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AN INVESTIGATION OF THE BEARING

CAPACITY OF A JOINTED ROCK SYSTEM

A THESIS

Presented to

The Faculty of the Graduate Division

by

George Frederick von Kolnitz, IV

In Partial Fulfillment

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Master of Science in Civil Engineering

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AN INVESTIGATION OF THE BEARING

CAPACITY OF A JOINTED ROCK SYSTEM

Approved:

Chairman

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol	
ϕ	Angle of internal friction
α	Semi-wedge angle
c	Cohesion
p_h	Horizontal splitting pressure
p_t	Bending tensile strength
M	Bending moment
C	Distance from neutral axis to edge of a section
I	Moment of inertia of a section
q	Unit bearing capacity
Q	Failure load
γ	Unit weight
b	Footing width
t	Rock layer thickness
W	Width of individual rock block
H	Height of system

SUMMARY

The purpose of the research was to investigate the bearing capacity of a jointed model rock system when loaded by a model footing. To simulate the jointed rock system, small blocks (4 in. x 4 in. x 1 in.) were cut from Indiana Limestone and arranged in a brickwork fashion one foot square and four inches thick. The system was confined by a bottomless plywood box. The model footings were cut from steel and ranged in size from 1.25 inches square to 6 inches square and thick enough to be rigid. The tests were performed by loading the footings which were placed at various positions on the system. The load was applied at a deformation rate of 0.15 inches per minute.

The main purpose of this research was satisfied in that a definite trend for the bearing capacity as a function of joint spacing was established for the range of testing. The conclusions reached for this series of tests follow:

(1) There was no significant transfer of stress across the discontinuities. The only blocks affected were those directly beneath the footing.

(2) Based on the above statement and results, no attempt should be made to analyze the bearing capacity of a jointed rock system by the general bearing capacity equation.

(3) The bearing capacity of the jointed system can be conveniently predicted by a simple modification to the Meyerhof equation for the bearing capacity of rock. The modification reduced the bearing capacity

to account for the horizontal discontinuities in the failure zone.

(4) a. <u>Small footings</u>: When the footing is small compared to the block size, the bearing capacity with the footing at the edge of a block is slightly higher than with the footing at the center. Further, there is a significant drop in the bearing capacity when the footing is moved to the corner of a block and over a discontinuity.

(4) b. <u>Large footings</u>: When the footing size approaches the block size, position of the footing affects the bearing capacity very little until a discontinuity is covered. This results in a significant drop in the bearing capacity.

(5) Failure occurs in a splitting manner followed by a punching out of lower blocks.

(6) Settlement depends greatly upon the tightness of the packing of individual blocks and would be most difficult to predict.

CHAPTER I

INTRODUCTION

Rock, the ultimate base of engineering structures, is probably the least investigated of any structural component. For many years, great effort has been expended to minimize cost and quantity of engineering materials in a structure by considering every conceivable stress condition to which the structure may be subjected. Recently, considerable effort has been expended in the field of soil mechanics. Early in the 20th Century, the importance of soil as the immediate foundation material of most structures was recognized. This was the actual beginning of that separate branch of civil engineering known as soil mechanics. Even more recently, the importance of rock as a foundation material has been recognized. This is not to infer that rock mechanics was neglected. The earlier structures in general did not exert an excessive stress on any rock mass. Therefore, rock mechanics was really not necessary. There are circumstances, such as tunneling, where the importance of stress conditions of the rock have been recognized for quite some time. The intricate network of shafts and tunnels of the mining industry require that accurate determination of the strength of rock be made. In addition, the problem of determining the stability of rock slopes along roadway cuts and in the area of arch dams has been recognized.

As the need for more accurate analysis increased, investigators began to attempt to duplicate field conditions in the laboratory. Possibly the main shortcoming was pointed out by Klaus W. John (1) who stated in effect that:

> The technological properties of a rock mass depend far more upon the system of geological separations within the mass than on the strength of the rock material itself. Therefore, rock mechanics is to be a mechanics of a discontinuum, that is, a jointed system.

John also hypothesized that the deformability or settlement expected from a rock mass results primarily from displacements of the unit blocks and not from deformity of the block itself.

Most laboratory tests on rocks have been performed on an intact rock mass devoid of macrogeological weaknesses such as joints which would appear in a prototype. This has probably been done for ease of analysis.

The purpose of this investigation was to simulate a simple jointed rock system in the laboratory and to analyze the failure characteristics with respect to bearing capacity using as strength parameters the cohesion and internal friction of the material. To simulate the system, the writer cut four-inch square blocks, one inch thick, and placed them in a bottomless plywood container. The system was one foot square and four inches deep. The idealized jointed or fractured system was loaded with model footings much like those which would be used to analyze the bearing capacity of soils.

As stated, most previous tests have been on intact rock masses. In addition, it seems that most tests on rocks have been some form of triaxial test rather than direct loading with a footing.

A notable exception to this was the work by G. G. Meyerhof (2) who investigated the bearing capacity of small concrete and rock blocks when loaded with a model footing.

According to Meyerhof,

At the bearing capacity (q) of a strip footing of width (b), resting on a block of thickness (H), and width (L) greater than or equal to H), the horizontal splitting pressure (p_h) can (in accordance with Coulomb-Mohr theory) be shown to be

$$p_h = q \tan^2\alpha - 2c \tan\alpha \qquad (1)$$

where the semiwedge angle $\alpha = 45 - \phi/2$. [See Figure 1.] The maximum bending tensile stress at the point of the wedge of material below the footing is

$$p_t = (1 + \frac{6H}{2H - b \cot\alpha})(\frac{b \cot\alpha}{2H - b \cot\alpha})p_h \qquad (2)$$

substituting for p_h from (1) into (2) and simplifying gives

$$q = \frac{(\frac{2H}{b} - \cot\alpha)^2(\cot\alpha)p_t}{\frac{8H}{b} - \cot\alpha} \qquad (3)$$

The intention of this writer was to verify this equation, modify it as necessary, or establish a new one, to arrive at some suitable means to predict the safe bearing capacity of a jointed system consisting of thin rock layers jointed in both the horizontal and vertical direction. The term "thin layer" as used here is meant to signify a rock layer thickness which does not exceed the width of the footing.

It would be well to note here that if the horizontal expanse of the rock mass (Figure 2) is great in relation to the size of the footing

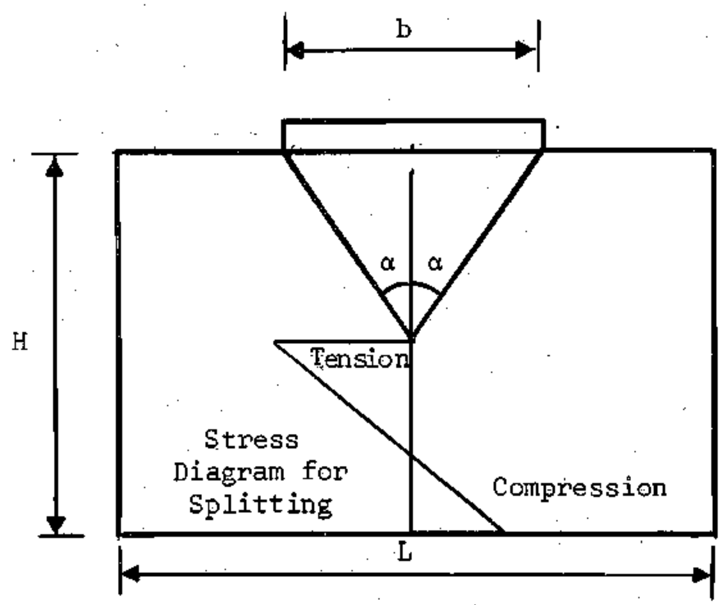


Figure 1. Failure of a Small Block, after Meyerhof.

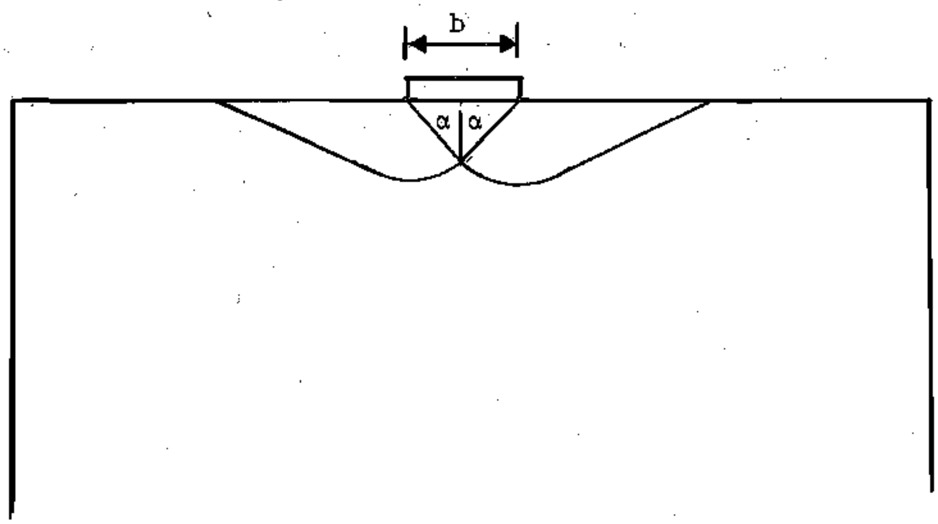


Figure 2. Bearing Failure of a Large Block, after Meyerhof.

or where splitting is prevented by reinforcement, the bearing capacity depends upon the shear strength of the material. A wedge is formed at failure and the material at the side is forced upward and outward along a curved shearing surface. This is similar to the bearing capacity failure in a soil mass and can probably be represented by the general bearing capacity equation (8)

$$q = c\ N_c$$

where N_c is the general bearing capacity factor.

The previous discussion was included mainly to show the reader that at least two different modes of failure are possible in rock. Since the individual blocks were small in relation to the footing and the individual blocks were in no way tied together, this type failure was not expected in this study.

In addition to the Meyerhof analysis, it was planned to calculate the predicted bearing capacity by the general bearing capacity equation (8).

$$q_o = \frac{\gamma b}{2} N_\gamma + c\ N_c$$

where γ = unit weight

b = footing width

N_c and N_γ = bearing capacity factors

The bearing capacity factors are dimensionless coefficients which depend upon the angle of internal friction and shape of the failure zone assumed by the investigator. It should be pointed out here that a general bearing capacity failure must take place for this analysis to be valid.

CHAPTER II

ROCK DESCRIPTION, EQUIPMENT, AND PROCEDURE

Rock Description

General

The rock used in this project was Indiana Limestone, a commercial product of the Indiana Limestone Company of Bedford, Indiana. The rock was obtained locally from the Sherwood Cut Stone Company of Atlanta. Perhaps the best way to describe the Indiana Limestone used in this project is to quote from the specifications pamphlet of the Indiana Limestone Company (5).

> Indiana Limestone is the type of rock termed by geologists as Oolitic Limestone. It is a calcite cemented calcareous stone formed of shells and shell fragments, practically non crystalline in character. It is characteristically a free stone without cleavage plane, possessing a remarkable uniformity of composition, texture, and structure and equality of strength in all directions regardless of the plane of its natural bedding.
>
> The average analysis (in per cent) as developed by carefully prepared composite samples is given below.

Carbonate of Lime	97.39
Carbonate of Magnesia	1.20
Silica	.69
Alumina	.44
Iron Oxide	.18
Water and Loss	.10
Total	100 %

> The average weight of dry (seasoned) Indiana Limestone is 144 pounds per cubic foot.

Webster's New Collegiate Dictionary (7) defines the term "free stone" as "any stone, but especially a sandstone or limestone, that may be cut freely without splitting."

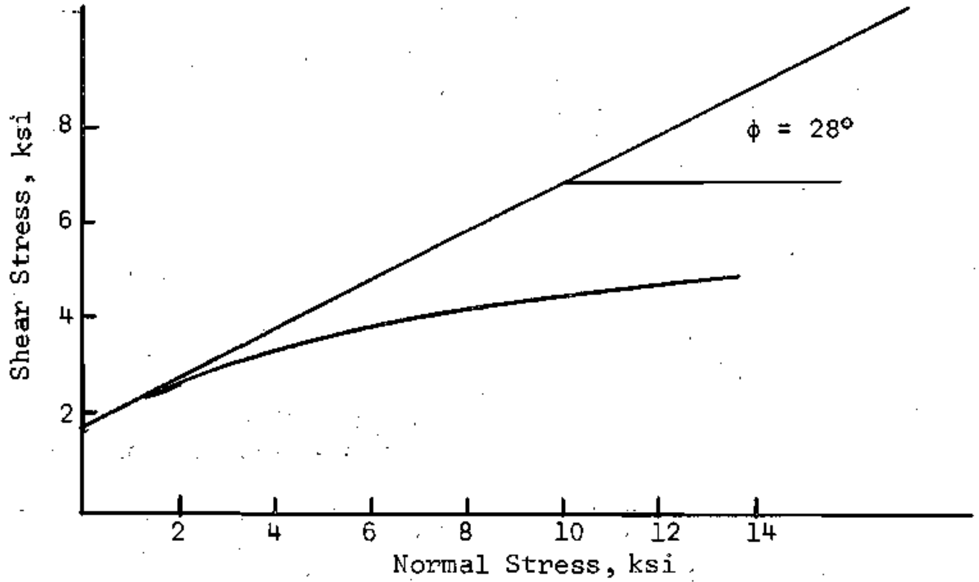
Anderegg (6) page 11, further describes the stone as having a compressive strength of from 6000 psi to 7000 psi.

Physical Properties

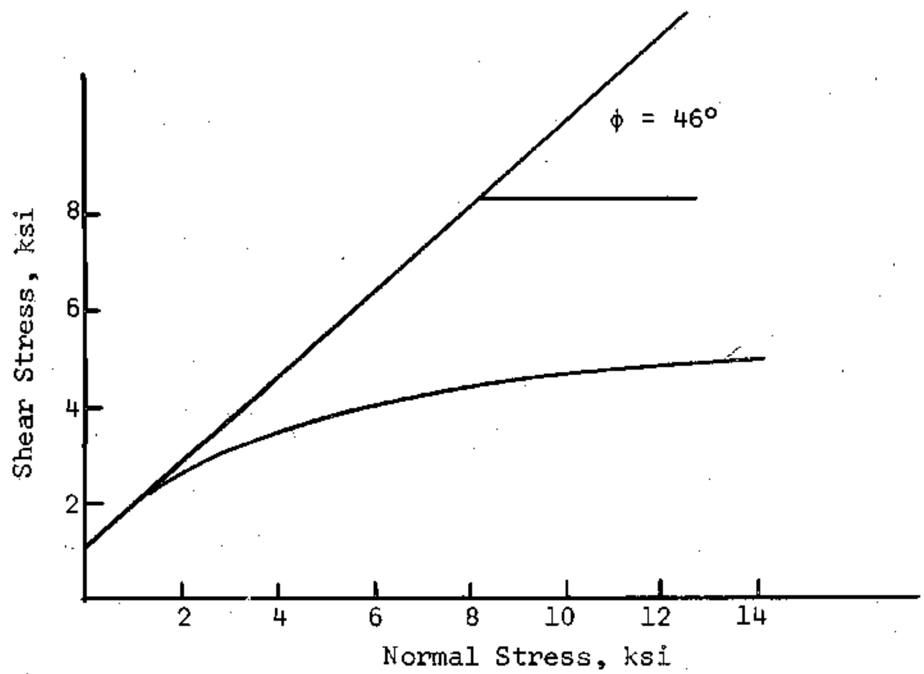
For the determination of the value of cohesion (c) and angle of friction (ϕ), this writer was fortunate enough to have access to Mohr Circles for Indiana Limestone presented by Schwartz (3) and Robertson (4). Both of these investigators have performed triaxial tests on Indiana Limestone and have presented the results in the form of Mohr Circles. The values of "c" and "ϕ" were obtained by drawing a tangent to the circles in the lowest range of confining pressure (Figure 3). For reference, the values are shown below.

Investigator	Cohesion (c)	Internal Friction (ϕ)
Schwartz	1,100 psi	46°
Robertson	1,800 psi	28°

A portion of this difference can be attributed to the human judgment factor involved in drawing the tangent line to the circles. The remainder of the difference can be attributed to differences inherent in the rock. It is well to note here that while there is an increase in the value of cohesion in the work of Robertson, there is also a corresponding decrease in the angle of internal friction. The end result of this is that the predicted bearing capacity is affected only slightly by changing from one to the other (see Figures 3 and 6). This shows the average difference between the predicted bearing capacities to be about



Mohr Envelope, after Robertson



Mohr Envelope, after Schwartz

Figure 3. Mohr Envelopes for Indiana Limestone.

250 psi. Most of the predicted bearing capacity results from the term 2cCotα in Equation 3. The decrease in "ϕ" with the increase in "c" results in an increased α angle ($45 - \phi/2$). In addition, the cotangent function of an angle decreases as the angle increases. Therefore, the increase in the term "c" in conjunction with a decrease in the term "cotα" results in little change in the predicted bearing capacity.

It is the opinion of this writer that the values shown by Robertson more closely approximate the values of "c" and "ϕ" in the rock tested. This is based on the fact that the observed angle $\alpha=(45-\phi/2)$ was in very close agreement with that obtained using Robertson's values (see Figure 11).

As a limiting extremity, the predicted bearing capacity was analyzed considering the unconfined case where "ϕ" is assumed zero and "c" is 3,400 psi. These results are still in good agreement with those actually measured (see Figure 6). The unconfined case was taken from an average of the unconfined circles shown by Schwartz and Robertson.

It was necessary to determine the bending tensile strength (p_t) for use in Meyerhof analysis. This value was determined by loading a simple beam at the one-third points and calculating the bending tensile strength by the equation:

$$p_t = \frac{MC}{I} \tag{4}$$

where p_t = bending tensile strength

M = maximum bending moment at failure

C = distance from natural axis to the extreme edge

I = moment of inertia of the section

This strength was used in analysis of results.

Also, Schwartz (3) performed direct tensile tests on the Indiana Limestone and found a tensile strength of slightly under 400 psi. While this is 20 per cent less than that determined by the beam test, the effect on the resulting prediction is very small. The term in Equation 3 using "p_t" accounts for less than 5 per cent of the total predicted bearing capacity.

Equipment

The main piece of apparatus used in this test was a standard compression testing machine. In order to achieve a greater accuracy, two different machines were used, depending upon the magnitude of load desired. The first, a Tinius Olsen machine, had a capacity of 20,000 pounds; the second, a Riehle machine, had a capacity of 475,000 pounds. A bottomless plywood box was used to confine the jointed rock system. To determine the amount and significance of stress transferred to the plywood, a flat, disc-shaped load cell was placed between the wall of the container and the rock system. To eliminate the edge effect of loading the cell, a thin, small diameter metal piece was centered on the cell and placed against the rock system. Strain readings were taken on an SR-4 strain indicator which was calibrated to a load of 17 pounds. The strain indicator was preloaded to a load of about 9 pounds. The strain readings showed that there was no significant transfer of stress across the discontinuity to the container.

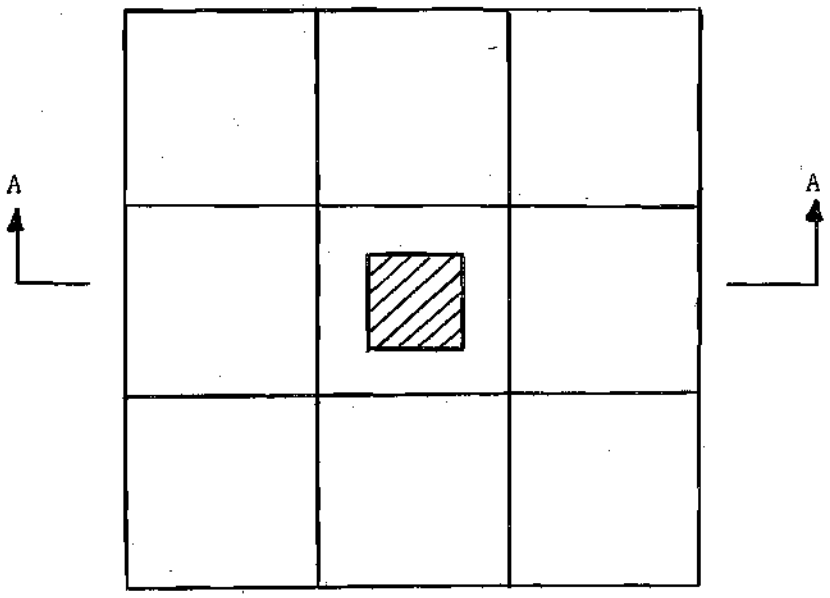
The footings used were cut from steel and were sized large enough so that there was no significant deflection of the footing during loading. In addition, the entire footing was covered with a steel loading cap. Settlement readings were made with a Starrett dial gage accurate to 0.0005 inches.

Procedure

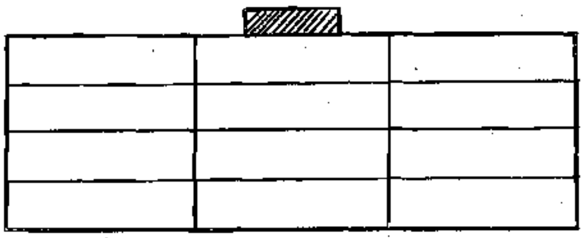
The Indiana Limestone used in the testing was cut into four-inch squares which were one-inch thick. These were arranged in a brickwork system one foot square and four inches thick to simulate a jointed rock. The dial gage was placed so that settlement readings of the footing were recorded. On selected tests, other dial gages were placed at various other positions on the surface of the rock system to note possible vertical movement during loading on the footing. Results of this are discussed later. Square footings of the following sizes were used in the tests: 1.25 inches, 2.0 inches, 3.0 inches, 4.0 inches, and 6.0 inches.

The first series of tests was performed with the footings centered on the center block of the system (Figure 4). Subsequent tests were performed with the footings on the edge and on a corner of the center block (Figure 5). Naturally, the four-inch footing could not be used in these tests since any movement of this footing would cover a vertical discontinuity. Further tests were performed with the footings centered over two blocks of the system and then centered over four blocks.

It should be noted here that the square blocks used in the tests were cut from larger rocks with a diamond saw. A tolerance of 1/32 inch was allowed on all dimensions. Blocks with a greater deviation than the



Plan View



Section A-A

Figure 4. Entire Jointed System, Footing in Center.

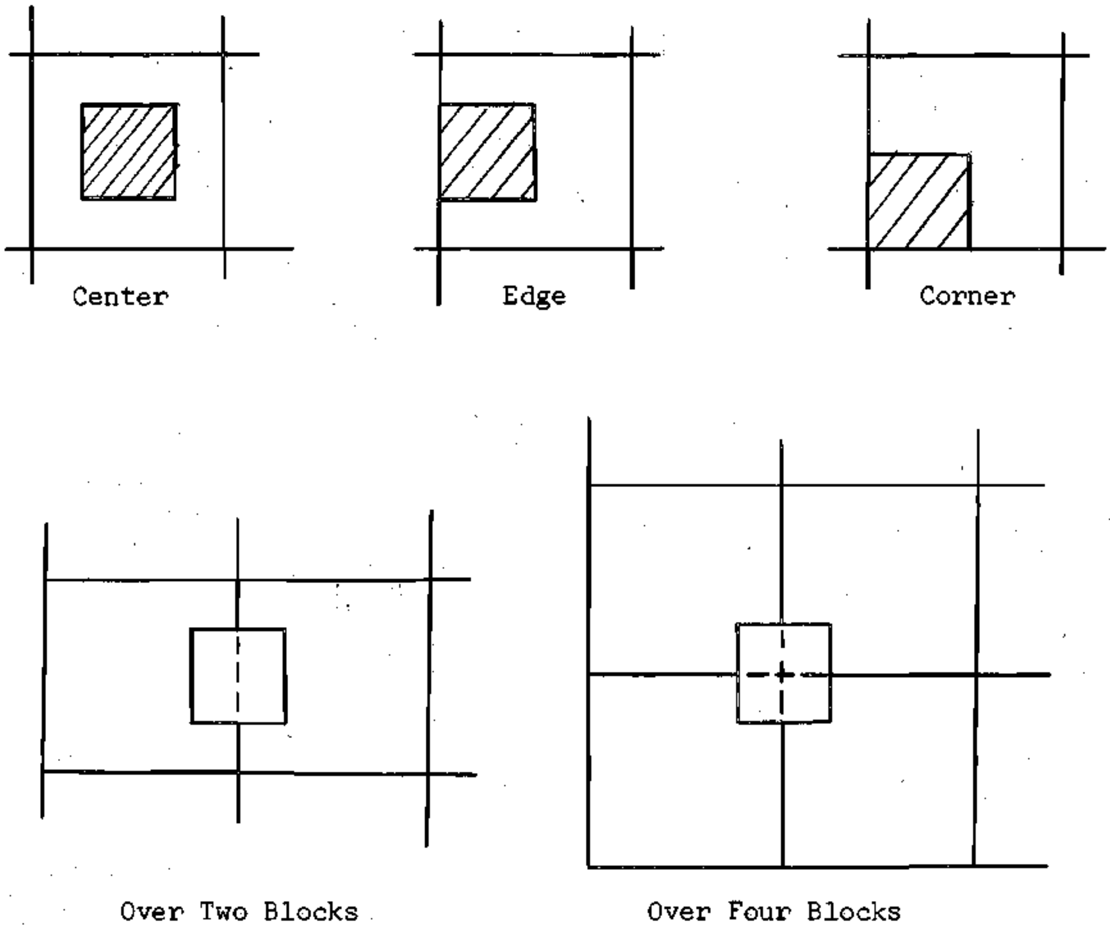


Figure 5. Position of Footings Referred to in the Text.

specified tolerance were discarded. Also, it should be noted that care was exercised to insure the tightest possible stacking of the blocks. Blocks with visual irregularities were not used in the testing.

For purposes of comparison, tests were performed with the footings on individual small blocks (4 in. x 4 in. x 1 in.) as well as solid blocks one foot square by five inches high. The deformation rate applied in each of the tests was 0.15 inches per minute.

Secondary tests of note were carried out to determine the extent of stress transfer across the discontinuities and to determine the bending tensile strength of the rock used. As mentioned previously, a load cell was used to determine the amount of stress transfer. The cell was calibrated to the SR-4 strain indicator by use of static weight, and it was then placed between the wall of the container and the rock system. A small seating load was placed on the apparatus and strain readings were taken as a test progressed to failure.

CHAPTER III

RESULTS AND DISCUSSION

Determination of Lateral Stress

One of the first questions to be resolved was whether there was any adverse effect on the tests from using the plywood container for confinement of the system. As mentioned, this effect was determined by using a load cell and an SR-4 strain indicator. The change in pressure along the wall of the container was recorded as the normal testing progressed. In all cases, there was very little, if any, change in pressure along the wall until after complete failure of the system had occurred. With the test load as high as 7000 pounds, there was a maximum pressure change on the wall of the container of less than one-fourth of a pound. The conclusion reached here was that there was no adverse effect on the test caused by the container. This also showed that there was no significant lateral transfer of stress across the vertical discontinuities or joints in the rock. This observation is further substantiated by the fact that at no time during the testing was any block damaged except those vertically below the footing.

Prediction of Bearing Capacity*

It must be remembered that there was no measured significant transfer of stress across the discontinuity in the rock system. There-

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* See Table 1 for this entire discussion.

fore, it was impossible for the normal bearing capacity failure surface to take shape. This completely rules out the possibility of using any of the standard bearing capacity factors. Since the normal failure surface did not form, it was expected that the predicted bearing capacity by the Terzaghi factors would be excessive. Table 1 shows quite conclusively that these predictions, with both general and local shear, are much too high. It is therefore concluded that the general bearing capacity analysis should never be used to predict the bearing capacity of a fractured or jointed rock system.

Meyerhof (2), on the other hand, has tested the bearing capacity of solid rock masses and has advanced the equation mentioned previously:

$$q_o = \frac{(2\frac{H}{b} - \cot\alpha)^2(\cot\alpha)p_t}{\frac{8H}{b} - \cot\alpha} + 2c \cot\alpha \qquad (3)$$

Since this equation applied to a solid rock mass, it was necessary to incorporate some modification to account for the discontinuities present in the system. It is suggested by the writer that the result of this analysis be modified by the factor t/b where "t" is the rock thickness and "b" is the footing width. The modified equation takes the form

$$q_o = (\frac{2\frac{H}{b} - \cot\alpha)^2(\cot\alpha)p_t}{\frac{8H}{b} - \cot\alpha} + 2c\text{Cot}\alpha)t/b \qquad (5a)$$

or

$$q_o \text{ modified} = (q_o)\ t/b \qquad (5b)$$

Table 1. Comparing Predicted and Actual Bearing Capacity Footings in Center of Block

Footing Size Inches	Terzaghi General Shear psi $\phi = 46°$	Terzaghi General Shear psi $\phi = 0$	Terzaghi Local Shear psi $\phi = 46°$	Terzaghi Local Shear psi $\phi = 0$	Meyerhof $\phi = 46°$ psi
1.25	165,000	18,500	33,000	12,500	5670
2	165,000	18,500	33,000	12,500	5670
3	165,000	18,500	33,000	12,500	5460
4	165,000	18,500	33,000	12,500	5500

Footing Size Inches	Meyerhof $\phi = 46°$ Modified By t/b psi	Meyerhof $\phi = 28°$ psi	Meyerhof $\phi = 28°$ Modified By t/b psi	Meyerhof $\phi = 0°$ psi	Meyerhof $\phi = 0°$ Modified By t/b psi	Observed psi
1.25	4530	6780	5400	7490	5980	4610
2	2840	6320	3200	7200	3600	3500
3	1820	6010	2000	7040	2240	2220
4	1400	6000	1550	6870	1740	1950

As the footing width increases, more discontinuities or joints are brought into the failure pattern so it should follow that the unit bearing capacity should diminish. It must be pointed out here that this modification holds true only when the footing width exceeds the rock thickness. If the reverse were true, the modification would result in an increase in bearing capacity. It should also be pointed out that this modification will probably not hold if the rock layers become very thin as in the case of laminated rock. If the thickness of the layer should approach zero, the predicted bearing capacity would also approach zero. This, obviously, should not occur. As shown in the following discussion, the results of this analysis for the range tested agree very closely with the actual measured bearing capacity.

Footings in the Center of the Block*

This section of the report will discuss the results of tests with the footings at the center of the individual block (Figure 5) and compare these values with those predicted. Figure 7 shows that the bearing capacity of the 1.25-inch footing was 4610 psi with a decrease to 1950 psi for the four-inch footing. The decrease is certainly reasonable since as the footing size is increased, the failure zone includes a greater number of discontinuities which would tend to reduce the bearing capacity. The actual failure patterns are shown in Figures 9 through 11.

In each case, failure cracks occurred from the corner of the footing to the corner of the rock block. Also, as mentioned, it is seen that the failure zone is increased by increasing the footing size. In the case of the six-inch square footing, the formation of a wedge was less apparent. The failure seemed to be more of a crushing failure. Table 1 shows in general a small decrease in unit bearing capacity for the Meyerhof analysis with no modification. This further substantiates the fact

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* See Table 3 and Figure 7.

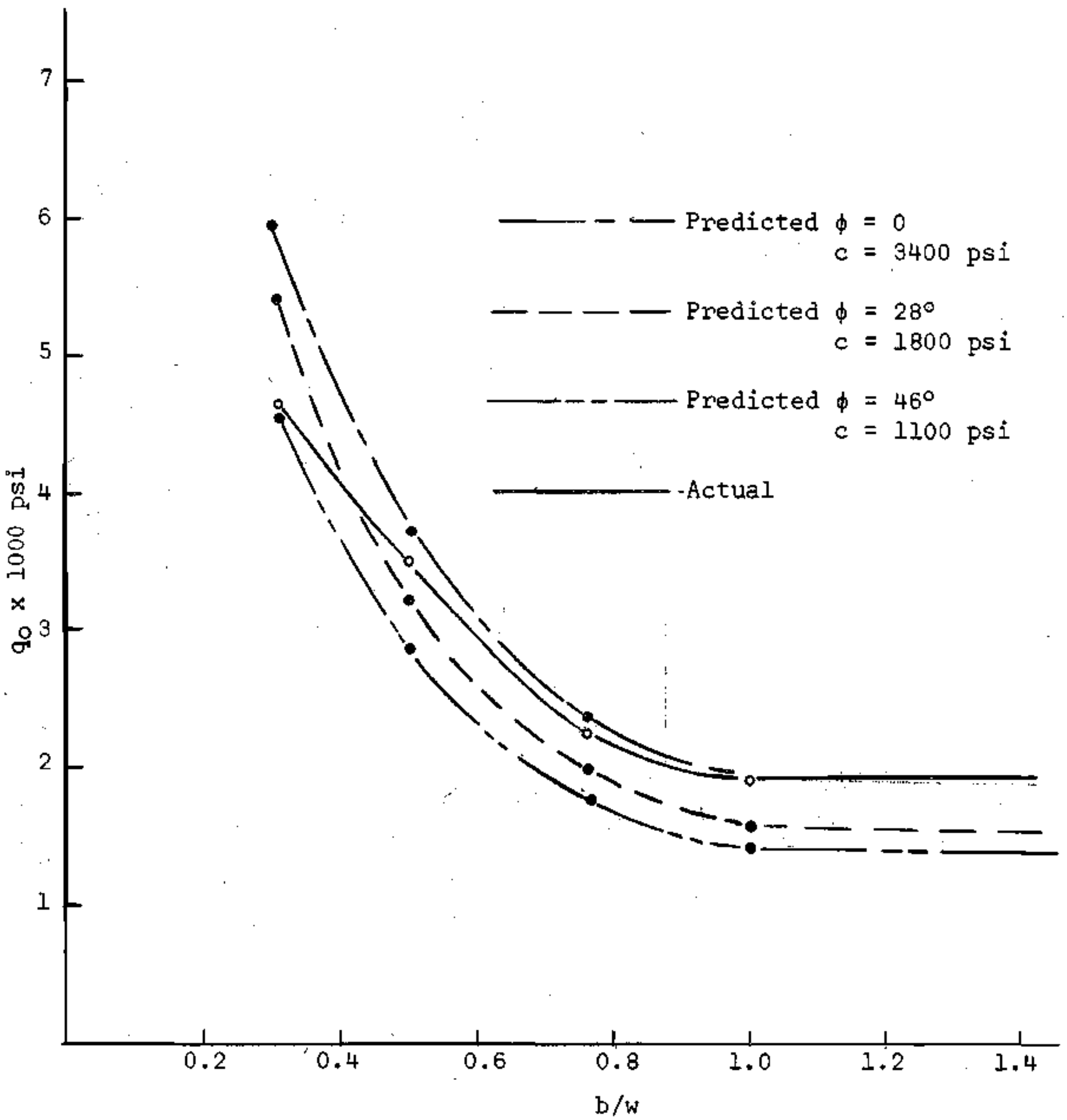


Figure 6. Predicted Unit Load (q_o) vs. b/w Footing in Center.

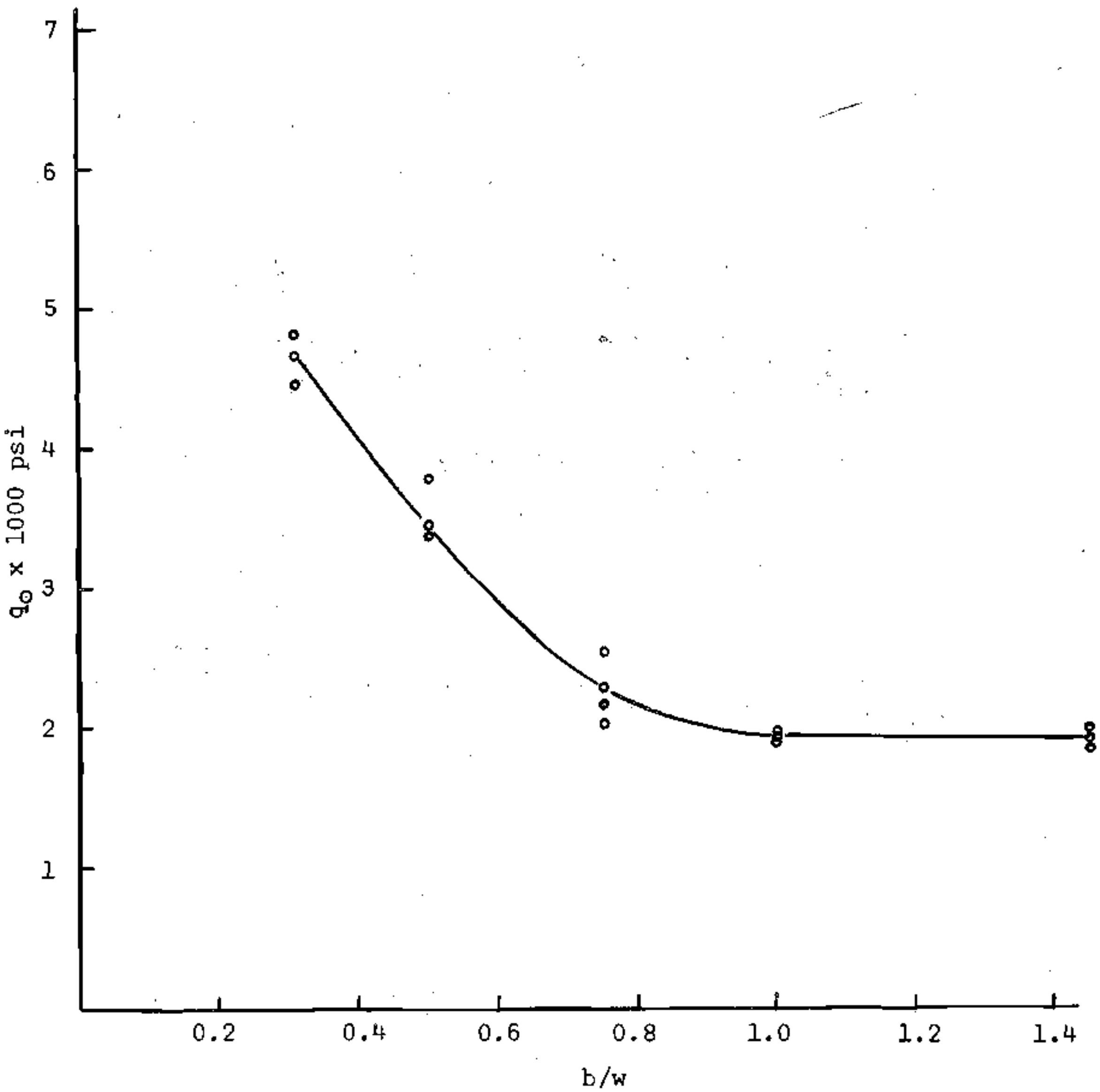


Figure 7. Unit Load (q_o) vs. b/w Footings in Center.

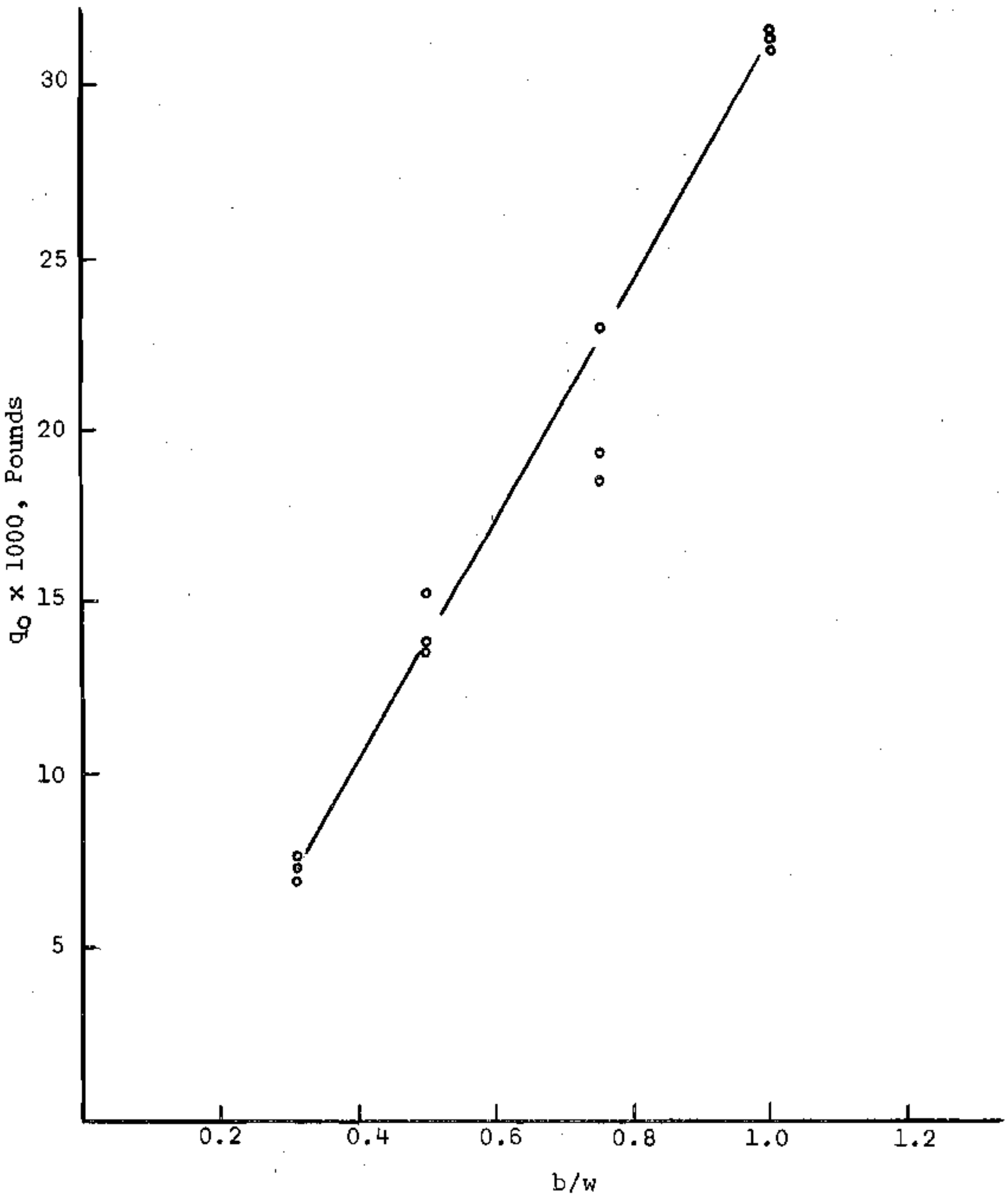


Figure 8. Failure Load (Q) vs. b/w Footing in Center.

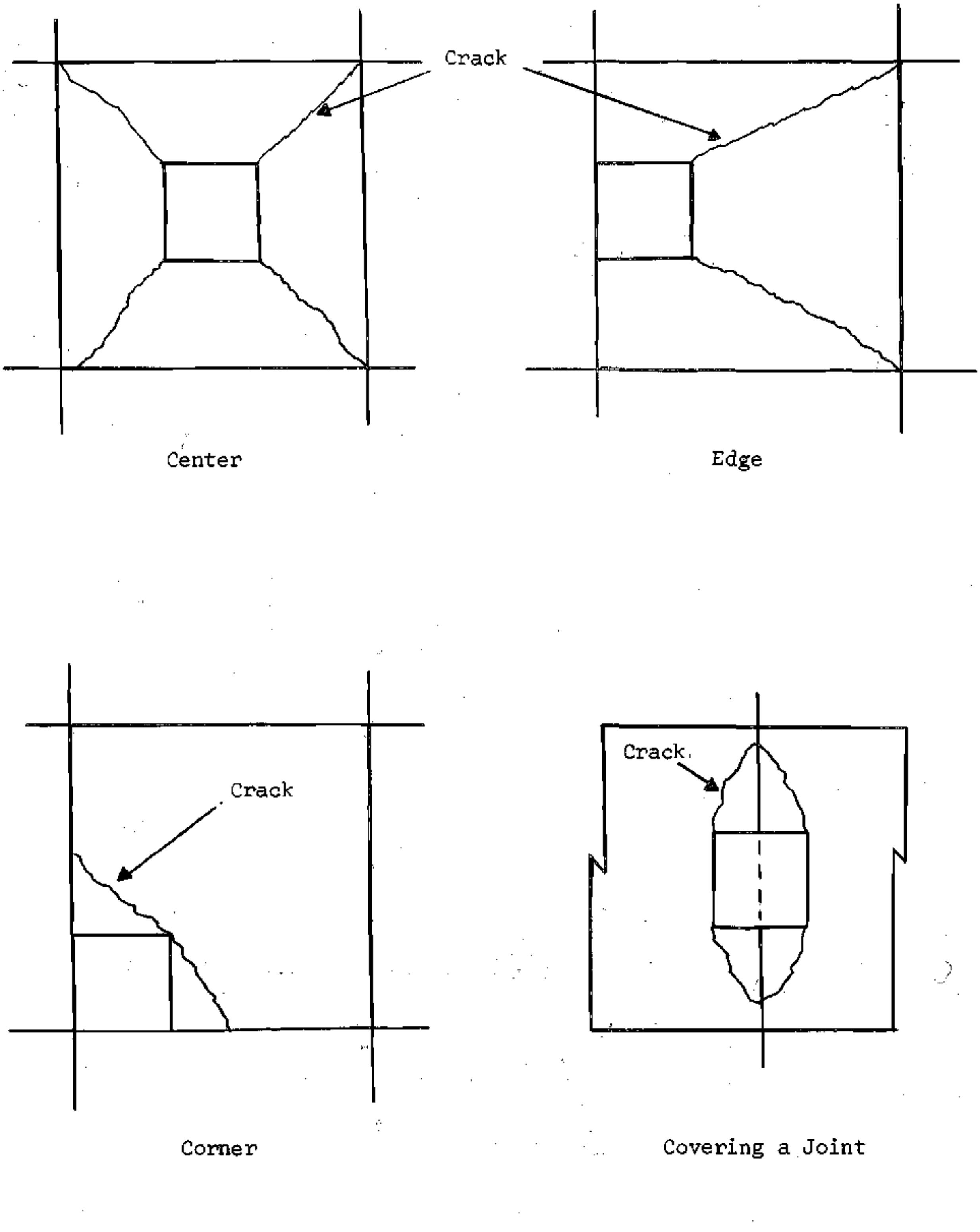


Figure 9. General Failure Pattern, Small Footing (1.25 in.) Plan View.

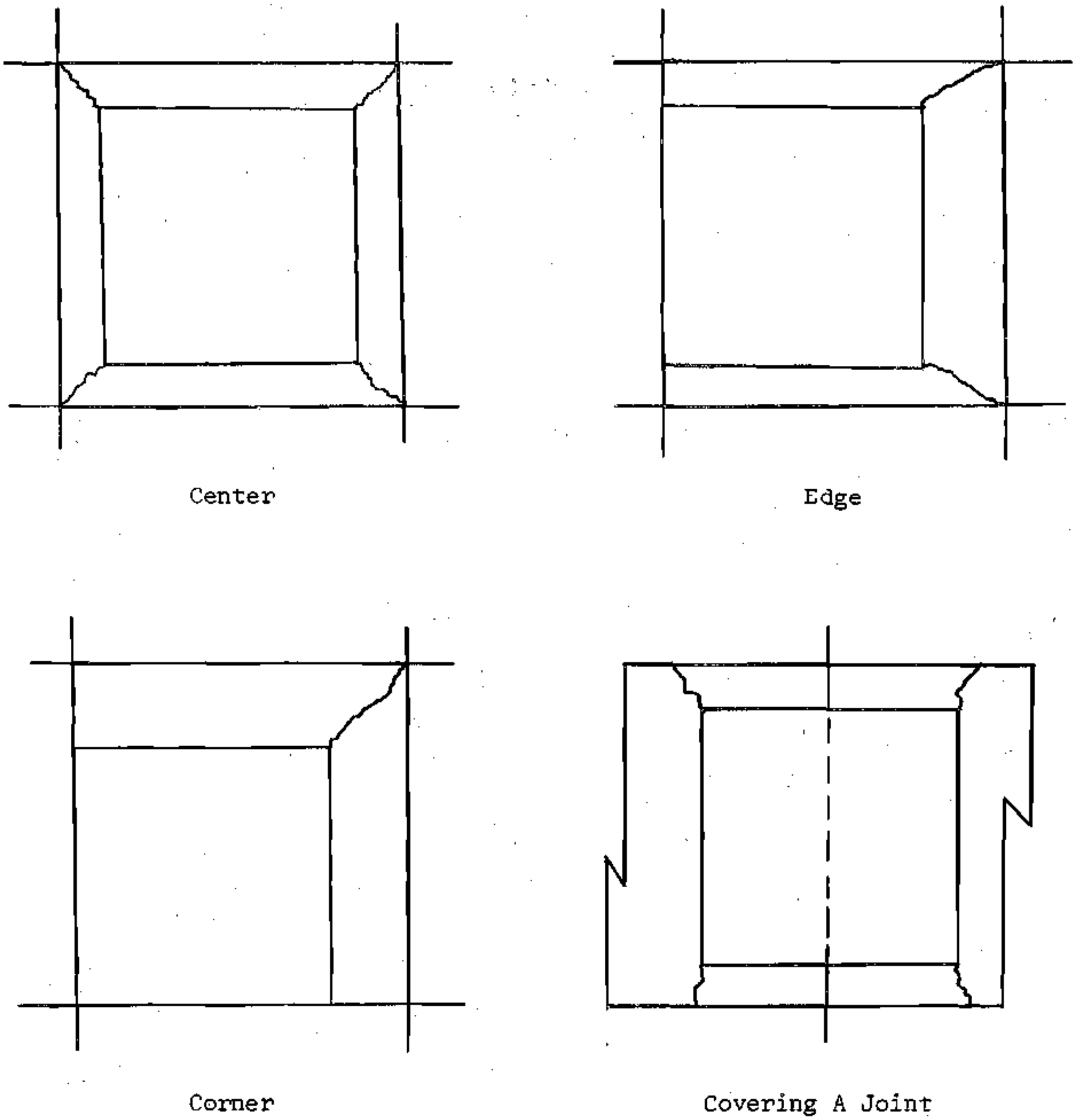
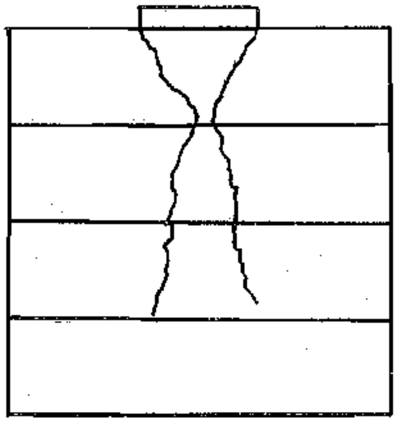
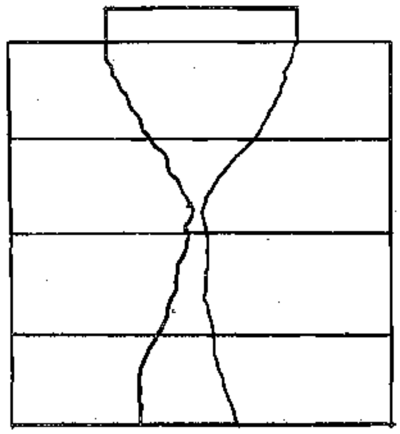


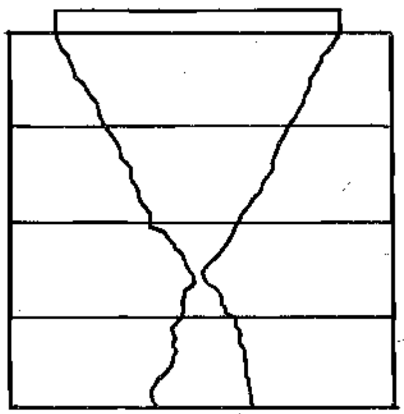
Figure 10. General Failure Pattern, Large Footing (3.0 in.), Plan View.



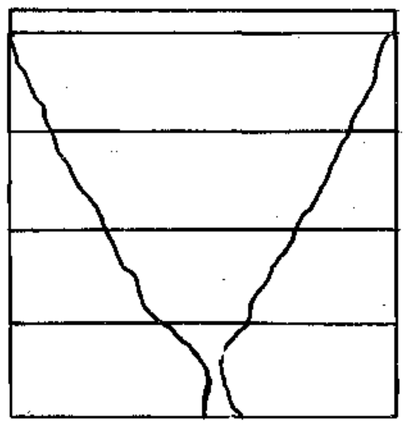
1.25 in. Footing
$\alpha \sim 31^{\circ}$



2.0 in. Footing
$\alpha \sim 30^{\circ}$



3.0 in. Footing
$\alpha \sim 31^{\circ}$



4.0 in. Footing
$\alpha \sim 32^{\circ}$

Figure 11. Observed Failure Pattern, Profile View Showing Wedge Angle = 2α.

that the larger decrease actually noted is caused by the discontinuities. As previously mentioned, analysis was made using several values of cohesion and angle of internal friction. The extreme cases ($\phi = 46^{\circ}$ and $\phi = 0^{\circ}$) analyzed resulted in values which bracketed the actual values very well. In addition, it is interesting to note that the difference in the calculated extremes is generally not great. This is due to the fact that the reduction in "ϕ" was accompanied by a corresponding increase in the value of cohesion. As previously stated, the writer feels that the values of "c" and "ϕ" shown by the work of Robertson are probably the more accurate considering the observed semiwedge angle "α." Analysis using these values and the modified Meyerhof equation resulted in predicted bearing capacity which was in very close agreement with that which was observed.

It appears that the modification does not hold when the footing size is increased so that it exceeds the block width as in the case of the six-inch footing. When this was done, the bearing capacity remained very nearly that for the four-inch footing which suggests that the bearing capacity reaches a lower limit when the footing width is just equal to the block width. Another interesting feature here was that the constant value for bearing capacity was reached when the footing was centered over a vertical discontinuity. This also held true when the smaller footings were centered over two blocks and then over four blocks. These will be discussed in more detail later.

The second plot, Figure 8, suggests that the total load increased linearly as the footing width increased. No analysis was advanced for this curve since it is a simple enough matter to convert the predicted unit bearing capacity to total load.

In conclusion to this section, it may be stated that the unit-bearing capacity of a thin layered jointed rock system may be effectively predicted by the Meyerhof equation for the bearing capacity of rock with the stated modification. In addition, it may be said that the bearing capacity becomes constant for at least the range of footing size tested after the footing width exceeds the crack spacing.

Footings at the Edge of the Block*

Figure 12 shows that the unit bearing capacity actually increased somewhat when the footing was moved from the center to coincide with the edge of the block. The curve of bearing capacity vs. b/w takes very nearly the same shape as that for the previous discussion with the footing at the center of the block. This, along with visual observations (Figures 9 and 10), suggests that the same type failure took place in this series as in the previous series of tests. The fact that the actual bearing capacity in this series was even slightly higher than that for the center series may be due to a slightly higher strength in the rock used in this series than in the center series. Another feasible explanation for this is suggested by comparing the observed failure cracks (Figures 9 and 10). While there were only two failure lines here, compared to four for the center series, the individual lines were somewhat longer, and it is probable that a higher stress was required at the footing to cause complete failure of the block. The stress was maximum in

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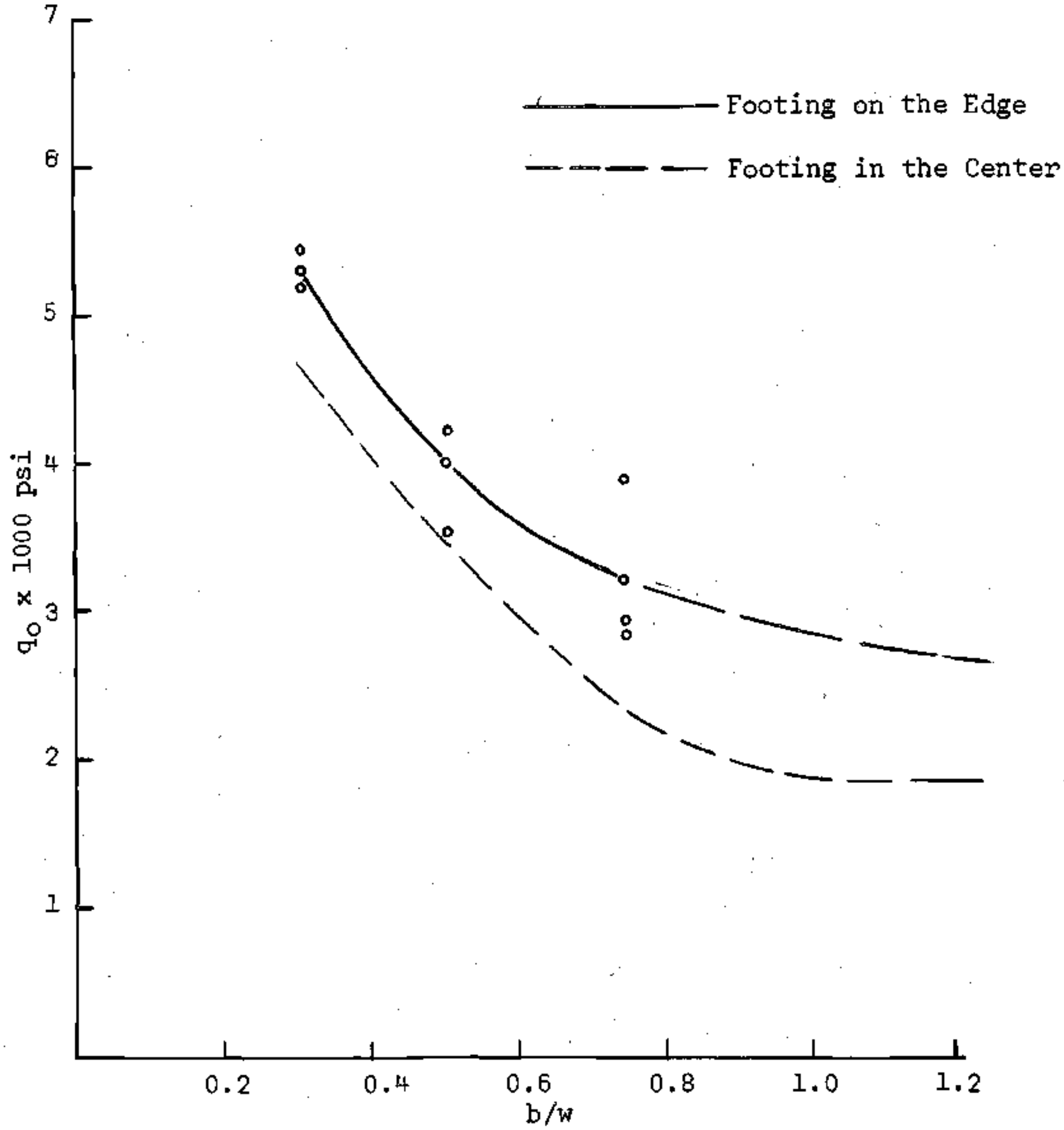
* See Table 4.



NOTE: Curve with footing at center shown for comparison.

Figure 12. Unit Load (q_o) vs. b/w Footing on the Edge.

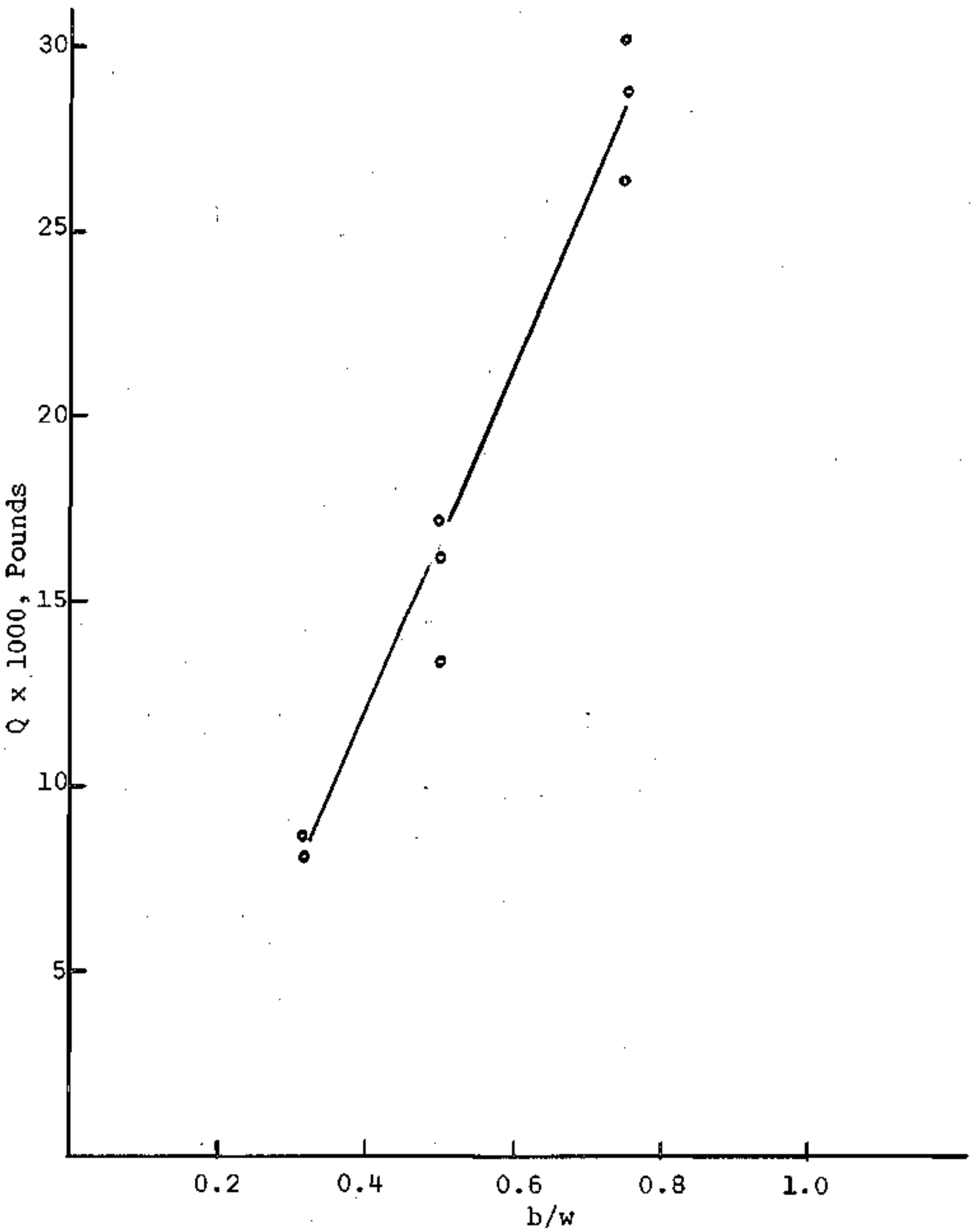


Figure 13. Failure Load (Q) vs. b/w, Footings at the Edge of the Block.

the area of the footing and decreased near the edge of the block. In any event, the predicted results are still in reasonably close agreement with the measured values.

Once again, the total failure load appears to be directly proportional to the footing width. This is further evidence that the failure was similar to that with the footing in the center.

Footing on the Corner of a Block*

For the smaller footings, there was a different mode of failure in this series of tests from the tests with the footings at the center and at the edge. For the smaller footings, the failure took place by merely breaking off the corner (Figure 9). Also, there was no wedge formation for the small footing. It is, therefore, quite reasonable that the bearing capacity should be somewhat less than in the previous tests. In addition, as the footing size increased, the unit bearing capacity also increased (Figure 14). This increase took place so that as the footing size approached the block size (three-inch footing), there was very little difference in the bearing capacity noted in this test from previous tests. This would indicate that as the footing size approaches the block size, it makes very little difference where the footing is placed on the block. This will be shown again in the discussion of different positions for the same footing.

Footings Centered Over Two and Four Blocks**

The results of these two series of tests show that the results

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* See Figures 14 and 15.

** See Figures 16 through 18.

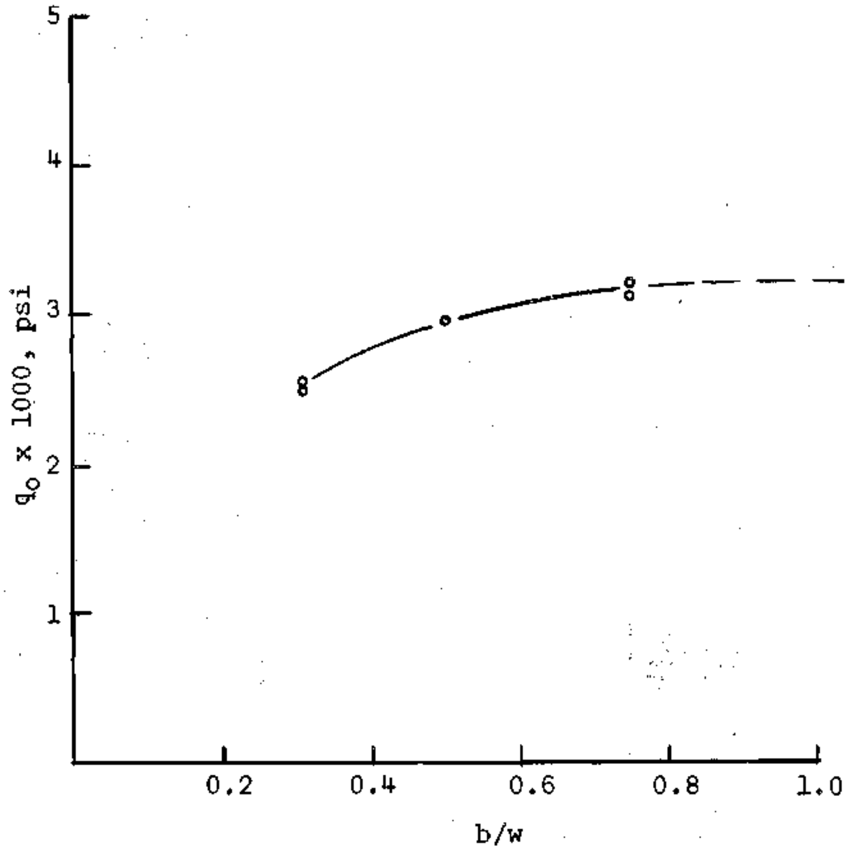


Figure 14. Unit Load (q_o) vs. b/w, Footings at a Corner of the Block.

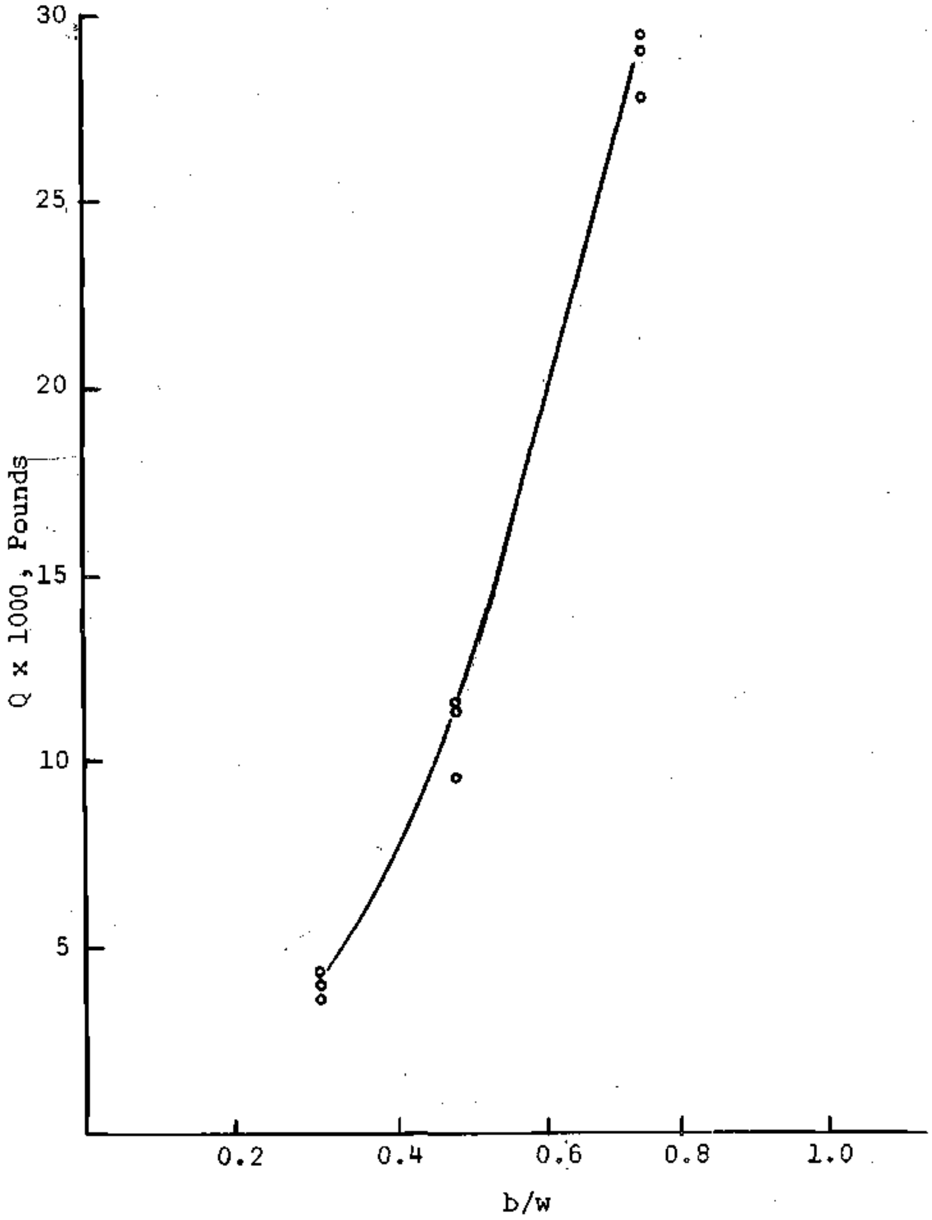


Figure 15. Failure Load (Q) vs. b/w, Footing at a Corner of the Block.

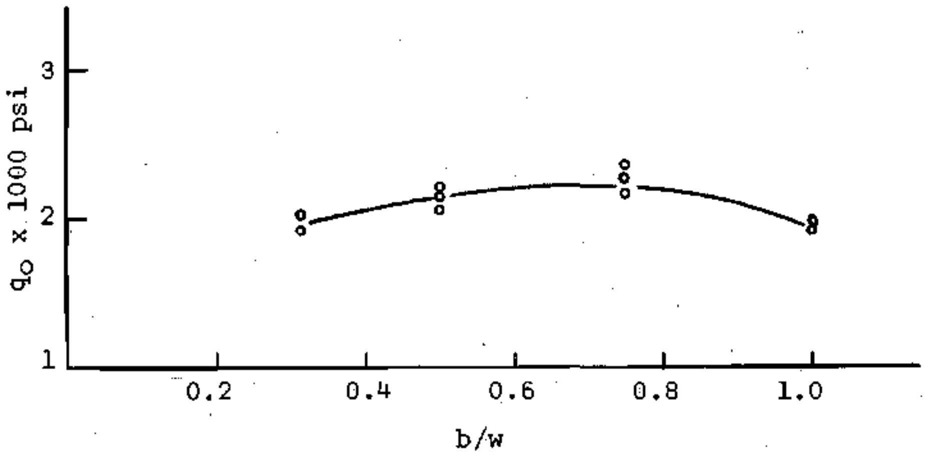


Figure 16. Unit Load (q_o) vs. b/w, Footing Centered Over Two Blocks.

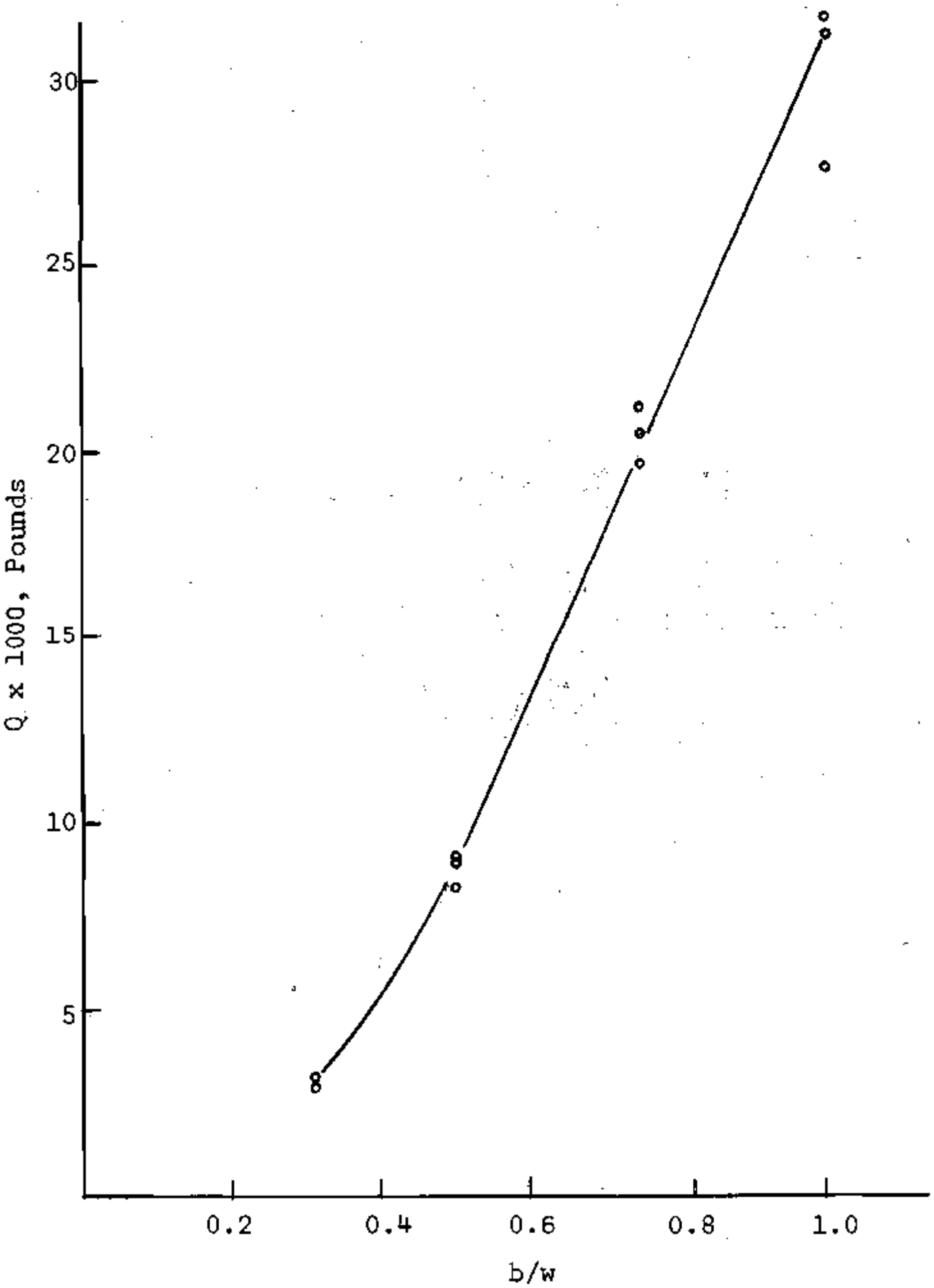


Figure 17. Failure Load (Q) vs. b/w, Footing Centered Over Two Blocks.

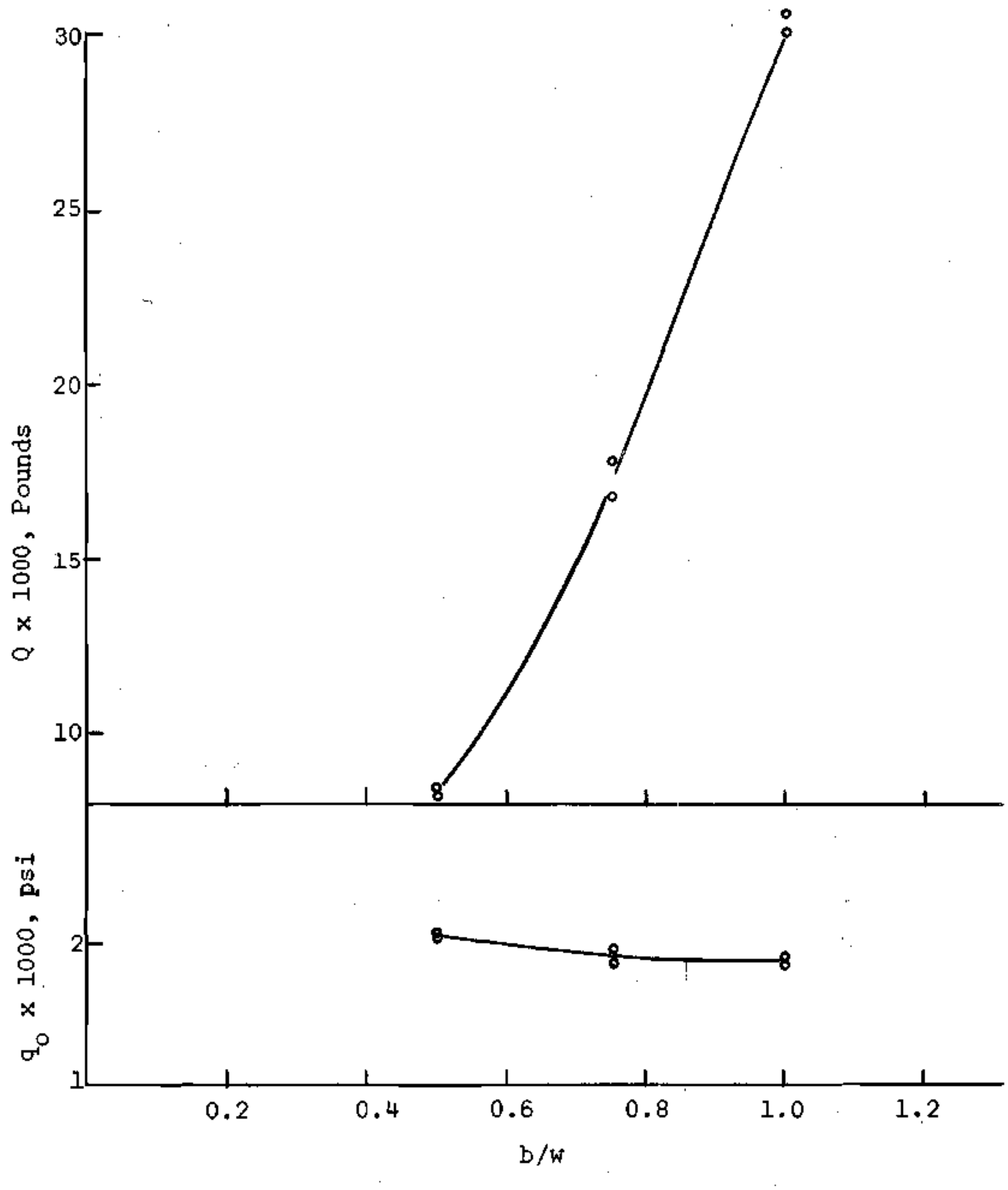


Figure 18. Unit Load (q_o) and Failure Load (Q) vs. b/w, Footing Centered Over Four Blocks.

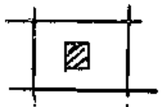
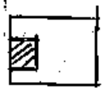
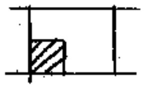
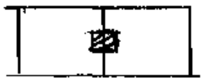
were relatively constant at approximately 2000 pounds per square inch (Figures 16 and 18). It is interesting that this is the approximate bearing capacity obtained in the test using the six-inch footing located at the center of the system. This seems to indicate that the lower limit of bearing capacity is reached when the footing is centered over a vertical discontinuity regardless of the footing size. This lower limit of bearing capacity is about 25 per cent of that predicted by the Meyerhof analysis with no modification. It should be pointed out that while the wedge did form here, vertical discontinuities in the wedge probably caused the system to react as if the corners of the rock were being broken off.

Comparison of Footing Positions

Table 2 shows a comparison of bearing capacity for footings at different positions with a constant size. As previously stated, the bearing capacity actually increased when the footing was moved from the center to the edge of the block. This held true for each footing size and has been previously explained.

With the 1.25-inch square footing on the corner of the block, there was a significant decrease in the bearing capacity compared to tests with footings on the center and at the edge. This decrease was somewhat less for the two-inch square footing and nonexistant for the three-inch square footing. This is evidence that as the footing size approaches the block size, the bearing capacity depends very little upon the position as long as there is no vertical discontinuity directly beneath the footing.

Table 2. Comparison of Footing Positions

Footing Size	Position	Sketch	q_{ave}	q_{ave}
1-1/4	Center		7200	4610
1-1/4	Edge		8260	5300
1-1/4	Corner		4000	2560
1-1/4	Two		3100	1985
2	Center		14,000	3500
2	Edge		16,000	4000
2	Corner		11,720	2930
2	Two		8510	2140
2	Four		8325	2080
3	Center		20,000	2220
3	Edge		28,800	3200
3	Corner		28,500	3170
3	Two		20,300	2250
3	Four		17,350	1930
4	Center		31,250	1955
4	Two		31,000	1940
4	Four		30,300	1890

There was a further decrease in bearing capacity when each footing was centered over a vertical discontinuity. It made little difference whether the footing was placed over two or over four blocks, suggesting that the greatest effect is brought about by the first discontinuity. The bearing capacity increased slightly when any footing was moved from the center to the edge of the block. The bearing capacity with the footing on the corner of the block decreased significantly for the smaller footings with no effect on the larger footing. Finally, any vertical discontinuity directly beneath a footing caused a decrease to the lower limit of bearing capacity.

Settlement

As mentioned in the introduction, John (1) has stated that any settlement depends primarily upon the degree of displacement of the individual blocks. Comparison of Figures 19 through 23 certainly tends to confirm John's hypothesis. The load-settlement curves show conclusively that there was much more settlement in the jointed system than in the solid block of the same size. The settlement in the jointed system averaged about 0.06 inches and ranged in excess of 0.1 inches. It should be noted here that care was exercised to insure the tightest possible packing of the individual blocks. The settlement in the solid mass averaged about 0.03 inches.

CHAPTER IV

CONCLUSIONS

The main objectives of this investigation were to (1) analyze the bearing capacity of a jointed rock system, (2) compare the bearing capacity with footings in different positions, (3) determine the type failure involved and extent of stress transmission, and (4) study the settlement characteristics of the jointed system.

Based on the series of tests carried out in the laboratory, the following conclusions have been reached:

(1) There was no significant transfer of stress across the discontinuities. The only blocks affected were those directly beneath the footing.

(2) Based on above statement and results, no attempt should be made to analyze the bearing capacity of a jointed rock system with the general bearing capacity equation (8).

(3) The bearing capacity of the jointed system can be conveniently predicted by a simple modification to the Meyerhof (2) equation for the bearing capacity of rock.

(4) a. Small footings: When the footing is small compared to the block size, there is a slight increase in bearing capacity when the footing is moved from the center to the edge of the block. Further, there is a significant drop in the bearing capacity when the footing is moved to the corner of a block and over a discontinuity.

(4) b. Large footings: When the footing size approaches the block size, position of the footing affects the bearing capacity very little until a discontinuity is covered. This results in a significant drop in the bearing capacity.

(5) Failure occurs in a splitting manner followed by a punching out of lower blocks.

(6) Settlement depends greatly upon the tightness of the packing of individual blocks and would be most difficult to predict.

CHAPTER V

RECOMMENDATIONS FOR FURTHER STUDY

(1) The investigation should be continued to observe the effect of increasing the footing size/block size ratio beyond the limit of this study.

(2) A study could be undertaken to examine the effect of another material, such as clay, in the joints.

(3) The layer thickness and horizontal size of the blocks should be increased to determine at what point a general bearing capacity failure takes place.

(4) A study of the effect of some type of reinforcement on the system could be undertaken.

APPENDIX

Table 3. Footings at Center of Blocks

Figure Size	b/w	Q_1	Q_2	Q_3	q_1	q_2	q_3	q_{avg}	q_{ave}	$Q_{Large\ Block}$	q_{Large}	Failure Settlement 1	2	3
1-1/4"	.313	7500	7200	6950	4800	4610	4450	7200	4610	50,000	32,000	.105	.085	.130
2"	.5	13,800	13,500	15,400	3450	3380	3850	14,000	3500	41,000	10,250	.030	.070	.055
3"	.75	19,100	18,300	22,900	2125	2035	2545	20,000	2220	84,000	9,333	.060	.080	.085
4"	1	31,500	31,300	31,000	1970	1955	1940	31,250	1950			.080	.080	.100
6"	1.5	67,400	70,000		1875	1950		68,700	1910			.150	.125	

Footings Are in the Center.

Table 4. Footings at the Edge of Blocks

Figure Size (Inches)	b/w	Q_1	Q_2	Q_3	q_1	q_2	q_3	Q_{ave}	q_{ave}	Settlement		
										1	2	3
1-1/4	.313	8540	8200	8100	5470	5250	5190	8260	5300	.047	.063	.058
2	.5	17,120	13,200	16,300	4280	3300	4075	16,000	4000	.068	.088	.053
3	.75	30,800	25,500	26,200	3920	2835	2910	28,800	3200	.060	.060	.055
4	1.0	No	Test									

Footings Are on the Edge.

Table 5. Footings on a Corner of Blocks

Figure Size (Inches)	b/w	Q_1	Q_2	Q_3	q_1	q_2	q_3	Q_{ave}	q_{ave}	Settlement 1	Settlement 2	Settlement 3
1-1/4	.313	3880	4060	4050	2485	2600	2595	4000	2560	.070	.038	.053
2	.5	11,740	9200	11,760	2935	2300	2940	11,720	2930	.060	.070	.085
3	.75	27,400	29,600		3045	3295		28,500	3170	.072	.050	

Footings Are on a Corner.

Table 6. Footing Centered Over Two Blocks

Figure Size (Inches)	b/w	Q_1	Q_2	Q_3	q_1	q_2	q_3	Q_{ave}	q_{ave}	Settlement 1	Settlement 2	Settlement 3
1-1/4	.313	3000	3200		1920	2050		3100	1985	.049	.030	
2	.50	8200	8850	8650	2050	2212	2163	8570	2140	.042	.051	.053
3	.75	19,400	21,200		2150	2350		20,300	2250	.105	.055	
4	1.0	26,500	31,500	30,800	1660	1970	1930	31,000	1950	.100	.110	.090

Footings Are Over Two Blocks.

Table 7. Footings Centered Over Four Blocks

Figure Size (Inches)	b/w	Q_1	Q_2	Q_3	q_1	q_2	q_3	Q_{ave}	q_{ave}	Settlement		
										1	2	3
1-1/4	.313	No	Test									
2	.50	8400	8250		2100	2062		8325	2080	.070	.062	
3	.75	17,900	16,800		1900	1870		17,350	1930	.105	.083	
4	1.0	30,600	30,000		1910	1875		30,300	1890	.098	.103	

Footings Are Over Four Blocks.

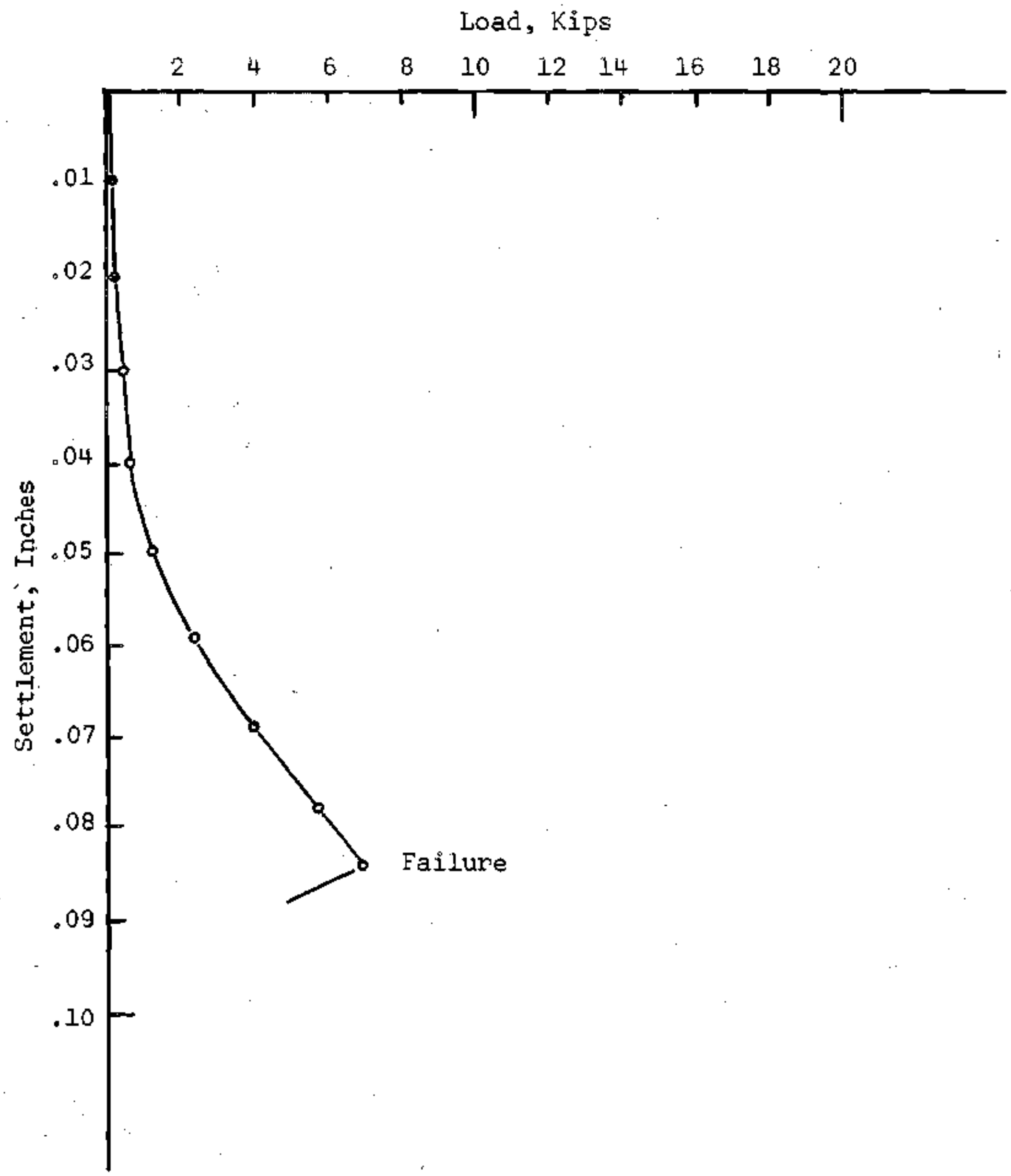


Figure 19. Typical Load-Settlement Curve
1.25 Inch Footing Centered on Jointed System.

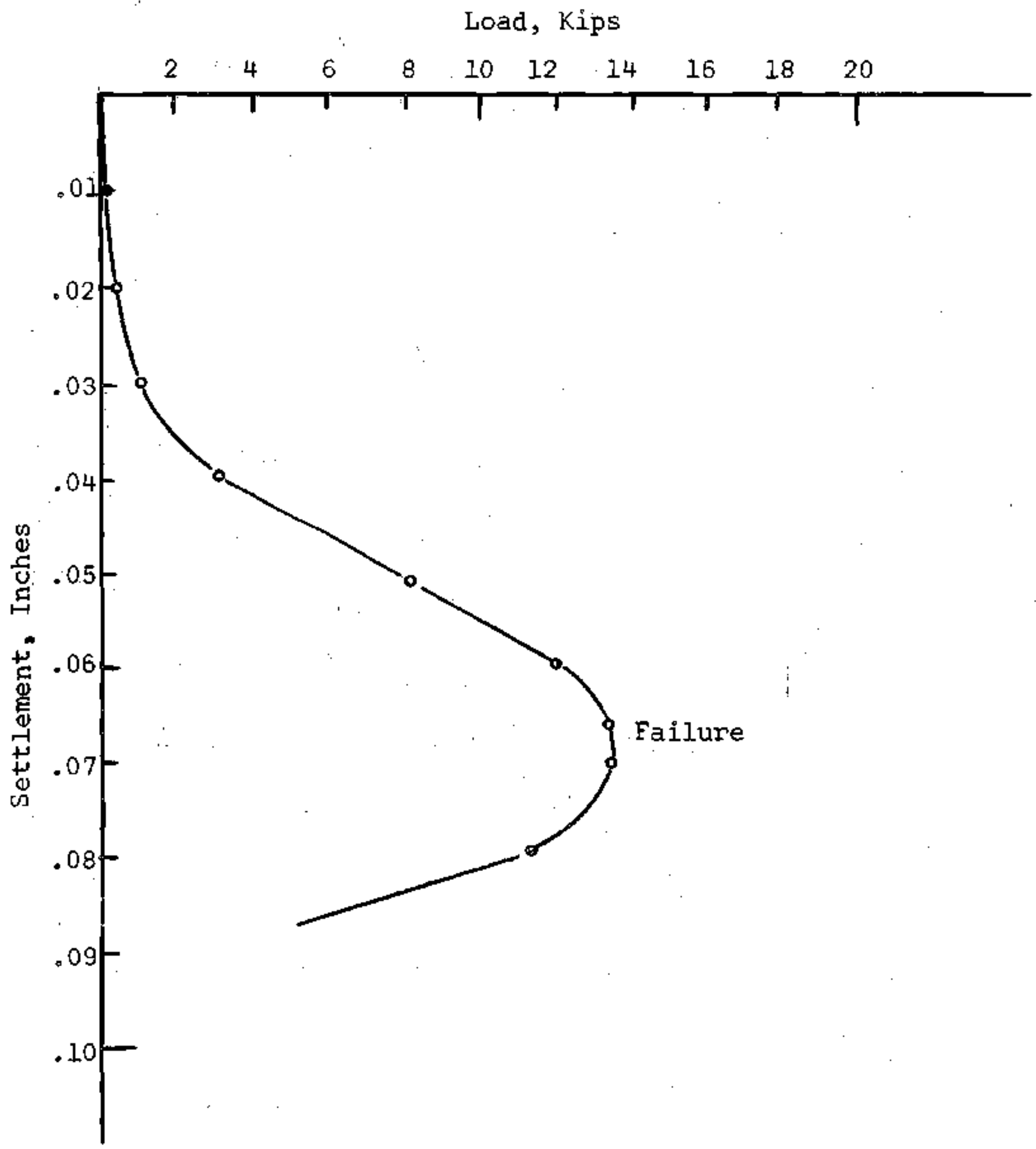


Figure 20. Typical Load-Settlement Curve
2.0 Inch Footing Centered on Jointed System.

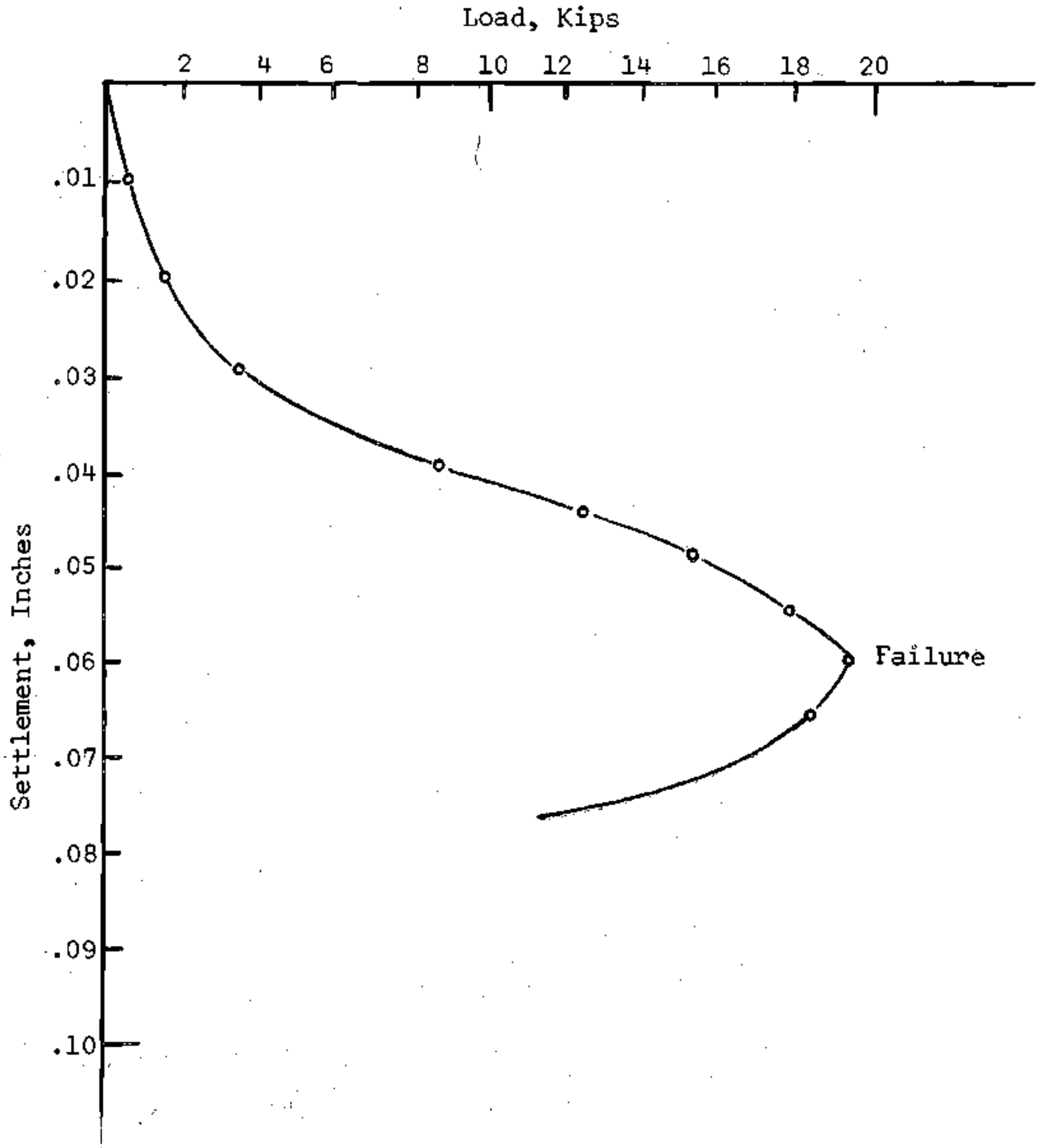


Figure 21. Typical Load-Settlement Curve
3.0 Inch Footing Centered on Jointed System.

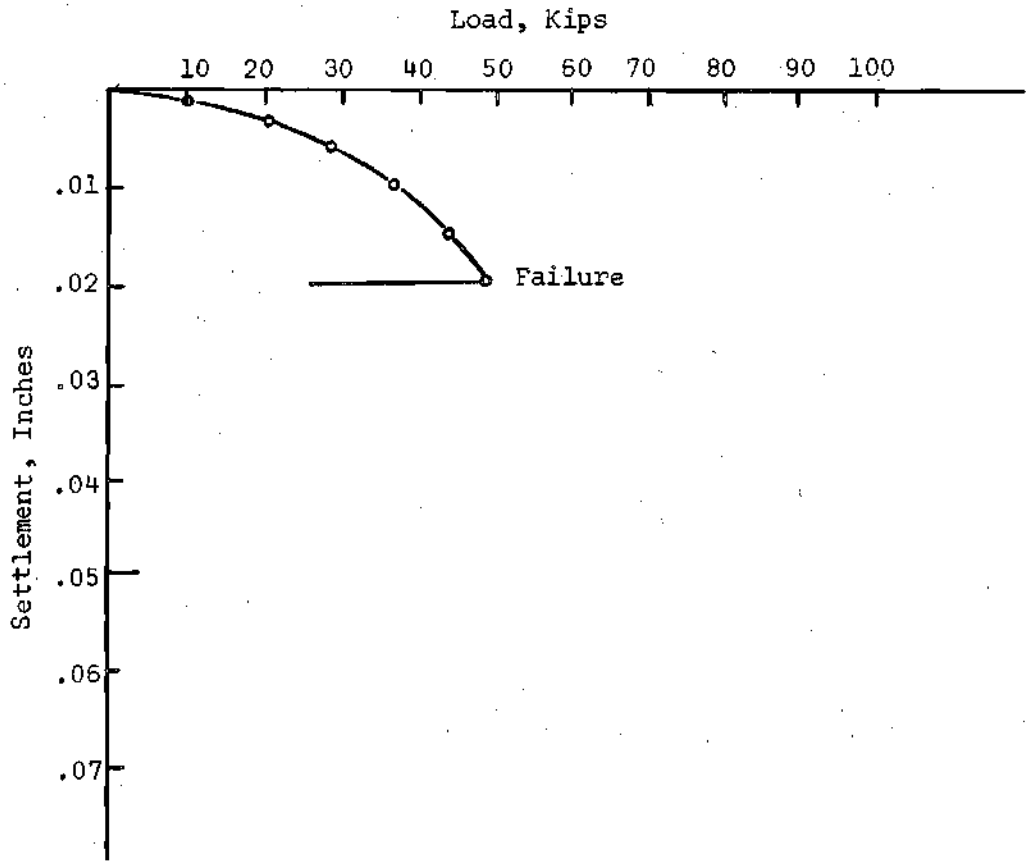


Figure 22. Typical Load Settlement Curve, 1.25 Inch Footing Centered on Solid Mass.

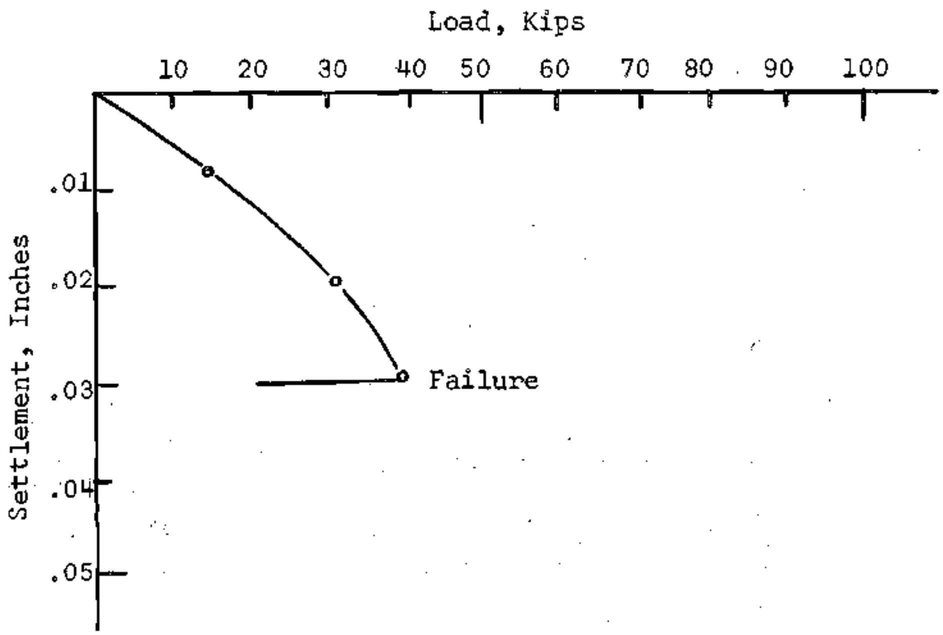


Figure 23. Typical Load-Settlement Curve
2.0 Inch Footing Centered on Solid Mass.

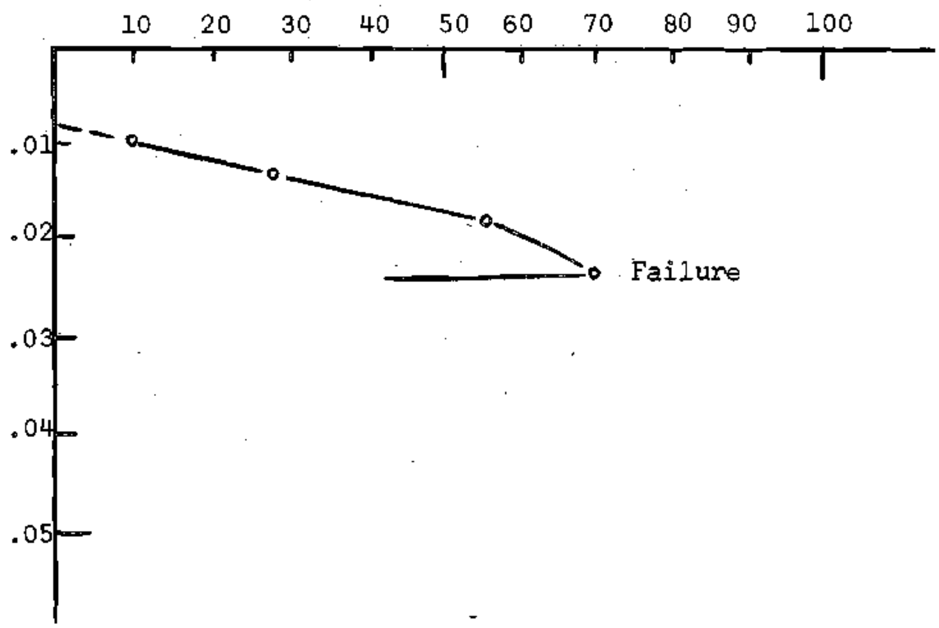


Figure 24. Typical Load-Settlement Curve
3.0 Inch Footing Centered on Solid Mass.

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