EXCESSIVE BEAM DEFLECTIONS ON THE KY-52 BRIDGE
OVER DIX RIVER (BOYLE-GARRARD COUNTY LINE)

by

Theodore Hopwood II
Transportation Research Engineer V

Vishwas G. Oka
Transportation Research Engineer I

David Q. Hunsucker
Transportation Research Engineer Associate

and

James H. Havens
Former Associate Director

Kentucky Transportation Research Program
College of Engineering
University of Kentucky
Lexington, Kentucky

in cooperation with
Transportation Cabinet
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16. Abstract

Upon rehabilitation, beams and deck of the Ky 52 bridge over Dix River were observed to have excessive deflections. An investigation of the problem included field inspections, structural analyses, and interviews with relevant KYDOH and other personnel.

Field measurements were made of the beam deflections. Those were compared to construction field measurements and dead-load calculations.

The beam deflection problem is attributed to 1) a failure to provide for sufficient camber in the beams to accomodate for construction dead loads, and 2) pre-existing sags in the beams when placed prior to casting the deck. Part of the deck deflection problem can be attributed to a lack of provision for construction deflection in the top of slab elevations furnished to the contractor. However, cause of the remaining deck deflection was not determined.

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INTRODUCTION

In 1985, the Kentucky Transportation Cabinet (KyTC) rehabilitated the KY 52 bridge over Dix River. The bridge is located on the Boyle-Garrard County line (Figure 1). The six-span composite steel beam bridge originally had a 24-foot wide deck that was increased to 32 feet as part of the rehabilitation work. The original rolled beams were used as interior stringers in five simple spans (No.s 1, 2, 3, 5, and 6). Those were removed and cleaned during widening. Studs were welded to the upper flanges to provide composite action with the new deck. New steel beams were used for the exterior stringers. Also, a 150-foot truss in span 4 was removed and replaced by a span containing four welded plate girders.

After the new deck was placed on the stringer spans 1-3, the beams in those spans (including the new exterior beams) were found to possess excessive deflections. Field measurements revealed top of slab (deck) deflections of approximately 1.5 inches at the mid points of the three spans (1). Those deflections created noticeable humps and dips on the bridge deck and caused poor ride quality. Eventually, a smoother riding surface was provided by scarifying the humps and applying an overlay on the deck.

KyTC personnel investigated the problem and concluded that proper construction procedures were followed by both the contractor and state inspection personnel (op. cit. 1). Initially, the excessive deflections were attributed to the old beams. However, the new beams exhibited similar deflections.

The bridge designer for the renovation used a flexural tensile stress of 18,000 psi for design of the steel beams. KyTC normally uses a flexural tensile stress of 55 percent of 30,000 psi (16,500 psi) as a design stress for old beams (2). The beams were analyzed by the working stress method using an allowable flexural tensile stress of 16,500 psi. The analysis indicated that the existing beams were acceptable, even at the lower design stress level.

KyTC officials requested that Kentucky Transportation Center (KTC) personnel investigate the problem, determine probable cause(s), and recommend solutions to prevent similar occurrences. KTC personnel first inspected the site on April 9, 1986. Investigators visually confirmed the excessive sag on the exterior beams. Shortly after the initial inspection, KTC personnel submitted a proposal for the investigation.

KTC personnel proposed to make precise measurements at the site and relate those to the original beam elevations, to inspect shop and construction drawings, and to review construction records. Personnel knowledgeable of original construction and the bridge renovation were to be interviewed to obtain insight relative to possible causes of the excessive deflections. Work on the study began in June 1986.

KTC personnel grouped the possible causes of excessive deflections into four categories:

    1)   pre-existing causes

a) insufficient camber in the old beams,
b) corrosion damaged steel, and
c) substructure settlement.

2) design-related causes
a) miscalculation of camber in the beams,
b) incorrect selection of beam size, and
c) incorrect design assumptions.

3) fabrication-related causes
a) incorrect shop drawings,
b) improper steel fabrication, and
c) use of the wrong size of steel beams.

4) construction-related causes
a) improper placement of the beams with respect to camber,
b) excessive concrete in the deck, creating excessive dead loads,
c) weak forms, and
d) incorrect concrete placement.

KTC personnel intended to address all possibilities if an obvious cause of the excessive deflections was not determined.

FIELD INVESTIGATIONS

During the initial field investigations, KTC personnel discussed the beam deflection problem with the KyTC inspector and the contractor's personnel. Both parties mentioned that some difficulties were experienced during erection of the structural steel. Neither could recall any events related to the deflection problem.

In June 1986, KTC personnel surveyed the bridge. An optical theodolite was used to determine the profile of the exterior beams (beams 1 and 8) and elevations of pier caps. KTC personnel established temporary benchmarks on the north and south sides of the bridge. Five control points on the bottom flanges of the exterior beams were selected in each span. KTC personnel placed optical targets attached to C-clamps at those points (Figure 2). Division of Maintenance personnel provided a snooper allowing access to place an optical target at each of the control points on the exterior beams. KTC personnel measured horizontal sweep and the vertical angle with respect to station points at all control points and on top of pier caps.

To investigate the possibility that the dead-load deflections were due to excess concrete in the bridge deck, KTC personnel obtained 7 cores from the deck. Core locations are shown in Figure 3. The average thickness of the cores was 8.9 inches. That conforms reasonably well with the design thickness of 8.5 inches.

Inspections of the original beams revealed some corrosion at their ends. That is usually caused by joint leakage. Corrosion damage did not appear sufficiently severe to render the beams unfit for further service or to affect their deflections.

Visual inspections revealed transverse tilting in beams of all the stringer spans. The beams were upright at the piers and abutments. Beam tilt was greatest at mid span with the lower flanges rotating slightly in an opposite direction of the upper flange.

KTC personnel measured the lateral tilt in the web areas of all spans including the plate-girder span (Figure 4). A level and ruler were used to measure tilt over a length of 24 inches in the center of the webs. No tilting was detected in the plate-girder span. Measured values of web tilt are listed in Table 1.

Due to the differences in tilt between the plate-girder and rolled beam spans, KTC personnel erroneously assumed that excessive beam deflection was related to insufficient transverse restraint. KTC personnel initially attributed transverse tilting to plastic deformation.

On November 9, 1988, KyTC and KTC personnel inspected the bridge. KyTC personnel revealed that though the rolled beams tilted, they maintained their cross-sectional shape and were not plastically deformed. During the inspection, the transverse beam spacing was measured at a center diaphragm in span 1. That diaphragm is located near the upper flange of the beams. The spacing between the webs of two adjacent beams was greater at the top flange than at the lower flange. That indicated the beam tilt probably was due to installation of the diaphragms.

KTC personnel inspected concrete on the deck and plinths. The concrete haunches over the beams were constructed to a greater elevation near the midpoint of span 6 than those in span 1. That was due to the KyTC requirement that the designer furnish the contractor with new top of slab elevations for spans 5 and 6 (3). The new elevations were intended to compensate for anticipated deflections similar to those in spans 1-3. Those revised elevations increased the height of the slab 2 inches at the mid points with no change at the ends of the spans.

The bottom surface of the deck slabs outside the exterior beams followed the profile of the upper flanges of those beams. A rustication groove was present at the construction joint between the deck slab and the plinth. The groove followed the profile of the beams and exhibited a similar downward deflection since it was located at a constant spacing from the bottom of the slab (Figure 5). The plinths were about one inch higher at the mid points than at the ends on all the spans. That provided a uniform plinth height across each span.

KTC personnel measured the distance from the deck to the clean-line edge at the base of each plinth. That distance is normally 3 inches. However, in spans 1-3, that distance measured about 2 inches on both sides at the midpoint of the deck. That is probably due to the overlays applied to those spans.

DATA ANALYSES

A computer program was developed to convert field measurements of the control points to respective elevations using the known station elevations. The computer program also calculated the deflection of the exterior beams at the center of each span. Computed deflections varied from 0.64 to 1.90 inches. Table 2 lists deflections calculated by that program. Graphs were plotted showing the profile of those beams. Figure 6 is a typical profile for the lower flange of an exterior beam.

Computer-calculated pier elevations based on horizontal and vertical angle measurements matched elevations of the piers shown on the construction drawings. That indicated the piers had not settled and that no construction problems or errors existed related to the substructure.

KTC personnel reviewed top-of-beam elevations obtained on site by KyTC personnel before casting the deck slabs on spans 1-3. Those measurements are used to set the forms under the deck to provide constant slab thickness. KTC personnel calculated deflections for the midpoints of the beams (Table 3). All of the new beams in those spans sagged downward between 0.23 to 0.60 inch at the midpoints. Most of the original beams which are now interior beams in spans 1-3 also lacked upward cambers. Midpoint deflections of those beams varied from 0.07 inch upward to 0.61 inch downward.

KTC personnel plotted the top of slab elevations and the corresponding top of beam elevations versus distance along the beams in spans 1-3. Figure 7 shows the top of slab construction elevations and the top of beam profile for beam 1. The lower line represents the top of slab construction elevations and relates to the left ordinate. The top of slab elevations were obtained from the design drawings. The upper line represents the top of the beam profile and relates to the right ordinate. The top of beam elevations were obtained from the KyTC field readings.

The top of slab construction elevation points approximate a straight line in spans 1-3. That conforms to correspondence stating the designer had omitted a construction camber (for deck dead load). That construction camber should have been about 0.5 inch for the new rolled beams (op. cit. 1). If the designer had provided for dead loads, the top of slab profiles would arch upward slightly between the beam supports. Those supports are located at the 0-, 60-, 120-, and 180-foot distances along the beam. Pre-existent sags on the top of beam profiles occur between peaks at the beam support locations.

Shop drawings from the steel fabricator indicated that the rolled beams were manufactured with the natural (as-rolled) camber up.

Beam deflections were computed (Appendix). Beam dead load would account for 0.18 inch of deflection at mid point when simply supported. Most of the beams had greater deflections when placed on their supports (as determined by field measurements).

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The dead load of the deck and plinth would account for 0.60 inch of deflection at the midpoint. Based upon top of beam measurements and deflection calculations, the exterior beams of spans 1-3 should deflect from 0.82 to 1.20 inches. That compares to field measurements that vary from 1.21 to 1.90 inches.

KTC personnel reviewed the designer's calculations for accuracy of assumptions and computations. The selection of beams satisfied the minimum depth requirements of current AASHTO standard specifications for highway bridges. Those specifications require the ratio of the overall depth of the girder (concrete slab plus steel girder) to the length of the span should not be less than 1/25 for composite girders. The ratio of the depth of the steel member alone to the length of the span should not be less than 1/30. The latter ratio was 1/18 for the plate girder span and 1/20 for the stringer spans.

Additional calculations revealed that the tilt measured would have a negligible effect on beam deflection.

CONCLUSIONS

The beam deflections were related to an initial downward sag when simply supported and an uncompensated construction dead load. In spans 1-3, the average sag of the exterior beams was 0.40 inch. The uncompensated construction dead load would provide about 0.60 inch deflection for the deck and plinths. The resulting total average deflection of the exterior beams in those spans is 1 inch.

Deflections based on field measurements averaged 1.53 inches for the exterior beams in spans 1-3. The difference between calculated and field-data average deflection values could be due to lack of precision in field measurements and/or uncertainties of exact construction or existing dead loads. In any case, there is no reason to believe that the beams deflected unusually when loaded.

The cause of the initial downward deflection of many of the beams in spans 1-3 (including the new beams) was never determined. The contractor placed the beams properly. If they had sufficient initial camber, they would not have sagged under their own weight.

The original deck deflection measured by KyTC for spans 1-3 was similar to the average exterior beam deflections derived from the KTC field measurements, about 1.5 inches. As noted, the uncompensated construction dead load would account for 0.60 inch of deflection. Initial beam sagging should not be a factor in deck deflection. The contractor normally compensates for beam sag when setting the rails for the finishing machine. The reason for deck sagging in excess of the uncompensated construction dead load was not determined.

There was no evidence that any pre-existing problem in the original bridge contributed to this event. The only design-related problem was failure to provide

initial construction camber. The only question about fabrication of the new beams relates to the provision for beam camber. There is no direct evidence of a construction-related cause or factor.

The diaphragms exhibited irregular workmanship and beam connections that varied from the design drawings. It is likely that camber differences between adjacent beams contributed to those circumstances.

REFERENCES

1.   Lile, J. R., Memorandum from Kentucky Transportation Cabinet, Division of Construction, to Johnson, R. A., Chief District Engineer, District 7, on KY 52 bridge, dated September 25, 1985.

2.   Roberts, J. A., Memorandum from Kentucky Transportation Cabinet, Division of Bridges, to file on KY 52 bridge dated October 18, 1985.

3.   Roberts, J. A., Letter to Koja, B., WMB Engineers Inc., Lexington, KY, on KY 52 bridge, dated September 23, 1985.

Table 1.   Tilt Measurements Taken in Web Areas of Rolled Beams

Beam Number	Span Number	Tilt (Inch)
1	1	0.20
	2	1.00
	3	0.72
	4	--
	5	0.28
	6	1.00
8	1	0.44
	2	0.08
	3	0.24
	4	--
	5	0.36
	6	0.52

Notes 1. Negligible beam tilt was detected for the plate girders in span 4.

Table 2.   Measured Deflections on Exterior Beams

Beam Number	Span Number	Deflection (Inch)
1	1	1.90
	2	1.65
	3	1.35
	4	1.88
	5	0.97
	6	0.64
8	1	1.21
	2	1.85
	3	1.25
	4	1.60
	5	1.10
	6	1.54

Notes 1. Span 4 is a plate girder span.
2. Spans 1, 2, 3, 4, 5, and 6 are approximately 61 feet long.
3. Span 4 is 150 feet long.
4. Negligible deflection was calculated for Beam 1 Span 6.
5. Numbering of beams is from North to South.
6. Numbering of span is from West to East.

Table 3.   Deflection at the Center of the Spans Calculated from the X Dimensions

Beam No.	Span 1 Deflection (Inch)	Span 2 Deflection (Inch)	Span 3 Deflection (Inch)
1	0.45	0.60	0.23
2	0.34	0.50	0.35
3	0.32	0.41	0.50
4	0.61	0.41	0.22
5	0.16	0.12	0.14
6	-0.07	0.35	0.17
7	0.04	0.11	0.08
8	0.34	0.32	0.47

Note 1. Positive numbers represent downward deflection.
      2. Negative numbers represent upward deflection (camber).

Figure 1.   Six-Span Composite Steel Beam Bridge on KY 52 at
the Boyle-Garrard County Line (Facing Westward).

Figure 2.   Optical Targets on Lower Flange of a Beam.
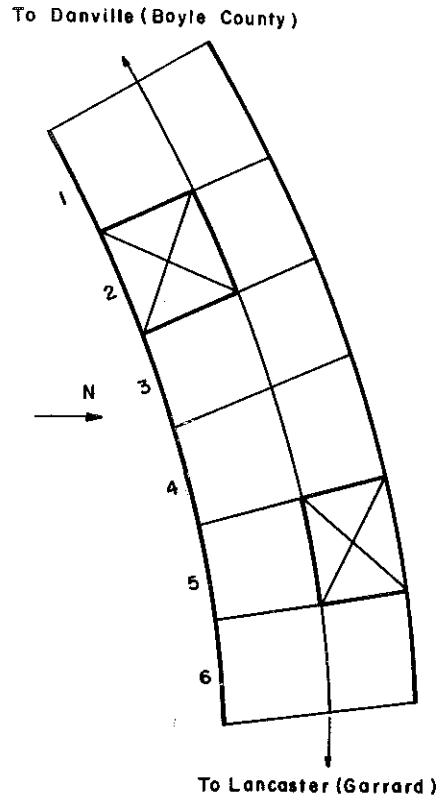
Figure 3a. Core Locations on KY 52 Bridge.

To Danville (Boyle County)

N →

1
2
3
4
5
6

To Lancaster (Garrard)

Figure 3b. Core Locations on Span 5, Westbound.

Height of Core
① 10 1/2"
② 8"
③ 9"
④ 7 1/2"

① 28"
② 35'
③ 11'0"
25'5"
22'7" 24'9"
④ 40"
37"

N →

Height of Core
⑤ 9"
⑥ 9 1/2"
⑦ 9"

45"
45" ⑤
11'0" 24'9"
45" ⑥
11'0" ⑦

Figure 3c. Core Locations on Span 2 Eastbound.

N →

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Figure 4. Measurement of Transverse Tilt on a Beam.

Figure 5. Rustication Groove in Spans 5 & 6 (Facing Eastward).

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BEAM 1— SPAN 2 PROFILE
AFTER CASTING DECK

ELEVATION

HORIZONTAL DISTANCE - Ft.
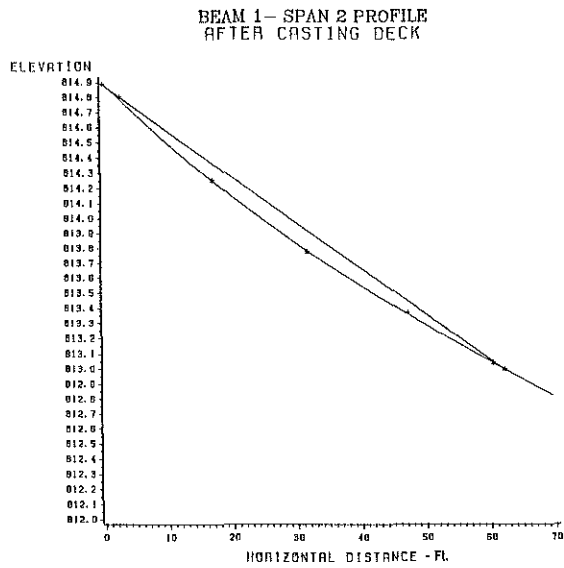
Figure 6.   Lower Flange Profile of Beam 1 (Exterior Beam) on
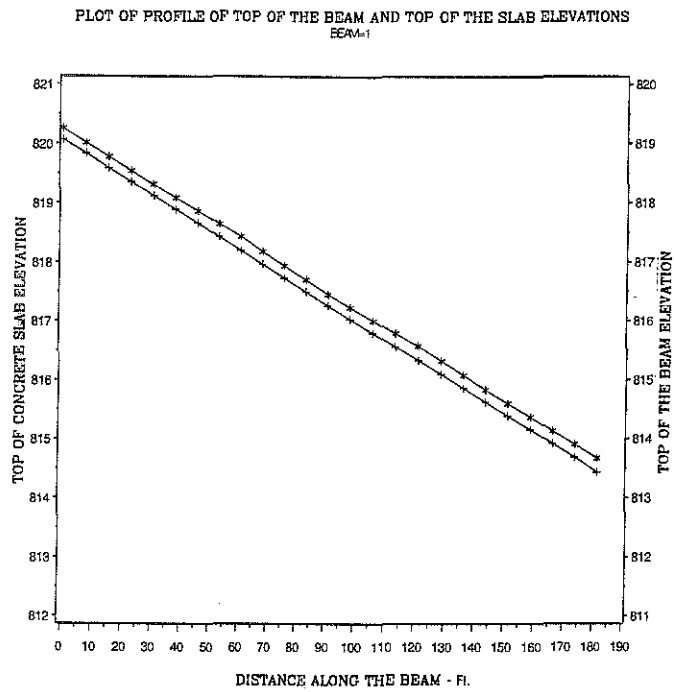Span 2 Showing Deflection after Casting Deck.

PLOT OF PROFILE OF TOP OF THE BEAM AND TOP OF THE SLAB ELEVATIONS
BEAM=1

DISTANCE ALONG THE BEAM - Ft.

Figure 7.   Top of Slab Elevations (Left Ordinate - Lower Line) and
Top of Beam Elevations (Right Ordinate - Upper Curve)
Versus Their Locations Along the Beam 1 (Spans 1-3).

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APPENDIX

1. CALCULATION OF STRINGER BEAM MIDSPAN DEFLECTION

1.1 Load Calculations

Thickness of deck slab = 8.5 inches = 0.7083 feet

Dead load carried by WF 36x150 steel beam

1. Weight of concrete slab = 0.7083 ft x 4.0833 ft x 0.150 kips/cuft
   = 0.4339 kips/ft

2. Beam weight   = 0.1500 kips/ft

3. Weight of plinth:
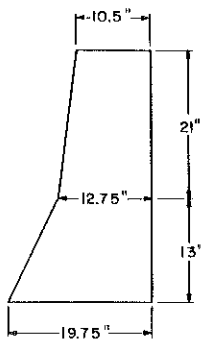
SECTION OF PLINTH

Volume of plinth per foot =

$$\frac{21 \text{ in.}(10.5 \text{ in.} + 12.75 \text{ in.})/2}{12 \text{ in./ft} \times 12 \text{ in./ft}} + \frac{13 \text{ in.}(12.75 \text{ in.} + 19.75 \text{ in.})/2}{12 \text{ in./ft} \times 12 \text{ in./ft}}$$

= 1.6953 + 1.4670 = 3.1623 cuft/ft

Weight of plinth = 3.1623 cuft/ft x 0.150 kips/cuft
   = 0.4743 kips/ft

Weight of two plinths = 0.9486 kips/ft

Weight of plinths/stringer = 0.1185 kips/ft

Dead load carried by stringer alone

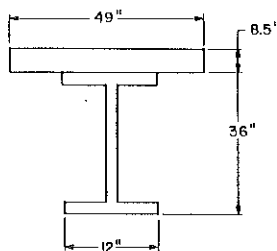1. Concrete slab = 0.4339 kips/ft

Dead load carried by composite section

1. Plinths = 0.1185 kips/ft

1.2 Properties of Composite Section

1.2.1 Composite Section Having Modular Ratio n = 24

Effective width of flange is minimum of

Effective Cross-Sectional Area:

1. 1/4th of span 1/4 x 61 in. x 12 in./ft = 183 inches

2. Stringer spacing = 4.0833 x 12 in./ft = 49 inches

3. 12 x slab thickness - 12 x 8.5 in. = 102 inches

   Use 49 inches

Effective cross-sectional area:

1. I section $= 44.16$ in.^{2}

2. Concrete
   top flange 49 in. x 8.5 in./n(=24) $= 17.35$ in.^{2}

   Total (1)+(2) $= 61.51$ in.^{2}

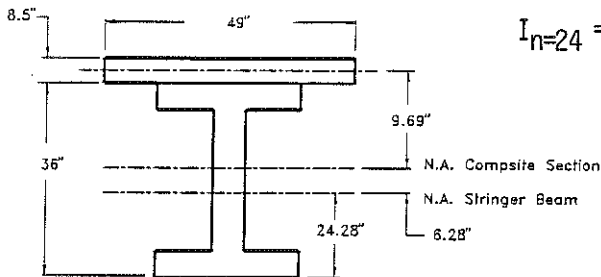Moment of concrete and steel area about bottom of the I-section

$$44.16 \text{ in.}^2 \times 18 \text{ in.} + 17.35 \text{ in.}^2 (4.25 \text{ in.} + 36 \text{ in.}) = 61.51 \text{ in.}^2 \times y_{24}$$

where y_{24} = distance of neutral axis from bottom of I-section

$$= 24.28 \text{ inches}$$

$d_{top} = 36 - 24.28 = 11.72$ inches and $d_{bottom} = 24.28$ inches

1.2.2 Moment of Inertia of Composite Section Having Modular Ratio n = 24

$$I_{n=24} = I_{stringer} + I_{slab}$$

$$= 9,012.1 \text{ in.}^4 + 44.16 \text{ in.}^2 (6.28 \text{ in.})^2$$

$$+ 1/12(17.35 \text{ in.}^2 \times (8.5 \text{ in.})^2)$$

$$+ (17.35 \text{ in.}^2)(9.69 \text{ in.} + 6.28 \text{ in.})^2$$

$$= 15,283.12 \text{ in.}^4$$

1.3 Deflection Calculations

1.3.1 Deflection due to dead load of concrete slab on stringer

$$D_s = 45WL^4/2E_sI_s \qquad \text{(From USS Highway Structures Design Handbook, Volume II)}$$

$$= [45 \times 0.4339 \text{ Kips/ft} \times (61 \text{ ft})^4]/[2 \times 29 \times 10^3 \text{ksi} \times 9,012.1 \text{ in.}^4] = 0.52 \text{ inches}$$

where D_s = dead-load deflection at mid span in inches,
        W = dead load of concrete slab in kips/foot,
        L = span in feet,
        E_s = modulus of elasticity of steel = 29×10^3 ksi, and
        I_s = moment of inertia of steel beam at mid span about centroidal axis in in.^{4}

1.3.2 Deflection due to dead load of plinths

$$D_p = 45W_p L^4/2E_s I_{(n=24)}$$

$$= [45 \times 0.1185 \text{ kips/ft} \times (61 \text{ ft})^4]/[2 \times 29 \times 10^3 \text{ksi} \times 15{,}283.12 \text{ in.}^4]$$
$$= 0.08 \text{ inches}$$

where D_p = deflection at mid span due to weight of plinths,
        $I_{n=24}$ = moment of inertia at mid span of composite section
            about centroidal axis having modular ratio n=24, and
        W_p = weight of plinths in kips/foot

1.3.3. Deflection due to dead load of stringer

$$D_{st} = 45WL^4/2E_s I_s \quad \text{(From USS Highway Structures Design Handbook, Volume II)}$$

$$= [45 \times 0.1500 \text{ Kips/ft} \times (61 \text{ ft})^4]/[2 \times 29 \times 10^3 \text{ksi} \times 9{,}012.1 \text{ in.}^4] = 0.18 \text{ inches}$$

where D_{st} = dead-load deflection at mid span in inches,
        W = dead load of a steel beam in kips/foot,
        L = span in feet,
        E_s = modulus of elasticity of steel = 29×10^3 ksi, and
        I = moment of inertia of steel beam at mid span
            about centroidal axis in in.^{4}

1.4 Deflection at mid span due to dead load of deck and plinths

$$D_t = D_s + D_p$$

0.52 inches + 0.08 inches = 0.60 inches

where D_t = deck and plinth dead load deflection at mid span in inches

        D_s = deflection due to dead load of concrete slab in inches

        D_p = deflection due to dead load of plinths in inches