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The ultimate load carrying capacity of single angle, single bolted connections.

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THE ULTIMATE LOAD CARRYING CAPACITY OF SINGLE ANGLE, SINGLE BOLTED CONNECTIONS

Submitted in partial fulfillment
of the requirements for the
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from the University of Windsor.

By

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NOTATION

P = Ultimate Test Load, Pounds

t = Thickness of Test Angle, Inches

d = Diameter of Bolt, 5/8 Inches

σ_y = Yield Stress, Lbs. Per Square Inch

σ_u = Ultimate Tensile Stress, Lbs. Per Sq. Inch

ϕ_y = Failure Function, Based on Yield Stress

ϕ_u = Failure Function, Based on Ultimate Tensile Stress

x = End Distance, Inches

y = Edge Distance, Inches

σ_n = Stress on Net Section At Ultimate Load, Lbs. Per Square Inch

σ_b = Bearing Stress at Ultimate Load, Lbs. Per Square Inch

P' = Intermediate Load on Specimen, Pounds

ϕ_y' = Load Function, Based on Yield Stress

1.0 INTRODUCTION

Tower structures and power substation structures are usually fabricated from equal and unequal angle sections because of their ease of fabrication and erection. These advantages are a result of the basic simplicity of the cross-section. Unlike wide-flange, H or I sections, angles can be connected together with a minimum of gusset plates or elaborate joints. In particular, single angle members have been used almost exclusively in the majority of tower structures.

In the design of the structures of a Power Transmission Line the economics of the project is related directly to the repetition which exists because of the quantity of structures of any one type required. In general, a transmission line consists of a large number of standard suspension towers and smaller quantities of special structures. These latter structures are used at points where the line changes direction or where other particular load carrying capabilities are required. In a long power line, as many as 1000 or more standard suspension towers may be required.

Most members of a tower occur at least four times in the structure or even oftener because of its symmetry and the fact that at least the opposite faces are alike. Thus, a particular member in a suspension tower may be repeated over 4000 times in the full transmission line. Obviously, if the design load in such a member can be adequately carried by a single bolt rather than two bolts in each end connection, the possible saving amounts to 8000 bolts and the punching of 16,000 holes. It is savings of this order which make the single bolted, single angle connection economically attractive to industry. Note that this feature is equally attractive when the field erection of the structures is considered. Every bolt which can be eliminated in the design, is one less bolt to be installed and tightened by hand.

In addition to having an effect on the design of connections, the multiplicity of one tower type in a transmission line necessitates the use of the lightest members possible if the design is to be selected from amongst competitive designs.

Thus, the aim of the designer of a transmission tower is to arrive at a structure consisting of a combination of minimum size, minimum thickness members connected by a minimum number of bolts. This structure, in turn, must support a load which exceeds the loading imposed during a full scale tower test.

Obviously, the minimum connection in a minimum size member is the single bolted single angle as discussed herein.

In the design of guyed masts, the long straight-sided shafts of these structures are similarly suited to the use of single angle members. In such masts the repetition of the basic bracing system makes the use of single bolted connections economically attractive for the same reasons which apply to Transmission Towers.

1.1 SINGLE ANGLE MEMBERS

Photograph No. 1 indicates typical single angle members fastened by single bolts as used in a transmission tower. Photograph No. 2 shows the angle bracing members of a guyed mast connected by single bolts. Figure 1.1 shows the end portion of a single angle with one hole for a connecting bolt. This figure defines the end and edge distances and relates these to the punching gauge as well as to the heel and toes of the member. Photograph No. 3 shows a single angle as fabricated. Note that the end of the piece has been sheared and the hole punched. These are typical fabrication techniques used in production tower shops for all but the very largest angle sizes. In general, shearing of angles is not replaced by saw cutting unless the section is 8 x 8 or larger. Punching is used in medium steels for all thicknesses up to and equal to the bolt diameter. For thicker material the holes are drilled from the solid. In some special connections or special structures where fit is of particular importance the holes may be sub-punched and reamed to the desired final size or drilled full size using a steel templet. In that these latter methods are special, they are not considered in this paper.

1.2 RESULTS OF PAST TESTS

Testing of angle connection was done in 1929 by The Canadian Bridge Company and The American Bridge Company working in collaboration. These tests indicated that with the steels then available, with a guaranteed minimum yield point of 33,000 p.s.i., some of the smaller angle sizes would not develop the bearing capacity of the material or the shear capacity of a standard tower bolt. In particular, 1-1/2 x 1-1/2 x 1/8, 1-1/2 x 1-1/2 x 3/16 and 1-3/4 x 1-3/4 x 3/16 angles were found to fail through the edge at relatively low loads. This fact was taken into consideration by introducing appropriate reduction factors into the permissible bearing stress or bolt shear values used with these sections. Note that 1-3/4 x 1-3/4 x 1/8 angles are not included in the list of exceptions. It was found that with standard gauges the bearing capacity of 1/8" material, which cannot be developed in an angle 1-1/2" wide, can be developed with 1-3/4" of width. For all other angles tested, it was found that full bearing or the bolt shear capacity could be developed with standard gauges and end distances.

One of the very significant results of these 1929 tests was the conclusion that permissible bearing stresses equal to twice the permissible tension stresses of the connected material were acceptable when using unfinished tower bolts.

In 1961, Dosco Industries Limited, Canadian Bridge Division, carried out another series of tests. These tests were instituted in view of the increased use of high strength steels, both for tower members and bolts. Table 1.1 lists the steels which are commonly used in the fabrication of towers in Canada, while Table 1.2 gives the physical properties of both standard tower bolts and high strength structural bolts.

The purpose of the 1961 tests was to evaluate the reliability of the reduced bearing values for the smaller angles, as concluded from the 1929 tests, when applied to the newer high strength steels. Unfortunately, these newer tests were neither extensive enough nor documented enough to lead to any reliable conclusions. In order to overcome these shortcomings and yield some significant results, the tests reported herein were instituted.

1.3 TESTS ON FLAT BARS

George Winter has reported in his "Tests on Bolted Connections in Light Gage Steel" (8) that four distinct failure patterns are possible depending upon the magnitude of the pertinent variables. These failure types are:

1. "Longitudinal shearing of the sheet along two practically parallel planes whose distance equals the bolt diameter; this type occurred for relatively short 'edge' distances e."

II. "Shearing-Tearing along two distinctly inclined planes with considerable "piling-up" of the material in front of the bolt; this type of failure occurred chiefly for long "edge" distances e".

III. "Transverse Tension-Tearing across the sheet"

IV. "Shearing of the bolt, with more or less pronounced preceding elongation of the hole."

(Note that "edge" e as referred to by Winter is equivalent to "end" distance as referred to here and as defined by Figure 1.1).

Based on his findings, Winter derived empirical relationships for flat bars which will indicate the expected ultimate capacity of the bar and the mode of failure, either I, II, III or IV as listed above.

The work reported here was undertaken in the hope that similar empirical relationships could be derived for single angle members.

2.0 EXPERIMENTAL WORK

A total of 604 samples were tested and the results thus obtained were combined with the results of 117 tests conducted by Canadian Bridge in 1961. All samples were fabricated by Canadian Bridge Division as regular production work, not as special work. That is, all pieces were cut to length by shearing, all holes were punched and no special efforts were given to accuracy of shearing or punching beyond what might be expected in regular production of small members. Each test section was fabricated from material available from Canadian Bridge Division stock with no attempt made to pick material with any particular adherence to the specified size. That is, no checking was done on either width or thickness against the appropriate fabrication specification (9).

For the 604 new tests each sample was made nine inches long while the 1961 tests used samples from nine inches up to 1'- 4-1/4" long. Most of the 1961 test pieces were the former length.

Figure 2.1 shows the general detail of all test pieces. Note that two bolts were used for connecting one end while the test end used only one bolt. All three bolt holes were punched on a common gauge line for simplicity of fabrication. In most cases all samples were galvanized the same as regular transmission tower steel. A few tests were strain gauged, in which cases galvanizing was eliminated in order to provide a surface which was easier to prepare for attachment of the gauges. Some test pieces were left ungalvanized in order to expedite delivery.

Preparation for galvanizing included shot blast cleaning in a tumble blast machine followed by a short "flash" pickle in acid.

2.1 VARIABLES CONSIDERED

Consideration of the load carrying capacity of a single angle connection leads to recognition of a number of factors which do have, or may have, a significant effect on the load carrying capacity of the connection.

In this report, the following factors have been considered:

(a) End Distance
(b) Edge Distance
(c) Thickness
(d) Yield Strength of the Material
(e) Ultimate Tensile Strength of the Material.

In addition to the foregoing, the following factors will also contribute to the ultimate capacity of a connection:

(f) Tolerance on End Distance
(g) Tolerance on Edge Distance
(h) Variations in Yield Strength throughout the material.
(i) Variations in Tensile Strength throughout the material.
(j) Mill Tolerances on the section profile (9).
(k) General Condition of Sheared or Punched Surfaces.
(l) General Condition of the Rolled Surfaces, that is, as rolled, compared to slightly or highly rust pitted.
(m) Hole Diameter

In that items (f) to (m) inclusive are factors which exist in industry and cannot always be economically or conveniently measured or regulated, this study has made no attempt to investigate their significance.

Item (m), hole diameter, results directly from bolt diameter if tolerances in the punched size are ignored. The majority of tower structures built in Canada use 5/8" diameter bolts in 11/16" diameter holes, therefore, this study has been restricted to investigating connections using this size holes.

Items (h) and (i), the variations in yield strength and tensile strength, throughout the material, are recognized in industry, but ignored for practical purposes. It is recognized that in a hot rolled structural section, such as an angle, the mechanical properties will change across the profile. Also, changes can be expected throughout the length of the material as rolled. In spite of these variations, it is standard practice to accept the mill test report for one test of a given profile in a given heat as being representative of all of that profile produced from the heat.

For the investigation reported herein, two criteria have been adopted relative to yield and tensile strengths. The 1961 Canadian Bridge Division tests were conducted in two different groups but tension coupons were not tested at that time. In order to evaluate the yield and ultimate tensile stresses in these tests, for each section size represented in a group, one tension test coupon was cut from a bolt test sample. Note that this procedure assumes, for example, that all of the 1-1/2 x 1-1/2 x 1/8 material in the April, 1961 tests was cut from one stock length.

For the newer tests, fabrication was carried out at four different times, thus four different sets of tension test coupons were made and tested. Also, when it was known that all of the pieces of a particular section in a production run could not be cut from one stock length, a tension coupon was prepared from each stock length used in production.

2.2 SCOPE OF INVESTIGATION

The following variables have been considered:

End distances from 3/4 to 1-3/4 inches in increments of 1/8"

Edge distances from 5/8 to 1-3/8 inches in increments of 1/8"

Thickness of 1/8", 3/16" and 1/4"

Two grades of steel, medium and high strength.

Figure 2.2 shows the total number of tests made for each combination of end and edge distance used.

Both the end and edge distances investigated go beyond the ranges usually used in industry for single bolt connections in order to permit the fitting of a reasonably long curve to the plotted results.

The thicknesses used, 1/8", 3/16" and 1/4" are by far the commonest thicknesses for single angles used with single bolts.

The grades of steel used are in two very broad classes, medium and high strength. The medium grade includes C.S.A. G40.4 and A.S.T.M. A36 steels with properties as per Table 1.1. The high strength grade includes C.S.A. G40.6, C.S.A. G40.8 and RB60 steels, again with properties as per Table 1.1. It should be noted that both the G40.4 and A36 specifications place no upper limit on yield strength, thus it is possible to have so-called medium steel with a yield strength higher than that of some of the high strength steels. For example, referring to Table 3.5, the 2-1/2 x 2-1/2 x 1/8 A36 steel of the 500 Series Tests has a yield strength of 51,200 p.s.i., while the 2-1/2 x 2-1/2 x 1/8 G40.8 of the 600 Series Tests has a yield strength of only 44,550 p.s.i.

In all cases, the high strength samples were selected from available material in Canadian Bridge Division stock. This criteria accounts for the rather random assortment of materials used,but was unavoidable, in that the total of all samples of one grade and profile would not represent enough material to justify a special rolling of the requirement by a steel mill. Most Canadian Mills will not accept orders for less than 25 tons of one angle section.

The desire to investigate two grades of steel, combined with the necessity of using available stock material, resulted in some of the samples being fabricated from material which had been stock-piled for some time and, as such, was badly rusted. After shot blasting, pickling and galvanizing, the resulting surface of the samples was noticeably pitted, as shown in Photograph 4.

2.3 TENSION TEST SPECIMENS

All tension test specimens were fabricated in accordance with Figure 2.3. The specimens for the new tests were not galvanized when tested in that under normal circumstances in industry, tension tests are performed on the black, as-rolled steel at the rolling mill. The results thus obtained are applied directly to the galvanized members of a complete structure without modification.

For the 1962 Canadian Bridge Division tests the tension specimens have been cut from the old bolt test members and, as such, were galvanized. Any change in either yield strength or ultimate tensile strength due to shot blasting, pickling or galvanizing has been disregarded as a minor effect. For practical purposes, in industry, this effect is absorbed in the Factor of Safety of the complete structure under consideration.

Note that the tension test specimens used are shorter and narrower than those specified in C.S.A. Standard G40.1 (9). The length used was derived from the full length of the 1962 test pieces. The width used was dictated by the need to cut the 1962 test pieces without including the piece mark which is stamped into the steel.

2.4 TESTING PROCEDURES

All tests, both the 1962 series and the newer series, were made on a Tinius-Olson Hydraulic Testing Machine of 200,000 pounds capacity, with three load ranges, of which the two highest, 30,000 lb. and 120,000 lb. respectively were used.

In all tests, the ultimate load recorded is the maximum reading observed before the load fell off. All attempts to note a distinct yield point as was done with the tension coupons failed. For all tests, the load indicating needle rose continuously from the beginning of the loading cycle until a maximum was reached, and then the needle fell back. In no cases was the load observed to increase again after the first fall-off at maximum load.

In many cases, particularly with a combination of large end and edge distances, the rate of loading was observed to slow down noticeably until failure occurred.

2.4.1 SINGLE SHEAR TESTS

Most of the tests were made with the bolts in single shear as shown in Photograph 5. The heavy plates used to form the connections with the test angles were 7/8" thick A36 steel. It was found that after several tests the holes in these plates elongated to such an extent that the bolts would move considerably out of their line at right angles to the plane of the plate. Later tests were made using hardened steel plates in order to minimize this secondary effect.

Connection slip was eliminated by applying an initial load of approximately 200 to 400 lbs. with the bolts finger tight. After application of this load all bolts were tightened to the maximum possible using a 14 in. ratchet wrench. This hand torqueing is similar to that used in field erection of tower structures.

For those tests where the expected ultimate load was greater than the shear capacity of a standard tower bolt, high strength structural bolts to A.S.T.M. Specification A-325 were used. All bolts were used with a flat washer under the nut.

2.4.2 DOUBLE SHEAR TESTS

When test loads were expected to exceed the capacity of high strength bolts, a double-shear type connection was used, as shown in Photograph No. 6. This connection does not provide equal loads on the two shear planes of the bolt but did provide enough redistribution of the load to the second shear plane to eliminate bolt failure before failure of the test angle.

A small initial bolt load was applied before final tightening in all double shear tests, as described above for single shear tests.

2.4.3 STRAIN GAUGED SPECIMENS

Four tests were conducted with the specimens strain gauged at points of critical stress concentrations. Figure 2.4 indicates the dimensions of these specimens and shows the location of the gauges. Note that pieces 719-A and B are made to fail through the edge while pieces 734-C and D are made to fail through the end.

Special plate washers, which are visible in Photograph No. 7 were used on these four tests. These washers had longitudinal grooves machined in one face to provide clearance for the wiring to the gauges. This technique had the effect of spreading the clamping force of the bolt over a larger area than is the case when a standard washer is used. Also, because of the grooves in the plate washer, the clamping force at the two edges of the hole at right angles to the longitudinal axis of the test angle was eliminated. It is not believed that these variations in clamping pressures had any significant effect on the ultimate loads carried by the specimens.

Photograph No. 8 shows a specimen in the test machine and the equipment used in reading gauges. This equipment consisted of Budd Datran Digital Strain Indicator, Switch and Balance Unit, Polarity Transposer and Printer Control Unit, along with a Victor Digit-Matic Printer.

All gauges used were Series EA Strain Gauges as manufactured by Micro-Measurements, Inc., Romulus, Michigan, with a gauge length of one-sixteenth of an inch.

Photograph No. 9 shows strain gauged specimens after testing. The arrangement of gauge wires parallel to the length of the angles is provided so that the grooved plate washer will straddle these wires.

2.4.4 DERIVATION OF YOUNG'S MODULUS

The derivation of Young's Modulus was based on values of strain read from electrical resistance strain gauges. Each tension coupon was fitted with one gauge on each face and the average strain readings thus available from the two gauges were used in calculating Young's Modulus.

Photograph No. 10 shows two tension coupons after testing with the gauges visible adjacent to the fractures. These gauges are the same as those used on connection tests.

2.4.5 MEASUREMENT OF HOLE ELONGATION

On some of the connection tests, hole elongation was measured by use of a dial gauge calibrated in thousandths of an inch. The arrangement used provided a measurement of total elongation between the loading plates and a point on the specimen just beyond the connecting bolt. With this arrangement, the elongation measured includes some axial stretch of both the angle under test and the loading plate. In view of the total elongation of the holes these effects are quite insignificant.

3.0 EXPERIMENTAL RESULTS

Tables 3.1 to 3.6 inclusive record the results of tension tests for all steels used in the connection tests. This record includes coupon cross-sectional dimensions, yield load, ultimate load, yield stress and ultimate tensile stress. In those cases where more than one test coupon was made for a given angle size in a given test series, the test results are averaged. This averaging is shown in Tables 3.2, 3.4 and 3.6.

Tensile coupon testing revealed the justification for one of the design engineer's concerns in using high strength steels. As noted in Table 3.6 the 2 x 2 x 1/8 Angle used for Tests 701 to 704 inclusive was called for as RB60 steel with a minimum yield strength of 60,000 p.s.i. In that the tension coupons gave an average yield strength of 48,600 p.s.i., it is obvious that a mistake has been made in the handling of the stock material used here. Such a mistake is obviously easy to make and can have serious consequences in the fabrication of a structure.

Tables 3.7 and 3.8 record the results of all connection tests. These results have been modified by use of the following equations:

$$\phi_y = \frac{P}{t \times d \times \sigma_y} \quad \text{and} \quad \phi_u = \frac{P}{t \times d \times \sigma_u}$$

Where: P = Ultimate Test Load - Pounds

t = Thickness of Test Specimen - Inches

d = Diameter of Bolt - 5/8 inches

σ_y = Yield Stress of Material - Pounds Per Square Inch

σ_u = Ultimate Tensile Stress of Material - Pounds Per Square Inch

Note that ϕ_y and ϕ_u are dimensionless quantities. All of the values recorded in Tables 3.7 and 3.8 are the averages of the numbers of tests given in Figure 2.2. Thus, in the extreme cases the values of ϕ_y and ϕ_u are based on the average of 3 or 23 tests.

3.1 STRAIN GAUGED TESTS

The results of strain gauged tests to evaluate Young's Modulus are given in Figures 3.1 and 3.2. Figures 3.3, 3.4 3.5 and 3.6 show stress versus connection load for the four strain gauged connection specimens. Note that in these latter figures the values are shown for only three of the five gauges for simplicity only. In each test the results for Gauges 1 and 2 compared favourably while Gauges 3 and 4 were similarly comparable.

The significant points in these tests are the relatively low loads at which the first yielding of the material occurs. Taking Test No. 719-A as an example, this specimen failed through the edge at 11,800 lbs. but yielding occurred at the side of the bolt hole adjacent to the edge at a load of 3000 lbs.

The side of the hole remote from the toe of the angle yielded at 5,000 lbs. load but the end of specimen did not yield until the load reached 10,250 lbs.

3.2 MEASUREMENT OF HOLE ELONGATION

Figures 3.7, 3.8 and 3.9 record load versus hole elongation for 1/8, 3/16 and 1/4 inch thick samples respectively. These samples were all made with large end and edge distances in order to develop high values of the failure functions ϕ.

The 1/8 inch thick specimen, No. 316-4, to which Figure 3.7 applies, was loaded in single shear using a high strength bolt. In this case the choice of a single shear connection was not a good choice as loading had to be stopped before the angle failed as the possibility of a bolt failure became imminent. Bolt failure, particularly failure of a high strength bolt, was deliberately avoided because of the possibility of injury to personnel or damage to equipment if a bolt sheared. In those cases where high strength bolts did shear unexpectedly the bolt head literally shot across the labratory with rather frightening force.

The 3/16 inch thick specimen, No. 335-4, of Figure 3.8 failed prematurely as indicated in the figure. This failure of course terminated the test. In the case of the 1/4 inch thick specimen, No. 367-4, failure was not reached as loading was stopped to eliminate double shear failure of the bolt. The three test pieces of this series are shown in Photograph No. 11.

3.3 TYPES OF FAILURES

Disregarding the few tests where bolt failure occurred, which are not of immediate interest in this study, the angle failures are broadly designated as end or edge failures as shown in Figure 3.10. This figure clearly indicates that failure through either the end or edge of a specimen is a distinct function of the particular combination of edge and end distance involved. Also indicated is the definite transition line between failure types whether either end or edge type failures can occur.

More particularly, failure types can be classified into four types somewhat as done by Winter (8). These types are:

I. Failure of the end in a distinct combination of bending and shear acting upon the portion of the member between the end and the bolt hole. The top specimen in Photograph No. 9 shows this failure type.

II. Failure of the end along two distinct inclined shearing-tearing planes radiating from the sides of the hole toward the end of the specimen.

III. Failure of the end, as in Type II, along two distinct inclined shearing-tearing planes radiating from the sides of the hole toward the end of the specimen, accompanied by noticeable piling-up of the material in front of the bolt.

IV. Failure at the edge of the specimen with the fracture originating at the edge of the hole, progressing to the toe of the angle and accompanied by necking down of the edge material.

These failure types are illustrated by photographs 12 and 13. Photograph No. 12 shows the pattern of failures for a constant 1-1/8 inch edge distance with end distances ranging from 3/4 inch to 2 inches in 1/8 inch increments. Photograph No. 13 shows failures for a constant 1-1/8 inch end distance for edge distances from 5/8 inch to 1-3/8 inches in increments of 1/8 inch.

As shown by the two specimens on the left of photograph No. 12, Type I failures occur only with very short end distances. In all cases of Type II and III failures the actual final failure and drop-off of the load occurred only when a crack formed at the extreme end of the specimen and propogated inward to join one of the cracks propogating from the edge of the hole toward the end of the specimen. Type II and III failures are basically the same but Type III was more noticeable in 3/16 and 1/4 inch thick material than in 1/8 inch thick material. Also, and quite obviously, Type II failures occur with shorter end distances than those which develop Type III failures.

A secondary effect was noticed in the case of some 1/4 inch thick high strength angles. This was the bending of the bolted leg of the specimen outside the plane of that leg. Photograph No. 14 illustrates the nature of this localized bending.

In most cases the overall bending of the specimens, due to the eccentric application of the load relative to the neutral axis of the cross-section, was quite apparent. This effect showed up only at or very near the ultimate load but at this load the effect was quite pronounced in some cases. Photograph No. 12 illustrates this bending in some of the specimens.

Elongation of bolt holes at failure is an expected result which Photographs 12 and 13 clearly indicate.

3.4 ERRATIC RESULTS

Review of individual results in terms of the dimensionless quantities ϕ revealed that some quite erratic results were obtained. When it is recalled that all specimens were produced as regular production work in a structural steel fabricating shop and no attempt was made to select material which conformed exactly to its specified size then it is obvious that erratic results should be expected.

C.S.A. Specification G40.1 - 1966, Reference 9, permits the width of 2 or 3 inch angles to underrun by a maximum of 1/16 inch. In practice the gauge distance may over-run by 1/32 inch, thus the edge distance may under-run by a total of 3/32 inch. Such an under-run when compared with the net width of a 5/8 inch edge distance represents a 30% loss of effective area. Also, C.S.A. G40.1-1966 permits the thickness of 2 to 3 inch wide angles to under-run by as much as 6.9%

As mentioned under 3.0, "Experimental Results", a material mixup occurred in the fabrication of Test Pieces 701 to 704 inclusive. The detection of this obvious error in one case automatically introduces the possibility that similar errors may have occurred in other cases.

Thus the combination of sections which under-run, gauge distances which over-run and the possibility of the wrong material being used introduces a strong possibility of erratic results and these are accepted as a natural result of the general conditions prevailing in these tests.

4.0 ULTIMATE LOADS AND FAILURE FORMULAE

As indicated under "Experimental Results", all ultimate test loads were converted into the quantities ϕ_y and ϕ_u which are functions of the yield stress and ultimate tensile stress respectively. The form of ϕ is attributable more or less directly to the work reported by Winter (8) for his Type I and II failures.

In Figures 4.1 and 4.2, ϕ_y is plotted against edge distance for various values of end distance. These figures are alike except for the end distances represented in each. Two figures have been used to avoid the confusion of points which results from superimposing too many points on one figure. In each case, for a given end distance the average value of ϕ_y for end failures has been indicated by a horizontal dashed line. The length and horizontal location of these lines indicate the range of edge distances over which they apply. These ranges in turn are as indicated in Figure 3.10 for end failures.

The average values of Figures 4.1 and 4.2 for end failures are shown in Figure 4.3 with ϕ_y plotted against end distance. Note that this figure represents end failures only. For each of the plotted points the average value of the function is shown along with the number of tests represented by the average. Use of these points with the method of least squares and regression, see Neville and Kennedy Reference (12), gives the expression

$$\phi_y = 2.011\ x + 0.374 \tag{4.1}$$

Where x = End distance in inches

This line is shown in Figure 4.3 by the dashed line. The calculation of this equation is given in Appendix A as an example of the least squares and regression method.

In a similar manner, the plotting of ϕ_y against end distance in Figure 4.4 and averaging values of the function for edge failures at particular values of edge distance leads to Figure 4.5. This figure is similar to Figure 4.3 which indicates the variation of ϕ_y with end distance for end failures, while figure 4.5 indicates the variation of ϕ_y with edge distance for edge failures.

In Figure 4.5 the linear equation derived by the method of least-squares and regression is:

$$\phi_y = 4.0245\ y - 0.687 \tag{4.2}$$

Where y = Edge distance in inches

This line is shown by the dashed line in Figure 4.5 and has also been shown in Figures 4.1 and 4.2. Similarly, Equation 4.1 has been plotted in Figure 4.4.

4.1 FAILURE FORMULAE BASED ON ULTIMATE TENSILE STRENGTH

All of the Figures 4.1 to 4.5 inclusive and the two equations 4.1 and 4.2 indicate the variations of ϕ_y which is defined as a function of yield stress. With the substitution of ϕ_u for ϕ_y or ultimate tensile stress for yield stress, Figures 4.6 to 4.10 result. Again, by using the method of least squares and regression the relationships between the function and end and edge distances are:

$$\phi_u = 1.447\,x + 0.268 \tag{4.3}$$

$$\phi_u = 3.058\,y - 0.650 \tag{4.4}$$

Where x = End Distance in Inches

Where y = Edge Distance in Inches

These lines are shown as dashed lines in Figures 4.8 and 4.10 respectively, and have in turn been superimposed on Figures 4.9, 4.6 and 4.7.

It can be noted here that Equations 4.3 and 4.4 for ϕ_u, when taken simultaneously, define two planes in the three dimensional x, y, ϕ_u co-ordinate system. Similarly, Equations 4.1 and 4.2 define two planes in the x, y, ϕ_y co-ordinate system. More is said about these double planes later.

4.2 STATISTICAL BASIS OF FAILURE FORMULAE

For the four Equations 4.1 to 4.4 a check of the correlation of each was carried out using the methods of Neville and Kennedy (12). This study indicated that for each line the correlation is virtually perfect. An example of these calculations is contained in Appendix A. The correlation coefficients found are:

Equation 4.1	$r \approx$	0.997
Equation 4.2	$r \approx$	0.994
Equation 4.3	$r \approx$	0.998
Equation 4.4	$r \approx$	0.998

In order to justify the use of the method of least squares and regression in arriving at Equations 4.1 to 4.4, Bartlett's Test was performed to compare the variances of the data. For simplicity these tests were performed on mean values of the function ϕ rather than individual values. This simplification is justified when it is observed that in Figure 4.3 for example, one point is the mean of 112 tests. This same point is represented in Figure 4.1 by the horizontal dashed line labelled 1" End, Ave. 2.467. The average in this case is derived from the five plotted points of Figure 4.1. Thus, in Bartlett's Test for the data of Figure 4.3 an n value of 5 replaced an n of 112. The reduction in computational effort is thus obvious.

For all four Bartlett's Tests the results indicate that the variances are homogeneous and as such the use of the methods of least squares and regression is permissible. The values of χ^2 as calculated are given below along with tabulated values for a 10% level of significance. A sample calculation is contained in Appendix "A".

Equation No.	Values of χ^2 Calculated	Values of χ^2 Tabulated, 10%
4.1	13.289	15.507
4.2	9.745	11.050
4.3	8.298	15.507
4.4	6.645	11.050

In addition to the above noted statistical investigations all of the values of ϕ_y and ϕ_u for constant end distances as plotted in Figures 4.1, 4.2, 4.6 and 4.7 and the values for constant edge distances plotted in Figures 4.4 and 4.9 were investigated statistically. In each case the method of least squares and regression was used to obtain the regression line over the appropriate range. For each regression line thus derived the slope was tested for significant difference from zero. These tests indicate that of the thirty regression lines thus investigated four are different from zero at the ten per cent level of significance. The statistical significance of four in thirty was not investigated and the average values of ϕ for constant end and edge values were used in developing equations 4.1 to 4.4 as previously described.

4.3 CONFIDENCE LIMITS FOR FAILURE FORMULAE

For the failure formulae given in Section 4.1 it should be noted that the failure function ϕ is an estimate. Using the methods of Neville and Kennedy (12), the confidence limits of the regression estimates were calculated for each formula.

As pointed out in Reference (12) these calculations lead to a hyperbolic curve, which may be mathematically correct but is inconvenient to use in practice.

F. S. Acton in his book "Analysis of Straight Line Data", Reference (13) gives a method of converting the confidence limits from their hyperbolic form to two straight lines. Obviously the two straight lines are preferable for practical purposes but not as convenient as a single straight line. When it is remembered that it is hoped that the results contained here will be put into daily use in industry the possible convenience of a single straight line is quite apparent. With this in mind the following approximation was introduced.

Because of the symmetry of the confidence limits about the mean value of the function, the confidence interval at two points equally spaced above or below x or y are equal. Also, the confidence interval increases with increasing distance from the mean value of x or y. Taking the regression line of Figure 4.3 as an example the mean value of end distance x is 1-1/4 inches. Thus the confidence interval at two points equally spaced above or below x equal to 1-1/4 inches are equal. If now we calculate a given confidence interval at the points most distance from the mean value of x , that is at 3/4 and 1-3/4 inches and apply this interval over the complete range shown, we have a line which possesses the given level of significance at its extreme ends and a lesser level of significance at other points.

Appendix A contains the calculation of the confidence limits for Equation 4.1 which leads to the Equation

$$\phi_y = 2.011 \ x + 0.279 \qquad (4.5)$$

This line is shown in Figure 4.3 and indicates the value of ϕ_y at the ten per cent level of significance.

Similar calculations give:

$$\phi_y = 4.024 \ y - 0.901 \qquad (4.6)$$

$$\phi_u = 1.447 \ x + 0.208 \qquad (4.7)$$

$$\phi_u = 3.058 \ y - 0.745 \qquad (4.8)$$

These lines are each shown in Figures 4.5, 4.8, and 4.10 respectively and each represents the given function at the ten per cent level of significance.

4.4 RANGE OF END AND EDGE FAILURES

As mentioned in Section 4.1 the foregoing equations for ϕ_y and ϕ_u each occur in pairs as functions of x and y thus defining two planes in the x, y, ϕ_y and x, y ϕ_u co-ordinate systems respectively. By combining the equations of the preceding section the following equations result.

In the x, y, ϕ_y system $y = 0.500 \ x + 0.293$ (4.9)

In the x, y, ϕ_u system $y = 0.473 \ x + 0.312$ (4.10)

These equations define the dividing line between end and edge type failures in the x - y plane. Superimposing these equations on Figure 3.1 gives Figure 4.11. The agreement between these lines with those combinations of end and edge distances which produced both end and edge type failures should be noted.

5.0 COMPARISON OF RESULTS WITH TESTS ON FLAT BARS

George Winter, in his paper "Tests on Bolted Connections in Light Gage Steel", (8) found that failures which correspond with Types I, II and III as used here, group in a satisfactory manner around the straight line

$$\frac{P}{\sigma_y \; t} = 1.40 \; e \qquad (5.1)$$

Where P, σ_y and t are as previously defined

While e = End Distance x as used herein

If this equation is re-written in terms of the failure function ϕ_y the result is

$$\phi_y = \frac{P}{d \times t \times \sigma_y} = \frac{1.40 \; x}{d}$$

Substitution of 0.625 inches for the bolt diameter d gives the following straight line which has been shown in Figure 4.3

$$\phi_y = 2.24 \; x \qquad (5.2)$$

The agreement between this expression and equations 4.1 and 4.5 is quite good.

Winter indicates that Equation 5.1 applies for values of x/d not exceeding 3.5

This limit can be restated for 5/8 inch diameter bolts as x not exceeding 2.19 inches. Use of this upper limit gives

$$\phi_y \text{ Max} = 2.24 \times 2.19 = 4.91$$

For the tests reported herein, an end failure was not produced at a value of x as high as 2.19 inches and a value of ϕ_y as high as 4.91 was not produced. Thus, Winter's upper limit is not confirmed in these tests but neither is it refuted. In those cases where x was greater than 2.19, end failure was not obtained as edge failure occurred first. The highest average value of ϕ_y obtained was 4.784 for an end and edge distance of 2 inches and 1-3/8 inches respectively. The test of Figure 3.7 with an x of 2 inches was loaded to a maximum ϕ_y value of 4.6 without failure thus approaching Winter's maximums.

For transverse tearing Winter suggests that these failures can be represented by

$$\sigma_n = (0.10 + 3.0 \frac{d}{s}) \quad \sigma_u \leq \sigma_u \qquad (5.3)$$

Where σ_n = Stress on the Net Section - P.S.I.

σ_u = Ultimate Tensile Stress - P.S.I.

d = Bolt Diameter - Inches

s = Width of Bar - Inches

This expression applies to single bolted bars of width s with the bolt on the longitudinal centre line of the bar. For comparison with the failure formulae 4.4 developed herein for angles, the width of the equivalent flat bar must be calculated.

Reference (14) defines the net section of a single angle connected by one leg as the net area of the connected leg plus one half of the area of the unconnected leg. Using this criteria and assuming that angles with equal legs are gauged such that the gauge distance and edge distance are equal we have

$$S = 3.0\,y \qquad (5.4)$$

Substitution of this value of S into Equation 5.3 gives

$$\phi_u = 4.8\,y - 1 \qquad \text{for } y < 0.694 \text{ Inches} \qquad (5.5)$$

$$\phi_u = (4.8y-1)\left(0.10 + \frac{0.625}{y}\right) \text{ for } y > 0.694 \text{ Inches} \qquad (5.6)$$

These equations are shown in Figure 4.10 labelled $S = 3.0\,y$ This curve is not very close to Curves 4.4 and 4.8 as shown in the figure. By trial and error the use of $S = 2.25\,y$ leads to

$$\phi_u = 3.6\,y - 1 \qquad \text{For } y < 0.926 \text{ Inches} \qquad (5.7)$$

$$\phi_u = (3.6\,y - 1)\left(0.10 + \frac{0.833}{y}\right) \quad \text{For } y > 0.926 \text{ Inches} \qquad (5.8)$$

These curves are also shown in Figure 4.10, labelled $S = 2.25\,y$, and are in fair agreement with Equation 4.4 for values of edge distance up to about 1-1/8 inches. The break in the curve corresponds with $0.30\,S = d$ as given by Winter.

6.0 LIMITING BEARING VALUES

As mentioned in Chapter 5.0, Winter suggests that his results for end type failures level off at a limiting bearing stress equal to 4.9 times the yield stress. As shown in Figures 4.3 and 4.5 the angle connection tests reported here did not level off at a limiting value of bearing stress. Such a limit had been expected and the range of end and edge distances used was expected to produce this stress but did not. In view of this result, or lack of an expected result, the definition of ultimate bearing stress is open to question.

Bearing stress at a bolt, by definition, is given by

$$\sigma_b' = \frac{P'}{t \times d}.$$

Where P' = Load on the bolt - Lbs.

t = Thickness of the Connected Material - Inches

d = Bolt Diameter - Inches

Thus the ratio of bearing stress to yield stress is

$$\frac{\sigma_b'}{\sigma_y} = \frac{P'}{t \times d \times \sigma_y} = \phi_y'$$

This equation is of the same form as the failure formula for ϕ_y except that the load is not necessarily the ultimate load on the connection hence the use of the prime marks in P' and ϕ_y'. This relationship between σ_b' and σ_y, in fact, determined the form of ϕ_y as used in this study. Thus in all cases reported herein the value of ϕ_y is the ratio of bearing stress to yield stress at failure of the test connection. Also, in Figures 3.7, 3.8 and 3.9 the right hand scale shown is ϕ_y' as defined above.

Reference 15 gives allowable working tension in A.S.T.M. A-325 bolts as 40,000 P.S.I. and allowable bearing stress as 1.35 σ_y. As A.S.T.M. A-325 bolts have an ultimate tensile stress of 120,000 P.S.I., Table 1.2, the factor of safety in tension on these bolts is 3. If it is assumed that the same factor applies in bearing the ultimate bearing stress is 4.05 σ_y or 0.83 times the ultimate bearing value suggested by Winter.

If the factor of safety included in Reference 15 is applied to the maximum bearing stress found by Winter the following results

$$\sigma_b \text{ Max.} = 4.9\ \sigma_y \tag{6.1}$$

Factor of safety = 3

$$\therefore\ \sigma_b \text{ allowable} = \frac{4.9}{3} = 1.63\ \sigma_y$$

As noted above, reference 15 gives

$$\sigma_b \text{ Allowable} = 1.35\ \sigma_y$$

Factor of safety = 3

$$\sigma_b \text{ Maximum} = 1.35 \times 3\ \sigma_y = 4.05\ \sigma_y \tag{6.2}$$

As a compromise value, the average of (6.1) and (6.2) gives approximately

$$\sigma_b \text{ Max.} = 4.5\ \sigma_y \tag{6.3}$$

Reference to Table 3.7 shows that for the average values of ϕ_y tabulated, a value of σ_b greater than 4.5 σ_y was exceeded only with an edge and end distance combination greater than 1-1/4 and 1-7/8 inches respectively. Referring to the tests of Figures 3.7, 3.8 and 3.9 which lie within this range of end and edge distances the results are of interest. In Figure 3.7 a value of ϕ_y' equal to 4.6 was obtained with residual hole elongation of 0.207 inches. For Figure 3.8 loading was terminated by failure of the specimen at a value of ϕ_y equal to 4.36. In this case the hole elongation was in excess of 3/16 inch. The specimen of Figure 3.9 was loaded to give a value of ϕ_y' of 3.5 and residual hole elongation of 0.223 inches.

The commentary appended to Reference 15 states "Tests have shown that bearing pressure on rivets in double or single shear, computed on the basis of an area equal to the product of the part thickness and nominal rivet diameter has no significant effect on the strength of the connected parts of A7 steel when this pressure is not more than 2.25 times the tensile stress applied to the net area of these parts." In Figures 3.7, 3.8 and 3.9 this statement is confirmed by the elongations of 0.025, 0.020 and 0.037 inches respectively at values of ϕ_y' equal to 2.25. By comparison, Figures 3.3, 3.4, 3.5 and 3.6 indicate that, for the strain gauged specimens, yielding first occurred at values of ϕ_y' equal to 0.629, 0.539, 0.670 and 0.670 respectively. These values are well below the value 2.25 given above and as quoted from the commentary on Reference 15. In Table 3.7 a value of ϕ_y greater than 2.25 is not developed with end and edge distances less than one inch and 3/4 inch respectively.

In Photograph No. 15 pairs of specimens are shown in which the left hand specimen has been loaded to failure while the right hand one has been unloaded before failure occurred. For these particular tests the following data applies to the unbroken specimens of Photograph No. 15 taken from left to right.

Test No.	Maximum Load P'	P'/P	ϕ_y'	Hole Elongation
800	11,370	1.112	2.807	0.025
803	14,500	0.950	3.664	0.024
808	14,330	0.743	3.440	0.154
813	17,500	0.866	3.147	0.069

These test results illustrate the capacity of an angle connection to support a large percentage of its ultimate load successfully with relatively small distortion being caused at the bolt hole. Note that for Test No. 800 the unbroken specimen was loaded to 112% of the average ultimate load of the specimen as given by the other two pieces of the same mark.

7.0 RELATIONSHIP BETWEEN ϕ_y AND ϕ_u

As developed herein the failure function ϕ can be predicted by use of either the yield stress or the ultimate tensile stress of the specimen. By equating predicted ultimate load a relationship between σ_y and σ_u results, as follows, taking end failures as an example.

$$\phi_y = 2.011\ x + 0.374 = \frac{P}{d \times t \times \sigma_y} \qquad (4.1)$$

$$\phi_u = 1.447\ x + 0.268 = \frac{P}{d \times t \times \sigma_u} \qquad (4.3)$$

Equating values of ultimate load P gives

$$\sigma_y\ (2.011\ x + 0.374) = \sigma_u\ (1.447\ x + 0.268)$$

or

$$\frac{\sigma_y}{\sigma_u} = \frac{1.447\ x + 0.268}{2.011\ x + 0.374} \qquad (7.1)$$

Substitution of values of end distance x into this equation gives an average value of the ratio of σ_y to σ_u of about 0.72.

Similarly, for ϕ_y and ϕ_u as functions of edge distance y this ratio averages out at very nearly the same figure.

Reference to Table 1.1 for typical steels used in tower structures indicates that for these steels the average value of the ratio σ_y/σ_u is 0.645 for the physical properties as specified.

Reference to Tables 2.1 to 2.6 inclusive for the actual steels used in these tests gives σ_y/σ_u' as 0.718. This is the mean value of 78 values. Thus the derived equations for the failure functions ϕ_y and ϕ_u give ratios of yield stress to ultimate stress equal to those found from the tensile test coupons of these steels, but higher than the ratios derived from the specifications for the same steels.

In view of these ratios, for a given connection, if the ultimate load is predicated based on the formulae derived herein and using specified physical properties of the steel the values derived from ϕ_y will be less than those derived from ϕ_u and will be on the conservative side. This result is very convenient in industry as most specifications for tower structures as well as those covering buildings and bridges relate allowable working stresses to yield strength.

8.0 MULTI-BOLT CONNECTIONS

The application of the results reported here to Multi-Bolt Connections requires further study and testing but two points appear reasonable. In an angle connection with two bolts on one gauge line in one leg of the angle only, it is expected that the bolt nearest the end of the angle will develop load as predicted by the formulae given herein with either end or edge type failure depending upon the detail dimensions involved. At the second bolt it is expected that the maximum load developed will be a function of edge distance only. The inter-action of the two bolts could be quite complicated. Obviously, with a small edge distance failure at the bolt removed from the end would occur before the end bolt could develop its full load. Conversely, with a short end distance at the end bolt this bolt could fail through the end leading to overloading of the second bolt.

As indicated above the ultimate capacity of Multi-Bolt Connections requires further study before the results given here can be applied with a reasonable degree of confidence.

9.0 VARIATIONS IN BOLT DIAMETER

As stated earlier, most tower structures are built with 5/8 inch diameter bolts in 11/16 inch diameter holes. This fact dictated the bolt diameter used in these tests. It should be noted, however, that testing with bolts of other diameters was not covered. In view of this the failure formulae should not be applied to connections using other sizes of bolts without further testing and review of these results.

10.0 CONCLUSIONS

Based on the results of 721 single angle single bolted connections,using 5/8 inch diameter bolts, the following conclusions can be reached.

1. Failure through either the end or edge is a distinct function of end and edge distance and can be predicted from Figures 3.1 or 4.11.
2. For end type failures the ultimate load is reasonably given by

   $P \cong 5/8 \times t \times \sigma_y \; (2.011\, x + 0.279)$

   Where x = End distance in inches

   t = Thickness of angle in inches
3. For edge type failures the ultimate load is reasonably given by

   $P = 5/8 \times t \times \sigma_y \; (4.024\, y - 0.901)$

   Where y = Edge distance in inches

   t = Thickness of angle in inches
4. Failure in bearing occurs at a nominal bearing stress equal to 4.5 times the yield stress.
5. Bearing stresses equal to 2.25 times the yield stress can produce insignificant hole elongation depending upon end and edge distance.
6. Bearing stresses equal to 2.25 times the yield stress

cannot be developed with end distances less than one inch or edge distance less than 5/8 inch.

7. The development of local stresses in the immediate neighbourhood of the hole, equal to or greater than the yield stress is not a reasonable indication of approaching failure of the connection.

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7. The Algoma Steel Corporation Limited Algoma's Guide to the Selection of Steel Specifications for Structurals & Plate.

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13. F. S. Acton, Analysis of Straight Line Data, Wiley Publications in Statistics, John Wiley and Sons, Inc. 1959.

14. Design Section, Transmission Division, Engineering Design Services, Hydro-Quebec, Technical Specification for the Design, Fabrication and Supply of Steel Transmission Towers, Document No. 58-1965.

15. Specifications for Structural Joints Using A.S.T.M. A-325 or A-490 Bolts Approved by Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, Endorsed by American Institute of Steel Construction, Endorsed by Industrial Fasteners Institute, March, 1964.

PHOTOGRAPHS

SINGLE ANGLE SINGLE BOLTED

CONNECTION IN A TRANSMISSION TOWER

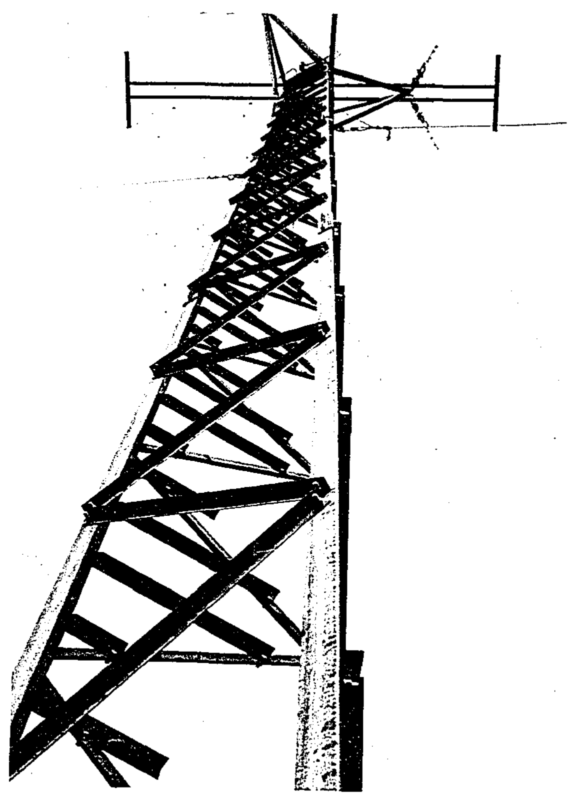
PHOTOGRAPH NO. 1

SINGLE ANGLE SINGLE

BOLTED CONNECTIONS

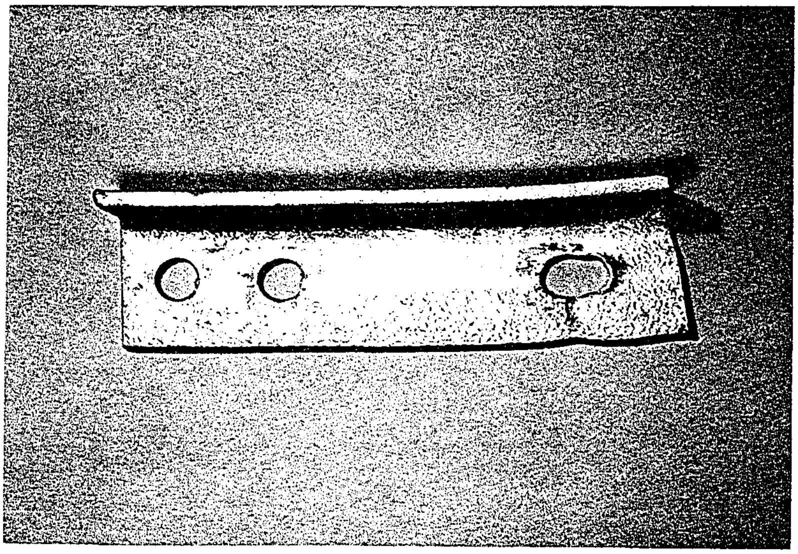
IN A GUYED MAST

PHOTOGRAPH NO. 2



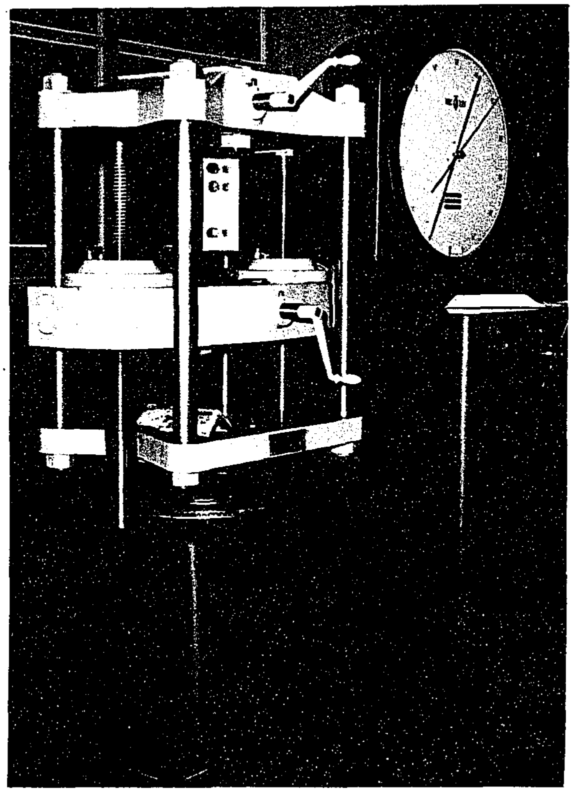
SPECIMEN AS SHEARED & PUNCHED

PHOTOGRAPH NO. 3



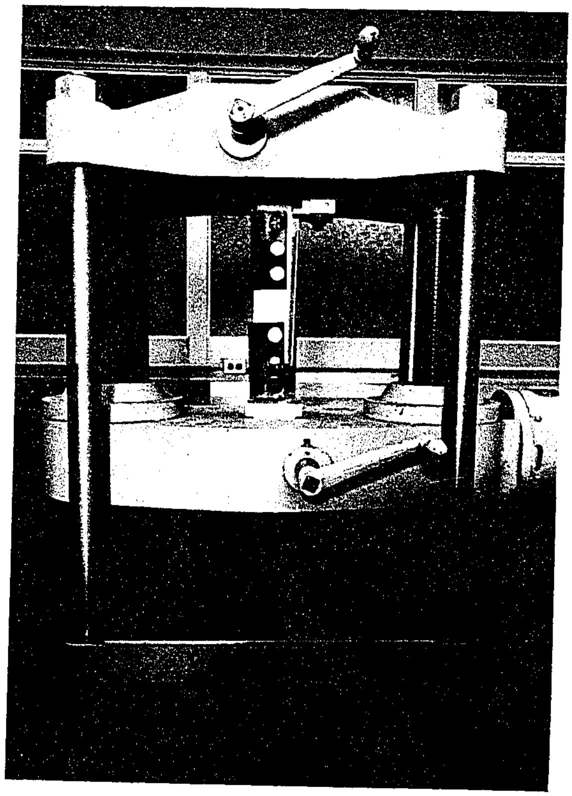
RUST-PITTED SPECIMEN

PHOTOGRAPH NO. 4



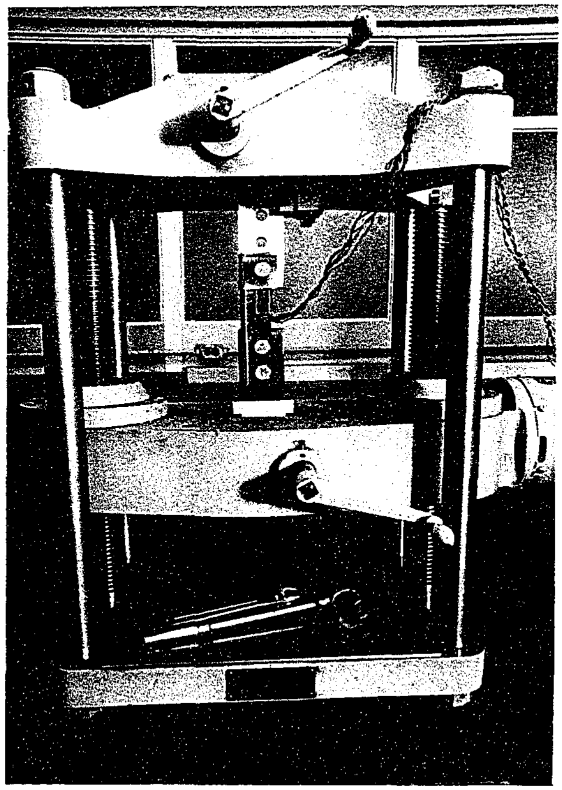
SINGLE SHEAR TEST

PHOTOGRAPH NO. 5



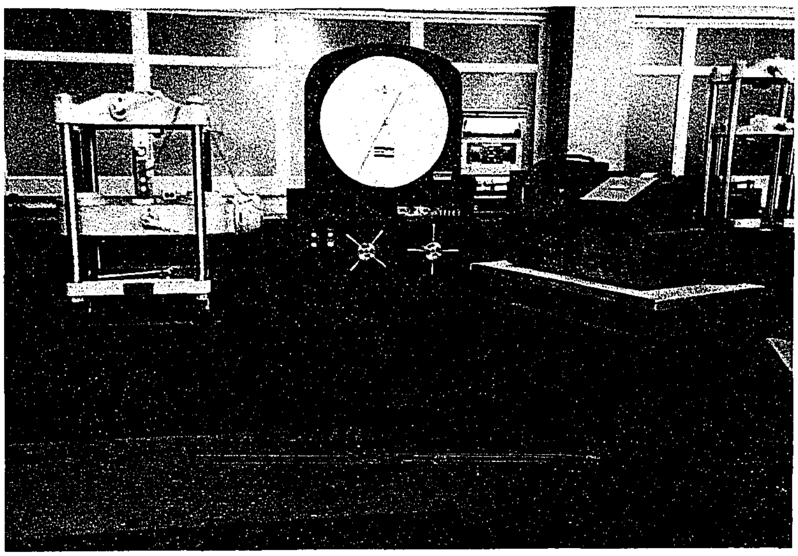
DOUBLE SHEAR TEST

PHOTOGRAPH NO. 6



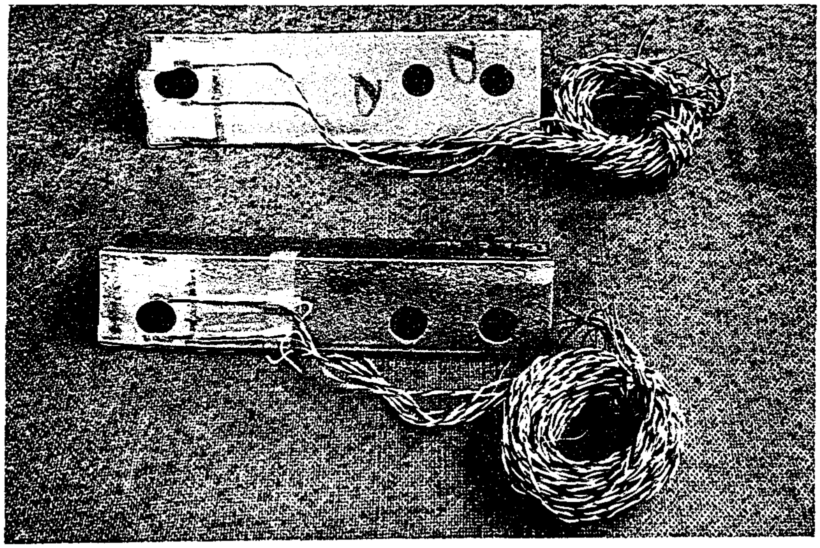
STRAIN GAUGED SPECIMEN

PHOTOGRAPH NO. 7



STRAIN GAUGED SPECIMEN UNDER TEST

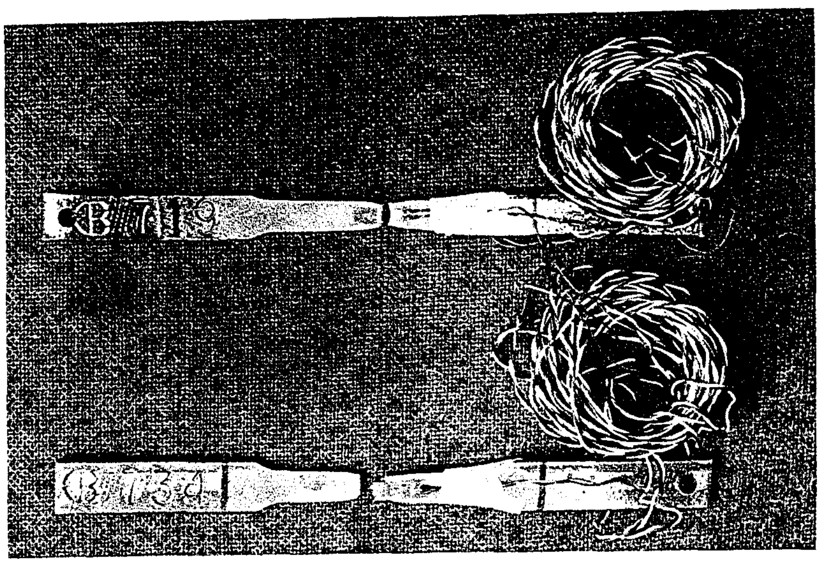
PHOTOGRAPH NO. 8



STRAIN GAUGED SPECIMENS

AFTER TESTING

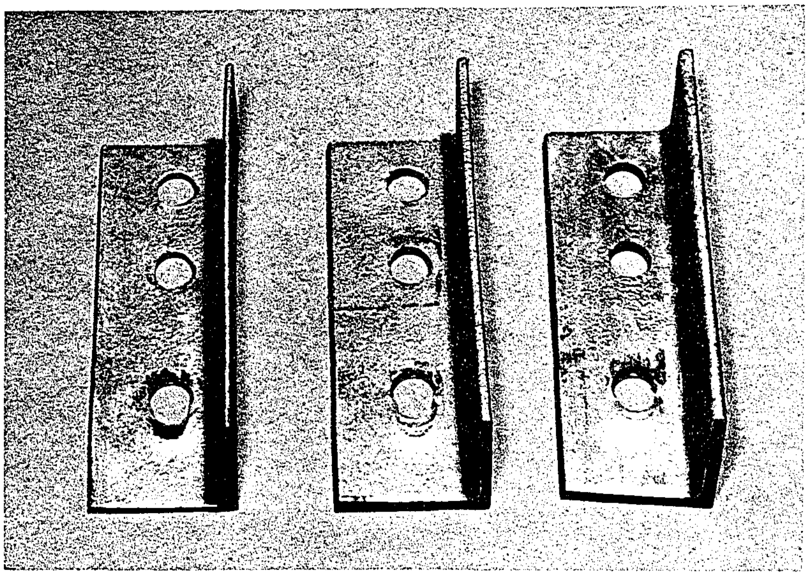
PHOTOGRAPH NO. 9



STRAIN GAUGED TENSION COUPONS AFTER

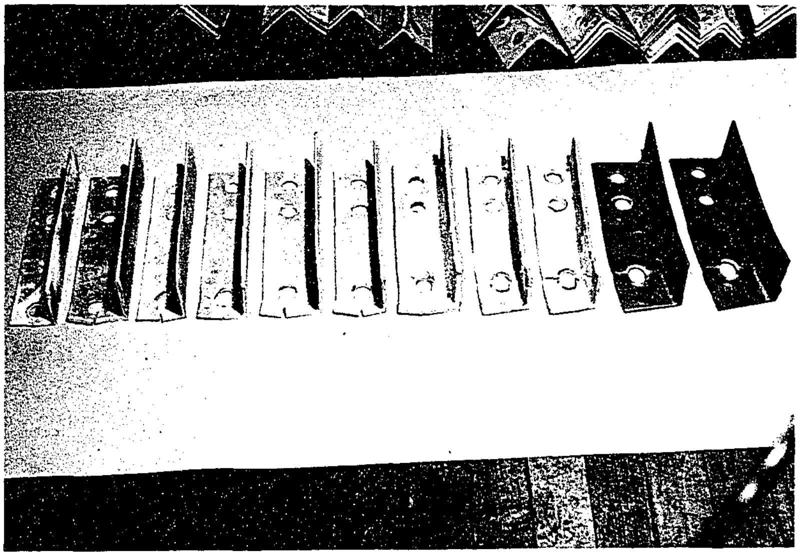
TESTING

PHOTOGRAPH NO. 10



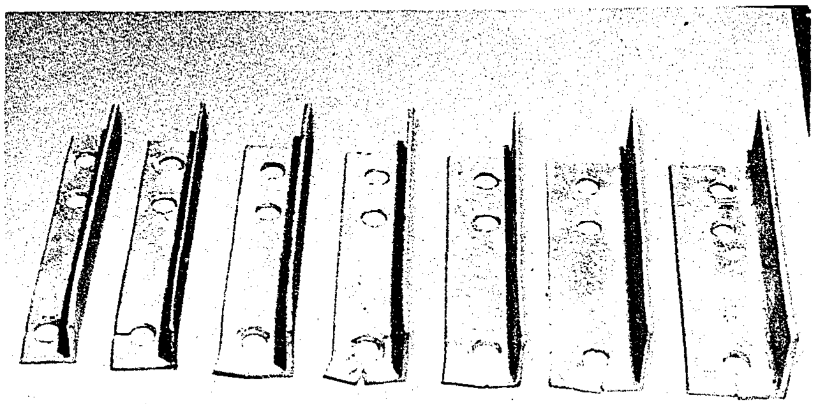
SPECIMENS FROM HOLE ELONGATION TESTS

PHOTOGRAPH NO. 11



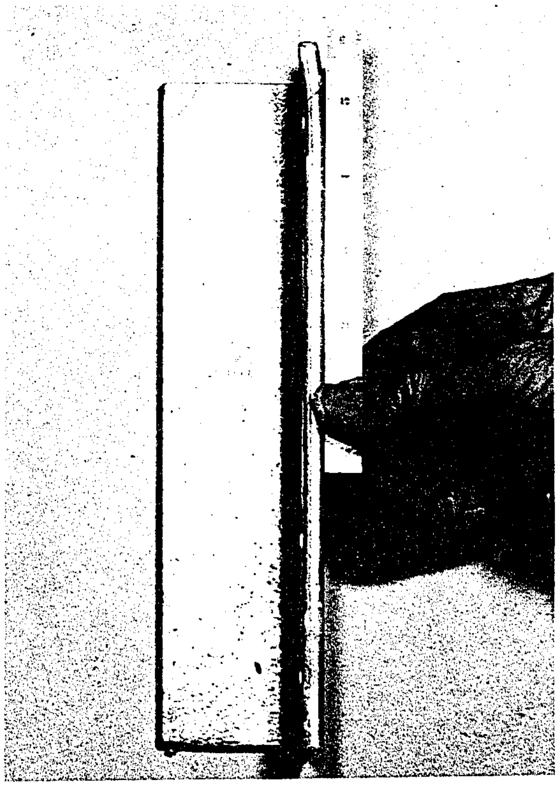
SPECIMENS WITH CONSTANT EDGE DISTANCE

PHOTOGRAPH NO. 12



SPECIMENS WITH CONSTANT END DISTANCE

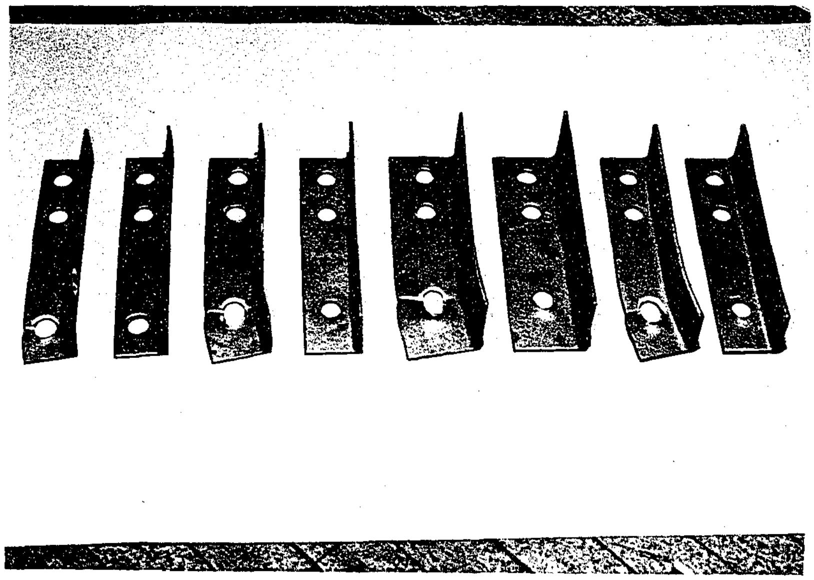
PHOTOGRAPH NO. 13



OUT OF PLANE BENDING OF

SPECIMEN

PHOTOGRAPH NO. 14

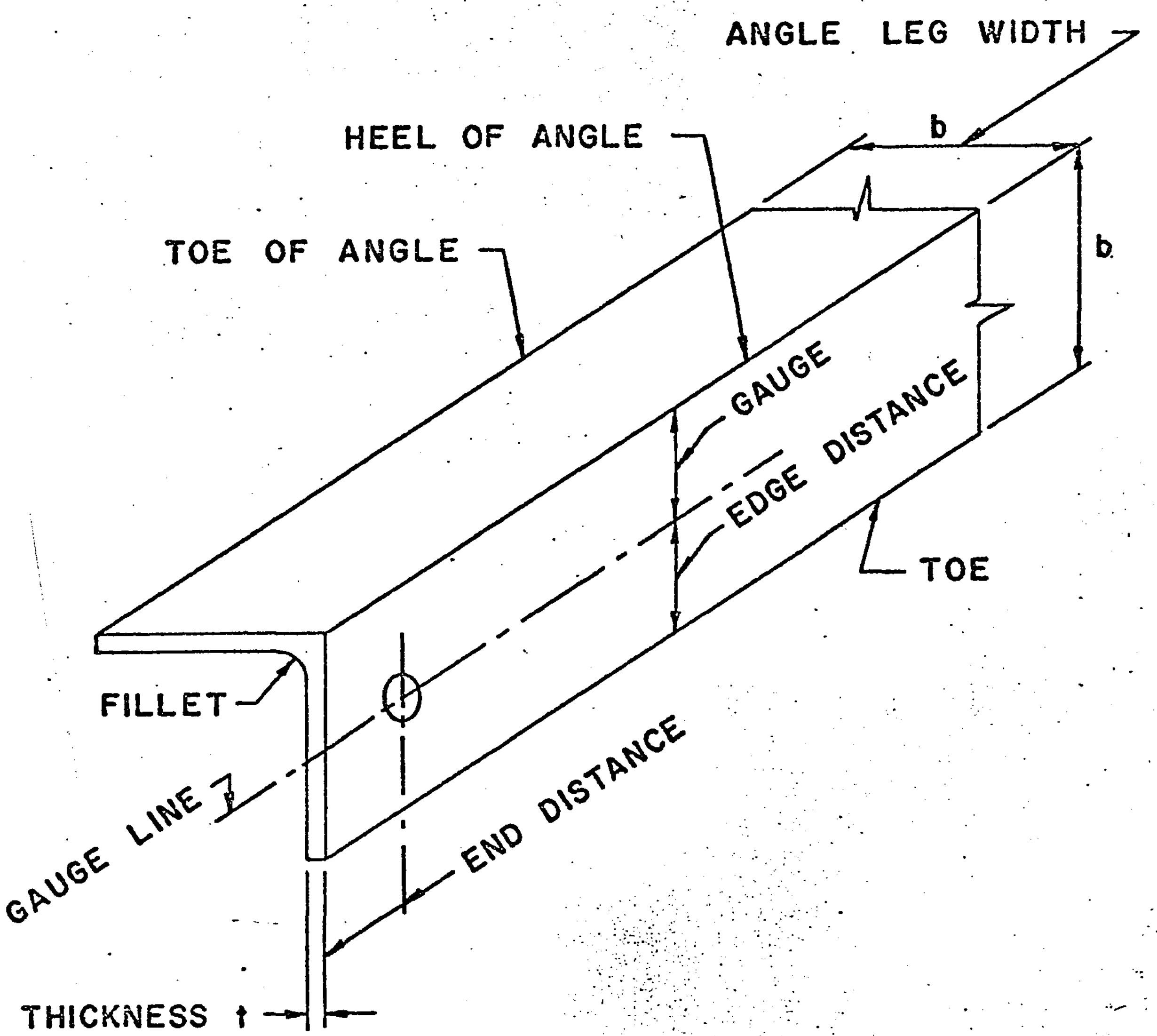


BROKEN AND UNBROKEN PAIRED

SPECIMENS

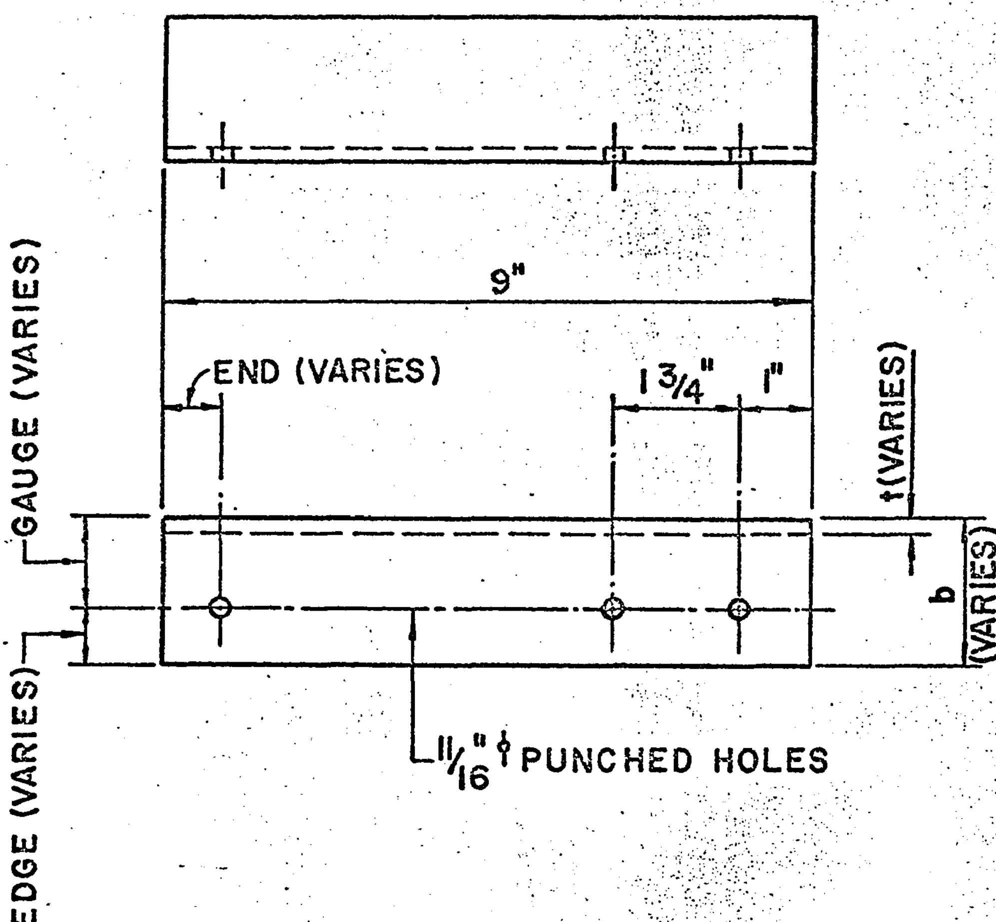
PHOTOGRAPH NO. 15

FIGURES



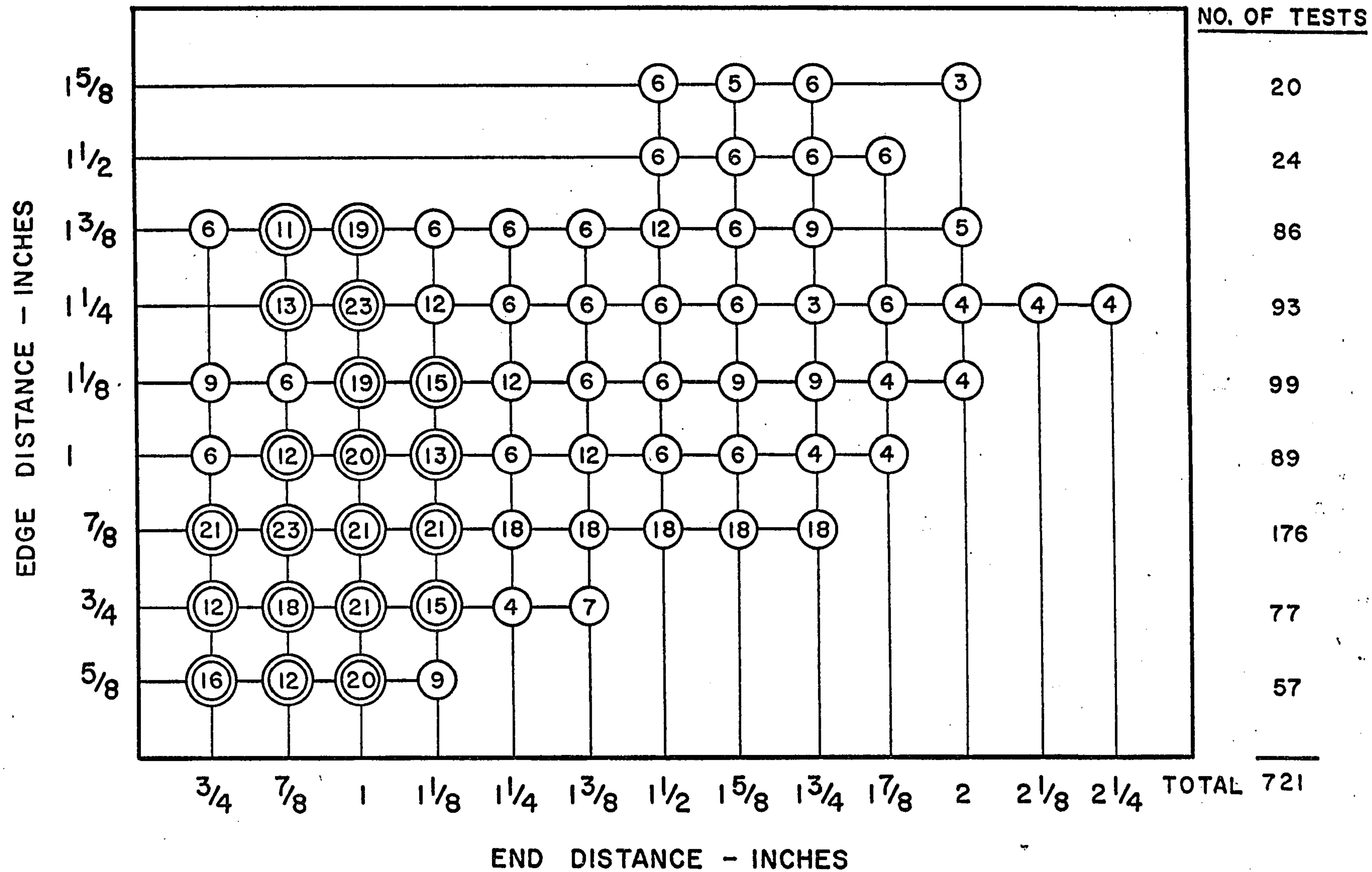
END PORTION OF A SINGLE ANGLE MEMBER

FIGURE 1.1



DETAIL OF TEST SPECIMEN

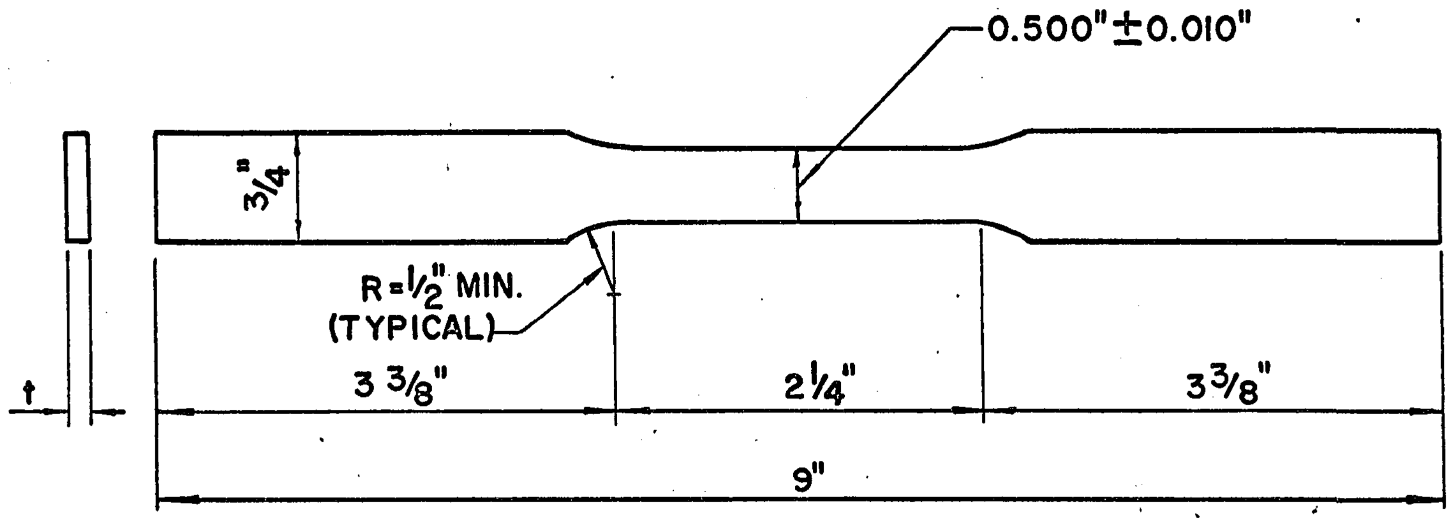
FIGURE 2.1



TABULATION OF NUMBER OF TEST SPECIMENS

INDICATES SAMPLES THAT WERE TESTED DURING 1962 & THE LATER TEST SERIES

FIGURE 2.2



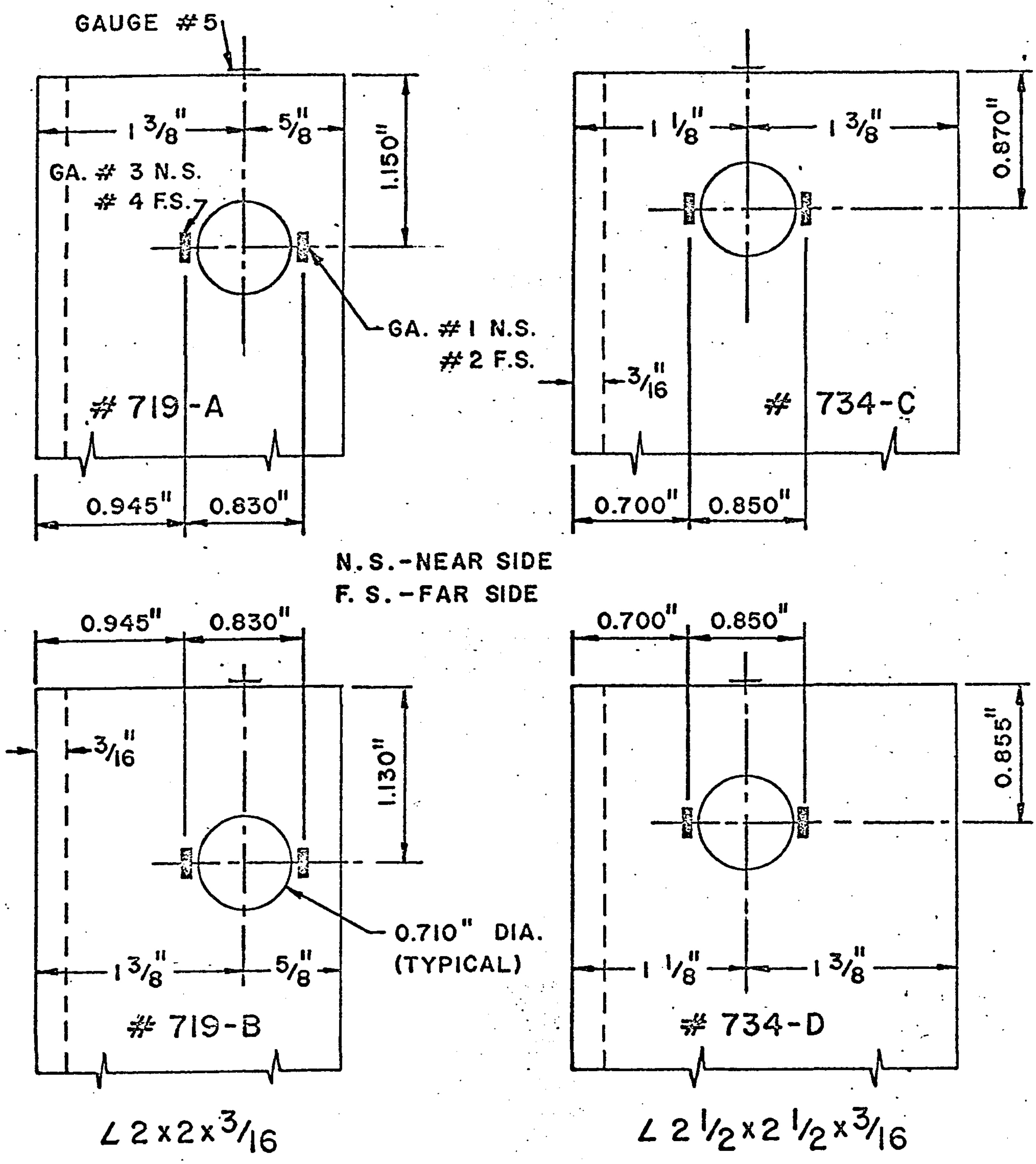
(1) t = THICKNESS OF ORIGINAL ANGLE SECTION

(2) THE EDGES OF THE REDUCED SECTION SHALL BE PARALLEL WITHIN 0.005" AND MAY HAVE A GRADUAL TAPER IN WIDTH FROM THE ENDS TOWARD THE CENTRE, WITH THE ENDS NOT MORE THAN 0.005" WIDER THAN THE CENTRE.

(3) THE ENDS OF THE SPECIMEN SHALL BE SYMMETRICAL WITH THE CENTRE LINE OF THE REDUCED SECTION WITHIN 0.05"

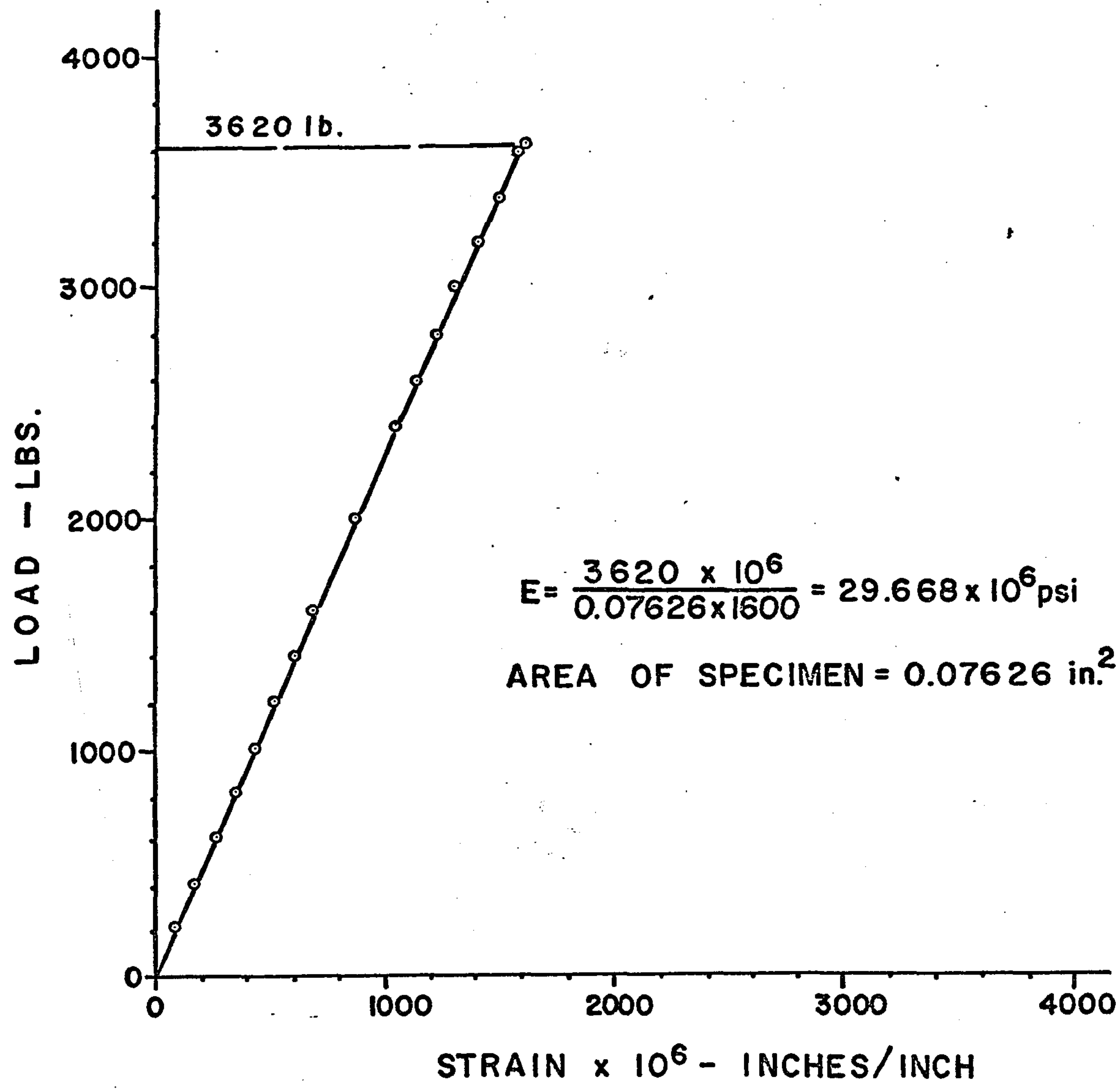
DETAIL OF RECTANGULAR TENSION TEST COUPON

FIGURE 2.3



DETAIL OF TEST PIECES SHOWING
LOCATIONS OF STRAIN GAUGES

FIGURE 2.4



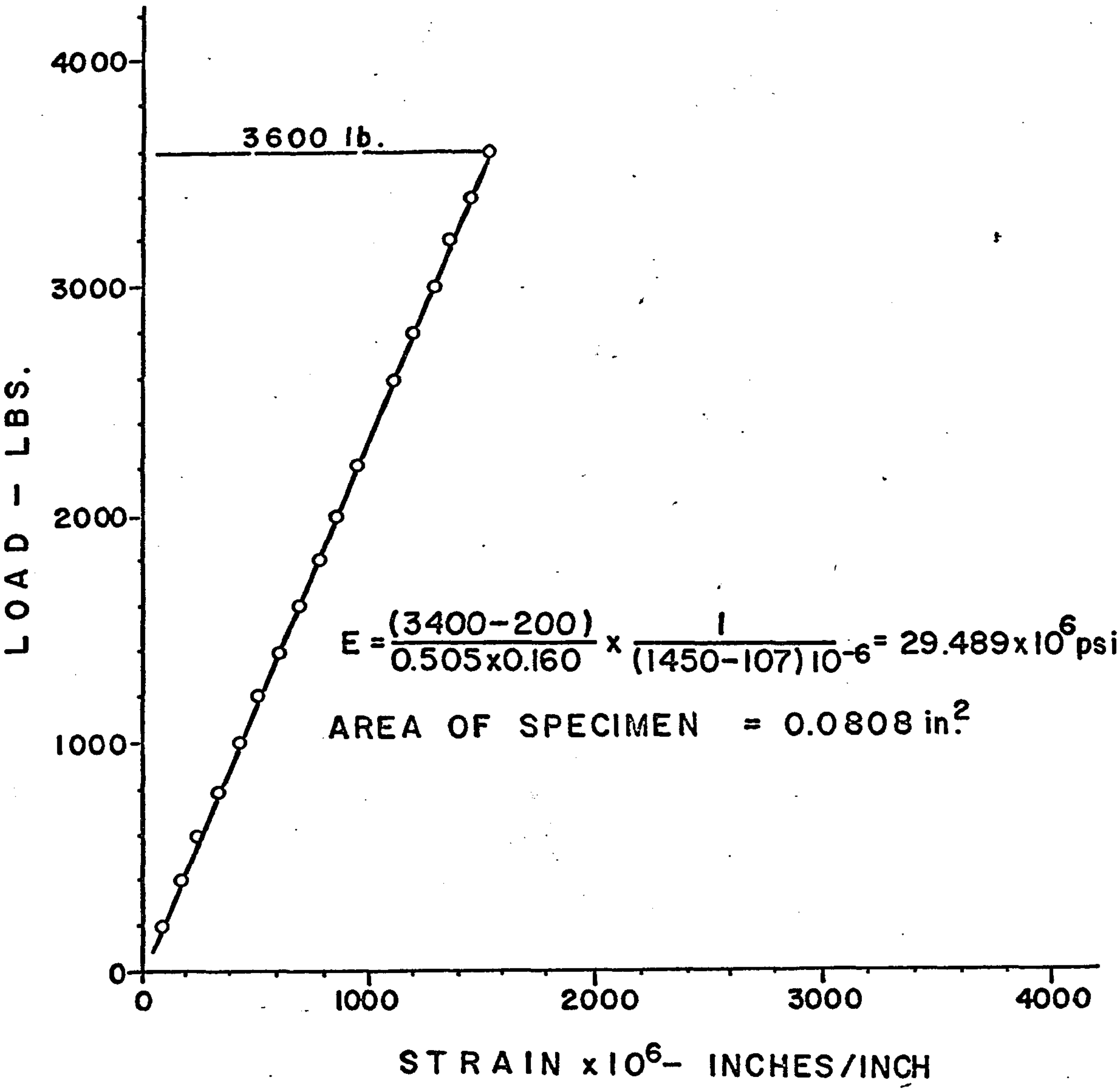
LOAD-STRAIN CURVE FOR TENSION

TEST SPECIMEN #719

$$\text{Yield Stress} = \frac{3620}{0.07626} = 47{,}469 \text{ \#/Sq. in.}$$

$$\text{Ultimate Stress} = \frac{5485}{0.07626} = 71{,}925 \text{ \#/Sq. In.}$$

FIGURE 3.1



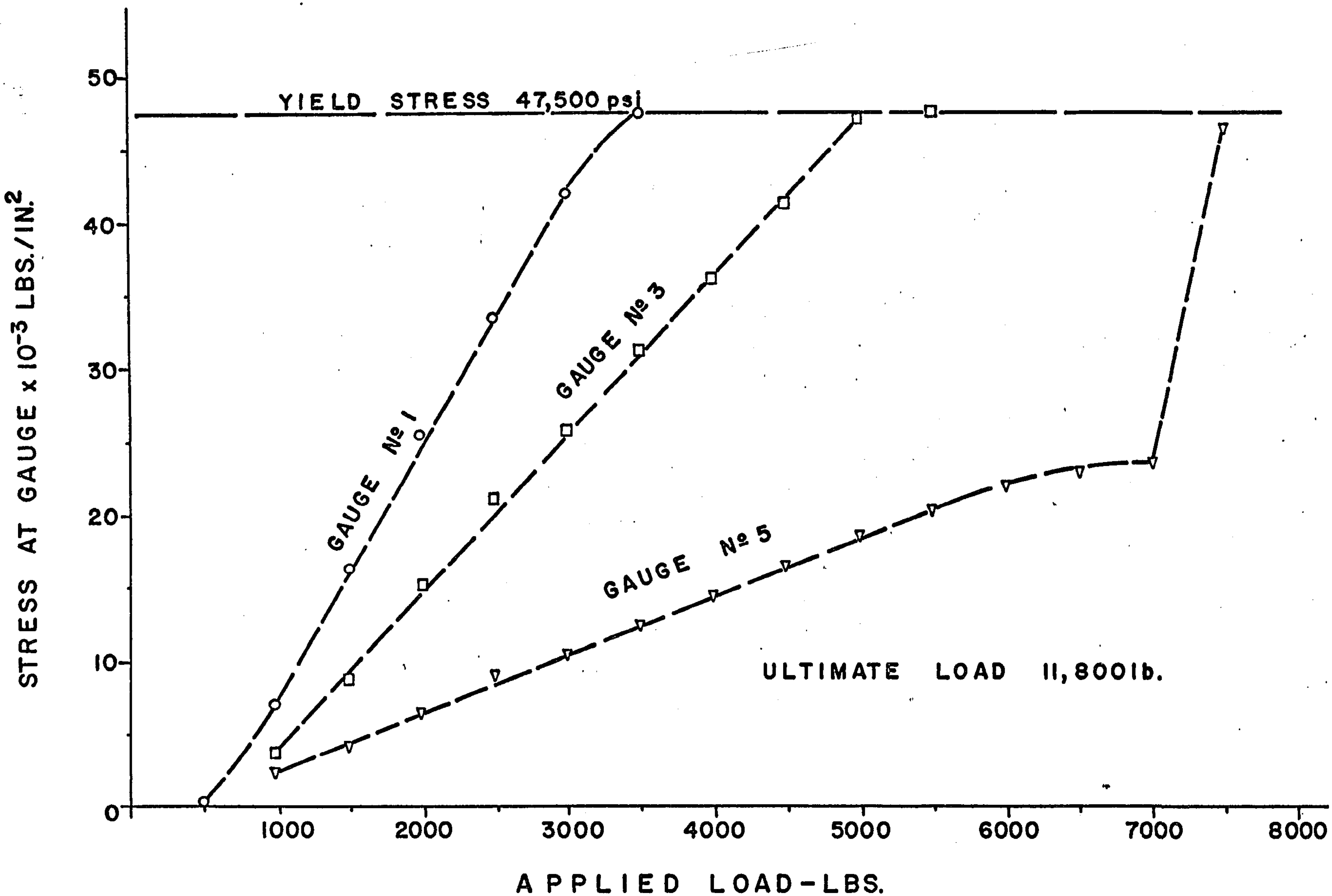
LOAD–STRAIN CURVE FOR TENSION

TEST SPECIMEN #734

Yield Stress $= \dfrac{3600}{0.0808} = 44{,}554$ #/Sq. In.

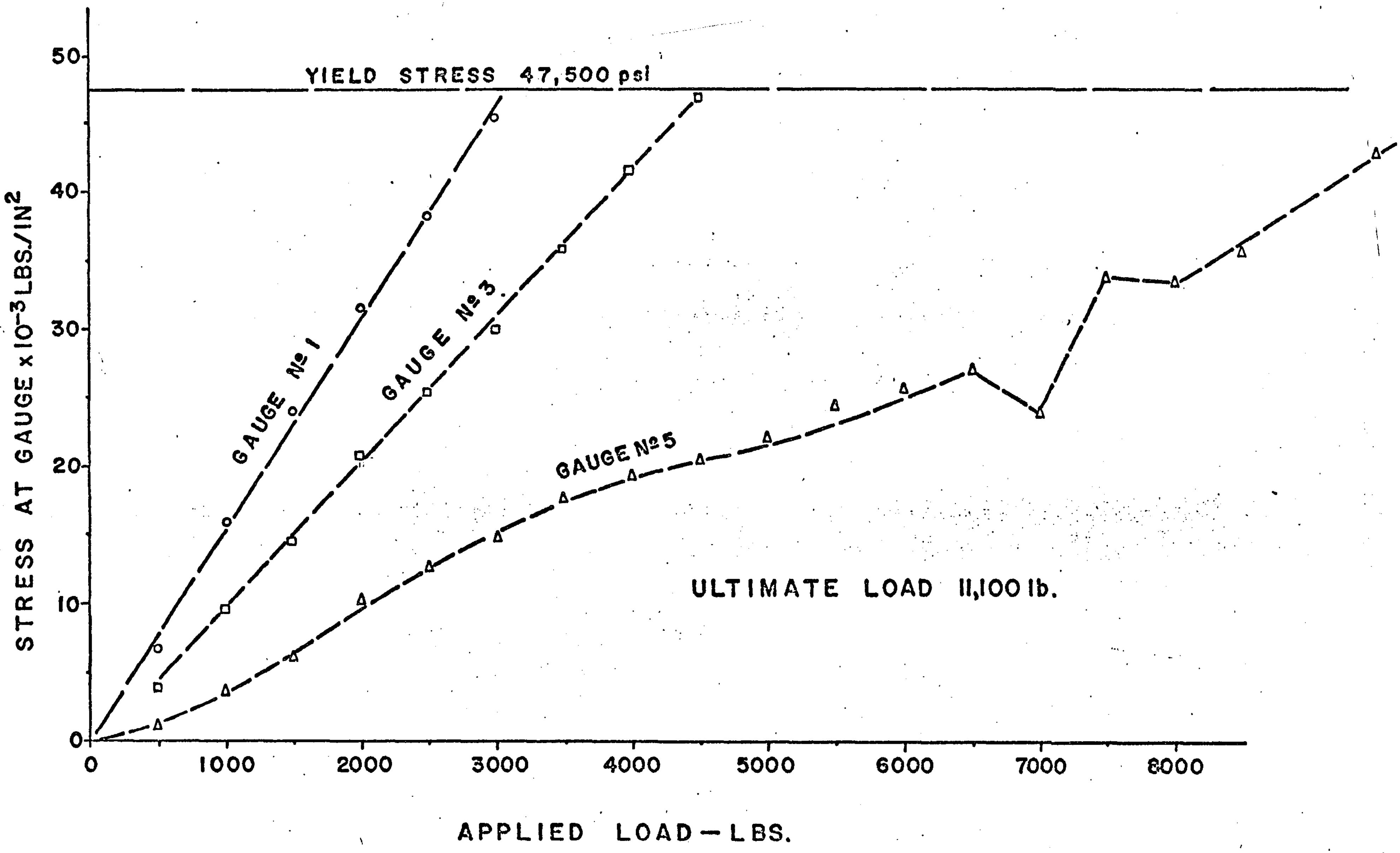
Ultimate Stress $= \dfrac{5720}{0.0808} = 70{,}792$ #/Sq. In.

FIGURE 3.2



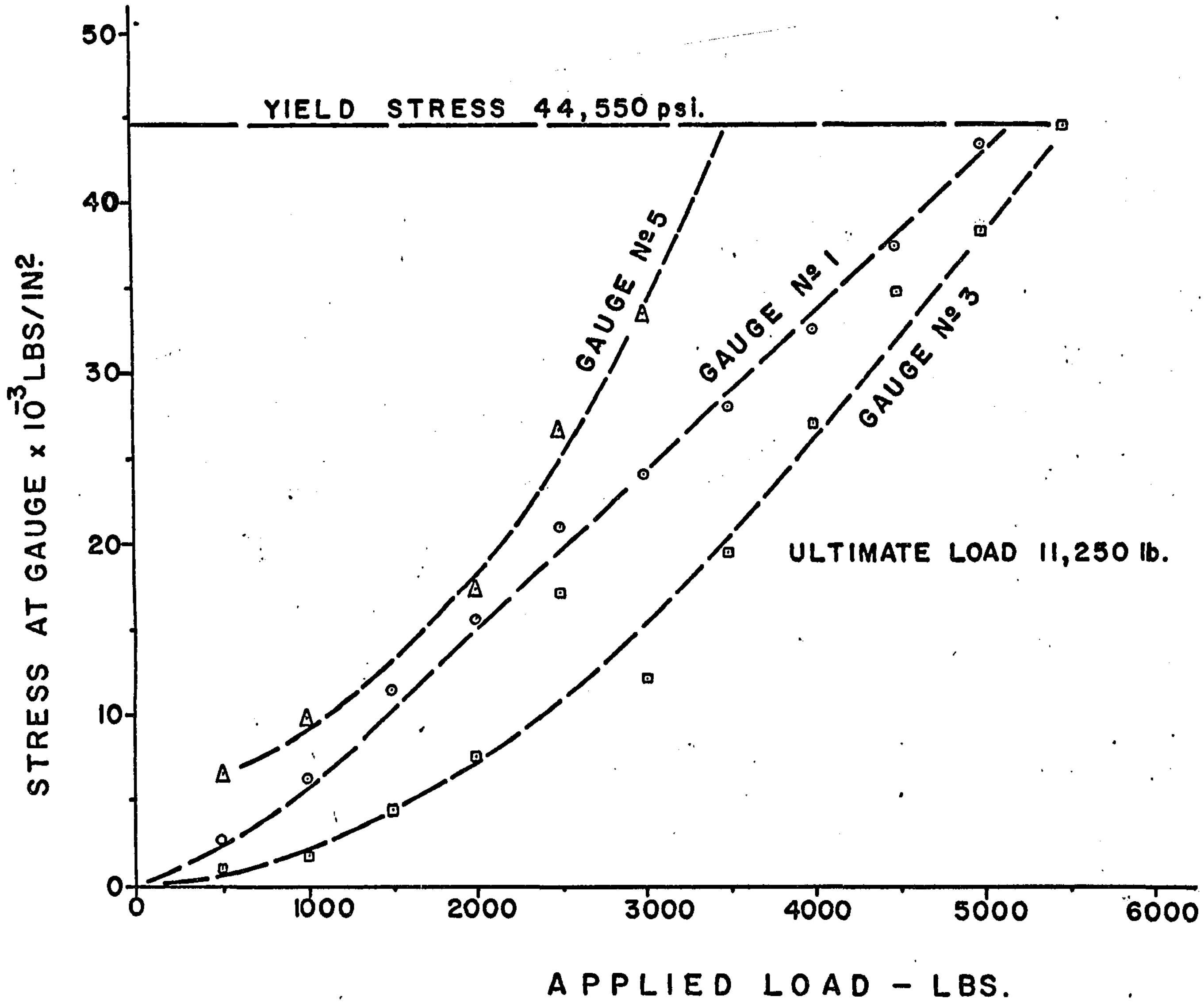
STRESS vs. CONNECTION LOAD FOR TEST NO. 719-A

FIGURE 3.3



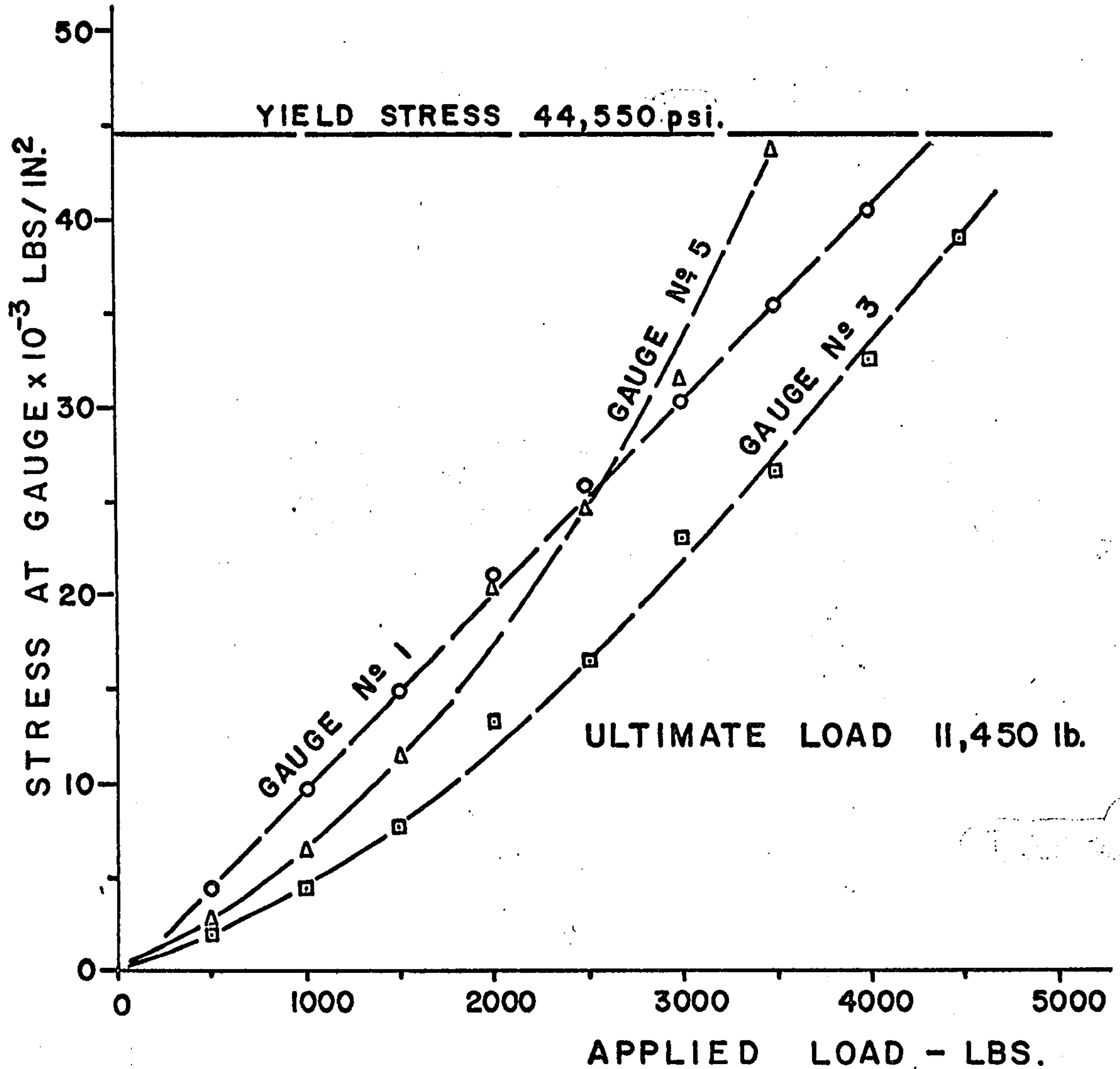
STRESS vs CONNECTION LOAD FOR TEST NO. 719-B

FIGURE 3.4



STRESS vs CONNECTION LOAD FOR TEST NO. 734–C

FIGURE 3.5



STRESS VS CONNECTION LOAD FOR TEST NO. 734-D

FIGURE 3.6

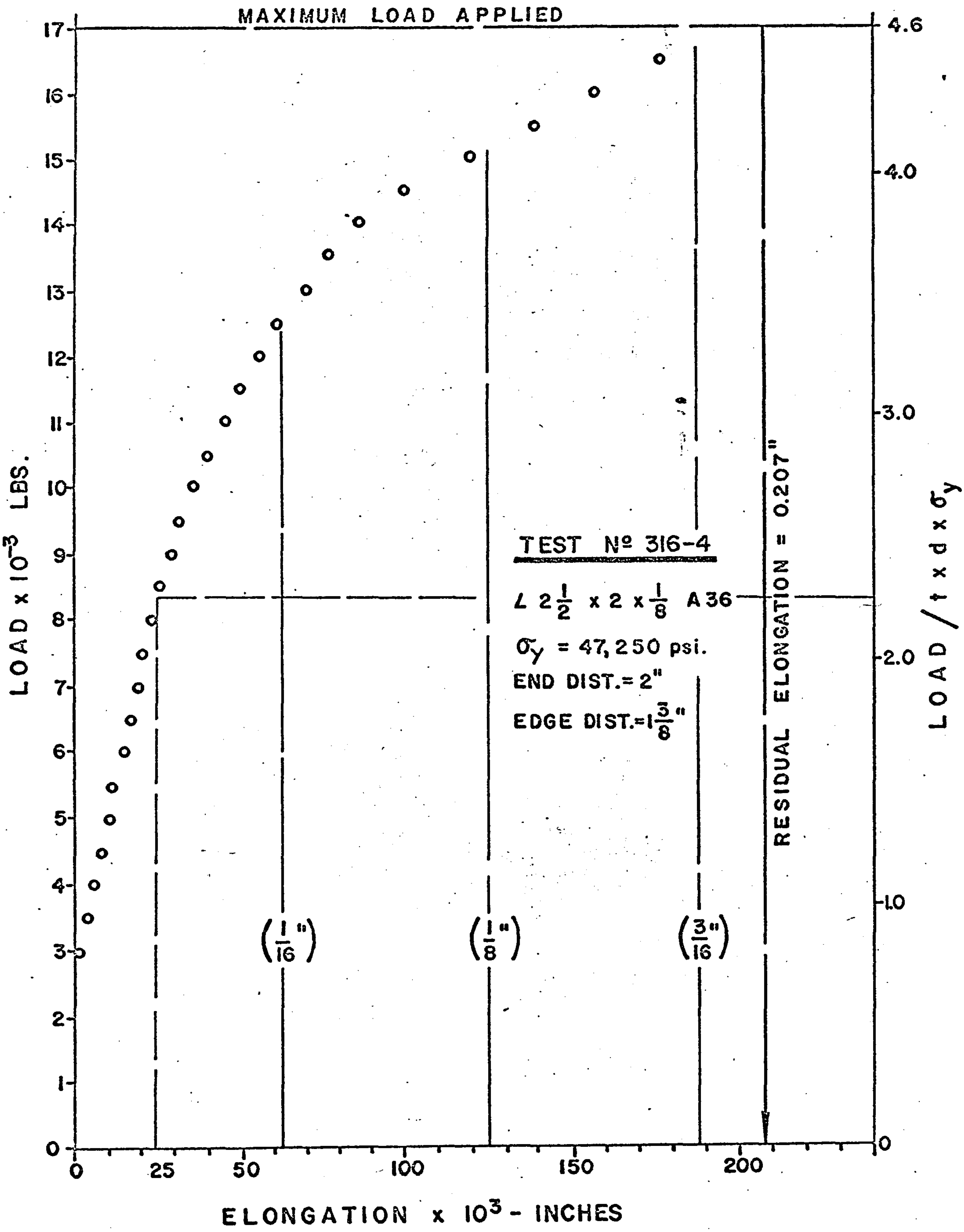

MAXIMUM LOAD APPLIED
TEST Nº 316-4
∠ 2 1/2 x 2 x 1/8 A36
σy = 47,250 psi.
END DIST.= 2"
EDGE DIST.=1 3/8"
RESIDUAL ELONGATION = 0.207"
(1/16")
(1/8")
(3/16")
LOAD x 10^-3 LBS.
LOAD / t x d x σy
ELONGATION x 10^3 - INCHES
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
25
50
100
150
200
1.0
2.0
3.0
4.0
4.6

FIGURE 3.7

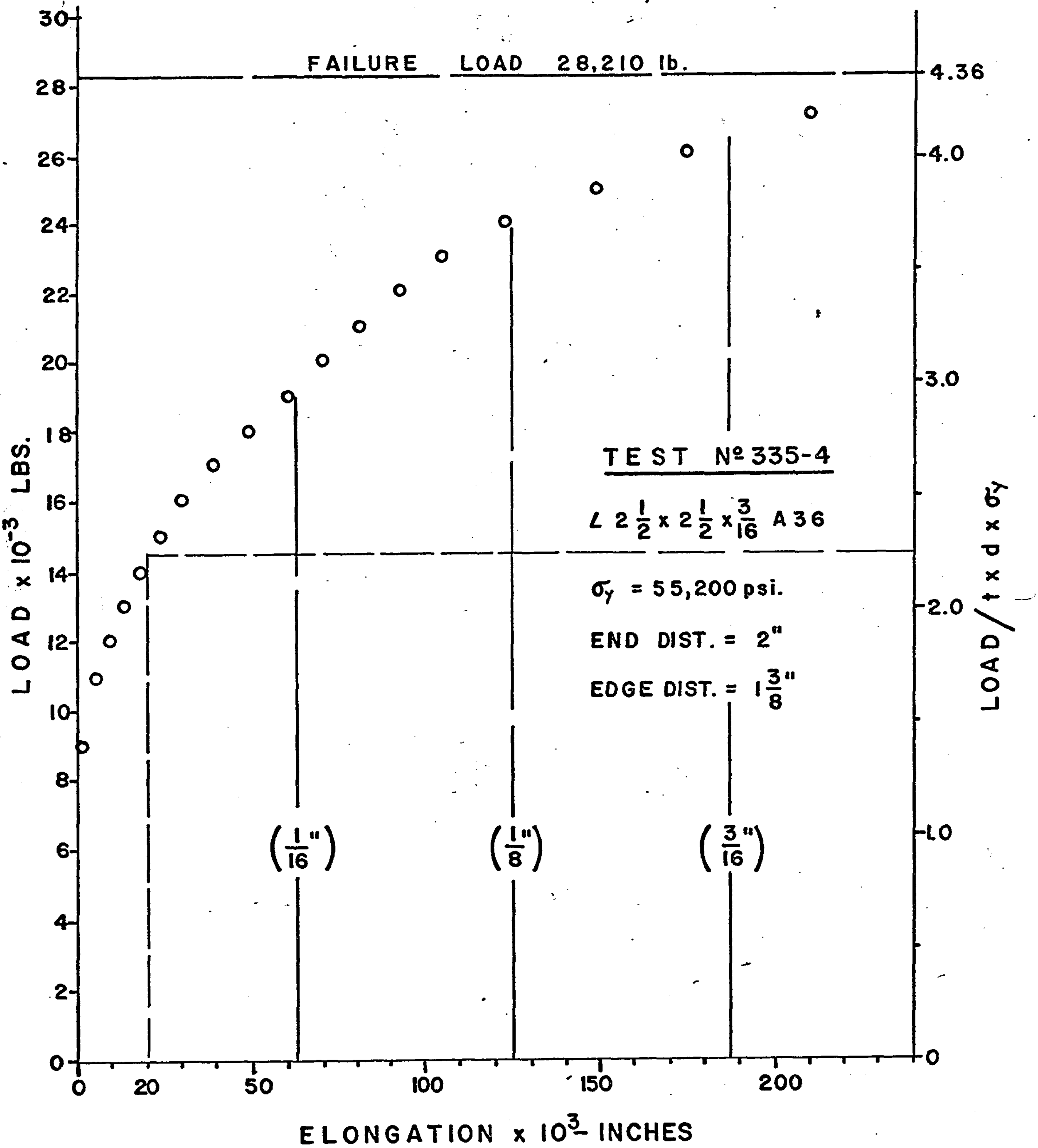

FAILURE LOAD 28,210 lb.
TEST Nº 335-4
∠ 2 1/2 x 2 1/2 x 3/16 A36
σY = 55,200 psi.
END DIST. = 2"
EDGE DIST. = 1 3/8"
(1/16")
(1/8")
(3/16")
LOAD x 10-3 LBS.
LOAD / t x d x σY
ELONGATION x 10-3 INCHES
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
0
1.0
2.0
3.0
4.0
4.36
0
20
50
100
150
200

FIGURE 3.8

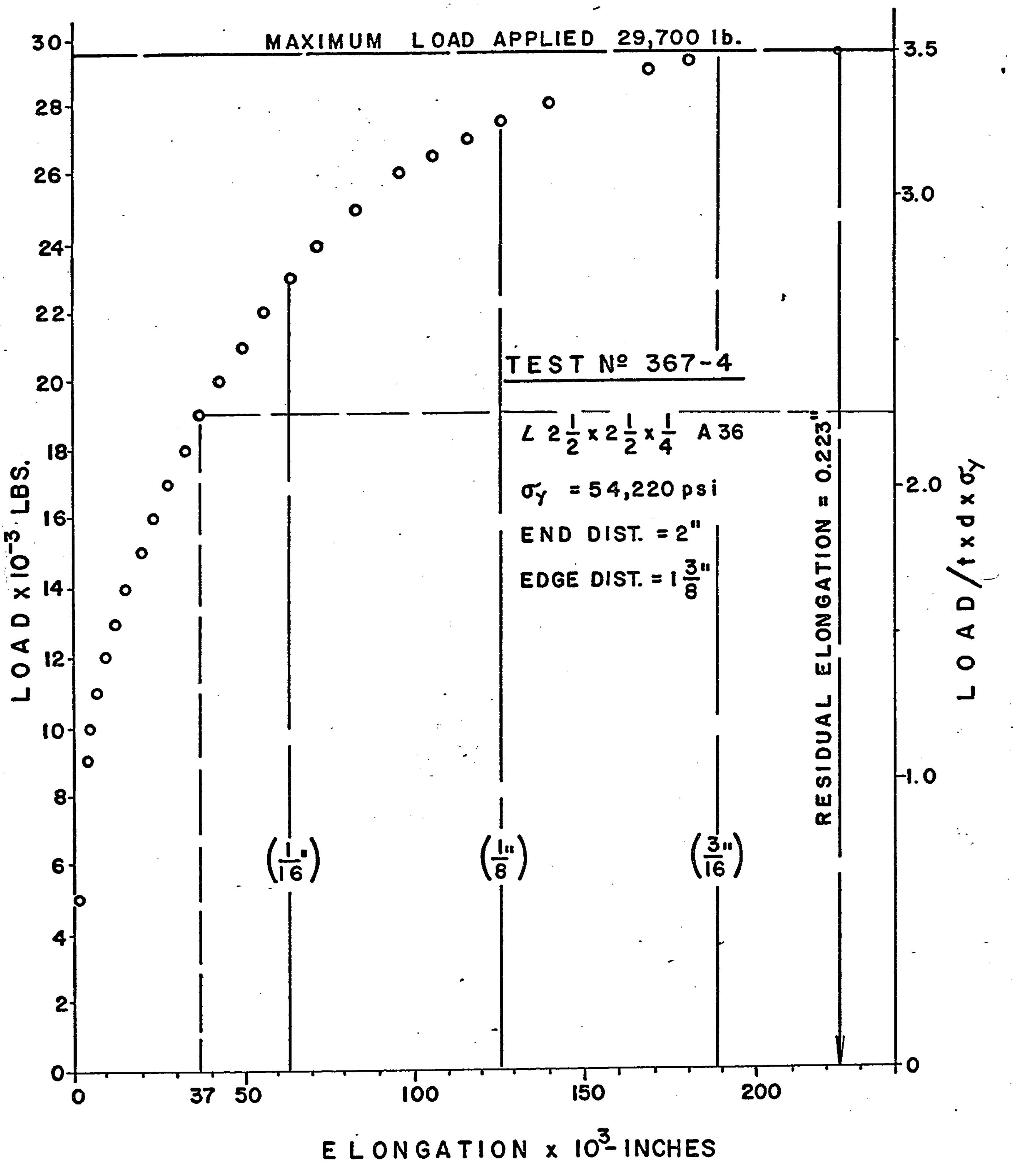
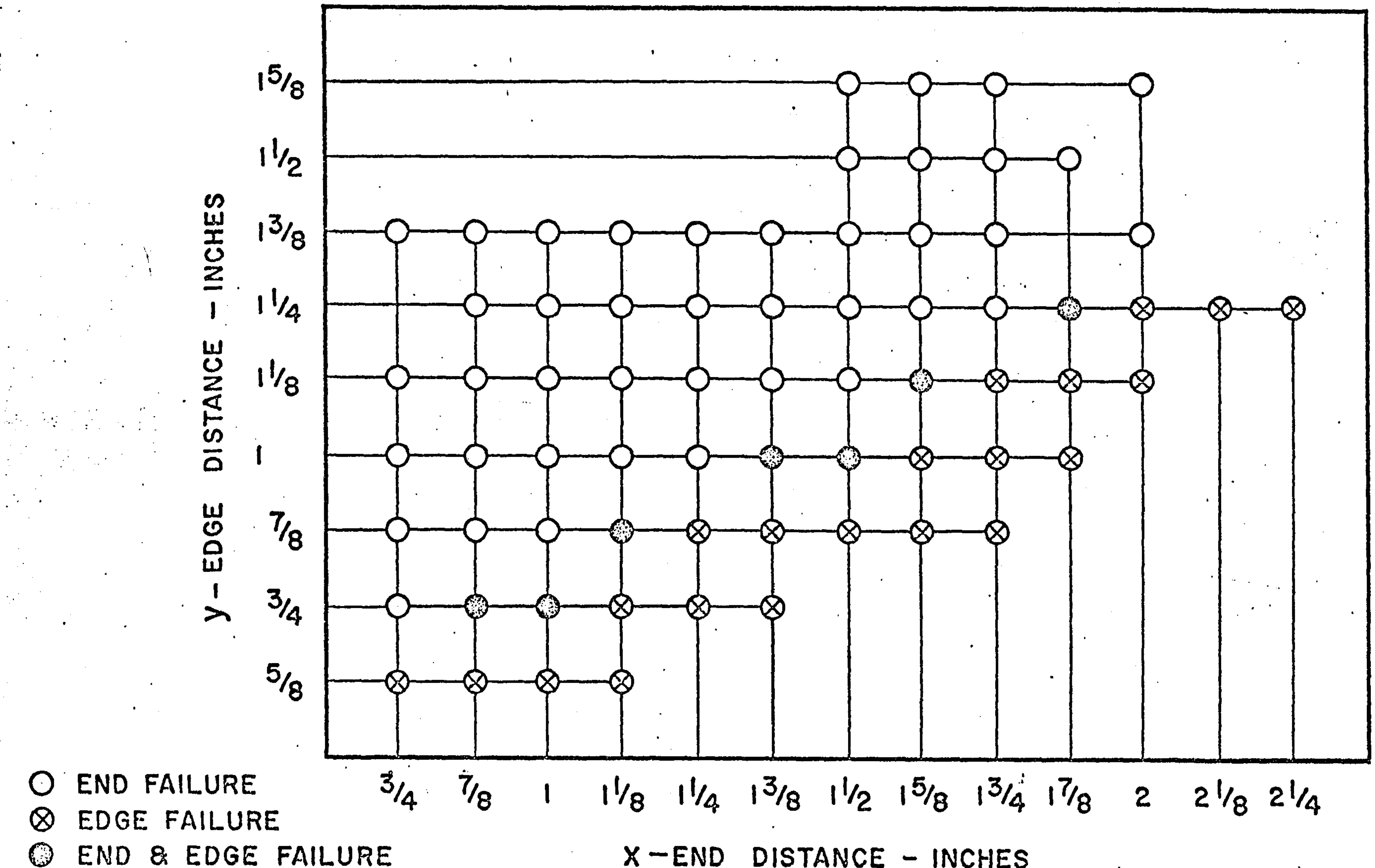

MAXIMUM LOAD APPLIED 29,700 lb.
TEST Nº 367-4
∠ 2 1/2 x 2 1/2 x 1/4 A 36
σy = 54,220 psi
END DIST. = 2"
EDGE DIST. = 1 3/8"
RESIDUAL ELONGATION = 0.223"
(1/16")
(1/8")
(3/16")
LOAD x 10^-3 LBS.
LOAD/t x d x σy
ELONGATION x 10^-3 INCHES
0
37
50
100
150
200
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
1.0
2.0
3.0
3.5

FIGURE 3.9



DISTRIBUTION OF END & EDGE TYPE FAILURES

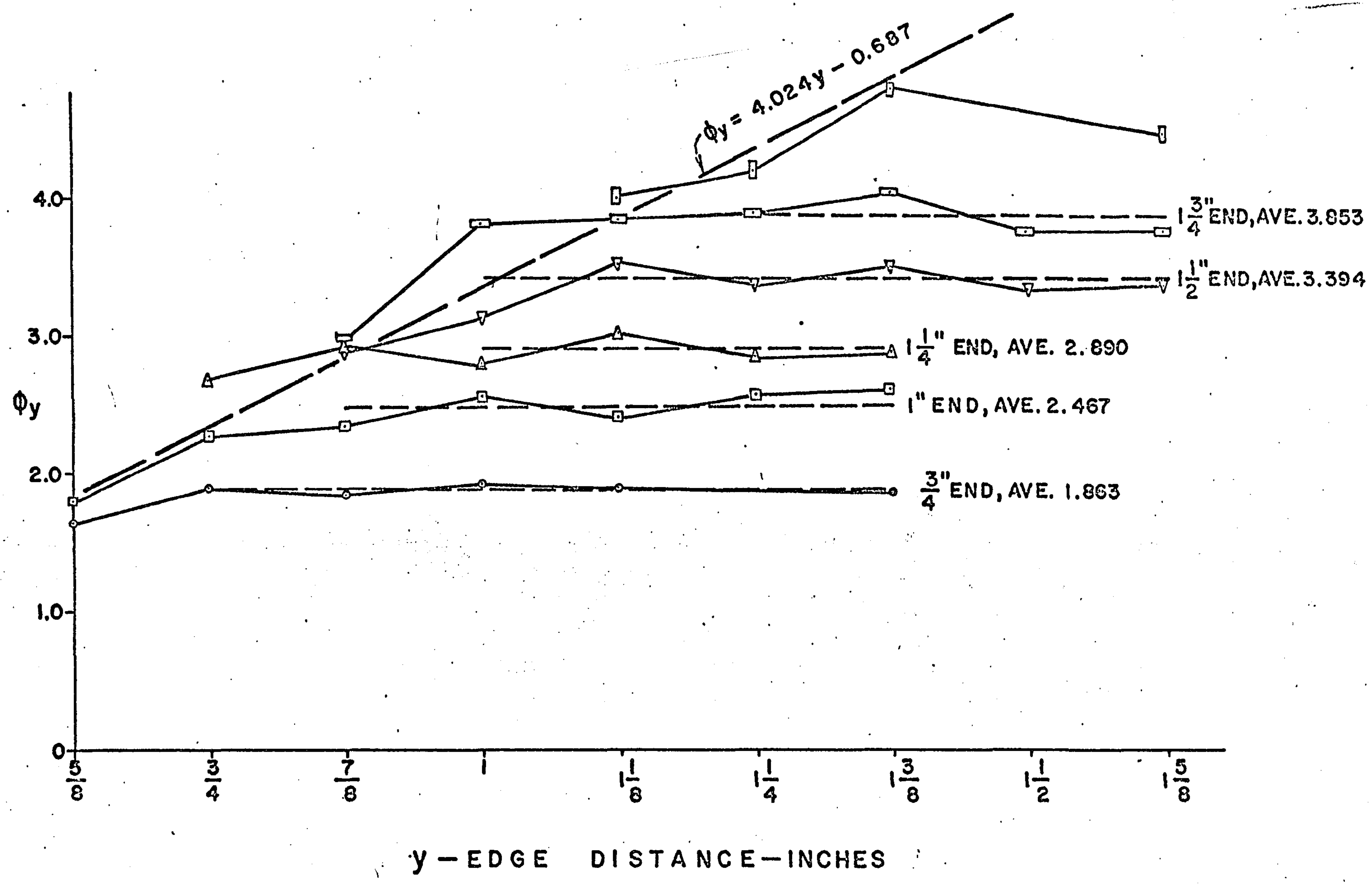
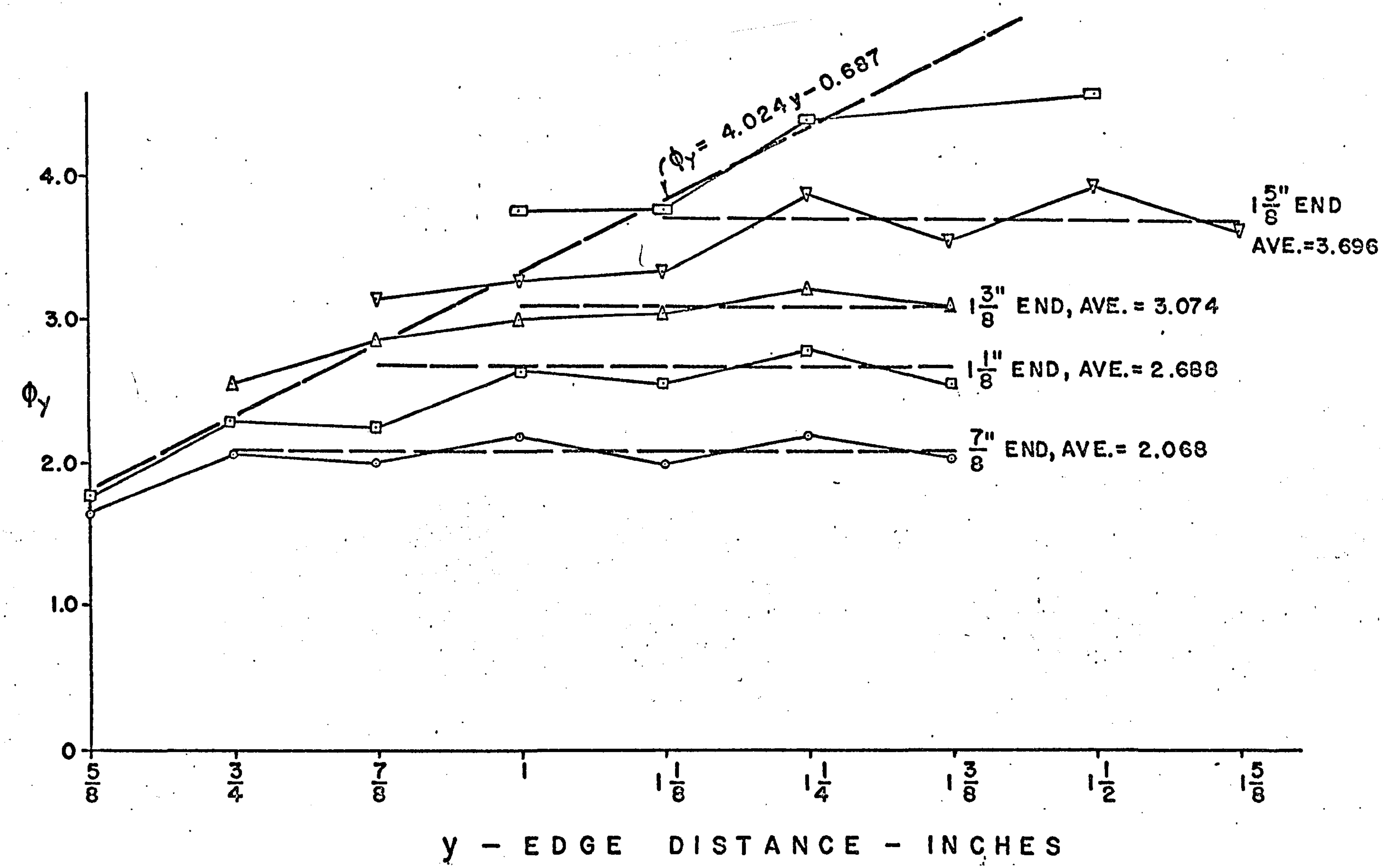
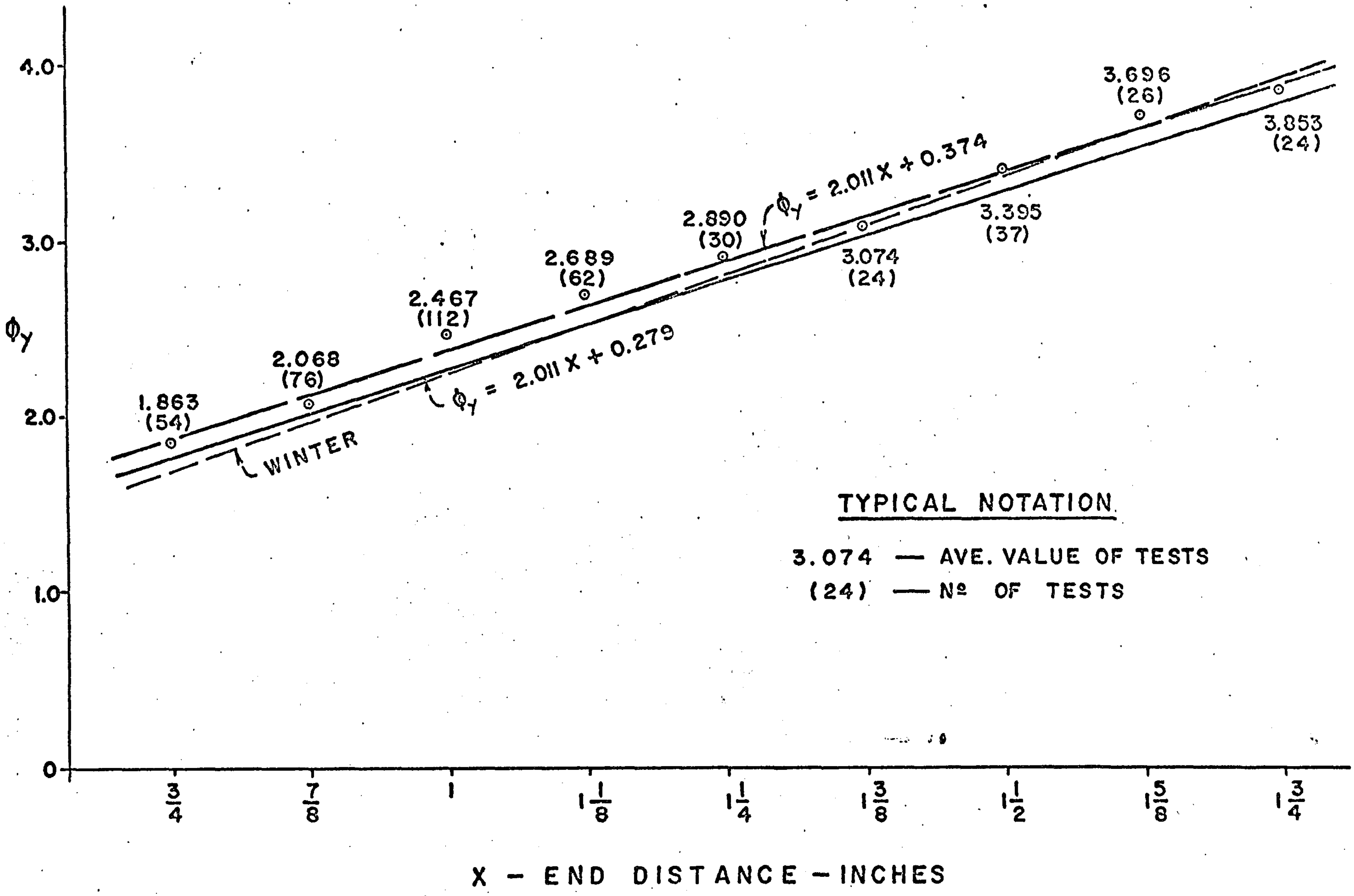

$\phi_y = 4.024y - 0.687$
$1\frac{3}{4}$" END, AVE. 3.853
$1\frac{1}{2}$" END, AVE. 3.394
$1\frac{1}{4}$" END, AVE. 2.890
1" END, AVE. 2.467
$\frac{3}{4}$" END, AVE. 1.863
ϕ_y
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$
$\frac{3}{4}$
$\frac{7}{8}$
1
$1\frac{1}{8}$
$1\frac{1}{4}$
$1\frac{3}{8}$
$1\frac{1}{2}$
$1\frac{5}{8}$
y — EDGE DISTANCE — INCHES

FIGURE 4.1



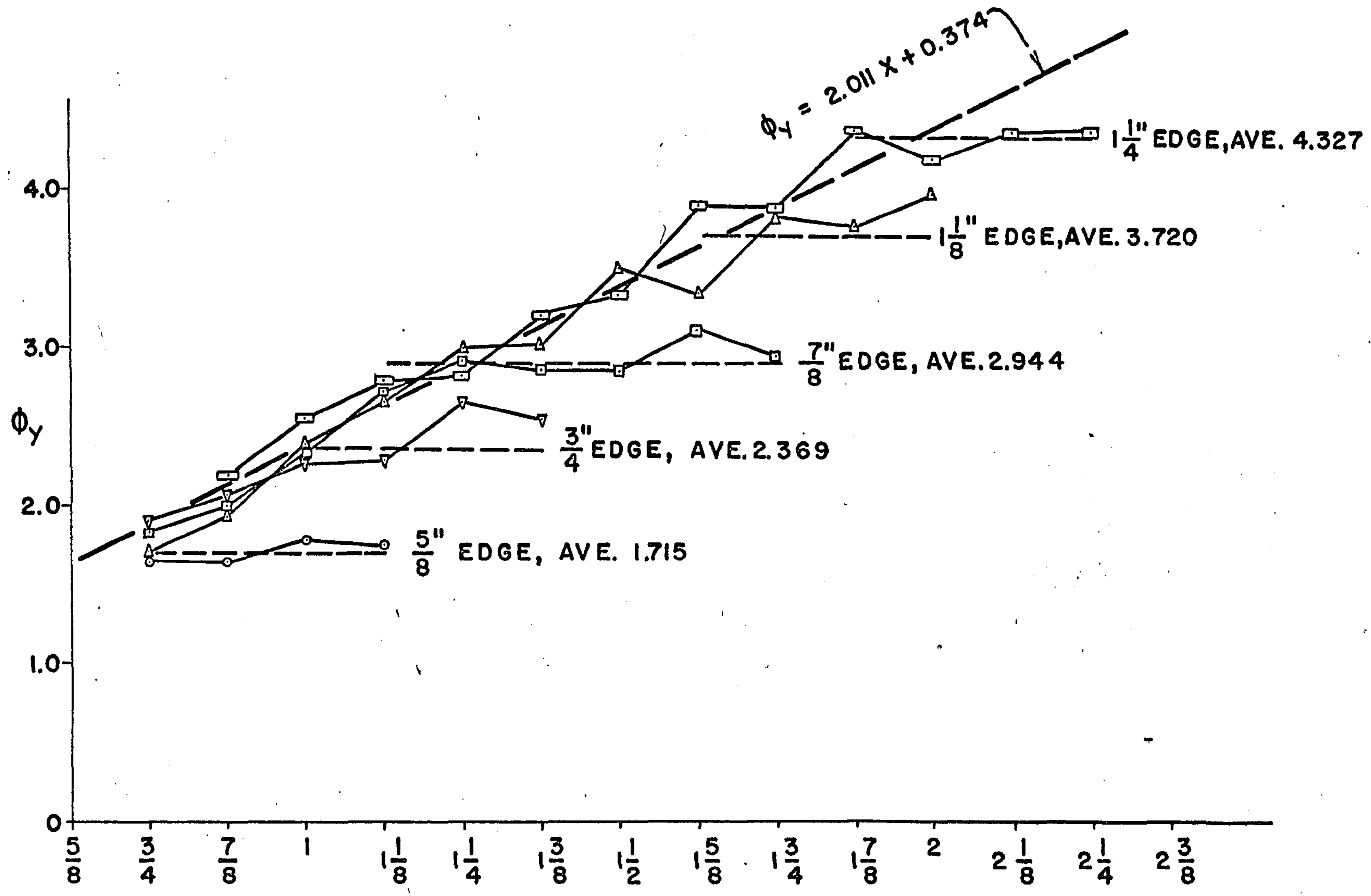
$\phi_y = 4.024y - 0.687$
$1\frac{5}{8}$" END
AVE.=3.696
$1\frac{3}{8}$" END, AVE. = 3.074
$1\frac{1}{8}$" END, AVE.= 2.688
$\frac{7}{8}$" END, AVE.= 2.068
ϕ_y
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$
$\frac{3}{4}$
$\frac{7}{8}$
1
$1\frac{1}{8}$
$1\frac{1}{4}$
$1\frac{3}{8}$
$1\frac{1}{2}$
$1\frac{5}{8}$
y - EDGE DISTANCE - INCHES

Figure 4.2



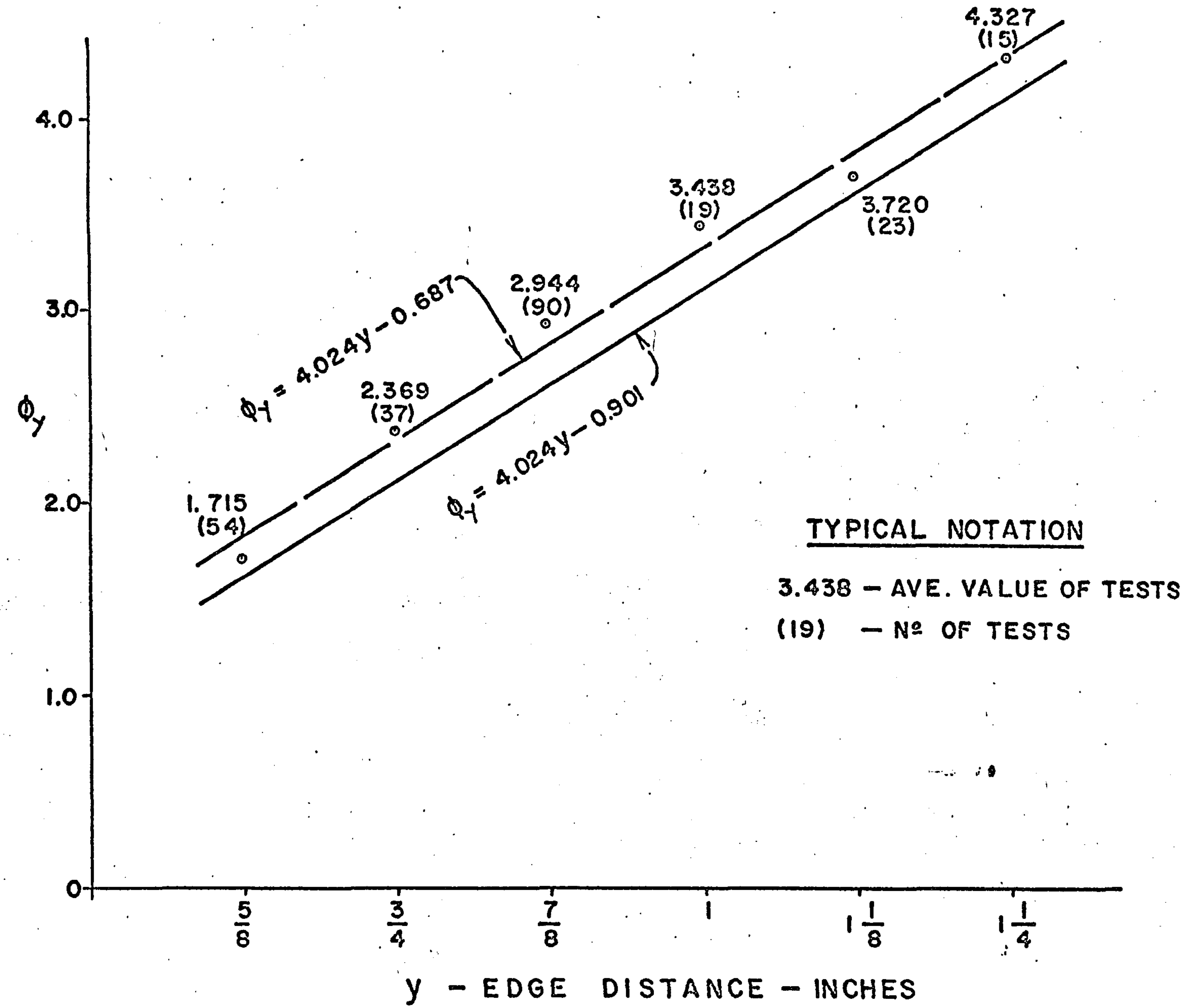
RELATIONSHIP BETWEEN ϕ_y AND END DISTANCE FOR END FAILURE

FIGURE 4.3



$\phi_y = 2.011X + 0.374$
$1\frac{1}{4}$" EDGE, AVE. 4.327
$1\frac{1}{8}$" EDGE, AVE. 3.720
$\frac{7}{8}$" EDGE, AVE. 2.944
$\frac{3}{4}$" EDGE, AVE. 2.369
$\frac{5}{8}$" EDGE, AVE. 1.715
ϕ_y
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$ $\frac{3}{4}$ $\frac{7}{8}$ 1 $1\frac{1}{8}$ $1\frac{1}{4}$ $1\frac{3}{8}$ $1\frac{1}{2}$ $1\frac{5}{8}$ $1\frac{3}{4}$ $1\frac{7}{8}$ 2 $2\frac{1}{8}$ $2\frac{1}{4}$ $2\frac{3}{8}$
X – END DISTANCE – INCHES

Figure 4.4



RELATIONSHIP BETWEEN ϕ_y AND EDGE DISTANCE FOR EDGE FAILURE

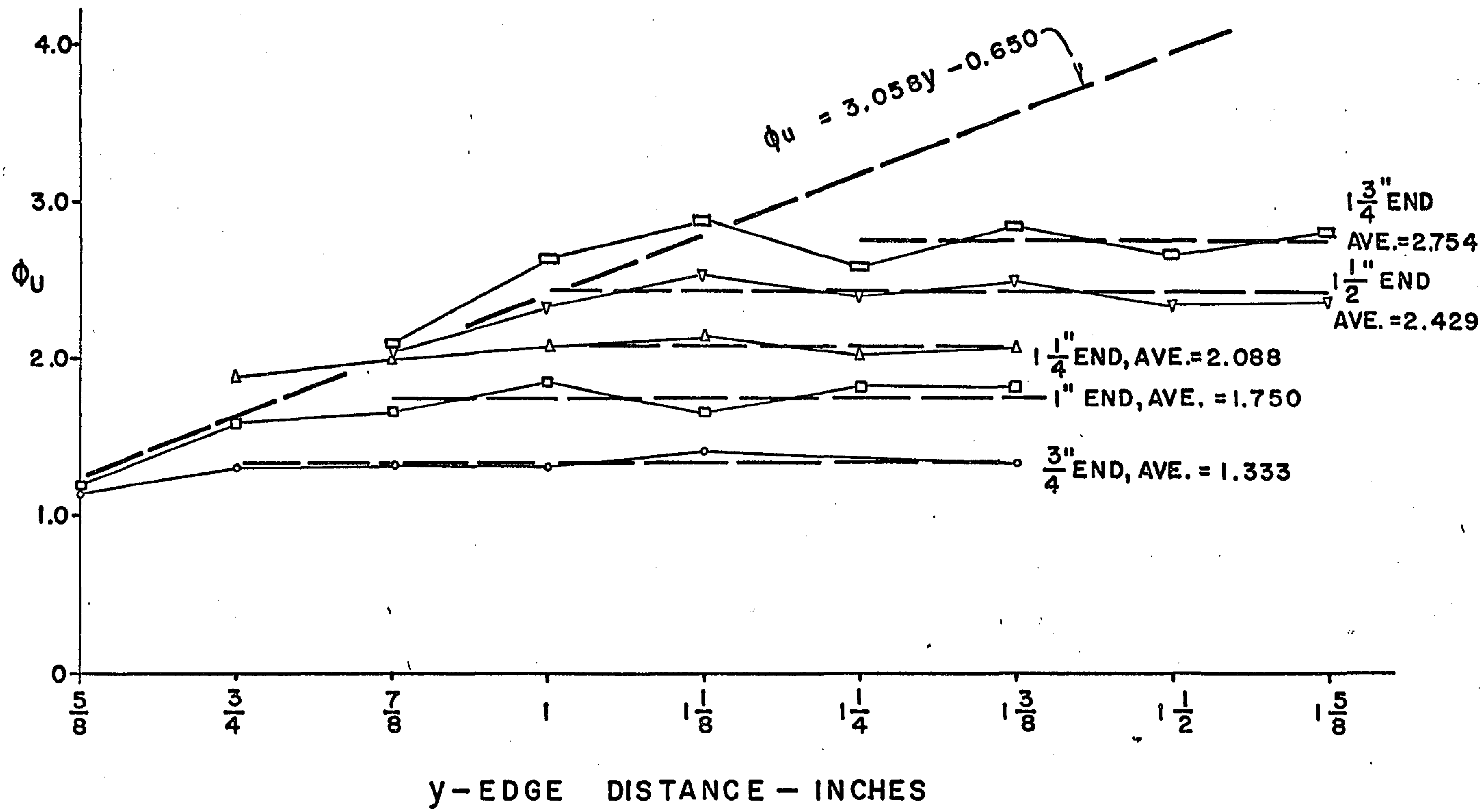
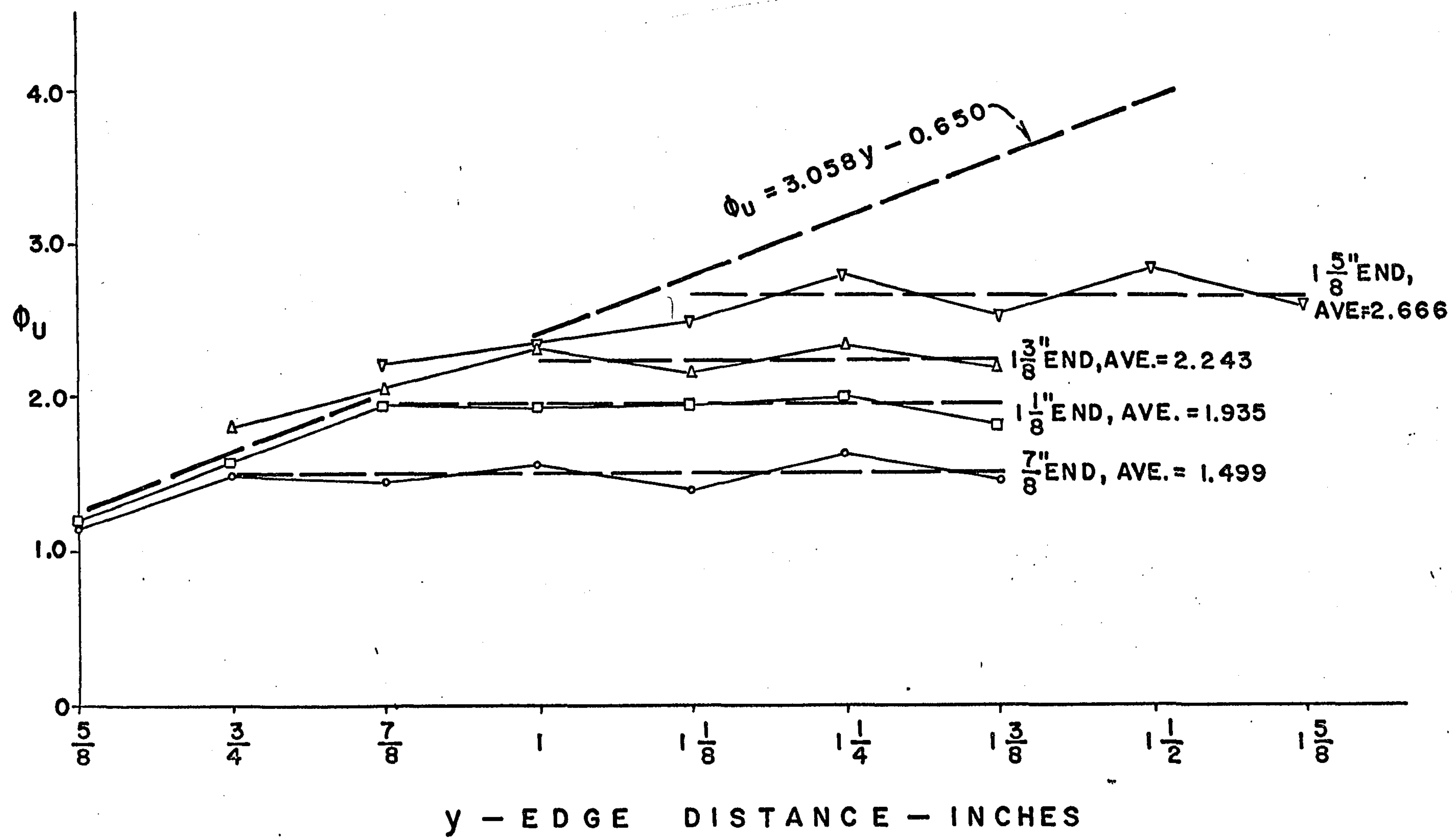

$\phi_u = 3.058y - 0.650$
$1\frac{3}{4}$" END
AVE.=2.754
$1\frac{1}{2}$" END
AVE.=2.429
$1\frac{1}{4}$" END, AVE.=2.088
1" END, AVE.=1.750
$\frac{3}{4}$" END, AVE.=1.333
ϕ_U
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$
$\frac{3}{4}$
$\frac{7}{8}$
1
$1\frac{1}{8}$
$1\frac{1}{4}$
$1\frac{3}{8}$
$1\frac{1}{2}$
$1\frac{5}{8}$
y-EDGE DISTANCE — INCHES

FIGURE 4.6



$\phi_u = 3.058y - 0.650$
$1\frac{5}{8}$" END, AVE.=2.666
$1\frac{3}{8}$" END, AVE.= 2.243
$1\frac{1}{8}$" END, AVE. = 1.935
$\frac{7}{8}$" END, AVE. = 1.499
ϕ_u
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$
$\frac{3}{4}$
$\frac{7}{8}$
1
$1\frac{1}{8}$
$1\frac{1}{4}$
$1\frac{3}{8}$
$1\frac{1}{2}$
$1\frac{5}{8}$
y — EDGE DISTANCE — INCHES

FIGURE 4.7

RELATIONSHIP BETWEEN ϕ_u AND END DISTANCE

IN THE RANGE OF END FAILURES

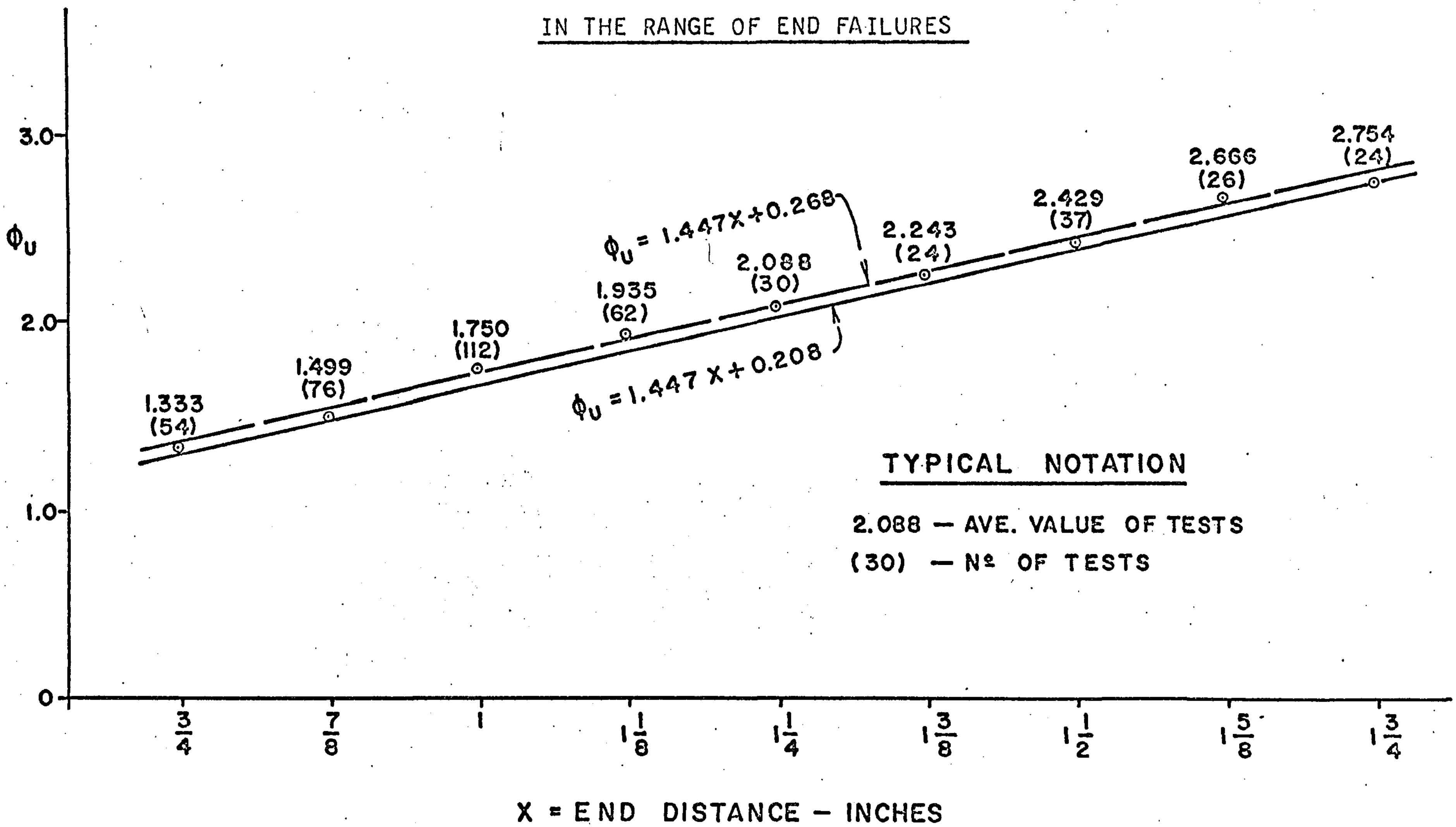
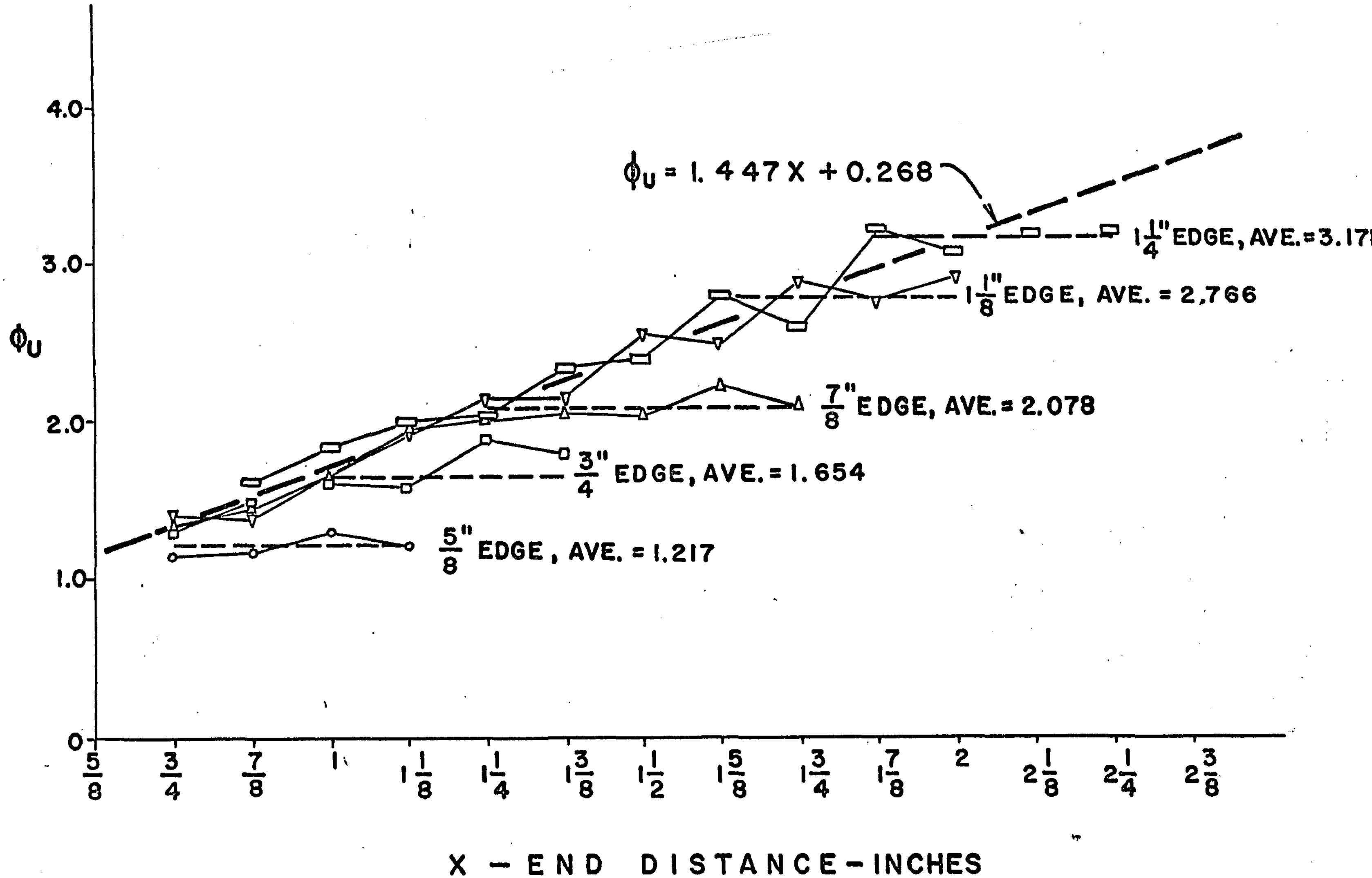


Figure 4.8



$\phi_U = 1.447X + 0.268$
$1\frac{1}{4}$" EDGE, AVE. = 3.171
$1\frac{1}{8}$" EDGE, AVE. = 2,766
$\frac{7}{8}$" EDGE, AVE. = 2.078
$\frac{3}{4}$" EDGE, AVE. = 1.654
$\frac{5}{8}$" EDGE, AVE. = 1.217
ϕ_U
4.0
3.0
2.0
1.0
0
$\frac{5}{8}$ $\frac{3}{4}$ $\frac{7}{8}$ 1 $1\frac{1}{8}$ $1\frac{1}{4}$ $1\frac{3}{8}$ $1\frac{1}{2}$ $1\frac{5}{8}$ $1\frac{3}{4}$ $1\frac{7}{8}$ 2 $2\frac{1}{8}$ $2\frac{1}{4}$ $2\frac{3}{8}$
X - END DISTANCE-INCHES

FIGURE 4.9

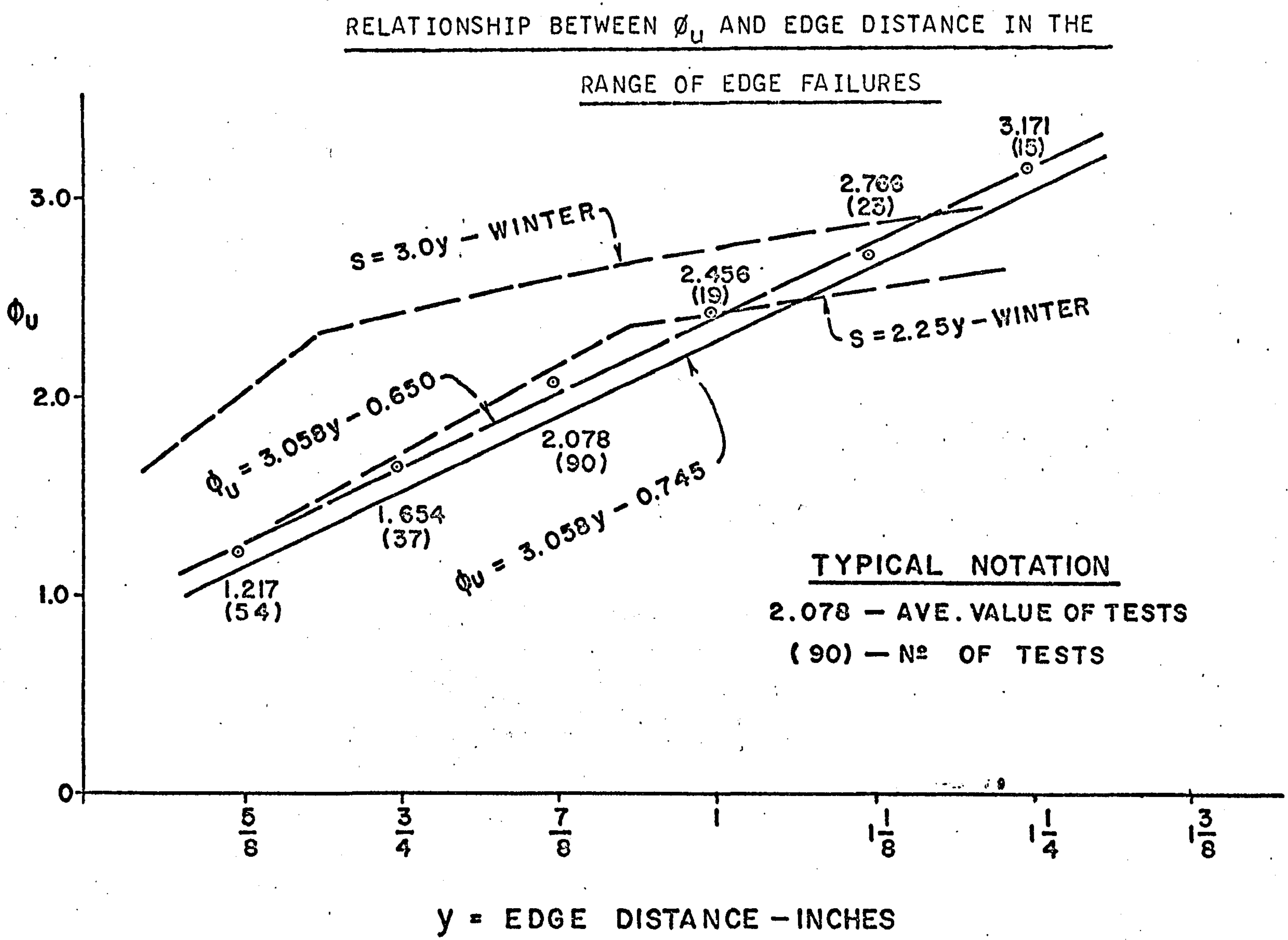
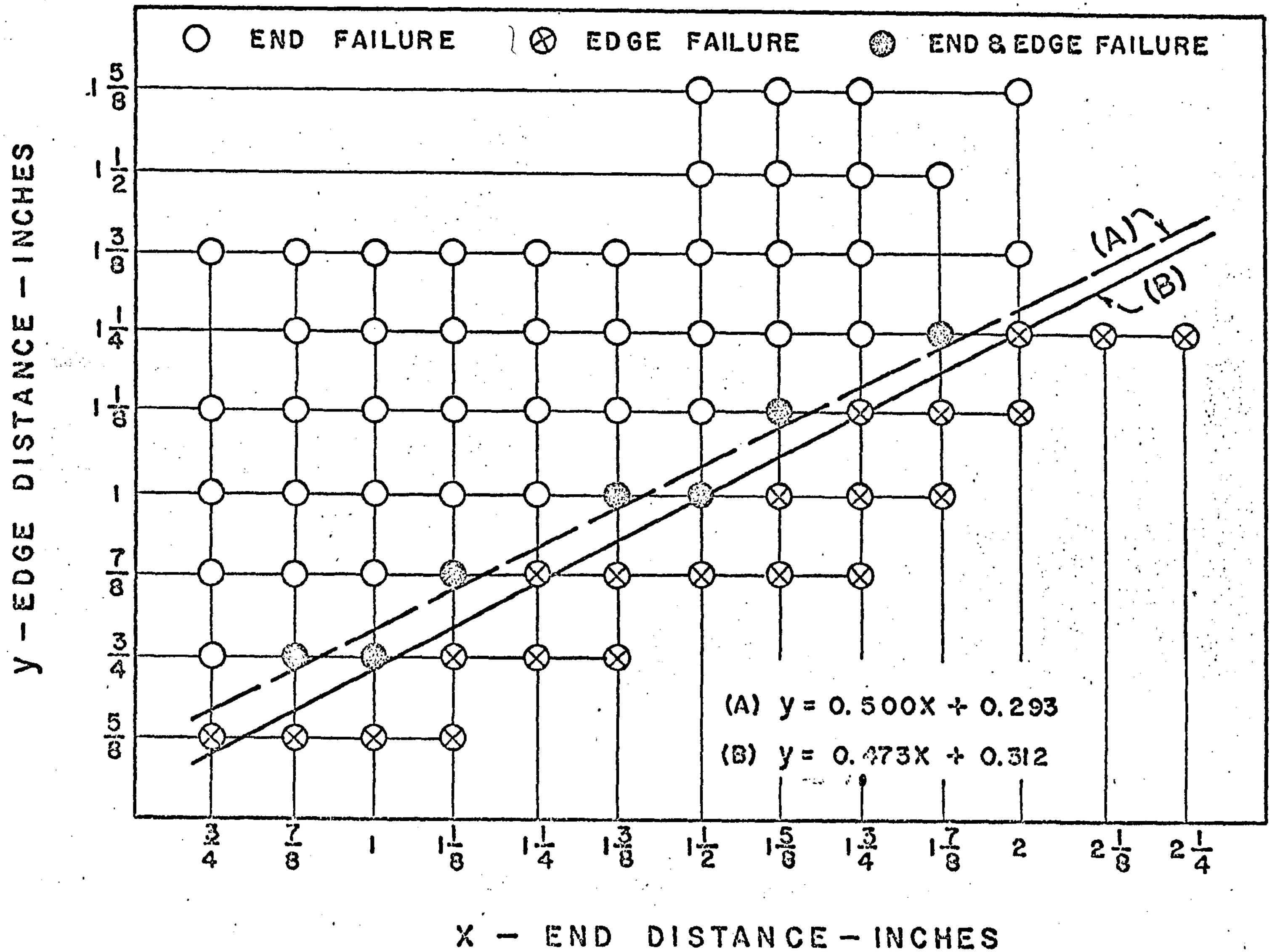

RELATIONSHIP BETWEEN Ø_u AND EDGE DISTANCE IN THE
RANGE OF EDGE FAILURES
ϕu
3.0
2.0
1.0
0
S = 3.0y – WINTER
S = 2.25y – WINTER
ϕu = 3.058y – 0.650
ϕu = 3.058y – 0.745
3.171
(15)
2.768
(23)
2.456
(19)
2.078
(90)
1.654
(37)
1.217
(54)
TYPICAL NOTATION
2.078 – AVE. VALUE OF TESTS
(90) — Nº OF TESTS
5/8
3/4
7/8
1
1 1/8
1 1/4
1 3/8
y = EDGE DISTANCE – INCHES

FIGURE 4.10



DISTRIBUTION OF END AND EDGE TYPE FAILURES

(Indicates End Failure

(Indicates Edge (Failure

(Indicates end and (Edge Failures

FIGURE 4.11

TABLES

SPECIFICATION	YIELD STRENGTH MINIMUM - P.S.I.	TENSILE STRENGTH - P.S.I.	REFERENCE	THICKNESS LIMITS-INCHES
C.S.A. - G40.4	33,000	60,000 - 72,000	1	
A.S.T.M. - A36	36,000	60,000 - 80,000	2	$t \leq 4$
C.S.A. - G40.6	45,000	80,000 - 95,000	3	
C.S.A. - G40.8	40,000	65,000 - 85,000	4	$t \leq 5/8$
C.S.A. - G40.8	38,000	65,000 - 85,000	4	$5/8 < t \leq 1$
C.S.A. - G40.8	36,000	65,000 - 85,000	4	$1 < t \leq 1\text{-}1/2$
C.S.A. - G40.12	44,000	62,000 Min.	5	$t \leq 3/4$
C.S.A. - G40.12	40,000	62,000 Min.	5	$3/4 < t$
RB-50	50,000	70,000 Min.	6	$t \leq 3/4$
RB-55	55,000	70,000 Min.	6	$t \leq 3/4$
RB-60	60,000	75,000 Min.	6	$t \leq 3/4$
T-50	50,000	75,000 Min.	7	$t \leq 1\text{-}1/8$
T-55	55,000	80,000 Min.	7	$t \leq 1\text{-}1/8$
T-60	60,000	80,000 Min.	7	$t \leq 1\text{-}1/8$

PROPERTIES OF STEELS USED IN TOWER STRUCTURES

TABLE 1.1

STANDARD TOWER BOLTS

UNIT TENSILE STRENGTH	- 60,000 P.S.I.	(1)
UNIT SHEAR STRENGTH THROUGH THREADS	- 45,000 P.S.I.	(1)
UNIT SHEAR STRENGTH THROUGH SHANK	- 34,000 - 37,200 P.S.I.	

HIGH STRENGTH BOLTS

UNIT TENSILE STRENGTH	- 120,000 P.S.I.	(2)

NOTE:

(1) As Per A.S.T.M. Specification A394, Reference 10

(2) As Per A.S.T.M. Specification A325, Reference 11

PROPERTIES OF BOLTS USED IN TOWER STRUCTURES

TABLE 1.2

TEST RESULTS FOR TENSION COUPONS

Section	Mat'l.	Test No.	Dimensions Width	Thick- Ness	Yield Load	Stress	Ult. Tensile Load	Stress
L1½x1½x1/8	G40.4	21	0.502	0.140	3,100	44,100	4,280	60,900
L1-3/4x1-3/4x1/8	G40.4	30	0.501	0.139	3,840	55,200	4,710	67,650
L2 x 2 x 1/8	G40.4	39	0.500	0.144	3,580	49,740	4,800	66,650
L2½x2½x 1/8	G40.4	51	0.503	0.136	3,280	47,960	4,400	64,350
L1½ x 1½ x 3/16	G40.4	26	0.496	0.206	4,770	46,700	6,480	63,400
L1-3/4x1-3/4x3/16	G40.4	35	0.505	0.194	4,480	45,750	5,975	61,050
L2 x 2 x 3/16	G40.4	45	0.505	0.184	4,630	49,820	6,310	67,950
L2½x2½ x 3/16	G40.4	55	0.506	0.193	5,100	52,200	6,730	68,900
L2½x2½ x 1/4	G40.4	59	0.510	0.259	5,975	45,250	8,550	64,750
L1½x1½ x 1/8	G40.6	23	0.504	0.141	4,050	57,000	5,910	83,200
L1-3/4x1-3/4x1/8	G40.6	32	0.496	0.148	4,280	58,300	6,110	83,300
L 2 x 2 x 1/8	G40.6	42	0.505	0.138	4,450	63,900	6,070	87,150
L2½x2½ x 1/8	G40.6	53	0.491	0.130	4,160	65,200	5,690	89,100
L1½x1½ x 3/16	G40.6	28	0.495	0.205	6,360	62,720	8,700	85,750
L1-3/4x1-3/4x3/16	G40.6	37	0.506	0.206	6,150	59,000	8,750	83,900
L 2 x 2 x 3/16	G40.6	47	0.509	0.194	6,350	64,350	8,600	87,150

TABLE 3.1

TEST RESULTS FOR TENSION COUPONS

Section	Mat'l.	Test No.	Dimensions Width	Dimensions Thick-Ness	Yield Load	Yield Stress	Ult. Tensile Load	Ult. Tensile Stress
L1½x1½ x 1/8	G40.4	105 to 107	0.492	0.121	3,120	52,450	4,280	71,900
L1½x1½ x 3/16	G40.4	108	0.491	0.194	4,740	49,800	6,580	69,130
L1-3/4x1-3/4x1/8	G40.4	111 to 119	0.501	0.131	3,590	54,720	4,860	74,050
L1-3/4x1-3/4 x 3/16		120 to 122	0.499	0.186	4,120	44,400	5,750	61,970
L2 x 2 x 1/8	G40.4	123 to 133	0.496	0.142	3,500	49,720	4,950	70,300
L2 x 2 x 3/16	G40.4	134 to 140	0.503	0.177	4,730	53,100	6,200	69,700
L2½ x 2½ x 1/8	G40.4	141 142	0.500	0.139	3,690	53,100	4,790	68,950
L2½ x 2½ x 3/16	G40.4	143 to145	0.497	0.200	5,020	50,500	6,840	68,850
L2½ x 2½ x 1/4	G40.4	146 to 148	0.512	0.266	7,150	52,500	9,270	68,100
L1½x1½ x 1/8	G40.6	195 to 197	0.494	0.122	3,850	63,900	5,540	91,950
L1-3/4x1-3/4 x 1/8	G40.6	198 to 206	0.496	0.132	4,180	63,900	5,990	91,500
L1-3/4x1-3/4x1/8	G40.6	"	0.489	0.134	4,240	64,750	6,200	94,700
L2 x 2 x 1/8	G40.6	207 to 217	0.494	0.135	4,160	62,400	5,750	86,250
L2 x 2 x 3/16	G40.6	218 to 224	0.500	0.190	6,100	64,250	8,625	90,800
L2½x2½x1/8	G40.6	225 226	0.499	0.131	4,480	68,550	6,100	93,300
L2½x2½x3/16	G40.6	227 to 229	0.507	0.184	6,150	66,000	8,440	90,500
L2½x2½x3/16	G40.6	"	0.4935	0.188	6,280	67,650	8,280	89,250
L3 x 2½ x 1/4	G40.6	230 to 232	0.511	0.251	7,820	61,000	11,220	87,500
L1-3/4x1-3/4x1/8	G40.6		AVERAGE VALUES			64,325		93,100
L2½x2½x3/16	G40.6		AVERAGE VALUES			66,825		89,875

TABLE 3.2

TEST RESULTS FOR TENSION COUPONS

Section	Mat'l.	Test Series	Dimensions Width	Thick-Ness	Yield Load	Stress	Ult.Tensile Load	Stress
L1-3/4x1-3/4x1/8	A36	300	0.502	0.134	3,140	46,700	4,850	72,100
L2 x 2 x 1/8	A36	300	0.512	0.128	3,520	53,750	5,040	76,900
L2½x2½x 1/8	A36	300	0.513	0.128	3,100	47,250	4,490	68,400
L1½x1½ x 3/16	A36	300	0.508	0.189	4,420	46,080	6,530	68,100
L1-3/4x1-3/4x3/16	A36	300	0.497	0.183	4,350	47,850	6,150	67,670
L2 x 2 x 3/16	A36	300	0.493	0.184	4,330	47,700	5,990	66,100
L2½x2½x 3/16	A36	300	0.503	0.200	5,550	55,200	7,350	73,100
L2 x 2 x 1/4	A36	300	0.508	0.250	6,130	48,280	8,540	67,250
L2½x2½ x 1/4	A36	300	0.510	0.242	6,690	54,220	8,950	72,520
L 3 x 3 x 1/4	A36	300	0.504	0.262	6,270	47,500	9,130	69,150

TABLE 3.3

TEST RESULTS FOR TENSION COUPONS

Section	Mat'l.	Test Series	Dimensions Width	Thick-Ness	Yield Load	Yield Stress	Ult. Tensile Load	Ult. Tensile Stress
L 2 x 2 x 1/8	RB60	400	0.508	0.135	4,400	64,200	6,440	94,000
L 2 x 2 x 1/8	RB60	400	0.490	0.139	4,175	61,300	6,800	99,800
L2½x2½ x 1/8	G40.8	400	0.511	0.139	3,350	47,170	4,670	65,750
L1-3/4x1-3/4x3/16	G40.8	400	0.491	0.194	4,700	49,350	6,740	70,800
L 2 x 2 x 3/16	RB60	400	0.498	0.193	6,710	69,800	8,560	89,050
L2½x2½x 3/16	G40.6	400	0.490	0.200	5,260	53,650	8,350	85,200
L2½x2½ x 3/16	G40.6	400	0.5006	0.204	6,175	60,500	8,970	87,850
L 3 x 2 x 1/4	G40.12	400	0.489	0.268	6,500	49,620	9,175	70,070
L2½x2½ x 1/4	G40.8	400	0.508	0.274	6,570	47,200	9,380	67,400
L2½x2½ x 1/4	G40.8	400	0.5075	0.263	6,350	47,600	8,990	67,400
L 3 x 3 x 1/4	RB60	400	0.4933	0.2584	9,170	72,050	11,180	87,900
L2½ x 2½ x 3/16	G40.6	400	0.498	0.201	6,160	61,530	9,050	90,450
L 2½ x 2½ x 1/4	G40.8	400	0.510	0.268	6,650	48,670	9,220	67,450
L 2½ x 2½ x 3/16	G40.6	400	0.4983	0.202	6,025	59,900	9,000	89,420
L 2½ x 2½ x 3/16	G40.6	400	Average Values		–	58,895	–	88,230
L 2½ x 2½ x 1/4	G40.8	400	Average Values		–	47,400	–	67,400

TABLE 3.4

TEST RESULTS FOR TENSION COUPONS

SECTION	MAT'L.	TEST SERIES	DIMENSIONS WIDTH	DIMENSIONS THICKNESS	YIELD LOAD	YIELD STRESS	ULT. TENSILE LOAD	ULT. TENSILE STRESS
L1-3/4x1-3/4 x1/8	A36	500	0.495	0.134	2,995	45,160	4,520	68,180
L2½x2½ x 1/8	A36	500	0.503	0.134	3,450	51,200	4,600	68,300
L 2 x 2 x 3/16	A36	500	0.5015	0.183	4,470	48,710	6,495	70,800
L2½x2½ x 3/16	A36	500	0.503	0.183	3,970	43,110	5,955	64,750
L 2 x 2 x 1/4	A36	500	0.4996	0.245	6,075	49,670	8,510	69,550
L2½x2½ x 1/4	A36	500	0.4932	0.2588	6,580	51,500	9,520	74,500
L2 x 2 x 1/8	RB60	600	0.5038	0.136	4,870	71,050	5,970	87,350
L2½x2½ x 1/8	G40.8	600	0.499	0.1333	2,965	44,550	4,195	63,000
L2½x 2 x 3/16	RB60	600	0.4973	0.200	6,120	61,500	8,140	81,740
L2½x2½ x 3/16	G40.6	600	0.5021	0.200	5,490	54,750	8,380	83,650
L3 x 2 x 1/4	G40.12	600	0.499	0.257	6,780	53,000	9,650	75,450
L3x2½ x 1/4	G40.12	600	0.5021	0.2650	7,020	52,750	10,110	76,000

TABLE 3.5

TEST RESULTS FOR TENSION COUPONS

Section	Mat'l.	Test No.	Dimensions Width	Thick-Ness	Yield Load	Stress	Ult. Load	Tensile Stress
L2 x 2 x 1/8	RB60	494	0.490	0.139	4,175	61,300	6,800	99,800
L 3 x 3 x 1/4	RB60	475	0.493	0.258	9,170	72,050	11,180	87,900
L2½x2½ x 1/4	A36	522	0.493	0.259	6,580	51,500	9,520	74,500
L3 x 2½ x 1/4	G40.12	622	0.502	0.265	7,020	52,750	10,110	76,000
L2 x 2 x 1/8	RB60	701	0.491	0.133	3,200	49,000	4,180	64,000
L 2 x 2 x 1/8	RB60	702	0.489	0.134	3,090	47,200	4,230	64,600
L 2 x 2 x 1/8	RB60	703	0.492	0.134	3,220	48,850	4,310	65,400
L2 x 2 x 1/8	RB60	704	0.491	0.133	3,220	49,350	4,380	67,100
L 2 x 2 x 1/8	RB60*	Average Values			–	48,600	–	65,275

* This material is obviously not RB60 Steel

Section	Mat'l.	Test No.	Width	Thickness	Yield Load	Yield Stress	Ult. Load	Tensile Stress
L1-3/4x1-3/4x1/8	A36	800-801	0.500	0.125	3,240	51,840	4,580	73,280
L2 x 2 x 1/8	A36	802-803	0.495	0.130	3,260	50,660	4,660	72,416
L2½x2½ x 1/8	A36	804-808	0.490	0.130	3,400	53,375	4,520	70,958
L1-3/4x1-3/4x3/16	A36	810-811	0.492	0.186	4,800	52,452	6,860	74,963
L 2 x 2 x 3/16	A36	812-813	0.498	0.183	4,320	47,403	6,240	68,471
L2½x2½ x 3/16	A36	814-818	0.499	0.190	4,860	51,260	6,820	71,933

TABLE 3.6

AVERAGE VALUES OF ϕ_y

EDGE DISTANCE - INCHES													
$1\frac{5}{8}$	–	–	–	–	–	–	3.336	3.612	3.729	–	4.426	–	–
$1\frac{1}{2}$	–	–	–	–	–	–	3.286	3.922	3.734	4.558	–	–	–
$1\frac{3}{8}$	1.837	2.030	2.601	2.552	2.858	3.077	3.475	3.544	4.010	–	4.784	–	–
$1\frac{1}{4}$	–	2.199	2.558	2.790	2.830	3.221	3.343	3.872	3.870	4.382	4.184	4.372	4.382
$1\frac{1}{8}$	1.872	1.948	2.387	2.658	2.994	3.021	3.510	3.352	3.843	3.769	3.980	–	–
1	1.912	2.182	2.545	2.648	2.775	2.978	3.113	3.262	3.801	3.746	–	–	–
$\frac{7}{8}$	1.836	1.994	2.339	2.723	2.915	2.859	2.865	3.124	2.960	–	–	–	–
$\frac{3}{4}$	1.894	2.046	2.271	2.281	2.676	2.538	–	–	–	–	–	–	–
$\frac{5}{8}$	1.644	1.646	1.800	1.764	–	–	–	–	–	–	–	–	–
	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	1-5/8	1-3/4	1-7/8	2	2-1/8	2-1/4

END DISTANCE - INCHES

TABLE 3.7

AVERAGE VALUES OF ϕ_u

EDGE DISTANCE - INCHES	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	1-5/8	1-3/4	1-7/8	2	2-1/8	2-1/4
1-5/8	-	-	-	-	-	-	2.371	2.594	2.805	-	-	-	-
1-1/2	-	-	-	-	-	-	2.329	2.830	2.656	3.296	-	-	-
1-3/8	1.328	1.456	1.815	1.814	2.067	2.187	2.489	2.522	2.842	-	3.438	-	-
1-1/4	-	1.619	1.832	1.991	2.018	2.328	2.380	2.798	2.583	3.218	3.062	3.203	3.214
1-1/8	1.410	1.393	1.660	1.942	2.137	2.150	2.535	2.485	2.890	2.758	2.915	-	-
1	1.320	1.538	1.848	1.912	2.082	2.306	2.339	2.325	2.646	2.592	-	-	-
7/8	1.324	1.055	1.660	1.951	1.998	2.046	2.030	2.217	2.099	-	-	-	-
3/4	1.299	1.494	1.598	1.576	1.882	1.781	-	-	-	-	-	-	-
5/8	1.159	1.173	1.290	1.212	-	-	-	-	-	-	-	-	-

END DISTANCE - INCHES

TABLE 3.8

TO DERIVE RELATIONSHIP BETWEEN ϕ_Y AND END DISTANCE FOR END FAILURES.

FOR DATA SEE FIGURES 4.1 AND 4.2

END DIST. x	ϕ_Y	$X = x - \bar{x}$	$\Phi = \phi_Y - \bar{\phi}_Y$	$X\Phi$	X^2
0.750	1.863	-0.500	-1.025	+0.51250	+0.250
0.875	2.068	-0.375	-0.820	+0.30750	+0.140625
1.000	2.467	-0.250	-0.421	+0.10525	+0.062500
1.125	2.688	-0.125	-0.200	+0.02500	+0.015625
1.250	2.890	0	+0.002	0	0
1.375	3.073	+0.125	+0.185	+0.023125	+0.015625
1.500	3.394	+0.250	+0.506	+0.12650	+0.062500
1.625	3.696	+0.375	+0.808	+0.30300	+0.140625
1.750	3.853	+0.500	+0.965	+0.48250	+0.250000
Σ = 11.250	25.992	0	0	+1.885375	0.937500

$$\bar{x} = \frac{11.250}{9} = 1.250 \qquad \bar{\phi}_Y = \frac{25.992}{9} = 2.888$$

FOR $\phi_Y = bx + a$

$$b = \frac{\Sigma X\Phi}{\Sigma X^2} = \frac{1.885375}{0.93750} = 2.011,067$$

$$\phi_Y - \bar{\phi}_Y = b(x - \bar{x})$$

$$\phi_Y - 2.888 = 2.011067(x - 1.250)$$

$$\phi_Y = 2.888 - 2.011067 \times 1.25 + 2.011067x$$

$$= 2.888 - 2.513833 + 2.011067x$$

$$\underline{\phi_Y = 2.011x + 0.374} \qquad \text{(EQ'N 4.1)}$$

TO CALCULATE STANDARD DEVIATION OF ϕ_Y AS GIVEN BY EQUATION 4.1

END x	$\hat{\phi}_Y = 0.374 + 2.01107\ x$	$\mathcal{E} = \phi_Y - \hat{\phi}_Y$	$\mathcal{E}^2$
0.750	0.374 + 1.508 300 = 1.882 300	−0.019 300	
0.875	0.374 + 1.759 683 = 2.133 683	−0.065 683	
1.000	0.374 + 2.011 067 = 2.385 067	+0.081 933	
1.125	0.374 + 2.262 450 = 2.636 450	+0.051 550	
1.250	0.374 + 2.513 833 = 2.887 833	+0.002 167	
1.375	0.374 + 2.765 217 = 3.139 217	−0.066 217	
1.500	0.374 + 3.016 600 = 3.390 600	+0.003 400	
1.625	0.374 + 3.267 983 = 3.641 983	+0.054 017	
1.750	0.374 + 3.519 367 = 3.893 367	−0.040 367	
		+0.001 500	0.023,005,443,434

$$S_{\phi_Y} = \sqrt{\frac{\Sigma \mathcal{E}^2}{n-2}} = \sqrt{\frac{0.023\ 005\ 443\ 434}{9-2}}$$

$$= \sqrt{0.003\ 286\ 491\ 91} = 0.057,327,933$$

TO CHECK CORRELATION OF EQUATION 4.1

CORRELATION COEFFICIENT $= r = \dfrac{\Sigma X \phi}{\sqrt{\Sigma X^2 \Sigma \phi^2}}$

$\Sigma X \phi = +1.885\ 375$

$\Sigma X^2 = 0.937\ 500$ $\qquad \Sigma \phi^2 = 3.814\ 620$

$$r = \frac{1.885\ 375}{\sqrt{0.937500 \times 3.814620}} = 0.9970$$

TABLE A-11 OF REFERENCE (12) GIVES $r = 0.798$ FOR $\nu = n-2 = 7$ AT 1% LEVEL OF SIGNIFICANCE

∴ CORRELATION OF EQ^N 4.1 IS EXCELLENT.

TO COMPARE VARIANCES OF VALUES USED TO DERIVE EQUATION 4.1

USE BARTLETT'S TEST TO TEST THE HYPOTHESIS THAT ALL THE VARIANCES ARE HOMOGENEOUS. SEE REFERENCE (12), PAGE 160.

END x	n	$\sum'(\phi_Y - \bar{\phi}_Y)^2$	ν	$s^2 = \sum(\phi_Y - \bar{\phi}_Y)^2/\nu$	LOG s^2	$(n-1)$ LOG s^2
0.75	4	0.003,195,687,5	3	0.001,065,229,2	$\bar{3}.0274312$	$\bar{9}+0.0822936$
0.875	6	0.051,947,500,0	5	0.010,389,500,0	$\bar{2}.0165737$	$\bar{10}+0.0828685$
1.000	5	0.053,300,000,0	4	0.013,325,000,0	$\bar{2}.12466$	$\bar{8}+0.498640$
1.125	5	0.031,672,800,0	4	0.007,918,200,0	$\bar{3}.89863$	$\bar{12}+3.594520$
1.250	4	0.026,012,750,0	3	0.008,670,920,0	$\bar{3}.938025$	$\bar{9}+2.814075$
1.375	4	0.033,642,750,0	3	0.011,214,250,0	$\bar{2}.0497606$	$\bar{6}+0.1492818$
1.500	5	0.037,566,000,0	4	0.009,391,500,0	$\bar{3}.972735$	$\bar{12}+3.890940$
1.625	5	0.236,867,200,0	4	0.059,216,800,0	$\bar{2}.7724476$	$\bar{8}+3.0897904$
1.750	4	0.053,284,750,0	3	0.017,761,580,0	$\bar{2}.2494795$	$\bar{6}+0.7484385$
—	—	0.52748943	33	—	—	$\bar{80}+14.9508478$

$$\bar{s}^2 = \frac{\sum\sum(\phi - \bar{\phi})^2}{\sum(n-1)} = \frac{0.52748943}{33} = 0.0159845$$

$$\text{LOG } \bar{s}^2 = \bar{2}.2037015 = -1.7962985$$

$$\chi^2 = 2.3026\left\{\text{LOG } \bar{s}^2 \times \sum(n-1) - \sum'\left[(n-1)\text{ LOG } s^2\right]\right\}$$

$$= 2.3026\left[-1.7962985 \times 33 + 80 - 14.9508478\right]$$

$$= 2.3026 \times 5.7713 = \underline{13.289}$$

FOR $k = 9$, $\nu = 9 - 1 = 8$, TABLE A8 OF REF. (12) GIVES

$\chi^2 = 13.362$ AT 10% LEVEL OF SIGNIFICANCE

THUS THE NULL HYPOTHESIS IS ACCEPTED AND IT IS CONCLUDED THAT THE VARIANCES ARE HOMOGENEOUS.

TO PREDICT CONFIDENCE INTERVAL OF VALUES OF ϕ_Y AS GIVEN BY EQUATION 4.1

FROM REFERENCE (12), PAGE 181

$$S_{\phi_i}^2 = S_\phi^2 \left[1 + \frac{1}{n} + \frac{(x_i - \bar{x})^2}{\sum (x - \bar{x})^2} \right]$$

$$S_{\phi_i} = 0.057\,327\,933 \sqrt{1 + \frac{1}{9} + \frac{(x_i - \bar{x})^2}{0.937\,500}}$$

MINUM VALUE OF x = 0.750

$$S_{\phi_{0.75}} = 0.057\,327\,933 \sqrt{1 + 0.11111 + \frac{(0.75 - 1.25)^2}{0.937\,500}}$$

$$= 0.057\,327\,933 \sqrt{1.37778}$$

$$= 0.057\,327\,933 \times 1.173\,787\,777$$

$$S_{\phi_{0.75}} = 0.067\,291 = S_{\phi_{1.75}}$$

FOR 10% LEVEL OF SIGNIFICANCE IN A ONE-SIDED TEST SEE TABLE 3 OF "STATISTICAL MANUAL" BY E.L. CROW, F.A. DAVIS AND M.W. MAXFIELD

$t = 1.415$ FOR $\nu = 9 - 2 = 7$

THUS $\phi_{Y_{0.75}} = 1.8823 - 1.415 \times 0.067\,291$

$$= 1.8823 - 0.0952$$

NOTE THAT $S_{\phi_{0.75}} = S_{\phi_{1.75}}$

$\therefore \phi_{Y_{1.75}} = 3.8934 - 0.0952$

MEAN VALUE OF $x = 1.25$

$$S_{\phi_{1.25}} = 0.057\,327\,933 \sqrt{1 + 0.11111}$$

$$= 0.057\,327\,933 \times 1.054\,092\,555$$

$$= 0.060\,428\,947$$

FOR 10 % LEVEL OF SIGNIFICANCE

$$\phi_{Y\,1.25} = 2.773\,32 - 1.415 \times 0.060\,428\,947$$

$$= 2.773\,32 - 0.085\,506\,96$$

IF IN PLACE OF THIS VALUE OF $\phi_{Y\,1.25}$ WE USE

$$\phi_{Y\,1.25} = 2.773\,32 - 0.0952$$

THE VALUE OF t THUS INCLUDED IS

$$t = \frac{0.0952}{0.060\,428\,947} = 1.5754$$

REFERENCE TO "STATISTICAL MANUAL" TABLE 3 INDICATES THAT THIS VALUE OF t, AT $\nu = 7$, CORRESPONDS WITH A LEVEL OF SIGNIFICANCE OF APPROXIMATELY 8.3 %

THUS IF $t\,S_{\phi_i} = 0.0952$ IS USED OVER THE RANGE OF x FROM 0.750" TO 1.750" THE LEVEL OF SIGNIFICANCE WILL RANGE FROM 10% MAXIMUM TO 8.3 % MINIMUM.

$\therefore$ USE $\phi_Y = 2.011x + 0.374 - 0.095$

$$\underline{\phi_Y = 2.011x + 0.279} \qquad \text{(EQ'N 4.5)}$$

VITA AUCTORIS

1925	George Rennie Sinclair born October 15, Windsor, Ontario, Canada.
1938	Graduated from King George Public School, Windsor, Ontario, Canada.
1942	Graduated in Architectural Drafting from Windsor-Walkerville Vocational School, Windsor, Ontario, Canada.
1942	April, entered employment in the Drafting Office of Canadian Bridge Company, Walkerville, Ontario, Canada.
1944	February 10, enlisted in the Royal Canadian Air Force. December 7, honourably discharged from the Royal Canadian Air Force as excess aircrew.
1945	January 8, enlisted in the Canadian Army.
1946	June 6, honourably discharged from The Canadian Army, on demobilization.
1947	Graduated from Veteran's Rehabilitation School, Windsor, Ontario, Canada with Senior Matriculation.
1947	September, entered the Faculty of Applied Science, Queen's University, Kingston, Ontario, Canada.
1948	Granted the Science '46 Memorial Scholarship for First Year Science, Queen's University.
1949	Granted the W. P. Wilgar Memorial Scholarship for Second Year Science, Queen's University.
1950	Granted the W. W. Near Scholarship in Civil Engineering, Queen's University.
1951	Granted Bachelor of Science Degree, with honours, in Civil Engineering by Queen's University, Kingston, Ontario, Canada.
	Granted the Bronze Medal for first place standing in Civil Engineering by Queen's University.
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1951 September, entered the Civil Engineering Department of City and Guilds College, Imperial College, University of London, London, England.

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