RP232

STRESS DISTRIBUTION IN WELDED STEEL PEDESTALS

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ABSTRACT

Two welded steel pedestals were tested in compression and a study of the stress distribution was made. These pedestals were designed for the purpose of replacing cast-steel pedestals for bridges because they could be made in a shorter time and without the use of expensive patterns. In one of the pedestals, the inclined plates did not touch the bottom plates, and the parts of the pedestal could be clamped in their proper relative position before welding the inclined plates to the bottom plates. The stress distribution in this pedestal was uniform over the inclined plates and the stiffeners at mid height. The inclined plates of the other pedestal were machined so as to fit directly between the top and base plates and a fillet weld was used to maintain the relative position of the plates. The stresses in the stiffeners of this pedestal were more than twice as great as the stresses in the inclined plates for loads in the designed working range. It was evident that the uniform distribution of stress in the first pedestal was due to the even bearing obtained by the use of the large weld between the inclined and the base plates.

The maximum strengths of the two pedestals were, however, about the same and more than six times the load for which they were designed.

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I. INTRODUCTION

Pedestals are used to support the ends of bridges. They distribute the concentrated load over the top of the masonry or concrete bridge piers. If the lower surface of the pedestal is sufficiently large, the compressive stresses in the pier will be safe.

Bridge pedestals have in the past been either grey iron or steel castings. Cast pedestals have the disadvantages that the patterns are expensive and that the time required to make the castings sometimes delays the completion of the bridge.

Pedestals, especially the smaller sizes, can not be economically made by riveting together pieces of rolled structural steel. As welding is coming into use for the fabrication of steel structures, the American Bridge Co. designed pedestals of rolled steel plates, joining the pieces by electric arc welding. As the use of welded pedestals was a radical departure from present practice, the cooperation of the bureau was obtained in determining the compressive strength.

II. THE PEDESTALS

Drawings of the pedestals are shown in Figure 1. They were designed to carry a load of 350,000 pounds, and represent one size in a series of pedestals for different bridge spans and loadings.

The difference between the pedestals *F1* and *F2* lies wholly in the connection between the bottom plate and the inclined plates. In pedestal *F1*, the inclined plates (*P2*) do not touch the bottom plate, and the parts of the pedestal can be truly lined and clamped in their proper relative positions before the weld "A" is made. The corresponding inclined plates (*P6*) of pedestal *F2* must be carefully

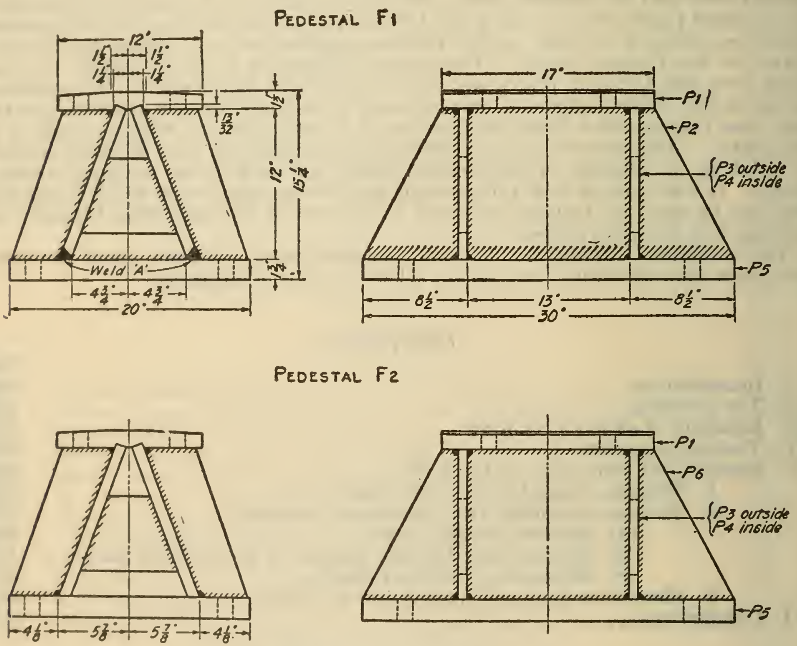

FIGURE 1.—*Drawing of the pedestals*

MATERIAL FOR PEDESTAL F1

P1, one plate 12 by 1½ by 17 inches.
P2, two plates 12¹³⁄₁₆ (finished width) by ⅞ by 30 inches.
P3, four plates 3½ by ¾ by 12¹³⁄₁₆ inches (neat length).
P4, two plates 6 by ¾ by 8 inches.
P5, one plate 20 by 1¾ by 30 inches.

MATERIAL FOR PEDESTAL F2

All material same as that in F1 except plates P6 used instead of plates P2.
P6, two plates 13¼ (finished width) by ⅞ by 30 inches.

NOTE.—All welds ⅜ inch except weld A for pedestal F1.

finished at both the bottom and the top, and any irregularity in any of the plates may result in uneven bearing after the seams are welded.

III. LOCATION OF STRAIN GAGE LINES

Twenty 2-inch gage lines were used for the study of stress distribution on each pedestal. It was recognized that this number was not sufficient to determine fully the stress distribution, but the results show that much was learned which would aid in perfecting the design. The location of the gage lines is shown in Figures 2, 3, and 4.

It was convenient to designate the four sides of the pedestal by the initials of the cardinal points of the compass toward which the sides faced when in the testing machine and the gage lines on each side by an additional number or letter to show the location.

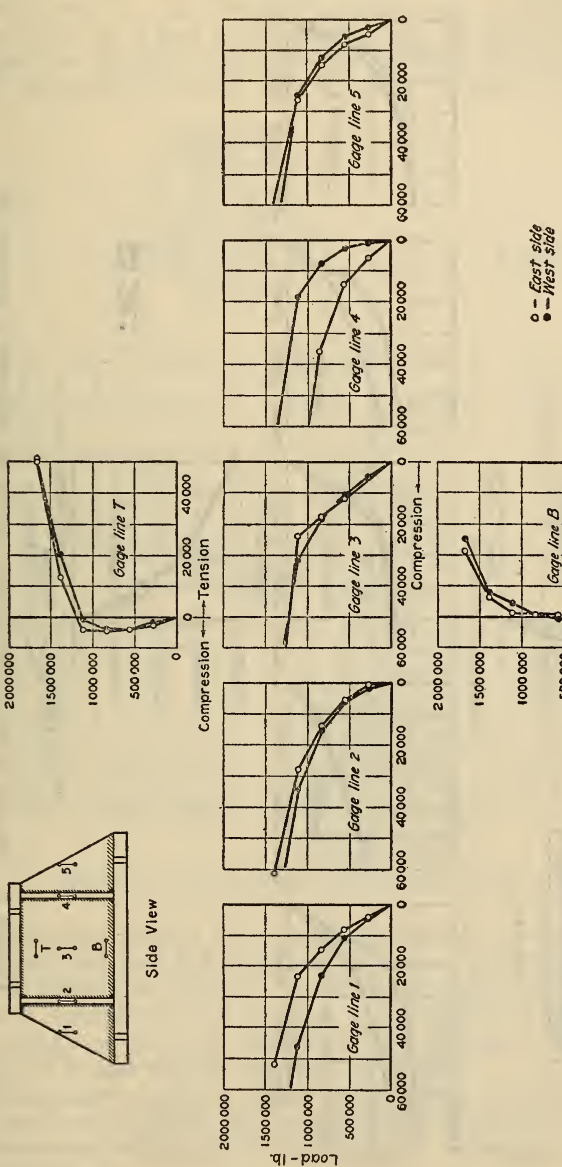

FIGURE 2.—*Stresses in the inclined plates and in the stiffeners of pedestal F1*

On the east and west sides, the five gage lines, 1 to 5, were at mid height between the top and bottom plates. Line *E1* was directly opposite *W1*, and they were near the north side of the pedestal. Other lines having corresponding numbers were also opposite each other. Lines 1, 3, and 5 were on the inclined plates and lines 2 and 4 on the outer surface of the stiffener plates. The horizontal gage lines E–T and W–T were near the top, and E–B and W–B were near the bottom of the inclined plates at mid length.

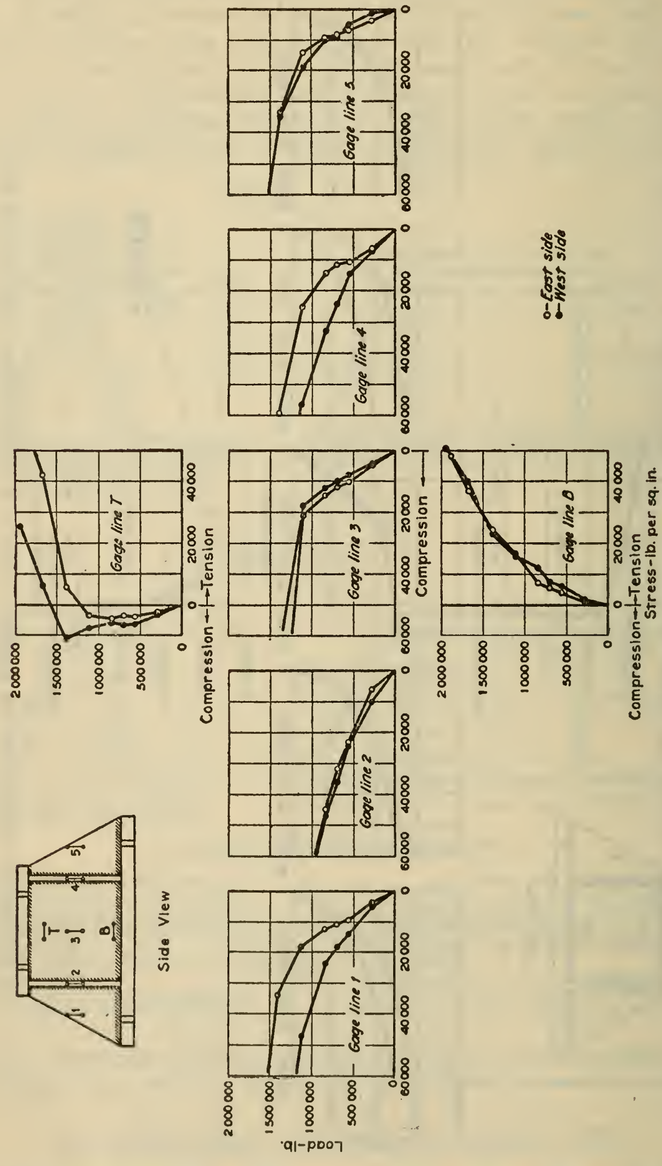

FIGURE 3.—*Stresses in the inclined plates and in the stiffeners of pedestal F2*

Three horizontal gage lines were located on both the north and south edges of the base plates, one-eighth inch below the top surface of this plate. Gage lines N–A and S–A were directly opposite each other, as were the remaining lines having similar designations.

IV. TESTING PROCEDURE

The pedestals were tested in compression in the 10,000,000-pound capacity testing machine at the bureau. In order to simulate the conditions under which bridge pedestals are used, these specimens were supported in the testing machine on concrete blocks, 3 feet 6 inches long by 2 feet 8 inches wide by 18 inches high, which had been aged one month. The distance between the base of the pedestal and the edge of the block was 6 inches at the sides and the ends.

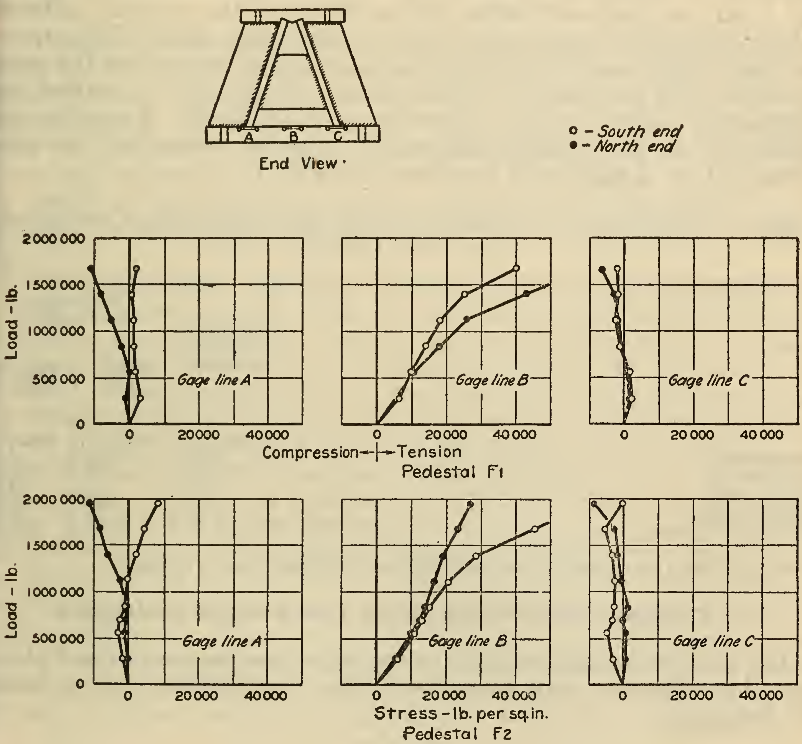

FIGURE 4.—*Stresses in the lower plates of the pedestals*

These concrete blocks were centered on the lower platen of the testing machine and set in a bed of plaster of Paris. The pedestal was then placed on top of the block in another bed of plaster of Paris and loaded through a steel bar 3 inches wide and having a length equal to the top length of the pedestal. Figure 5 shows pedestal *F2* in the testing machine after test.

A Whittemore fulcrum plate strain gage having a 2-inch gage length was used. This gage was provided with a Last Word dial, each division of which was 1/10,000 inch. Readings were estimated to 1/10 division.

V. RESULTS AND DISCUSSION OF THE TESTS

1. STRESSES COMPUTED FROM THE LOAD

The compressive stress in the pedestals at mid height for each load was computed by resolving the load into two components acting parallel to the inclined plates.

Each component was, then, divided by the cross-sectional area of the inclined plate and the two outside stiffeners on one side of the pedestal to obtain the stress. The nominal dimensions (given in fig. 1) of the inclined plates (*P2* or *P6*) and the outside stiffeners (*P3*) were used in computing the cross-sectional area. The area of the inside stiffeners (*P4*) was not included when computing the cross-sectional area because it did not seem probable that it carried any appreciable portion of the components of the load. A section perpendicular to the inclined plates was used in computing the area. These values of the stress are given in Table 1.

TABLE 1.—*Comparison of calculated stress from load with weighted average stress from strain readings*

Load	Calculated stress at mid height	Stress from strain readings	
		Pedestal F1	Pedestal F2
	Lbs./in.²	*Lbs./in.²*	*Lbs./in.²*
280,000 pounds	5, 790	3, 900	4, 500
560,000 pounds	11, 580	8, 800	11, 000
700,000 pounds	14, 475	----------	14, 400
840,000 pounds	17, 370	17, 300	18, 000
1,120,000 pounds	23, 160	[1] 30, 000	[1] 30, 200

[1] Probably above the proportional limit and therefore an incorrect value for the stress.

2. STRESSES COMPUTED FROM THE STRAIN READINGS

The stresses in the pedestals at the gage lines on the inclined plates and the stiffeners were calculated from the deformation, by using the formula:

$$S=\frac{e}{l}E$$

where

S = stress, lbs./in.².

e = deformation, inch, indicated by the gage readings.

l = gage length, 2 inches.

and

E = modulus of elasticity of the steel, assumed to be 30,000,000 lbs./in.².

As no coupon specimens of the material in these pedestals were furnished, the interpretation of the strain readings is somewhat unsatisfactory. This formula does not apply for values of the stress above the proportional limit. As the proportional limits are not known for the plates in the pedestals, it is impossible to determine whether the values of the stresses should be computed for the strains at the higher loads.

The calculation of the stresses in the other gage lines does not appear to be practicable for a structure as complicated as these pedestals.

Figures 2, 3, and 4 show curves which have been plotted with the load as ordinates and with the stresses in the gage lines, computed from the strain readings, as abscissas. The tensile stresses are plotted to the right, and the compressive stresses to the left of the vertical axis of each diagram.

(a) STRESSES AT MID HEIGHT

A comparison of the average stresses at mid height in the inclined plates and in the stiffeners can be made from the values given in Table 2. For pedestal F1, having large welds at the base of the inclined plates, the stresses in the inclined plates were practically equal to those in the stiffener plates, showing that in this design the stress was approximately uniform in the compressive members of the pedestal. The stresses in the stiffeners of pedestal F2 were, however, for all loads, more than twice the stresses in the inclined plates.

It is evident that the load was distributed more uniformly in pedestal F1, having closed single bevel T joints[1] between the inclined plates and the lower plate, than in pedestal F2, having the inclined plates machined to fit the lower plate and secured by fillet welds on one side. It seems probable that in other welded steel structures the one having closed single bevel T joints would be more satisfactory as well as cheaper than one in which the members were machined to fit and held in position by a fillet weld on one side.

To obtain an average compressive stress at mid height from the values computed from the strain-gage readings, the stresses given in Table 2 were used. The stress in the inclined plates for a given load was multiplied by the ratio of the cross-sectional area of the inclined plates to the cross-sectional area of both the inclined plates and the outside stiffeners. The stress in the stiffeners was multiplied by the ratio of the cross-sectional area of the outside stiffeners to the cross-sectional area of both the inclined plates and the outside stiffeners. The sum of these results is given in Table 1 for both pedestals.

TABLE 2.—*Comparison of average stress at mid height from strain readings in inclined plates and in stiffeners*

Load	Average stress from strain readings			
	Pedestal F1		Pedestal F2	
	Inclined plates	Stiffeners	Inclined plates	Stiffeners
	Lbs./in.²	*Lbs./in.²*	*Lbs./in.²*	*Lbs./in.²*
280,000 pounds	4,350	2,100	3,750	7,650
560,000 pounds	9,250	7,200	9,100	18,400
700,000 pounds			11,400	26,100
840,000 pounds	17,100	18,300	13,650	35,100
1,120,000 pounds	29,400	32,700	22,800	[a] 59,700

[a] Probably above the proportional limit and therefore an incorrect value for the stress.

[1] Welding and Cutting, Nomenclature, Definitions, and Symbols, Nov. 19, 1929, American Welding Society, 33 West Thirty-ninth Street, New York, N. Y., paragraph 460, 27.

The log sheets, Tables 3 and 4, show that the inclined plate *P2* for pedestal *F1* scaled at 840,000 pounds and the inclined plate *P6* of pedestal *F2* at a higher load (1,120,000 pounds). Scaling is an indication that the stress in the steel plate has exceeded the yielding point of the material.

As, however, the stress (17,370 lbs./in.^{2}) for the pedestal *F1* under a load of 840,000 pounds agrees very closely with the other values for this load in Table 1, it is probable that the amount of material in pedestal *F1* which had reached the yield point was so small that the error in this value is not appreciable.

TABLE 3.—*Log sheet for test of pedestal F1*

Load (in pounds)	Remarks
0	Strain readings taken.
280,000	Do.
560,000	Strain readings taken, some cracks heard.
840,000	Strain readings taken. Scaling on inside surface of *P2*. Scaling on outside surface of *P2* south of *E–T* at upper part of plate.
1,120,000	Strain readings taken.
1,400,000	Do.
1,680,000	Strain readings taken on some.
2,000,000	Top plate visibly bowing down at mid length.
2,119,000	Weld along stiffener scaling.
2,315,000	Plates *P2* visibly buckling. Also stiffener plates *P3*.
2,322,000	Maximum load. Welds on diaphragm *P4* broken at top.

TABLE 4.—*Log sheet for test of pedestal F2*

Load (in pounds)	Remarks
0	Strain readings taken.
280,000	Do.
560,000	Do.
700,000	Do.
840,000	Do.
1,120,000	Strain readings taken. Scaling on top east plate *P6*.
1,400,000	Strain readings taken.
1,680,000	Do.
1,960,000	Do.
2,156,000	Visible buckle in stiffener *P3*.
2,260,000	Top plate bowing down at center.
2,423,000	Maximum load. Welds on diaphragms *P4* broken.

The stresses computed from the strain-gage readings for the 1,120,000 pounds load, however, are, as was to be expected, much greater than the stress computed from the load.

The stresses (Table 1) in pedestal *F1* computed from the strain-gage readings are somewhat lower than for pedestal *F2* which was, in all probabity, due to the more uniform stress distribution in pedestal *F1*.

For pedestal *F2* the stresses from strain-gage reading agree very closely with the stresses computed from the load.

If it is assumed that the stresses computed from the load approximate the actual stresses, the agreement of the stresses from the strain-gage reading with these values is remarkable. It should be remembered that these pedestals were fabricated from many comparatively small pieces and that the stress distribution is far from simple. It is evident that the strain gage is of value in studying the stress distri-

FIGURE 5.—*Pedestal F2 in the 10,000,000-pound capacity testing machine after test*

FIGURE 6.—*Side view of the pedestals after test*

bution in structures of this kind for which the usual methods of computing the stresses in engineering structures do not give sufficiently accurate results.

(b) STRESSES NEAR TOP AND BOTTOM OF THE INCLINED PLATES

The stresses in the horizontal gage lines, *T*, near the top and *B*, near the bottom of the inclined plates are shown in Figures 2 and 3. Low loads appeared to induce compressive stresses in the top gage lines, but higher loads caused tensile stresses which rapidly increased beyond the proportional limit of the material. A similar tendency was shown by the bottom gage lines, but the compressive stresses were much less, and tensile stresses occurred at lower loads than was the case for the top of the plate.

(c) STRESSES IN THE LOWER PLATE

Figure 4 shows the stresses in the lower plate of the pedestal. The stresses were low in the gage lines *A* and *C*, which were directly below the inclined plates. Those in gage line *B*, at the center, were comparatively large tensile stresses, induced probably by the horizontal components of the forces exerted by the inclined plates and by bending of the lower plate.

3. MAXIMUM LOAD AND DESCRIPTION OF FAILURE

Tables 3 and 4 are copies of the log sheets for the tests. The maximum loads carried by the two pedestals, notwithstanding the differences in the stress distributions, were very nearly equal. The factor of safety for the design load of 350,000 pounds was 6.6 for pedestal *F1* and 6.9 for pedestal *F2*.

Figures 5 and 6 show that, at failure, the top plate had bowed downward at mid length, the stiffener plates were buckled, and the inclined plates were deformed so that the welds between them and the inside stiffener plates, *P4* (fig. 1), were broken. The base plate was not visibly deformed, although the strain readings represented in Figure 4 showed that the material had been highly stressed at least along the longitudinal center line at the top of the plate.

No strain measurements were taken on the top plate, and consequently the load necessary to cause dangerously high stresses in that plate are unknown.

VI. CONCLUSIONS

1. The compressive tests of two electric arc welded steel pedestals for bridges, designed for a load of 350,000 pounds showed that the factor of safety was more than six.

2. The stress distribution in the inclined plates (subjected to compressive forces) was more uniform in the pedestal having closed single bevel T joints between the inclined plates and the lower (base) plate, than in the pedestal in which the members were machined to fit and held in position by a fillet weld on one side.

3. The compressive stresses in the sides of the pedestal computed from the strain-gage readings were nearly the same as the stresses computed from the loads on the pedestals.

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