STRAIN MEASUREMENT IN THE REINFORCEMENT FOR THE DOME OF THE NATURAL HISTORY BUILDING

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ABSTRACT

Slight outward movement of the brick piers supporting the dome over the rotunda of the Natural History Building of the National Museum caused cracks along the joints in the masonry and in the stone arches under them. To reinforce the structure the pendentives were surrounded by a framework of structural steel. Inwardly directed forces were exerted on the masonry by many screw jacks, to prevent further movement.

To control the jacking operation, readings were taken on a large number of gage lines on the frame work, using a Whittemore strain gage. The stress in the framework was computed from these readings. Approximately the desired stress in each member was obtained.

The safety of the dome and its supporting structure was assured.

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I. INTRODUCTION

The Natural History Building of the United States National Museum,¹ one of the group of buildings of the Smithsonian Institution in Washington, D. C., was built during the period 1904 to 1911. It covers an area of 168,752 square feet, including 32,581 square feet in two interior courts, the principal length, east to west, being 561 feet and from north to south 364 feet. In style it is of modern classic architecture of monumental character executed in granite.

The commanding architectural feature of the building is a pavilion 118 feet square at the center of the south range inclosing an open

¹ Description taken from Bulletin 80, National Museum Smithsonian Institution, by Richard Rathbun, 1913.

rotunda, $81\frac{1}{2}$ feet in diameter, surmounted by ceiling and roof domes 125 and 142 feet above the main floor. On the exterior this central feature is accentuated by a Roman Corinthian portico, the order of which extends through three stories, a height of 56 feet. Behind the portico the main wall rises to a height of 102 feet, or within 14 feet of the level of the base of the roof dome over the rotunda.

The rotunda through three stories ($62\frac{1}{2}$ feet) is of irregular octagonal plan. The sides parallel to the principal axes are 47 feet long and those in the diagonal $16\frac{1}{2}$ feet. The longer sides have arched openings 38 feet wide extending through four stories, leaving the massive masonry piers on the four diagonals to support the upper part and the roof. Two of these piers are shown in Figure 1. Pendentives of corbeled brick masonry merge the octagonal section into a 72-foot diameter circular drum above the arched openings, thus bringing the inside face of the drum $5\frac{1}{4}$ feet inside of the inside face of the piers on which the load is supported. This inside diameter of the drum is maintained for a height of some $8\frac{1}{4}$ feet above the arches, and above which the diameter is increased to 78 feet, the thickness of the wall above this offset being 3 feet 9 inches. Just below this offset in the wall thickness the Guastavino ceiling dome rests on a ledge corbeled from the inside face of the drum. The main or roof dome is also of Guastavino construction and rests on the top of the drum near its outside.

II. CONDITION OF THE DOME SUPPORTS

A few years after the building was occupied, opening of the joints at the keystone of the semicircular arches in the rotunda was noticed. Observations made on the progress of this movement over a period of 10 years indicated continued progress. The keystone in the east arch which had moved more than the others is shown in Figures 3 and 4.

A section of one of the pendentives between the fourth floor and the domes is shown in Figure 2. The estimated weight of the steel-banded main dome and its supports above the piers is about 6,000,000 pounds. If this weight is assumed to be equally distributed to the four piers, the action lines are vertical and 74 inches inside the center line of the pendentive at the thinnest section. The outside of the main dome is, in fact, 18 inches inside of the inner surface at this place. This dome and its supporting structure would, therefore, under the worst condition exert an eccentric compressive force of about 1,500,000 pounds on each of the piers. The bending stresses were increased by the smaller, but more eccentric, resultant forces exerted by the weight of the ceiling dome.

It is probable that these forces were sufficiently eccentric to cause tension in the outer surfaces of the pendentives. Cracks (see fig. 5) which were thought to indicate tensile stresses, were noticed in the mortar joints on these surfaces.

The reinforcing of the dome supports was planned by the staff of the Office of the Supervising Architect, Treasury Department.

W. C. Lyon, a member of the structural division of the staff, was in direct charge of the work which was performed by contractors.

FIGURE 1.—*The rotunda*
Note the piers and the arches supporting the dome.

FIGURE 3.—*Lower part of the keystone in the east arch*

The displacement of the keystone with relation to the adjacent voussoirs is quite evident. The displacement was caused by a slight outward movement of the piers supporting the dome over the rotunda.

FIGURE 2.—Section through brick pendentive and drum

III. REINFORCING THE DOME SUPPORTS

1. STEEL FRAMEWORK

To prevent further movement of the piers, a riveted steel framework was designed to surround the structure as shown in Figures 2 and 6. Six vertical steel I-beams were placed back of, and above, each pier and supported at their upper and lower ends by horizontal built-up beams. The horizontal beams were held in place by connecting the ends of adjacent beams by means of tension members so as to form two steel frames or "rings," one at the upper end of the vertical beams and the other at the lower end. The

FIGURE 6.—Isometric view of steel framework

tension members in the lower "ring" consisted of two steel plates, while those in the upper "ring" consisted of two channels placed back to back. The tension members had an over-all length of about 58 feet and were spliced near the middle.

2. JACKS

Eight screw jacks were placed between each beam and the wall. The base of each jack was bolted to the flange of the beam and the bearing plate was bedded in plaster of Paris against the outer surface of the masonry.

The arrangement of the jacks is shown in Figures 2 and 7. By rotating the jackscrews, inwardly directed forces could be exerted to prevent further outward movement. No attempt was made to bring the piers back to their original position. To apply the

FIGURE 4.—*Upper part of the keystone in the east arch*

The displacement of the keystone with relation to the adjacent voussoirs is quite evident. The displacement was caused by a slight outward movement of the piers supporting the dome over the rotunda.

FIGURE 5.—*Tensile crack in mortar joint of brick pendentive supporting the dome of the rotunda*

forces exerted by the jacks slowly and as nearly uniformly as possible above each pier, the jacking operation schedule shown in Table 1 was followed. This operation was started on January 3, 1929, and completed February 11, 1929.

TABLE 1.—*Jacking schedule*
(The location of each tier of jacks is shown in fig. 2)

Working day	Jackscrews turned 30°	Working day	Jackscrews turned 30°	Working day	Jackscrews turned 30°
	<i>Tiers</i>		<i>Tiers</i>		<i>Tiers</i>
1.....	2 3 1 4	11.....	6 7 2 3	21.....	5 6 7 8
2.....	2 3 5 2	12.....	1 4 5 6	22.....	1 3 2 4
3.....	3 1 4 2	13.....	7 8 2 3	23.....	5 6 7 8
4.....	3 6 2 3	14.....	1 4 2 3	24.....	1 3 2 4
5.....	1 4 5 7	15.....	1 4 2 3	25.....	5 6 7 8
6.....	2 3 1 4	16.....	5 6 7 8	26.....	1 3 2 4
7.....	5 6 8 2	17.....	2 3 3 4	27.....	5 6 7 8
8.....	3 1 4 5	18.....	1 3 2 4		
9.....	6 7 8 2	19.....	5 6 7 8		
10.....	3 1 4 5	20.....	1 3 2 4		

The pitch of the screw was one-quarter inch so the jack advanced one-forty-eighth inch for each 30° turn of the screw. As it was impracticable to turn all of the 24 jackscrews in a tier simultaneously, the jackscrews in the given tier on Beam III for each of the four piers were turned at a given signal. Ten minutes later, the jackscrews on Beam IV were turned. Similarly, the remaining jacks in the tier were turned in sequence II, V, VI, I until at the end of an hour each of the 24 jacks in the tier had been turned 30°.

On the first working day all the jackscrews in tiers 2, 3, 1, and 4 were turned as listed in Table 1. Four hours were required. The jacks were not moved again until the second working day when all the jackscrews in tiers 2, 3, 5, and 2 were turned in four hours. The schedule was completed in 27 working days. The forces exerted by the jacks were increased very slowly to allow time for adjustments in the masonry and framework.

The schedule was so arranged that the greater forces were exerted by the lower tiers of jacks where computation showed the maximum tensile stresses in the masonry. To exert the desired restraint, it was estimated that a tensile stress of about 14,000 lb./in.² would not be exceeded at any point in the framework.

During the latter part of the jacking operation, some modifications in the schedule were made to obtain approximately the same stress in gage lines similarly located with respect to the piers. The stresses computed from the strain-gage readings were used as a guide in making these modifications.

IV. STRAIN MEASUREMENTS

1. STRAIN GAGE

As the force exerted by a screwjack can not be determined satisfactorily by measuring the torque applied to the screw, the stresses in the framework were computed from strain-gage readings, obtained by members of the staff of the National Bureau of Standards.

The Whittemore fulcrum plate strain gage shown in Figure 8 was used on gage lines having a length of 10 inches. This instrument

was designed for the Committee on Arch Dam Investigation, at the request of the Engineering Foundation.² A geared dial micrometer, graduated to 0.0001 inch and having a range of 0.1 inch, is usually used on this instrument. For this investigation, however, it was found that with care the holes could be drilled so that the maximum difference in the lengths of the gage lines did not exceed 0.01 inch. This close control of the gage length permitted the use of a Last Word dial micrometer graduated to 0.0001 inch and having a range of 0.024 inch. The error of this dial did not exceed one division, and its use was, therefore, preferred. Stresses up to about 20,000 lbs./in.² in either tension or compression could, therefore, be measured.

2. METHOD OF READING STRAINS

The temperature of the framework inside the building was so nearly constant during these tests that it was not considered necessary to correct the readings of the strain gage for differences of temperature.

Although the reading of the instrument depends somewhat upon the skill of the observer, it was found that the errors in these readings in all probability did not exceed 0.00016 inch, equivalent to 500 lbs./in.².

The stresses in the framework caused by assembling and by riveting the members were neglected, as they were probably low. The work would have been delayed if readings had been taken on the members resting horizontally on a continuous support before assembly. The gage holes (0.043 inch diameter) were, therefore, drilled after the framework had been completely erected. The initial strain-gage readings were taken with the jacks tightened by turning the screw with the hand.

The strain gage was applied to the standard bar (mild steel similar to the framework) and then in succession to several gage lines on the framework and the readings recorded. Readings were made by estimation to the nearest one-tenth of one division of the dial (0.00001 inch). This procedure was then repeated on the same gage lines. Additional readings were taken on any gage line for which the difference between the two readings was one division of the dial (0.0001 inch) until two consecutive readings did not differ as much as one division.

Figure 9 shows an observer taking strain-gage readings on the lower "ring." Four observers, each provided with a strain gage, obtained readings simultaneously.

Before the jacking schedule for the day was started, readings were taken on all gage lines. After the jackscrews in each tier had been turned, readings were taken on the lower gage lines on the vertical beams. Upon the completion of the jacking schedule for the day, readings were again taken on all gage lines.

3. LOCATION OF GAGE LINES

The gage lines were laid out on the framework in the positions shown in Figure 6. Those adjacent to each pier were numbered consecutively from 1 to 22; 1 to 5 being on the lower "ring," 6 to 17 on the beams, and 18 to 22 on the upper "ring." The total number of gage lines was 88.

²American Society of Civil Engineers, Papers and Discussions, Engineering Foundation Committee on Arch Dam Investigation, Arch Dam Investigation, 1, p. 64; November, 1927.

FIGURE 7.—Screw jack in place between the vertical beams and the masonry

FIGURE 8.—Whittemore strain gage having a 10-inch gage length

V. STRESSES IN THE FRAMEWORK

1. METHOD OF COMPUTING STRESSES

The stresses were computed from the strain-gage readings using the formula

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

in which

S = stress, lbs./in.².

L = gage length, 10 inches.

E = Young's modulus of elasticity for steel, assumed to be 29,000,000 lbs./in.².

e = the difference in inches between the initial reading and the reading after forces had been exerted on the framework.

A difference of one division on the dial (0.0001 inch) was, therefore, equivalent to a change in stress of about 290 lbs./in.² in the framework.

As the temperature was practically constant, the reading of the standard bar served only to show that the strain gage was operating satisfactorily and that the zero on the dial had not been displaced accidentally.

2. STRESSES IN THE VERTICAL BEAMS

The changes in the stress in the vertical beams from day to day caused by the jacking are shown graphically in Figure 10. The ordinates of the graphs represent tensile stress, while the abscissas represent the number of calendar days elapsed since the beginning of the jacking operations. The location of each gage line is shown in Figure 6.

The stresses in the outer beams (6 and 11 near the edges of the pier) are, in most cases, higher than in the inner beams (8 and 9). On the same day there was not a great deal of difference between the stresses in beams similarly placed with respect to the piers. The highest observed stress was about 7,000 lbs./in.², indicating that the factor of safety for these beams was very high.

3. STRESSES IN THE "RINGS"

(a) BEHIND PIERS

The stresses in the "rings" behind the piers are shown in Figure 11, using the same method of presentation as for the beams. The stresses are about the same behind each pier showing that the final adjustment of the jacks produced the desired result.

The stresses in the upper "ring" were higher than in the lower "ring," approaching 14,000 lbs./in.² in many of the beams. These are the highest observed stresses in the framework. They are, however, considerably lower than the stresses usually allowed in steel structures and undoubtedly have an adequate factor of safety.

For designing the framework it was assumed that the resultant of the forces exerted by the jacks on a beam would act at about one quarter of the distance between the two "rings" above the lower "ring." The reaction at the lower end of a beam would then be about three times the reaction at the upper end.

FIGURE 10.—Stresses in the vertical beams

FIGURE 9.—An operator taking strain readings on the lower tension "ring"

The lower "ring" was, therefore, designed to have a section modulus behind the piers about three times that of the upper "ring." (Lower "ring" 1,100 in.³, upper "ring" 350 in.³.) On the basis of these assumptions, the stresses in the gage lines on the "rings" behind the piers (4, 5, 18, and 19, fig. 6) would be expected to be about

FIGURE 11.—Stresses in the tension "ring" behind the piers

the same. The higher stresses in the upper "ring" indicate that the action line of the resultant of the forces exerted by the jacks was considerably higher than was assumed.

(b) BETWEEN PIERS

The stresses in the "rings" between the piers are shown graphically in Figure 12. They are for all gage lines very low. Except in two gage lines (A-1, B-1) they do not exceed 5,000 lbs./in.²

As the two plates (5/8 by 30 inches in section) forming each of the tensile members in the lower "ring" between the piers were riveted to the horizontal beams forming the members behind the piers, the deflection of the members behind the piers caused by the jacks resulted in eccentric loading of these tensile members. This is the

FIGURE 12.—Stresses in the tension "ring" between the piers

reason the stresses in gage lines 1 are much greater than those in gage lines 2 and 3.

On the upper tensile member the stresses in gage lines 22 (fig. 12) were smaller than in gage lines 20 and 21. Gage line 22 is on the edge of the flange of the lower channel and gage lines 20 and 21 are

on the web of the upper channel. The length of these members (about 58 feet) made it necessary to splice them. This was done by inserting a plate between the two channels which were back to back and riveting it to the webs. Near the splices, higher stresses in the webs than in the flanges were to be expected. Gage lines 20, 21, and 22 were about 5 feet from a splice.

FIGURE 13.—Deflections of the vertical beams

VI. DEFLECTIONS

For measuring the bending deflection of the vertical beams under the forces exerted by the jacks, small steel wires were stretched between pins in the webs at tiers 1 and 8. A mirror and a scale graduated to 0.1 inch were attached to the web back of the wire at the elevation of tier 4 as shown in Figure 2. The bending deflections are shown graphically in Figure 13.

Noting that in these graphs the outer beams are near the top and bottom of the page, it is evident that the deflections of the outer beams were considerably greater than those of the inner beams. By

comparing these graphs with those shown in Figure 10 it is seen that the beams which had the greatest deflection had the greatest tensile stress and that roughly the deflections are proportional to the stresses.

VII. CONCLUSIONS

1. The conclusion was reached, when planning the reinforcement of the supporting structure for the dome of the Natural History Building of the National Museum, that the hand strain gage was the only instrument which it was practicable to use to determine the stresses in the steel framework.

2. The engineers of the office of the Supervising Architect were enabled by using the hand strain gage to direct and control accurately the jacking operations when reenforcing the supporting structure for the dome of the Natural History Building of the National Museum.

3. When the jacking operation was completed, the stresses in the steel framework surrounding the supporting structure of the dome of the Natural History Building of the National Museum were in satisfactory agreement with those deemed necessary to insure the safety of the structure.

WASHINGTON, August 25, 1930.