FINAL REPORT

A STUDY OF GIRDER DEFLECTIONS DURING BRIDGE DECK CONSTRUCTION

by

M. H. Hilton
Highway Research Engineer

Virginia Highway Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia Department of Highways and the University of Virginia)

In Cooperation with the U. S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

June 1971
VHRC 70-R49

SUMMARY

Problems involved in obtaining the desired thickness of bridge decks were investigated. The study, which was limited to decks which were longitudinally screeded during construction, included (1) field measurements of the girder deflections during construction, and (2) a theoretical frame analysis of the girder deflections under the field loading conditions. Each of the two spans investigated were simply supported steel plate girder designs.

When full span length longitudinal screeding is used, the finished grade elevations are set on the screeding edge of the machine, and remain independent of the bridge girder deflections during deck placement. Consequently, any factor affecting the girder deflections, and thus the forming elevations, will, in turn have bearing on the final thickness of a bridge deck. In addition, all factors which, in effect, cause the deck forming to be too high at the time the concrete is screeded to grade have the potential of causing a shy deck thickness. The most significant factors were found to be:

(1) Plan dead load deflection values which are in error,

(2) the differential temperatures existing between the top and bottom flanges of the girders during concrete placement as opposed to those that may have existed when the forming elevations were established, and

(3) the transverse position of the concrete dead loading at the time a final screeding pass is made over a given point on a span.

Based on the results of the study, certain recommendations are offered regarding the computation of dead load deflections and precautions to be observed during construction when longitudinal screeding of the concrete deck is used.

CONCLUSIONS

The following conclusions are based on the results of a field and analytical study of the deflections of two simply supported steel plate girder spans during the construction of the bridge decks. The conclusions pertain to bridge decks constructed by use of the full span longitudinal concrete placement and screeding technique.

1. Differential temperatures between the top and bottom flanges of steel girders can be quite high -- due to solar radiation -- when the deck forms are in place. The resulting effect is an upward deflection of the girders. If bridge deck forms are established to grades complying essentially with a thermally neutral condition on the girders, but the concrete deck is longitudinally screeded to grade during differential thermal conditions, a shy deck thickness could result. Upward midspan deflections on the order of 0.40 inch due to solar radiation were measured on a 96'-2" long steel girder span.

2. It is apparent that exact steel girder elevations cannot be established when any degree of solar radiation is present. On different days having similar weather, temperature, and solar conditions, however, the elevations of the girders will be close to identical at approximately the same time of day.

3. The heat of hydration of plastic concrete prior to initial set would have an insignificant effect on girder deflections for warm weather deck placement conditions. The evidence suggests that solar radiation, changes in air temperature, and the initial temperature of the plastic concrete influence girder temperatures more than does the heat of hydration. In this respect, it should be noted that differential temperatures could develop during cold weather concrete placement as well as during warm weather placement.

4. The average compression of the neoprene bearing pads due to the dead load of the concrete deck was on the order of 0.02 inch, which does not warrant consideration in the calculation of dead load deflections.

5. This study and others[2] indicate that there is a tendency for plan dead load deflections to be in error on the high side. Thus, the deck forms would be set too high and with full span longitudinal screeding a shy deck thickness would result. Plan deflection errors are believed to be due to designers including the dead weights of all superstructure components rather than that of the concrete deck only.

6. The field deflection measurements show that the structural steel framing of each of the two spans tested acted as a unit due to the diaphragm connections between the girders.

2358

7. A comparison of the field deflection data with a theoretical analysis of deflections of semirigidly connected girders suggests that the bolted diaphragm connections on the two study spans act in a semi-rigid fashion. It was estimated from the comparison that the connections have an end fixity factor of approximately 0.20, which, in effect, is not greatly different from a rigid connection with an end fixity factor of 1.0.

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

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

RECOMMENDATIONS

When full span longitudinal screeding is to be used for constructing simple span bridge decks, it is recommended:

1. That initial girder elevations to be used for calculating the deck forming elevations be established when the thermal conditions on the girders will approximate those expected to prevail at the time the concrete is screeded to grade. The deck forms should be checked and adjusted vertically when solar radiation is representative of the most extreme conditions that might be expected on the day of deck placement. Assuming sunny conditions, early-to mid-afternoon on the day before concrete placement would normally be representative of the most extreme (hottest) solar conditions. Very early or very late concrete placement operations, when solar radiation is not present, would of course eliminate this problem. In this case the forming elevations should be established when the girders are most likely to be in a thermally neutral position, i.e., when the top and bottom girder flanges are at the same temperature.

2. That the plan dead load deflections of the girders be checked before construction to assure that the values are based on the dead load of the deck concrete only.

3. That the correct plan dead load deflection values be reduced by 25% to provide a compensating safety factor for instances when the last pass of the screeding machine follows too close behind concrete placement.

4. That during deck placement the final pass of the longitudinal screeding machine lag behind concrete placement by at least three bay lengths whenever practical. A bay length is defined, for this purpose, as the distance between adjacent girders.

5. That for bridge spans with large skew angles (10° or greater) the dead load deflections be checked by a computer frame analysis similar to that used in this study and included in Appendix C. For the frame analysis the deflections of each girder should be based on the three-bay-lag behind concrete placement principle. If the resulting values are lower than those obtained by conventional calculations, the lower values should be used for establishing deck forming elevations.

FINAL REPORT

A STUDY OF GIRDER DEFLECTIONS DURING BRIDGE DECK CONSTRUCTION

by

M. H. Hilton
Highway Research Engineer

INTRODUCTION

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

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

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

One factor of concern regarding the deck thickness problem involves the effect on deflections of interconnecting diaphragms between the bridge girders. Conventional procedures for computing plan dead load deflection values assume that diaphragm connections are hinged, i.e., that each girder is free to deflect independently under the dead load of that portion of the concrete deck it would carry. When concrete is placed down one side of a bridge span, as is the case when a deck is to be longitudinally screeded, the deflections of girders directly under the load will be partially restrained by the interconnecting diaphragm action with the unloaded girders.

Figure 1. A longitudinal type bridge deck screeding machine. A longitudinal work bridge lies to the right behind the screeding machine.

Thus, if the concrete deck is struck off to grade over one girder before concrete is placed over the remaining girders, then the deflection of this girder will not be as much as calculated and the deck will be shy by the difference. An earlier theoretical analysis(1) of bridge girder deflections during concrete decking indicated that deficient deck thicknesses could result where longitudinal screeding follows too closely behind concrete placement. This analysis, however, assumed full rigidity at all diaphragm connections. For bolted connections, which currently are widely used, the assumption of a rigid joint may not be applicable under the variable loading conditions existing during deck placement. An earlier investigation(2) of a shy bridge deck thickness, for example, suggested that partial restraint of deflections during deck placement will occur where rigid cross frame diaphragms are used. In the present study, field measurements and a theoretical analysis of semirigidly connected simple span bridge girders were used to investigate the actual vs. the theoretical deflections occurring during bridge deck construction. The theoretical analysis of semirigidly connected bridge girders was based on a computer program developed especially for the study by Lisle.(3)

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

PURPOSE AND SCOPE

The general purpose of the study was to determine the order of magnitude of the effects of several variables on the deflections of a simple span bridge girder system during the placement and screeding of the concrete deck. More specifically, the objectives of the investigation were as follows:

1. To investigate the girder deflections at progressive stages of concrete deck placement, and to evaluate the adequacy of the conventional method of computing plan dead load deflections for a bridge deck that is to be placed and screeded longitudinally over the full span length.

2. To estimate — by use of a comparison of the theoretical and field data — the degree of diaphragm connection rigidity on the particular spans selected for study.

3. To investigate the theoretical effects of diaphragm connection rigidity on the deflections of a girder system, and to compare the results with actual deflection data obtained during progressive stages of deck placement.

4. To obtain field data on the differential thermal conditions between the upper and lower flanges of steel girders due to solar heating prior to and the hydration heat of concrete subsequent to concrete deck placement.

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

Structures Studied

One span on each of two bridges was selected and instrumented for field study during the construction of the decks. These spans, which were constructed by the Central Contracting Company of Farmville, Virginia, were:

1. Span #3 of the Rte. 607 bridge over interstate Rte. 64, Louisa County, construction project 0064-054-101, B609, and

2. Span #4 of the southbound lane of Rte. 15 over interstate Rte. 64, Louisa County, construction project 0064-054-101, B606.

The Rte. 607 span was composed of six parallel girders; the Rte. 15 span was composed of seven. The steel framing diagrams showing dimensions and locations of the test instrumentation (described later) for the Rte. 607 and Rte. 15 spans are given respectively in Figures 2 and 3. Typical cross sectional views of the superstructure showing the girder and diaphragm configurations of the two bridges are shown later in Figures 19 and 28.

LEGEND

- High precision level.
- ○ Scales attached to lower flange of girders.
- Reference target mounted on face of pier cap.
- ⊗ Thermocouples on top and bottom flanges of girders.
- Thermocouples on top and bottom flanges and mid-depth of web of girders.
- Thermocouples on top flanges of girders only.
- Dial gages at girder bearings.
- ⊠ Strain gages on diaphragms.

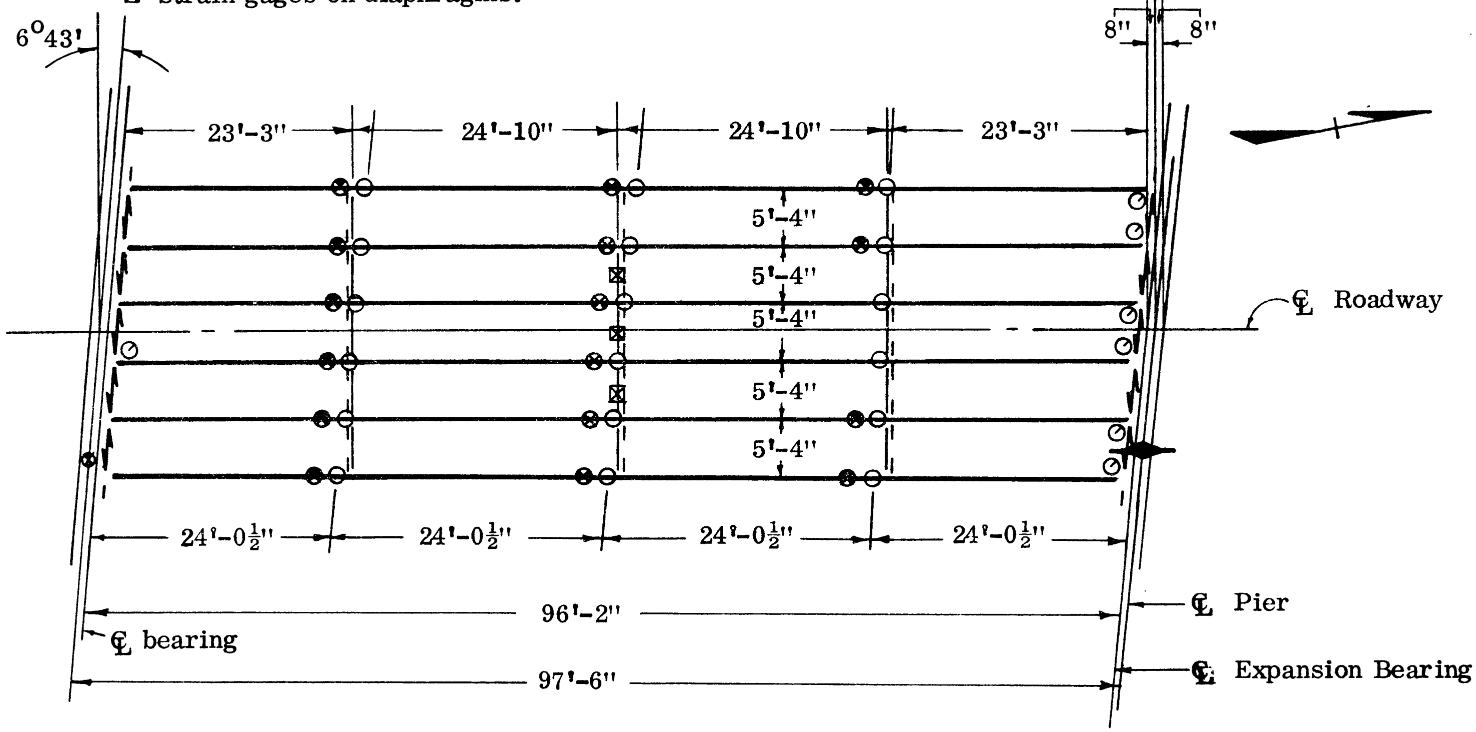


Figure 2. Framing diagram of Span #3, Rte. 607 over Rte. 64 showing location of instrumentation.

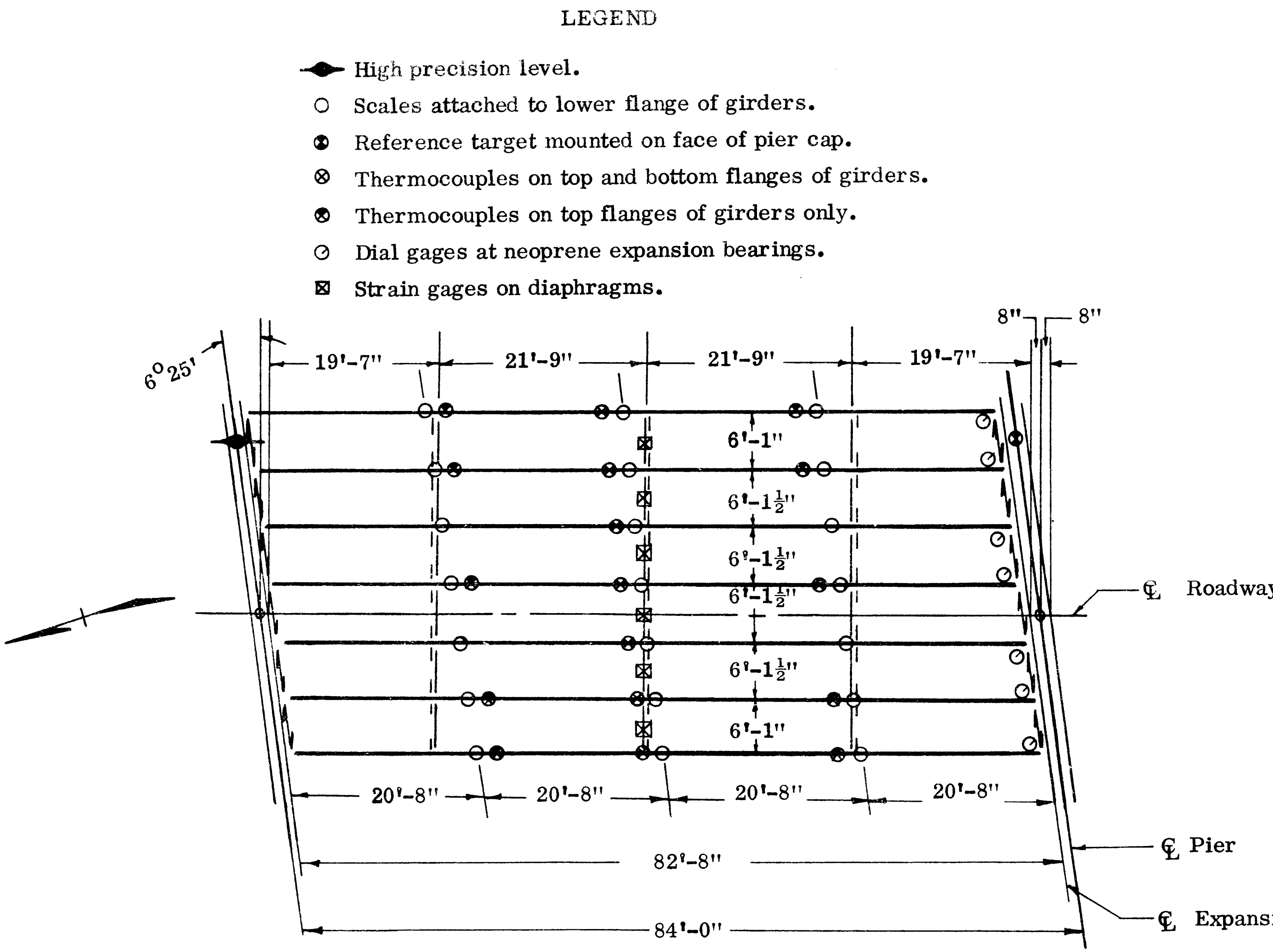


Figure 3. Framing diagram of Span #4, Rte. 15, SBL over Rte. 64 showing location of instrumentation.

INSTRUMENTATION, TESTS, AND PROCEDURES

Since the field measurements were made on actual structures during the construction of the bridge decks, 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 spans 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 roadway grading. Thirdly, for obvious reasons, 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 operations was limited to that which could be handled in approximately ten to fifteen 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 described below.

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 as the most feasible instrument for use in the study. 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.

For each of the two spans studied the level was mounted on a trivet that in turn was set in stationary lugs on the top of the lowest elevation pier cap at one end of the span. In addition, the level was centered on the cap directly above one of the circular pier columns. The line of sight of the level was thus slightly below the bottom flanges of the steel girders. Figure 4 shows the level mounted on top of a pier cap of the Rte. 607 bridge. In order to sight through the level it was necessary that the operator lie in a prone position. This requirement was facilitated by erecting scaffolding behind the pier as shown in Figure 5.

Special design rod and scale units were installed at the quarter points of each girder on the spans tested. As illustrated in Figure 6, the rod and scale unit was mounted in an adjustable bracket that in turn was attached to a large "C" clamp. The "C" clamp, which was fabricated for use in this particular study, was 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, one-foot long, engineer's scales with half inch major graduations were mounted to the rods and adjusted vertically so that all scales would intersect the line of sight of the level. Finally, a reference scale was mounted to the pier cap at the opposite end of the span from that of the position of the level instrument. A view of the rod and scale attachments on the underside of the Rte. 607 span is shown in Figure 7. The locations of all the deflection measurement instrumentation are given on the framing diagrams of Figures 2 and 3 for each span tested.

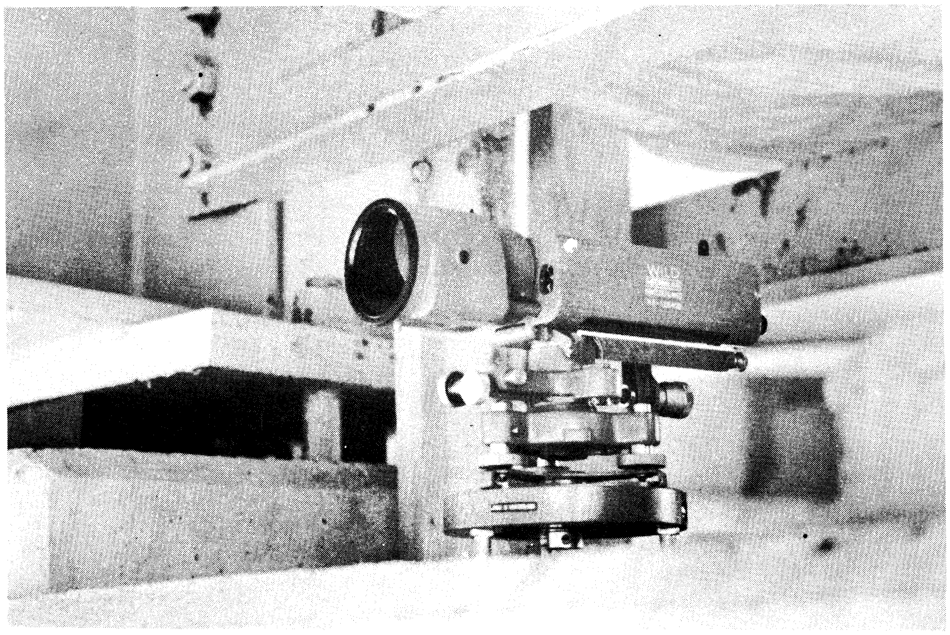


Figure 4. A view of the precision level and trivet positioned on top of a bridge pier cap.

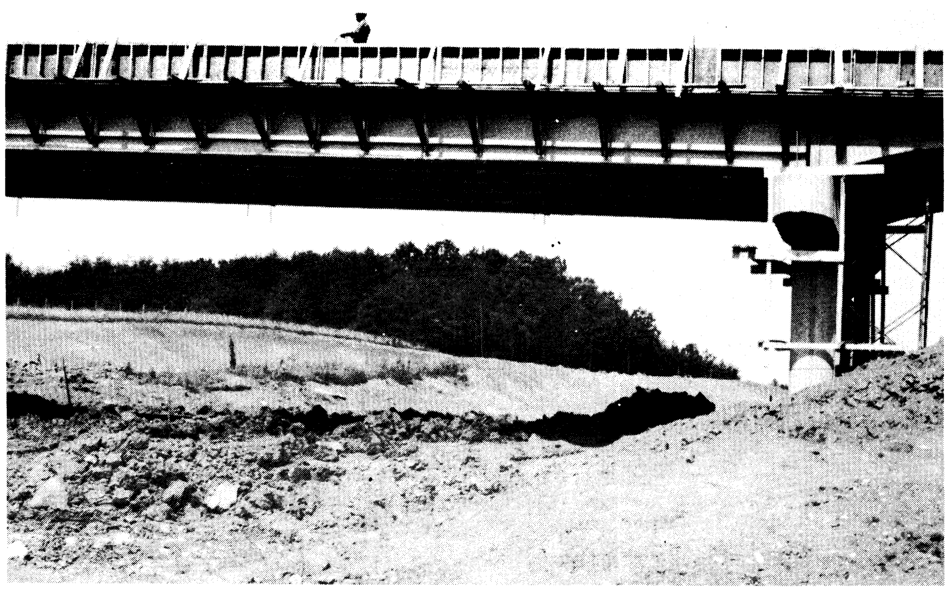


Figure 5. A view of part of the instrumented span on the Rte. 607 bridge showing the scaffolding which was erected around the pier. The level operator was positioned on top of the scaffolding to the right of the pier shown.

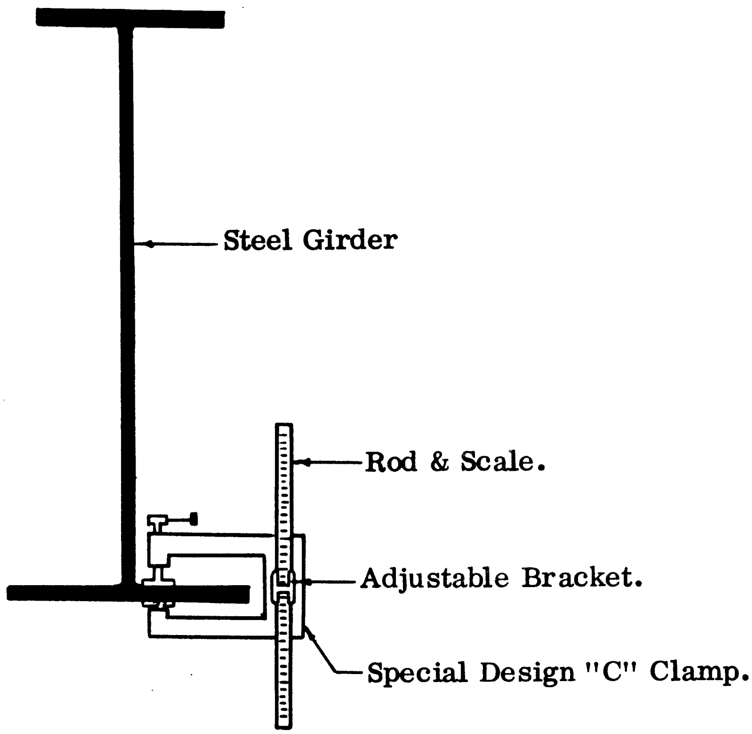


Figure 6. Details of a typical rod and scale unit attached to the lower flange of a bridge girder.

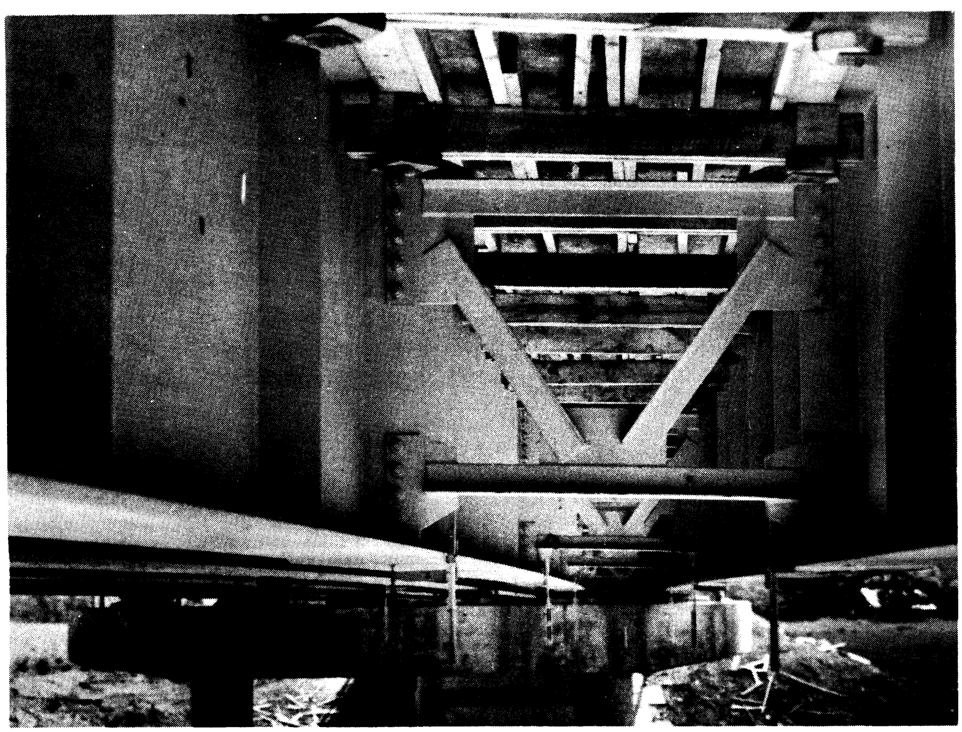


Figure 7. The scale units attached to the lower flanges of the steel girders as viewed from the position of the level instrument. A typical diaphragm connection is also shown (Rte. 607).

Thermal Instrumentation

A twenty-four channel Honeywell thermocouple recorder powered by a portable generator (Figure 8) was used to collect temperature data on the steel girders. Thermocouples, using a type J iron-constantan wire, were placed on the top and bottom flanges of the girders at the midspan length points. The remaining channels on the recording device were utilized by placing thermocouples on the girders at the quarter-span length points. The top flanges were emphasized in this case since it was expected that temperature variations would be greatest on the top side due to cloud cover and other factors affecting the sun's radiation. In addition, during concrete placement operations, the larger number of gages were needed on the top flanges to monitor the effects of variations in the positioning of the fresh concrete on the thermal conditions of the steel girders. On the Route 607 span, thermocouples were placed at mid-depth of the web of the two outside girders. A typical installation on the top flange of a girder is shown in Figure 9. Locations of all the gage points on each span tested are given in Figures 2 and 3.

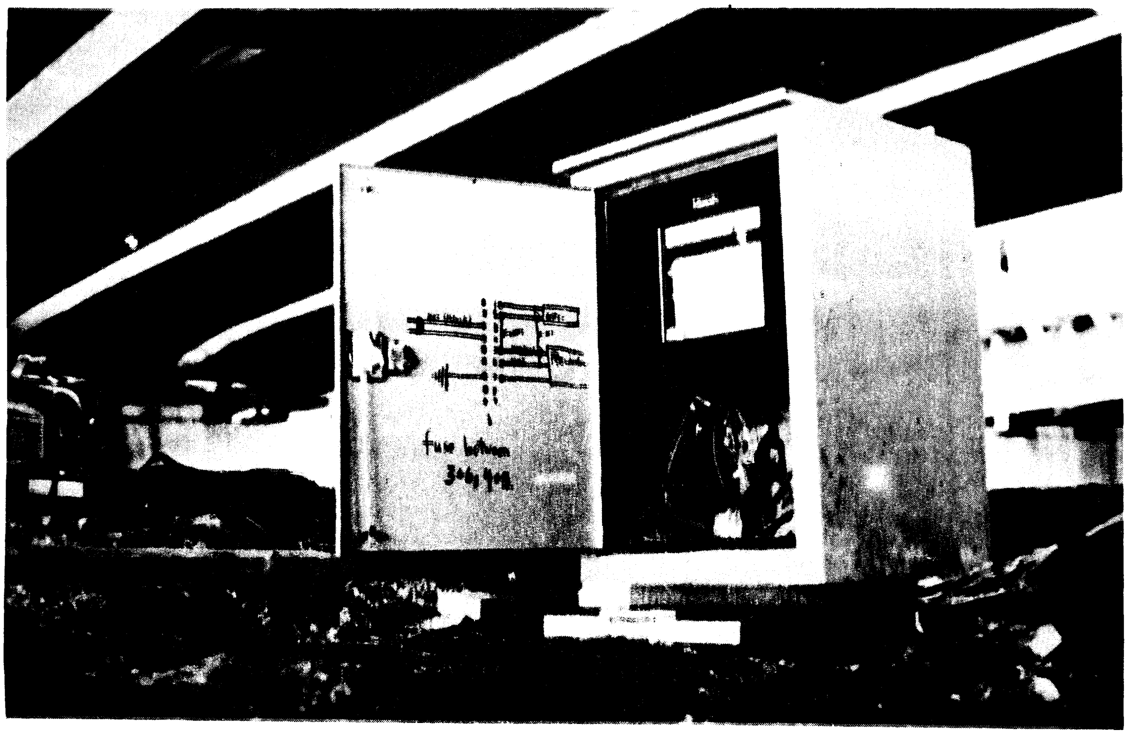


Figure 8. The Honeywell thermocouple recorder mounted in a steel cabinet for field use. A portable generator to the left of the recorder supplied the operating power (Route 15 span).

2370

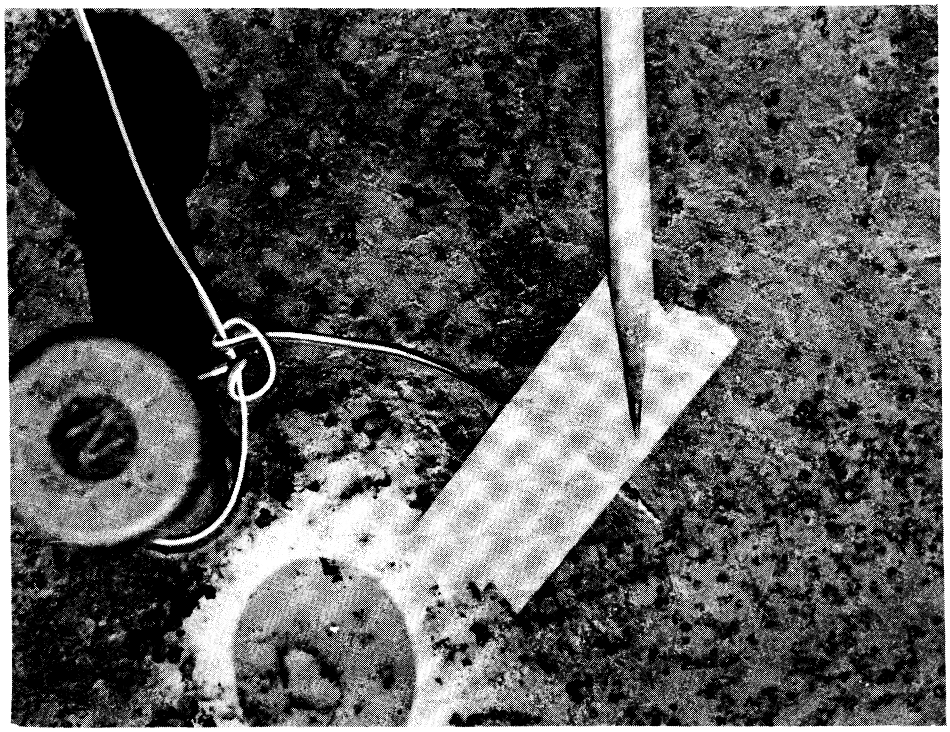


Figure 9. Typical thermocouple gage in place on the top flange of a steel girder.

During the placement of the concrete decks the Honeywell recorder was in continuous operation. A complete cycle of the twenty-four thermocouple locations 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 on the Rte. 607 span to determine the independent effect of solar radiation on girder deflections.

Bearing Deflection Instrumentation

Both of the structures instrumented were designed with neoprene bearing pads located at the expansion ends of the spans. In order to measure and account for the dead load deflections of these pads, dial gages were set as close to the centerline of bearing of each girder as possible. In addition, on the Rte. 607 span, deflection measurements were taken at a fixed end steel bearing point to determine the order of magnitude of the vertical movement at these types of assemblies.

The dial gages were mounted to a heavy steel stand, which in turn was secured to the top of the pier cap by use of 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 point. A typical installation of a dial gage at a neoprene bearing is shown in Figure 10.

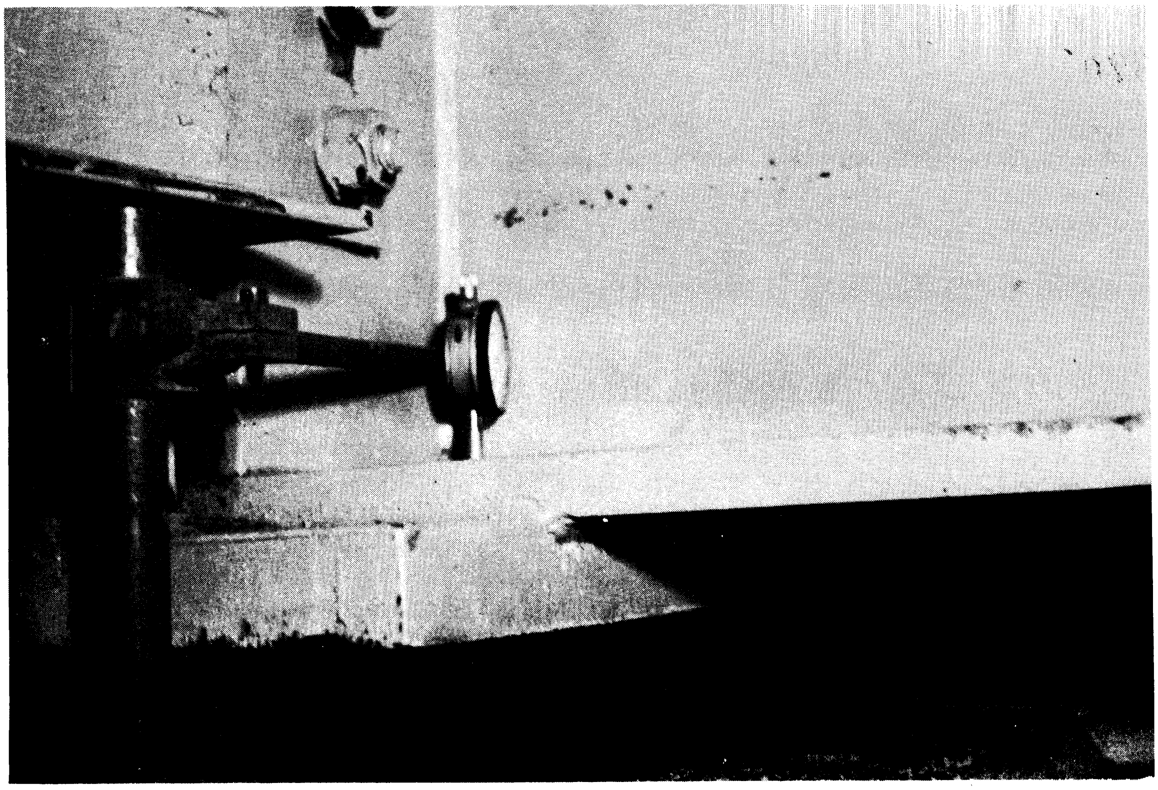


Figure 10. A typical dial gage installation used to measure bearing deflections during placement of the concrete bridge decks.

Strain Gage Instrumentation

As noted in the original Working Plan for this study[4], strain measurements were considered optional since conditions during construction were not expected to be amenable to the successful performance of strain instrumentation. Furthermore, strain data were not required to fulfill the objectives of the study. As shown in the framing diagrams of Figures 2 and 3, however, a limited number of SR-4 wire gages were mounted on the horizontal leg of the angle members of the diaphragms located nearest to midspan. The purpose of these gages was to provide strain data on the behavior of the diaphragms during deck placement.

Only two days between the completion of the deck forming and the placement of the concrete deck were available for installation of all the instrumentation on the Rte. 15 span. Consequently, a lack of time prevented the setting up of the strain indicator at a location out from under the span. Unexpectedly, the contractor used water to wet down the deck forming just prior to the beginning of concrete placement, and some of the water came through the forms and splashed down on the strain indicator and switching unit. On the Rte. 607 span, the strain gages were installed, waterproofed, and the recording instruments moved from under the span. Subsequently, however, several days of heavy rain delayed placement of the deck. During placement of the concrete on each span strain readings were taken but in each case malfunctioning of the system was apparent during strain readouts. The strain data were plotted and reviewed but found to be completely unreliable — due probably to the unfavorable conditions cited above. As a result no further discussion or presentation of these data will be given in this report.

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 on each of the two spans:

1. The time of initial and final set (ASTM C403-68) was run on three representative batches of the concrete. Samples were selected near the beginning, the half-way point, and the conclusions of the deck placement operations.

2. Unit weight determinations (ASTM C138-63) were made on six samples selected at intervals to be generally representative of the concrete placed in each area between the girders.

3. The temperature of the concrete was measured at discharge from the mixer trucks, and the ambient air temperature was recorded continuously during the placement operations.

Field Study Procedures

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

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

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 Virginia Department of Highways' inspectors.

During deck concreting a record of the time and sequence of events was made. The following information was recorded:

1. The time at which all measurements were taken,

2. the position of the screeding machine relative to the concrete loading position when deflection measurements were made, and

3. the time at which the final pass of the screed was made over each girder and the position of the concrete loading prevailing at that time.

At the outset of the study it was expected that the above information would be used for relating the field measurements to any differences between the plan and actual thicknesses of the completed bridge decks. For reasons to be explained in the results of the study an analysis of any differences between the plan and the actual deck thicknesses would be of no value and, in fact, fruitless. Similarly, final elevations on the surface of the completed decks were not required as outlined in the working plan[4] since these data would also have been used to examine differences in deck thicknesses. The actual thicknesses of the decks, however, were determined by depth probe measurements taken through the plastic concrete after the screeding machine had struck the decks to grade. These measurements, which are taken routinely by the project inspector, were made at the quarter points of each span tested.

The placing and screeding of the Rte. 15 deck was recorded by time lapse photography, and on the Rte. 607 span, photographs were taken of the various stages of deck placement at which deflection measurements were made.

Environmental Conditions During Deck Placement

The deck concrete on each of the two spans that were instrumented was placed during warm and sunny weather. The Rte. 15 span was placed on May 28 with the air temperature ranging from 66°F to 89°,and the Route 607 span was placed on July 14 when the air temperature ranged from 64° to 92° during the decking operations.

RESULTS (RTE. 607 SPAN #3)

Although the Rte. 15 span was placed the earliest and thus was instrumented and tested first, the results of the Rte. 607 measurements can more logically be presented first. There are two reasons for this. First, the Rte. 607 bridge has the narrower roadway (28 ft. as opposed to 38 ft. on the Rte. 15 bridge) and one less girder in the superstructure framing. Secondly, and more importantly, there was sufficient time (due to rain and other construction delays) to study the independent effects of solar radiation on the steel girder deflections for several days prior to the placement of the deck concrete on the Rte. 607 bridge.

Solar Radiation and Thermal Differentials

The Rte. 607 span generally runs in a north-south direction. Accordingly, the morning sun generally falls on the east side of the superstructure and gradually passes over to the west side in the evening. During several sunny days in June and July, differential thermal and deflection readings were taken on the girders while only the deck forming was in place. As shown in Figure 11, the deck forming shielded the lower flanges of the interior girders from the sun. The exterior girder on the east side was exposed to the sun in the morning and the exterior girder on the west side was exposed to the afternoon sun. In addition, the vertical forming on each side of the span tended to shield the top of the east girder in the morning and the top of the west girder later in the afternoon. A transverse section of the steel framing of this span is shown directly above Figures 12(a) and 12(b), which show, respectively, the average differential temperatures recorded between the top and bottom flanges of the girders and the resulting upward midspan deflections of the girders.

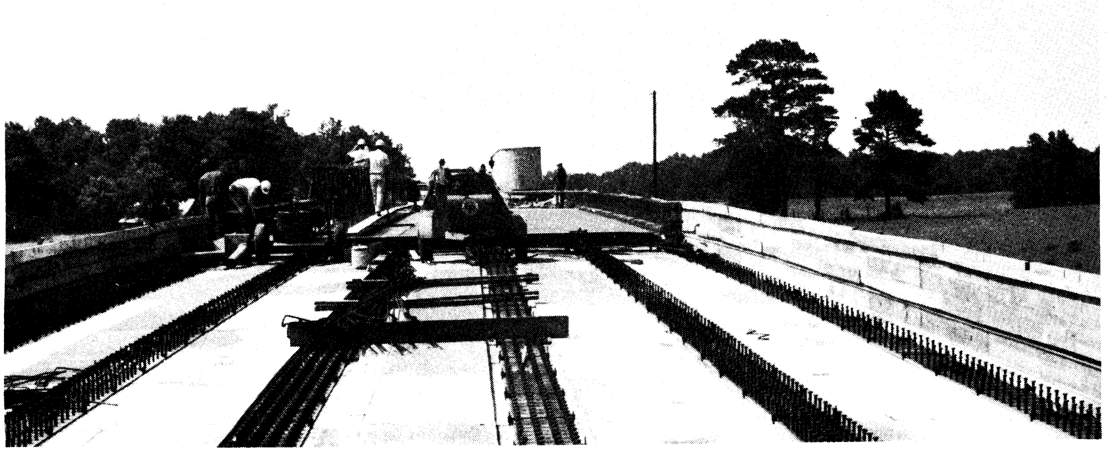


Figure 11. The 96'-2" length span (Rte. 607) with only the deck forming in place.

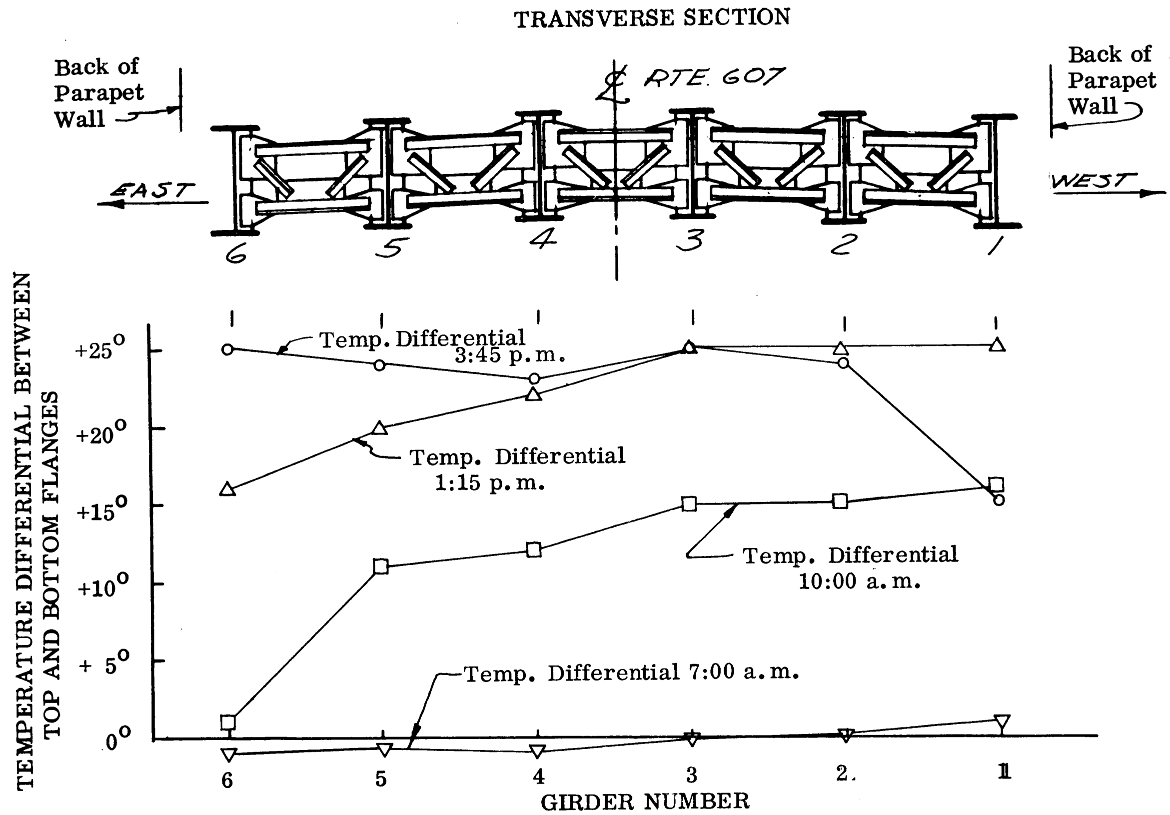


Figure 12(a). Differential temperatures between the top and bottom flanges of each girder at the times shown.

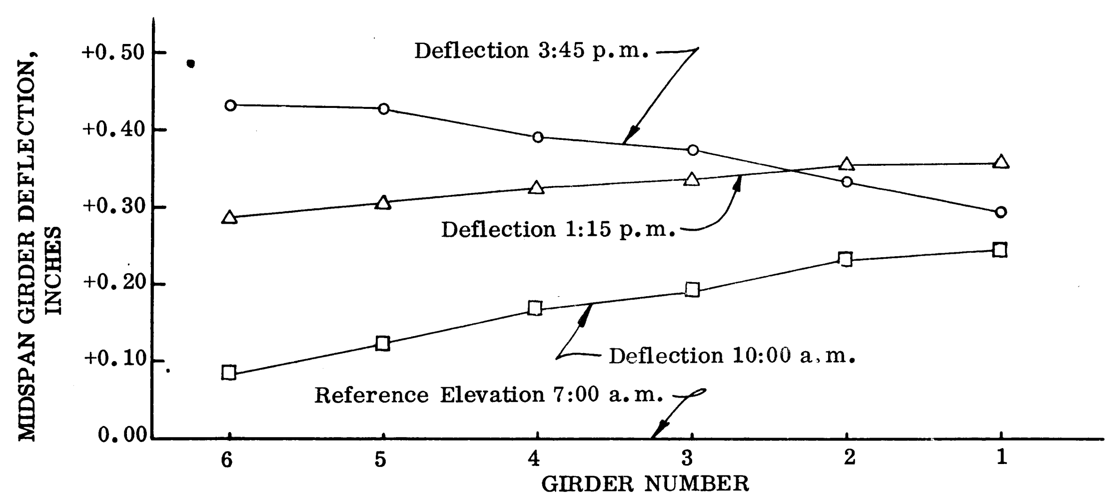


Figure 12(b). Upward midspan girder deflections due to the differential temperatures shown in Figure 12(a).

At 7:00 a.m. on July 1, the temperature differential between the top and bottom flanges was virtually neutral (Figure 12(a)) and the corresponding girder elevations at midspan were recorded at that time and used as a reference (Figure 12(b)). Comparisons of the temperature differentials at 10:00 a.m., 1:15 p.m., and 3:45 p.m. with the corresponding midspan deflections generally show that the upward deflection of the steel girders increases with increasing temperature differentials. In addition, transfer of the thermal loading between girders via the diaphragm connections is indicated by the smooth transverse deflection pattern. Upward midspan deflections of 0.43 inch were recorded on girders number 5 and 6 at 3:45 p.m. All the girders reached an upward deflection level of approximately 3/8 inch above the reference level during the early afternoon. As will be discussed in more detail in the next section, thermal deflections of this order of magnitude could have a significant bearing on bridge deck thicknesses.

It can also be noted that the differential temperatures varied transversely across the span width due to its orientation to the angle of the sun. Thus, the midspan girder elevations not only varied significantly in magnitude but the slope of the transverse pattern of upward deflections reversed during the course of the day. This transverse "warping" effect, due to the sun moving toward the west, is illustrated in Figure 13 where the midspan girder elevations for two different days are referenced to the elevations existing at 12:00 noon. Observing the upward movement of girder #6 and the downward movement of girder #1, a difference in the relative elevation of these two girders on the order of 1/4 inch occurred between 12:00 noon and 3:45 p.m. on June 30. It can also be noted from Figure 13 that during days of similar climatic conditions, and at nearly the same time of day, the differential temperatures and thus the upward deflections of the girders are quite similar. For the two comparative days illustrated, the maximum difference in elevation was 1/32 inch at girder #6. It might be concluded from these data that for two different days having similar weather, temperature, and solar conditions, the elevations of the girders will be close to identical at approximately the same time of day. It is apparent, however, that exact girder elevations cannot be established when any degree of solar radiation is present.

Figure 14 shows the temperatures on the upper and lower flanges and at mid-depth of the exterior girders. Temperatures on the order of 120°F were measured on the top flanges, but at mid-depth of the web the temperatures were about the same as those on the lower flanges. It is likely that some of the heat from the top flanges is conducted down into the web as shown, but becomes insignificant before reaching the mid-depth level.

While the maximum temperature differentials recorded between the upper and lower flanges in this study were on the order of 25°F, it is possible to experience differentials of a higher order of magnitude. In a study of the thermal behavior of a box section type bridge in the London area, for example, Capps[5] has reported extreme temperature differentials on the order of 50°F.

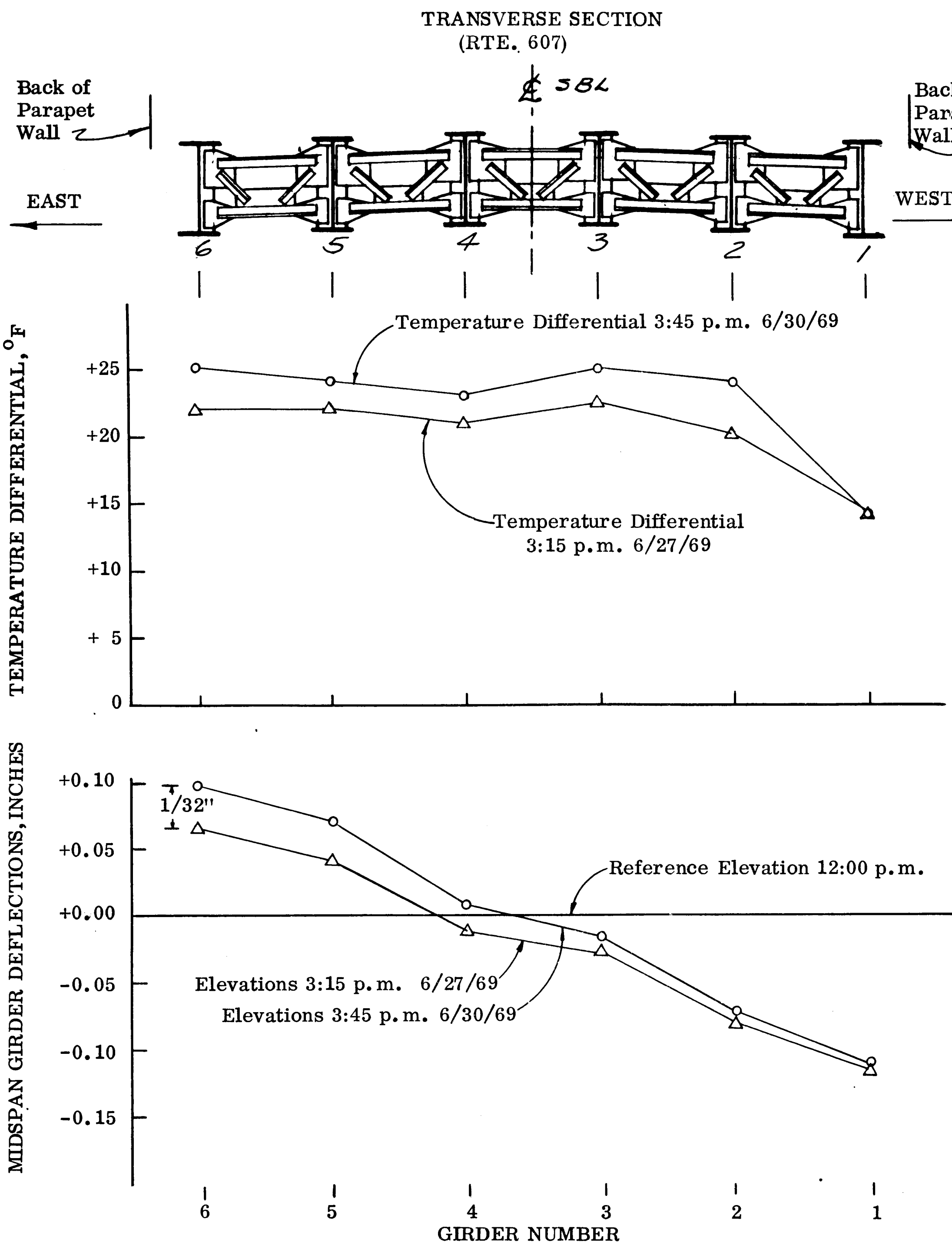


Figure 13. Midspan girder deflections and temperature differentials at similar times of day but on different days. The girder deflections are with reference to the elevation existing at noon of each day.

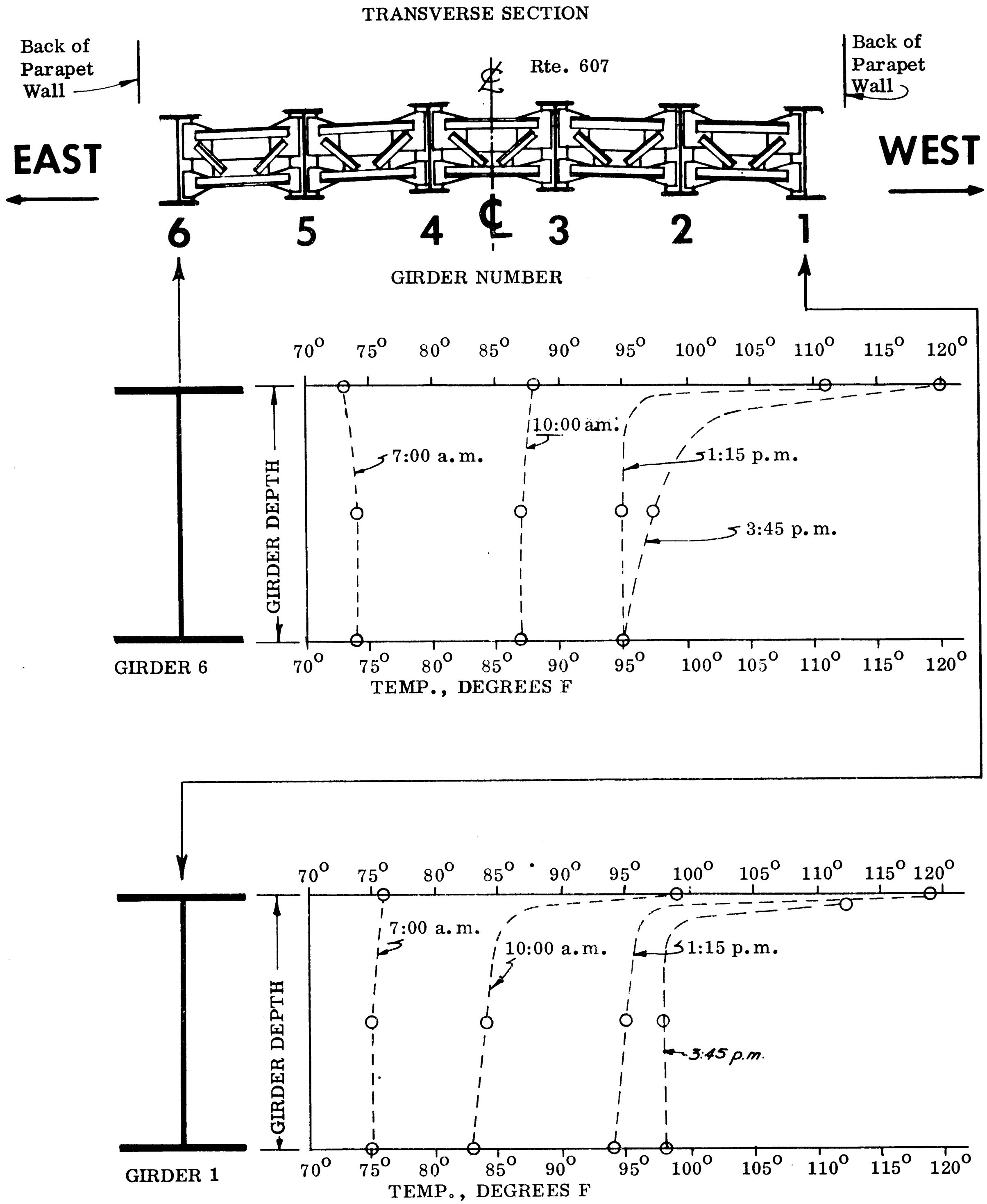


Figure 14. Thermal gradients on the exterior girders of the Rte. 607 span. (Temperatures measured during June and July.)

Solar Radiation During Deck Placement

It is important to note that solar radiation can also cause changes in elevations of bridge girders during day time deck placement operations. When girder elevation changes are considered relative to the initial elevations measured for calculation of forming elevations, significant deck thickness can be lost if the span is longitudinally screeded. This fact can best be illustrated in Figure 15. If no temperature differential exists between the top and bottom flanges of a simply supported bridge girder, it is in a thermally neutral position (Figure 15A). Under conditions of solar radiation, differential temperatures will generate an expansive force, F, in the upper flange which is resisted by an opposing force in the lower flange to create a bending moment, M, as shown. The resulting effect is an upward deflection of the girder (Figure 15B). If the deck forms are established to grades complying with the neutral position of the girder, but the concrete deck is screeded to grade under differential thermal conditions, the thickness of the deck will be decreased by an amount Δ (Figure 15C).

In order to minimize the effects of solar radiation: (1) deck forming elevations should be established when the thermal conditions on the girders will approximate those anticipated at the time of concrete placement; and/or (2) the deck forms should be adjusted vertically at a time when the thermal condition of the girders will approximate the condition expected to prevail at the time the concrete is screeded to grade. The latter precaution is important since the in-place forming will shield the lower portion of the girders from solar radiation and thus cause high differential temperatures on hot, sunny days. Differential thermal effects can be virtually negated, of course, by very early or very late deck placement operations, i.e., placement when solar radiation will not be a problem.

General Thermal Differentials During Deck Placement

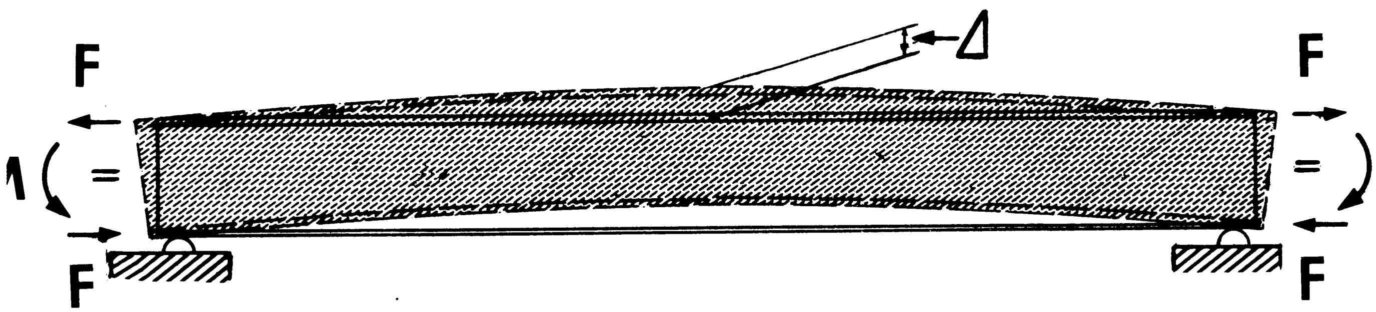
The temperatures on the upper and lower flanges of each girder, the temperature of the plastic concrete, and the ambient air temperature on the day of deck placement are shown in Figure 16 for each girder on the Rte. 607 span. The times of initial and final sets of the concrete were determined from test data (see Appendix Figure A-1). Some observations from Figure 16 indicate the following facts:

1. In early morning (6:30 a.m.) the lower flanges of the steel girders were warmer than the upper flanges.

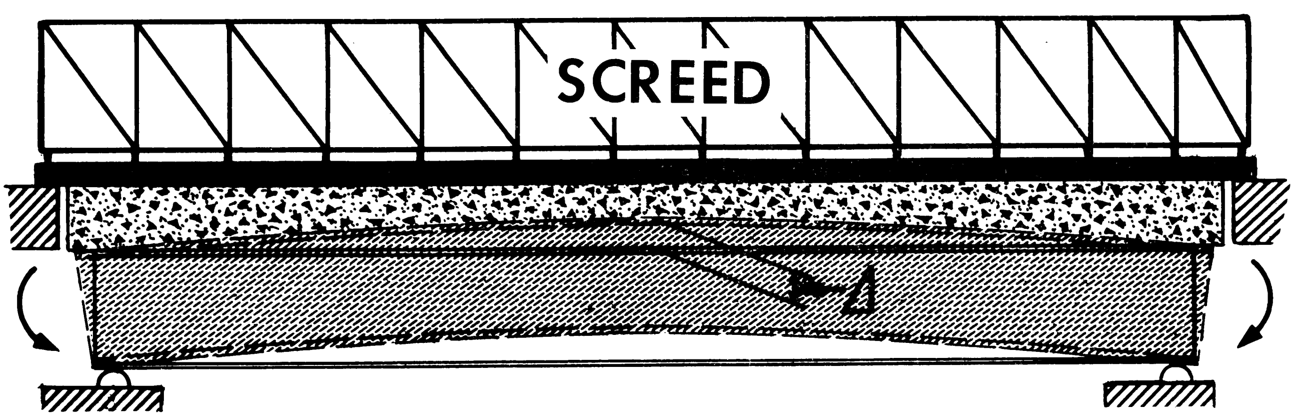
2. The temperature on the top flange of all the girders increased rapidly due to solar radiation until the concrete was placed over the top flanges.

3. The rate of temperature increase on the bottom flanges was no greater than the rate of increase in the ambient air temperature. In addition, the temperature of the lower flanges remained lower than the air temperature in the afternoon.

A. Neutral Position

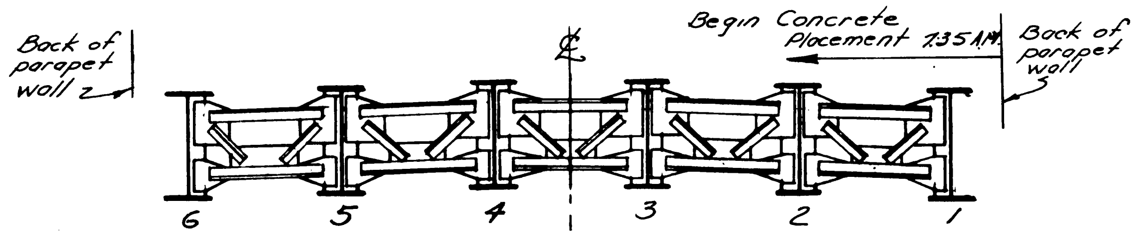


B. Top Flange Hot



C. Top Flange Hot, Decking

Figure 15. An illustration of the possible effects of solar radiation on bridge deck thickness when longitudinal screeding is used.



TRANSVERSE SECTION (RTE. 607)

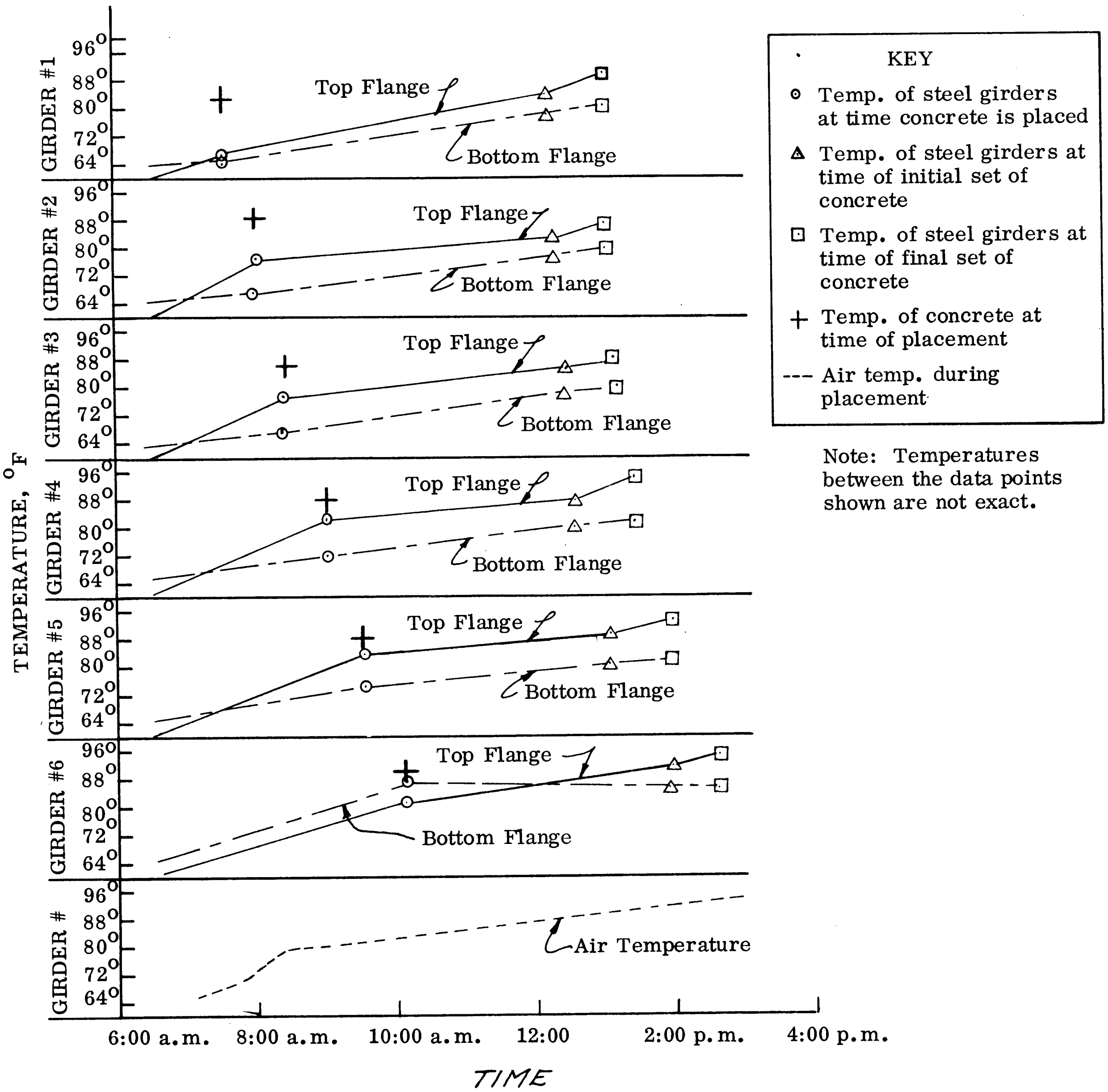


Figure 16. Temperature conditions on the steel girders during deck placement.

4. After the concrete was placed over each girder, the general rate of rise in temperature on the top flanges decreased, and in most cases was usually less than the rate of increase in the ambient air temperature.

5. The temperature on the top flanges of each girder at the time of initial set was not significantly higher than, and in most cases was nearly the same as, the initial temperature of the plastic concrete. This suggests that the heat of hydration of the concrete has very little if any direct effect on the temperature of the upper flanges of the girders prior to initial set of the concrete.

6. Between the initial set and the final set of the concrete, the rate of temperature rise on the top flanges increased, which indicates that the effects of heat of hydration are more significant after initial set has occurred. It should be noted, however, that concrete deck finishing would have to be completed prior to final set and that the concrete would be very difficult to work subsequent to initial set (500 psi penetration resistance).

In summary the above observations suggest that the heat of hydration of the concrete would have an insignificant effect on girder deflections during deck placement and finishing. The top flanges of the steel girders, however, are at a higher temperature than the lower flanges during deck finishing operations, due to solar radiation, a general rise in ambient air temperature, and the initial higher temperature of the fresh concrete. Since the Rte. 15 concrete placement began at a later hour and lasted later in the day, the data paralleling that shown above, and presented later, is more profound and substantiates these general conclusions.

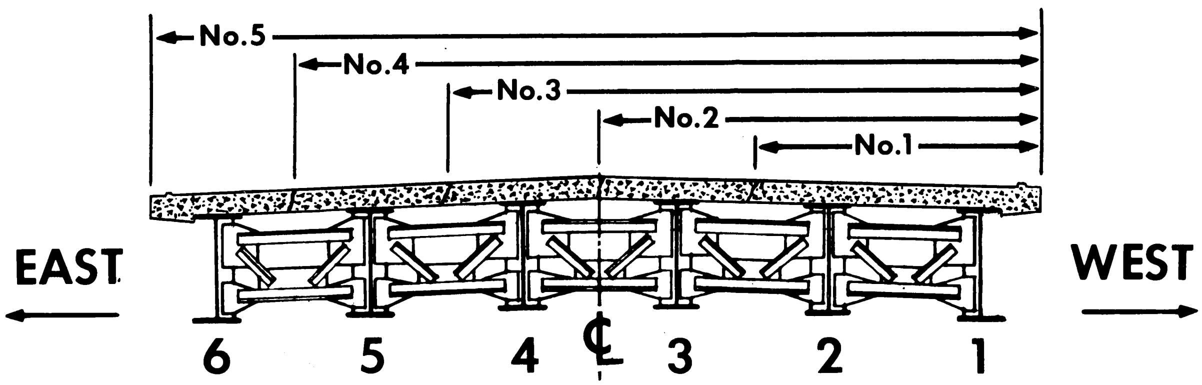
Bearing Pad Deflections

Figure 17 shows the results of measurements made at the neoprene expansion bearing pads during placement of the concrete on the Rte. 607 span. The top portion of the figure shows the approximate deck placement loading intervals at which the corresponding pad deflections were measured. (Note that the same loading intervals were used for the girder deflection measurements, which are discussed later.)

As would be expected, the neoprene pads compress under the direct loading of the concrete deck. It can be observed that each of the deck loading intervals has an effect on the pads under the adjacent girders. The effects of loading interval #1, for example, are transmitted beyond the second bearing pad, and result in a slight compressive effect on pads three and four and a slight uplift at pads five and six. This general transverse deflection pattern continued with each loading interval until all pads had deflected at least 0.013 inch under full loading. The greatest pad deflection, 0.035 inch, occurred under the first girder. In general the first pads loaded compressed the most; the last several pads loaded compressed to a lesser degree, with the pad under girder #3 compressing the least. The average pad compression was on the order of 0.02 inch.

TRANSVERSE SECTION (RTE. 607, SPAN #3)

A. Deck placement intervals for deflection measurements.



B. Neoprene bearing deflections for each deck placement interval shown above.

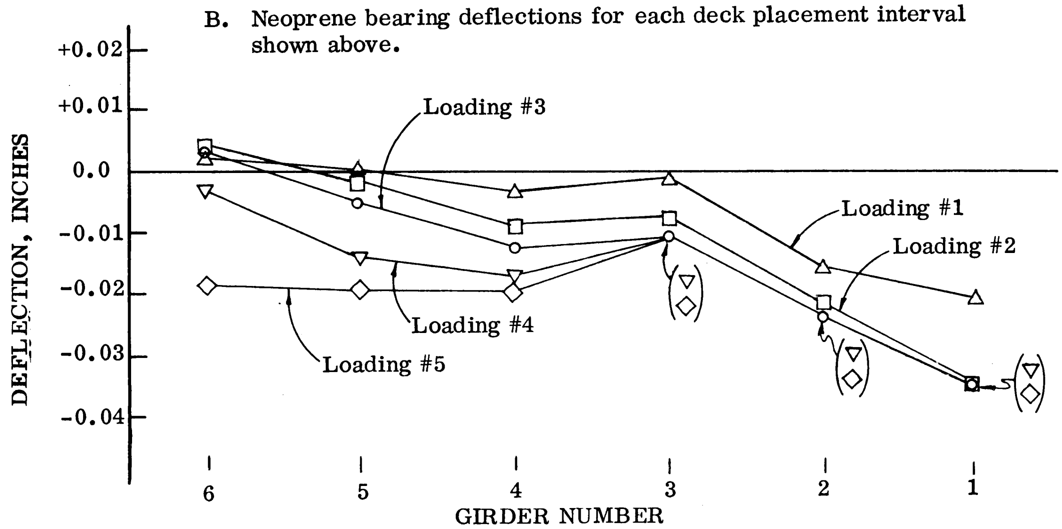


Figure 17. Neoprene bearing deflections of the expansion end of each girder for the deck placement loading shown.

It can be concluded from these data that considerable transfer of load from the loaded to the unloaded girders takes place. The average pad compression of 0.02 inch would have an additive but insignificant effect on the deck thickness and does not warrant consideration in design or field calculations.

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

Plan Girder Deflections

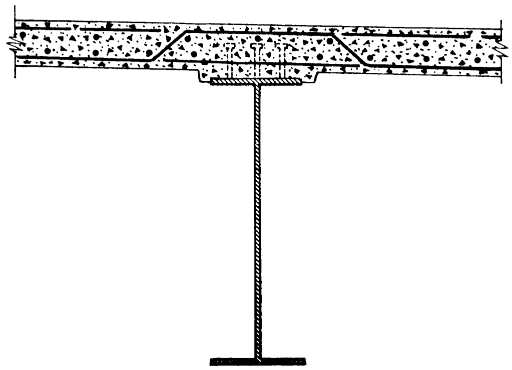
Deflections given on the bridge plans for simply supported spans are usually calculated by assuming each girder to be free to deflect as an individual unit. Thus, plan dead load deflections are calculated by assuming that each interior girder, for example, will carry an equal portion of the concrete deck as shown in Figure 18. Using this method, the midspan deflections for the Rte. 607 interior girders were found to be equal to 1.0 inch.* The plans, however, give the value as 1-5/8 inches, or 0.63 inch too high. Had the plan value been used, the forms would have been set too high; and with the longitudinal screeding the deck thickness would have been shy by 0.63 inch (assuming the correct conventionally calculated deflection represents the true situation, and that all thermal factors are neglected). However, a shy deck thickness had resulted earlier on another bridge deck, and the contractor had made adjustments in the forming elevations to avert a similar occurrence on the Rte. 607 span. As shown in Appendix Figure A-2, actual depth probe data indicate that the completed deck is very close to the required $7\frac{1}{2}$ inch thickness.

Due to the deck forming adjustments described and the findings regarding the effects of solar radiation, it would be fruitless to attempt to calculate the actual thickness of the deck. At any rate, such an analysis would be of little value to this study.

Plan deflection errors on the high side, as other studies[2] have shown, are a major cause of shy deck thickness when longitudinal screeding is used and would have caused a deficient deck on the study span if adjustments had not been made.

---

*For this calculation 150 lb./ft.^{3} was used as the weight of the reinforced concrete and the midspan moment of inertia value was used. Moment of inertia values are given in Appendix Figure A-3.



$$\text{MIDSPAN DEF'L.} = \frac{5wl^4}{384\ E\ I}$$

w = uniform loading per unit length of span

ℓ = length of span

E = modulus of elasticity of the girder

I = moment of inertia of the girder

Figure 18. Conventional calculation of girder deflections for simple spans due to deck placement.

Field and Theoretical Girder Deflections

As shown previously, differential temperatures can have a significant bearing on the elevations of steel bridge girders. Thus, the field deflection measurements taken during the deck placement operation automatically incorporate the existing thermal conditions on the girders. Accordingly, the actual midspan deflections of the girders for each deck loading increment are shown in Figure 19. Additional measurements taken approximately three hours after completion of the deck finishing (2:55 p.m. data, Figure 19) clearly show that continued heating of the top girder flanges results in an upward deflection of the whole span. Viewed as a proportion of the total dead load deflection at 11:40 a.m., this average 18% "thermal uplift" demonstrates the remarkable forces generated by thermal differentials.

The general transverse pattern of the midspan girder deflections for all loading intervals shows that the structural steel framing is acting as a unit due to the diaphragm connections between the girders. Note that girder #6 is uplifted by the first and second loading increments. Thus, the basic questions are:

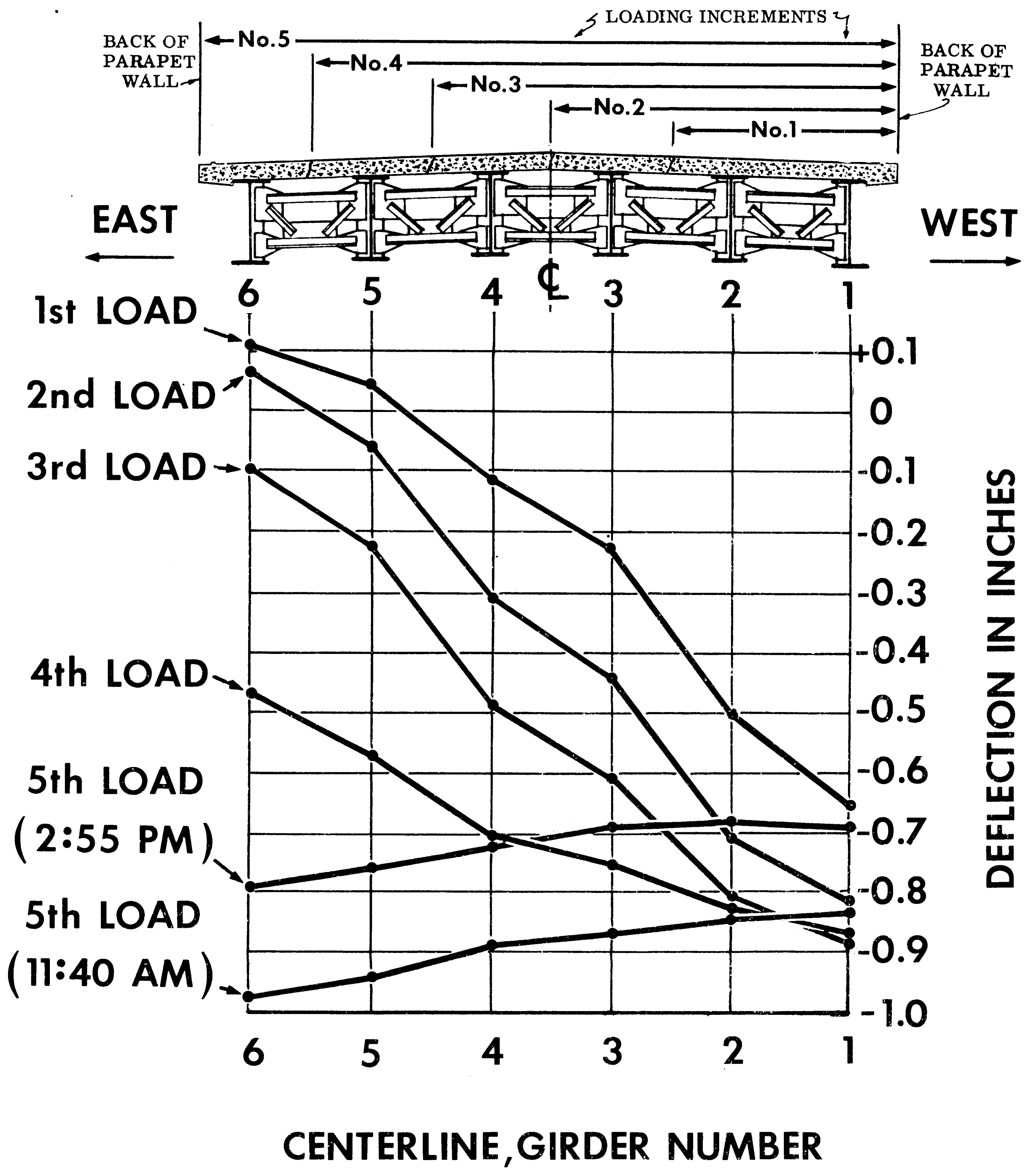


Figure 19. Midspan girder deflections due to concrete deck placement. (Rte. 607, Span #3.)

a. How do the actual deflections for each loading increment compare to those conventionally calculated?

b. How do the actual deflections compare with those computed by assuming rigid or semirigid connections between all the girders?

To study the latter question, a theoretical analysis of deflections of semi-rigidly connected girders was used. This analysis, which was developed by Lisle(3), utilizes a modified stiffness matrix and has been programmed in Extended Algol 60 for solution on a Burroughs B5500 computer. (The computer program listing is given in Appendix C.) The program can be used for computing deflections of bridge girder systems with any degree of end fixity at the diaphragm connections. Thus, an end fixity factor of one would represent a rigid connection and zero would represent a pinned connection. Any value between zero and one would represent a semi-rigid connection.

In using the program the structural framing of a span is considered as a series of segments -- each segment usually terminating at a connection. Referring back to Figure 2, the Rte. 607 framing would consist of four segments for each girder plus 25 individual diaphragms. The moments of inertia of each segment of the girders were calculated by conventional procedures and are given in Appendix Figure A-3. The differential thermal conditions existing on the girders at each loading increment can be accounted for in the program by applying moments at the girder ends (as shown earlier in Figure 15C) and at changes in the sectional dimensions of the girders. An estimate of the thermal moments can be calculated from:

$$M = EA\,\alpha\,\Delta Td$$

where

M = thermal moment

E = modulus of elasticity of steel

α = thermal coefficient of expansion of steel

ΔT = differential temperature between upper and lower flanges

A = area of the heated flange

d = distance from centroid of A to the neutral axis.

Since independent solar radiation deflection data were available for the Rte. 607 span, the moments as determined from the above formula were checked in the computer program and found to be slightly low. A multiple of 1.2M checked closely and was used for the thermal input moments in the analysis.

For the loading on the frame the actual unit weights of the concrete (see Appendix Table A-1) were used. The total weight of the concrete was determined from these values and the volumes placed in each loading increment. The total weight was proportioned to each girder according to the plan dimensions, i.e., each exterior girder received 15.6% and each interior girder 17.2% of the load. Since much of the concrete on the span had not been screeded to grade at each loading increment, this was considered to be a reasonable procedure. Where the loading increments varied slightly from the ideal, this was noted in the field and taken into account in the analysis. Thus, the programmed loading corresponds as nearly as is practical to that existing during the field deflection measurements.

In presenting the results of the analysis, it is convenient to show the field and the computed deflections for each loading increment on the same figure. In this manner comparisons can readily be made. Only the midspan deflections are presented since these are of the most basic importance. (Quarter-span field deflection measurements were found to be on the order of 70 to 75% of the midspan values, as was expected.) A transverse section of the steel framing, which is given at the top of each figure, shows the actual loading intensities and thermal differentials existing at the time the field measurements were made. The results are shown in Figures 20 through 26.

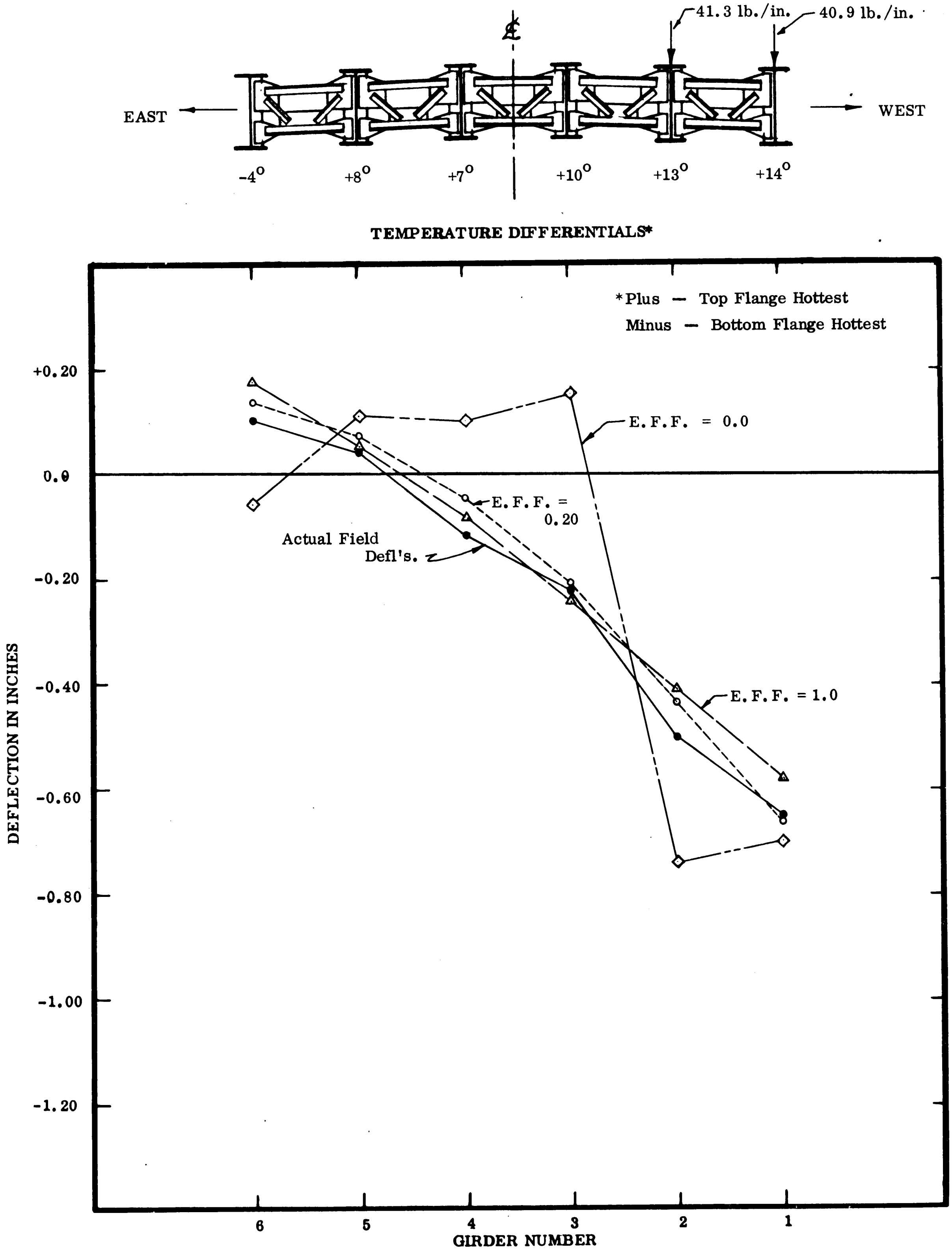


Figure 20. A comparison of field deflections with computed deflections using several degrees of diaphragm connection rigidity. (Span #3, Rte. 607).

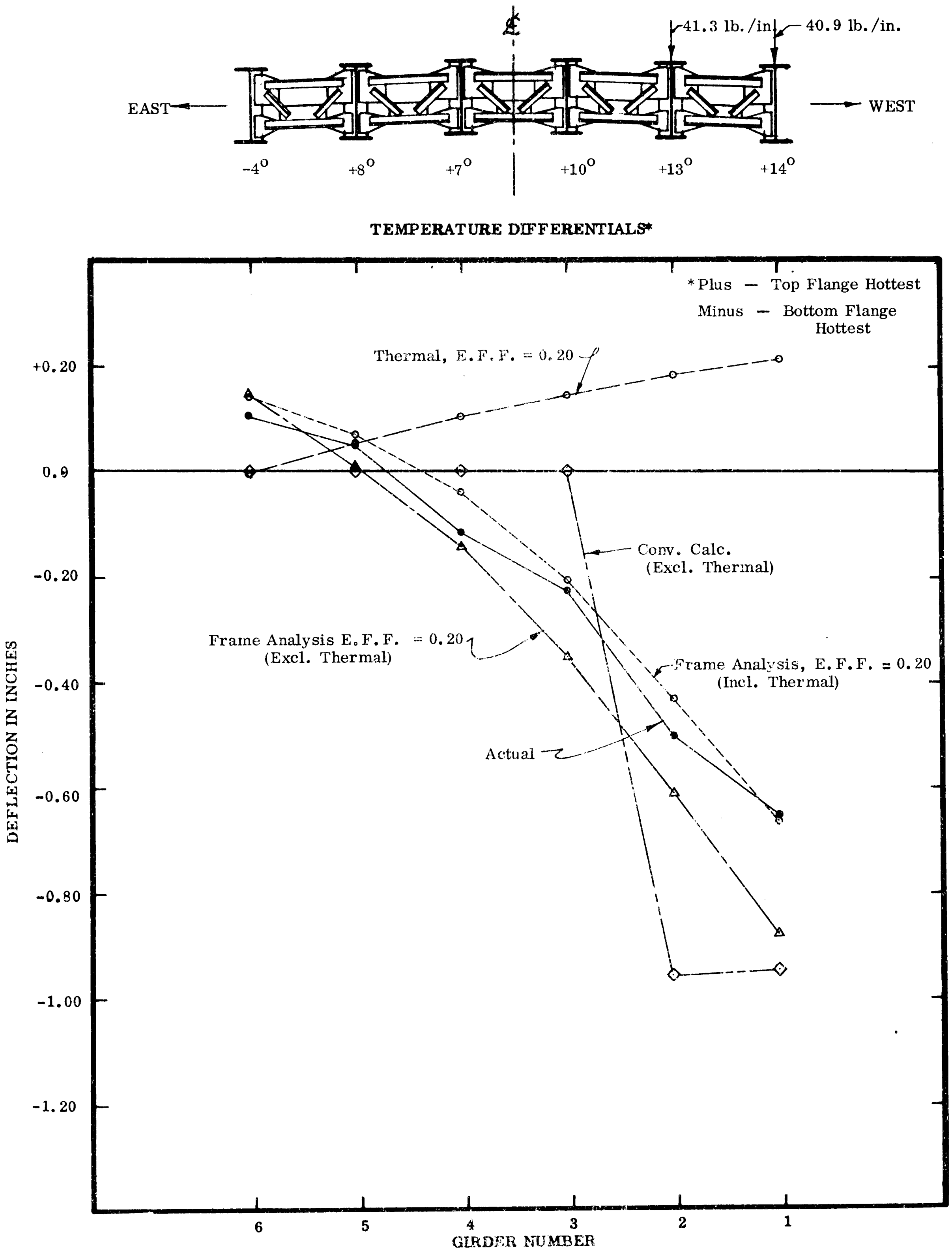


Figure 21. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 1, 8:15 a.m., Span #3, Rte. 607.)

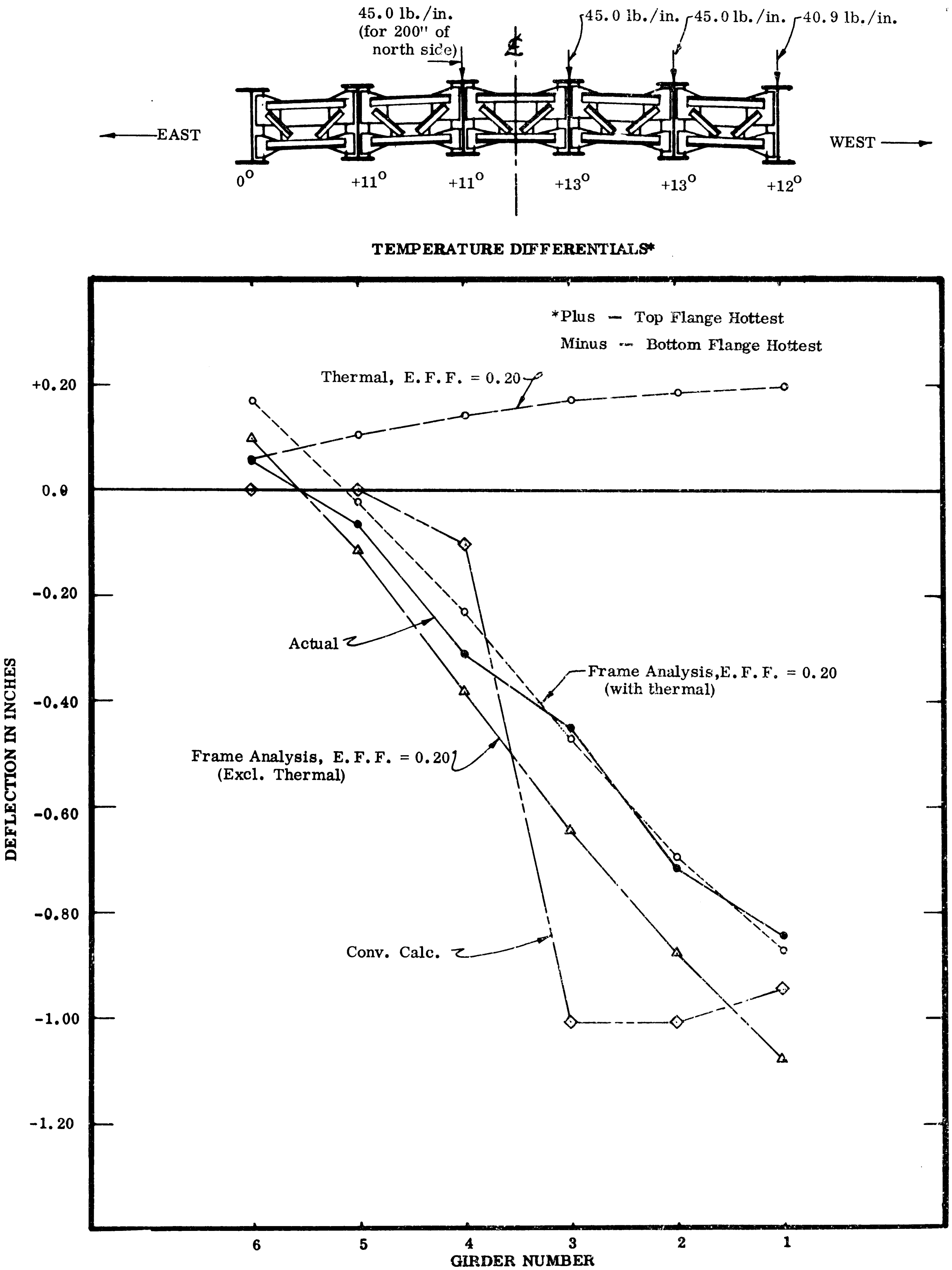


Figure 22. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 2, 8:50 a.m., Span #3, Rte. 607.)

2392

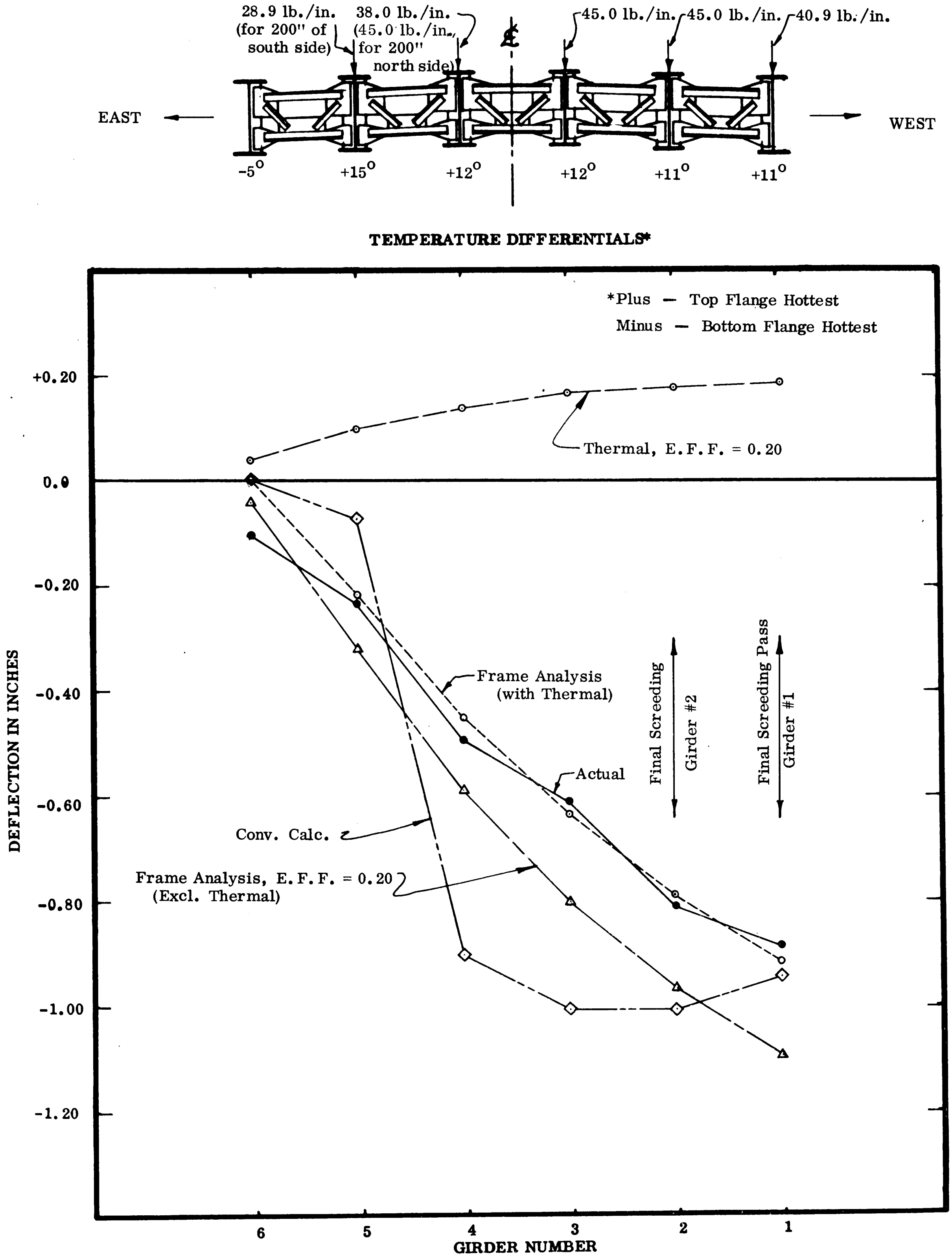


Figure 23. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 3, 9:15 a.m., Span #3, Rte. 607.)

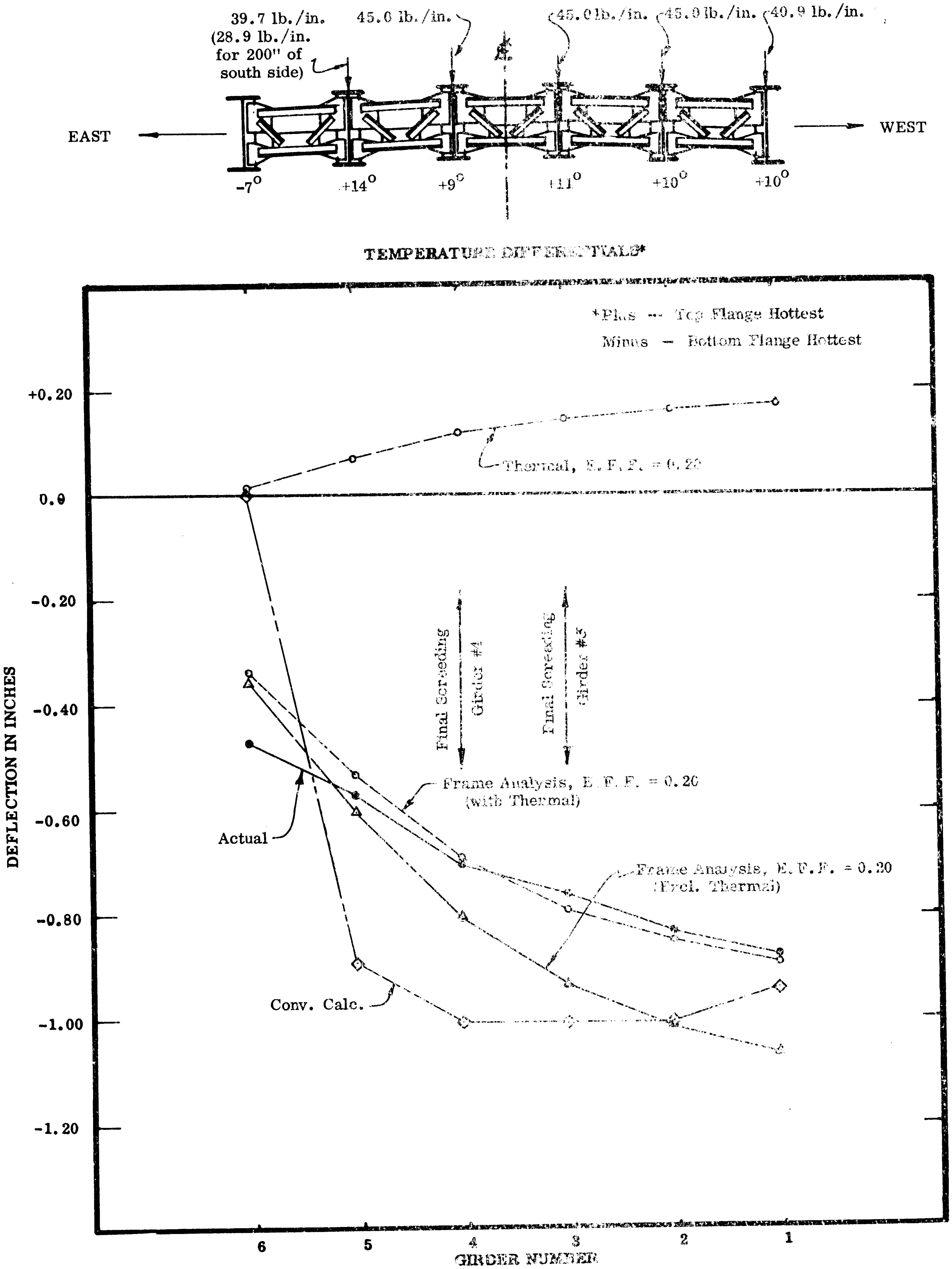


Figure 24. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 4, 9:45 a.m., Span #3, Rte. 607.)

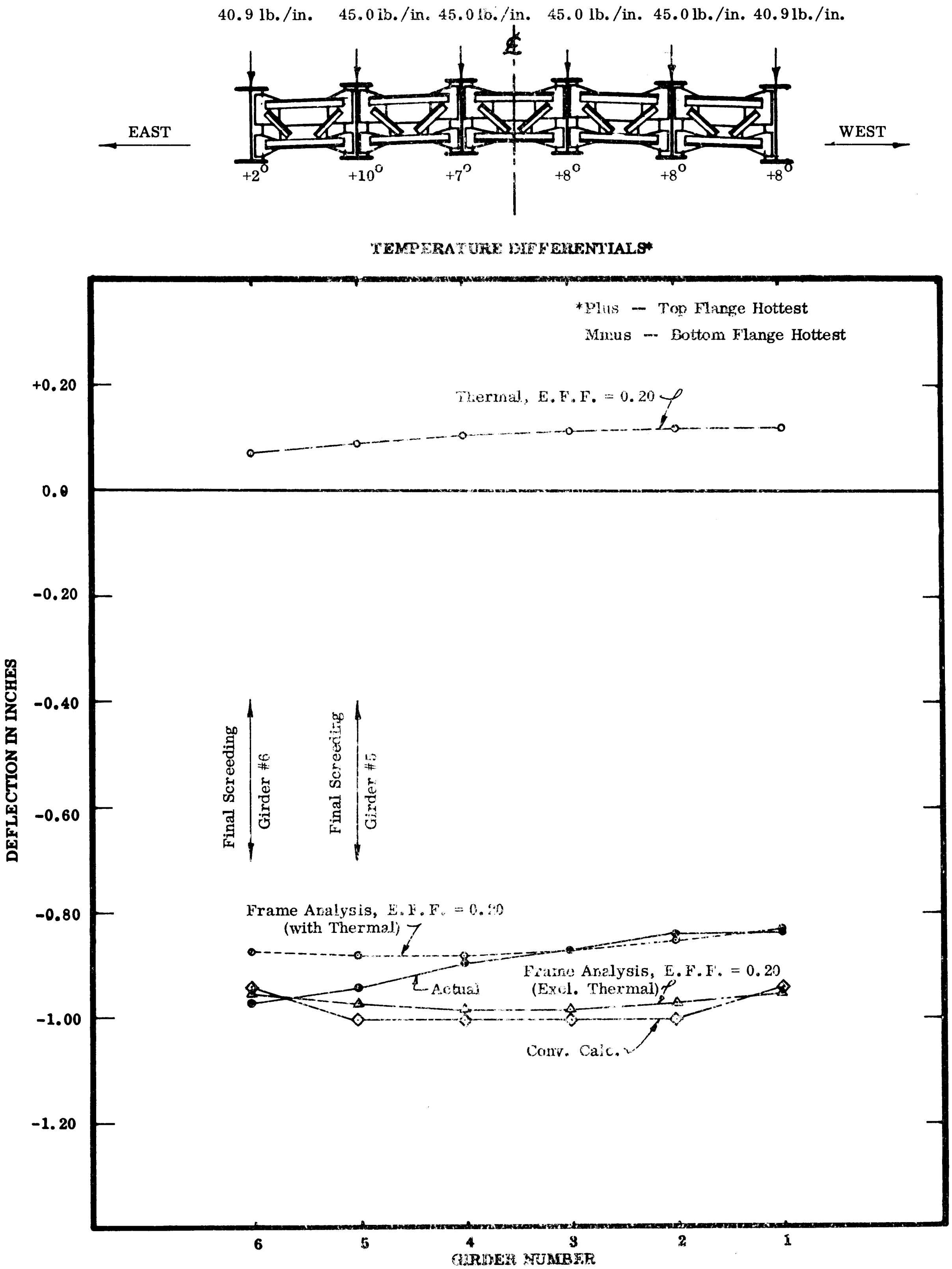


Figure 25. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 5, 11:45 a.m., Span #3, Rte. 607.)

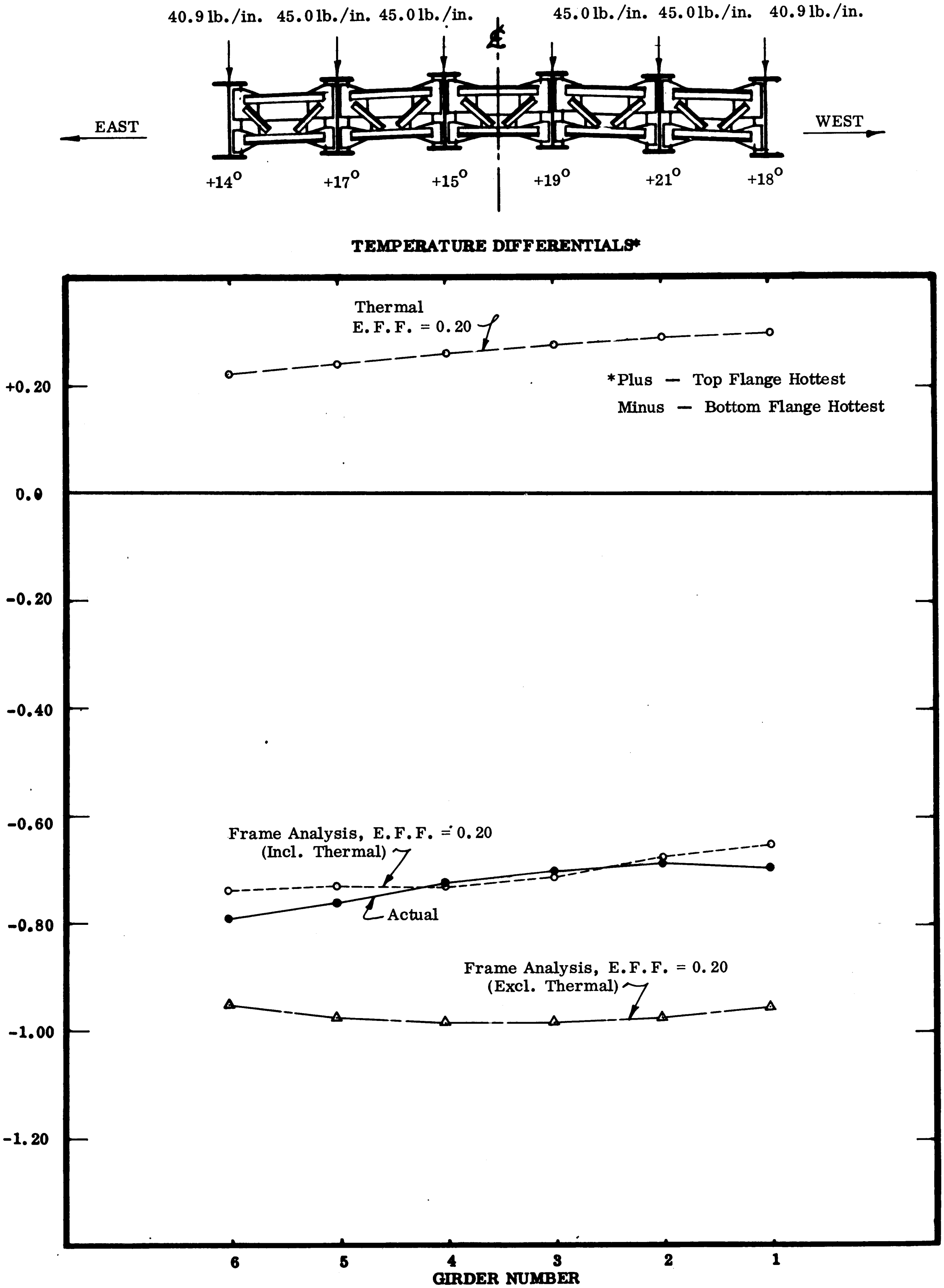


Figure 26. A comparison of actual field midspan deflections with computed deflections. (Loading interval No. 5, 2:50 p.m., Span #3, Rte. 607.)

Diaphragm Connection Rigidity

A thorough theoretical analysis of a wide range of end fixity factors (E.F.F.) was made for each loading increment designated in Figure 19. In general, very little difference was found between the deflections obtained by assuming E.F.F.'s ranging from 0.10 to 1.0. An E.F.F. of 0.20, however, appeared to match the actual deflection patterns the closest. An example is given in Figure 20 showing several E.F.F. conditions compared with the field data for the first loading increment. Note that the thermal differentials are included in the curves shown. Similar results were found for all other loading increments. Thus it is concluded that the diaphragm connections on the Rte. 607 span are semirigid in nature and have an end fixity factor of approximately 0.20. An E.F.F. of 0.20 was also found to closely check deflection deficiencies occurring on the Brambleton Avenue bridge as analyzed by Lisle.[3] For practical purposes this could be assumed to be a rigid connection since an insignificant error would be involved.

Actual and Theoretical Deflection Comparisons

Figures 21-26 compare the actual and the computed deflections for each loading increment. The theoretical deflections given are based on an E.F.F. of 0.20, and the computed values are shown both excluding and including the superimposed differential thermal conditions on the girders. The conventional deflections are based on a unit weight of 150 lb./ft.^{3} for concrete, which is commonly used for calculating plan deflections. The following observations are made from the six data figures:

1. In general the deflections by the frame analysis including thermal conditions are in excellent agreement with the actual field deflections.

2. The frame analysis excluding thermal conditions shows that the deflections would be considerably greater if differential thermal conditions did not exist. Noting girder #1 during the first loading increment (Figure 21), for example, the downward deflection would be 0.22 inch, or 33%, greater by neglecting the thermal factor. Viewed conversely, the actual deflections were less due to hotter temperatures on the top flanges of the girders.

3. Both the actual field and the frame analysis results were markedly different from the conventionally calculated deflections.

4. Noting girder #2 during the third loading increment (Figure 23), it can be observed that the concrete placement was 2 to 2½ bays (the distance between adjacent girders) ahead of the final pass of the screeding machine over girder #2. Excluding thermal effects, the frame analysis deflection value is very nearly equal to the conventional deflection. Observing the same point on Figure 24, which would constitute a 3 to 3½ bay lead, these two deflection values are almost identical. Thus, the greater the lag of the final screeding pass, the greater the chances of the actual deflections

being the same as the conventional plan deflections, and the less the chances of the deck thickness being shy. For the span in question, a final screeding pass lag of 3 bays behind concrete placement appears to be ideal.

5. General observations from the data presented indicate that the final screeding pass over the concrete averages a 2 to 3 bay lag behind concrete placement. Quite often only a 2 bay lag was noted over some areas.

6. In one instance the final pass of the screed was made with only a one-bay lag. Referring to Figure 24, the difference between the field and conventional deflections at the final pass over girder #4 is 0.30 inch. The deck could possibly have been shy by 0.30 inch in that vicinity had the forms been set utilizing conventional deflections and the initial girder elevations (taken for bolster calculations) measured when the girders were in a thermally neutral condition.

7. Considering a hypothetical situation which would assume the same conditions listed in the former situation, a 0.40 inch shy deck could occur between girders 3 and 4 during the third loading increment (Figure 23) if only a one-bay lag were used in screeding the deck to grade.

8. The thermal uplift of the span, which occurred in a three hour period subsequent to completion of the deck finishing, was verified by the frame analysis results, which check very closely with the field deflections at that time (Figure 26).

RESULTS (S.B.L., RTE. 15, SPAN #4)

The results of the field measurements and data analysis on the Rte. 15 span were much the same as those presented for the Rte. 607 bridge span. Since the latter results have been discussed in considerable detail, treatment of the Rte. 15 data will be brief and confined mostly to observations of general differences between them and the Rte. 607 data.

All the procedures described previously were used in the analysis of the Rte. 15 data.

Thermal Differentials During Deck Placement

Thermal data taken during placement of the Rte. 15 span are presented in Figure 27. The determinations of the times of initial and final sets of the concrete are included in Appendix Figure B-1. Note in Figure 27 that the air temperature data are applicable to the whole bridge span. The following observations are presented from Figure 27:

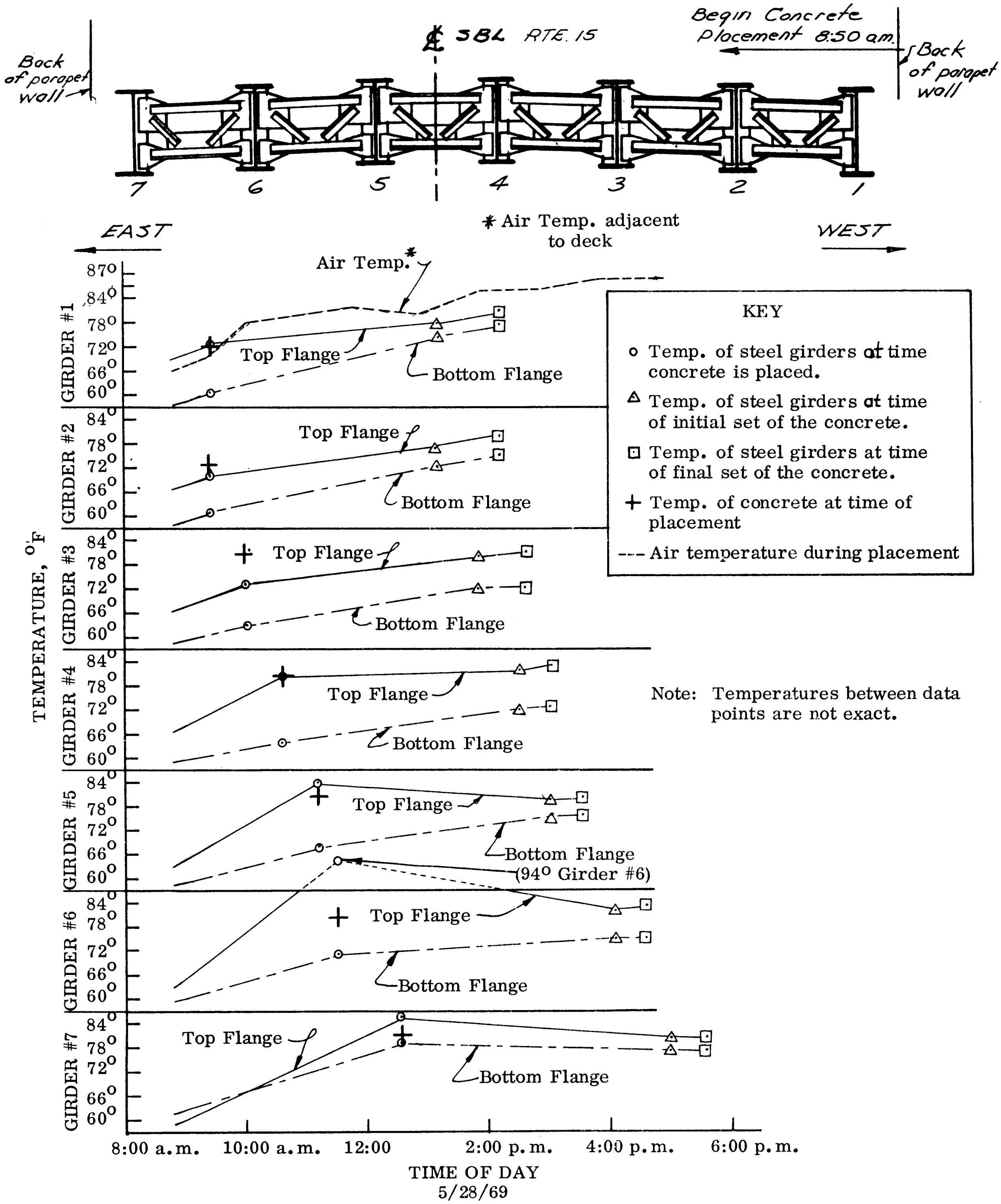


Figure 27. Temperature conditions during bridge deck placement (Rte. 15, Span #4).

1. The temperature on the top flanges of the girders increased rapidly due to solar radiation until the concrete was placed over them.

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

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

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

Bearing Pad Deflections

The neoprene bearing pad deflections shown in Figure 28 are, in general, like those for the Rte. 607 span. Due possibly to the wider span width, there appears to have been more uplift at the center three pads during the first loading interval. This uplift, in a practical sense, was very insignificant (0.0015 inch). The maximum pad deflection was approximately 0.03 inch and the average was on the order of 0.02 inch.

Plan Girder Deflections

Using the conventional method, the midspan deflection due to the dead load of the concrete deck was calculated to be 1.04 inch* for the interior girders, and 0.88 inch for the exterior girders. The dead load midspan deflections for both the interior and exterior girders were given as $1\frac{1}{2}$ inches on the bridge plans. Thus, as for the Rte. 607 span, had the contractor used the plan deflection values a midspan shy deck thickness of 1/2 inch over the interior girders and 5/8 inch over the exterior girders would have resulted (assuming no other factors would have influenced the outcome of the results). Due to ample allowances in the forming elevations, however, the minimum 8 inch final deck thickness was obtained (see depth probe results, Appendix Figure B-2).

---

*Calculations based on the midspan moment of inertia value and 150 lb./ft.^{3} unit weight of reinforced concrete.

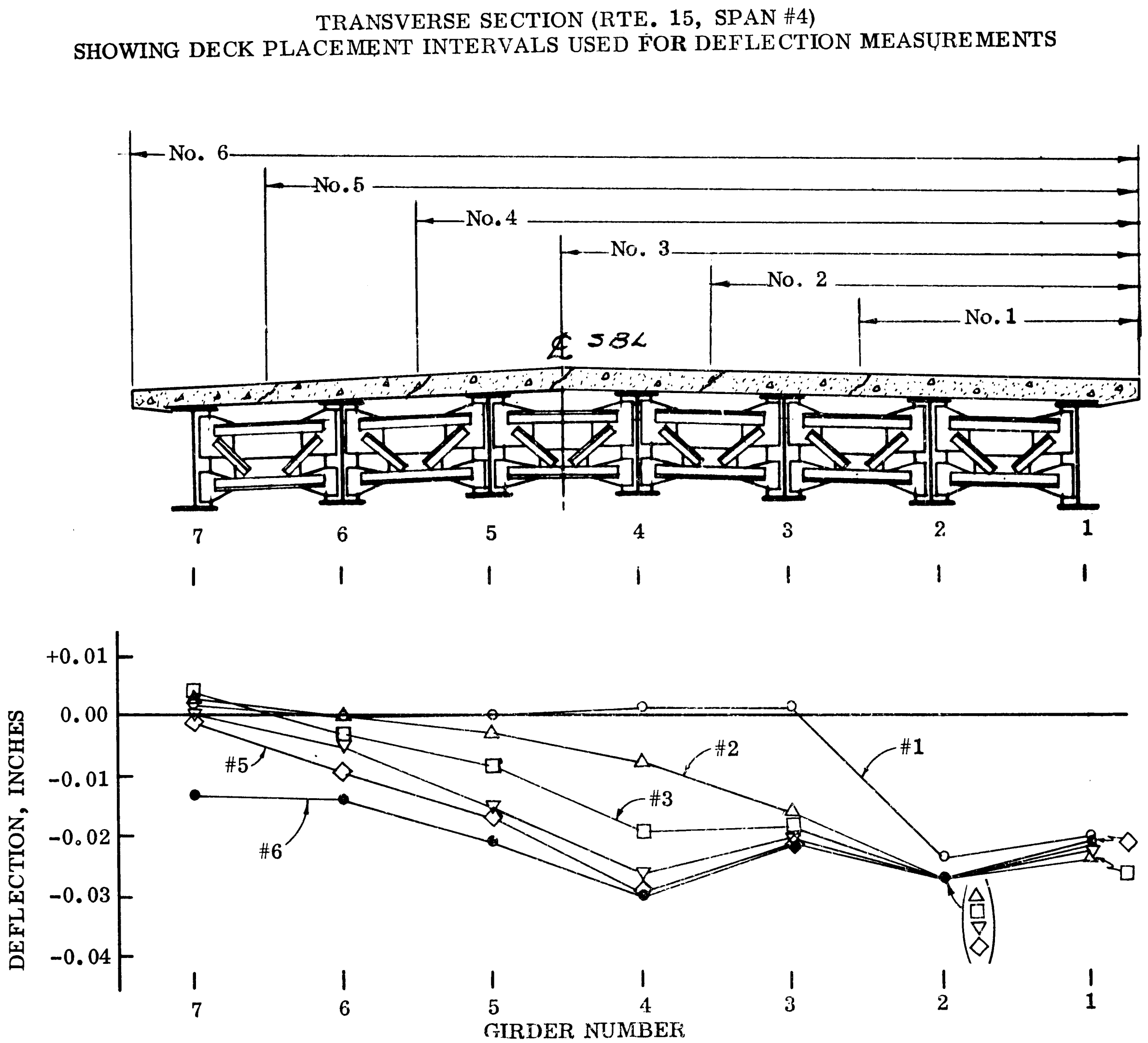


Figure 28. Neoprene bearing deflections at the expansion end of each girder for the deck placement loading intervals shown.

Field and Theoretical Girder Deflections

The theoretical analysis approach to the Rte. 15 data was the same as that for the Rte. 607 span, with one exception. Whereas independent solar radiation data were available to verify the thermal moment inputs on the Rte. 607 span, these data were not available for the Rte. 15 study. Consequently, the thermal moment inputs for the computer program were calculated directly from the formula presented earlier. The unit weights of concrete used in the analysis are given in Appendix Table B-1.

Diaphragm Connection Rigidity

The theoretical deflection analysis was performed on connection rigidity (E.F.F.) factors of zero, 0.05, 0.10, 0.20, 0.50, and 1.0. As was the case with the former span, an E.F.F. of 0.20 appeared to best match the field deflection results; but there is little difference between E.F.F.'s of 0.20 and 1.0.

Actual and Theoretical Deflection Comparisons

Figures 29-34 show the actual and the calculated deflections of the girders for each loading interval. The theoretical frame analysis deflections are based on an E.F.F. of 0.20 — excluding and including the average thermal differentials existing for each particular loading interval.

In general the data verify the observations made on the previous structure. These data clearly indicate that temperature differentials on the general order of 10^{o} to $15^{o}F$ will exist during summer daytime deck placement operations. Top girder flanges exposed to solar radiation for a number of hours before the concrete is placed can become very hot as evidenced by girder #7, Figure 31. The resulting upward deflections of the girders at midspan are on the general order of 10 to 20% of the plan deflections as calculated by conventional procedures.

At a point where roughly three-quarters of the deck had been placed, there was a tendency on both study spans for the final pass of the longitudinal screeding machine to follow too closely behind concrete placement (Figure 23). While this may have been due to the delays for study measurements, any type of delay during normal operations could cause the same result. Again, a three-bay lag behind concrete placement appears to be ideal. Finally, it should be noted again that the conventionally calculated deflections shown in Figures 30-34 are based on the midspan moments of inertia and exclude thermal differentials. This would explain much of the difference between the final conventional and the frame analysis deflections which take all changes of moments of inertia into account.

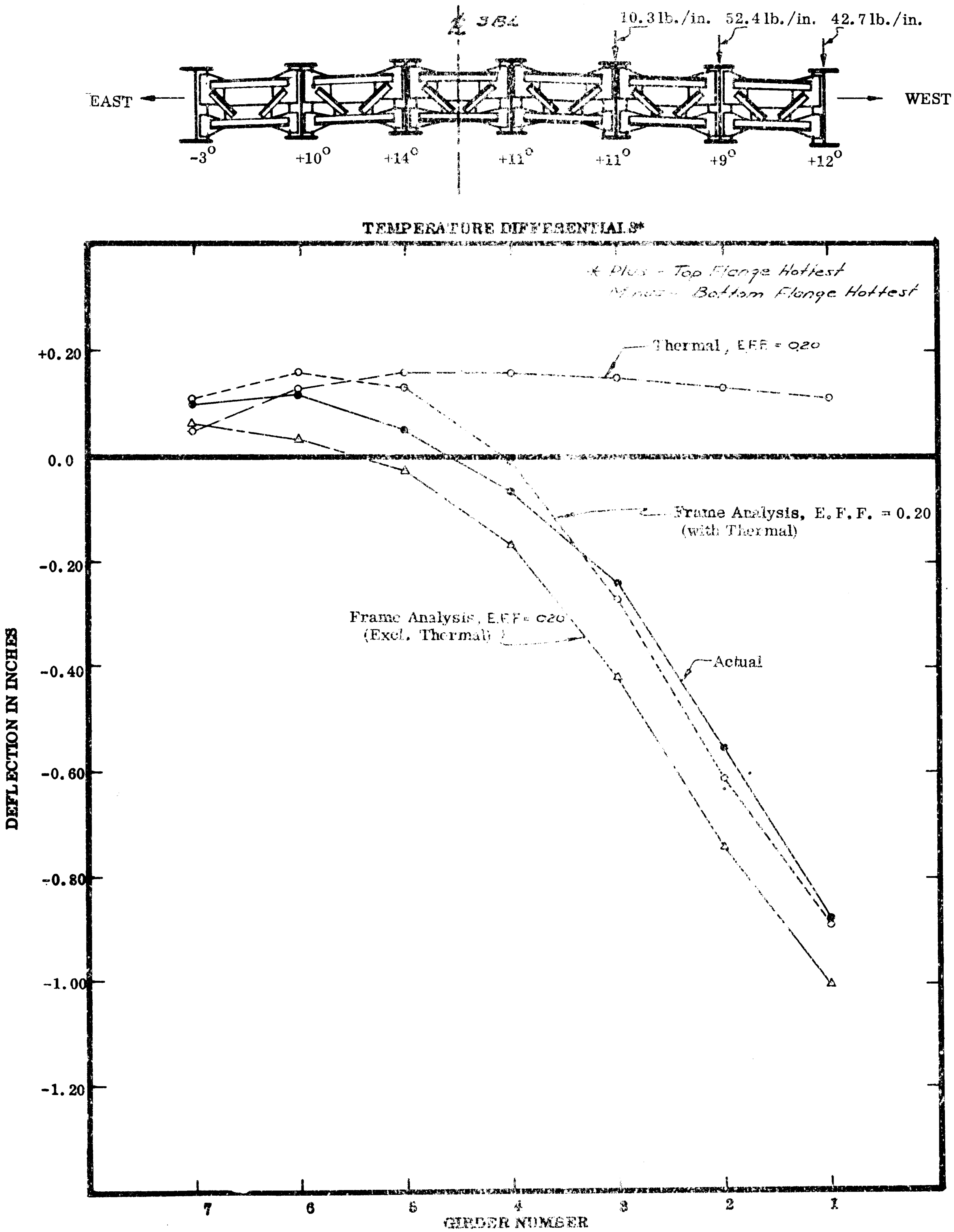


Figure 29. A comparison of actual field midspan deflections with computed deflections. Loading interval No. 1, Span #4, SBL Rte. 15.

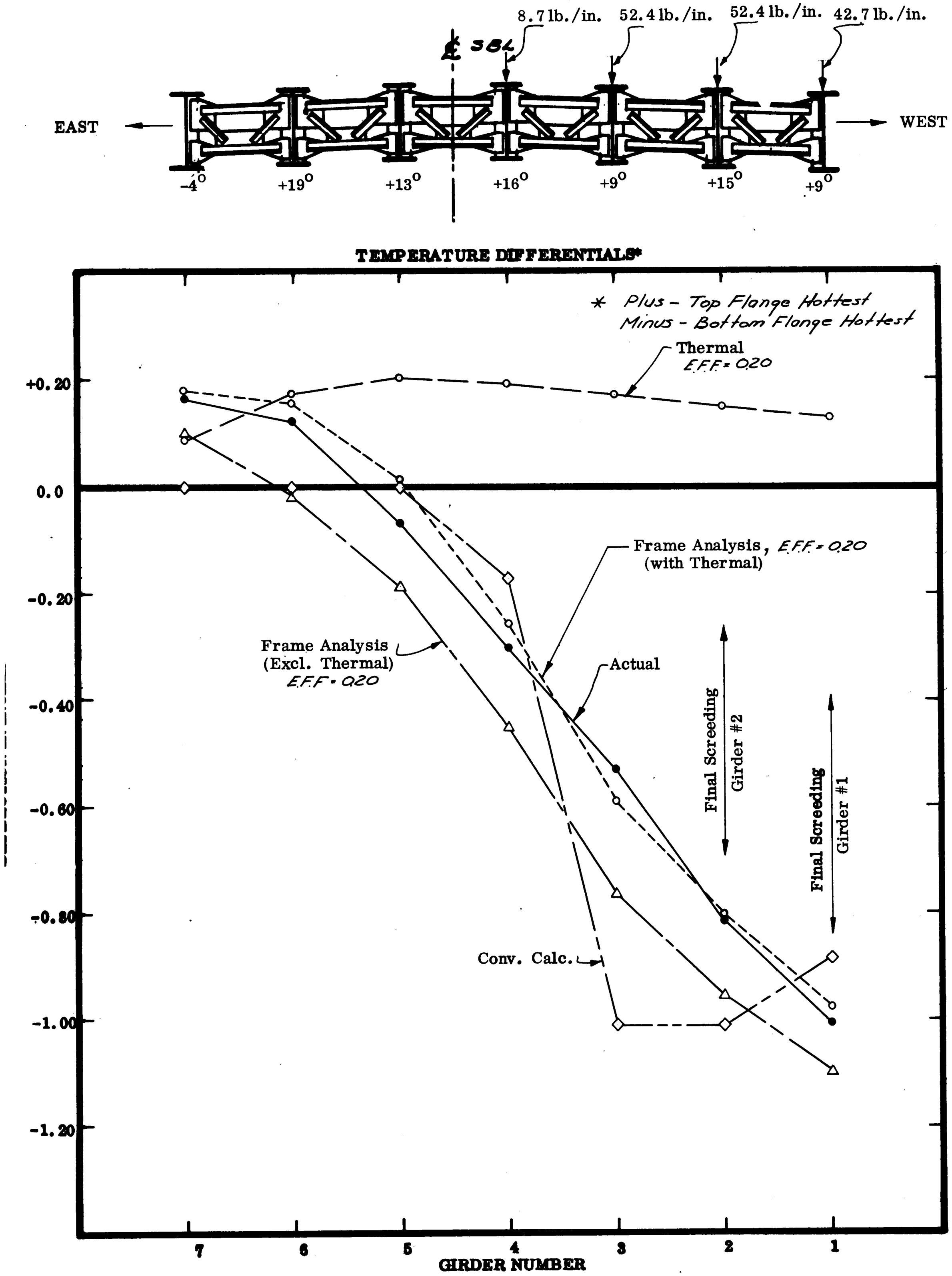


Figure 30. A comparison of actual field midspan deflections with computed deflections. Loading interval No. 2, Span #4, SBL Rte. 15.

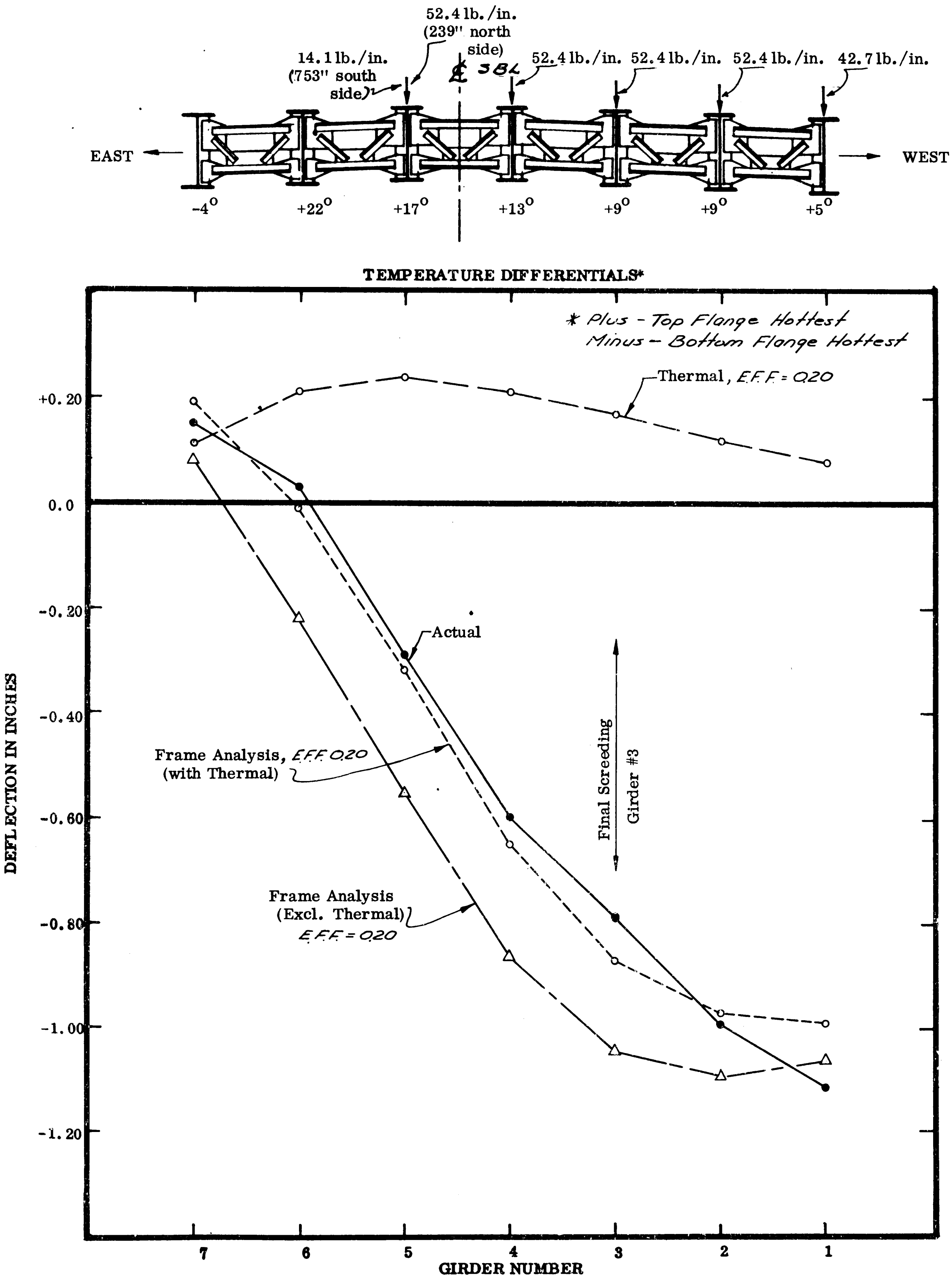


Figure 31. A comparison of actual field midspan deflections with computed deflections. Loading interval No. 3, Span #4, SBL Rte. 15.

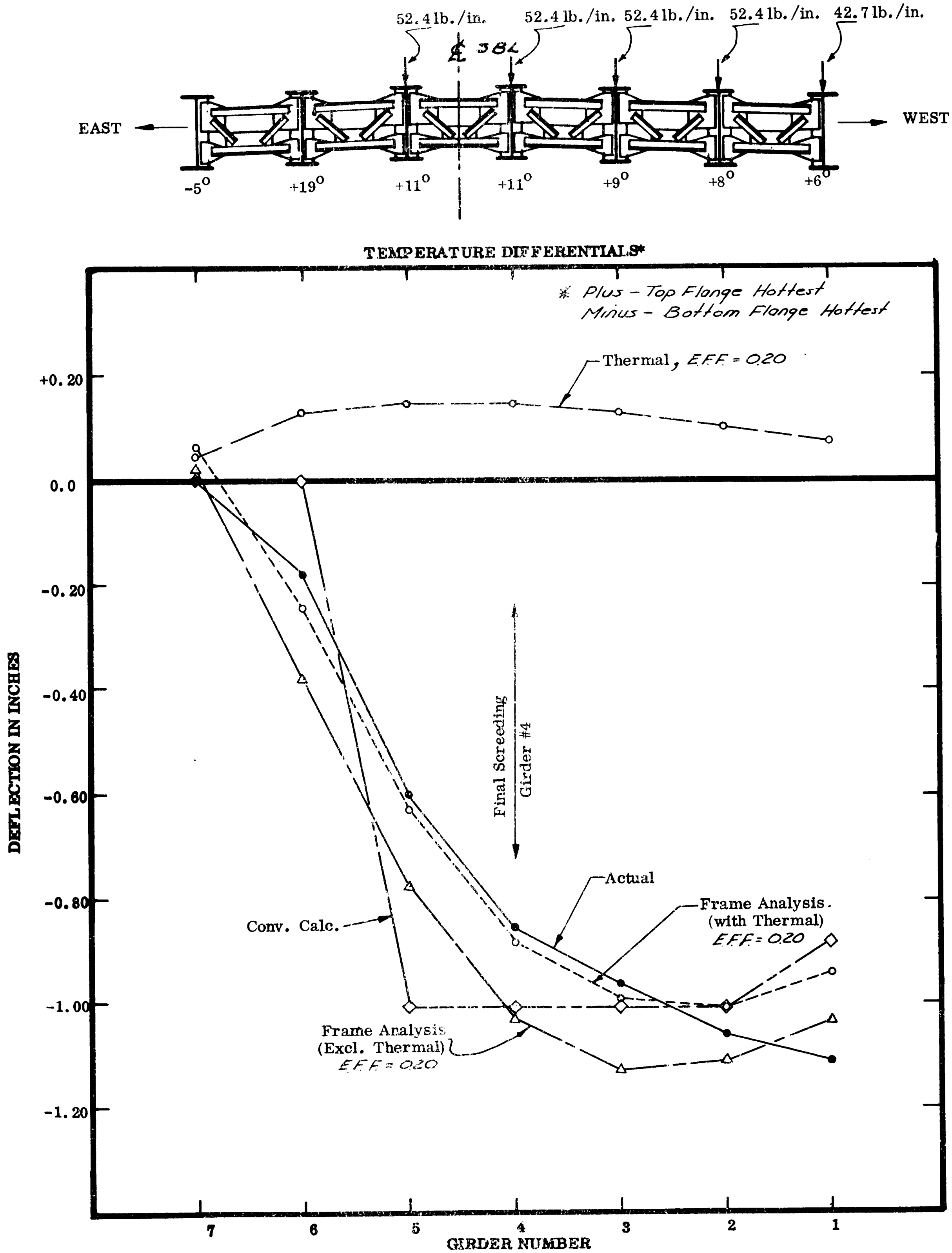


Figure 32. A comparison of actual field midspan deflections with computed deflections. Loading interval #4, Span #4, SBL Rte. 15.

2406

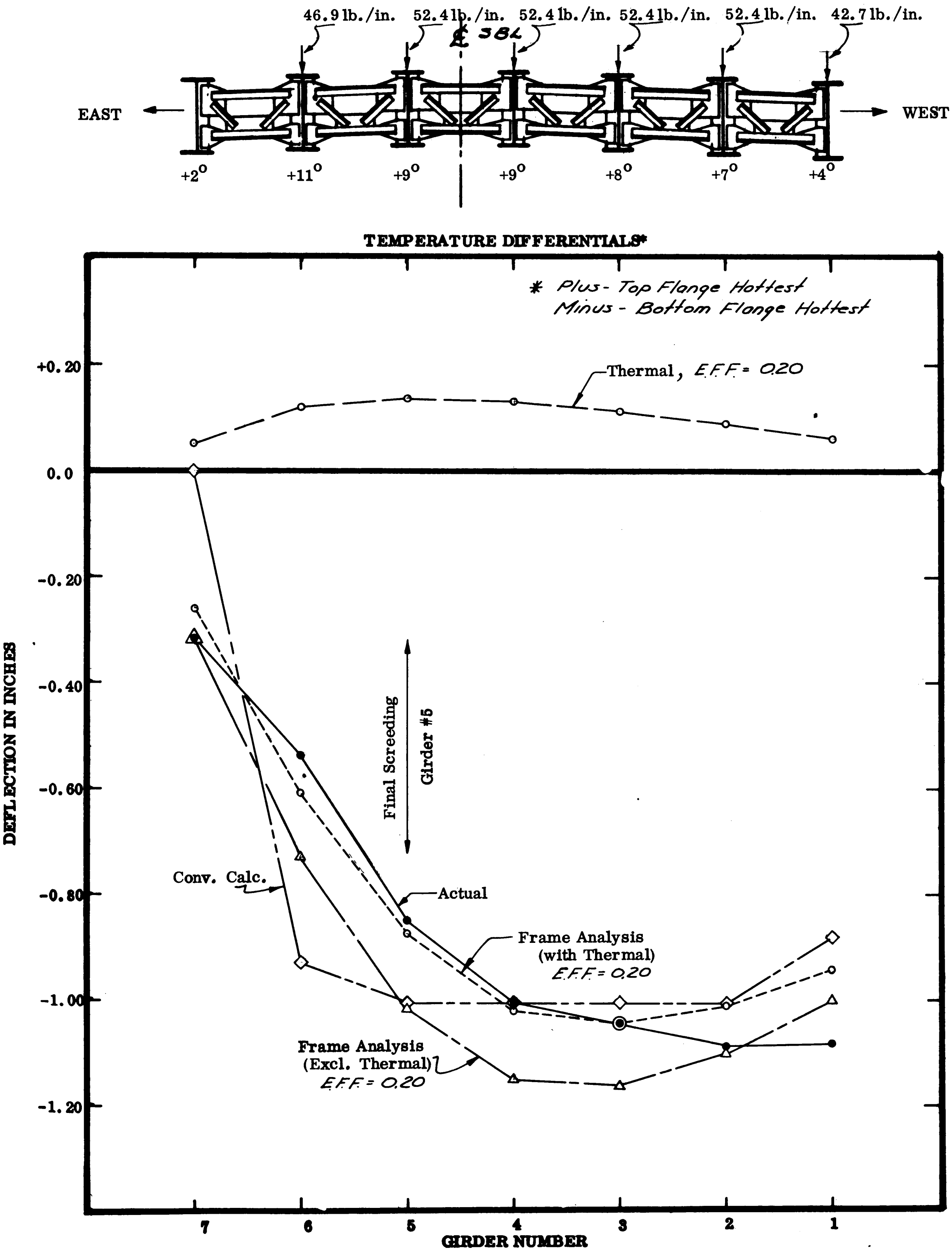


Figure 33. A comparison of actual field midspan deflections with computed deflections. Loading interval No. 5, Span #4, SBL Rte. 15.

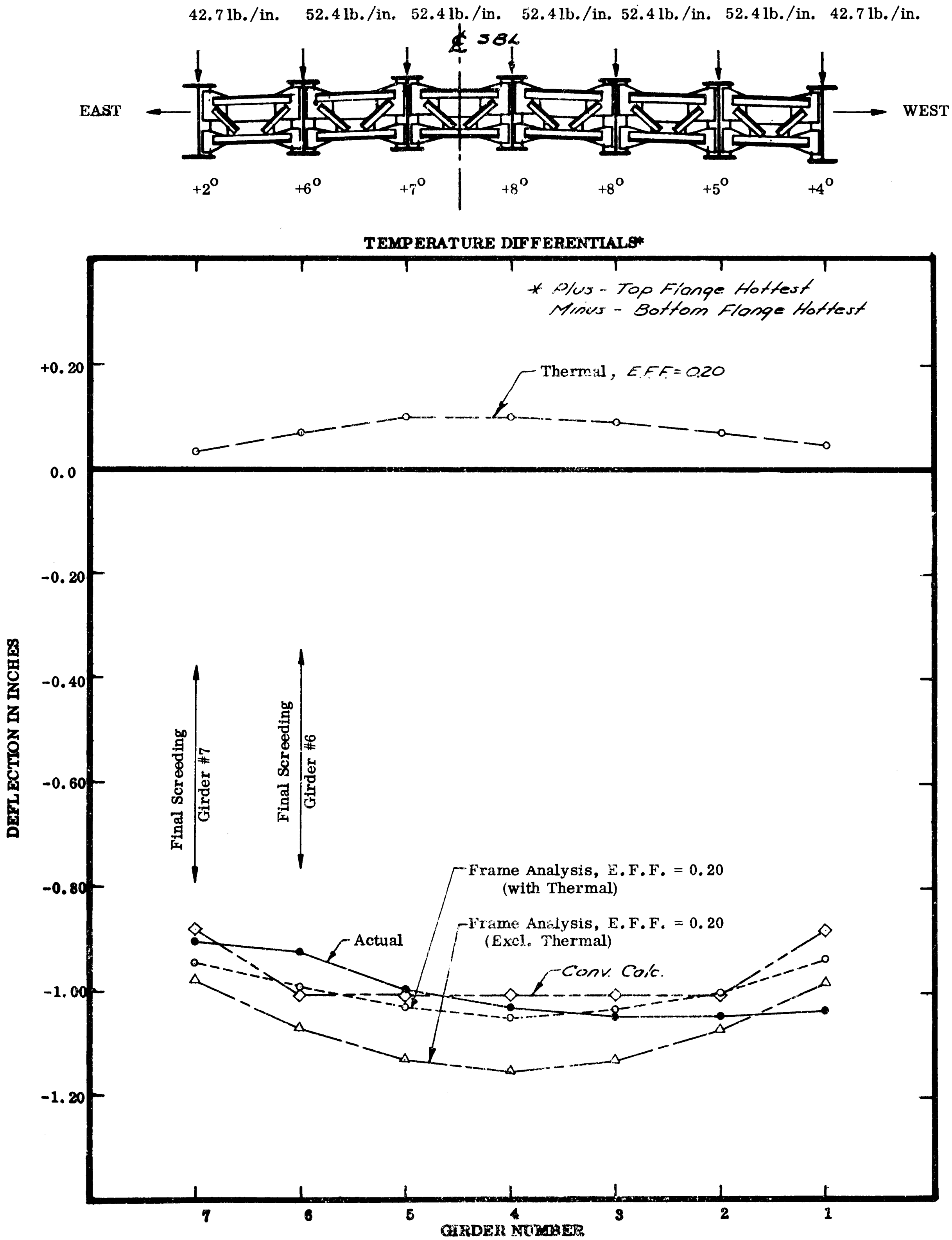


Figure 34. A comparison of actual field midspan deflections with computed deflections. Loading interval No. 6, Span #4, Rte. 15.

DISCUSSION OF RESULTS

While there was usually a reasonable lag behind concrete placement before the final longitudinal screeding pass was made over a given area, occasionally there was only a one- or two-bay lag. There would appear to be a need to compensate for such instances when conventional plan deflections have been used to establish forming elevations. If one considers the hypothetical situation discussed in the Rte. 607 results, a 40% reduction in the plan girder deflections would have been needed to avert a 0.40 inch shy deck thickness. On the other hand, if deck forming elevations were established to minimize the potential thermal differentials, only a 26% reduction in conventional plan deflections would be needed. Clearly, a routine reduction in conventional plan deflections values of at least 25% appears warranted. Since plan dead load deflections for simple span bridges seldom exceed 2 inches, a 25% reduction could conceivably result in a 1/2 inch extra thickness in the midspan area of some decks. Current Virginia Department of Highways specifications,[6] however, allow payment for up to 1/2 inch in excess of the plan deck thickness.

The results have indicated that the bolted diaphragm connections act in a semirigid fashion on the two spans tested, but for most practical calculations they could be assumed to be rigid. Use of conventionally calculated dead load deflections appears to be adequate as long as final longitudinal screeding follows concrete placement by approximately three-bay lengths. This result, however, must be qualified to structures similar to those tested. Bridges on heavy skews, for example, would represent a different situation and the use of conventional plan dead load deflection values and longitudinal screeding would be quite risky. The deflections obtained from the computer program (included in Appendix C) compared favorably with the field results when an E.F.F. of 0.20 was used; and the program could be utilized for checking deflections in questionable situations where longitudinal screeding is to be used.

Since the plan dead load deflections were too high for each of the study spans and have also been found to be too high in other investigations[2], it appears that much of the shy deck problem is due to this factor rather than to diaphragm rigidity.

ACKNOWLEDGEMENTS

This study was conducted in cooperation with the Federal Highway Administration and financed with HPR funds.

The author wishes to express his appreciation to F. L. Burroughs, assistant construction engineer, Virginia Department of Highways, for his advice and his assistance in arranging the field study with the Central Contracting Company. The excellent cooperation of the contractor and his personnel are also greatly appreciated.

Thanks are extended to W. T. McKeel, Jr., John Hagen, Clyde Giannini, John Stulting, Ray Gunn, Gene Leake, Jimmy Mills, Dick Hunt, L. E. Wood, and C. E. Maddox, all of the Virginia Highway Research Council, for their assistance in the instrumentation and data collection phase of the study.

Finally, the assistance of John Hagen in the analysis of the data is greatly appreciated.

This project was conducted under the general direction of Jack H. Dillard, state highway research engineer.

REFERENCES

1. Zuk, William, "Preliminary Report on the Theoretical Investigation of Bridge Beam Distortion During Construction," Virginia Highway Research Council, September 1967.

2. Hilton, M. H., W. Zuk, and W. T. McKeel, Jr., "Investigation of Shy Deck Thickness on the Route 264 Bridge over Brambleton Avenue," Virginia Highway Research Council, February 1968.

3. Lisle, F. N., "Theoretical Deflections of Semirigidly Connected Bridge Girder Systems During Construction", (an unpublished master's degree thesis to be presented to the University of Virginia).

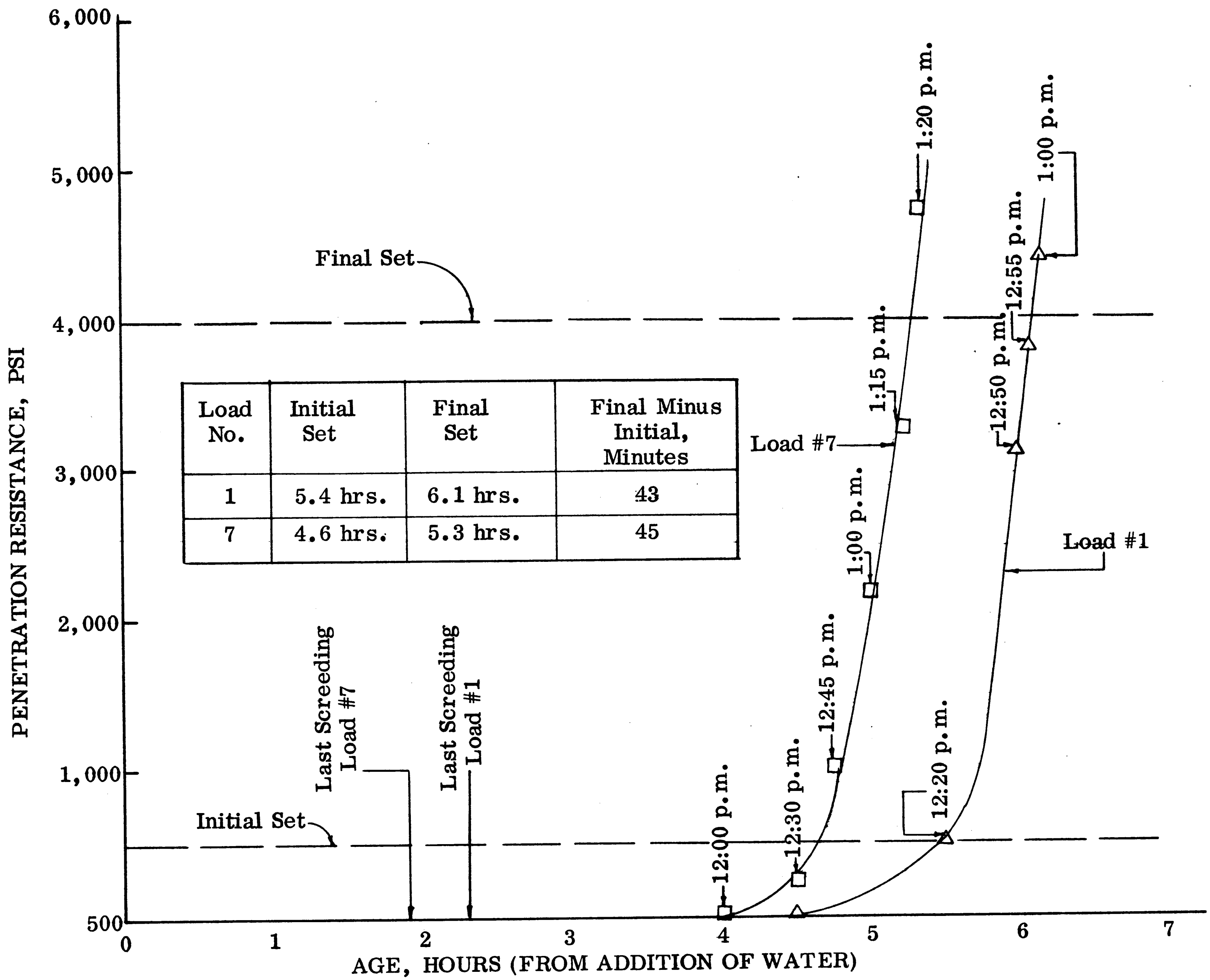
4. Hilton, M. H., "A Study of Girder Deflections During Bridge Deck Construction", Working Plan, Virginia Highway Research Council, March 1969.

5. Capps, M. W. R., "The Thermal Behaviour of the Beachley Viaduct/Wye Bridge", Road Research Laboratory, Report LR234, 1968.

6. Road and Bridge Specifications, Virginia Department of Highways, Richmond, Virginia (1970), p. 367.

APPENDIX A

SUPPLEMENTAL DATA FOR THE RTE. 607 SPAN



Load No.	Initial Set	Final Set	Final Minus Initial, Minutes
1	5.4 hrs.	6.1 hrs.	43
7	4.6 hrs.	5.3 hrs.	45

Figure A-1. Time of setting of concrete (Rte. 607, Span #3).

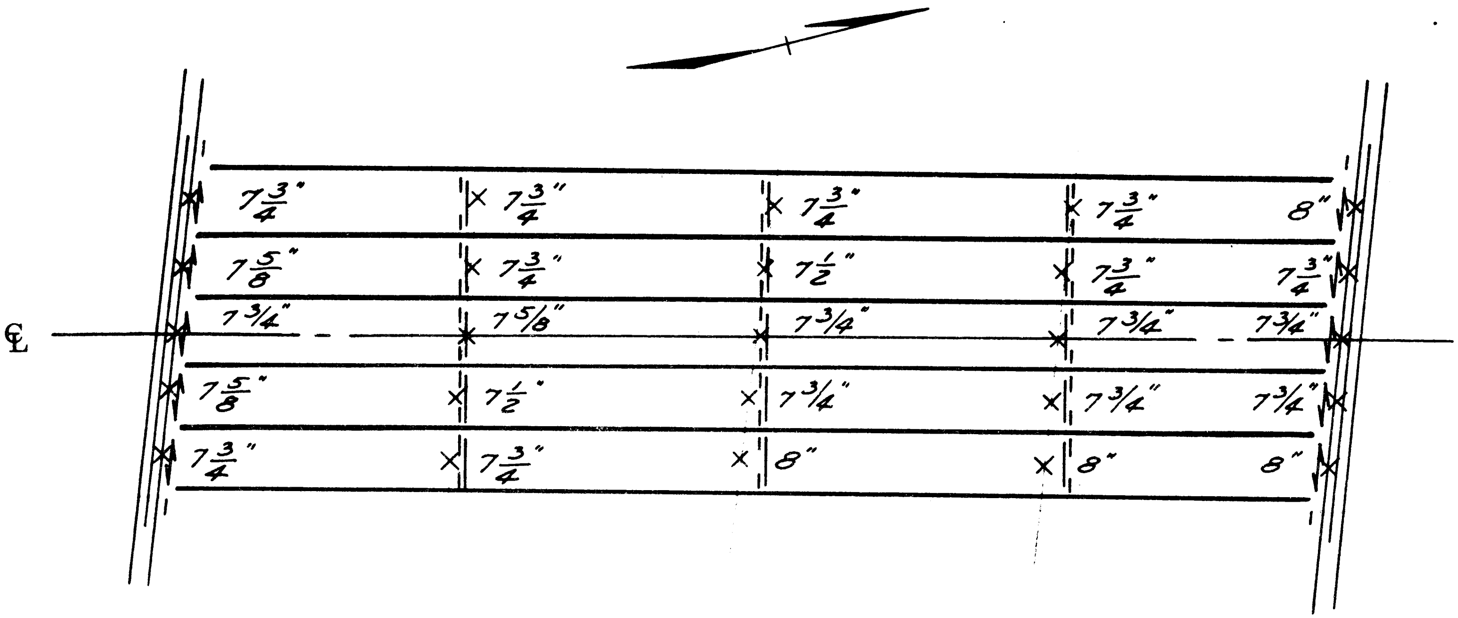
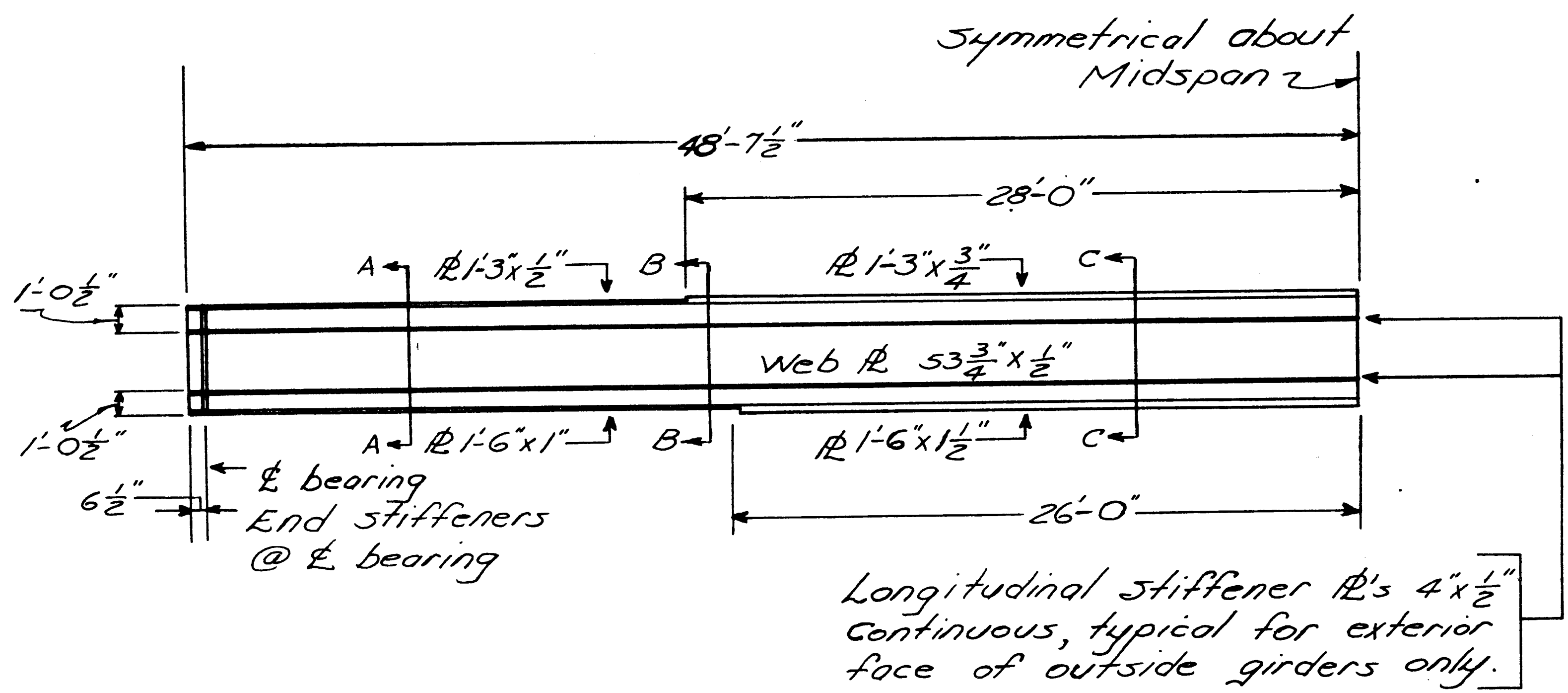


Figure A-2. Approximate locations of deck depth probe measurements on the Rte. 607 span. (Data furnished by the project inspector, Virginia Department of Highways.)



	I, Interior Girders	I, Exterior Girders	J, Polar Moment
Section AA	23,898	24,953	9
Section BB	27,806	28,776	11
Section CC	32,305	33,433	26

Figure A-3. Plate girder dimensions and moments of inertia of sections shown. Rte. 607, Span #3.

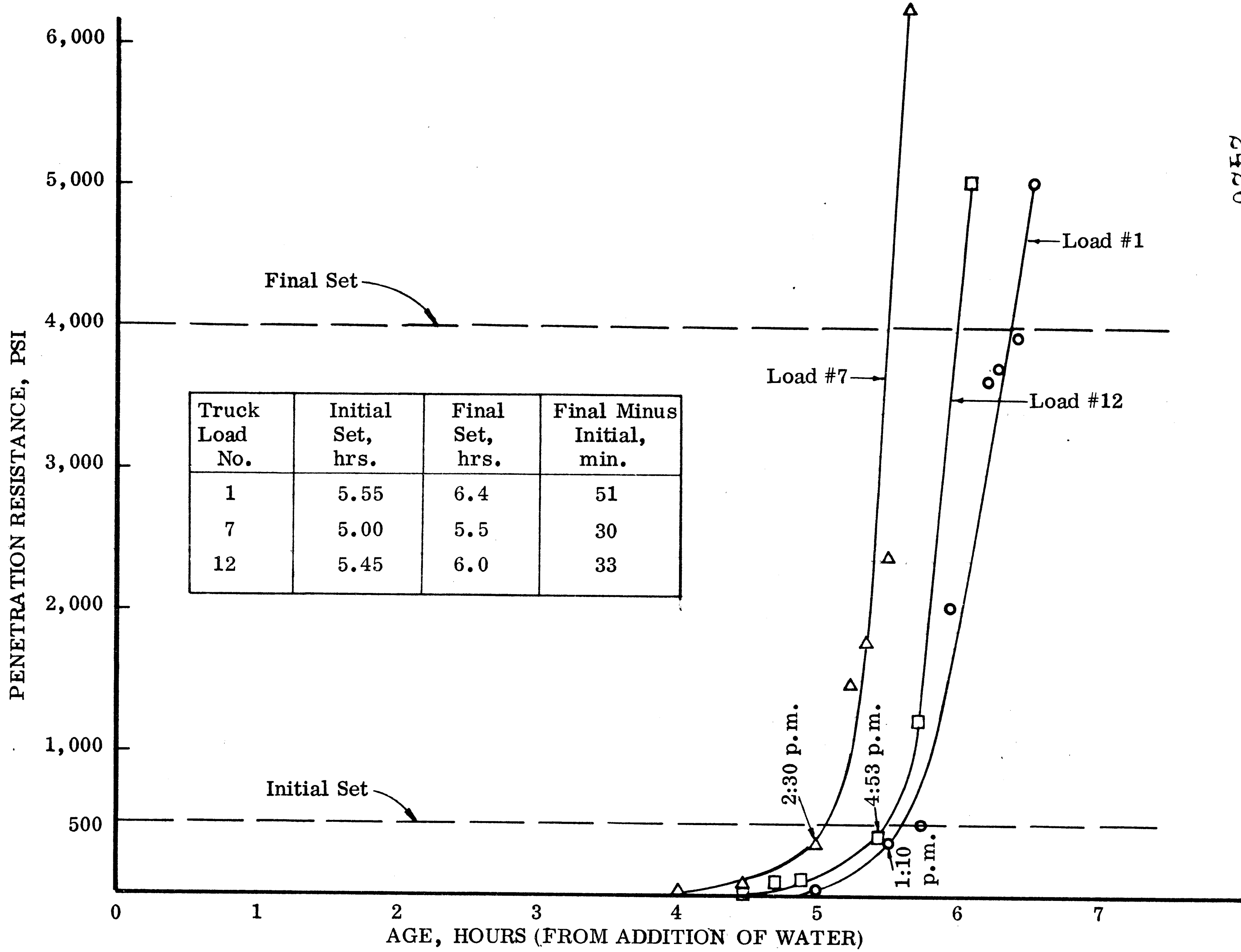
TABLE A-1

UNIT WEIGHTS OF CONCRETE
Rte. 607

Truck Load Number	Unit Weight $lb./ft.^3$
1	140.8
3	143.44
5	142.0
7	141.45
9	146.0
11	145.76

APPENDIX B

SUPPLEMENTAL DATA FOR THE RTE. 15 SPAN



Truck Load No.	Initial Set, hrs.	Final Set, hrs.	Final Minus Initial, min.
1	5.55	6.4	51
7	5.00	5.5	30
12	5.45	6.0	33

Figure B-1. Time of setting of concrete (Rte. 15, Span #4).

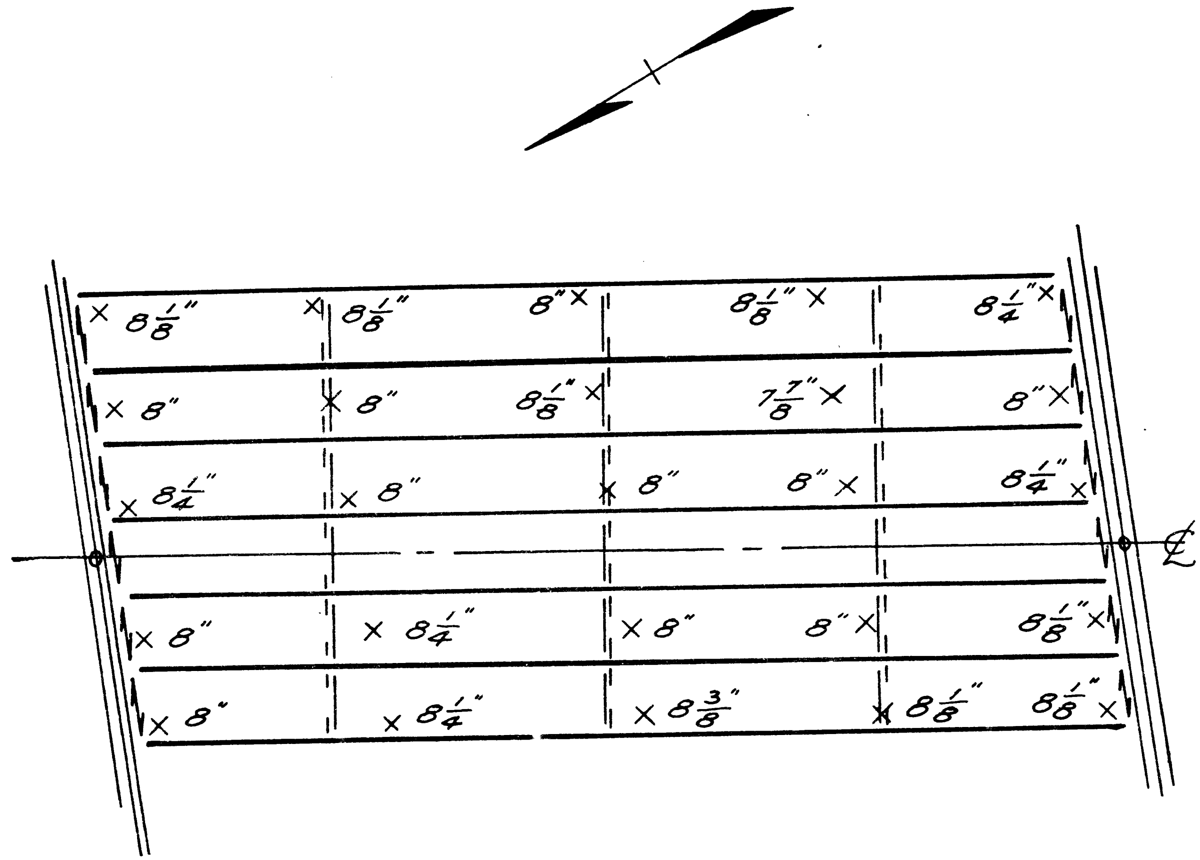


Figure B-2. Approximate locations of deck depth probe measurements on the Rte. 15 span. (Data furnished by the project inspector, Virginia Department of Highways.)

2422

TABLE B-1

UNIT WEIGHTS OF CONCRETE
Rte. 15

Truck Load Number	Unit Weight lb./ft.^{3}
1	140.0
3	141.0
5	140.0
7	140.0
10	139.0
12	140.5

APPENDIX C

COMPUTER PROGRAM FOR GIRDER DEFLECTIONS BY FRAME ANALYSIS

Appendix C provides a printout of the computer program developed by Lisle.(3) The program, which is written in Burroughs B5500 Extended Algol 60, can be used for analyzing any grid system under vertical loads and moments out of the grid plane. Loads can be applied directly to the joints or they can be applied along the members and converted to equivalent joint loads with consideration being given to semirigid connections. The results for each loading condition analyzed are tabulated as vertical movements and as rotations.

```
 09:24  AM     MARCH 22, 1971     *****************     HAGEN, J.J. HWY BRIDGE
BEGIN

PROCEDURE TIMEDATE(X);
    VALUE X;
    INTEGER X;
BEGIN
    LABEL FIN;
    IF X=2 THEN
BEGIN
    WRITE(LP[DBL],<"ELAPSED PROCESSOR TIME=",F10.2,X1,"SECONDS">,TIME(2)
     /60);
    GO TO FIN;
END;
    IF X=3 THEN
BEGIN
    WRITE(LP[DBL],<"ELAPSED I/O TIME=",F10.2,X1,"SECONDS">,TIME(3)/60);
    GO TO FIN;
END;
    IF X=0 THEN
BEGIN
    WRITE(LP[DBL],<"CURRENT DATE:",X5,A5>,TIME(0));
    GO TO FIN;
END;
    IF X=1 THEN
    WRITE(LP[DBL],<"ELAPSED TIME SINCE LAST START TIME=",F10.2,X1,
    "SECONDS">,TIME(1)/60);
FIN:
END OF PROCEDURE TIMEDATE;

PROCEDURE LOADMULT(A,B,C,N,SLOW);
    VALUE N;
    INTEGER N;
    LABEL SLOW;
    REAL ARRAY A[*,*],B,C[*];
BEGIN
    INTEGER I,J;
    FOR I←1STEP 1UNTIL N DO
BEGIN
    C[I]←0;
    FOR J←1STEP 1UNTIL N DO
    C[I]←C[I]+A[I,J]×B[J];
END;
END OF PROCEDURE LOADMULT;

PROCEDURE SYMMULT(A,B,C,N,SLOW);
    VALUE N;
    INTEGER N;
    LABEL SLOW;
    REAL ARRAY A[*,*],B[*,*],C[*,*];
BEGIN
    INTEGER I,J,K;
    FOR I←1STEP 1UNTIL N DO
    FOR J←1STEP 1UNTIL N DO
BEGIN
    C[I,J]←0;
```

```
    FOR K←1STEP 1UNTIL N DO
    C[I,J]←C[I,J]+A[I,K]×B[J,K];
END;
END OF PROCEDURE SYMMULT;

PROCEDURE PARTINV(A,N,SLOW);
    VALUE N;
    INTEGER N;
    LABEL SLOW;
    REAL ARRAY A[*,*];
BEGIN
    INTEGER NN,I,K,J,M,Q;
    REAL ARRAY G,B[1:N];
    REAL D,E;
    TIMEDATE(2);
    Q←2;
    A[1,1]←1.0/A[1,1];
    NN←N-1;
    FOR M←1 STEP 1 UNTIL NN DO
BEGIN
    K←M+1;
    FOR I←1STEP 1UNTIL K DO
    B[I]←A[K,I];
    FOR I←1 STEP 1 UNTIL M DO
BEGIN
    G[I]←0;
    FOR J←1 STEP 1 UNTIL M DO
    G[I]←G[I]+A[I,J]×B[J];
END;
    D←0;
    FOR I←1 STEP 1 UNTIL M DO
    D←D+B[I]×G[I];
    E←B[K]-D;
    B[K]←1.0/E;
    FOR I←1 STEP 1 UNTIL M DO
    B[I]←-G[I]×B[K];
    FOR I←1 STEP 1 UNTIL M DO
BEGIN
    FOR J←1STEP 1UNTIL M DO
    A[I,J]←A[I,J]-G[I]×B[J];
    A[I,K]←A[K,I]←B[I];
END;
    A[K,K]←B[K];
END;
    TIMEDATE(2);
END OF PROCEDURE PARTINV;

PROCEDURE FEL(FL,COO,SII,ID,Y1,Y2,W,L,R1,R2,R12,P);
    VALUE Y1,Y2,FL,ID,W,R1,R2,R12,SII,COO,L;
    REAL Y1,Y2,W,R1,R2,R12;
    REAL COO,SII,L;
    INTEGER FL,ID;
    REAL ARRAY P[0];
BEGIN
    INTEGER AI,AJ,BIJ,BJI;
    LABEL LL0,LL1,LL2,LL3,LL4;
```

```
    LABEL FAYE;
    REAL C,A,B,DELTAI,DELTAJ,AL,BA,G,FI,FJ,Q;
    AI←AJ←4;
    BIJ←BJI←2;
    IF ID≠0 THEN GO TO LL0;
    A←Y1;
    B←L−A;
    DELTAI←(W×B×(L×L−B×B))/(6×L);
    DELTAJ←(W×A×(L×L−A×A))/(6×L);
    GO TO FAYE;
LL0: IF ID≠1 THEN GO TO LL1;
    C←Y2−Y1;
    A←Y1+C/2;
    B←L−A;
    Q←W×C;
    DELTAI← (W×B×C×(4×A×(B+L)−C×C))/(24×L);
    DELTAJ←(W×A×C×(4×B×(A+L)−C×C))/(24×L);
    GO TO FAYE;
LL1: IF ID≠2 THEN GO TO LL2;
    C←Y2−Y1;
    A←Y1+2×C/3;
    B←L−A;
    AL←A/L;
    BA←B/L;
    G←C/L;
    Q←W×C/2;
    DELTAI←W×L×L×L×G×(270×(AL−AL×AL×AL)−G×G×(45×AL−2×G))/3240;
    DELTAJ←W×L×L×L×G×(270×(BA−BA×BA×BA)−G×G×(45×BA+2×G))/3240;
    GO TO FAYE;
LL2: IF ID≠3 THEN GO TO LL3;
    C←Y2−Y1;
    A←Y1+C/3;
    B←L−A;
    AL←A/L;
    BA←B/L;
    G←C/L;
    Q←W×C/2;
    DELTAI←W×L×L×L×G×(270×(AL−AL×AL×AL)−G×G×(45×AL+2×G))/3240;
    DELTAJ←W×L×L×L×G×(270×(BA−BA×BA×BA)−G×G×(45×BA−2×G))/3240;
FAYE:
    FI← (AI×R1×DELTAI−BIJ×R12×DELTAJ)/L;
    FJ←(BIJ×R12×DELTAI−AJ×R2×DELTAJ)/L;
    P[2]←FI×SII;
    P[3]←FI×COO;
    P[5]←FJ×SII;
    P[6]←FJ×COO;
    P[4]←(FI+FJ)/L;
    P[1]←−P[4];
    P[1]←P[1]+(B×Q)/L;
    P[4]←P[4]+(A×Q)/L;
    GO TO LL4;
LL3: WRITE(LP[1],<"ERROR IN LOADING DIScRIPTION−ID">);
LL4:
END OF PROCEDURE FEL;

    INTEGER NN,FR;
```

```
    REAL E,G;
    LABEL NOMO;
    ALPHA ARRAY NAME[0:12];

    READ(CR,<13A6>,FOR NN←0STEP 1UNTIL 12DO NAME[NN]);

    READ(CR,/,NN,FR,E,G);

BEGIN
    INTEGER CN,FL,ID;
    INTEGER U,MORE;
    INTEGER CFL;
    REAL X1,Y1,V1,X2,Y2,V2,IP,B,J,KP;
    REAL COUNT,PLOAD;
    REAL DX,DY,L,SI,CO,V,AI,AJ,A1,A2,E2;
    REAL BIJ,C1,C2,D,T,F,H,EPS;
    REAL RR1,RR2,RR12,RR;
    REAL I,X3,W;
    REAL ARRAY R1,R2,R12,R,LL,COO,SII[0:NN];
    REAL ARRAY P,LOAD,DEFL,CLAIN[0:FR];
    REAL ARRAY ST[1:6,1:6],N[0:NN,0:6],K[1:FR,1:FR];
    REAL ARRAY FAYEMC[0:FR];
    REAL ARRAY MATRIX[1:FR,1:FR];
    REAL ARRAY INP[0:6];
    ALPHA ARRAY DIR,MOM[0:6],DIRECT,MOMENT[0:FR];
    LABEL EX,NXT,SLOW,MC;
    LABEL  CFML;
    LABEL OU,START,AOK,POINT;
    LABEL K1,FIN,EXET;
    FORMAT F1("MEMBER",X3,"ORIGIN END COORD",X3,"END FIXITY",X3,
    "N-ORIGIN END COORD",X3,"END FIXITY",X3,"PROPERTIES-",X4,"E",X10,"G"
    ,X9,"I",X6,"J");
    FORMAT F2(X1,I3,X3,"X=",F7.2,X2,"Y=",F7.2,X2,"V=",F5.3,X4,"X=",F7.2,
    X2,"Y=",F7.2,X3,"V=",F5.3,X2,"L=",F7.2,X3,E10.3,X1,E10.3,X1,F7.1,
    X1,F6.1);
    FORMAT F3("DISPLACEMENT NO.",X5,"LOAD DIRECTION ON JOINT",X5,
    "LOAD MAGNITUDE");
    FORMAT F4(X7,I3,X20,A6,X12,F15.5);
    FILL MOM[*] WITH "FORC Z","MOM @X","MOM @Y","FORC Z","MOM @X",
    "MOM @Y";
    TIMEDATE(0);
    WRITE(LP[6],<X21,13A6>,FOR ID←0STEP 1UNTIL 12DO NAME[ID]);
    WRITE(LP[DBL],<"NUMBER OF MEMBERS=",I5>,NN);
    WRITE(LP[DBL],<"DEGREES OF FREEDOM=",I5>,FR);
    WRITE(LP[DBL],F1);
    FOR COUNT←1STEP 1UNTIL NN DO
BEGIN

    READ(CR,/,FL,X1,Y1,X2,Y2,FOR F←1STEP 1UNTIL 6DO INP[F],I,IP);
    READ(CR,/,V1,V2);

    FOR F←1 STEP 1 UNTIL 6 DO
    N[FL,F]←INP[F];
    DX←X2-X1;
    DY←Y2-Y1;
    L←SQRT(DX×DX + DY×DY);
```

```
    WRITE(LP,F2,FL,X1,Y1,V1,X2,Y2,V2,L,F,G,I,IP);
    LL[FL]←L;
    CO←(X2-X1)/L;
    COO[FL]←CO;
    SI←(Y2-Y1)/L;
    SII[FL]←SI;
    V←V2×V1;
    R1[FL]←3×V1/(4-V);
    R2[FL]←3×V2/(4-V);
    R12[FL]←3×V/(4-V);
    AI←4;
    AJ←4;
    BIJ←2;
    E2←E×I/L;
    A1←AI×R1[FL]×E2;
    A2←AJ×R2[FL]×E2;
    B←BIJ×R12[FL]×E2;
    C1←(A1+B)/L;
    C2←(A2+B)/L;
    D←(C1 +C2)/L;
    T←G×IP/L;
    ST[1,1]←ST[4,4]←D;
    ST[1,2]←ST[2,1]←C1×SI;
    ST[1,3]←ST[3,1]←-C1×CO;
    ST[1,4]←ST[4,1]←-D;
    ST[1,5]←ST[5,1]←C2×SI;
    ST[1,6]←ST[6,1]←-C2×CO;
    ST[2,2]←A1×SI×SI+T×CO×CO;
    ST[2,3]←ST[3,2]←-A1×CO×SI+T×CO×SI;
    ST[2,4]←ST[4,2]←-C1×SI;
    ST[2,5]←ST[5,2]←B×SI×SI-T×CO×CO;
    ST[2,6]←ST[6,2]←-B×CO×SI-T×CO×SI;
    ST[3,3]←A1×CO×CO+T×SI×SI;
    ST[3,4]←ST[4,3]←C1×CO;
    ST[3,5]←ST[5,3]←-B×CO×SI-T×CO×SI;
    ST[3,6]←ST[6,3]←B×CO×CO-T×SI×SI;
    ST[4,5]←ST[5,4]←-C2×SI;
    ST[4,6]←ST[6,4]←C2×CO;
    ST[5,5]←A2×SI×SI+T×CO×CO;
    ST[5,6]←ST[6,5]←-A2×CO×SI+T×CO×SI;
    ST[6,6]←A2×CO×CO+T×SI×SI;
    FOR F←1 STEP 1 UNTIL 6 DO
BEGIN
    IF INP[F]=0 THEN GO TO EX;
    KP←INP[F];
    FOR U←1STEP 1UNTIL 6 DO
BEGIN
    IF INP[U]=0 THEN GO TO NXT;
    H←INP[U];
    K[KP,H]←K[KP,H]+ST[F,U];
NXT:
END;
EX:
END;
END;
```

```
    FOR FL←1 STEP 1 UNTIL NN DO
BEGIN
    FOR I←1 STEP 1 UNTIL 6 DO
BEGIN
    KP←N[FL,I];
    F←I-1;
    MOMENT[KP]←MOM[F];
    IF I>3 THEN U←21×(I-3) ELSE U←I×21;
    CLAIN[KP]←U;
END;
END;

    WRITE(LP[1]);
    WRITE(LP[6],<X45,"STRUCTURE LOADINGS">);
    COUNT←0;

    READ(CR,/,MORE);

    WRITE(LP[DBL],<"NUMBER OF LOADINGS=",I3>,MORE);
START:
    WRITE(LP[DBL],<X38,"ADDITIONAL LOADING NO.",I3>,COUNT+1);

K1: READ(CR,/,FL,ID,Y1,Y2,W)[EXET];

    IF ID=4 THEN GO TO POINT;
    IF FL=0 THEN GO TO EXET;
    WRITE(LP,<"LOADING ON MEMBER NO.=",I3,X2,"OF TYPE NO.",I3,X2,
    "BETWEEN",X2,F6.2,X2,"AND",X2,F6.2,X2,"OF INTENSITY",X2,F10.5,X2,
    "IN THE Z DIRECTION">,FL,ID,Y1,Y2,W);
    CO←GOO[FL];SI←SII[FL];L←LL[FL];RR1←R1[FL];RR2←R2[FL]; RR12←R12[FL];
FEL(FL,GO,SI,ID,Y1,Y2,W,L,RR1,RR2,RR12,P);
    FOR F←1 STEP 1 UNTIL 6 DO
BEGIN
    G←N[FL,F];
    LOAD[G]←P[F]+ LOAD[G];
END;
    GO TO K1;
POINT:
    WRITE(LP[DBL],<X45,"POINT LOADS ON JOINTS">);
MC:

    READ(CR,/,I,PLOAD)[EXET];

    IF I=0 THEN GO TO EXET;
    WRITE(LP,<"POINT LOADING ON JOINT DISPLACEMENT NO.",I3,X2,"(",A6,
    ") OF MAGNITUDE=",F10.2>,I,MOMENT[I],PLOAD);
    P[I]←PLOAD;
    LOAD[I]←P[I]+LOAD[I];
    GO TO MC;
EXET:
    WRITE(LP[4]);
    WRITE(LP[DBL],<X43,"TOTAL EQUIVALENT JOINT LOADS">);
    WRITE(LP[DBL],<X20,"(NOTE:*****STRUCTURE LOADING IS SUMMATION OF ",
    "-ALL- PREVIOUS LOADINGS*****)">);
    WRITE(LP[DBL],F3);
    FOR FL←1 STEP 1 UNTIL FR DO
```

```
        WRITE(LP,F4,FL,MOMENT[FL],LOAD[FL]);
        IF COUNT ≠0 THEN GO TO OU;
        FOR F←1STEP 1UNTIL FR DO
        FOR G←1STEP 1UNTIL FR DO
        MATRIX[F,G]←K[F,G];
    PARTINV(K,FR,SLOW);
    OU:
    LOADMULT(K,LOAD,DEFL,FR,SLOW);
        LOADMULT(MATRIX,DEFL,P,FR,SLOW);
        FOR F←1STEP 1UNTIL FR DO
        FAYEMC[F]←P[F]-LOAD[F];
        LOADMULT(K,FAYEMC,P,FR,SLOW);
        WRITE(LP[4]);
        WRITE(LP[DBL],<X45,"RESULTING  DISPLACEMENTS">);
        WRITE(LP[DBL],<"DISPLACEMENT NO.",X12,"DEFLECTION Z",X10,"ROTATION "
        "@X",X10,"ROTATION @Y",X20,"APPROX. MAX. ERROR">);
        FOR FL←1STEP 1UNTIL FR DO
    BEGIN
        WRITE(LP[NO],<X105,E10.3>,ABS(P[FL]));
        WRITE(LP,<X4,I4,X*,F10.5>,FL,CLAIN[FL],DEFL[FL]);
    END;
        WRITE(LP[1]);
        COUNT←COUNT+1;
        IF COUNT< MORE THEN GO TO START;
        WRITE(LP[6]);
        TIMEDATE(2);
        TIMEDATE(3);
        GO TO NOMO;
    SLOW:
        WRITE(LP,<"SLOW ERROR FINISH">);
        TIMEDATE(2);
        TIMEDATE(3);
    END;
    NOMO: END OF PROGRAM.
   SQRT IS SEGMENT NUMBER 0036,PRT ADDRESS IS 0200
   OUTPUT(W) IS SEGMENT NUMBER 0037,PRT ADDRESS IS 0035
   BLOCK CONTROL IS SEGMENT NUMBER 0038,PRT ADDRESS IS 0005
   INPUT(W) IS SEGMENT NUMBER 0039,PRT ADDRESS IS 0060
   GO TO SOLVER  IS SEGMENT NUMBER 0040,PRT ADDRESS IS 0212
   ALGOL WRITE   IS SEGMENT NUMBER 0041,PRT ADDRESS IS 0014
   ALGOL READ    IS SEGMENT NUMBER 0042,PRT ADDRESS IS 0015
   ALGOL SELECT  IS SEGMENT NUMBER 0043,PRT ADDRESS IS 0016
COMPILATION TIME(SECONDS):    PR =    19    I/O =    77
NUMBER OF ERRORS DETECTED = 000. LAST ERROR ON CARD #
NUMBER OF SEQUENCE ERRORS COUNTED =    0.
NUMBER OF SLOW WARNINGS =    0.
PRT SIZE= 158;    TOTAL SEGMENT SIZE= 1330 WORDS.
DISK STORAGE REQ.=    76 SEGS.;    NO. SEGS.=  44.
ESTIMATED CORE STORAGE REQUIREMENT = 32000 WORDS.
```