



Experimental testing of a saddle type hyperbolic paraboloid using three different load conditions
by Dennis Nottingham

A THESIS Submitted to the Graduate Faculty in partial fulfillment of the requirements for the degree
of Master of Science in Civil Engineering at Montana State College

Montana State University

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Abstract:

Results of tests performed on a saddle type ten foot square hyperbolic paraboloid with rigid supports and a two inch shell are shown in the following text in the form of tables and graphs. Three types of loads were employed -uniform load, concentrated center load, and uniform load over half the surface. These are probably the most common loads used and were chosen for that reason.

The basic design and problems encountered in the construction of forms and placing the concrete are also covered. Photographs and illustrations give a good picture of the procedure and apparatus used.

SR-4 strain gages were used to determine strains in the concrete. Methods employed in attaching gages to concrete and the nature of the readings are discussed.

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USING THREE DIFFERENT LOAD CONDITIONS

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DENNIS NOTTINGHAM

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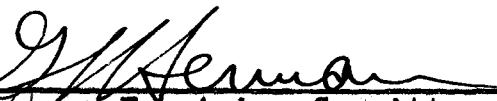
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ABSTRACT

Results of tests performed on a saddle type ten foot square hyperbolic paraboloid with rigid supports and a two inch shell are shown in the following text in the form of tables and graphs. Three types of loads were employed - uniform load, concentrated center load, and uniform load over half the surface. These are probably the most common loads used and were chosen for that reason.

The basic design and problems encountered in the construction of forms and placing the concrete are also covered. Photographs and illustrations give a good picture of the procedure and apparatus used.

SR-4 strain gages were used to determine strains in the concrete. Methods employed in attaching gages to concrete and the nature of the readings are discussed.

INTRODUCTION

HISTORY

Shell forms which are becoming increasingly popular today are not a new type of construction. These forms found their beginning in vaults, barrels, and domes in Middle Eastern and Byzantine building. Structures of this type were early attempts at spanning space beyond the capacity of post and lintel construction. Economically, these early shells were also justified. Brick and stone provided the materials for craftsmen with no formal mathematical training, yet many of these ancient buildings still stand.¹ The dome of St. Peter's spans 131 feet and weighs 10,000 tons. This may seem ponderous when compared with concrete shell domes in the market hall in Leipzig which span 240 feet and weigh 2,160 tons, but the latter was built in 1929.²

Even before early building began, nature was at work carving shells from stone. These shells are usually doubly curved, thus avoiding the tendency for bending moments to occur. This shows that shapes of this type can be utilized when building with materials which have a low tensile

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1. Chermayeff, Serge, "History of Thin Concrete Shells," Proceedings of a Conference on Thin Concrete Shells, June 21 to 23, 1954, MIT, p.2.
 2. "Shell Concrete for Spanning Large Areas," Architectural Forum, December 1949, p.101.

strength.³

The appearance of modern shells evolved from two independent sources, namely, a development in mathematical analysis and a perfection in structural materials. Lamé and Clapeyron formulated some mathematical expressions for stress analysis of shells in the early nineteenth century. Results of these findings led to further studies by Love in 1892, which involved the way that shells support loads. By this time, the advancement in analytical methods was sufficient for practical application to the design of actual structures.

Construction materials, such as concrete, were available in the late nineteenth century, but the greatest contribution in this field was the introduction of steel reinforced concrete. This discovery may be partly attributed to J. Monier, a French gardener, who built a reinforced concrete flower pot in 1867.

The combination of concrete and steel plus the proper methods of analysis set the stage for a period of increasing shell construction.

Antoni Gaudi was probably one of the first men to use reinforced concrete in imaginative fashions foreign

3. Candela, Felix, "The Shell As a Space Encloser," Proceedings of a Conference on Thin Shells, June 21 to 23, 1954, MIT, p.5.

to existing styles of construction. In 1909 he designed a parochial school in Barcelona which may have been the first modern shell roof ever built. Gaudi probably didn't analyze his shell mathematically. An example of a mathematically designed shell was the Zeiss dome built in 1924. The formal analysis was based on mathematical expressions by Love. In countries where material was scarce, the popularity of shells began to increase. This growth in interest accounts for the improved design practices of the present time.⁴

One great advantage of concrete shell structures is their resistance to fire. Severe fires in a shell concrete textile plant in Buenos Aires and a shell concrete hanger in the U. S. both failed to collapse the structures. A similar building with a steel framework would have surely fallen down. Shell type buildings have withstood impact due to bombs during wartime. The shell roof of the Fronton Ricalto's in Madrid was hit by a shell which knocked a six foot hole in it, but the roof remained standing. These examples refer to cylindrical shells which are surfaces of single curvature and are more exposed to failure than shells of double curvature.⁵

4. Levy, Matthys P., "Thin Shells: Some Basic References for Architects and Engineers," Architectural Record, June 1959, p.224.

5. "Shell Concrete for Spanning Large Areas," Architectural Forum, December 1949, p.103.

Although shell technique is now at home in the United States, many more advanced ideas are being produced outside our borders. The United States was slow to start building shells because of the high cost of labor and forming. New construction methods, movable forms, and high strength materials are gradually overcoming costs. The customer is just beginning to realize that shell roofs are not just a novelty, but relatively inexpensive handsome structures. The men who have done much to start this trend in the United States began more than a decade ago working in South America and Mexico. Two of these men are Felix Candela and Guillermo Gonzalez.⁶

The one doubly curved shell that cuts costs through easier forming is the hyperbolic paraboloid. The use of reinforced concrete in the hyperbolic paraboloid offers the same advantages inherent to all shells of this material --lightness, incombustibility, economy of materials, security against impact, and little sensitiveness to foundation settlement. Felix Candela is probably greatly responsible for the present interest in hyperbolic paraboloids. He has built many of these shells in Mexico where the cost of labor is relatively low.

6. Candela, Felix, "Market Project, Bandshell, "Architectural Forum, January 1957., p.132.
7. Candela, Felix, "Structural Applications of Hyperbolic Paraboloidal Shells," Journal of the American Concrete Institute, January 1955, p.397.

Shells of this type have been used for entrance canopies, churches, footings, warehouse roofs, gas stations, dwellings, factories, bowling lanes, and many other buildings.

Actual controlled tests on concrete hyperbolic paraboloids have been run recently by the Portland Cement Association. Investigations of this nature are rapidly increasing with the coming interest in shells.

The simple beauty and many advantages of the hyperbolic paraboloid mark it as a structure which will be progressively utilized in the future.

THEORY AND DESIGNANALYSIS AND PROOF

The following is an analysis of a hyperbolic paraboloid loaded with a uniform load.

A description of the shell surface is obtained in the following manner. Referring to Figure 1:

$$c/h = x/a \text{ and } z/c = y/b \text{ from similar triangles.}$$

Therefore

$$c = hx/a = bz/y; z = xyh/ab.$$

If $k = h/ab$, then

$$z = kxy.$$

For convenience an axis rotated an angle theta from the original axis is chosen. See Figure 1. Using this new axis

$$x = x' \cos \theta + y' \sin \theta \text{ and}$$

$$y = x' \sin \theta - y' \cos \theta$$

If theta is equal to 45° then

$$x = 0.707(x' + y') \text{ and}$$

$$y = 0.707(x' - y').$$

Substituting the values of x and y,

$$z = 0.5k(x'^2 - y'^2).$$

When $x' = 0$ then $z = 0.5k(-y'^2)$ and when $y' = 0$ then $z = 0.5k(x'^2)$;

these are both equations for parabolas.

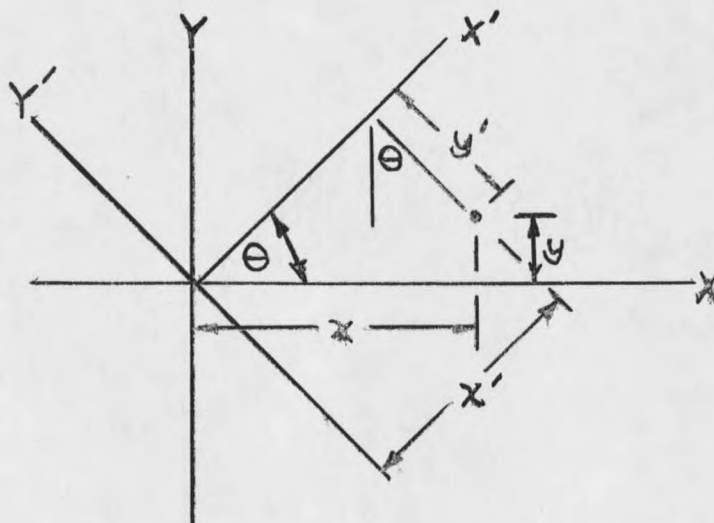
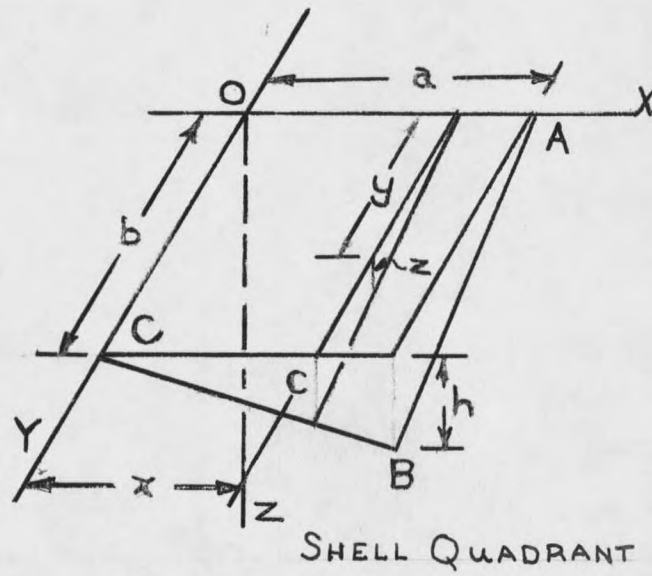


FIGURE 1.
DERIVATION FIGURES.

A parabolic arch under uniform load has zero bending moment throughout. Using this fact, the horizontal thrusts can be found. Note Figure 1a. The sum of the moments about the center of the span is equal to zero or $H(-h') = (wL/4)(L/2) - (w/2)(L/2)(L/4) = wL^2/16$. $w/2$ is used as the uniform load since it is assumed that half the load goes to the tension parabola and half to the compression parabola.

Therefore

$$H = -wL^2/16h'.$$

$z = -0.5ky'^2$. If $y' = L/2$ and $z = h'$ then $h' = -0.5kL^2/4$.

Therefore $H = (-wL^2/16)(-4/0.5kL^2) = w/2k$ or

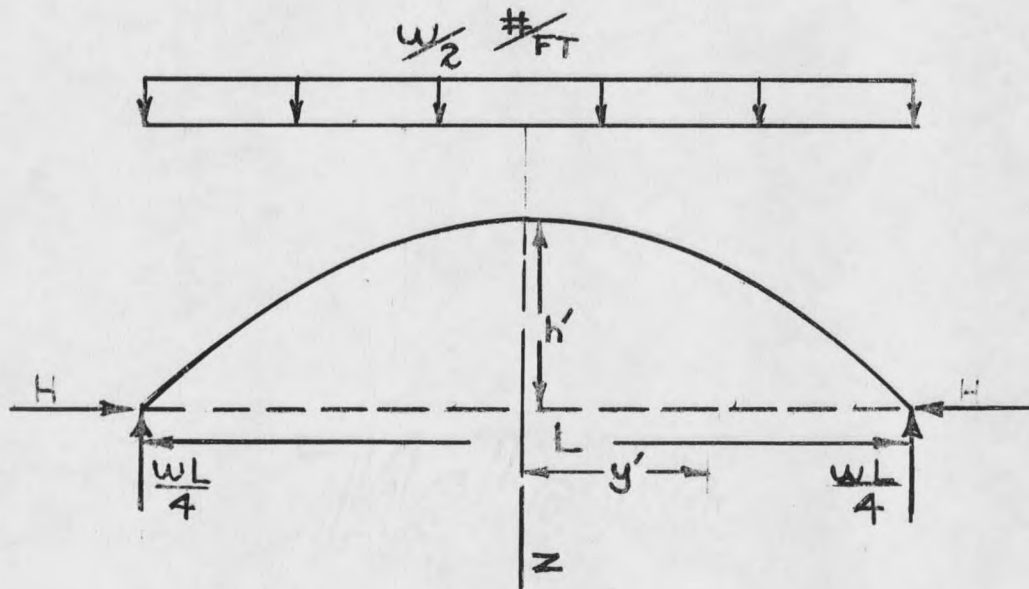
$$H = wab/2h.$$

Up to this point, it has been assumed that the edges of the structure are rigidly restrained. It will now be shown that this assumption is valid. From Figure 1a it can be shown that the thrusts perpendicular to the edge beam are equal in magnitude and opposite in direction. Therefore, the edge beam is restrained from moving laterally. A force exists along the beam horizontally. This force $(S) = 2H \sin \theta \, ds/dx = 2H \sin \theta \cos \theta$.

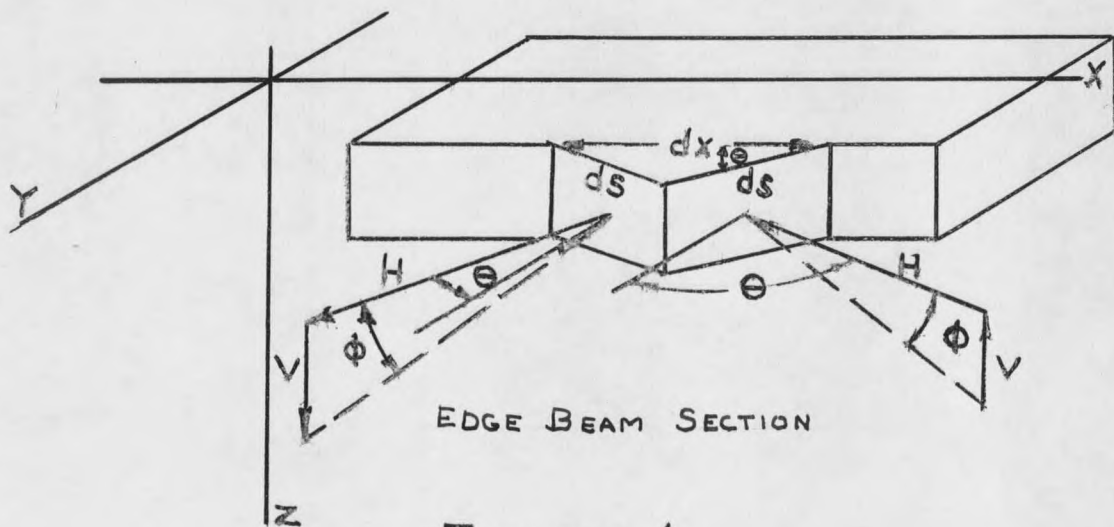
When θ equals 45° and $H = wab/2h$ then

$$S = wab/2h \text{ per unit length of beam.}$$

The vertical forces at the edge beam are equal to



TYPICAL PARABOLIC ARCH



EDGE BEAM SECTION

FIGURE 1a.

DERIVATION FIGURES.

$$V = \sum H \tan \phi = H dz/dy' + H dz/dx'.$$

Since $z = -0.5k(x'^2 - y'^2)$, then $dz/dy' = -ky'$ and $dz/dx' = kx'$. At any point on the horizontal edge $x' = y'$ and the vertical components cancel each other. Along the sloping edge AB, y' will not equal x' , but instead equals $x' - a\sqrt{2}$. Now $dz/dy' = -k(x' - a\sqrt{2})$ and $dz/dx' = kx'$.

Substituting these values, $V = \sum H \tan \phi = H[-k(x' - a\sqrt{2})] + Hkx'$ or

$$V = a\sqrt{2}kH = a\sqrt{2} H(h/ab) = Hh\sqrt{2}/b.$$

V in this case is applied to a length ds . V' per unit length of beam is equal to $Vds/dx = Hh\sqrt{2} \cos \theta/b$. For angles equal to 45° , $V' = Hh/b$ along an edge parallel to edge b . Similarly, along an edge parallel to edge a , $V' = Hh/a$. This shows that the vertical force in the edge beam is then $H \tan \gamma$ where γ is the angle of rise of the edge beam. Therefore, the edge beam is in direct tension or compression.

The preceeding proof and analysis was taken mainly from Elementary Analysis of Hyperbolic Paraboloid Concrete Shells, an edition of the Portland Cement Association.

DESIGN

Available laboratory space limited the size of the hyperbolic paraboloid to ten feet square in plan. A rise of four feet was chosen as a maximum height. This was to keep the fresh concrete from moving during placing

and vibration. A shell thickness of two inches was selected as being a minimum for good concrete workability and ease of placing. The following design loads were used:

two inch shell	-	25 psf,
edge beam	-	1 psf,
live load	-	40 psf,
for a total of		66 psf.

The horizontal component of the force in the edge beam equals

$S = wab/2h = 66(5)(5)/2(2) = 413$ pounds per foot of edge beam.

Total edge beam compression equals

$413 \sqrt{1116} = 4450$ pounds.

Vertical reactions at the supports equal

$2(4450) \sin 21.8^\circ = 5300$ pounds. Where 21.8° is the slope of the beam. Horizontal thrusts at the supports equal

$2(4450) \cos 21.8^\circ \cos 45^\circ = 5840$ pounds.

Most edge beams are designed as columns, but here they were designed as being in direct bearing. With a $f'_c = 3000$ psi, then $f_c = .25f'_c = 750$ psi. Total edge beam cross sectional area equals

$4450/750 = 5.94$ square inches.

Since eccentric loads were to be used, the assigned edge beam dimensions were three inches wide and four inches

deep on the outside edge. A number three reinforcing rod was run along each beam, two inches up from the bottom and continued into the footings. Shell tension and compression along the parabolic arches was equal to $413/24 = 17.2$ psi.

This was reinforced with six by six number ten welded wire fabric. The steel in the shell and beams was used primarily to prevent shrinkage or temperature cracks. Footing dimensions were 5 ft x $3\frac{1}{2}$ ft x $5\frac{1}{2}$ in reinforced two ways with number five rods on six inch centers. The footings were greatly over designed to insure no footing failure under eccentric loads. The length of connection between the shell and footing was arbitrarily chosen. The tie rod had a $7/8$ in diameter with a turnbuckle in the center. Six inch lengths of $3\frac{1}{2}$ in x $3\frac{1}{2}$ in x $\frac{1}{2}$ in angle were welded to the ends of the tie rod as bearing plates.

The preceeding shell design was based on the assumption that the structure was uniformly loaded. In the following text a comparison will be made between stresses and strains predicted by the design and those that are created by several types of loading.

CONSTRUCTION AND PROCEDURECONSTRUCTION

To begin forming, the bottom edge two by four beams were fitted together in the proper position. These beams constituted all that was necessary to generate the required surface. The beams were braced and the interior two by four beams were added parallel to one set of parallel sides. Stringers were run under the beams at the one-third points and shored at the end and one-third points. The entire group of shoring was strengthened with diagonal one by four bracing. Sheeting was placed parallel to the other set of parallel edge beams. Six inch shiplap had to be ripped into three inch strips for easy handling, since the boards assumed a warped form. By placing the covering in this manner, small cracks were developed near the edges of the forms. These cracks were eliminated when the entire surface was covered with roofing paper. Edge forms consisting of two by six lumber were placed next and braced to the existing structure. The bottom of the trough was closed off by one by three boards which gave the finished concrete edge beam the required three inch width. Completion of the footing and abutment forms finished the forming. See Figure 4.

Next the reinforcing steel and tie rod were placed and held in position with wires and blocks.

Concrete was brought to the laboratory in a ready-mix truck which was located as close as possible to the forms. However, it was still necessary to use a wheelbarrow. Slump of the one and one half cubic yards of concrete was about one to two inches. This presented some problems in placing, most of which were eliminated when a vibrator was put to use. Surface finishing was done by troweling followed by a light brooming. See Figure 3.

The structure was kept damp with a wet canvass for a five day curing period. Stripping the forms began after five days and was accomplished with little difficulty. Test cylinders broke at a five day strength of 1950 psi and 3910 psi.

Many curious and interested people viewed the completed hyperbolic paraboloid during the annual High School Week festivities. See Figure 5.

GAGE PLACEMENT

A-1 and AR-1 were the two types of SR-4 strain gage chosen for use on the hyperbolic paraboloid. The A-1 type measures strain in one direction while the AR-1 type is a rosette made up of three gages similar to the A-1 gage.

Twelve positions were selected for gage placement. Their exact location can be seen on Figure 2, Figure 9, and Figure 2a. By symmetric placement of the gages at the

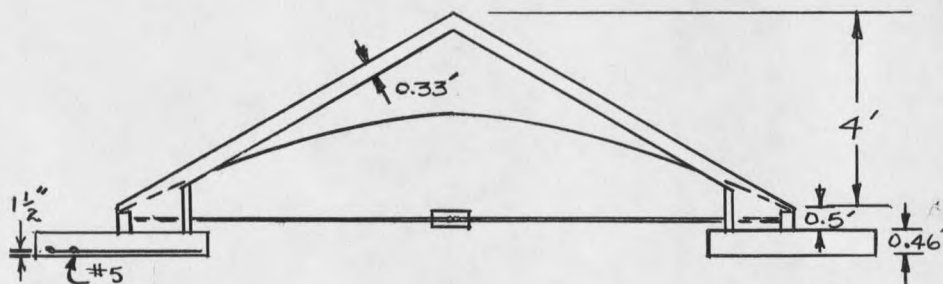
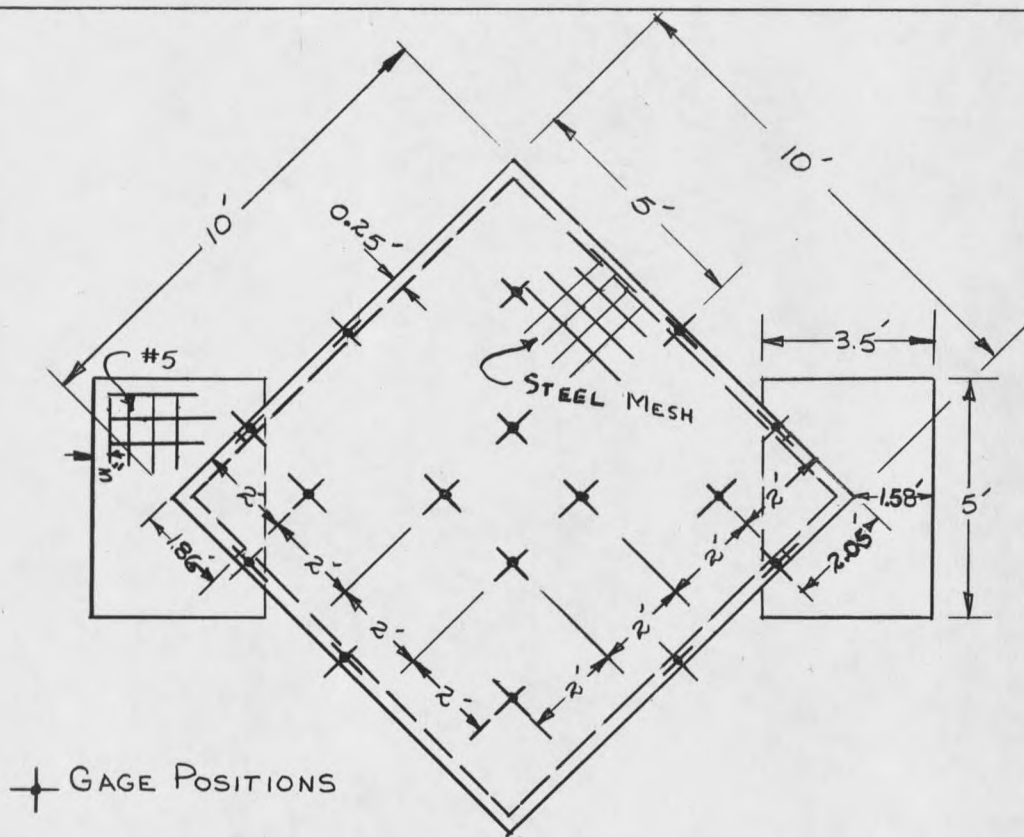


FIGURE 2.
DIMENSIONS AND GAGE POSITIONS

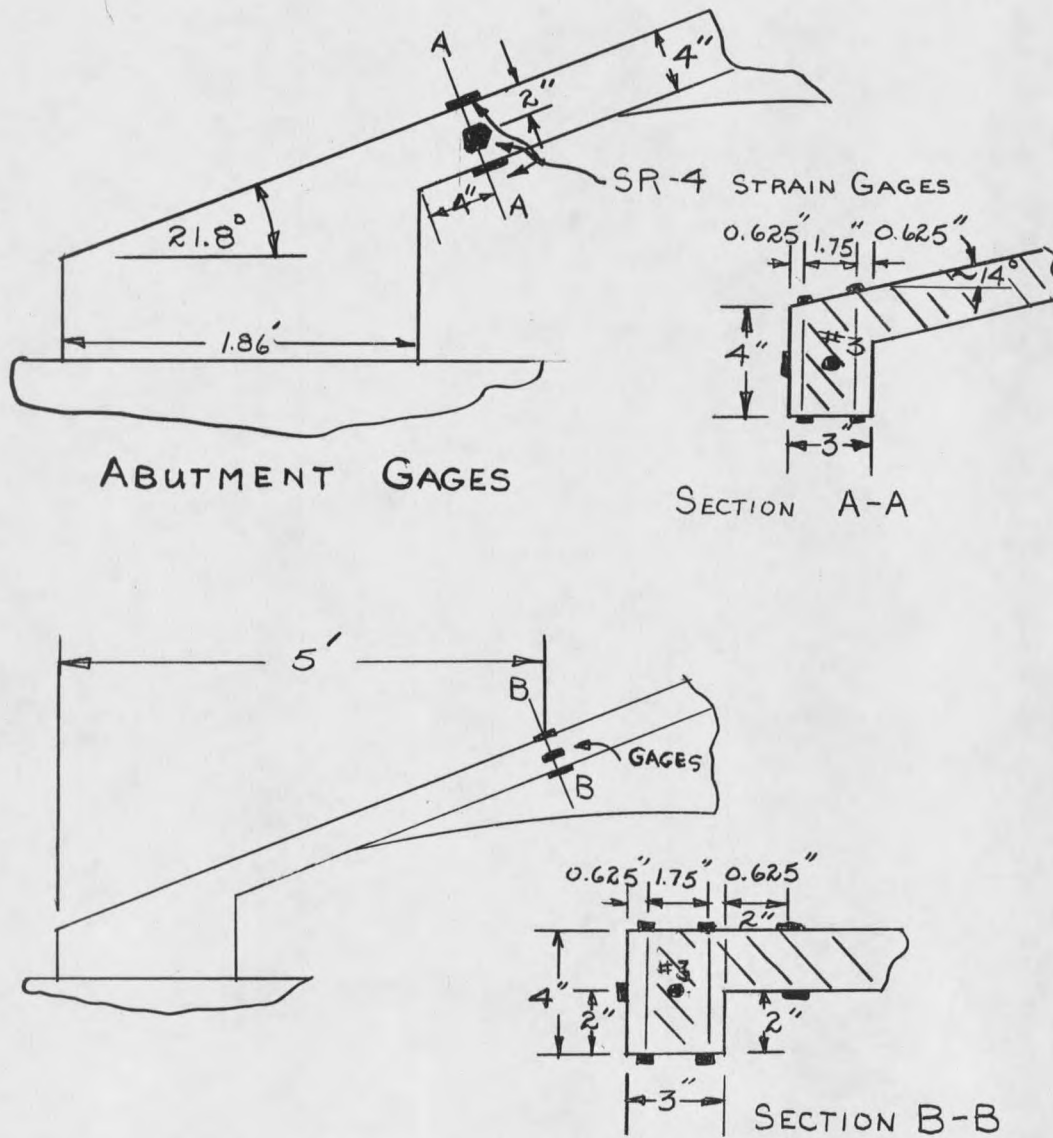


FIGURE 2a.
GAGE LOCATION.



Figure 3. Finishing The Shell Surface.

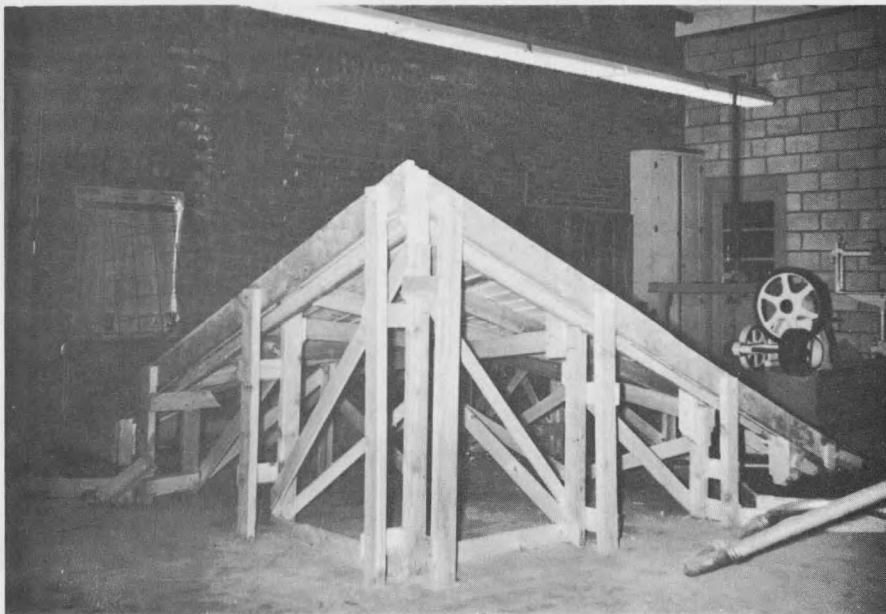


Figure 4. Completed Formwork.



Figure 5. The Finished Shell.

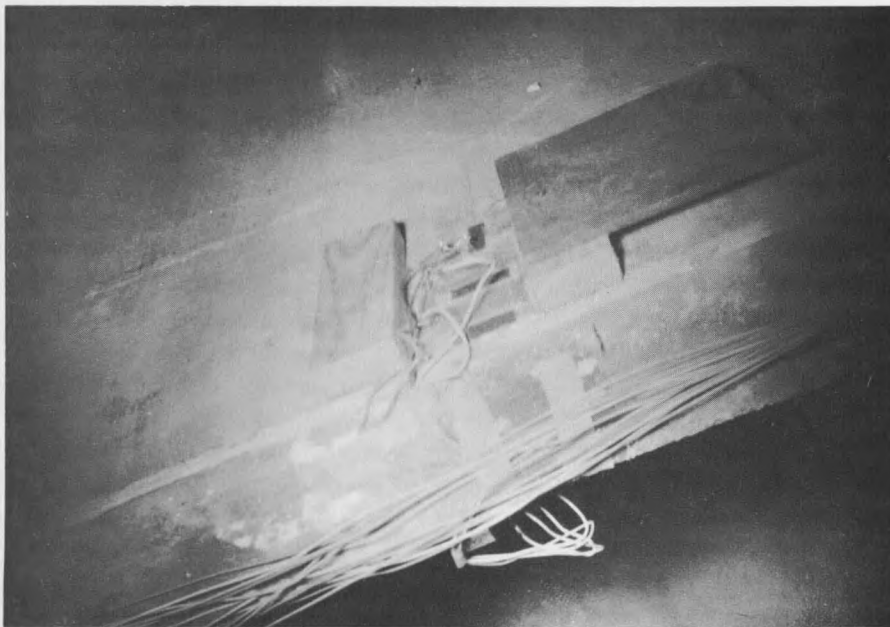


Figure 6. SR-4 Strain Gages Located At The Beam Center.

TABLE I - Gage Location Key

11-T1b	21-T3b	31-T5b	41-T7b	51-T9r	61-T11r
12-T1w	22-T3w	32-T5w	42-T7w	52-T9y	62-T11y
13-T1r	23-B3b	33-T5r	43-B7b	53-B9r	63-B11r
14-B1b	24-B3w	34-B5b	44-B7w	54-B9y	64-B11y
15-B1r	25-T4b	35-B5w	45-T8b	55-T10r	65-T12r
16-B1w	26-T4w	36-B5r	46-T8w	56-T10y	66-T12y
17-T2b	27-T4r	37-T6b	47-T8r	57-B10r	67-B12r
18-T2w	28-B4b	38-T6w	48-B8b	58-B10y	68-B12y
19-B2b	29-B4w	39-B6b	49-B8w	59-C8y	
110-B2w	210-B4r	310-B6w	410-B8r	510-C5y	

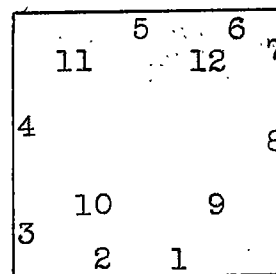
71-T2y
72-C2y
73-B2y
74-T3y
75-C3y
76-B3y
77-T6y
78-C6y
79-B6y

81-T7y
82-C7y
83-B7y
84-T5b
85-T5r
86-T5w

Gage
Positions

Plan
View

Abutment



Abutment

Measuring

Table

Example: 1 1 T 1
 Box No. Box Switch No. Location (top) Gage Position

b
Color:

Symbols:

T - Top of Beam or Shell
B - Bottom of Beam or Shell
C - Center of Beam on the Outside Edge
b - Blue Wire
w - White Wire
r - Red Wire
y - Yellow Wire

For gage numbers from 11 through 410 blue is on the outside of the edge beam, white is on the inside edge of the edge beam and red is located on the shell at the numbered positions.

Gages numbered from 51 through 58 and from 61 through 68 are located on the shell, with yellow being closer to the center in plan.

Gages numbered from 71 to 83 are on rosettes and are located at the center of the beams near the supports.

Gages numbered from 84 to 86 are on a rosette located on the shell at position 5. 84 is perpendicular to the edge beam, 85 lies along the compression parabola and 86 lies along the tension parabola.

center of each beam, at the supports and on the shell, it was possible to obtain readings of a similar nature. The following graphs were produced by using from one to an average of eight readings for each point. The exact number of readings represented by each point can be seen by referring to the tables containing the data. Gages placed on the beams were located $5/8$ in. from the edge because of the rough nature of the concrete near the edge, and also to keep away from irregular strain near the corners. Location of the gages four inches from the abutments along the edge beams was also to get away from irregular strains. Gages on the bottom of the shell are located beneath gages on the top such that the perpendicular distance between them is two inches.

Attachment of the gages to the concrete created some problems.

These were overcome when the following procedure was used:

1. the surface was ground to a fairly smooth finish with a power grinder and carborundum wheel,
2. remaining pores were filled with hydrostone capping compound, which was allowed to set,
3. the surface was sanded smooth and thoroughly cleaned,
4. a coat of SR-4 cement was applied and allowed to dry,

5. a second coat of SR-4 cement was applied and the gage placed over the fresh cement,
6. the gage was held in place and a pressure applied either by a weight or by hand for about twenty minutes.

Gages placed on test cylinders in the same manner served as compensating gages.

WIRING AND STRAIN MEASUREMENT

Coded wires from the various gages located on the structure were run to a common point near which the instrument table was located. See Figure 10. A Baldwin SR-4 strain indicator was used for taking strain readings. Seventy-three pairs of gage leads required the use of eight switch boxes containing ten sets of terminals each for connection to the circuit. These boxes were controlled by the master switch box which was connected to the indicator. See Figure 10. Both types of compensating gages were connected to the master switch.

DEFLECTION MEASUREMENT

Deflections were only measured on the free ends of the shell. Readings were taken using a dial gage supported on a wooden post. See Figure 11.

LOADING

Sand bags provided the means for applying loads to the structure. Paper feed bags were procured from a local



Figure 7. Gages Located On The Underside Of The Structure.

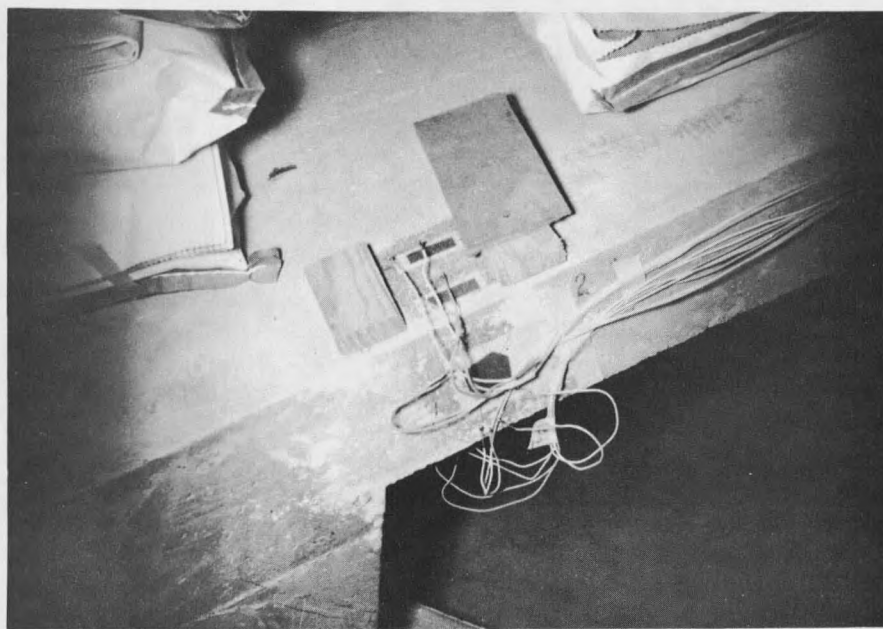


Figure 8. SR-4 Strain Gages Positioned Near The Abutment.

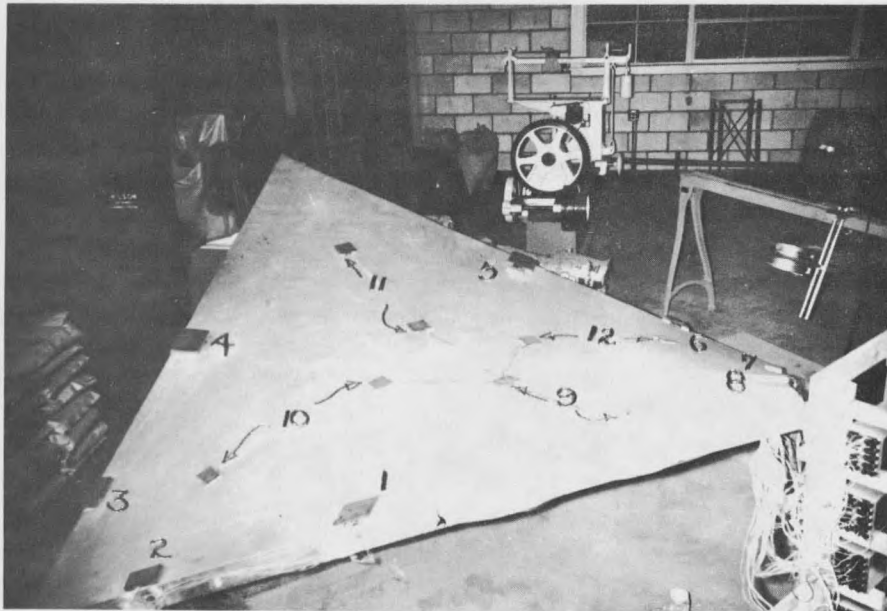


Figure 9. Strain Gage Positions On The Hyperbolic Paraboloid.

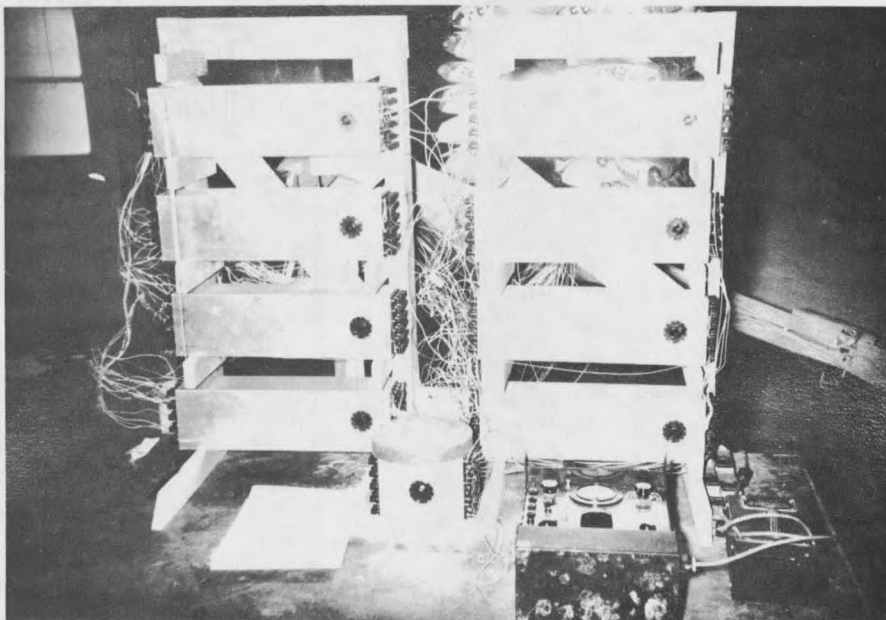


Figure 10. Switching And Measuring Equipment.

flour mill for this purpose. Forty pounds of sand and gravel were loaded into each of 150 sacks to make up the necessary load. Later, while loading the half uniform load condition it was necessary to fill additional twenty pound bags.

Twenty-five bags constituted a uniform load of 10 psf. See Figure 12. The loading points where the bags were placed were located in plan by the intersection of lines parallel to two adjacent edges of the shell. These lines were on two foot centers starting one foot in from the edge.

The condition termed half uniform load means a uniform load over half the surface area. The loading points for this condition were the same as for the uniform load. See Figure 14.

The area one foot square was used as the contact surface for the concentrated center load. Sand bags were placed on a wooden platform atop this area to attain the one ton weight. See Figure 13.

Complete loading for each condition consisted of a run where increments of load were added and strain readings taken, and a run where the total load was applied at once and readings taken.

EXPERIMENTAL RESULTS

General

Tests of this type have many variables. The following is an attempt to discuss these variables, the accuracy of the results and some of the assumptions made.

Average time for one complete increment loading was about three hours. During this time the temperature (usually about 95°F) was held to within one degree of the initial. After each increment of load was added, all seventy-three gage readings were taken, a procedure which took about fifteen minutes. A check back on the first gages read was done following the completion of each such reading. Maximum variation was usually around three to five micro inches per inch. Several things could contribute to this, mainly variable switch resistance. The erratic behavior of the lines plotted on the following graphs for small values of strain show this variation quite clearly. As a result these small values have no value except to indicate practically no strain. When large values of strain are obtained, small variations are not as noticeable, however, readings do not have pinpoint accuracy.

On the example graph (Figure 17) individual gage strains are plotted for a typical point. These strains jump back and forth, but on the average one gage will

have higher readings than the others. This also shows up under other types of loadings. Again the many variables involved with this type work can account for these differences. Non-homogeneity of the concrete, slight beam size or shell thickness variation, gage placement over paste as opposed to over aggregate as well as many others could be cited. At best then, the information that is shown by these tests is approximate and shows more of a trend than absolute accuracy.

Modulus of elasticity for concrete is needed to give some relation of stress or total force to strain. The stress strain relationship for concrete is not linear, but an approximation that $E = 1000f'_c$ is sometimes used. A design $f'_c = 3000$ psi was prescribed, but test cylinders broke at much higher values. A 3 in by 6 in cylinder cured in the same fashion as the shell broke at 4660 psi at a 30 day age, while two 3 in by 6 in moist room cured cylinders broke at 9560 psi and 9050 at the same age. The structure was tested at about two to three months age. It would not seem unreasonable that a strength at this age might be around 6000 psi. Therefore, for purpose of approximate comparison the value of the modulus of elasticity will be assumed to be equal to 6×10^6 psi.

Uniform Load

Points of highest stress for this load condition

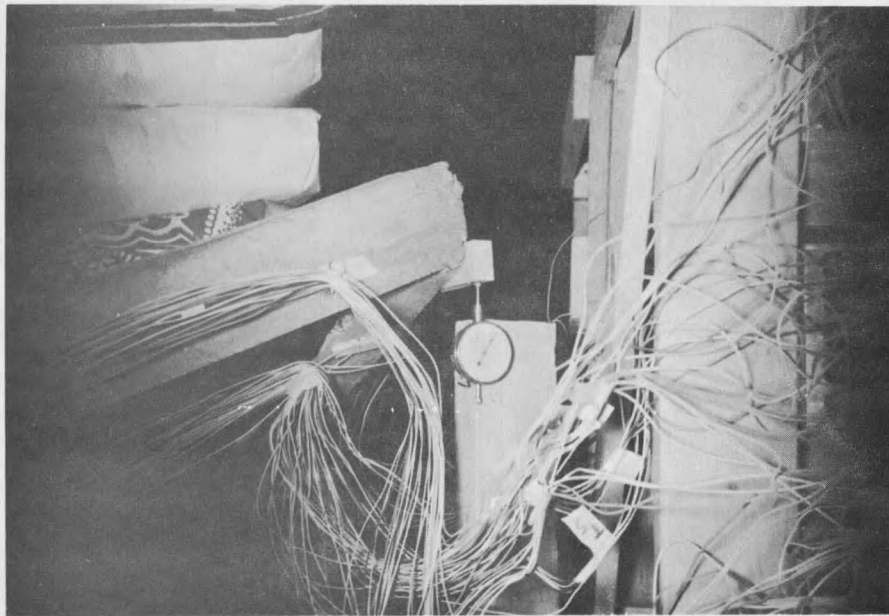


Figure 11. Strain Gage Leads And Dial Gage Position.



Figure 12. Uniform Load Over The Entire Surface.

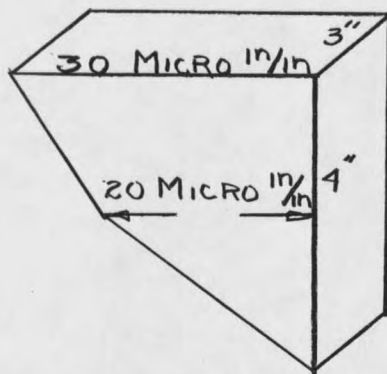
are located mainly on the edge beams. Shell stresses are quite low and as previously discussed, low strains tend to give erratic readings from the indicator.

Gages (13, 27, 33, 47, 15, 210, 36, 410) placed perpendicular to the edge beam at the beam center, show some interesting strains. The gages are located five inches in from the edge of the beam. From the theory it seemed justified that there would be no force perpendicular to the edge beam, yet these gages show a definite tensile force. The strains varied with two separate loadings from around 20 micro in/in to lower numbers. Perhaps deflection of the edge beam caused a slight tensile force. More tests with more gages applied would have to be run before a definite answer could be obtained.

The average values of strain for the edge beam at the center showed the following strain distribution. See Figures 20-24 for strain values. If we looked at the strain distribution across a plane perpendicular to the beam, a 40 psf live load would give average values as shown in the following sketch. Column action due to an eccentric axial load seems to be functioning here.

Assuming $E = 6 \times 10^6$ psi and that the total force (F) is carried by the edge beam, then

$$F = (20 + 30 + 20)(2)(3)(6)/2 = 1260 \text{ lbs.}$$



The theoretical force equals

$$F = 40(5)(5) 116/2(2)(2) = 1350 \text{ lbs.}$$

From this distribution it appears that the free

end should deflect up, however, this is a localized condition due to the column action by the eccentrically located edge beam.

Stresses in general were very low, but some in the vicinity of the abutments provided some interest. From Figures 25-29 the strains at various locations can be seen. Tensile stress developed in the concrete near the abutment was of most concern and appeared to be caused by bending in both directions. The beam at this point seemed to bend down and in. See Figure 15. The main reason for bending here is the rigidity of the abutment. The structure naturally will deflect under loads, but here the abutment prevents the deflection thus causing fairly large bending stresses. Using $E = 6 \times 10^6$ psi, maximum average tensile stress reached for uniform loading was 240 psi and maximum average compressive stress was 666 psi. Final end deflection at 50 psf live load was downward about 10 thousandths of an inch. The eccentric location of the edge beam undoubtedly limited

this to a lesser value than would have been seen had the beam been more symmetrically located.

Concentrated Center Load

A concentrated load was built up over the center of the hyperbolic paraboloid on a one foot square area. This area was large, thus only small contact and shearing forces were developed under the load.

Gages placed along the compression arch showed some strain was present and that it increased nearer the support. See Figure 19. Local bending or the dent caused by the one ton load extended somewhere beyond a 2.83 foot diameter as shown by the shell gages. See Figure 16.

Compression on the top and tension on the bottom changed to compression on the bottom and tension on top of the arch somewhere between the inside and outside shell gages. Near the supports, the direction of the strain in the shell agrees with that in the edge beam. Maximum average compressive stress developed in the shell under a gage was 274 psi and the maximum average tensile stress was about 48 psi. Both of these are sufficiently small to term negligible. Higher tensile stresses definitely existed directly under the load, but none of these were recorded.

At the center of the edge beams, the SR-4 strain gages showed bending in one direction. See Figures 20-24.

Compressive stresses were small, but tensile stress reached



Figure 13. Concentrated Load At The Shell Center.



Figure 14. Uniform Load Over Half The Surface Area.

values up to 190 psi. This stress distribution or one similar was apparently prominent over most of the beam because the final end deflection was up.

Strains at the abutment for the maximum concentrated load condition are very similar to those created by a uniform load of 40 psf. The differences being that the compressive stresses on the top of the beam are lower, and the tensile stresses on the bottom are higher for the concentrated case. This condition could have been caused by most of the force created by the load going to the abutments on the compressive parabola. Maximum average tensile stress reached at the abutment was about 270 psi for a one ton load.

Free end deflection was up with a magnitude of about 16 thousandths of an inch.

Half Uniform Load

The half uniform load was a uniform load over half the projected surface area of the shell. A load of this type is entirely possible in practice, and presents an eccentric condition.

Again, the shell stresses were very low as for the previous loadings. In fact, stresses of any magnitude failed to show up except at the abutment.

Strains recorded for the beam center location under the load showed an almost identical strain plot as did

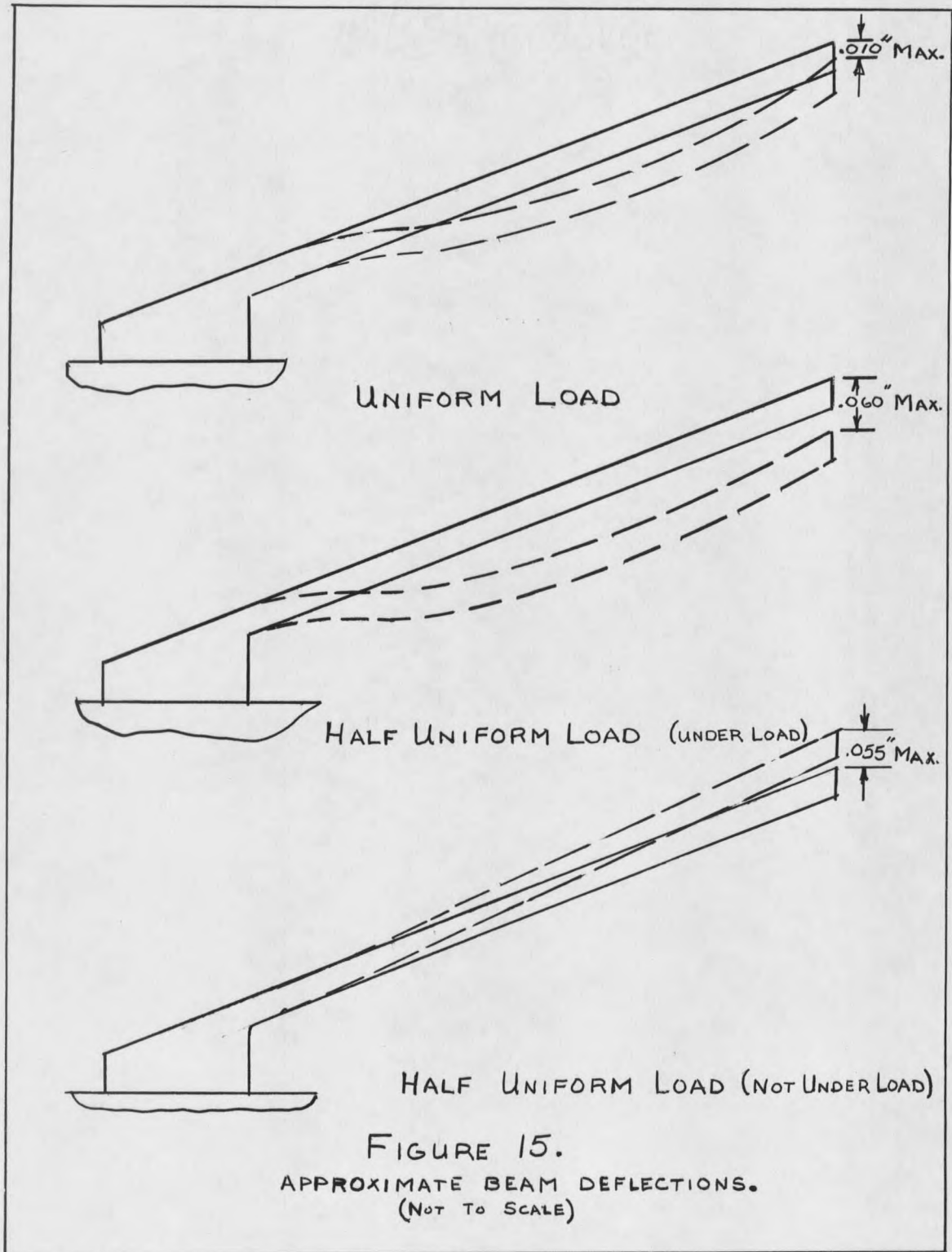
the pure uniform load. See Figures 20-24. For the beam center not under the load, strains were of negligible size.

The free end under the load deflected down 60 thousandths of an inch, and the other end went up 55 thousandths of an inch. With this information, the stress distributions can almost be visualized. Edge beams under the load at the abutment are acting somewhat as cantilevers. They develop high tensile stresses on the top and high compressive stresses on the bottom. Maximum average tensile stress on the top reaches 441 psi, while average compressive stress on the bottom reaches 1030 psi. See Figures 25-29. For edge beams on the other side of the abutment, the strain distribution is reversed. Here, maximum average tensile stress occurs on the bottom with a magnitude of 330 psi, while average compressive stress is about 252 psi.

Loading was carried to only 40 psf for this condition because high tensile stresses were being developed at the abutment and failure was not the present objective. Further tests on the shell and beams around the abutment for eccentric loads may be done in the future.

Stresses discussed in the preceding test and approximate, varying with the modulus of elasticity and were used only to give some idea of existing stresses. Maximum average stress as termed in the text means the

average of the number of readings at one gage and not an average of the stresses along a side or position. Principal stresses were not found in these tests, just stresses perpendicular to desired planes.



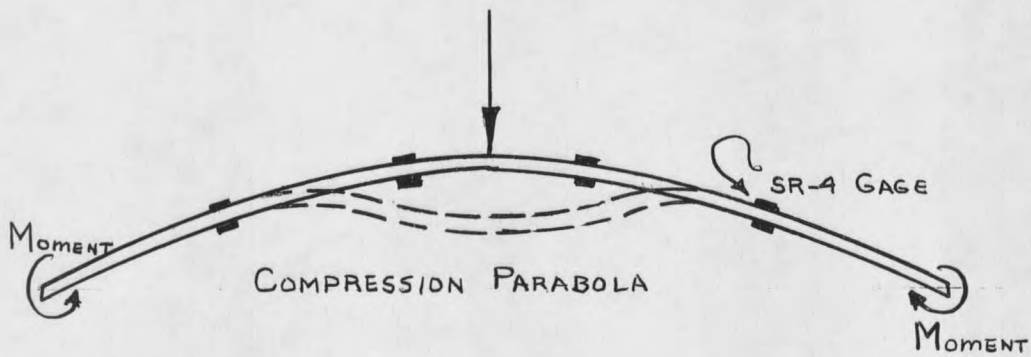
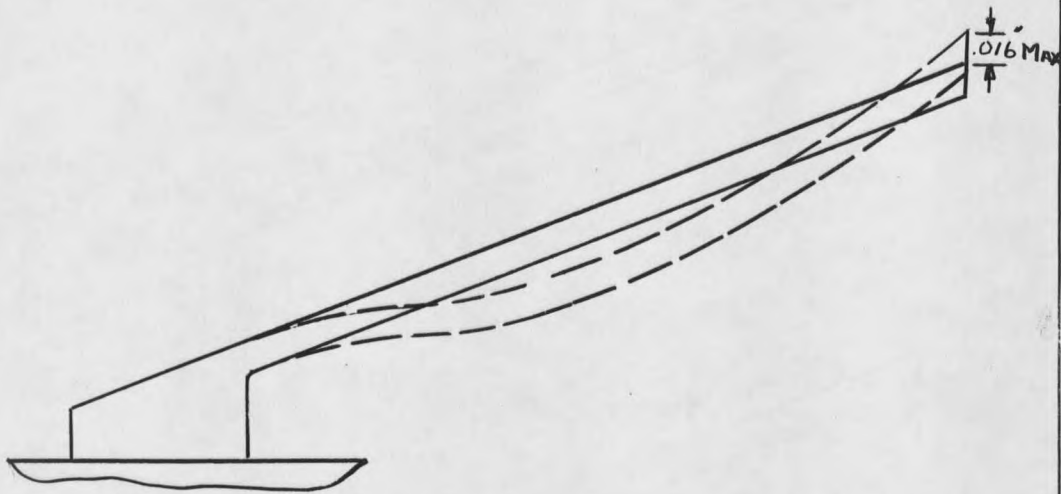


FIGURE 16.
CONCENTRATED LOAD DEFLECTION CURVES
(NOT TO SCALE)

CONCLUSIONS

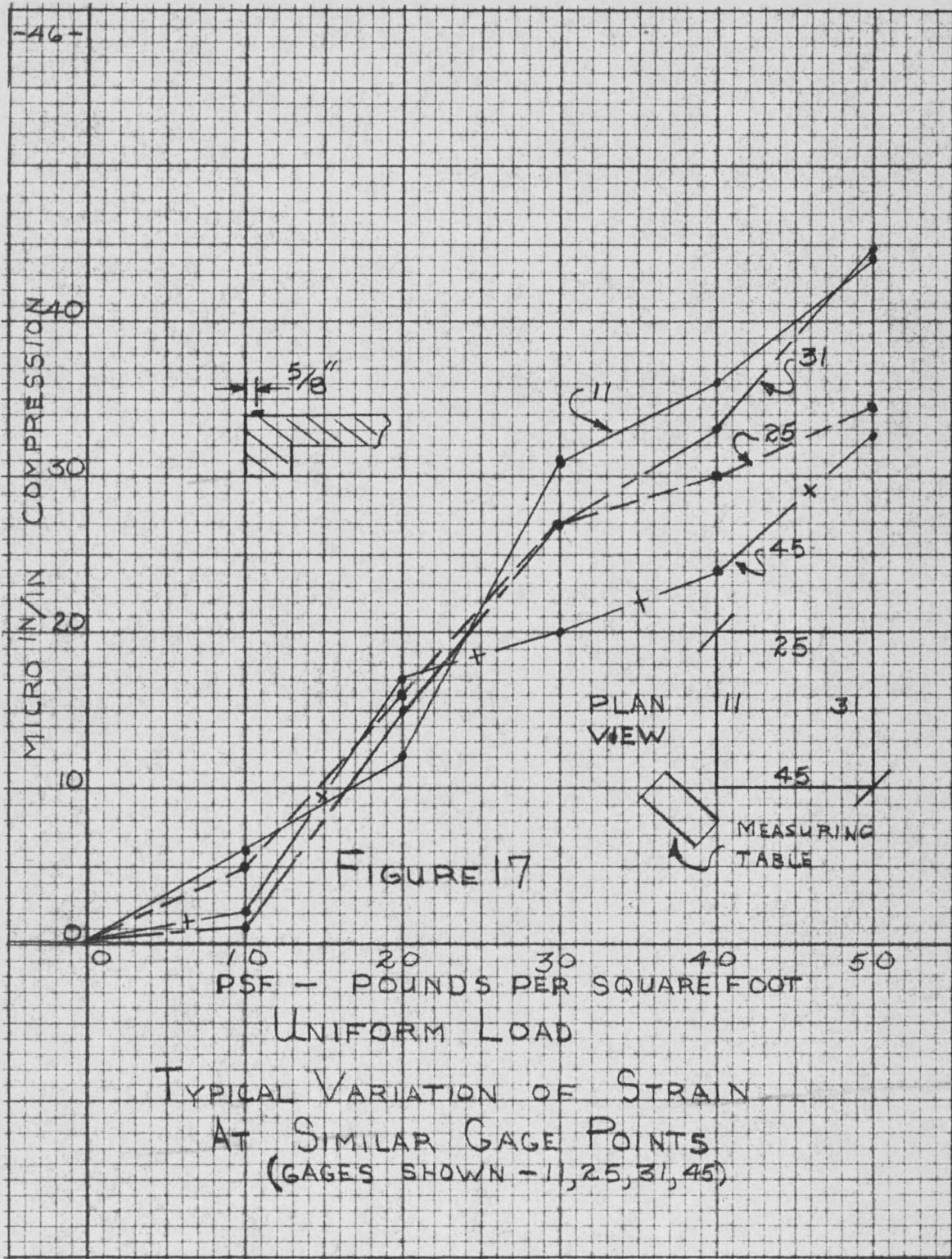
The results of these tests do not entirely uphold the uniform load theory as derived when applied to a hyperbolic paraboloid with fixed abutments. When abutments are extended along the edge beams with the intent of taking eccentric loads, then some design alterations might have to be made. The present system of designing edge beams as tied columns should be sufficient where abutments are rigidly fixed to the shell along short lengths, but do not take the entire eccentric load. Edge beams, down to a point somewhere between the center of the beam and the abutment seem to exhibit column action mainly, except for a concentrated load. Here, the beam would be more efficient if it were more symmetrically located. The sharp departure of the edge beam from the shell probably causes some stress concentration and should be eliminated. This procedure is presently being done.

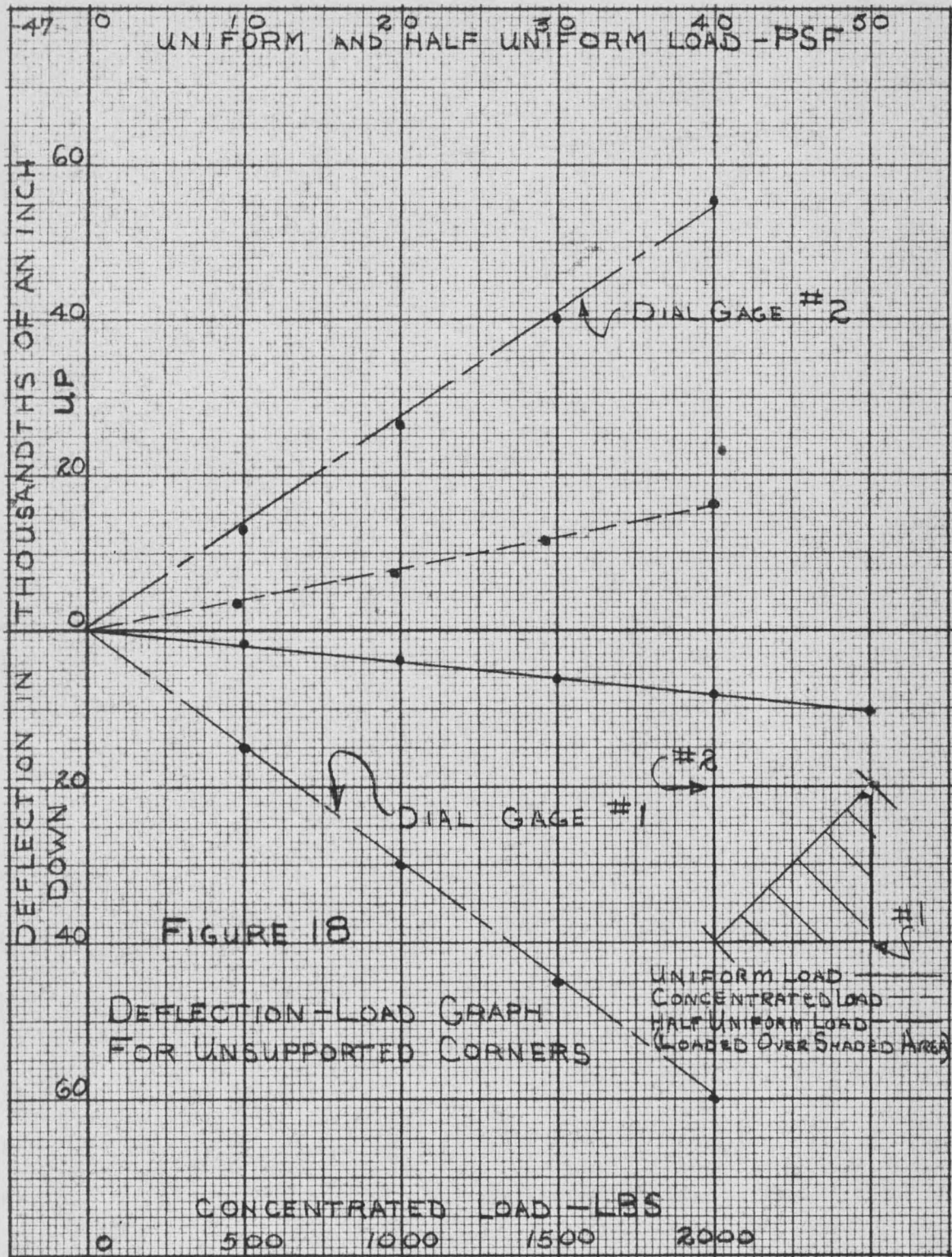
Near the abutments, an edge beam with a variable cross-section should take care of eccentric loads and stress caused by the rigidity of the abutment. The beam should have some tensile reinforcement on top and bottom.

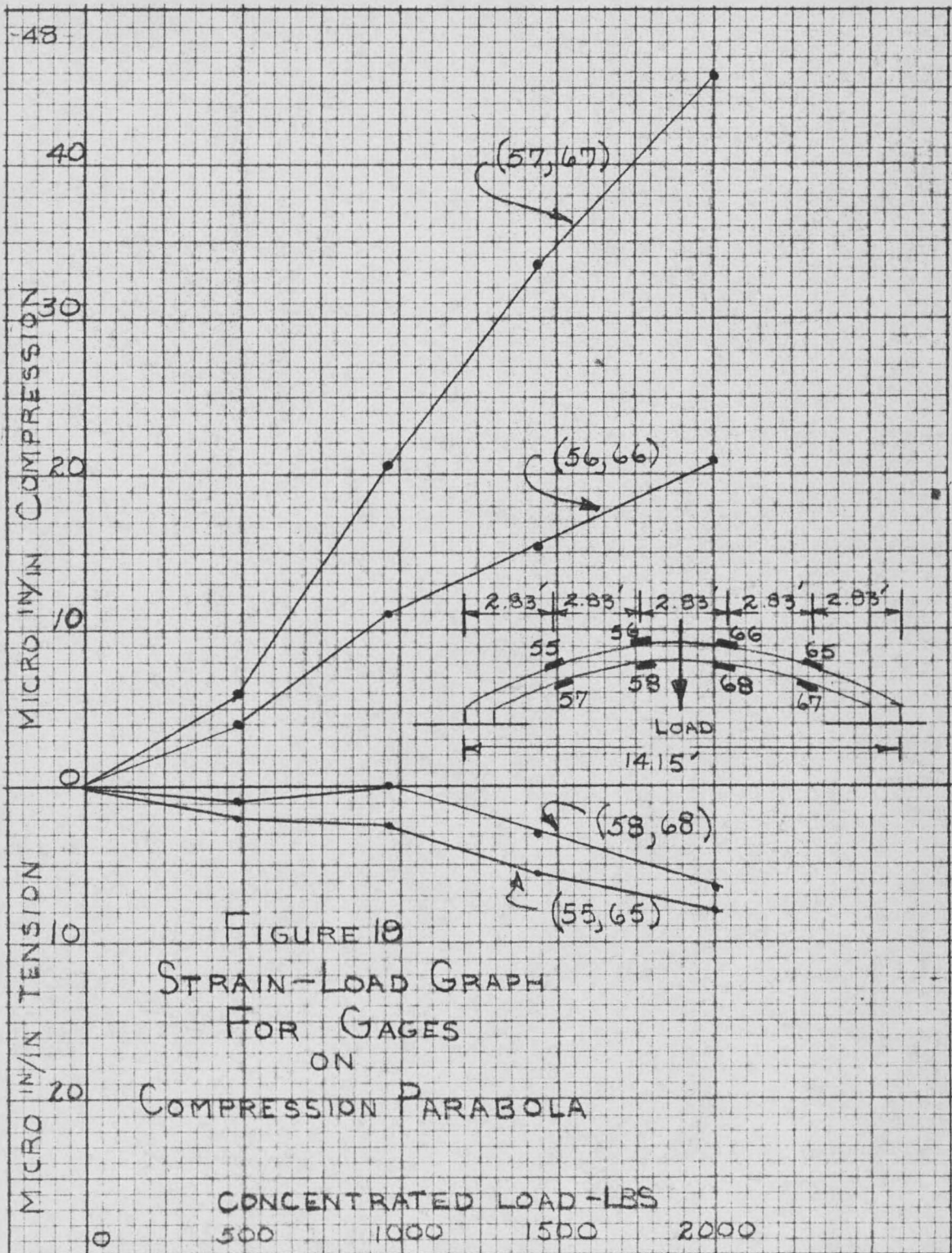
Shell stresses at the highest are of a very low magnitude and seem to be the least of the worries in design.

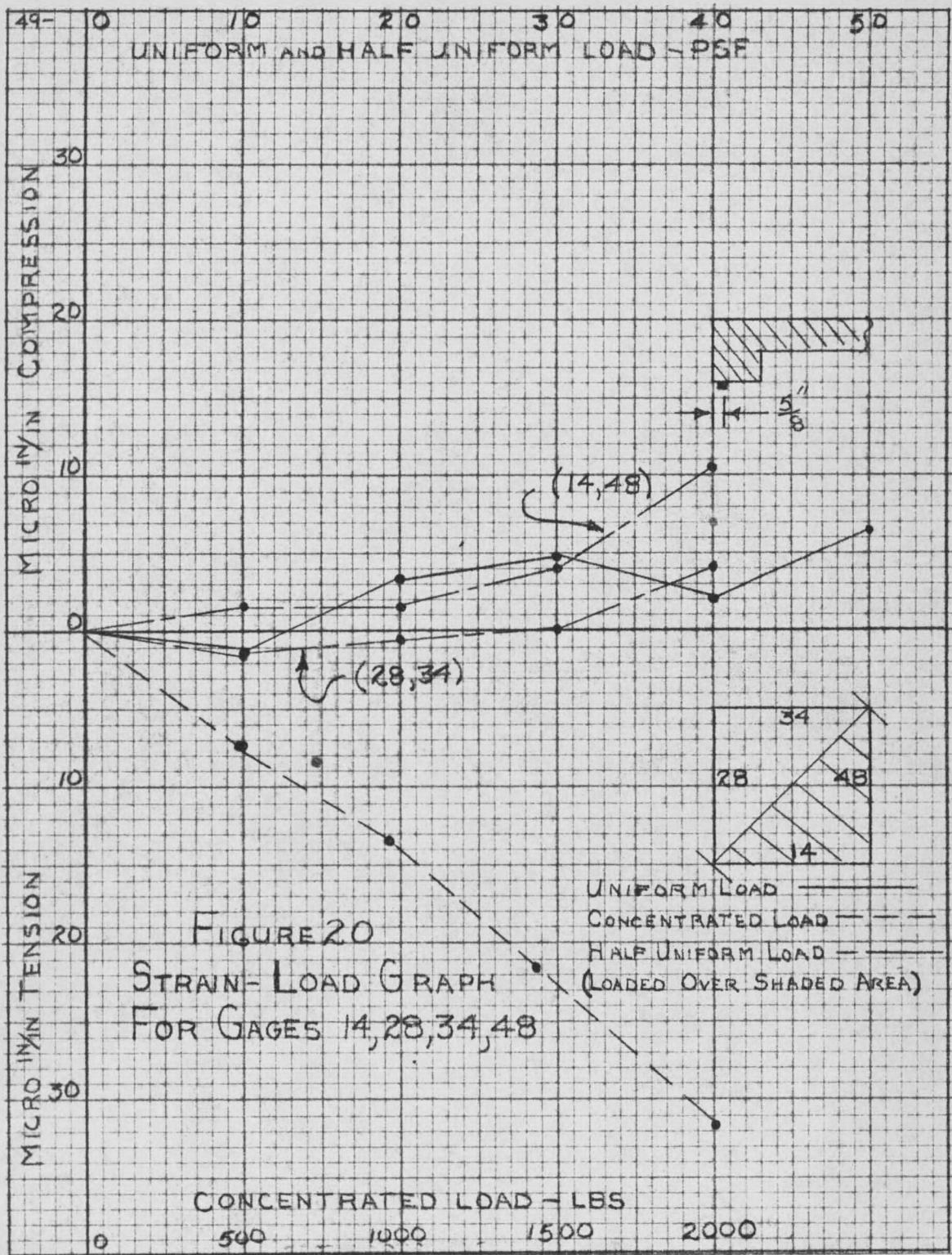
The preceeding discussion is basically what showed

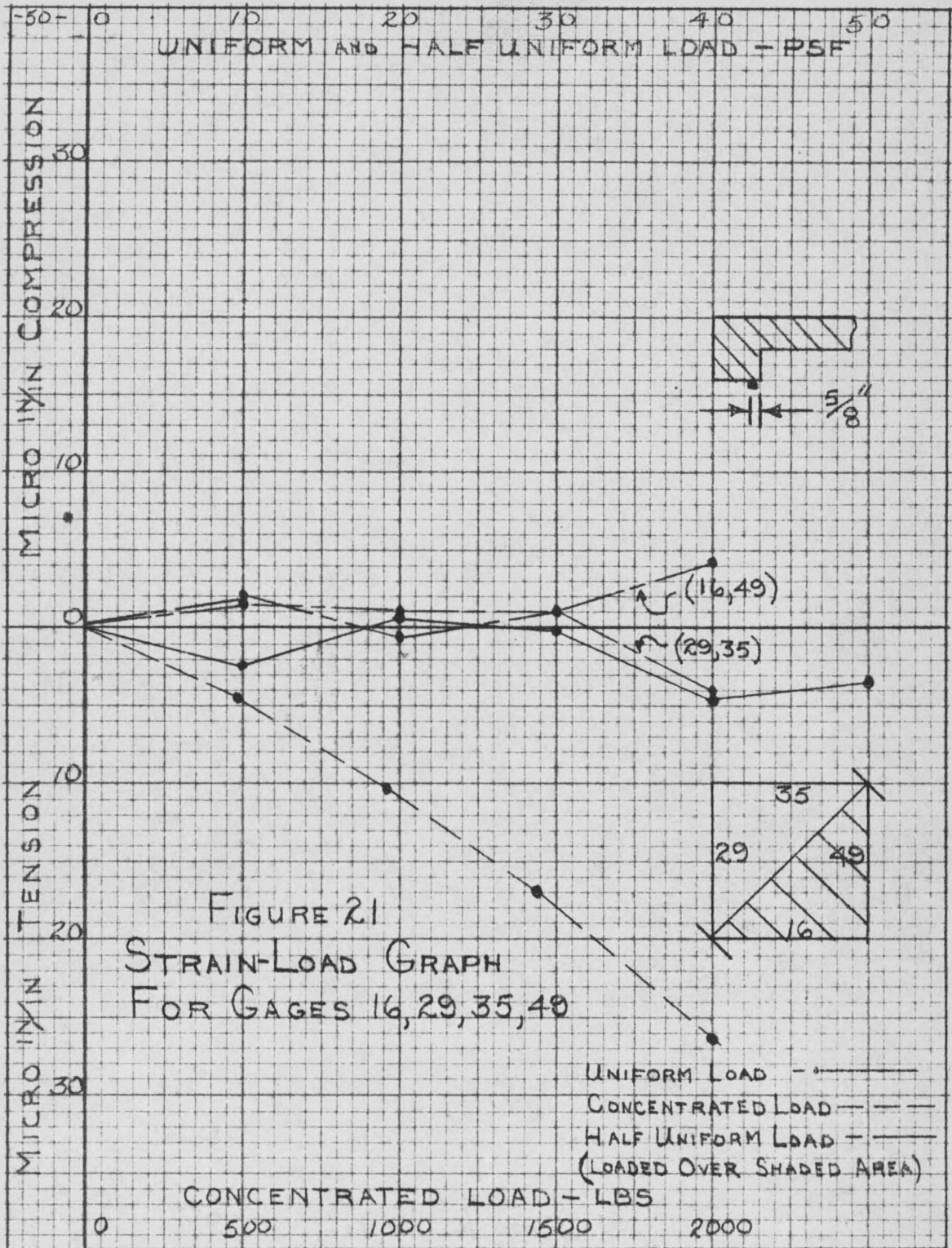
up in these tests on the hyperbolic paraboloid. The magnitudes of the results are probably of little use for the design of larger structures, but the stress distributions should give a more basic idea of what is actually happening in a shell of this type.

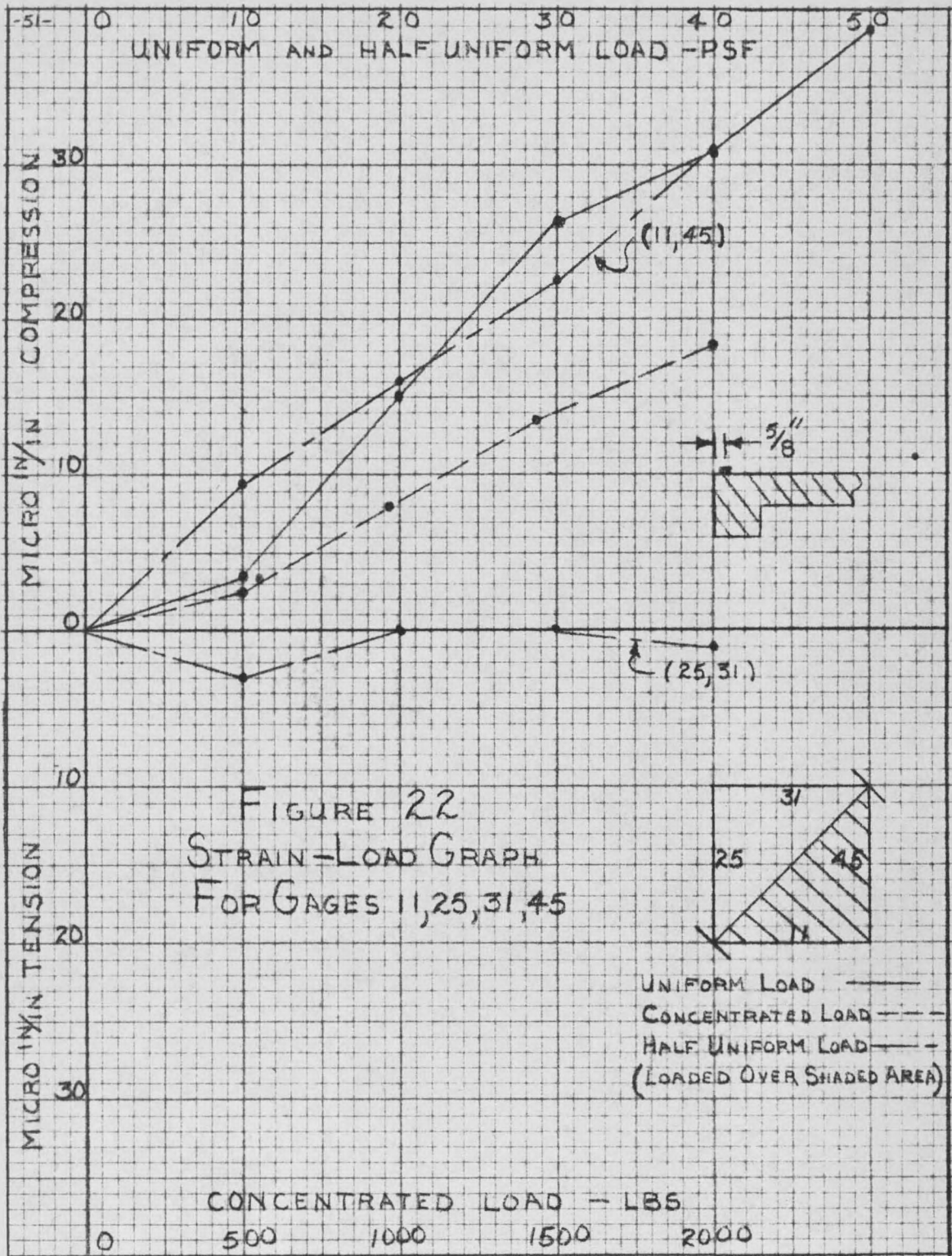


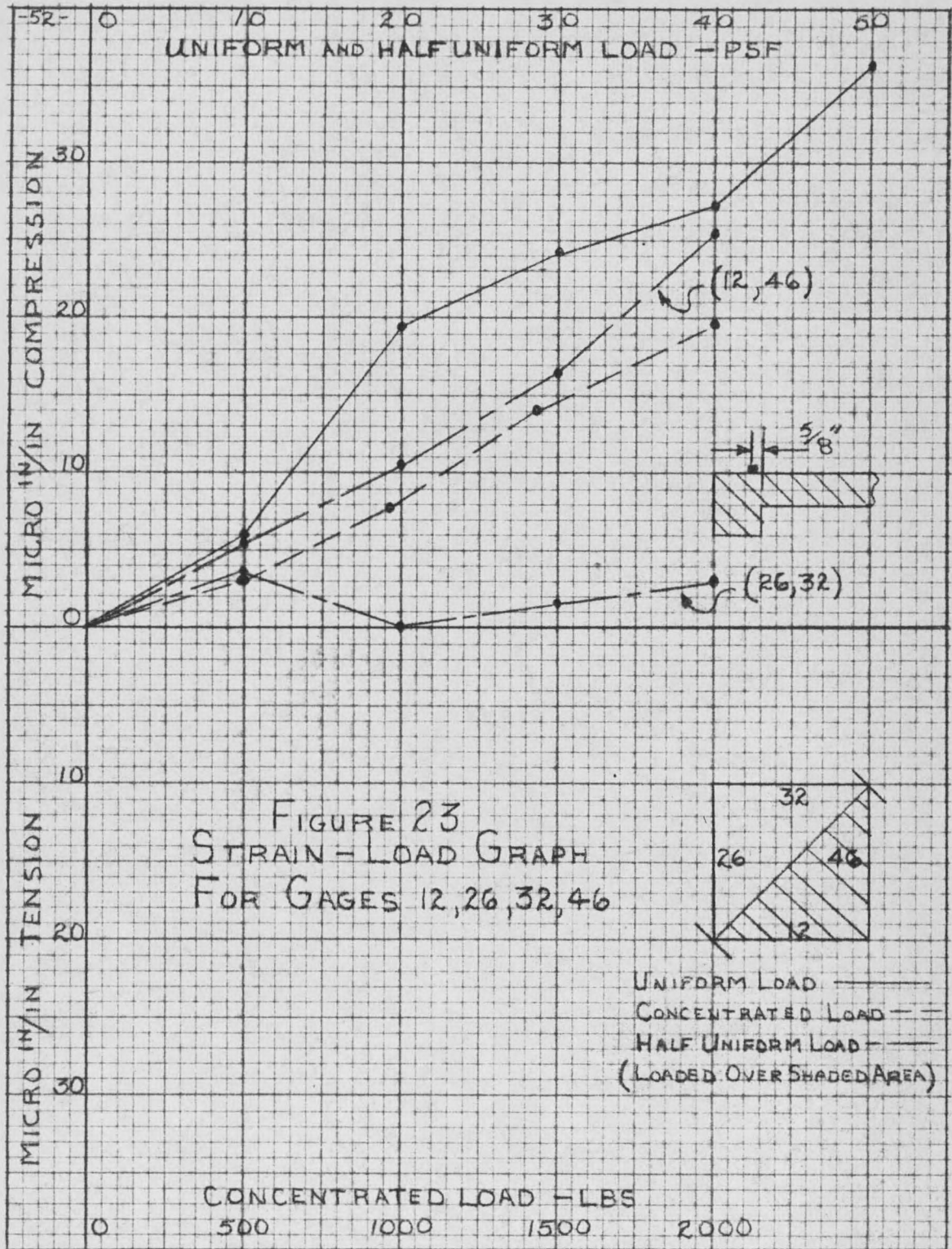


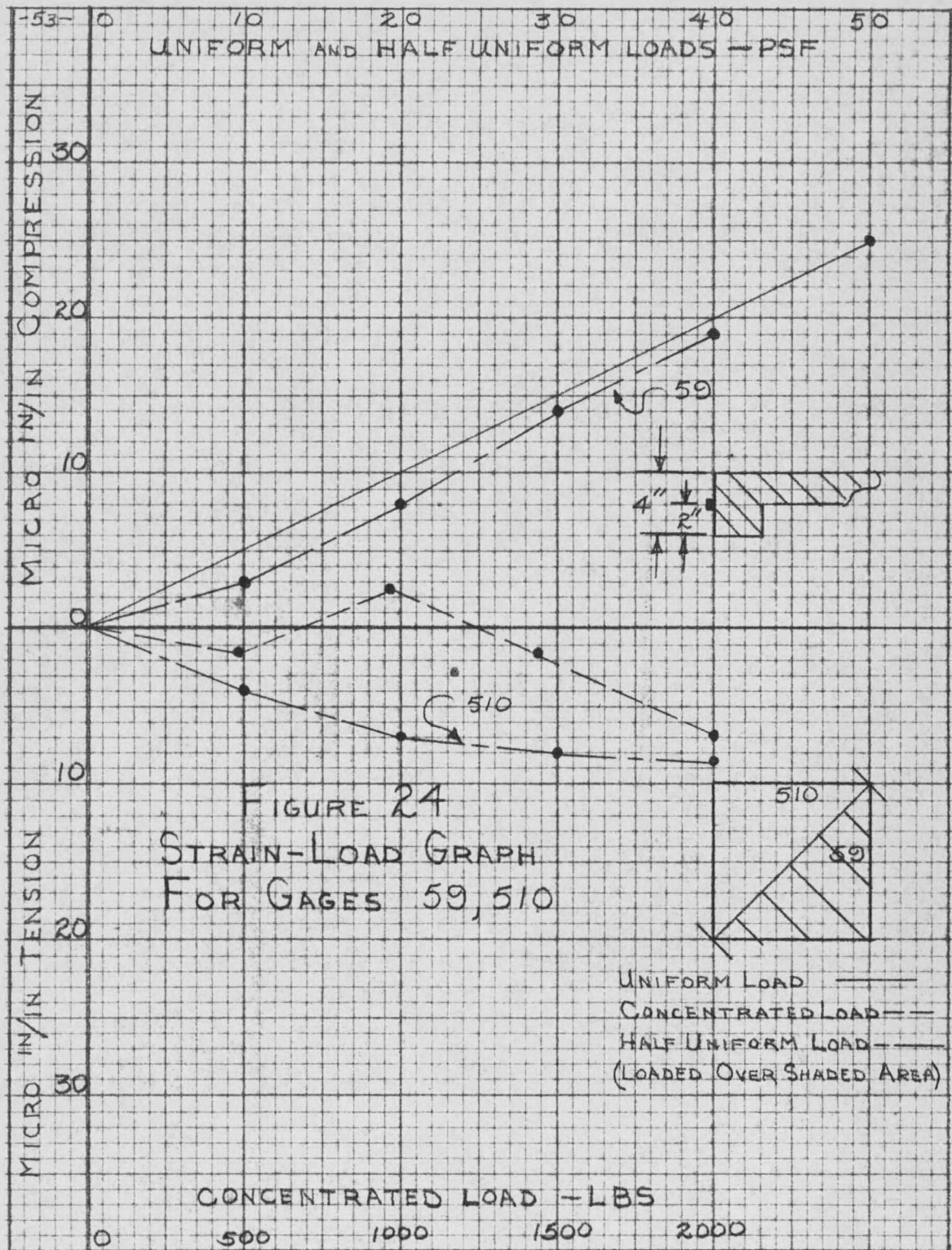


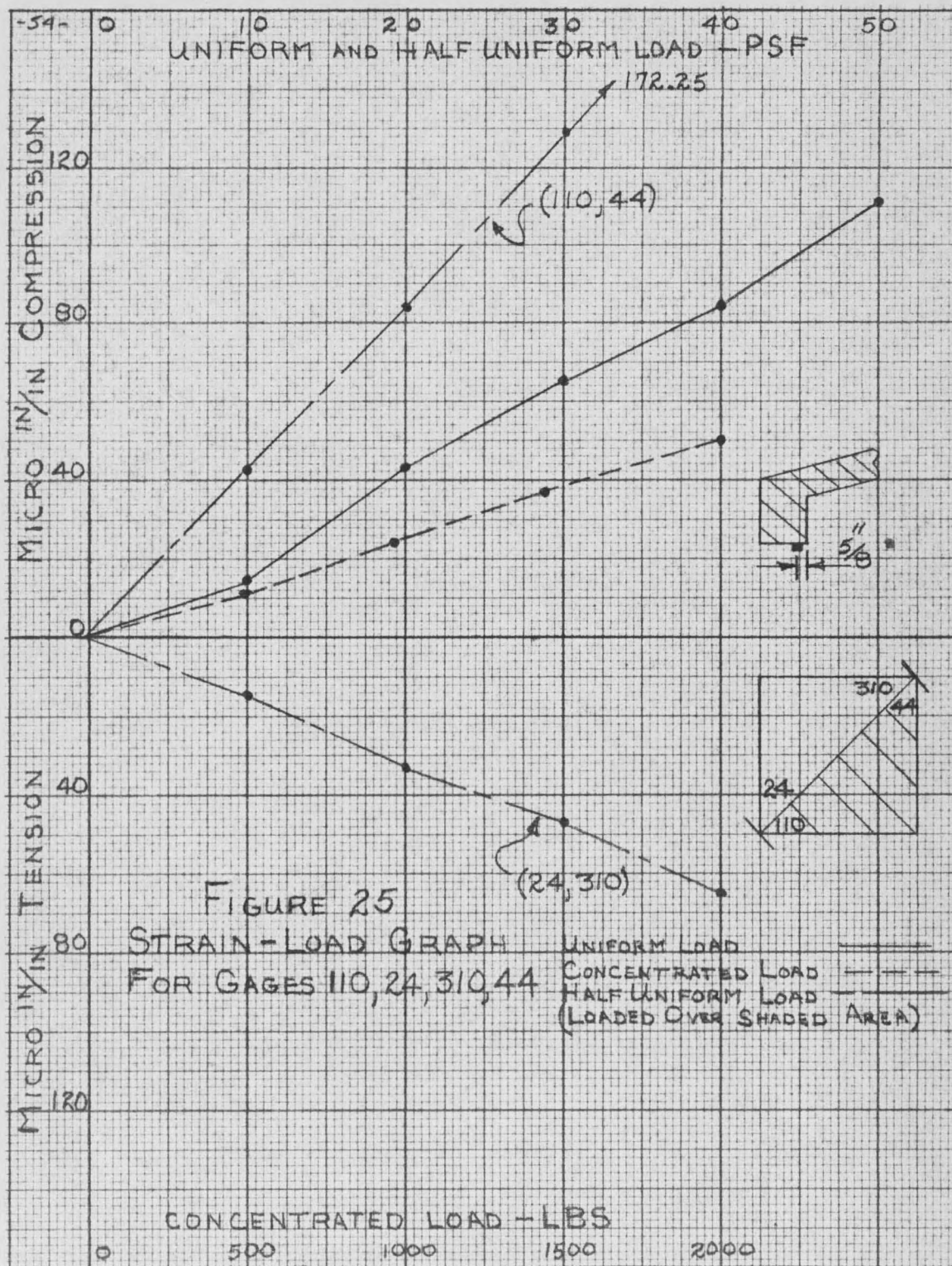


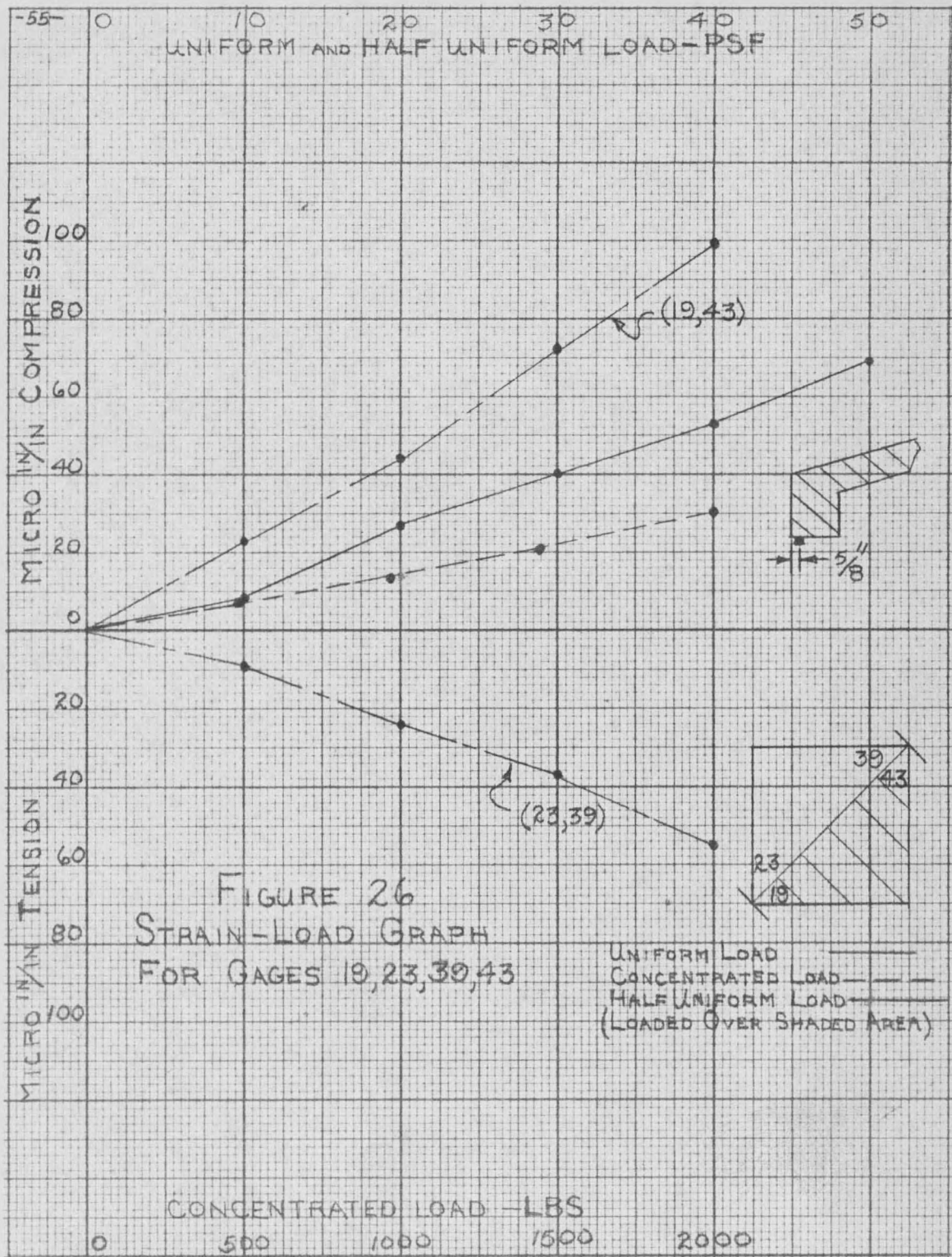


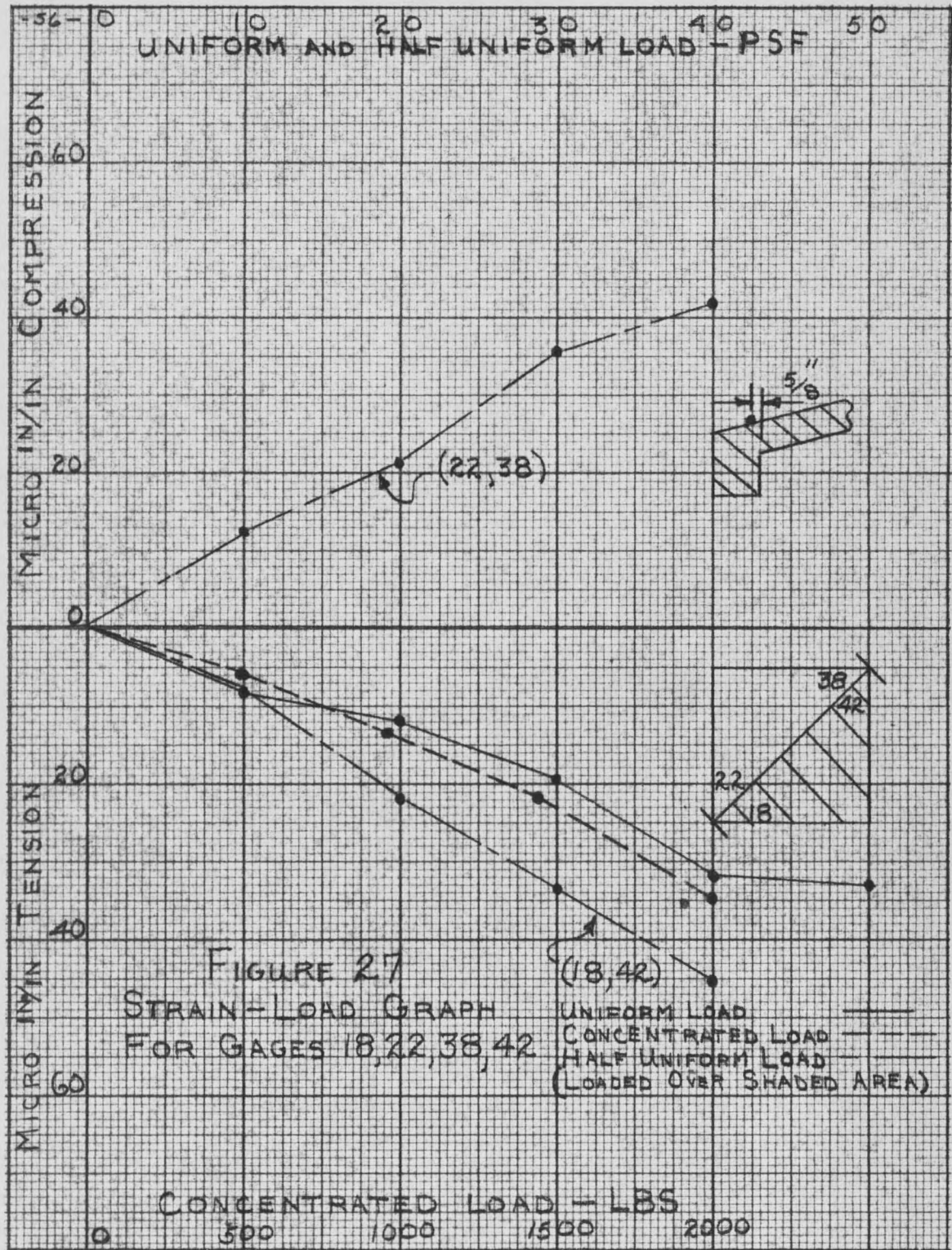


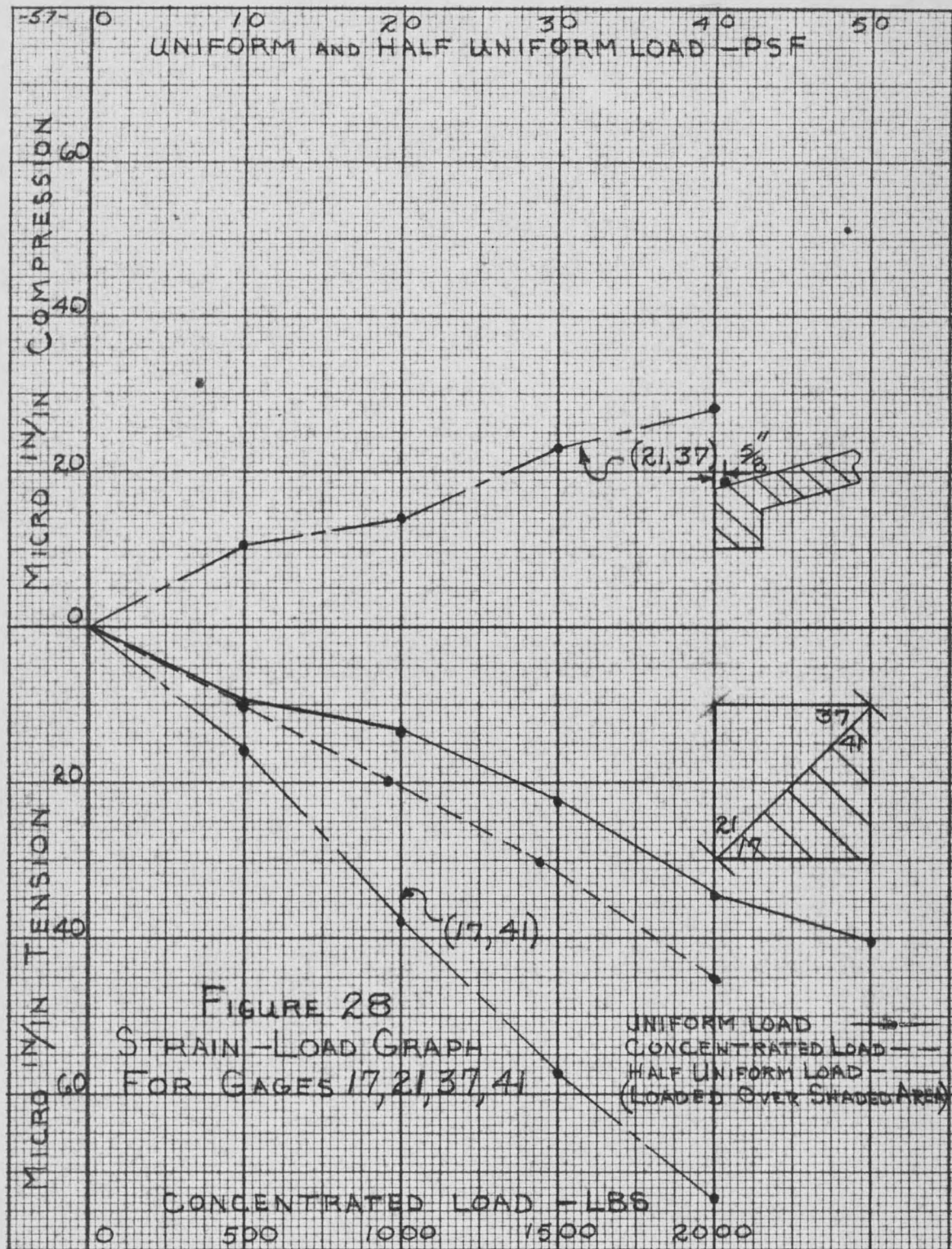


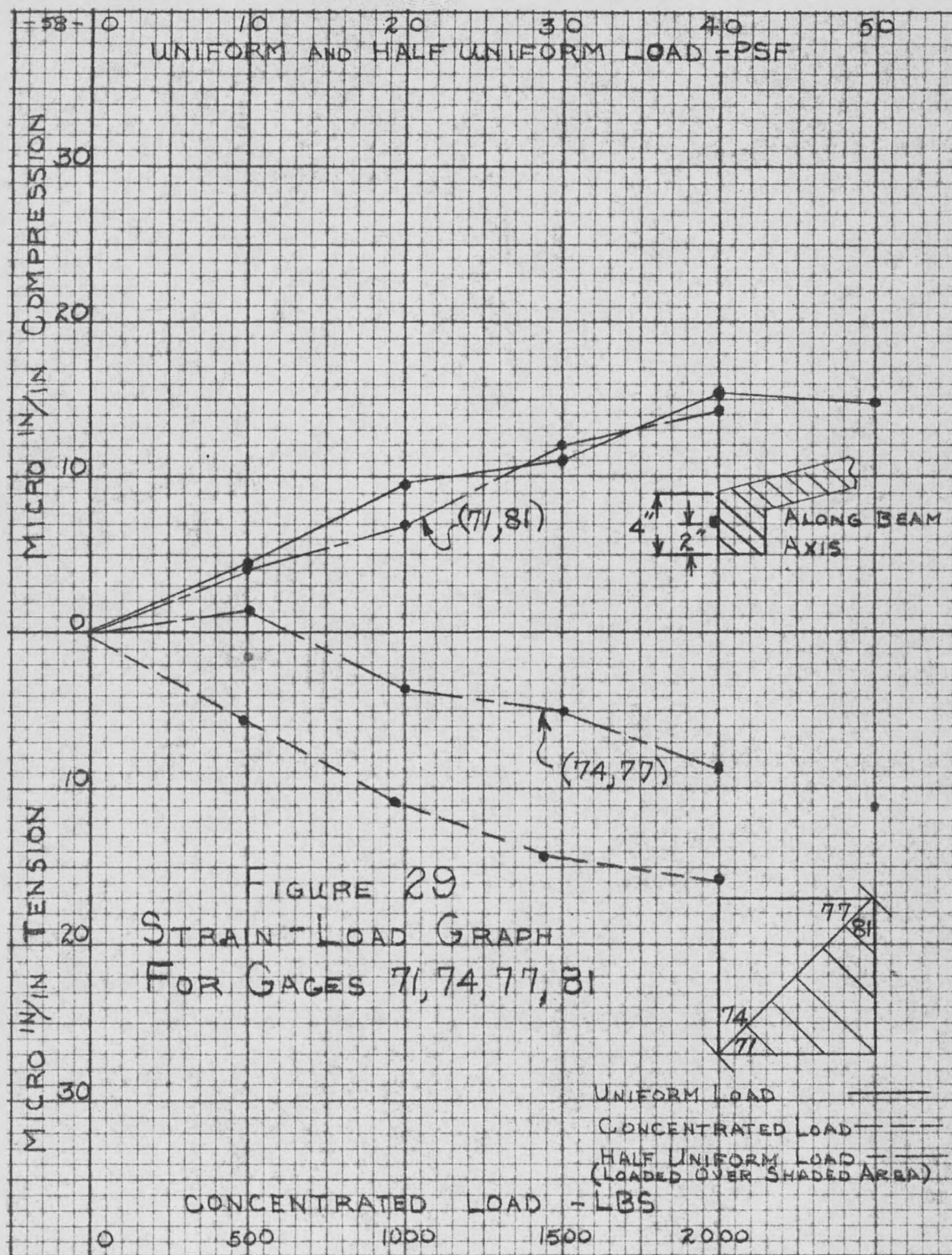












APPENDIX

TABLE II - Uniform Load Data

Gage Number	10 psf	20 psf	30 psf	40 psf	50 psf	50 psf
11	- 6	- 12	- 31	- 36	- 46	- 42
25	- 5	- 16	- 27	- 30	- 36	- 33
31	- 1	- 15	- 27	- 33	- 42	- 47
45	- 2	- 17	- 20	- 24	- 32	- 33
12	- 8	- 23	- 28	- 32	- 39	- 39
26	- 5	- 18	- 25	- 28	- 30	- 35
32	- 3	- 15	- 22	- 25	- 33	- 39
46	- 8	- 22	- 22	- 24	- 37	- 38
14	- 1	- 6	- 8	- 3	- 3	- 4
28	+ 5	- 0	- 1	+ 2	+ 3	- 4
34	0	- 6	- 10	- 8	- 9	- 12
48	+ 1	- 1	0	+ 1	+ 2	- 5
16	+ 1	- 3	- 2	+ 3	+ 6	- 3
29	+ 3	0	+ 1	+ 6	+ 6	0
35	+ 6	+ 1	0	+ 5	+ 6	- 6
49	0	0	+ 2	+ 5	+ 8	- 2
17	+ 9	+ 14	+ 23	+ 39	+ 49	+ 40
21	+ 8	+ 7	+ 15	+ 23	+ 31	+ 22
37	+ 10	+ 16	+ 23	+ 36	+ 45	+ 31
41	+ 10	+ 17	+ 29	+ 41	+ 55	+ 49
18	+ 6	+ 9	+ 16	+ 27	+ 33	+ 20
22	+ 10	+ 9	+ 15	+ 29	+ 35	+ 28
38	+ 10	+ 17	+ 27	+ 42	+ 56	+ 31
42	+ 7	+ 12	+ 20	+ 30	+ 37	+ 23
19	- 7	- 25	- 37	- 47	- 62	- 57
23	- 8	- 28	- 44	- 55	- 70	- 71
39	- 10	- 30	- 46	- 59	- 75	- 83
43	- 9	- 26	- 33	- 48	- 61	- 72
110	- 17	- 49	- 71	- 91	-116	-116
24	- 13	- 39	- 60	- 79	-100	-103
310	- 13	- 34	- 55	- 70	- 92	-105
44	- 18	- 49	- 72	- 96	-127	-128
59	-	-	-	-	-	- 27
510	-	-	-	-	-	- 23

TABLE III - Uniform Load Data

Number	Strain in Micro in/in At Given Uniform Loading					
	10 psf	20 psf	30 psf	40 psf	50 psf	50 psf
13	+ 1	+ 3	+ 9	+ 19	+ 21	+ 4
27	+ 12	+ 10	+ 15	+ 24	+ 26	+ 6
33	+ 10	+ 11	+ 16	+ 23	+ 29	0
47	+ 6	+ 12	+ 19	+ 26	+ 30	+ 6
15	+ 5	+ 3	+ 7	+ 17	+ 22	+ 2
210	+ 3	+ 8	+ 10	+ 20	+ 24	+ 5
36	+ 2	+ 2	+ 7	+ 17	+ 20	- 3
410	+ 2	+ 8	+ 15	+ 20	+ 25	+ 2
71	- 5	- 13	- 15	- 20	- 23	- 19
74	- 3	- 6	- 7	- 9	- 11	- 11
77	- 4	- 9	- 11	- 14	- 13	- 9
81	- 6	- 10	- 11	- 18	- 20	- 11
72	+ 3	+ 5	+ 11	+ 11	+ 16	+ 15
75	+ 8	+ 11	+ 18	+ 26	+ 37	+ 30
78	+ 5	+ 8	+ 14	+ 19	+ 28	+ 27
82	+ 7	+ 12	+ 24	+ 30	+ 41	+ 36
73	- 3	- 5	- 3	- 3	- 1	+ 3
76	+ 4	+ 4	+ 8	+ 8	+ 15	+ 8
79	- 2	- 3	0	0	+ 1	+ 4
83	0	0	+ 7	+ 5	+ 7	+ 8
84	-	-	-	-	-	+ 12
85	-	-	-	-	-	- 7
86	-	-	-	-	-	+ 6
51	+ 1	- 3	+ 5	+ 8	+ 15	+ 8
61	- 11	- 15	- 12	- 5	0	+ 3
53	0	- 4	+ 1	+ 7	+ 12	- 5
63	- 1	- 4	0	+ 8	+ 12	+ 5
52	+ 4	- 4	+ 1	+ 6	+ 7	- 4
62	- 7	- 13	- 6	- 3	- 2	- 2
54	+ 1	0	+ 7	+ 16	+ 20	+ 2
64	+ 1	- 1	+ 6	+ 7	+ 17	+ 8

TABLE IV

Gage Number	10 psf	20 psf	30 psf	40 psf	50 psf	50 psf
55	- 6	- 16	- 8	- 6	- 7	- 14
65	- 10	- 12	- 4	- 2	+ 3	- 7
57	- 3	- 12	- 8	- 5	- 2	- 17
67	- 6	- 11	0	+ 2	+ 5	- 7
56	- 7	- 15	- 9	- 8	- 8	- 20
66	- 6	- 11	- 5	- 8	- 4	- 14
58	- 2	- 4	+ 2	+ 8	+ 15	- 4
68	- 2	- 4	+ 4	+ 10	+ 16	+ 6
Dial Gage #1	+1.5	+3.6	+6.0	+8.9	+11.4	
	+2.3	+4.8	+7.3	+8.5	+10.4	+7.0
	+2.0	+4.5	+7.0	+9.5	+12.5	
Dial Gage #2	+1.5	+2.5	+5.0	+7.8	+11.3	
	+1.0	+2.4	+4.1	+6.1	-	+9.3
	+2.0	+5.0	+7.0	+9.0	+10.0	

* Dial Gage readings are in thousandths of an inch with plus indicating a downward deflection.

* Minus preceeding the strain readings indicates compression and plus indicates tension

* The two identical load columns represent separate loading. The first belongs with the increment loads, the second was loaded all at once.

TABLE V - Concentrated Center Load Data

Number	Strain in Micro in/in At Given Concentrated Load				
	480 lbs	960 lbs	1440 lbs	2000 lbs	2000 lbs
11	+ 1	- 4	- 11	- 17	- 22
25	- 4	- 8	- 13	- 17	- 13
31	- 3	- 10	- 17	- 23	- 22
45	- 4	- 10	- 13	- 17	- 16
12	+ 1	- 4	- 12	- 20	- 22
26	- 4	- 8	- 15	- 17	- 19
32	- 5	- 11	- 16	- 23	- 23
46	- 4	- 8	- 13	- 17	- 17
14	+ 14	+ 20	+ 28	+ 40	+ 34
28	+ 11	+ 20	+ 31	+ 42	+ 40
34	- 1	+ 3	+ 11	+ 21	+ 26
48	+ 5	+ 11	+ 17	+ 26	+ 26
16	+ 3	+ 10	+ 15	+ 21	+ 26
29	+ 7	+ 13	+ 21	+ 27	+ 29
35	+ 2	+ 7	+ 15	+ 24	+ 30
49	+ 6	+ 11	+ 17	+ 26	+ 28
17	+ 9	+ 21	+ 32	+ 43	+ 48
21	+ 9	+ 16	+ 24	+ 38	+ 35
37	+ 4	+ 15	+ 23	+ 33	+ 38
41	+ 17	+ 28	+ 42	+ 63	+ 66
18	+ 1	+ 8	+ 13	+ 21	+ 27
22	+ 9	+ 18	+ 24	+ 39	+ 32
38	+ 5	+ 15	+ 29	+ 44	+ 47
42	+ 9	+ 13	+ 21	+ 34	+ 36
19	- 11	- 17	- 22	- 31	- 24
23	- 2	- 5	- 10	- 17	- 16
39	- 6	- 14	- 22	- 35	- 24
43	- 10	- 17	- 28	- 30	- 24
110	- 15	- 31	- 47	- 66	- 54
24	- 7	- 15	- 24	- 39	- 30
310	- 9	- 19	- 31	- 46	- 39
44	- 15	- 32	- 48	- 62	- 59
59	+ 2	- 1	+ 1	+ 7	+ 9
510	+ 1	- 4	+ 2	+ 3	+ 9

TABLE VI - Concentrated Center Load Data

Number	Strain in Micro in/in At Given Concentrated Load				
	480 lbs	960 lbs	1440 lbs	2000 lbs	2000 lbs
13	+ 9	+ 9	+ 9	+ 10	+ 7
27	+ 3	+ 4	+ 5	+ 11	+ 7
33	+ 1	+ 2	+ 1	+ 4	+ 6
47	+ 2	+ 4	+ 5	+ 10	+ 8
15	- 2	0	0	- 4	- 3
210	- 1	0	- 1	- 1	- 2
36	- 2	- 1	0	- 5	- 1
410	- 2	- 3	- 2	- 4	- 6
71	+ 4	+ 9	+ 11	+ 19	+ 9
74	+ 6	+ 12	+ 18	+ 22	+ 12
77	+ 6	+ 13	+ 18	+ 20	+ 23
81	+ 6	+ 9	+ 10	+ 13	+ 7
72	+ 9	+ 18	+ 25	+ 35	+ 27
75	+ 8	+ 20	+ 30	+ 43	+ 35
78	+ 9	+ 19	+ 28	+ 37	+ 34
82	+ 12	+ 21	+ 31	+ 45	+ 42
73	+ 2	+ 5	+ 4	+ 8	+ 1
76	+ 3	+ 8	+ 9	+ 10	+ 4
79	+ 3	+ 5	+ 4	+ 5	+ 1
83	+ 2	+ 3	+ 7	+ 7	0
84	+ 7	+ 9	+ 14	+ 14	+ 10
85	+ 4	+ 5	+ 8	+ 6	- 1
86	+ 3	0	0	- 6	- 14
51	- 4	- 2	+ 3	+ 5	+ 3
61	+ 6	+ 8	+ 7	+ 10	+ 2
53	+ 4	+ 2	+ 5	+ 6	+ 5
63	+ 3	+ 6	+ 4	+ 8	+ 4
52	0	- 3	- 6	- 10	+ 7
62	+ 5	+ 6	+ 6	+ 6	+ 7
54	- 4	- 1	+ 7	+ 13	+ 25
64	+ 6	+ 19	+ 12	+ 17	+ 16

TABLE VII - Concentrated Center Load Data

Gage Number	Strain in Micro in/in At Given Concentrated Load	480 lbs	960 lbs	1440 lbs	2000 lbs	2000 lbs
55	-	1	- 4	0	+ 1	+ 8
65	+	5	+ 9	+ 11	+ 13	+ 10
57	-	5	- 21	- 30	- 43	- 37
67	-	7	- 20	- 37	- 53	- 50
56	-	5	- 15	- 20	- 26	- 18
66	-	3	- 7	- 11	- 18	- 21
58	-	1	- 6	- 1	- 1	+ 7
68	+	3	+ 6	+ 7	+ 9	+ 15
Dial Gage #1	-	2.7	- 5.8	- 9.9	-15.0	-12.5
Dial Gage #2	-	4.2	- 8.8	-12.5	-17.7	-19.2

- * Dial Gage readings are in thousandths of an inch with plus indicating a downward deflection.
- * Minus preceeding the strain readings indicates compression and plus indicates tension.
- * The two identical load columns represent separate loading. The first belongs with the increment loads, the second was loaded all at once.

TABLE VIII - Uniform Load Over Half the Surface Date

Strain in Micro in/in At Given Load Over Half the Surface

Number	10 psf	20 psf	30 psf	40 psf	40 psf
11	- 11	- 18	- 23	- 34	- 36
45	- 8	- 14	- 22	- 29	- 25
25	- 1	+ 5	+ 5	+ 4	- 1
31	- 5	- 5	- 5	- 7	0
12	- 5	- 8	- 12	- 21	- 27
46	- 6	- 13	- 21	- 28	- 26
26	- 3	+ 3	+ 2	+ 3	- 4
32	- 4	- 3	- 5	- 5	0
14	- 1	- 2	- 9	- 15	- 23
48	- 2	- 1	+ 1	+ 1	- 4
28	+ 3	+ 10	+ 11	+ 14	+ 7
34	- 9	- 11	- 13	- 14	+ 1
16	- 1	- 1	- 3	- 6	- 12
49	- 2	- 1	+ 1	+ 1	0
29	+ 3	+ 7	+ 7	+ 9	+ 4
35	- 7	- 6	- 9	- 9	+ 4
17	+ 12	+ 32	+ 49	+ 69	+ 56
41	+ 20	+ 44	+ 66	+ 89	+ 80
21	- 9	- 9	- 19	- 30	- 27
37	- 12	- 19	- 27	- 34	- 21
18	+ 6	+ 19	+ 30	+ 44	+ 37
42	+ 10	+ 25	+ 37	+ 50	+ 50
22	- 9	- 9	- 20	- 32	- 33
38	- 16	- 33	- 51	- 63	- 39
19	- 23	- 42	- 66	- 100	- 94
43	- 23	- 46	- 78	- 106	- 97
23	+ 11	+ 31	+ 45	+ 67	+ 56
39	+ 7	+ 18	+ 29	+ 45	+ 51
110	- 41	- 79	- 125	- 176	- 163
44	- 44	- 88	- 133	- 187	- 163

TABLE IX - Uniform Load Over Half the Surface Data

Gage Number	Strain in Micro in/in At Given Load Over Half The Surface				
	10 psf	20 psf	30 psf	40 psf	40 psf
24	+ 16	+ 38	+ 55	+ 79	+ 69
310	+ 13	+ 29	+ 39	+ 59	+ 53
59	- 3	- 8	- 14	- 20	- 18
510	+ 4	+ 7	+ 8	+ 7	+ 10
13	+ 1	+ 4	+ 4	+ 1	- 6
47	+ 3	+ 6	+ 7	+ 7	+ 1
27	- 2	+ 5	+ 5	+ 6	+ 2
33	- 5	- 4	- 4	- 4	+ 2
15	+ 3	+ 6	+ 10	+ 14	- 1
410	- 4	- 3	+ 2	+ 4	+ 2
210	- 1	+ 6	+ 3	+ 2	- 3
36	- 11	- 12	- 13	- 12	- 2
71	- 6	- 9	- 15	- 20	- 18
81	- 2	- 5	- 9	- 10	- 9
74	0	+ 4	+ 5	+ 8	+ 9
77	- 3	+ 3	+ 5	+ 9	+ 10
72	+ 4	+ 12	+ 16	+ 19	+ 21
82	+ 10	+ 20	+ 27	+ 39	+ 42
75	- 2	- 3	- 5	+ 3	- 1
78	- 2	0	+ 1	+ 4	+ 7
73	+ 2	+ 7	+ 11	+ 13	+ 14
83	+ 4	+ 8	+ 13	+ 18	+ 18
76	0	- 2	- 5	- 4	- 7
79	- 3	- 2	- 4	- 3	0
84	+ 3	+ 7	+ 5	+ 8	+ 6
85	+ 1	+ 6	+ 5	+ 10	+ 6
86	0	+ 2	+ 2	+ 2	- 4
51	+ 12	+ 16	+ 20	+ 22	+ 5
53	- 2	- 2	- 1	+ 1	0

TABLE X. - Uniform Load Over Half the Surface Data

Gage Number	Strain in Micro in/in At Given Load Over Half The Surface				
	10 psf	20 psf	30 psf	40 psf	40 psf
61	- 3	- 2	+ 1	+ 2	- 7
63	- 1	- 3	- 3	- 1	- 2
52	+ 8	+ 5	- 2	- 5	- 12
54	+ 7	+ 11	+ 15	+ 18	+ 10
62	- 2	+ 1	+ 6	+ 9	+ 4
64	- 2	- 3	- 1	- 2	- 2
55	- 1	0	- 1	- 5	- 10
66	- 6	- 8	- 7	- 9	- 14
57	- 2	0	0	0	- 2
67	- 3	- 5	- 7	- 12	- 11
56	+ 1	+ 2	+ 1	0	- 9
66	- 6	- 8	- 7	- 12	- 14
58	0	+ 2	+ 2	+ 5	+ 3
68	0	+ 1	+ 6	+ 8	+ 5
Dial Gage #1	+15.0	+30.0	+45.0	+60.0	+60.0
Dial Gage #2	-13.0	-26.4	-40.0	-55.0	-56.0

- * Dial Gage readings are in thousandths of an inch with plus indicating a downward deflection.
- * Minus preceeding the strain readings indicates compression and plus indicates tension.
- * The two identical load columns represent separate loading. The first belongs with the increment loads, the second was loaded all at once.

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