to several months of service loading.

For the bridge investigated, some camber loss due to construction loading occurred shortly after placement of the concrete deck. The amount of loss, however, was only one-fourth of that determined from the AASHTO equation. In addition, there were no significant camber losses due to service loading over 6½ months. The total loss under both construction and service loading was only 13% of that predicted by the AASHTO equation. Therefore, the results of the study suggest that the relationship given in the specifications for the calculation of the potential camber loss in a heat-curved girder is not applicable to girders having radii of curvature greater than 800 ft. The order of magnitude of the camber losses further suggests that the specifications may not be completely applicable to girders having radii of curvature less than 800 ft.

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INTRODUCTION

During the past decade, the use of horizontally curved girders for the construction of bridges on curved alignments has increased significantly. The increased use has been most notable in interchange areas where the geometric design often includes elevated curved alignments. In these situations, bridges employing curved girders are usually more compatible with the prevailing geometrics than those with straight girders and provide improved esthetics.

A 1973 survey by a subcommittee of the joint AASHTO-ASCE Committee on Flexural Members indicated that 507 curved girder bridges had been constructed in the United States at that time.(1) The majority of these structures, however, had been built in only a few states, with California and New York having constructed the highest number. The survey also revealed that most of the curved bridges were plate girder types of composite design. The majority were reported to have span lengths in the 50 to 150 ft. range and to be composed of either two or four girders per span. In addition, the majority of the curved plate girders that had been built prior to the survey had been fabricated by cutting to the required curvature rather than heat curving or cold forming. Type A36 steel was used on 63% of the bridges whereas A588 steel was used on less than 5%. The use of A588 steel has obviously increased in recent years.

Three general approaches have been used to design curved girder bridges. One is the approximate (U. S. Steel) method,(2) the second is the grid, and the third the space frame method. The survey found that 76% of the curved girder bridges built were designed by the approximate method and 22% by the grid procedure.

A number of curved girder bridges have been built in Virginia during the last few years and this activity has given rise to at least one persistent question relating to the fabrication of the steel girders. This involves the AASHTO bridge specification requirement that additional camber be built into steel girders that are to be heat curved. This additional camber is to be provided to allow for the subsequent losses due to the dissipation of residual stresses imposed by the heat-curving process during the fabrication of the girders. The AASHTO specifications suggest that approximately 50% of the camber loss relating to the heat-curving process will occur during the construction of the bridge, and an additional 50% will occur after a few months under service loading. Therefore, the increase in camber should be included in the bridge forming during construction; and after construction is complete, the bridge profile should be higher than the plan grade between the supports. If the additional camber is lost as the bridge specifications suggest, then the final profile should be attained after several months of traffic loading.

Many bridge engineers and steel fabricators, however, question whether the additional camber is necessary. The fabricators would prefer not being required to provide the additional camber in heat-curved girders since this would, in most cases, add to the time and expense of fabrication. To determine the nature of the deflections and camber loss in a heat-curved girder bridge, one was instrumented and measurements taken both during and subsequent to construction, and the results are reported here.

STRUCTURE STUDIED

A curved girder bridge consisting of three simply supported spans, two of them relatively short at 36 ft. and 18 ft., and the third 140 ft., was selected for study during its construction. All the measurements, however, were confined to the 140-ft. span. The structure tested, shown in Figure 1, is located on Route 726 and crosses Route 460 four miles east of Lynchburg, Virginia.

Each span of the bridge has four steel plate girders spaced at 10 ft.-8 in. on center. The girders of the 140-ft. span are connected by truss type diaphragms, and lateral cross bracing is used on the exterior bays as illustrated in Figure 2. The four steel girders are curved on radii varying from 802.51 ft. on the inside to 834.51 ft. on the outside of the alignment curvature. On the centerline of the bridge the alignment is equal to a 7° highway curve.

The steel girders were fabricated from A588 steel and were heat treated to obtain the required degree of curvature. Compared to the 1973 survey cited earlier, the study structure differs from the majority that had been built prior to then in that few were heat-curved and few utilized A588 high strength steel.

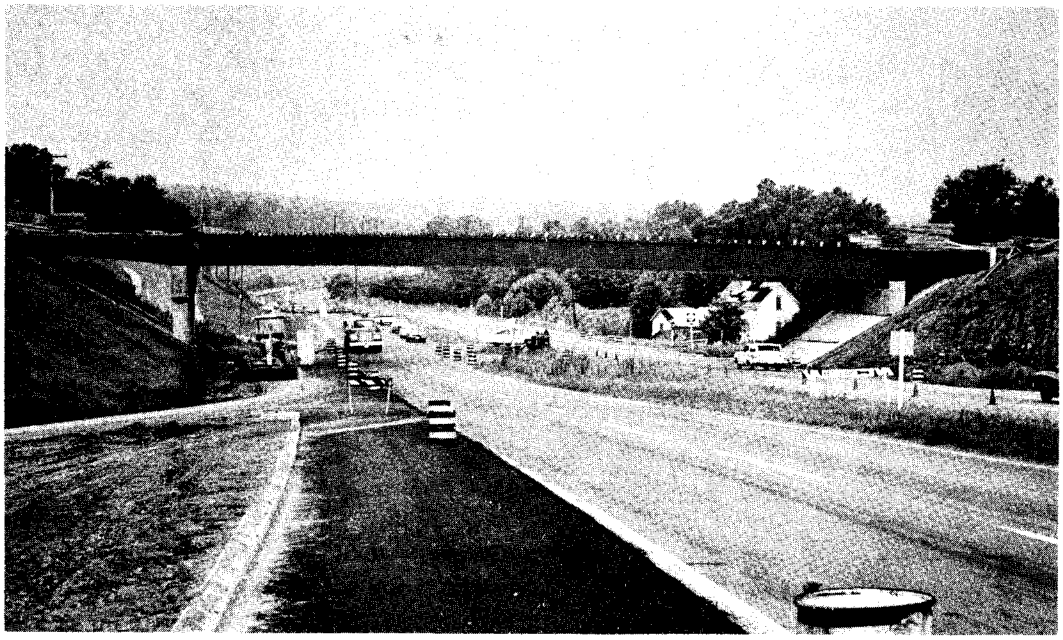

Figure 1. Profile view of the curved girder bridge showing the structural steel in place prior to construction of the deck.

Figure 2. View of the structural steel showing curvature in the girders and the design of the diaphragms and lateral cross bracing.

PURPOSE AND SCOPE

The purposes of the study were as follows:

1. To measure the dead load deflection of the curved steel plate girders during the placement of the concrete deck and parapet walls.

2. To measure the effects of thermal differentials on the steel girders due to solar radiation and to concrete temperature effects such as heat of hydration and shielding of the upper flanges of the girders during placement.

3. To determine the order of magnitude of the camber loss in the heat-curved girders both during construction and subsequent to the bridge being placed under service loadings.

4. To compare the results obtained from the field measurements with those calculated from the AASHTO formula for camber loss in heat-curved girders.

The scope of the study was limited to the testing of one span of the bridge described earlier. Primary measurements were taken only of those variables which, directly or indirectly, tend to influence the deflections and camber loss of the curved steel girders. Although measurements of strains in some selected diaphragm members were taken, the results are not germane to the main objective of the study and are not included in this report.

Deflection and thermal measurements were taken during all stages of construction, 1 month after the bridge deck was completed, and after it had been under traffic loading for approximately 6½ months.

INSTRUMENTATION, TESTS, AND PROCEDURES

Since measurements were made on the structure during the construction of the bridge deck, the work was subjected to several constraints. First, it was necessary that the deck forming be in place before most of the instrumentation could be installed on the span to be tested. Consequently, very little time was available to accomplish this task without causing excessive delay to the contractor. Secondly, the data collection techniques and measurement devices had to be designed for minimum obstruction and delay during the general construction of the bridge or the maintenance of traffic beneath the bridge. Thirdly, concrete placement operations could not be delayed for long periods of time to permit data collection. Thus, the number of measurements taken during each delay in construction operations was limited to that which could be handled in approximately 10 minutes. In addition, the weather and other uncontrollable construction factors excluded the use of some types of instrumentation that could not be depended upon to function properly under adverse conditions. All the aforementioned constraints were considered in selecting the methods and procedures of data collection.

The locations of the instrumentation installed on the structural steel of the 140-ft. span are shown in Figure 3. Additional strain gage instrumentation that was installed on some diaphragm members is not shown since data they provided are not relevant to the subject of this report.

Girder Deflection Instrumentation

Since some of the deflection increments to be measured were expected to be on the order of hundredths of an inch, a high precision, modified Wild "N-III" level was selected for use. The modified "N-III" level is capable of direct readings to 0.001 of an inch by nature of a plane-parallel glass plate mounted in front of the objective lense. When tilted, the glass plate displaces the line of sight, which serves as an optical micrometer that can be used to measure fractions of an observed rod graduation.

The high precision level was mounted on a trivet that in turn was set in stationary bronze lugs on the top of the pier cap at the north end of the 140 ft. test span. The line of sight of the level was thus slightly below the bottom flanges of the steel girders. Figure 4 shows the instrument mounted on top of the pier, which was designed with a solid concrete stem. Because of the location of the pier one could stand on the north fill slope and sight southward through the instrument when collecting deflection data.

Special design rod and scale units were installed at the mid-span points of each girder on the span tested. As illustrated in Figure 5, the rod and scale unit was mounted in an adjustable bracket that in turn was attached to a large C clamp. The C clamps, which were fabricated for use in this study, were attached to the girder flanges as close to the web as possible. By use of a hand level, the rod on each unit was set plumb. Flat, 1-foot long, engineer's scales with 1/2 in. major graduations were mounted to the rods and adjusted vertically so that all scales would intersect the line of sight of the level.

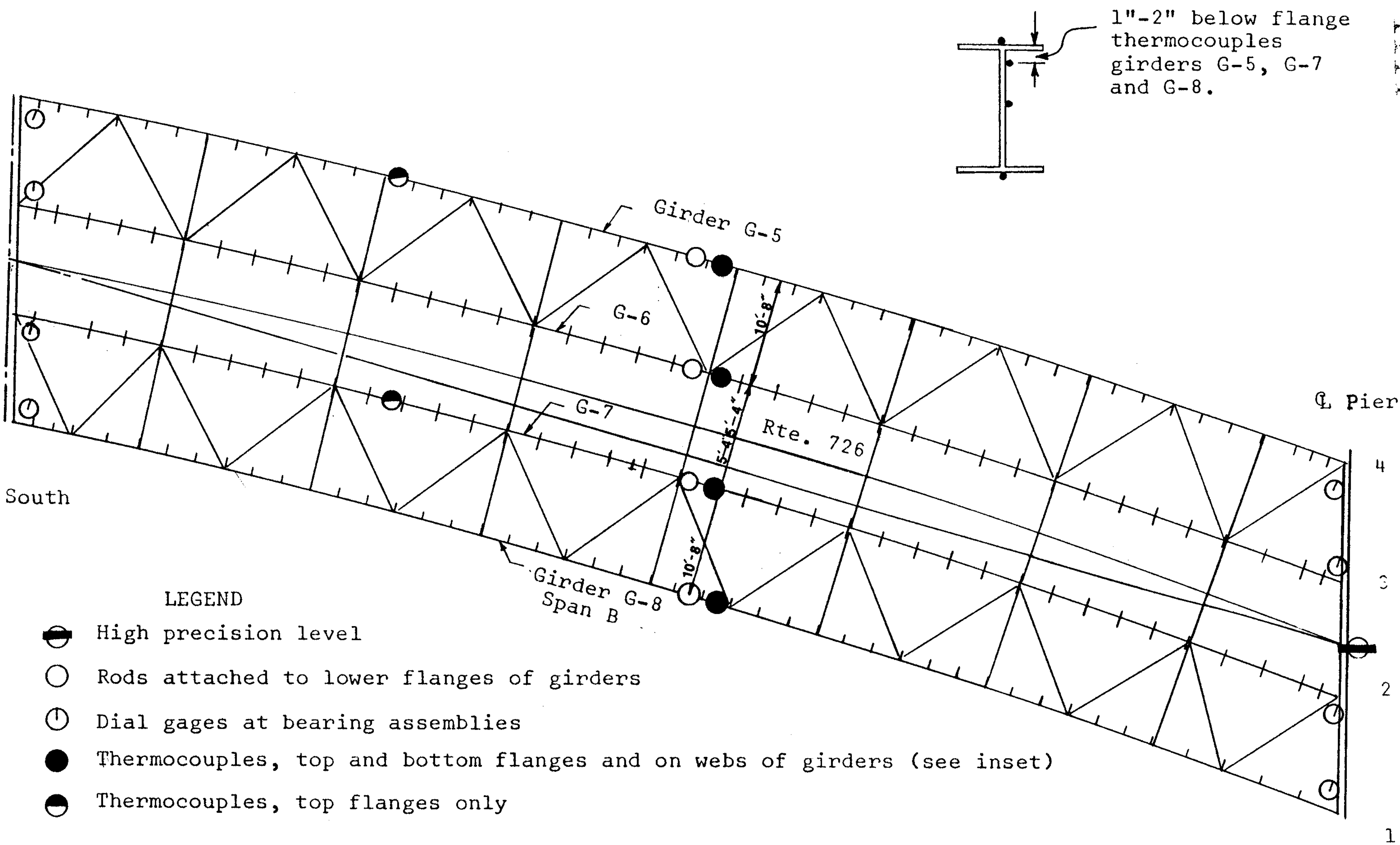

Figure 3. Framing diagram of curved girder span B of Rte. 726 over Rte. 460 showing locations of instrumentation.

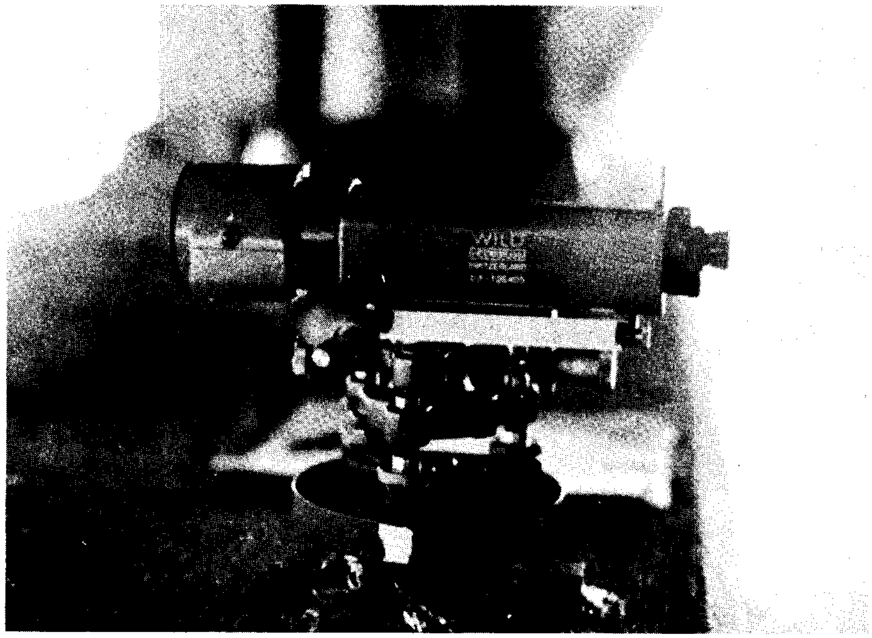

Figure 4. High precision level and trivet positioned on top of the bridge pier cap.

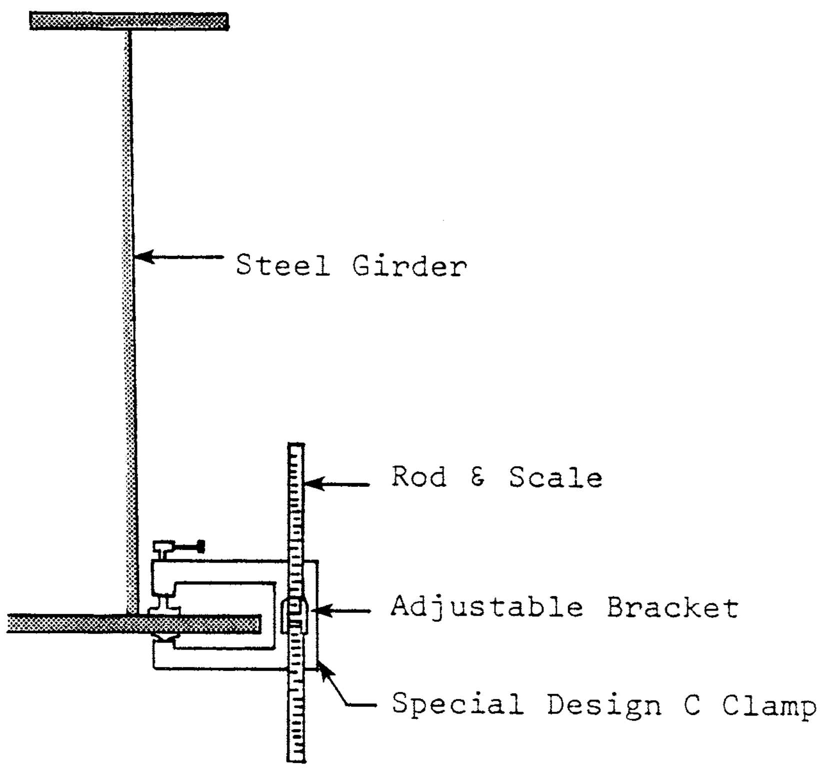

Figure 5. Typical rod and scale unit attached to the lower flange of the curved girders.

Thermal Instrumentation

A 24-channel Honeywell thermocouple recorder powered by a portable generator was used to collect temperature data on the steel girders. The recorder is shown in Figure 6 mounted in a heavy steel cabinet for protection during field use. Thermocouples using a type J iron-constantan wire were placed on the top and bottom flanges of the girders at the mid-span length points. Some additional thermocouples were placed on the girders at the quarter-span length points and at some selected positions on the web of the girders as shown in Figure 3.

During placement of the concrete deck and parapet walls, the temperature recorder was in continuous operation. A complete cycle of 24 thermocouples was made every 12 minutes; i.e., a temperature measurement was taken automatically at each location every 12 minutes. Other temperature measurements were taken prior to concrete placement and after each phase of the construction process was completed to determine the independent effect of solar radiation on the deflection of the curved girders.

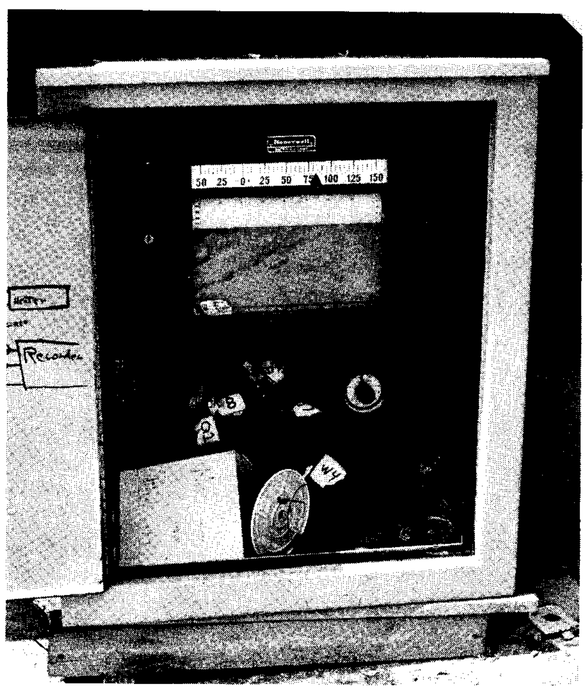

Figure 6. Honeywell recorder mounted in a steel cabinet for field use.

Bearing Deflection Instrumentation

To measure and account for possible dead load deflections of the bridge bearings, dial gages were set as close to the centerline of bearing of each girder as possible. The dial gages were mounted to a heavy steel stand, which in turn was secured to the top of the pier cap with an epoxy resin. The tops of the bottom flange of the steel girders were cleaned and all loose paint was scraped off at the contact point between the steel and the gage. A typical installation of a dial gage at a girder bearing is shown in Figure 7.

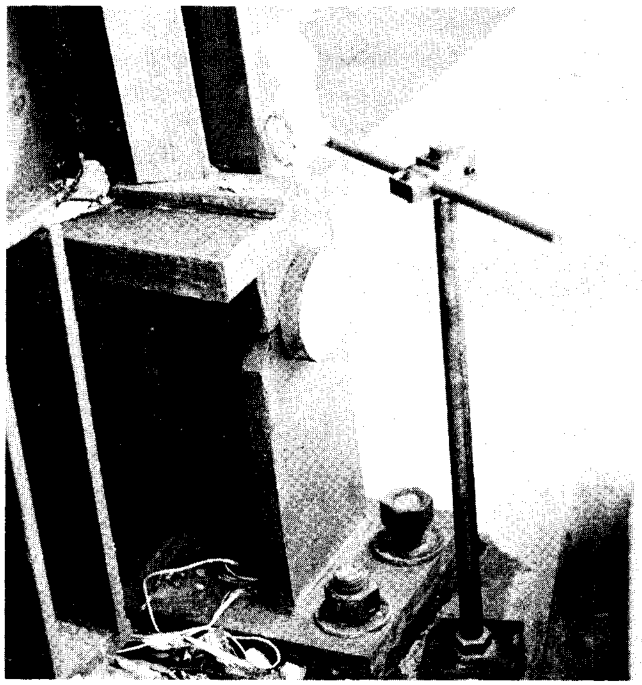

Figure 7. Dial gage installation used to measure bearing deflections during placement of the concrete bridge deck. Plastic covering was used to protect gage from moisture.

Tests on the Plastic Concrete

Tests made on the plastic concrete were restricted to the measurement of those properties which would have the most direct influence on the bridge girder deflections during deck placement.

The following tests and measurements were made during placement of the deck concrete.

1. The times of initial set and final set (ASTM C403-68) were run on three batches of the concrete. Samples were selected near the beginning and the conclusion of the deck placement operation.

2. Unit weight determinations (ASTM C-138-63) were made on three samples selected at intervals to be generally representative of the concrete placed in each area of the deck. Air content and slump measurements were also taken on the three samples.

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 operation.

Study Procedures

All of the instrumentation was installed on the test span while construction was in progress, and initial readings were taken on all systems as soon as the installation was complete. Subsequent measurements were taken during a full day after each major stage of construction to determine the independent effects of differential thermal conditions.

During placement of the deck concrete, construction operations were delayed to allow for measurements of deflections when the loading was approximately one-fourth, one-half, and three-quarters complete. Measurements were taken when all the concrete was in place and again several hours after completion of the deck finishing operation.

With the exception of the placement delays for measurements, the contractor's normal procedures were used during construction. All elevations and grades used to establish the position of the deck forming were set by the contractor's personnel and checked by the bridge construction inspectors.

After all construction was completed, the positions of all the deflection rods and scales were marked on the girders to establish their horizontal and vertical position. Initial readings were then taken on the vertical position of all girders and all thermocouples were scanned to establish the differential temperature conditions at that time. These data, then, were used as the basis for measurements of the long-term loss in camber to be taken after the bridge had been put into service. All gages were then dismantled to allow for removal of the false work and painting of the structural steel. The thermocouple wires were cut off in such a manner that they could be reconnected. Subsequently, all gages were reconnected or repositioned on the span to facilitate the measurement of any long-term losses in camber under service loadings.

RESULTS

Thermally-Induced Deflections in Steel Section

With the forming for the deck in place, the lower portion of the steel girders are shielded from the sun. Consequently, the top flanges of the girders are exposed to solar radiation whereas the lower portion of the girders are exposed only to the ambient air temperature. Since the alignment of the study bridge is generally in a southerly to northerly direction, the early morning sun strikes the web and lower flange of the eastern girder; the late afternoon sun strikes the web and lower flange of the western girder. The effect is a net thermal differential between the upper and lower flanges that develops an internal moment over the cross section of each girder. The internal moment causes the girder to deflect upward by an amount relating to the intensity of the solar radiation, time of day, etc. For a typical sunny day in early August the differential temperatures shown in Figure 8 were recorded at eight times from 7:35 a.m. to 3:20 p.m. At 7:35 a.m. the lower flanges were warmer than the upper flanges for all the girders. This was probably due to the lower flanges being somewhat protected from the elements during the night. However, with time the upper flanges heated up. By 3:00 p.m. a maximum temperature differential of 36° was recorded on each of the two center girders. The effect of the morning sunlight on the east girder (G-8) can be noted from these data.

The deflections corresponding to each of the reported differential temperatures are shown in Figure 9. The initial reference elevations of the girders were recorded at 7:30 a.m. As can be noted from these data, the upward mid-span deflections increased as the differential temperatures between the top and bottom flanges increased. Furthermore, since the morning sun warmed the lower portion of girder G-8, an uneven transverse deflection pattern developed. In the afternoon, when the solar radiation was more evenly distributed across the top of the span, the transverse deflection pattern was more evenly distributed. By 3:00 p.m. the deflections were, for all practical purposes, the same across the width of the span. Although data were not recorded beyond 3:20 p.m. (when the temperature differentials began to decline), it is apparent that the solar effects on the west girder (G-5) would reverse the transverse deflection pattern during the evening hours.

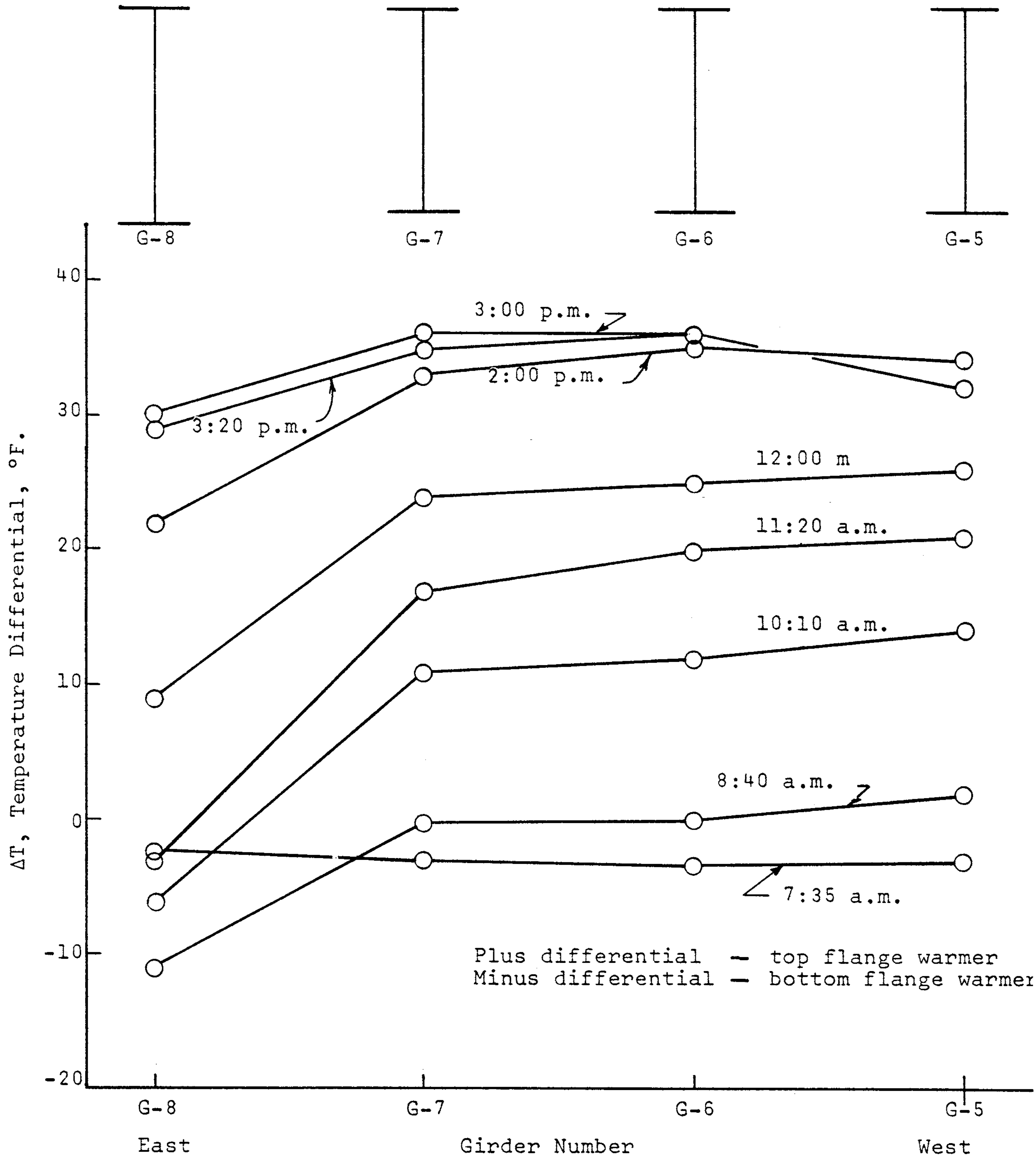

Figure 8. Temperature differential between top and bottom flanges of curved-girder span with deck forming in place: prior to deck placement.

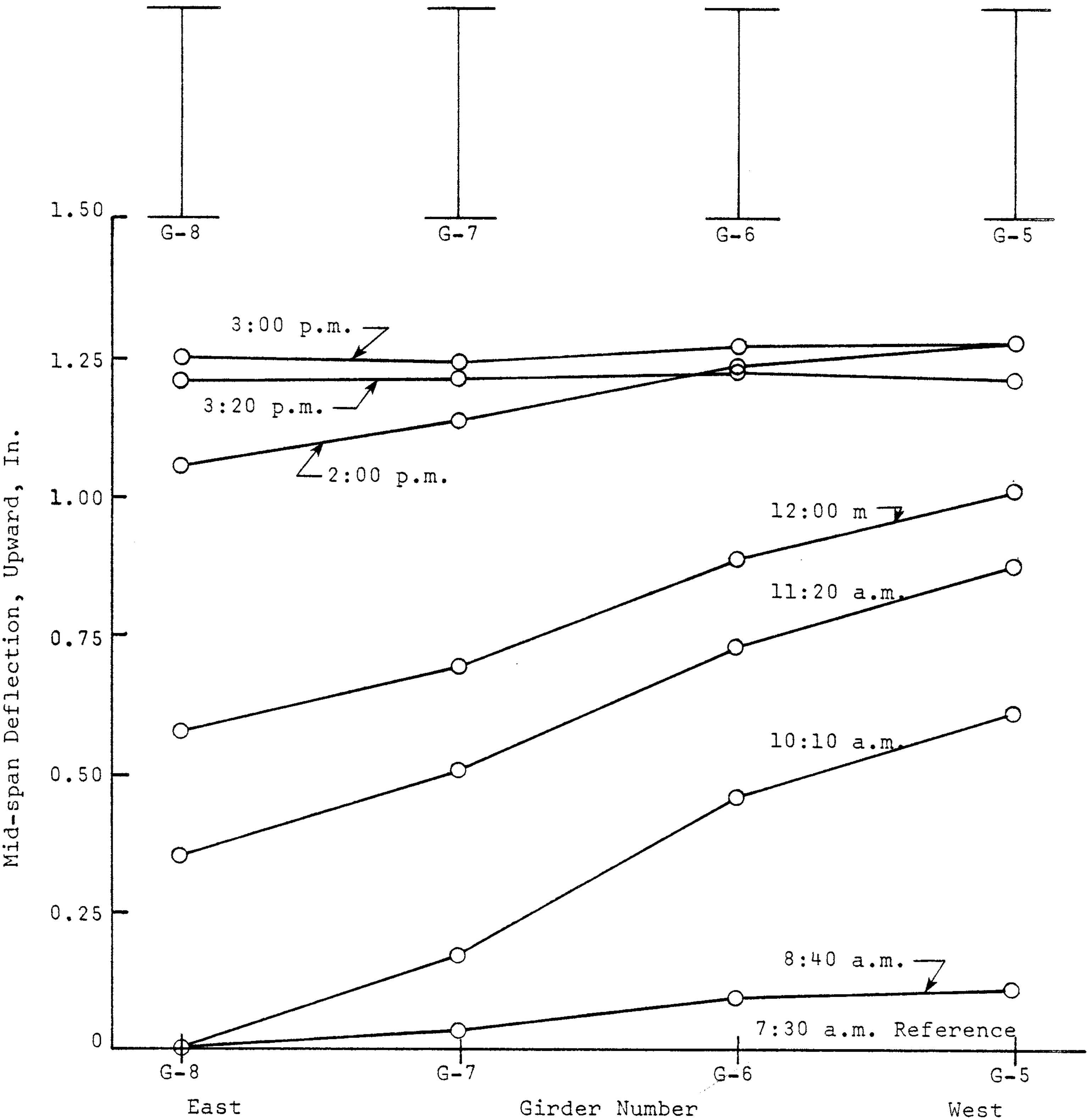

Figure 9. Upward deflections at mid-span of curved girder span resulting from solar radiation. Deck forming in place shields lower portion of girders from the sun.

As can be noted from Figure 9, the maximum mid-span deflections of the girders were on the order of 1.25 in. when data were recorded at 3:00 p.m. These data show that the thermal effects on girder deflections must be taken into account if one is to attempt to measure deflections due to loading and to sustained losses of camber due to deadweight or to service loads. Therefore, an important aspect of this study was to determine the nature of the differential thermal conditions and their effects on the deflections of the curved steel plate girders.

To determine the thermal gradients through the depth of the girders, thermocouples were placed on the webs of girders 5, 7, and 8. One was located at mid-depth of the web and another approximately 2 in. below the lower side of the top flange. The thermal gradients for the three girders are shown in Figure 10. These data show that the temperature increase due to solar radiation on the top flange of the girders was transmitted downward through the web. As the top flange temperature increased in the afternoon, the temperature of the web at mid-depth of the girder was greater than that of the lower flange. This was particularly true on girder G-5 on the west side of the bridge. It is, therefore, obvious that any calculations performed to determine thermal deflections must consider the upper portion of the web above the neutral axis to be participating in the development of the forces and moments. In some instances, as the data of Figure 10 suggest, a portion of the web below the neutral axis was at a higher temperature than the lower flanges of the girders.

For the bridge tested in this study the plate girder design incorporates changes in the moment of inertia at points of increasing flange plate thickness. This, in addition to the action of the rigid diaphragm connections between the girders, creates a complex system. The rigid diaphragm connections cause the thermally-related deflections to be distributed across the width of the span as indicated by the data in Figure 9. Since the 3:00 p.m. thermal deflection data were more uniformly distributed across the span width, theoretical calculations were made for several different assumptions to determine which most nearly agreed with the field results. For the first assumption an effective flange area and moment of inertia were determined by proportioning based on the length of the girder applicable to each section. The second calculation assumed that the moment of inertia and flange area of the center portion of the girder could be used with sufficient accuracy to predict the thermal deflections. The calculated deflections were based on the relationship

$$F = AE\,\alpha\,\Delta T, \qquad (1)$$

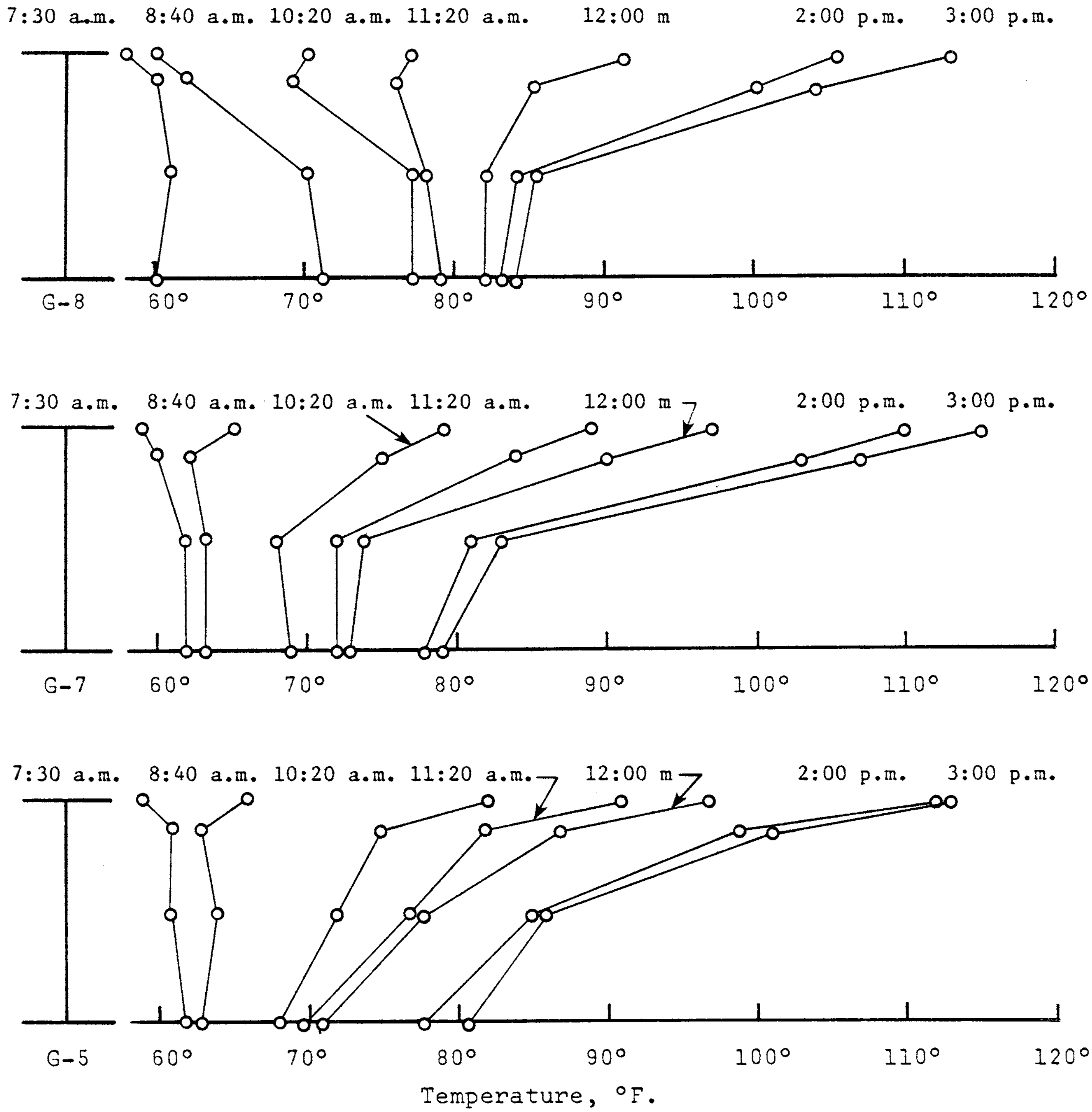

Figure 10. Thermal gradients through the depth of girders 5, 7 and 8 caused by solar radiation: deck forming in place.

where

F = force developed by expansion of the heated steel,

A = area of the section above the neutral axis,

E = modulus of elasticity of steel,

α = thermal coefficient of expansion for steel, and

ΔT = the difference in temperature between the upper and lower flanges.

Since the internal moment in the girder is developed by the product of the force and the distance to the neutral axis, the deflection, Δ, is

$$\Delta = \frac{A\ \alpha\ \Delta T\ d\ L^2}{8I}, \tag{2}$$

where

d = distance from the force center to the neutral axis,

L = length of span, and

I = moment of inertia.

The thermal deflections calculated from the latter relationship resulted in the values shown in Table 1. Δp, Δm, and Δa represent, respectively, the thermal deflections resulting from proportioning the properties of the girder, by using the mid-span properties and, lastly, the thermal deflections measured in the field at 3:00 p.m. (DST) in early August are given. The two calculated deflections are nearly the same, with Δm being slightly closer than Δp to the actual deflection. The calculated deflections are at most only 1/32 in. greater than the measured deflections. The average Δp and Δm for all four girders of the span are, respectively, 2.2% and 0.8% higher than the average Δa. This small difference is probably due in part to the fact that the temperature at the neutral axis is often higher than that of the lower flange. Only the portion of the web above the neutral axis was assumed to be participating in developing the thermally-induced moments, but the temperatures were assumed to be uniform. Based on these results, however, it can be concluded that the use of equation (2) gives a reasonably good estimate of the deflections caused by differential temperatures that are relatively uniform across the width of the span.

Table 1

Girder Deflections Caused by Differential Temperatures Between Top and Bottom Flanges with Deck Forming in Place, in.

Girder No.	Δp^a	Δm^b	Δa^c
G-5	1.26	1.24	1.27
G-6	1.27	1.25	1.27
G-7	1.30	1.28	1.24
G-8	1.30	1.29	1.24
Average	1.283	1.265	1.255

[a]Calculated using proportioned girder properties.

[b]Calculated using mid-span girder properties.

[c]Measured.

Dead Load Deflections in Steel Section

Reinforcing Steel Placement

The first deadweight loading to be placed on the span (excluding the deck forming) was the reinforcing steel. Temperature measurements and deflection rod readings taken on the girders prior to the placement of the deck concrete were compared to the initial readings taken approximately 12 days earlier. The thermal differentials on the girder were virtually the same for each of these two sets of data as shown in Table 2. There was only a half degree Fahrenheit difference on girders G-5 and G-6 between these two time periods. By using equation (2) and applying a correction to the measured deflections for girders G-5 and G-6, the thermal differentials would be the same for the two times at which measurements were taken. The resulting deflections are then thermally neutral and thus reflect only the downward movement caused by the deadweight of the reinforcing steel. The mid-span dead load deflections resulting from the weight of the reinforcing steel were on the order of 5/32 in. and are reported later in Table 4.

Table 2

Temperature Differentials Between Lower and Upper Flanges of Girders, Deg. F.

Time	Date	Girder No. G-8	G-7	G-6	G-5
7:30 a.m.	8-8	-2.5^{o}[a]	-3.0^{o}	-3.5^{o}	-3.0^{o}
6:30 a.m.	8-21	-2.5^{o}	-3.0^{o}	-3.0^{o}	-2.5^{o}

[a](-) indicates lower flange warmest.

Concrete Deck Placement

Concrete was placed on the 140-ft. span beginning at 6:30 a.m. on August 21. Placement began at the north end of the span and proceeded southward. At the beginning of the deck placement, the temperature conditions on the steel girders indicated that the girders were not in a thermally neutral position. Since the lower flanges of the girders were warmer than the top flanges, the girders were initially deflected downward. As the day went by, the temperature differential between the top and bottom flanges changed. By 10:00 a.m. the upper flanges were the warmer, but by 11:25 a.m. all the concrete had been placed on the span and the temperature on the top flanges began to drop relative to that on the lower flanges. By 2:45 p.m. the temperature differential between the upper and lower flanges of the girders was very slight and the span was, for all practical purposes, in a thermally neutral state. The differential temperature data recorded during the various stages of concrete placement are shown in Figure 11.

Initial readings on the deflection rods were taken at 6:30 a.m. prior to beginning the placement of the deck concrete. These readings were used as a reference to determine the dead load deflections of the girders due to the weight of the concrete. However, it should be noted that a correction should be made to obtain the correct deflection at a given time, since the girders were not in a thermally neutral position at 6:30 a.m.

Figure 12 shows the deflections of the girders at the various stages of concrete placement. The deflections shown are those that existed at the stage of concrete placement indicated on the graph. These data also correspond to the thermal differentials shown in Figure 11. At 11:25 a.m., approximately 5 hours after the beginning of placement, all of the concrete was in the forms. At that time, however, the top flanges were warmer than the lower, so it could be expected that a counter deflection upward existed. Therefore, additional deflection measurements were taken at 2:45 p.m. and 3:20 p.m., when the span was very close to a thermally neutral position. As would be expected, the downward deflection was greater these times, although the dead load on the girders remained unchanged from that which had existed at 11:25 a.m. Applying the corrections discussed above to account for the initial thermal differentials yielded the final thermally neutral dead load deflections resulting from the weight of the concrete deck. These final values are shown by the lower curve in Figure 12 and are reported later in Table 4.

Before calculating the thermal corrections for the deflection data, it was necessary to consider the setting time of the newly placed concrete. This is an important consideration because once the concrete begins to set, some degree of composite action between the concrete and steel begins to develop. To determine the times of initial set and final set, two samples — one at the beginning and one midway through the placement operation — were tested by the ASTM C403-68 test procedures. The results are shown in Figure A-1 of the Appendix. For the first sample, the initial and final sets occurred, respectively, at 10.1 and 11.3 hours from the time of addition of water to the concrete. Therefore, the time of initial set was approximately 4:30 p.m. and that of final set approximately 5:45 p.m. for the first concrete placed on the span. For the concrete located in the mid-span area the initial and final sets occurred, respectively, at 8:30 p.m. and 10:30 p.m. Based on these data, no composite action between the steel girders and concrete could be expected at 2:45 p.m. and 3:20 p.m., when the last deflection measurements were recorded. Accordingly, all calculations to determine corrections to the girder deflections to account for thermal differentials during deck placement are based on the section modulus of the steel section only. Corrections to the deflections measured subsequent to the time of final set of the concrete, as discussed later, must be calculated based on some degree of composite action between the girders and the concrete deck.

Some slight movements of the bearing assemblies did occur during the placement of the concrete deck. The final deflections, or settlements, in the bearings at the north and south ends of the span are given in Table 3. As would be expected, the placement of the concrete caused some downward settlement of the bearings. The settlement, however, was very slight, as the average values of Table 3 indicate. Girders 5 and 7, for example, experienced settlements of less than a hundredth of an inch. The maximum average settlement occurred at girder 8, which experienced a downward movement of 0.018 in. These average values are thus deducted from the girder deflections measured upon completion of the placement of concrete for the bridge deck.

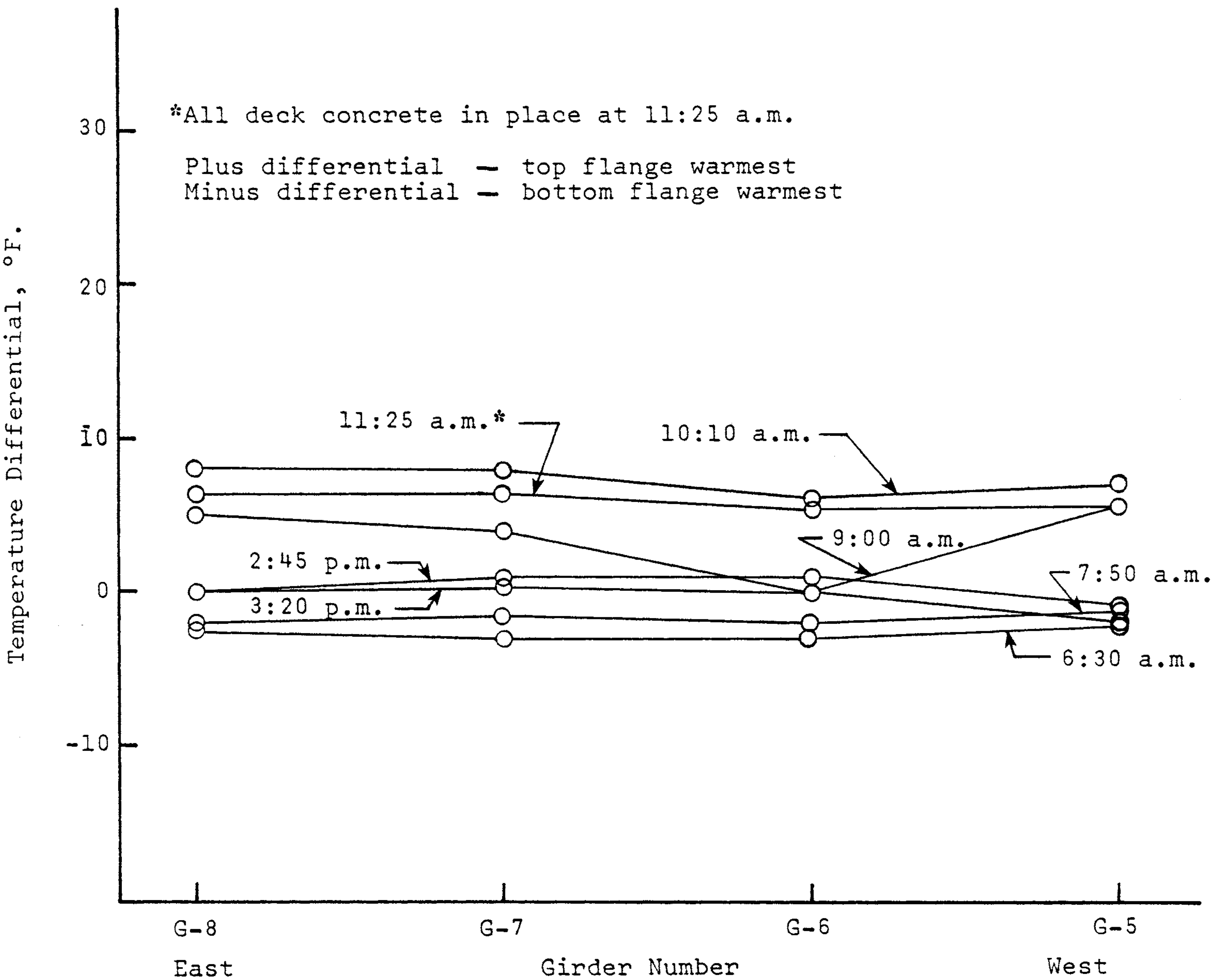

Figure 11. Temperature differential between top and bottom flanges during deck placement.

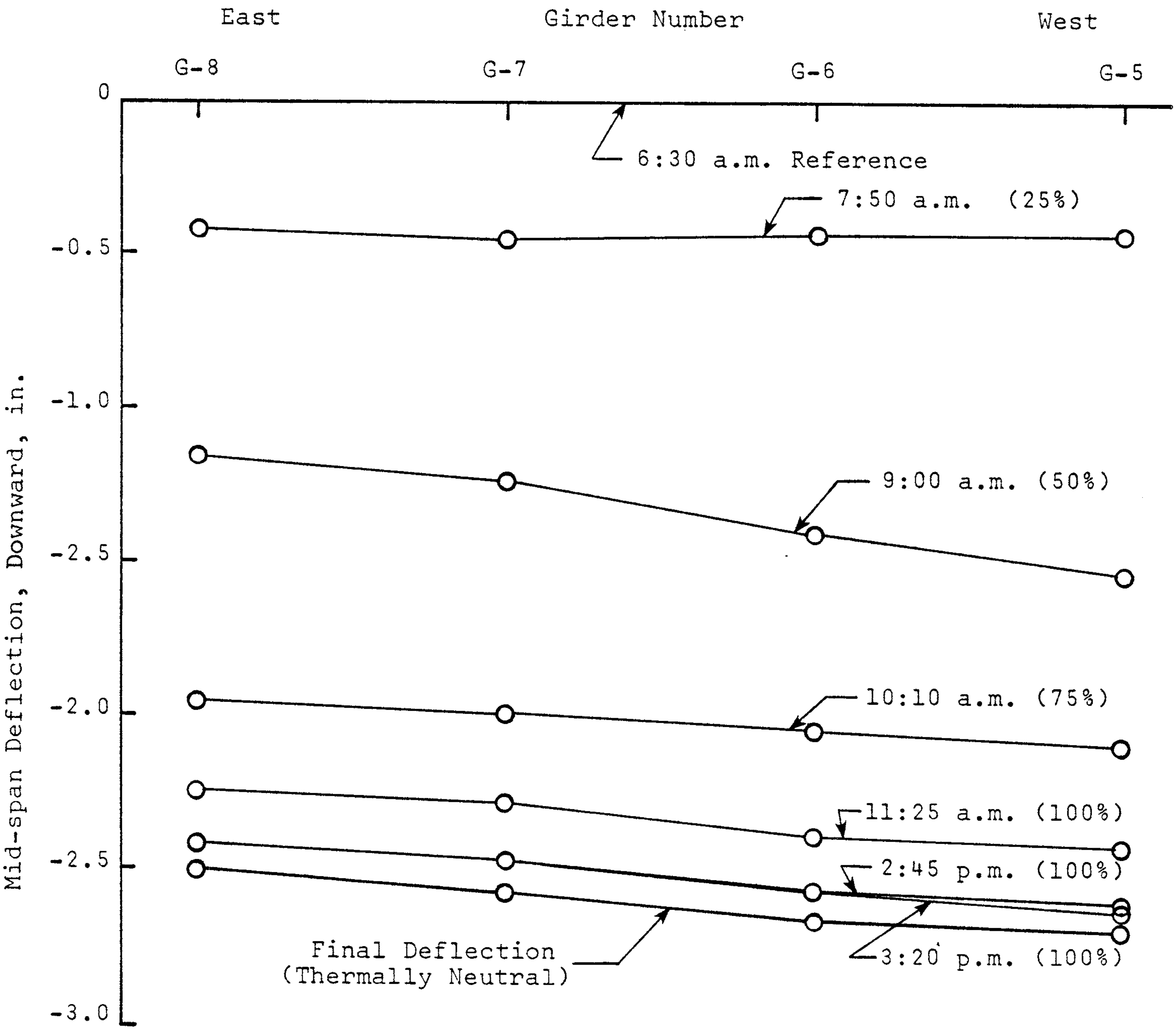

Figure 12. Downward deflections at mid-span due to placement of concrete deck.

Table 3

Bearing Settlements, in.

Bearing	Girder Number G-8	G-7	G-6	G-5
North Pier	0.013	0.001	0.012	0.001
South Pier	0.022	0.012	*	0.002
Average	0.018	0.007	0.012	0.002

*Dial gage was struck by debris during construction.

The total measured deflections of the curved girders due to the weight of the reinforcing steel and concrete are given in Table 4 and are lower than those given on the bridge plans. In addition, the measured deflections were progressively larger from girder G-8, the inside girder, to G-5, the outside girder, whereas the plan deflections alternate from lower to higher values between girders. This suggests that the diaphragm action between the girders tends to even out the actual deflection patterns. It is interesting to note that the average of the measured deflections is 2.766 in. Theoretical calculations that include deflections due to shear forces at the diaphragms yield an average deflection of 2.77 in. Thus, based on the average deflection, which tends to allow for diaphragm action between girders, there was excellent agreement between the measured and theoretical dead load deflections. When curved girders are interconnected by rigid diaphragms it would appear to be more accurate to calculate an average dead load deflection for the girders than to use the independently calculated deflection for each girder.

Two additional deflection measurements were recorded the day after the deck was placed. By this time the heat of hydration of the concrete was causing the top flanges of the girders to be warmer than the lower flanges. Temperature differentials on the order of 17° to 24°F. existed at 10:00 a.m. on August 22, and one would expect the girders to be at a higher elevation at mid-span than they had been at 2:45 p.m. the day before. The 10:00 a.m. data shown in Figure 13 show this to be the case, since the deflections were at that time less than the thermally neutral final deflections for the previous day. The thermal deflections were calculated by using equation (2) and are shown in the upper portion of Figure 13. By applying these as corrections to the measured data of August 22, the thermally neutral deflections shown by the lower curve in Figure 13 were within 0.01 in. or less of those measured on the previous day. In calculating the thermally-induced upward deflections, partial composite action between the concrete and

Table 4

Mid-Span Dead Load Deflections of Steel Girders Due to Placement of the Deck Reinforcing Steel and Concrete, in.

Loading	Girder Number G-8	G-7	G-6	G-5
Reinforcing Steel	0.144	0.165	0.171	0.209
Concrete	2.494	2.568	2.662	2.690
Steel Plus Concrete	2.638	2.733	2.833	2.899
Bearing Settlement	-0.018	-0.007	-0.012	-0.002
Total	2.620	2.726	2.821	2.897
Plan Values	3-5/8	3-3/4	3-1/4	3-3/8
Difference	+1	+1	+7/16	+15/32

NOTE: Thermally neutral deflections.

steel was assumed. Rather than using the normal design value for the ratio of the modulus of elasticity of steel to that of the concrete, E_s/E_c, it was assumed that the modulus of the 1-day old concrete would be on the order of 1,000,000 psi. Accordingly, a value of E_s/E_c of 30 was used in lieu of the normal design value of 10. The results shown in Figure 13 suggest this to be a reasonably valid assumption.

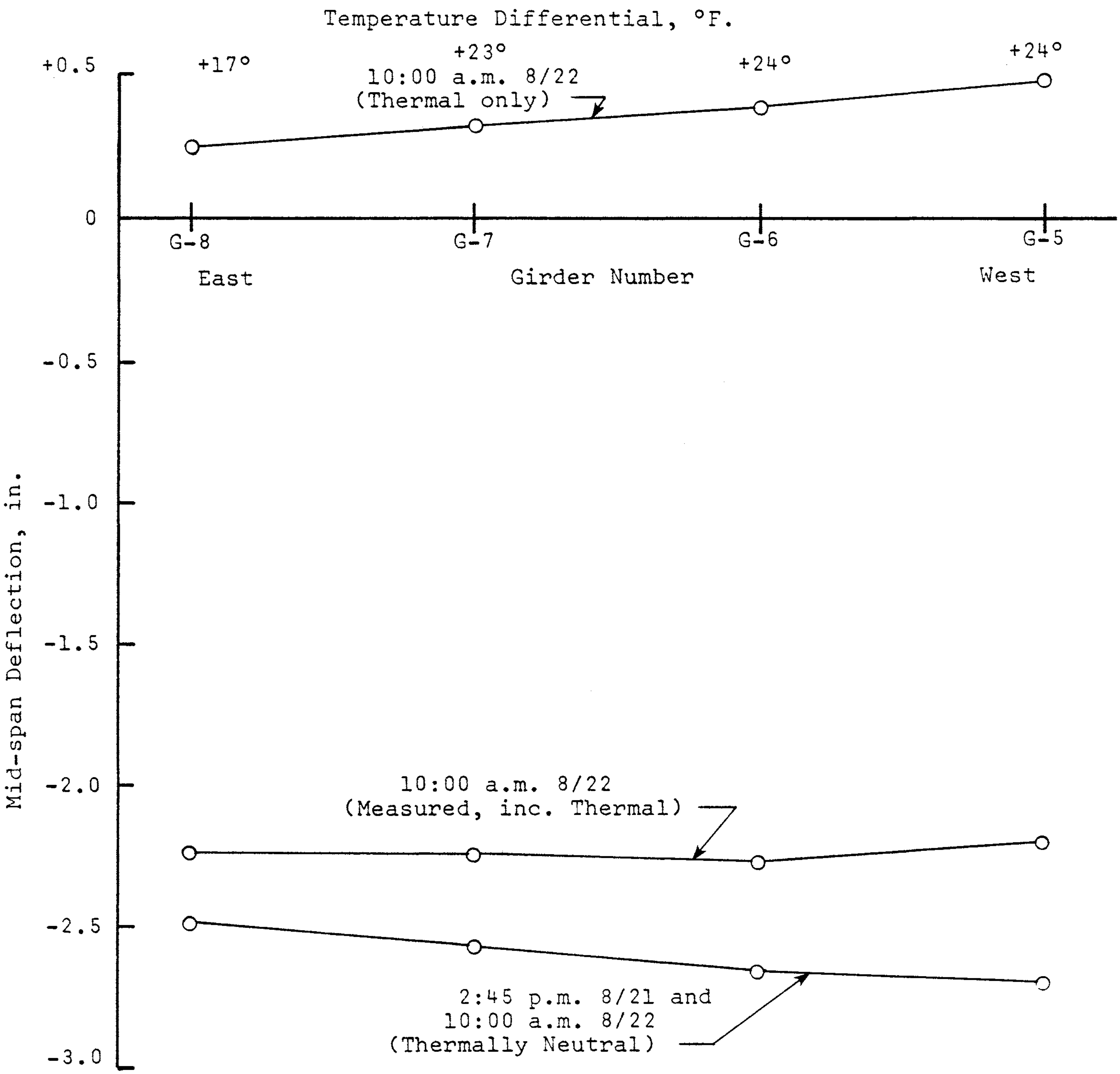

Figure 13. The deflection of the girders 1 day after deck placement showing the effect of differential temperatures resulting from the heat of hydration of the concrete.

Thermally-Induced Deflections and Short-Term Camber Loss in Composite Section

As discussed earlier, the 2:45 p.m. data on the day the concrete deck was completed best represent a thermally neutral condition of the span under study. Therefore, these deflections and thermal data were used as a new base reference for the comparison of the deflections resulting from subsequent thermal, dead-loading, or other conditions. Nineteen days after completion of the deck, additional temperature — deflection data were recorded to determine their order of magnitude under the new condition of the concrete deck and steel girders acting as a composite section. Unlike the thermal effects discussed earlier for the steel section only, the top flanges of the girders were then protected from the sun whereas the remaining portion of the girders were exposed to ambient conditions as well as to direct solar radiation on the east side of the bridge in the morning and on the west side in the evening. In addition, due to the composite action of the deck and girders, the moment of inertia and location of the neutral axis differ from those of the steel section only. Consequently, thermal deflection data were collected during a day in early September for two purposes: (1) to obtain data that could be used for making thermal corrections to all subsequent deflection measurements that would be recorded, and (2) to determine if any loss of camber in the heat-curved girders had occurred in the 19-day period since the application of the sustained dead loading of the concrete deck and reinforcing steel.

The results of these measurements, given in Figures 14 and 15, show, respectively, the net thermal differentials between the top and bottom flanges of the steel and the deflections resulting from the thermal loads. These data were recorded at seven times during the day, with the first measurement being used as a reference. Therefore, for the thermal data shown in Figure 14, the net temperature differential between the top and bottom flanges of each girder is the algebraic difference between the differential at 8:30 a.m. and that at the time of the subsequent measurement. In all cases, measurements taken subsequent to the 8:30 a.m. reference showed that the exposed lower portion of the steel girders were warmer than the top flanges within the concrete deck. Net temperature differentials on the order of 11° to 14°F. developed downward deflections of the composite girders on the order of 1/4 in. or more, as shown in Figure 15.

Because of the uneven distribution of the temperatures measured on the lower flange and web of each girder, it was difficult to utilize equation (2) to calculate the thermal deflections of the composite section. While the calculated thermal deflections were reasonably close to those measured, the temperature variation within the web of each girder made it virtually impossible to assume a reasonable degree of confidence that the true effects were being reflected in the calculations. Therefore, the experimental data were used for making thermal corrections to the composite section deflections.

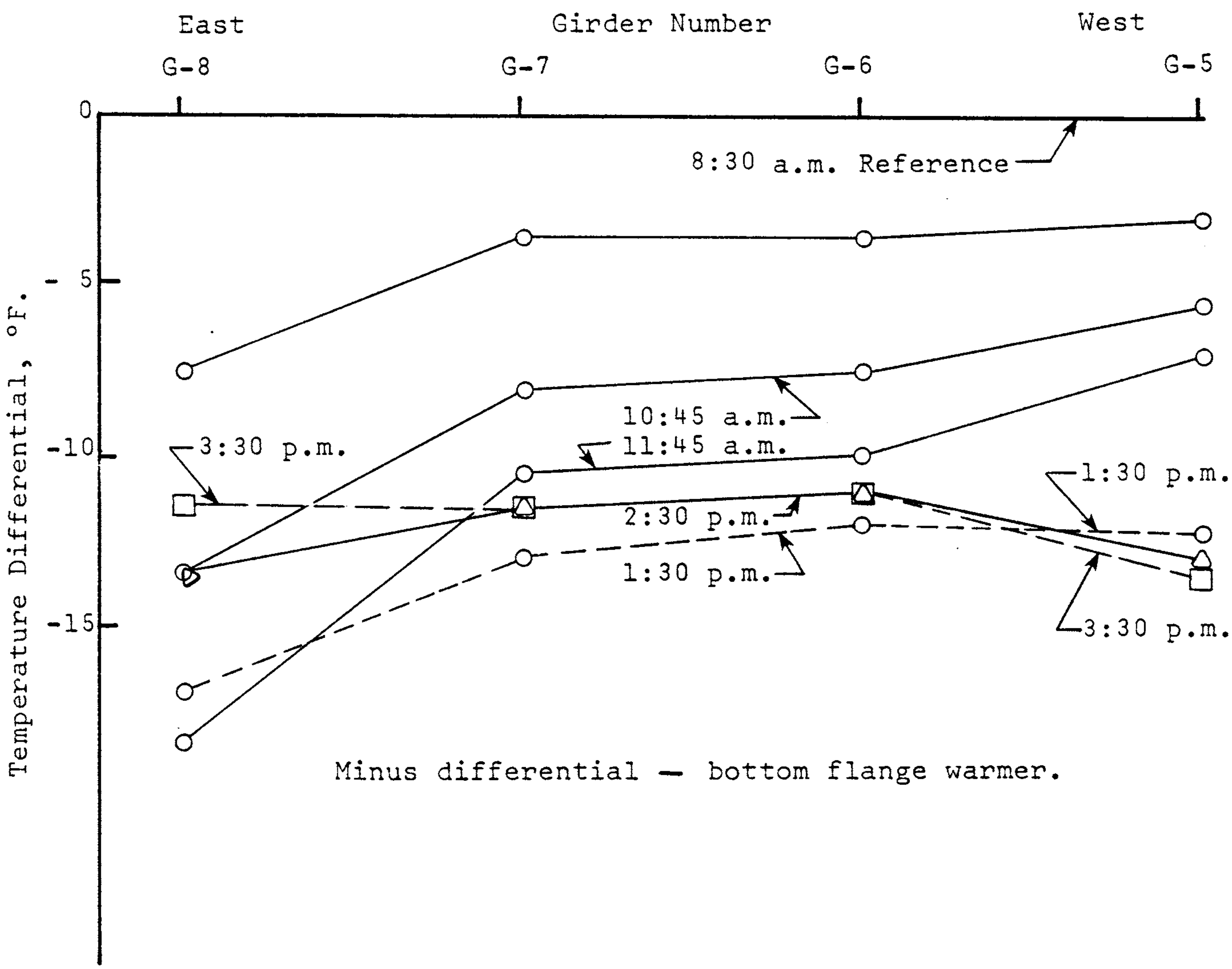

Figure 14. Temperature differentials between top and bottom flanges in composite section 19 days after deck placement.

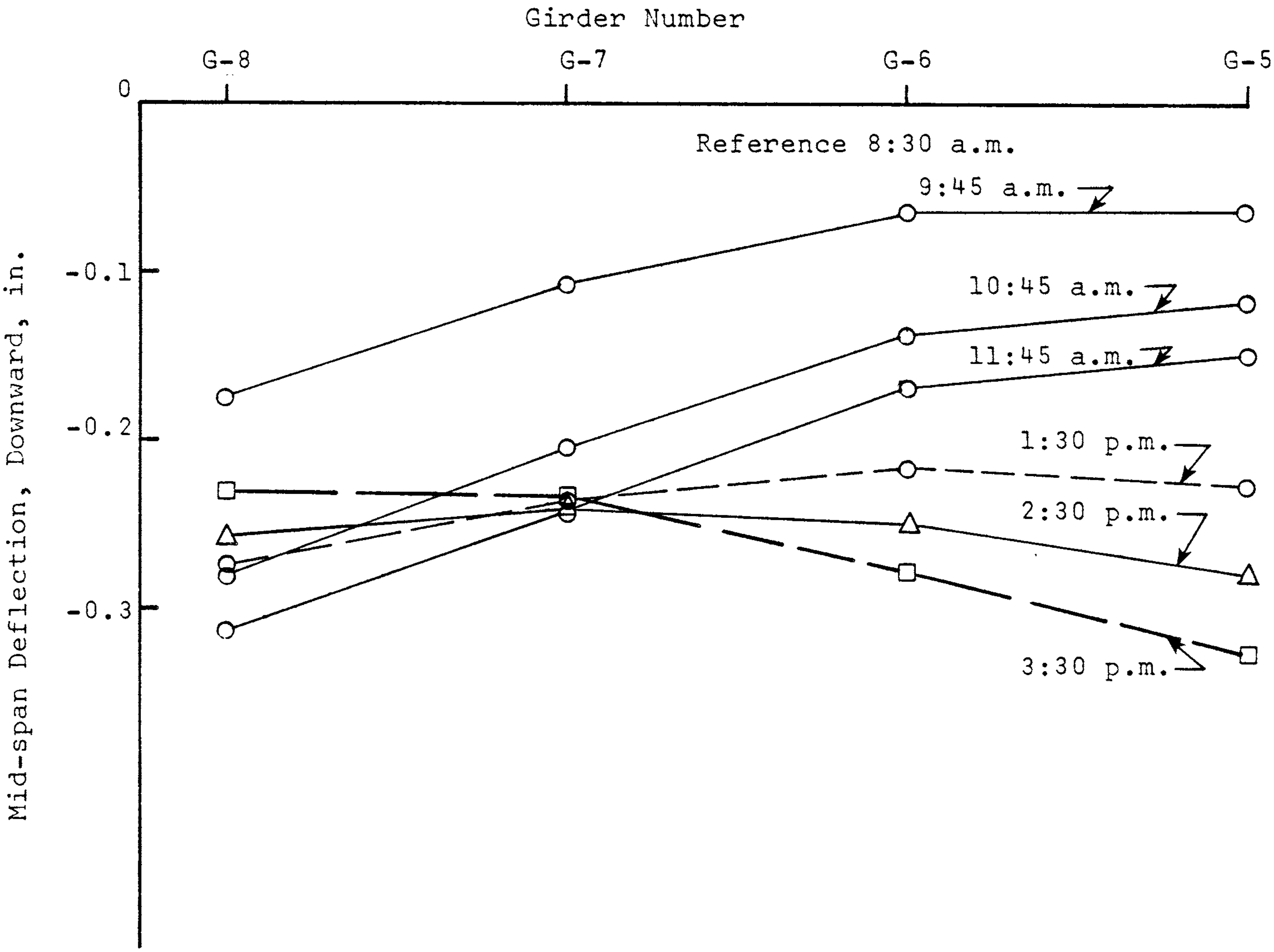

Figure 15. Downward deflections at mid-span due to differential temperatures between top and bottom flanges of the steel girders in composite section 19 days after deck placement.

To determine the deflections that occurred between the completion of the deck and 19 days later, the deflections that existed at 2:45 p.m. on August 21 were used as a base for comparison with those measured at 3:25 p.m. on September 9. The former data were selected because they were very nearly thermally neutral. The latter, however, showed the lower flanges to be approximately 10°F. warmer than the upper flanges. Using the 3:30 p.m. differential temperatures from Figure 14 and the corresponding deflections from Figure 15, thermal corrections were calculated by proportioning. Since the differential temperatures were nearly the same for each set of data, only a small reduction in the 3:30 p.m. thermal deflections had to be made. After calculating the corrections for the thermally-induced deflections and subtracting these algebraically from the measured deflections occurring between August 21 and September 9, deflections ranging from 0.243 in. on girder 6 to 0.313 in. on girder 8 still remained. These remaining deflections are thus a loss of camber in the steel girders that occurred sometime during the 19-day period and are reported in Table 5.

The camber loss, which ranges between 1/4 in. for girder 6 to 5/16 in. for girder 8, probably occurred within the first few days subsequent to placement of the deck. This is substantiated by a set of deflection measurements taken 5 days subsequent to the placement of the deck which indicated a camber loss on the order of 1/4 in. for all four of the girders.

Table 5

Deflections and Camber Loss 19 Days After Deck Placement, in.

Girder No.	Total Measured Deflections[a]	Thermal Deflections (Corrections)	Remaining Deflection[b] (Camber Loss)
G-5	-0.505	-0.226	-0.279
G-6	-0.504	-0.263	-0.241
G-7	-0.501	-0.201	-0.303
G-8	-0.510	-0.197	-0.313

[a]Difference between 2:45 p.m. August 21 and 3:25 p.m. September 9 (19 Days).

[b]Algebraic difference between measured and thermal deflections.

Dead Load Deflections in Composite Section

Twenty-two days after the deck was completed, concrete was placed for the last of the two parapet walls. The walls were placed on different days, thus allowing for the measurement of the steel girder deflections resulting from the weight placed first on the east and then on the west sides of the bridge.

The net dead load deflections of the girders were of the same order of magnitude for each wall. The deflections due to the placement of the west parapet wall are shown in Figure 16. As would be expected, the maximum deflection occurred at girder number 5. The deflection of each girder decreased transversely across the width of the span to girder number 8, which deflected the least. Deducting algebraically the movement due to the change in differential temperatures between the top and bottom flanges of the girders, the net deflections caused by the weight of the wall are designated in Figure 16 as the 6:10-p.m. (thermally neutral) curve.

The deflections of the girders due to the placement of the concrete for the east wall were quite similar to those shown in Figure 16, except that the maximum deflection occurred under girder 8 and the slope of the curves shown would be reversed.

The net dead load deflections of the girders due to the weight of both walls are given in Table 6. It can be seen that the net deflection is about the same for each girder, i.e., approximately 1/2 inch. Compared to the values given on the bridge plans, the actual deflections were lower on all girders except number 8, which was 1/8 in. higher. The plan deflections for the two walls varied from 3/8 in. for girder 8 to 7/8 in. for girder 5 — a difference of 1/2 in.—whereas the actual deflection varied by less than 1/16 in. between the maximum and minimum.

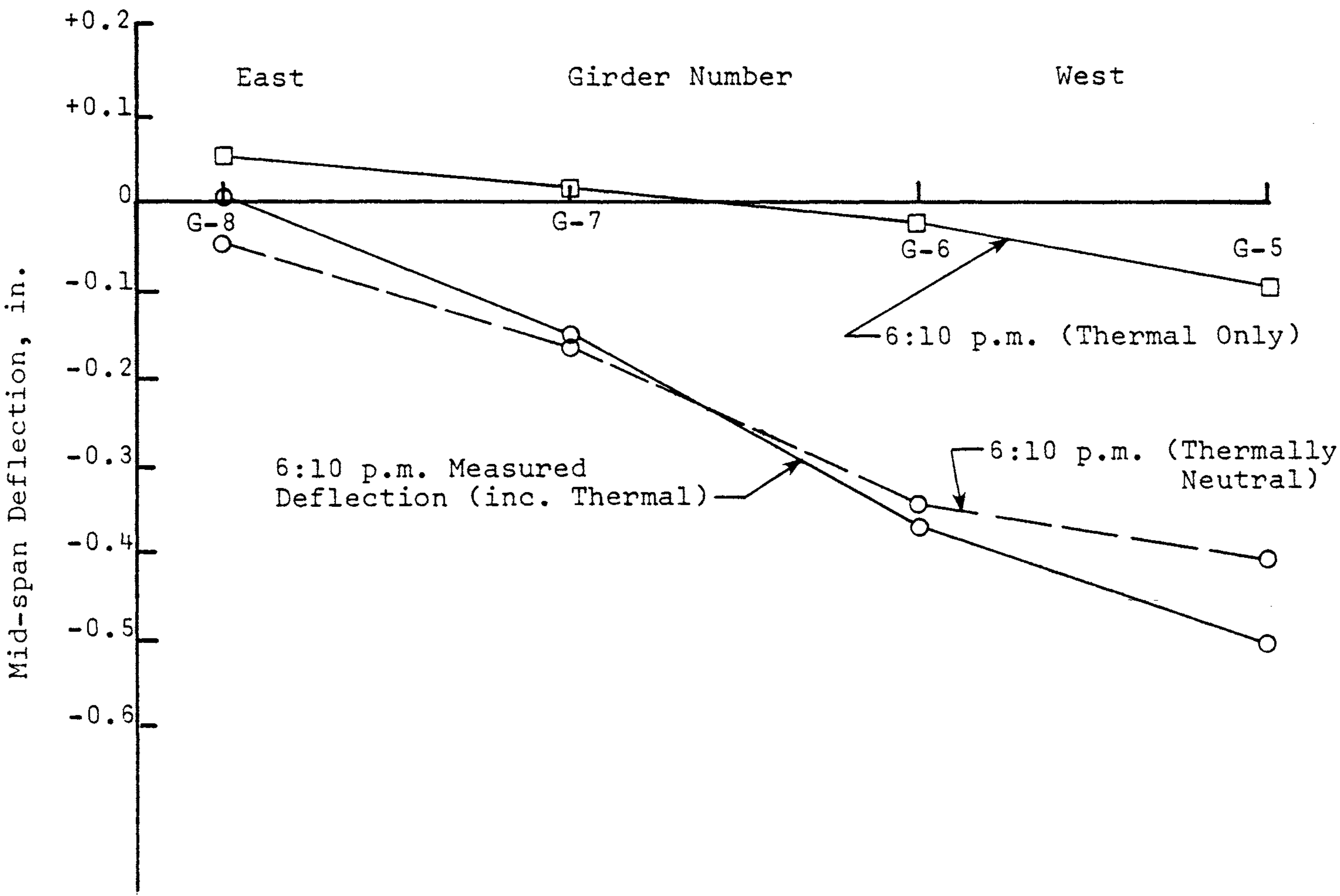

Figure 16. The deflections of the girders in the composite section due to placement of the concrete for the west parapet wall. The deflections due to differential temperatures are algebraically subtracted from the measured to yield those resulting from loading only.

Table 6

Mid-Span Dead Load Deflections of Steel Girders
Due to Placement of Parapet Walls, in.

Loading	Girder Number G-8	G-7	G-6	G-5
West Wall	0.048	0.169	0.351	0.412
East Wall	0.429	0.293	0.183	0.094
TOTAL	0.477	0.462	0.534	0.506
Plan Values	1/2"	5/8"	5/8"	3/4"
Difference	—	+5/32"	+1/8"	+1/4"

NOTE: Thermally neutral deflections.

Long-Term Camber Losses

With both the east and west parapet walls placed, the dead loading on the test span was essentially complete. Therefore, it was once again necessary to monitor the temperature-deflection characteristics of the span with all the construction loading in place. To allow the east parapet wall concrete to gain sufficient strength to be representative of that which would be effective over the next several months, a 4-day period was allowed to elapse before the thermal deflection data were recorded. These final temperature-deflection data were recorded in mid-September and the results are shown in Figure 17, which shows the differential temperature between the top and bottom flanges of the girders and the resulting deflections at various times of the day.

Once again, the differential temperatures showed the lower flanges to be warmer than the top, as would be expected. Accordingly, the thermally-induced deflections of the girders were downward. It can be noted from Figure 17 that a maximum downward deflection on the order of 0.21 in. was caused by a 10° F. thermal difference between the upper and lower flanges. Thus, for reasonably uniform differential thermal conditions a deflection of 0.022 in. per deg. F. could be expected.

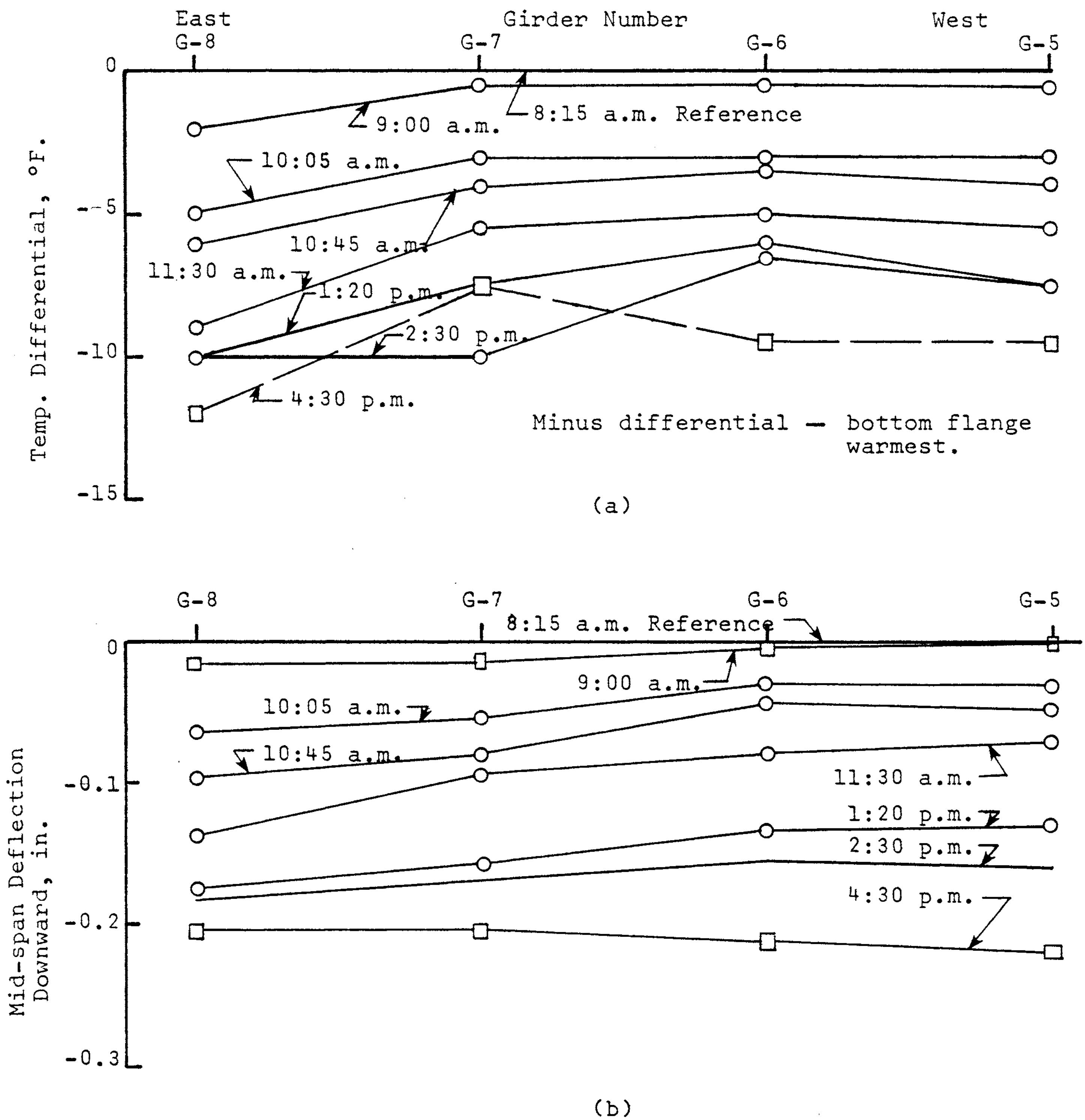

Figure 17. (a) Temperature differentials between top and bottom flanges in composite section, and (b) the resulting deflections 26 days after deck placement with parapet walls in place.

At the same time that the data discussed above were collected, initial readings for the measurement of long-term camber loss were recorded for each girder. Based on the thermal deflection data shown in Figure 17, corrections were applied to these initial readings to obtain a thermally neutral basis for subsequent comparisons. Since the structural steel was to be painted and other minor touch-up work remained to be finished, it was necessary to remove the camber measurement scales after the initial readings were acquired. As a result, it was necessary to carefully mark the horizontal and vertical position of the scales such that they could be reinstalled at a later date. It was recognized that it would not be possible to reinstall the scales at precisely the same vertical position in such a manner as to match the 0.001 in. capability of the high precision level; however, it was felt that the camber scales could be reinstalled to within plus or minus 1/32 in. of their original position.

On October 10, twenty-four days after the initial camber readings were taken, the bridge was opened to traffic. On April 29 of the following year, 202 days after the bridge was placed in service and 226 days after the initial long-term camber readings were recorded, the final camber measurements were recorded. Using the thermal data that were recorded simultaneously, the final readings were corrected to obtain the thermally neutral position of the girders. Both the initial and final long-term camber readings are reported in Table 7. With the exception of girder G-5, the initial and final readings were virtually the same. For girders G-6, G-7, and G-8 the data indicate that there was a 0.01 in. to 0.02 in. increase in camber. Since a difference of this order of magnitude is well within the expected experimental error, it is reasonable to conclude that there was no long-term camber loss of any practical consequence in either of these three girders. While the data indicate that the fourth girder, G-5, experienced an increase in camber of 0.13 in., it is not likely that this was the case. It is more likely that this result can be attributed to experimental error, although it is higher than that expected. Based on the average differences for all four girders, however, the results are very close to the expected error of plus or minus 1/32 in. related to removing and replacing the camber rods. It is, therefore, concluded that there was no camber loss of any practical significance in the span during the 226-day period that included 202 days under service (live) loading.

Due to vandalism, the bench-mark position used for the high precision level was lost and additional data were not collected beyond the time period discussed above. However, since no camber losses occurred during the almost 7-month period subsequent to construction, one would not expect camber losses beyond that time.

Table 7

Net Difference in Camber Over a 226-Day Period Subsequent to Construction

Camber Reading	Time Elapsed After Deck Placement: Days	Time Elapsed After Deck Placement: Months	Camber Rod Reading on Girder: G-8, in.	G-7, in.	G-6, in.	G-5, in.
Initial	26	0.87	6.15	7.18	10.27	8.87
Final	252	8.40	6.14	7.17	10.25	8.74
Difference	226*	7.53	+0.01	+0.01	+ 0.02	+0.13

*The bridge was under live load (service loading) for 202 days of the 226-day period.

CALCULATED VS. ACTUAL CAMBER LOSS OF THE HEAT-CURVED GIRDERS

When bridge girders are to be heat-curved to obtain horizontal curvature, the present AASHTO specifications for highway bridges require that an additional amount of camber be included in them during fabrication to compensate for possible losses during service as residual stresses dissipate.(3) The amount of camber, Δ, (including that which would be needed to offset anticipated dead load deflections) is given in the specifications as

$$\Delta = (\Delta_{DL}/\Delta_m)\,[\Delta_m + (0.02L^2F_y/EY_o)], \qquad (3)$$

where

Δ_{DL} is the camber in inches at any point along the length L calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads,

Δ_m is the maximum value of Δ_{DL} in inches within the length L,

E is the modulus of elasticity in ksi,

F_y is the specified minimum yield point in ksi of the girder flange,

Y_o is the distance from the neutral axis to the extreme outer fiber in inches (maximum distance for nonsymmetrical sections), and

L is the span length for simple spans or the distance between a simple end support and the dead load contraflexure point, or the distance between points of dead load contraflexure for continuous spans; L is measured in inches.

NOTE: Part of the camber loss is attributable to construction loads and will occur during construction of the bridge; total camber loss will be complete after several months of in-service loads. Therefore, a portion of the camber increase (approximately 50%) should be included in the bridge profile. Camber losses of this nature (but generally smaller in magnitude) are also known to occur in straight beams and girders.

Actually, only the second portion of the AASHTO formula pertains to the additional camber allowance for heat-curving. This part of the relationship was presented by Brockenbrough in 1970 as

$$\Delta_r = \frac{0.02\ L^2 F_y}{E Y_o}, \tag{4}$$

where Δ_r is the residual deflection to be offset by an increase in vertical camber at the point of maximum dead-load camber.[5]

Since the test structure is a simple span, curved girder design, the point of maximum dead-load camber is at mid-span. In addition, the flange plate thickness changes at certain points along the length of the girder, which results in a nonsymmetrical section. Noting that the specifications assume that 50% of the camber loss occurs during construction and the remainder under service loading, the values for additional camber were calculated from equation (4). For the construction loading, the maximum steel section, Y_o, was used and for the service loading the maximum composite section Y_o was used. This resulted in two calculations; one for the construction loading and one for the service loading. The calculated camber loss values are compared with the measured values in Table 8.

The camber losses calculated for the construction loads ranged from 1.16 to 1.20 in., with an average of 1.19 in. for the four girders. The measured camber losses ranged from 0.24 in. to 0.31 in., with an average of 0.28 in. — or only 24% of that predicted by the formula. The camber losses calculated for the service loads ranged from 0.90 to 0.95 in., with an average of 0.92 in. for the four girders. As discussed earlier, no service-load camber loss was detected in the field.

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Table 8

Calculated Vs. Actual Camber Loss of the Heat-Curved Girders

Girder No.	Span Length, ft.	Radius of Curvature, ft.	Steel Section Y_o, in.	Construction Loading Camber Loss		Composite Section Y_o, in.	Service Loading Camber Loss	
				Calculated, in.	Actual, in.		Calculated, in.	Actual, in.
G-5	138.45	834.51	41.1	1.16	0.28	49.6	0.95	0
G-6	138.58	823.84	39.9	1.20	0.24	52.2	0.91	0
G-7	138.72	813.18	40.3	1.19	0.30	52.5	0.91	0
G-8	138.91	802.51	40.4	1.19	0.31	53.3	0.90	0
		AVERAGE —		1.19	0.28		0.92	0

NOTE: A588 Steel; yield of 50 ksi

E = 29,000 ksi

The total camber losses calculated for the four girders ranged from 2.09 to 2.11 in., with an average of 2.10 in. as shown in Table 9. The total camber losses measured were only those classified as construction losses and averaged 0.28 in. — only about 13% of the average of those calculated for the four girders.

It should be noted that the radii of curvature of the four girders comprising the test span were greater than 800 ft. (Table 8), whereas those investigated by Brockenbrough were theoretically curved to radii in the 200 to 500 ft. range.[5] The shorter radii of curvature were developed by applying heat to a greater portion of the flange width. The relative residual vertical curvature remaining after loading was also greater in the shorter radii girders. Of the five girders investigated by Brockenbrough, all had radii of curvature less than 300 ft. when curved with type 3 heat (one-quarter of the flange width heated) and less than 470 ft. when curved with type 2 heat (one-sixth of the flange width heated). Brockenbrough's relationship for increase in vertical camber (equation 4) would thus appear to be applicable to girders heat-curved to considerably shorter radii than those tested in this study. Since the degree of heating and the radius of curvature appear to be related, residual stresses and thus loss of camber would probably be greater in girders curved to shorter radii.

The results of this study suggest that the AASHTO specifications relationship (equation 4) would not be applicable to girders heat curved to radii of 800 ft. or greater. Considering the magnitude of difference between the camber losses measured on the test structure and those calculated, equation (4) may not be completely applicable to girders heat-curved to radii in the 500-to-800-ft. range. In addition, the results suggest that the radius of curvature should be considered in calculating the potential camber loss in heat-curved girders.

Table 9

Total Camber Loss Comparison

Girder No.	Calculated, In.	Actual, In.
G-5	2.11	0.28
G-6	2.11	0.24
G-7	2.10	0.30
G-8	2.09	0.31
Average	2.10	0.28

Based on the results of the field measurements made in this study, some additional camber to offset that lost shortly after placement of the deck could be required. This would amount to approximately 24% of the amount that would be determined by calculating 50% of equation (4). Equation (4) could be modified to yield the camber loss found in this study by using a constant of 0.0024 to replace the 0.02. Thus, the average construction camber losses for the approximately 800-to-835 ft. radii of curvature girders that were studied could be determined from

$$\Delta = \frac{0.0024\ L^2 F_y}{E Y_o}$$

No service-load camber losses would be provided for.

SUMMARY OF CONCLUSIONS

The following conclusions are based on the results of the field study of deflections and camber loss of the heat-curved girders on the Route 726 bridge over Route 460.

Camber Loss

It should be noted that the camber losses discussed below do not include those that may have occurred between the steel fabrication plant and the job site, nor those that may have occurred prior to placement of the instrumentation on the structure. Camber losses may or may not have occurred during this period of time.

1. The results of the field study suggest that the relationship given for the calculation of the potential camber loss in heat-curved girders (article 1-7-14(c), AASHTO Standard Specifications for Highway Bridges, 1977) is not applicable to girders having radii of curvature greater than 800 ft.

2. Some camber loss due to construction loading occurs shortly after placement of the concrete deck. The amount of camber loss, however, is significantly less than that which would be predicted from the AASHTO specifications. For the study bridge, the camber loss due to construction loads was approximately one-fourth (24%) of that determined from the AASHTO equation.

3. There was no significant camber loss due to service loading after the bridge had been in service for approximately 6½ months. No measurements were taken beyond this point, but it would appear unlikely that additional losses of any consequence would have occurred.

4. The average total camber loss including both construction and service loading was approximately 13% of that predicted by the AASHTO equation.

5. Considering the magnitude of the differences between the camber losses measured on the test structure and those calculated by the AASHTO equation, the equation may not be completely applicable to girders heat-curved to radii in the 500-to-800-ft. range.

6. Since the amount of heat applied to the girders is related to the degree of curvature required, the results suggest that the radius of curvature should be considered in calculating the potential camber loss. The present AASHTO formula does not incorporate the radius of curvature as a factor in the calculation.

Dead Load and Thermal Deflections

1. The measured dead-load deflections were uniformly distributed across the four curved girders in the span — i.e., they were progressively larger from the inside girder to the outside girder—whereas the plan deflection values alternated from lower to higher values between girders. The diaphragm action between the girders tended to balance out the deflections. An average dead load deflection value is more representative of the actual deflection that occurred at each girder than are the independently calculated values.

2. The deflections of the girders resulting from the placement of the parapet walls were also more uniformly distributed across the span width than were the calculated plan values for each girder. An average dead load deflection value is more representative of the actual deflection that occurred at each girder than are the independently calculated values.

3. Settlement of the bridge bearings due to placement of the deck concrete was insignificant, averaging on the order of 0.01 in.

4. Differential temperatures between the top and bottom flanges of the curved steel girders can be significant, due to solar radiation, when the deck forms are in place prior to concrete placement. Upward deflections on the order of 1.25 in. were measured at mid-span during the early stages of construction. These deflections can be adequately calculated by an equation given in this report.

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RECOMMENDATIONS

Based on the results of this study, it is recommended that additional camber to compensate for losses expected in heat-curved girders be minimized, or omitted, for girders having radii of curvature greater than 800 ft. Some compensation on the order of 25% of that which would be calculated for construction losses only from the AASHTO equation could be added to the required camber to compensate for losses resulting from placement of the concrete bridge deck. No compensation for possible camber losses due to service loading should be required.

For heat-curved girders having radii of curvature in the 500-to-800-ft. range, the results of this study suggest that any compensation for camber losses calculated from the AASHTO equation would probably be too great. It is recommended, therefore, that the designers (or fabricators) use their discretion in this range until further information is available. Camber losses due to construction loading would appear, however, to be the most likely to occur.

For heat-curved girders having radii of curvature less than 500 ft., more study would be required to test the applicability of the AASHTO equation.

REFERENCES

1. "Survey of Curved Girder Bridges," by Subcommittee on Curved Girders, Joint AASHTO-ASCE Committee on Flexural Members, Civil Engineering, February 1973.

2. Structural Report: "Analysis and Design of Horizontally Curved Steel Bridge Girders," U. S. Steel Corporation, May 1963.

3. Standard Specifications for Highway Bridges, AASHTO, 11th ed., 1977.

4. Hilton, M. H., "A Study of Girder Deflections During Bridge Deck Construction," Virginia Highway Research Council, June 1971.

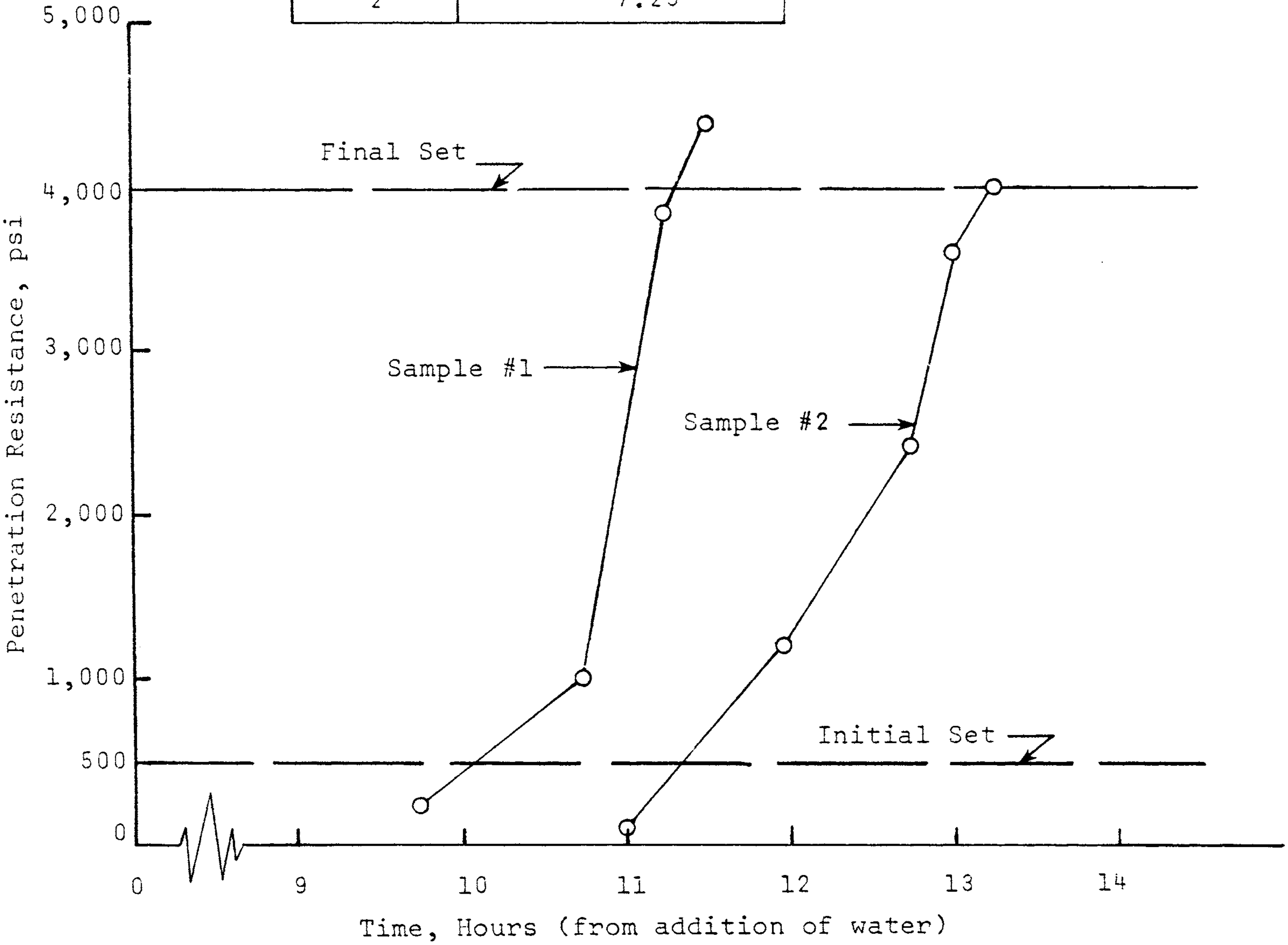
5. Brockenbrough, Roger L., "Criteria for Heat Curving Steel Beams and Girders," Journal of the Structural Division, American Society of Civil Engineers, Vol. 96, No. ST10, October 1970.

11.00

APPENDIX A

TIME OF SETTING OF CONCRETE

Sample No.	Final Set minus Deck Completion
	Hours
1	5.3
2	7.25



APPENDIX B

CONCRETE PROPERTIES

Sample No.*	Unit Weight, lb/ft^3	Slump, in.	Temperature, °F.	Air Content, %
1	145.4	3.25	80	5.6
2	143.4	2.5	82	6.6
3	144.1	3.25	85	5.5
Average	144.3	3.0	82.3	5.9

*Samples taken at the beginning and after one-quarter and three-quarters of the deck concrete had been placed.