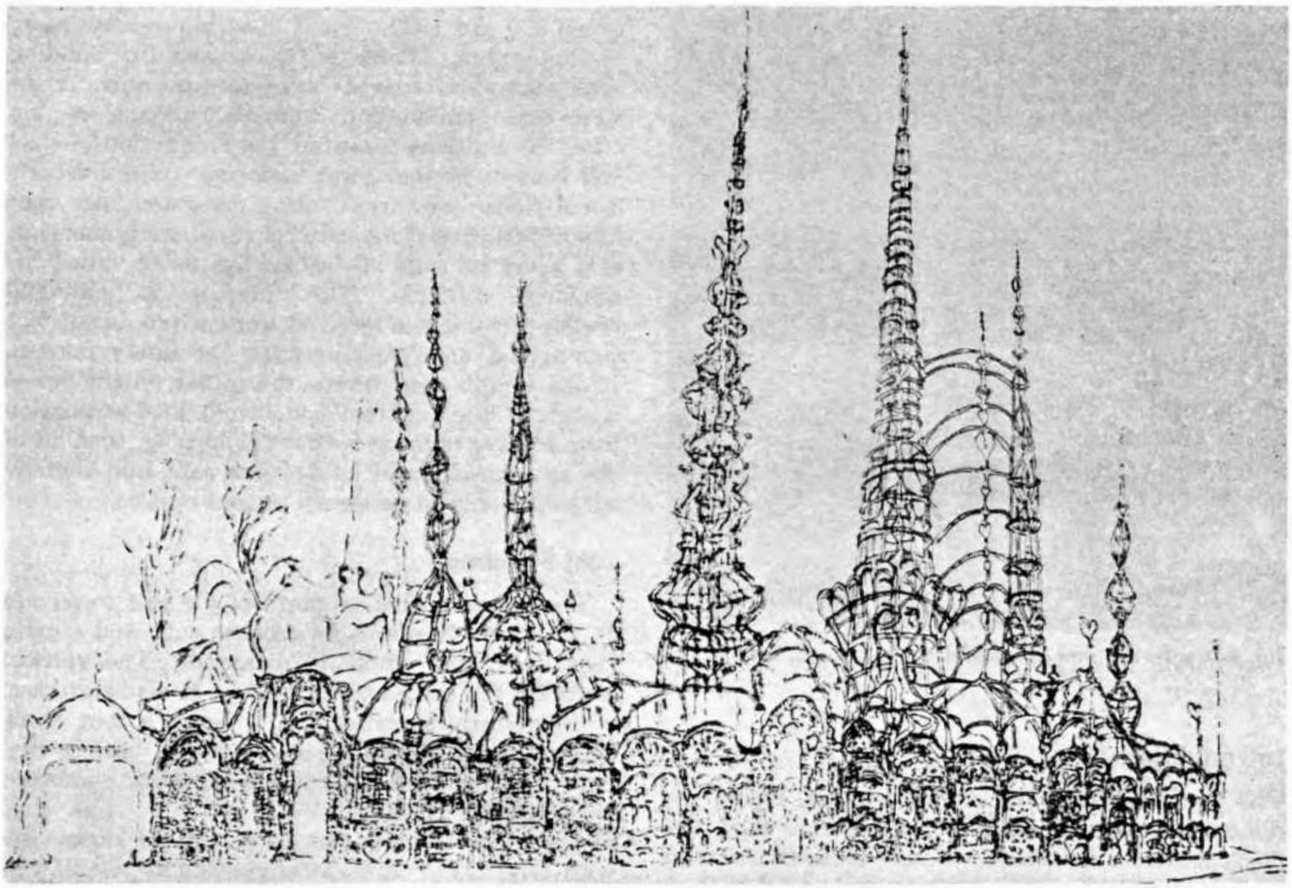

Fig. 1—Artist's rendering of seven towers at Watts, Calif.

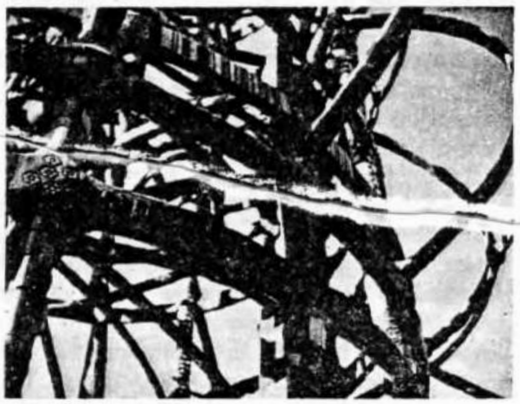

Fig. 2—Structural details of 99-ft test tower

Structural Test of Hand-built Tower

Problem of conducting potentially damaging load test on world-famous art monument is solved by using unorthodox testing procedure

by N. J. Goldstone

ABSTRACT—An unusual application of load-testing principles was undertaken by the author in gaging the strength of an internationally acclaimed work of art. The architectural structure was a 99-ft steel and cement tower, the tallest of seven similar towers hand-crafted by Simon Rodia more than 40 years ago in the Los Angeles district of Watts, Calif. Load testing was performed by author in 1959 to determine whether the towers met local building codes and were unhazardous to adjacent dwellings. This paper describes author's test specimen, load conditions, instrumentation, test procedure and results. Background of the monument is also offered.

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List of Symbols

(Los Angeles Building Code)

A_s = area of vertical steel reinforcement, sq. in.
A_g = area of over-all cross section, sq. in.
d = least lateral dimension of column, in.
f_c' = ultimate compression stress, psi
f_c = allowable compression stress, psi
f_s' = ultimate tensile stress in steel, psi
P = allowable short-column load, lb.
P' = allowable long-column load, lb.
p = allowable column stress, psi
h, L = distance between lateral support, column length, in.
r = least radius of gyration, in.

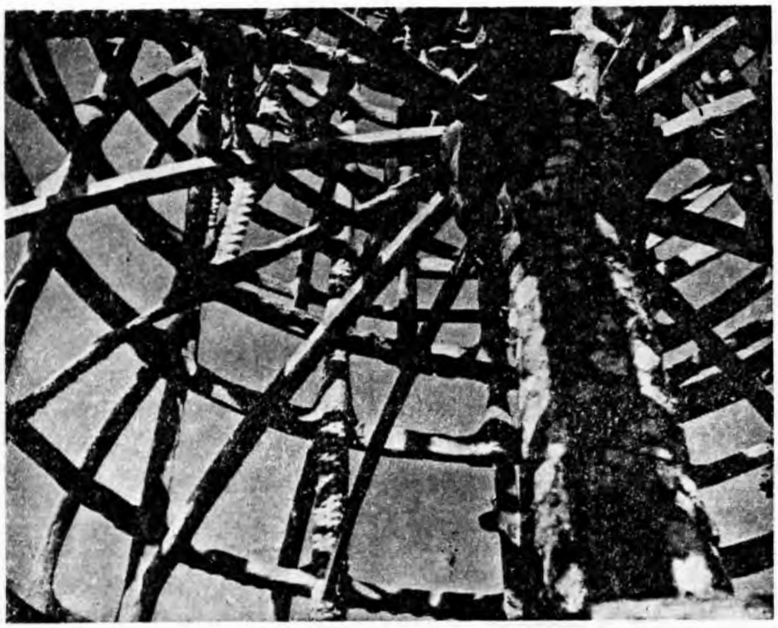


Fig. 3—Center column, rings and spokes of test tower

Introduction

The test specimen was an open-frame, highly redundant structure fabricated of cement, steel and chicken wire. Its members were embellished with broken glass and mosaic crockery embedded in their cement coating (see Figs. 1, 2 and 3).

Contrary to approved testing practice, the specimen rather than procedure was the primary consideration, since tower failure under testing would brand the attempt as artistic vandalism in the eyes of art lovers the world over.*

Two safeguards were employed in the test jig to guarantee control over the test specimen's maximum load; a short-travel hydraulic cylinder was selected to prevent excessive tower deflection (a 6-in. movement of the structure at 33 ft above ground level would bottom the cylinder, blocking further load application); and the 10-in. I-beam—the primary whiffletree member—was designed to yield, in bending, at 10,000 lb, preventing additional load application to the tower.

The test specimen, along with three other towers on the site, had been declared dangerous to adjacent dwellings by the Los Angeles Department of Building and Safety. All four structures were subsequently ordered to be demolished. Results of the load test, however, reversed that decision.

Why the Load Test Was Conducted

In a municipal hearing, Los Angeles Building and Safety Department engineers presented two stress analyses indicating negative safety margins of between 500 and 1500 percent in certain critical parts of the towers. These analyses used the building code allowable stresses (refer to Appendix, 1) for steel and combination columns, making certain other assumptions regarding the interaction of steel and cement in the tower members. The author's calculations, however (also presented at that municipal hearing) indicated positive safety margins, also based on code allowables but using values for reinforced concrete. This difference in analytical results remained unresolved until a proof-load test was agreed upon to determine the safety margin. If one sample test tower—the tallest on the site—withstood the required code wind load-simulation force for five minutes without collapsing, then all of the structures would be declared safe, and thereby subjected only to necessary cement repairs.†

Test Specimen

The main vertical supports of the test tower are 16 triangular legs, $4^1/_2$ in. on each side, and a main central column, 12 in. in diameter. The vertical members are embedded in a 13 $^1/_2$-ft diameter base, with footings extending 14 in. below a 2-in. thick cement patio. The base is filled with broken concrete and is a shell-like cover 4-in. thick, reinforced with circular rings of $1^1/_2 \times 1^1/_2 \times {}^1/_8$-in. steel angles. The 16 main legs are reinforced alternately with $1^1/_2 \times 1^1/_2 \times {}^1/_8$-in. angles and $2^1/_2 \times 2^1/_2 \times {}^5/_{16}$-in. tees (see Fig. 4).

Spoke connections extend from the legs to hubs on the central column at 47 elevations. The 752 joints where the spokes and legs intersect are connected by the 47 rings which encircle the tower. Scores of large, semicircular segments connect alternate ring-leg joints, both vertically and horizontally, over the entire structure. Sea shells, pieces of glass and ceramics, and mosaic tiles cover many of the tower members (as shown in Figs. 2 and 3).

Load transfer through splices and joints in the steel and cement members is accomplished by cement bond between overlapping angles and tees, and by steel-mesh hardware cloth and chicken-wire embedded in the cement. There are no rivets, welds or bolts in the structure. (Refer to Appendix, 2 for discussion of this point.)

Structural details were obtained from observation, drill probes into six members and core sample tests performed by the Smith-Emery Laboratory in Los Angeles.

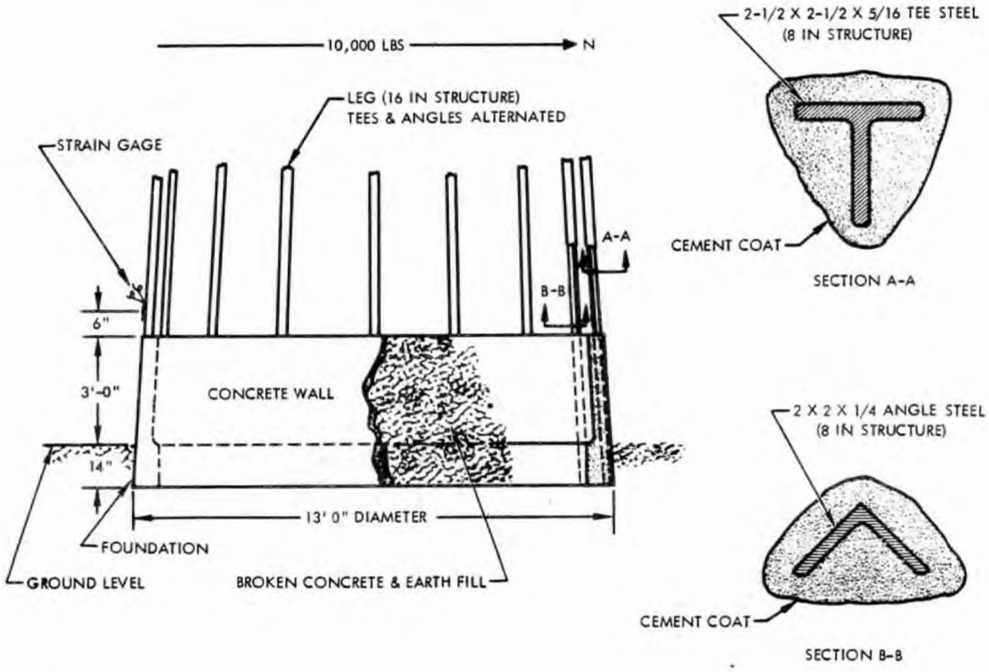
Test Condition

The 10,000-lb critical load at 33-ft elevation of the structure was based on the code wind pressure of 15 psi on the exposed surface area below 60-ft tower level and 20 psi above 60 ft. The pressures are computed using a drag coefficient of 1 for usual building shapes; however, most tower members have shapes with actual coefficients of smaller values. No re-

* *But recognition of Rodia's achievement came 16 years after the towers were completed. Rodia, an Italian immigrant who worked as a tile-setter, spent more than 30 years in erecting the towers by hand, using no equipment other than a window-washer's block-and-tackle. He worked without drawings.*

† *Based on their calculated negative margins, Building and Safety Department engineers anticipated failure of the tower during the test.*

Fig. 4—Base construction showing location of strain gages



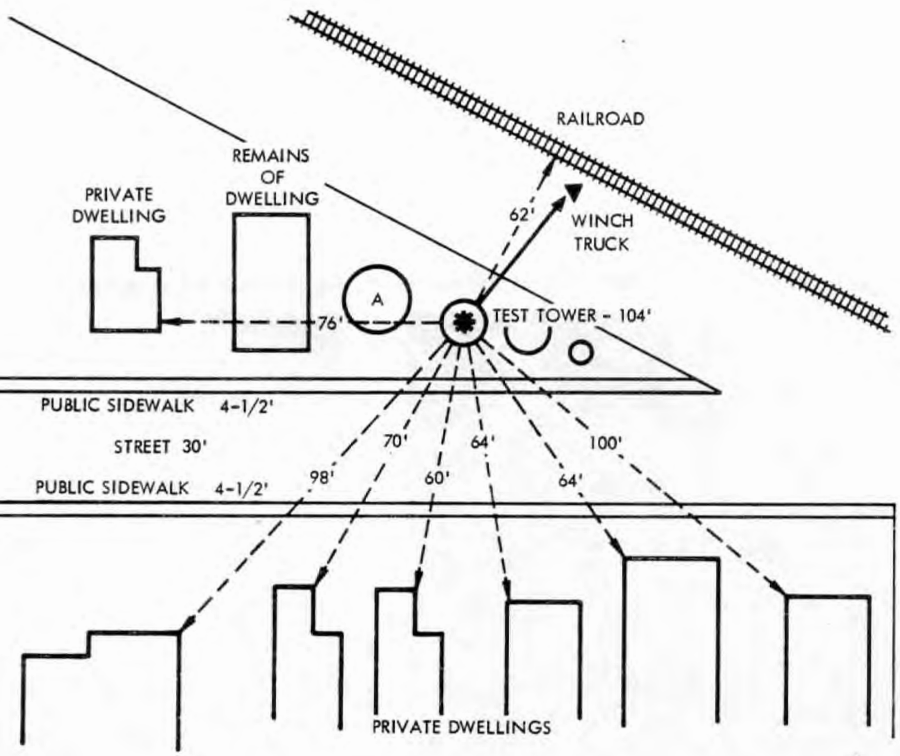


Fig. 5—Plot plan of test-tower site

duction in applied load was made for the drag coefficient difference, resulting in an extremely large, conservative test load.

Test Setup

Figure 5 illustrates the location of the test tower in relation to private dwellings in the general test area. The loading direction, therefore, was toward the railroad tracks and away from the dwellings to minimize the hazard of tower failure. A load and instrumentation control center was located at point A in Fig. 5.

The 10,000-lb load was applied by a hand-pump-actuated hydraulic jack, supported on a scaffold at the 33-ft elevation. The load distribution was accomplished through a three-stage whiffletree which applied the load to slings around the tower at four elevations (see Figure 6). The slings rested on 2 × 4's attached to the nine south tower legs. Synthetic rubber pads—located at leg-to-ring joints—carried the load into the tower joints. Load reaction was

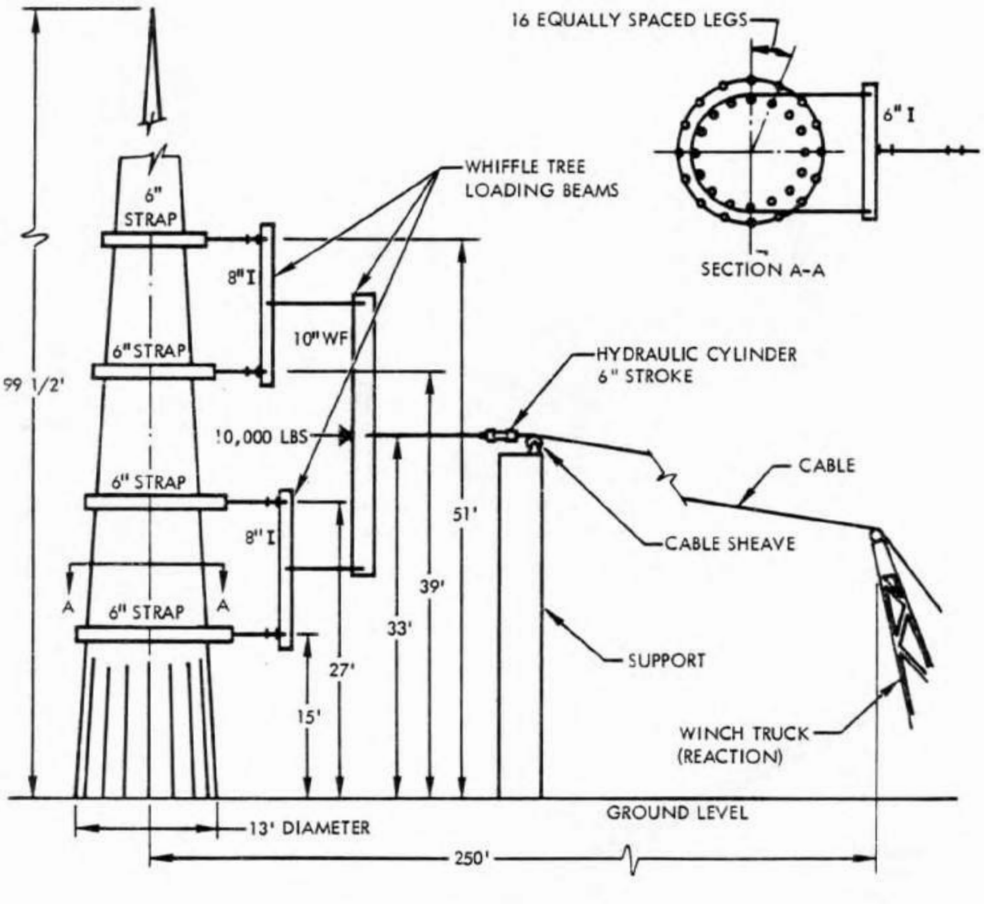


Fig. 6—Schematic of test setup

Fig. 7—Tower pads and 2 x 4's, southwest corner of test tower

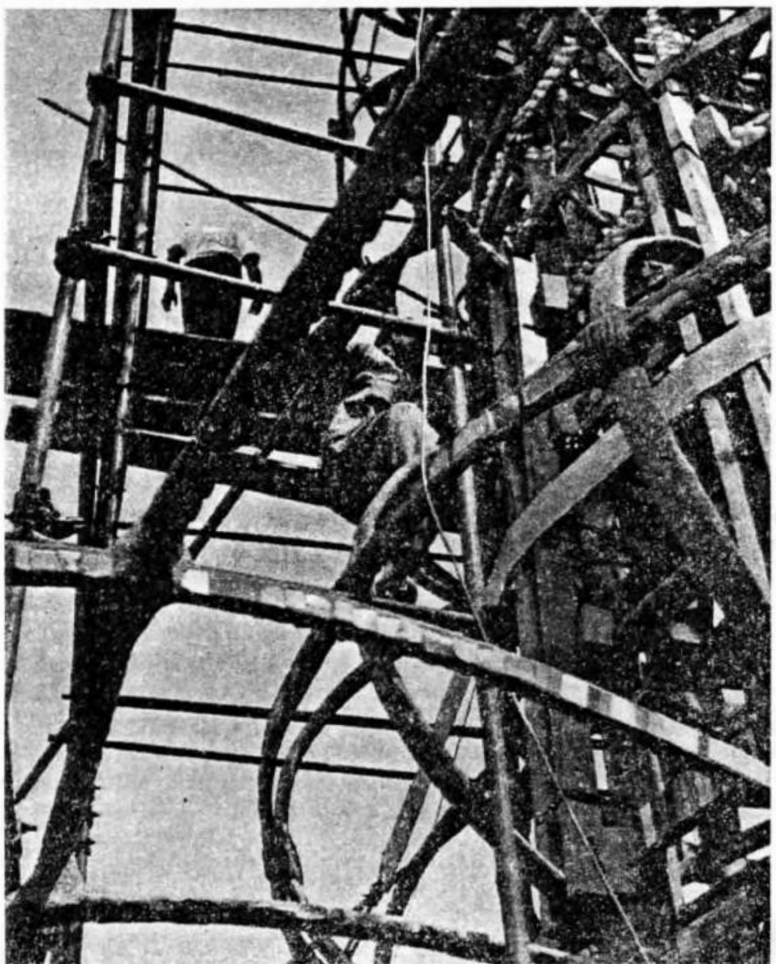
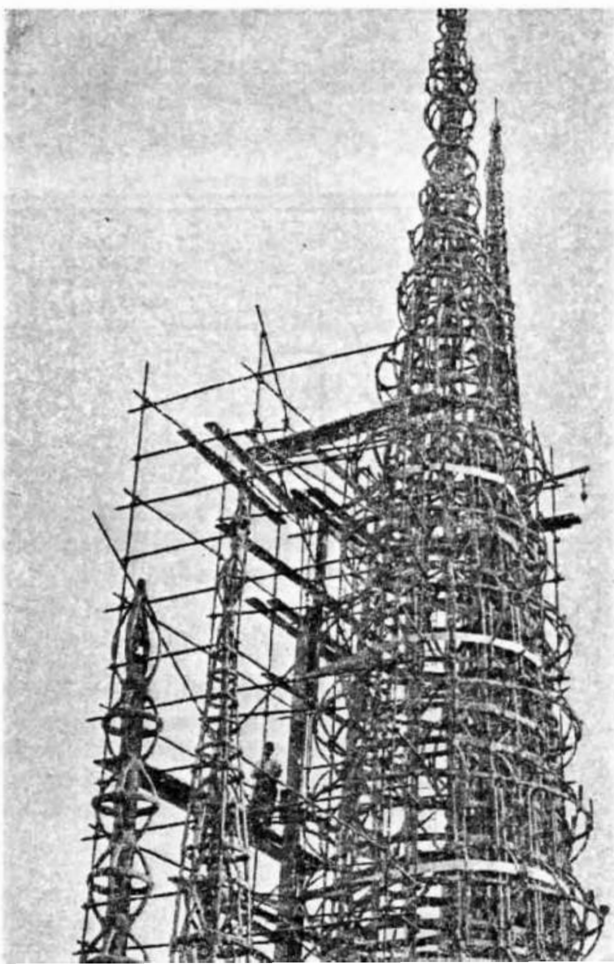


Fig. 8—Test setup and rig



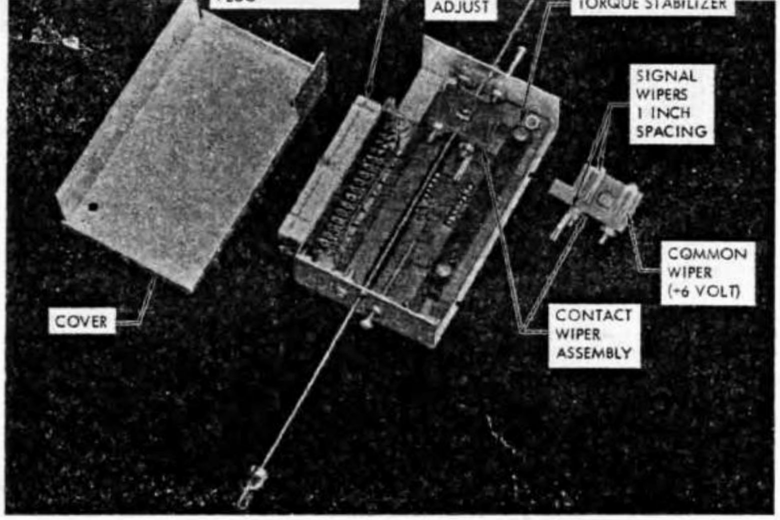


Fig. 9—Linear-deflection transducer with case open

supplied by a winch-truck cable, the winch used only to remove cable slack.

The synthetic rubber pads were employed not only to carry the load into the tower joints but also to protect mosaic inserts in the cement covering. The 2×4's were strapped along the legs, between rings, to distribute the load from four horizontal, cotton slings (see Fig. 7). The padding and the 2×4's were attached to each of the nine legs on the south side, from the base to an elevation of 55 ft above ground level. The four slings—each 6-in. wide—were located at 15, 27, 39 and 51-ft elevations (see Fig. 8). Horizontal, 6-in. I-beams joined the ends of the slings.

Two 8-in. I-beams joined the four 6-in. beams at the midpoints to complete a second stage of the whiffletree. A 10-in. WF beam joined the two 8-in. beams to complete the loading members (see Fig. 6). The 10,000-lb maximum load was thereby distributed—2500 lb to each sling and into the tower members through the 2×4's and padding. The weight of the beams was supported by cables to the scaffolding at the 60-ft elevation.

Instrumentation

Figure 9 shows a remote-reading deflection transducer designed and built for this test. Three such transducers were used: two measured bending deflections at 15 and 52-ft elevations, and one (shown in Fig. 10) measured overturning movement at the base. Deflections in $^{1}/_{32}$-in. increments were indicated for each transducer on a light panel at the control center. The total travel for each instrument was 2 in.

A four-gage strain bridge, wired to measure axial stress, was installed on the steel reinforcement angle of a leg on the south side of the tower. The installation was waterproofed and the cement cover replaced. Readings were taken during the test with a Baldwin SR-4 strain indicator (see Fig. 4).

Test observers equipped with binoculars studied predetermined portions of the structure throughout the test for any signs of local failures.

Test Procedure

Loads were applied in increments of 1000 lb with instrumentation recorded at each increment. Load was applied to 8000 lb and removed to measure permanent set. The final run to 10,000 lb was terminated after 1 min when the 10-in. WF beam began yielding (refer to Appendix, 3). This time for load application—although shorter than the 5 min specified—was satisfactory to the representative of the Building Department.

Results

The deflections and stress values are shown in Figs. 11 and 12, respectively. Stresses computed from the author's analysis are plotted for comparison with test values in Fig. 12. No cement cracking was observed during the test, although a $^{1}/_{16}$-in. chip of cement was found afterward on the tower base.

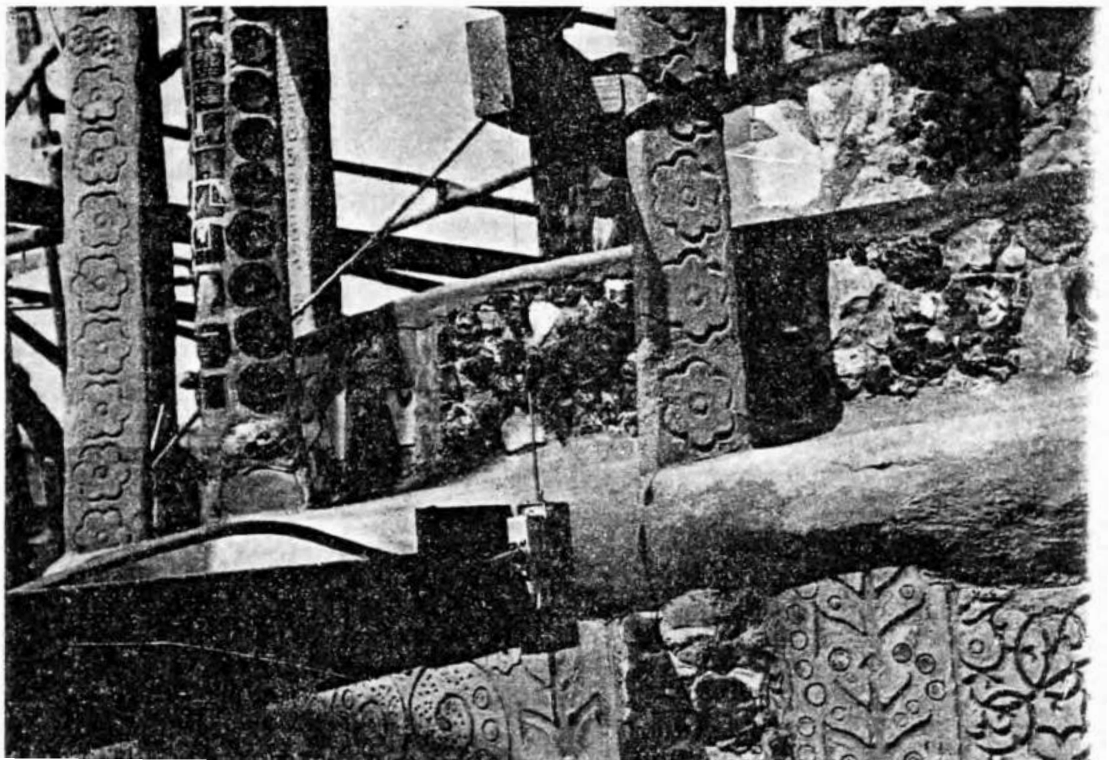


Fig. 10—Transducer installation at tower base (south side)

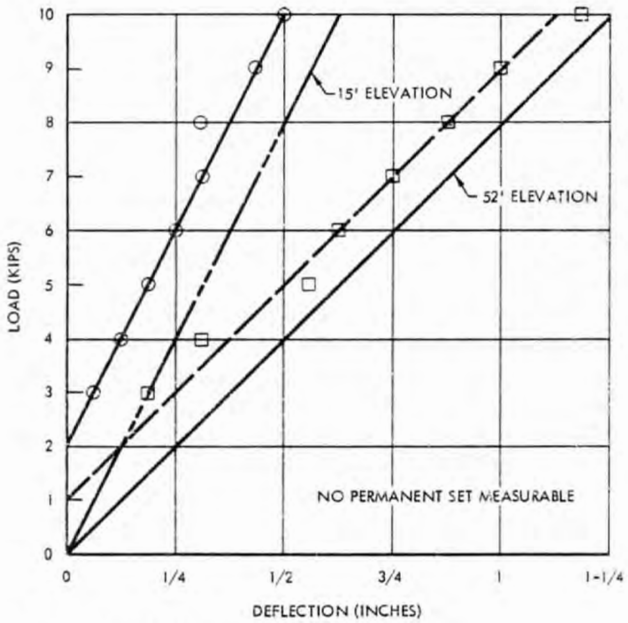


Fig. 11—Tower deflections graph

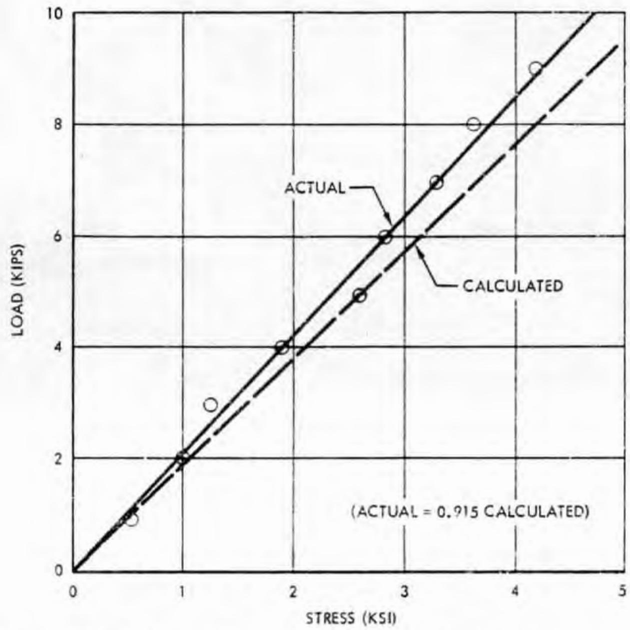


Fig. 12—Tower-leg stress graph, south (tension) leg

APPENDIX

1. The stress analyses presented by the Los Angeles Building Department differed from the author's analysis in the following major area:

(a) Department engineers analyzed the tower leg from three standpoints: (1) steel columns alone, (2) steel columns with lateral support from the cement covers, and (3) combination columns detailed in the *Uniform Building Code.*

These choices of analytic models assumed that the cement cover—reinforced with steel mesh—was only partially effective for carrying compression loads. The author's analysis assumed a reinforced concrete structure with resulting higher column allowables. Comparable compression allowables for the 6-ft long, unsupported leg at the base of the test tower were:

steel angle, alone:	1090 lb allowable
combination column:	2120 lb allowable
reinforced concrete:	5725 lb allowable

Column formulas:

$$\text{Steel, alone } P = \left\{1 + \frac{18{,}000/L^2}{18{,}000r^2}\right\} \left\{1.6 - \frac{L}{200r}\right\}$$

where:

$I = 0.04$ in.^{4}
$L = 60$ in.
$r = 0.32$ in.

$$\text{Combination column } P = A_r f_r' \,(1 + A_g/100A_r)$$

where:

$A_r = 0.27$ in.^{2}
$f_r' = 6260$ psi
$A_g = 7$ in.^{2}

$$\text{Reinforced concrete } P = 0.18A_g f_c + 0.8 f_s A_c$$

where:

$f_c = 3000$ psi
$f_s = 18{,}000$ psi

adjust for long column

$$P' = P(1.3 - 0.03h/d)$$

where:

$h = 60$ in.
$d = 4$ in.

(*b*) Department engineers computed anticipated stresses in the steel angle reinforcement of 16,000 psi under test load for the leg which had been instrumented. This prediction assumed that the cement did not contribute to the bending moment of inertia of the cross section.

2. Since the tower reinforcements have no rivets, welds or bolts for load transfer, the following calculations show the probable manner of transfer:

Calculations for the leg of the test tower:

Concrete in tension: $P = fA_c = (0.03\ f_c)\ 7 = 630$ lb
Steel-to-steel bond for 7-in. lap of $2 \times 1^1/_2$-in. angle, 120 psi bond stress: $(2 \times 7 + 1.5 \times 7)\ 120 = 2940$ lb
Tension through wire wrapping, 11 strands-$^1/_8$-in. diameter, 0.135 in^2: $0.135 \times 20{,}000$ psi $= 2700$ lb
Total allowable load $= 6270$ lb

3. It might seem that the most disastrous event which can befall a test engineer occurred for the author when the main whiffletree beam failed at 100 percent load. This failure was unfortunate, but nevertheless the test did accomplish its goal; and the narrow margins used throughout the jig and loading-apparatus design ensured a maximum load of 10,000 lb on the tower and little more.

Acknowledgments

The work described, herein, was carried out by the author as a private project. He is indebted to the following major contributors: William Cartwright; Attorney Jack Levine; Morley English, University of California at Los Angeles; Edward Farrell, AIA; Otto Steen; Dev Leahy; Vince Terry; and Robert Walker.