Deflections and Camber Loss in Heat-Curved Girders

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ABSTRACT

To check the camber loss in heat-curved girders, a 140-ft, simply supported span was instrumented during the construction of a bridge. The span was composed of four steel plate girders that have 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 before and after construction of the bridge deck. Some loss in camber from 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 for predicting such loss. In addition, no significant camber losses were caused by service loading over a 6.5-month period subsequent to construction.

The construction of a number of curved girder bridges during the past few years has given rise to at least one question relating to the fabrication of the steel girders. This involves the requirement that additional camber be provided in steel girders that are to be heat curved. This additional camber is to allow for subsequent losses from the dissipation of residual stresses imposed by the heat-curving process during fabrication of the girders. The AASHTO specifications suggest that approximately 50 percent of the camber loss relating to the heat-curving process occurs during construction of the bridge, and an additional 50 percent occurs after a few months under service loading (1). Therefore, the increase in camber should be included in 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 suggested by the AASHTO specifications, 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 that they not be required to provide the additional camber in heat-curved girders because, in most cases, this adds to the time and expense of fabrication. To determine the nature of the deflections and camber loss in a heat-curved girder bridge, one bridge was instrumented and measurements were taken both during and after construction. The camber losses measured do not include those that may have occurred between the steel fabrication plant and the job site, or those that may have occurred before placement of the instrumentation on the structure.

STRUCTURE STUDIED

A curved girder bridge consisting of three simply supported spans, two of them relatively short at 36 and 18 ft, and the third 140 ft, was studied during its construction. All the measurements, however, were confined to the 140-ft span. A general view of the curvature of the steel girders is shown in Figure 1.

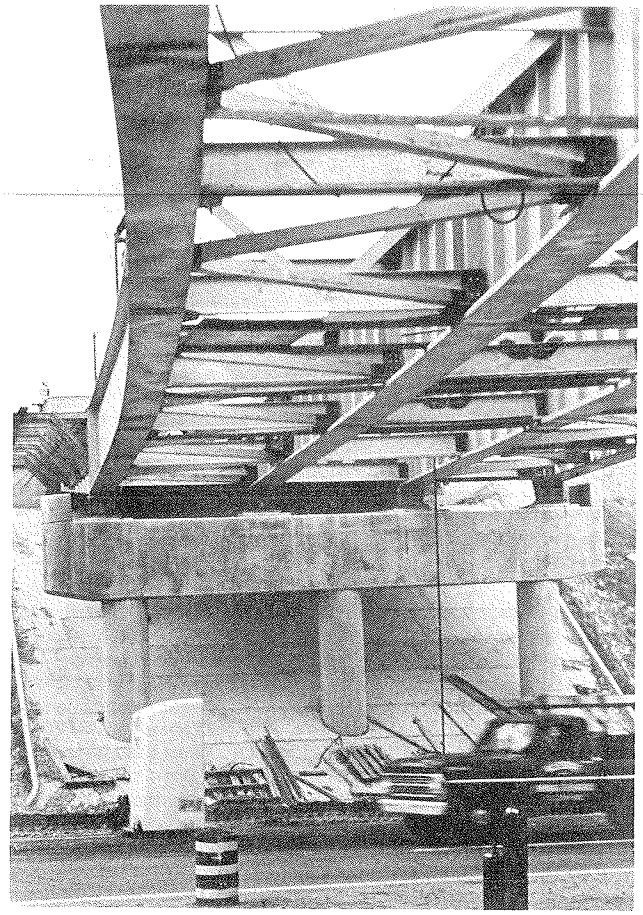

FIGURE 1 View showing curvature in the girders and design of diaphragms and lateral cross bracing.

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 1. The girders were fabricated from A588 steel and were heat treated to obtain the required degree of curvature. They 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-degree highway curve.

INSTRUMENTATION, TESTS, AND PROCEDURES

Girder and Bearing Deflection Instrumentation

Because 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 level, which is marked in 0.001-in. increments, was mounted on a trivet set in stationary bronze lugs on the top of the pier cap at the north end of the span. The line of sight of the level was thus slightly below the bottom flanges of the girders. Special design scales were installed at the midspan points of each girder and adjusted vertically to intersect the line of sight of the level. To measure and account for possible dead-load deflections of the bridge bearings, dial gauges were set as close as possible to the centerline of bearing of each girder.

Thermal Instrumentation

Thermocouples were placed on the top and bottom flanges at the midspan of the girders. They were also placed at the quarter-span points of the girders and at selected positions on the web of the girders.

During placement of the concrete deck and parapet walls a 24-channel temperature recorder scanned each gauge every 12 min. Other temperature measurements were taken before the concrete was placed and after each phase of the construction was completed to determine the effect of solar radiation on the deflection of the girders.

Tests on Plastic Concrete

Tests of the plastic concrete were restricted to the measurement of properties that would have the most direct influence on the girder deflections during deck placement. These included the times of initial and final set, unit weight, and temperature of the concrete.

Procedures

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 effects of differential thermal conditions. With the exception of brief delays during deck placement for taking measurements, the contractor's normal procedures were used during construction.

After construction was completed, the positions of 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 the girders, and the thermocouples were scanned to obtain data that were used to establish the differential temperature conditions. These data were then used as the basis for measurements of the long-term loss in camber after the bridge had been put into service. The gauges were then dismantled and later installed after the removal of the forming and painting of the structural steel.

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 is exposed to only the ambient air temperature. Because the alignment of the bridge is in a generally southerly to northerly direction, in the early morning the sun strikes the web and lower flange of the eastern girder and in late afternoon it 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 (2). The internal moment causes the girder to deflect upward by an amount relating to the intensity of the solar radiation, time of day, and so forth. The differential temperatures shown in Figure 2 were recorded at eight times from 7:30 a.m. to 3:20 p.m. on a typical sunny day in early August. At 7:30 a.m. the lower flanges were warmer than the upper flanges for all the girders, probably because they were 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°F was recorded on each of the two center girders.

The deflections that correspond to each of the reported differential temperatures are shown in Figure 3. The initial reference elevations of the girders were recorded at 7:30 a.m. As can be noted from these data, the maximum upward midspan deflections of the girders were on the order of 1.25 in. at 3:00 p.m. These data indicate that the thermal effects on girder deflections must be taken into account if there is an attempt to measure deflections that result from loading and from sustained losses of camber caused by dead weight or service loads.

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 girder 7 (Figure 4) indicate that the temperature increase caused by solar radiation on the top flange was transmitted downward through the web. Thus any calculations performed to determine thermal deflections must consider that the upper portion of the web above the neutral axis participates in the development of the forces and moments. In some instances, as the data in Figure 4 suggest, a portion of the web below the neutral axis was warmer 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 giders, 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 3. Because the 3:00 p.m. thermal deflection data were more uniformly distributed across the span width, theoretical calculations were made to determine the agreement with the field results. The calculated deflections were based on the following relationship:

$$F = AE\alpha\Delta T \tag{1}$$

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 = difference in temperature between the upper and lower flanges.

Because 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

$$\Delta = (A\alpha\Delta T d L^2)/8I \tag{2}$$

where

- d = distance from the force center to the neutral axis,
- L = length of span, and
- I = moment of inertia.

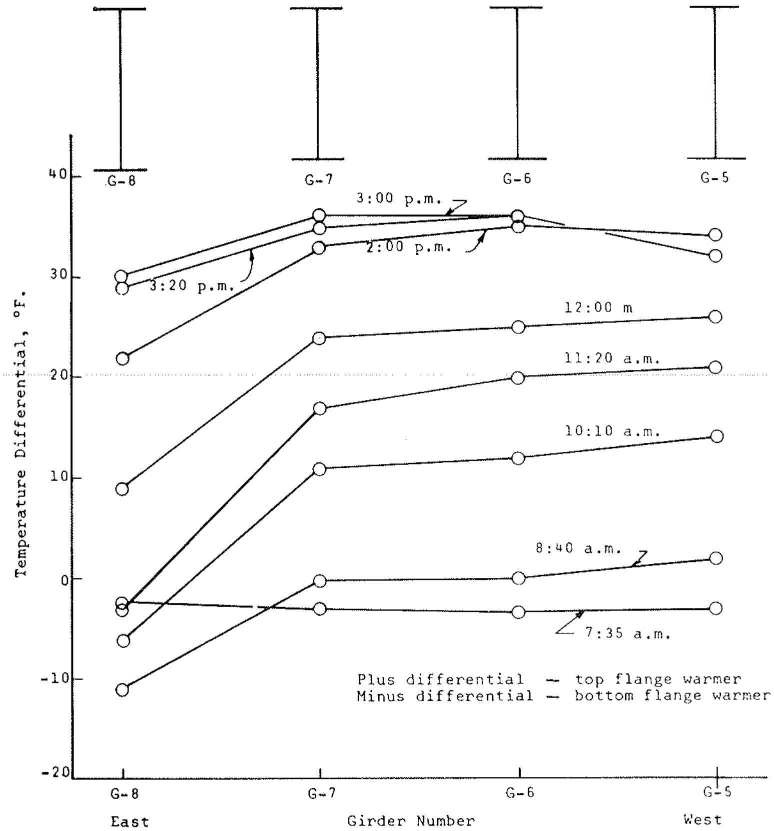

FIGURE 2 Temperature differential between top and bottom flanges of curved girder span with deck forming in place; before deck placement.

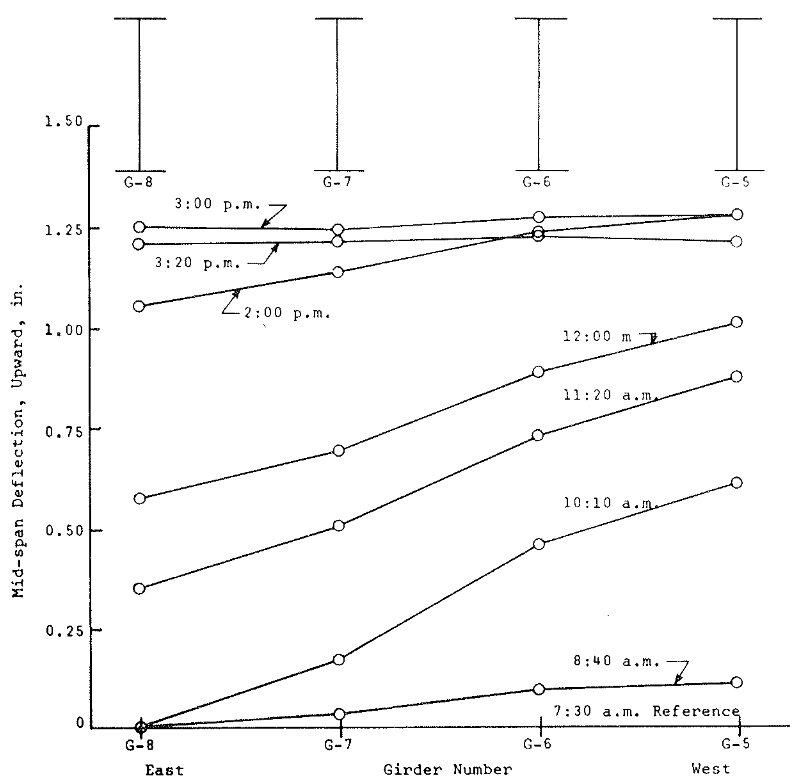

FIGURE 3 Upward deflections at midspan of curved girder span resulting from solar radiation; deck forming shielded lower portion of girders from sun.

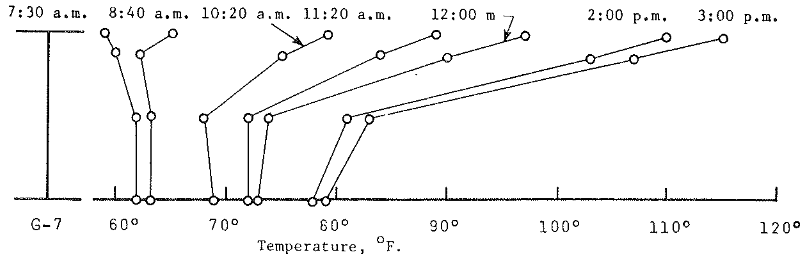

FIGURE 4 Thermal gradients through the depth of girder 7 caused by solar radiation; deck forming in place.

By using the midspan properties of the steel section, the thermal deflections calculated from Equation 2 were within less than 1 percent of the thermal deflections measured in the field at 3:00 p.m. in early August. Thus Equation 2 gives a reasonably satisfactory estimate of the deflections caused by differential temperatures that are relatively uniform across the width of the span.

Dead-Load Deflections in Steel Section

Reinforcing-Steel Placement

The first deadweight loading on the span (excluding the deck forming) was the reinforcing steel. Temperature measurements and deflection readings taken on the girders before placement of the deck concrete were compared with the initial readings taken approximately 12 days earlier. The thermal differentials were the same for these two periods except for a 0.5°F difference on two of the girders. By using Equation 2 and applying a correction to the measured deflections for those two girders, the thermal differentials were neutralized. The resulting deflections thus reflected only the downward movement caused by the deadweight of the reinforcing steel. The midspan dead-load deflections that result from the weight of the reinforcing steel were on the order of 0.15625 in. (Table 1).

TABLE 1 Midspan Dead-Load Deflections of Steel Girders Caused by Placement of Deck Reinforcing Steel and Concrete

	Midspan Dead-Load Deflection (in.) by Girder			
Loading	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	3.625	3.75	3.25	3.375
Difference	+1	+1	+0.4375	+0.46875

Note: Thermally neutral deflections.

Concrete Deck Placement

Concrete was placed on the 140-ft span beginning at 6:30 a.m. on August 21. At the beginning of the deck placement the temperature on the steel girders indicated that they were not in a thermally neutral position. Because the lower flanges 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 warmer than the lower flanges, but by 11:15 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 2:24 p.m. the temperature differential between the flanges was slight, and the span was, for all practical purposes, in a thermally neutral state.

Figure 5 shows the deflections of the girders at the various stages of concrete placement. These deflections existed at the stage of concrete placement indicated on the graph and include any amount caused by thermal differentials. At 11:25 a.m., approximately 5 hr 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 and 3:20 p.m., when the span was close to a thermally neutral position. As would be expected, the downward deflection was greater at these times, although the dead load on the girders remained unchanged from that which had existed at 11:25 a.m. Applying the corrections previously discussed to account for the initial thermal differentials yielded the final thermally neutral dead-load deflections that resulted from the weight of the concrete deck. These final values are shown by the lower curve in Figure 5 and are reported in Table 1.

Before calculating the thermal corrections for the deflection data, it was necessary to consider the setting time of the 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 determine the times of initial and final set, two samples--one at the beginning and one midway through the placement operation--were tested by using ASTM C403-68 procedures. It was found that the time of final set was approximately 5:45 p.m. for the first concrete placed on the span. For the concrete located in the midspan region, the final set occurred at 10:30 p.m. Based on these data, no composite action between the steel girders and concrete could be expected at 3:20 p.m., when the last deflection measurements were recorded. Accordingly, all calculations to determine thermal corrections to the girder deflections that occur during deck placement are based on the section modulus of the steel action only. Corrections to the deflections measured subsequent to the 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 average final deflections, or settlements, in the bearings at the north and south ends of the span are given in Table 1. As would be expected, the placement of the concrete caused some downward settlement of the bearings. These average

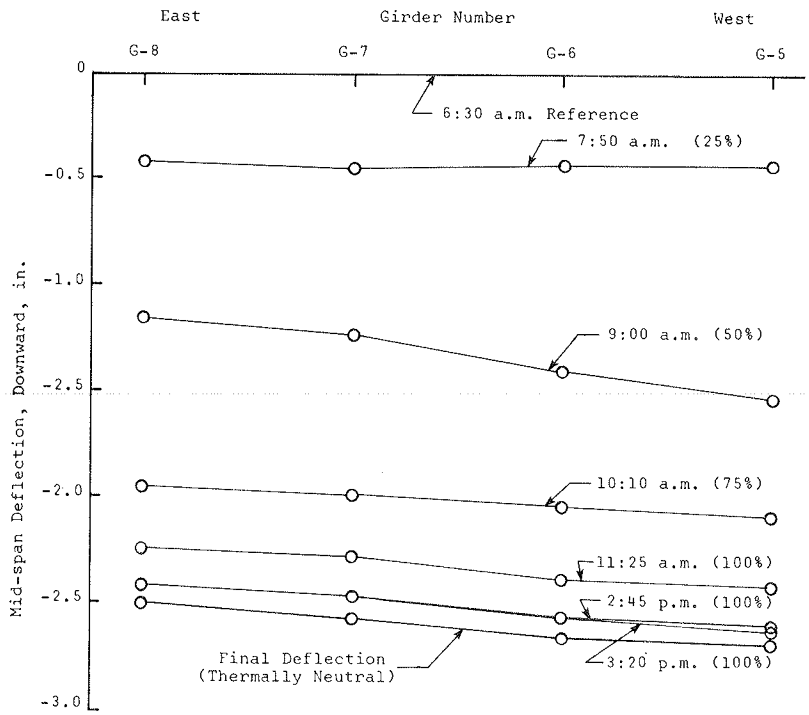

FIGURE 5 Downward deflections at midspan caused by placement of concrete deck.

values are thus deducted from the girder deflections measured on completion of the placement of the concrete deck.

The total measured deflections of the curved girders caused by the weight of the reinforcing steel and concrete are given in Table 1 and are lower than those given in the bridge plans. In addition, the measured deflections were progressively larger from the inside girder to 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 from 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.

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 it was expected that the girders would be at a higher elevation at midspan than they had been at 2:45 p.m. the day before. The 10:00 a.m. data in Figure 6 show this to be the case, as 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 6. By applying these as corrections to the measured data of August 22, the thermally neutral deflections shown by the lower curve in Figure 6 were within 0.01 in. or less of those measured on the previous day. Therefore, no deflections resulting from camber loss in the steel girders occurred during the first day after the deck was placed.

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

As discussed earlier, the data for 2:45 p.m. 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 was 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, because of 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: (a) to obtain data that could be used for making thermal corrections to all subsequent deflection measurements that would be recorded, and (b) 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 7 and 8, show the net thermal differentials between the top and bottom flanges of the steel and the deflections that result from the thermal loads, respectively. 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 7, the net temperature differ-

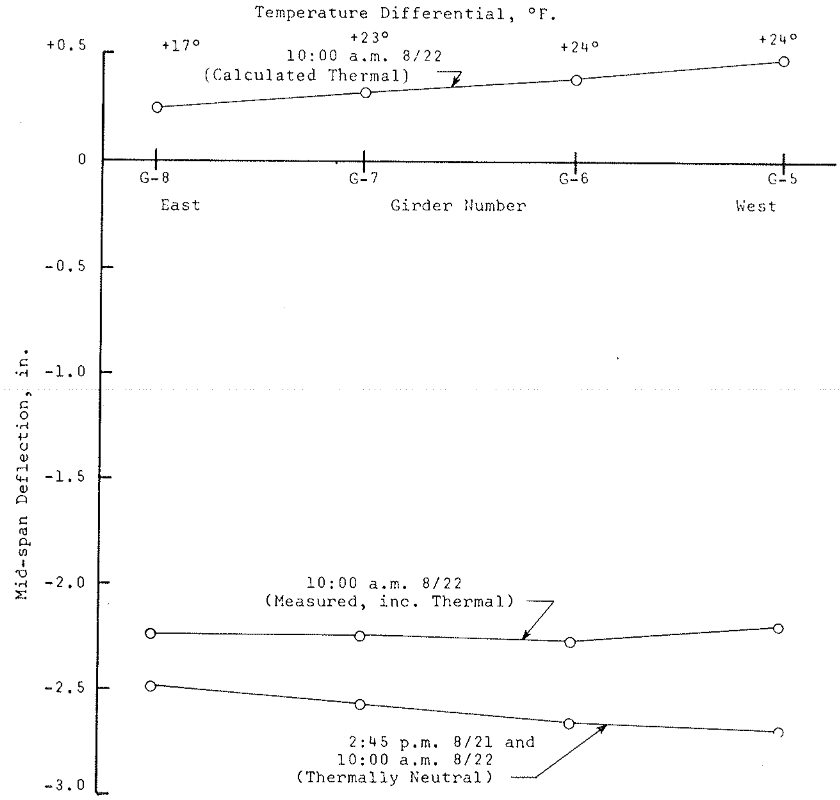

FIGURE 6 Deflections of girders 1 day after deck placement, showing effect of differential temperatures resulting from heat of hydration of concrete.

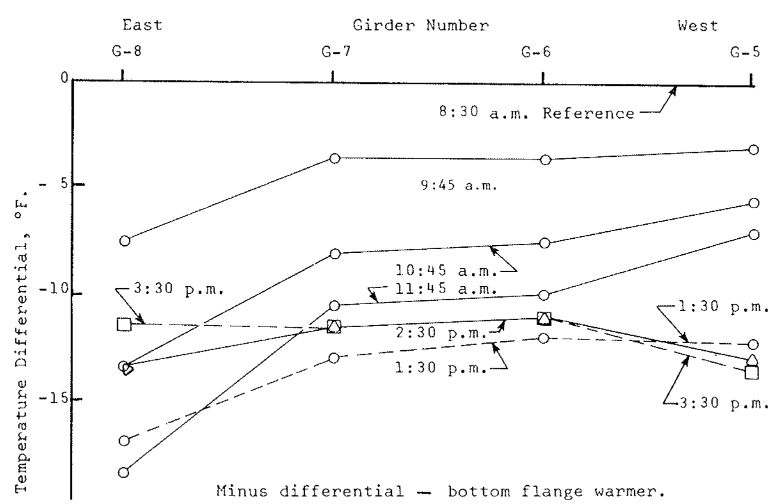

FIGURE 7 Temperature differentials between top and bottom flanges in composite section 19 days after deck placement.

ential 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 indicated that the exposed lower portion of the steel girders was 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 0.25 in. or more, as shown in Figure 8.

Because of the uneven distribution of the temperatures measured on the lower flange and web of each girder, it was difficult to use Equation 2 to calculate the thermal deflections of the composite section. Although 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, with 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. To determine the deflections that occurred between the

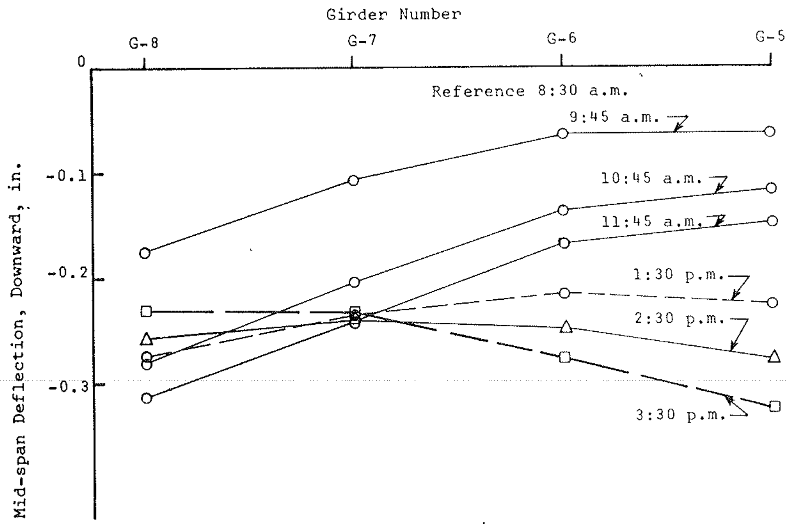

FIGURE 8 Downward deflections at midspan caused by differential temperatures between top and bottom flanges of steel girders in composite section 19 days after deck placement.

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 3:30 p.m. on September 9. The former data were selected because they were nearly thermally neutral. The latter data, however, indicated that the lower flanges were approximately 12°F warmer than the upper flanges. By using the 3:30 p.m. differential temperatures from Figure 7 and the corresponding deflections from Figure 8, thermal corrections were calculated by proportioning. Because 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 changes in 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 reported in Table 2 are thus a loss of camber in the steel girders that occurred sometime during the 19-day period.

TABLE 2 Deflections and Camber Loss 19 Days After Deck Replacement

Girder No.	Deflection and Camber Measurement (in.) 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	-0.300
G-8	-0.510	-0.197	-0.313

[a] Difference between 2:45 p.m. August 21, and 3:30 p.m. September 9 (19 days).
[b] Algebraic difference between measured and thermal deflections.

The camber loss, which ranges between 0.25 in. for girder 6 to 0.3125 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 0.25 in. for all four girders.

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 dead-load deflections of the girders from the weight of both walls are given in Table 3. The net deflection after correction for differential temperatures was about the same for each girder, that is, approximately 0.5 in. Compared with the values given on the bridge plans, the actual deflections were lower on all girders except number 8, which was 0.125 in. higher.

TABLE 3 Midspan Dead-Load Deflections of Steel Girders Caused by Placement of Parapet Walls

Loading	Midspan Dead-Load Deflection (in.) by Girder 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	0.5	0.625	0.625	0.75
Difference	–	+0.15625	+0.125	+0.25

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 completed span. 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, 4 days were allowed to elapse before the thermal deflection data were recorded. These final temperature-deflection data were recorded in mid-September, but are not reported here because they were similar to those data recorded on September 9. In general, however, 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 0.021 in. per degree F could be expected.

At the same time that the data just discussed were collected, initial readings for the measurement of long-term camber loss were recorded for each girder. Based on the thermal deflection data, corrections were applied to these initial readings to obtain a thermally neutral basis for subsequent comparisons.

On October 10, 24 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. By using the thermal data that were recorded simultaneously, the final readings were corrected to obtain the thermally neutral position of the girders. With the exception of girder 5, the initial and final readings were virtually the same. For girders 6, 7, and 8 there was a 0.01- to 0.02-in. increase in camber. Because a difference of this order of magnitude is well within the expected experimental error involved in reinstalling the deflection scales, it is reasonable to conclude that there was no long-term camber loss of any practical consequence in either of these girders. Although these data indicate that girder 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 expected. Therefore, it is concluded that there was no camber loss of any practical significance in the span during the 226-day period, which included 202 days under service loading.

CALCULATED VERSUS ACTUAL CAMBER LOSS OF HEAT-CURVED GIRDERS

When bridge girders are to be heat treated to obtain horizontal curvature, the current 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 (including that which would be needed to offset anticipated dead-load deflections) is given in the specification as

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

where

- Δ_{DL} = camber (in.) at any point along the length L calculated by usual procedures to compensate for deflection caused by dead loads or any other specified loads,
- Δ_m = maximum value of Δ_{DL} (in.) within the length L,
- E = modulus of elasticity (ksi),
- F_y = specified minimum yield point (ksi) of the girder flange,
- Y_o = distance from the neutral axis to the extreme outer fiber (in.) (maximum distance for nonsymmetrical sections), and
- L = span length (in.) for simple spans or the distance between a simple end support and the dead-load contraflexure point for continuous spans.

[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 percent) 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 (3) in 1970 as

$$\Delta_r = 0.02L^2F_y/EY_o \tag{4}$$

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

Because the test structure is a simple span, the maximum dead-load camber is at midspan. 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 percent of the camber loss occurs during construction and the remainder occurs under service loading, the values for additional camber were calculated from Equation 4. For the construction loading, the steel section Y_o was used, and for the service loading the composite section Y_o was used. This resulted in separate calculations for the construction and the service loadings. The calculated camber loss values are compared with the measured values in Table 4.

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 to 0.31 in., with an average of 0.28 in., only 24 percent 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.

The total camber loss calculated for the four girders ranged from 2.09 to 2.11 in., with an average of 2.10 in. The total camber losses measured were only those classified as construction losses and averaged 0.28 in., only about 13 percent 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, whereas those investigated by Brockenbrough were curved to radii in the 200- to 500-ft range (3). 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 the 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. Because the degree of heating and the radius of curvature appear to be related, residual stresses and thus loss of camber

TABLE 4 Calculated Versus Actual Camber Loss of Heat-Curved Girders

Girder No.	Span Length (ft)	Radius of Curvature (ft)	Steel Section Y_o (in.)	Construction Loading Camber Loss (in.)		Composite Section Y_o (in.)	Service Loading Camber Loss (in.)	
				Calculated	Actual		Calculated	Actual
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
Avg				1.19	0.28		0.92	0

Note: A588 steel; yield of 50 ksi. E = 29,000 ksi.

would probably be greater in girders curved to shorter radii.

The results of this study suggest that the AASHTO specifications relationship (Equation 4) might 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 might be a factor in calculating the potential camber loss in heat-curved girders.

SUMMARY OF CONCLUSIONS

1. The results of the study suggest that the relationship given for the calculations of the potential camber loss in heat-curved girders [article 1-7-14(c), AASHTO Standard Specifications for Highway Bridges (1)] may not be applicable to girders that have radii of curvature greater than 800 ft.

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

3. There was no significant camber loss caused by service loading after the bridge had been in service for approximately 6.5 months.

4. The average total camber loss, including both construction and service loading, was approximately 13 percent 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, the AASHTO equation might not be completely applicable to girders heat curved to radii in the 500- to 800-ft range.

6. Because the amount of heat applied to the girders is related to the degree of curvature required, the results suggest that the radius of curvature might be a variable that should be considered in calculating potential camber losses.

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