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

George R. Sinclair
University of Windsor

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

Submitted in partial fulfillment
of the requirements for the
Degree of Master of Applied Science
from the University of Windsor.

By

George R. Sinclair

May, 1968
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Last, but not least, appreciation is extended to my wife who has been left with our family of six children on endless occasions, while I busied myself with tests and reports. It is hoped that she feels her efforts have been worthwhile.
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\[ P \] = Ultimate Test Load, Pounds
\[ t \] = Thickness of Test Angle, Inches
\[ d \] = Diameter of Bolt, 5/8 Inches
\[ \sigma_y \] = Yield Stress, Lbs. Per Square Inch
\[ \sigma_u \] = Ultimate Tensile Stress, Lbs. Per Sq. Inch
\[ \phi_y \] = Failure Function, Based on Yield Stress
\[ \phi_u \] = Failure Function, Based on Ultimate Tensile Stress
\[ x \] = End Distance, Inches
\[ y \] = Edge Distance, Inches
\[ \sigma_n \] = Stress on Net Section At Ultimate Load, Lbs. Per Square Inch
\[ \sigma_b \] = Bearing Stress at Ultimate Load, Lbs. Per Square Inch
\[ P' \] = Intermediate Load on Specimen, Pounds
\[ \phi_y \] = Load Function, Based on Yield Stress
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 torquing 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.
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 2.2 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:

\[
\sigma_y = \frac{P}{t \times d \times \sigma_y}
\]
\[
\sigma_u = \frac{P}{t \times d \times \sigma_u}
\]
Where: 

\[ P = \text{Ultimate Test Load - Pounds} \]
\[ t = \text{Thickness of Test Specimen - Inches} \]
\[ d = \text{Diameter of Bolt - 5/8 inches} \]
\[ \sigma_y = \text{Yield Stress of Material - Pounds Per Square Inch} \]
\[ \sigma_u = \text{Ultimate Tensile Stress of Material - Pounds Per Square Inch} \]

Note that \( \sigma_y \) and \( \sigma_u \) 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 \( \sigma_y \) and \( \sigma_u \) 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 $\phi$.

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 laboratory 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 propagated inward to join one of the cracks propagating 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 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 $\sigma_y$ and $\sigma_u$ which are functions of the yield stress and ultimate tensile stress respectively. The form of $\sigma$ 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, $\sigma_y$ 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 $\sigma_y$ 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 $\sigma_y$ 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

$$\phi_y = 2.011 \times + 0.374$$  \hspace{1cm} (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 \( \phi_y \) 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 \( \phi_y \) with end distance for end failures, while figure 4.5 indicates the variation of \( \phi_y \) with edge distance for edge failures.

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

$$\phi_y = 4.0245 \times - 0.687$$  \hspace{1cm} (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 \( \phi_y \) which is
defined as a function of yield stress. With the substitution of $\sigma_u$ for $\sigma_y$ 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:

$$
\sigma_u = 1.447x + 0.268 \quad (4.3)
$$

$$
\sigma_u = 3.058y - 0.650 \quad (4.4)
$$

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 $\sigma_u$, when taken simultaneously, define two planes in the three dimensional $x, y, \sigma_u$ co-ordinate system. Similarly, Equations 4.1 and 4.2 define two planes in the $x, y, \sigma_y$ 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 \[ r = 0.997 \]
Equation 4.2 \[ r = 0.994 \]
Equation 4.3 \[ r = 0.998 \]
Equation 4.4 \[ r = 0.998 \]

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 $\phi$ 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 $\chi^2$ as calculated are given below along with tabulated values for a 10% level of significance. A sample calculation is contained in Appendix "A".
In addition to the above noted statistical investigations all of the values of $\phi_y$ and $\phi_u$ 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 $\phi$ 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 $\phi$ 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

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

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

Similar calculations give:

$$\phi_y = 4.024 y - 0.901 \quad (4.6)$$
$$\phi_u = 1.447 x + 0.208 \quad (4.7)$$
$$\phi_u = 3.058 y - 0.745 \quad (4.8)$$

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 $\phi_y$ and $\phi_u$ each occur in pairs as functions of $x$ and $y$ thus defining two planes in the $x, y$ and $x, y$ co-ordinate systems respectively. By combining the equations of the preceding section the following equations result.

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

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

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.

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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

$$\frac{P}{\sigma_y t} = 1.40 e$$

Where $P$, $\sigma_y$ and $t$ are as previously defined.

While $e = \text{End Distance}$ as used herein.

If this equation is re-written in terms of the failure function $\phi_y$ the result is

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

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

$$\phi_y = 2.24 x$$

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 $\sigma_y$ 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 $\sigma_y$ 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 $\sigma_y$ value of 4.6 without failure thus approaching Winter's maximums.

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

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

(5.3)

Where $\sigma_n$ = Stress on the Net Section - P.S.I.
$\sigma_u$ = 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

\[ S = 3.0 \ y \]  \ (5.4)  

Substitution of this value of \( S \) into Equation 5.3 gives

\[ \phi_u = 4.8 \ y - 1 \quad \text{for} \ y < 0.694 \ \text{Inches} \]  \ (5.5)  
\[ \phi_u = (4.8y-1) \left(0.10 + \frac{0.625}{y}\right) \quad \text{for} \ y > 0.694 \ \text{Inches} \]  \ (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

\[ \phi_u = 3.6 \ y - 1 \quad \text{for} \ y < 0.926 \ \text{Inches} \]  \ (5.7)  
\[ \phi_u = (3.6 \ y - 1) \left(0.10 + \frac{0.833}{y}\right) \quad \text{for} \ y > 0.926 \ \text{Inches} \]  \ (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 \( \phi_y \) except that the load is not necessarily the ultimate load on the connection hence the use of the prime marks in \( P' \) and \( \phi_y' \). This relationship between \( \sigma_b \) and \( \sigma_y \), in fact, determined the form of \( \phi_y \) as used in this study. Thus in all
cases reported herein the value of $\phi_y$ 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 $\phi_y^r$ 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 $\sigma_y$. 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 $\sigma_y$ 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

$$\sigma_b \text{ Max.} = 4.9 \sigma_y$$  \hspace{1cm} (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 $= 3$

$$\sigma_b \text{ Maximum} = 1.35 \times 3 \sigma_y = 4.05 \sigma_y$$  \hspace{1cm} (6.2)

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

$$\sigma_b \text{ Max.} = 4.5 \sigma_y$$  \hspace{1cm} (6.3)
Reference to Table 3.7 shows that for the average values of \( \varphi_y \) tabulated, a value of \( \sigma_D \) greater than 4.5 \( \sigma_y \) 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 \( \varphi_y \) 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 \( \varphi_y \) 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 \( \varphi_y \) 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 \( \varphi_y \) 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 \( \varphi_y \) 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 $\phi_y$ 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.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Maximum Load $P_l$</th>
<th>$P_l/P$</th>
<th>$\phi_y$</th>
<th>Hole Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>800</td>
<td>11,370</td>
<td>1.112</td>
<td>2.807</td>
<td>0.025</td>
</tr>
<tr>
<td>803</td>
<td>14,500</td>
<td>0.950</td>
<td>3.664</td>
<td>0.024</td>
</tr>
<tr>
<td>808</td>
<td>14,330</td>
<td>0.743</td>
<td>3.440</td>
<td>0.154</td>
</tr>
<tr>
<td>813</td>
<td>17,500</td>
<td>0.866</td>
<td>3.147</td>
<td>0.069</td>
</tr>
</tbody>
</table>

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 $\phi_y$ AND $\phi_u$

As developed herein the failure function $\phi$ 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 $\sigma_y$ and $\sigma_u$ results, as follows, taking end failures as an example.

$$\phi_y = 2.011 x + 0.374 = \frac{P}{d x t x \sigma_y}$$  \hspace{1cm} (4.1)

$$\phi_u = 1.447 x + 0.268 = \frac{P}{d x t x \sigma_u}$$  \hspace{1cm} (4.3)

Equating values of ultimate load $P$ gives

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

or

$$\frac{\sigma_y}{\sigma_u} = \frac{1.447 x + 0.268}{2.011 x + 0.374}$$  \hspace{1cm} (7.1)

Substitution of values of end distance $x$ into this equation gives an average value of the ratio of $\sigma_y$ to $\sigma_u$ of about 0.72.

Similarly, for $\phi_y$ and $\phi_u$ 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 $\sigma_y/\sigma_u$ 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 $\sigma_y/\sigma_u$ as 0.718. This is the mean value of 78 values. Thus the derived equations for the failure functions $\phi_y$ and $\phi_u$ 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 predicted based on the formulae derived herein and using specified physical properties of the steel the values derived from $\phi_y$ will be less than those derived from $\phi_u$ 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 interaction 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

\[ P = \frac{5}{8} \times t \times \sigma_y \left(2.011 \times x^{-0.279}\right) \]

Where \( x \) = End distance in inches
\( t \) = Thickness of angle in inches

3. For edge type failures the ultimate load is reasonably given by

\[ P = \frac{5}{8} \times t \times \sigma_y \left(4.024 \times y^{-0.901}\right) \]

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 \( \frac{5}{8} \) 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.
REFERENCES


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


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
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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

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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
END PORTION OF A SINGLE ANGLE MEMBER

FIGURE 1.1
DETAIL OF TEST SPECIMEN

FIGURE 2.1
TABULATION OF NUMBER OF TEST SPECIMENS

FIGURE 2.2

INDICATES SAMPLES THAT WERE TESTED DURING 1962 & THE LATER TEST SERIES
(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
FIGURE 2.4

DETAIL OF TEST PIECES SHOWING LOCATIONS OF STRAIN GAUGES

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LOAD—STRAIN CURVE FOR TENSION

TEST SPECIMEN #719

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

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

FIGURE 3.1

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
LOAD - LBS.

3600 lb.

3000 -

2000 -

1000 -

STRAIN x10^6 - INCHES/INCH

LOAD-STRAIN CURVE FOR TENSION

TEST SPECIMEN #734

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

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

FIGURE 3.2

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

FIGURE 3.3
YIELD STRESS 47,500 psi

ULTIMATE LOAD 11,100 lb.

STRESS vs CONNECTION LOAD FOR TEST NO. 719-B

FIGURE 3.4
FIGURE 3.5

STRESS vs CONNECTION LOAD FOR TEST NO. 734-C

YIELD STRESS 44,550 psi.
ULTIMATE LOAD 11,250 lb.

STRESS AT GAUGE x 10^3 LBS./IN^2.
Fig. 3.7

Maximum Load Applied

TEST NO. 316-4

L 2 1/2 x 2 x 1/8 A 36

\(\sigma_y = 47,250 \text{ psi.}\)

END DIST. = 2"

EDGE DIST. = 1 3/16"

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FAILURE LOAD 28,210 lb.

TEST No. 335-4

L 2 1/2 x 2 1/2 x 3/16 A 36

σ_y = 55,200 psi.
END DIST. = 2"
EDGE DIST. = 1 3/8"

ELONGATION x 10^3 INCHES

FIGURE 3.8

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MAXIMUM LOAD APPLIED 29,700 lb.

TEST NO. 367-4

\[ L \frac{1}{2} x 2 \frac{1}{2} x \frac{1}{4} \text{ A36} \]

\[ \sigma_y = 54,220 \text{ psi} \]

END DIST. = 2"

EDGE DIST. = 1 \frac{3}{8}"

RESIDUAL ELONGATION = 0.223

\[ \text{FIGURE 3.9} \]

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DISTRIBUTION OF END & EDGE TYPE FAILURES

● END FAILURE
○ EDGE FAILURE

X-EDGE DISTANCE - INCHES

Y-EDGE DISTANCE - INCHES

15/8 11/2 13/8 3/4 7/8 1 11/8 11/4 15/8 13/4 17/8 2 21/8 21/4

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Figure 4.2

Y - EDGE DISTANCE – INCHES
FIGURE 4.3

TYPICAL NOTATION

3.074 — AVE. VALUE OF TESTS  
(24) — NO. OF TESTS

X - END DISTANCE - INCHES

RELATIONSHIP BETWEEN $\phi_Y$ AND END DISTANCE FOR END FAILURE

$2.068 = 2.011x + 0.279$

$2.467 = 2.011x + 0.279$

$2.689 = 2.011x + 0.279$

$2.890 = 2.011x + 0.279$

$3.074 = 2.011x + 0.279$

$3.395 = 2.011x + 0.279$

$3.696 = 2.011x + 0.279$

$3.853 = 2.011x + 0.279$

$1.863 = 2.011x + 0.279$

$2.068 = 2.011x + 0.279$

$2.467 = 2.011x + 0.279$

$2.689 = 2.011x + 0.279$

$2.890 = 2.011x + 0.279$

$3.074 = 2.011x + 0.279$

$3.395 = 2.011x + 0.279$

$3.696 = 2.011x + 0.279$

$3.853 = 2.011x + 0.279$

$1.863 = 2.011x + 0.279$

$2.068 = 2.011x + 0.279$

$2.467 = 2.011x + 0.279$

$2.689 = 2.011x + 0.279$

$2.890 = 2.011x + 0.279$

$3.074 = 2.011x + 0.279$

$3.395 = 2.011x + 0.279$

$3.696 = 2.011x + 0.279$

$3.853 = 2.011x + 0.279$
\[ \phi_y = 2.011x + 0.374 \]

- 7/8" EDGE, AVE. 2.944
- 3/4" EDGE, AVE. 2.369
- 5/8" EDGE, AVE. 1.715

Figure 4
Figure 4.6

Y-EDGE DISTANCE - INCHES

\[ \Phi_U = 3.05S - 0.50 \]

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RELATIONSHIP BETWEEN $\phi_u$ AND END DISTANCE

IN THE RANGE OF END FAILURES

TYPICAL NOTATION

$2.088$ — AVE. VALUE OF TESTS

$(30)$ — NO. OF TESTS

$X =$ END DISTANCE — INCHES

Figure 4.8
\[ \phi_U = 1.447X + 0.268 \]

- 1/4" EDGE, AVE. = 3.171
- 1 1/8" EDGE, AVE. = 2.766
- 7/8" EDGE, AVE. = 2.078
- 5/8" EDGE, AVE. = 1.217

FIGURE 4.9
RELATIONSHIP BETWEEN $\phi_u$ AND EDGE DISTANCE IN THE RANGE OF EDGE FAILURES

$\phi_u = 3.058y - 0.650$

$\phi_u = 3.058y - 0.745$

$S = 3.0y - \text{WINTER}$

$S = 2.25y - \text{WINTER}$

2.078 - AVE. VALUE OF TESTS

(90) - NO. OF TESTS

$y =$ EDGE DISTANCE - INCHES

FIGURE 4.10
DISTRIBUTION OF END AND EDGE TYPE FAILURES

(A) $y = 0.500X + 0.293$
(B) $y = 0.473X + 0.312$

FIGURE 4.11
### TABLES
<table>
<thead>
<tr>
<th>SPECIFICATION</th>
<th>YIELD STRENGTH MINIMUM - P.S.I.</th>
<th>TENSILE STRENGTH - P.S.I.</th>
<th>REFERENCE</th>
<th>THICKNESS LIMITS - INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td>C.S.A. - G40.4</td>
<td>33,000</td>
<td>60,000 - 72,000</td>
<td>1</td>
<td>t ≤ 4</td>
</tr>
<tr>
<td>A.S.T.M. - A36</td>
<td>36,000</td>
<td>60,000 - 80,000</td>
<td>2</td>
<td>t ≤ 8</td>
</tr>
<tr>
<td>C.S.A. - G40.6</td>
<td>45,000</td>
<td>80,000 - 95,000</td>
<td>3</td>
<td>5/8 &lt; t ≤ 1</td>
</tr>
<tr>
<td>C.S.A. - G40.8</td>
<td>40,000</td>
<td>65,000 - 85,000</td>
<td>4</td>
<td>1 &lt; t ≤ 1 - 1/2</td>
</tr>
<tr>
<td>C.S.A. - G40.8</td>
<td>36,000</td>
<td>65,000 - 85,000</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>C.S.A. - G40.12</td>
<td>44,000</td>
<td>62,000 Min.</td>
<td>5</td>
<td>t ≤ 3/4</td>
</tr>
<tr>
<td>C.S.A. - G40.12</td>
<td>40,000</td>
<td>62,000 Min.</td>
<td>5</td>
<td>3/4 &lt; t</td>
</tr>
<tr>
<td>RB-50</td>
<td>50,000</td>
<td>70,000 Min.</td>
<td>6</td>
<td>t ≤ 3/4</td>
</tr>
<tr>
<td>RB-55</td>
<td>55,000</td>
<td>70,000 Min.</td>
<td>6</td>
<td>t ≤ 3/4</td>
</tr>
<tr>
<td>RB-60</td>
<td>60,000</td>
<td>75,000 Min.</td>
<td>6</td>
<td>t ≤ 3/4</td>
</tr>
<tr>
<td>T-50</td>
<td>50,000</td>
<td>75,000 Min.</td>
<td>7</td>
<td>t ≤ 1 - 1/8</td>
</tr>
<tr>
<td>T-55</td>
<td>55,000</td>
<td>80,000 Min.</td>
<td>7</td>
<td>t ≤ 1 - 1/8</td>
</tr>
<tr>
<td>T-60</td>
<td>60,000</td>
<td>80,000 Min.</td>
<td>7</td>
<td>t ≤ 1 - 1/8</td>
</tr>
</tbody>
</table>

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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
<table>
<thead>
<tr>
<th>Section</th>
<th>Mat'l.</th>
<th>Test No.</th>
<th>Width</th>
<th>Thickness</th>
<th>Yield Load</th>
<th>Tensile Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_1\frac{3}{8}\times1\frac{3}{8}\times1/8$</td>
<td>G40.6</td>
<td>21</td>
<td>0.502</td>
<td>0.140</td>
<td>3,100</td>
<td>4,280 60,900</td>
</tr>
<tr>
<td>$L_1-3/4\times1-3/4\times1/8$</td>
<td>G40.4</td>
<td>30</td>
<td>0.501</td>
<td>0.139</td>
<td>3,840</td>
<td>55,200 67,650</td>
</tr>
<tr>
<td>$L_2 \times 2 \times 1/8$</td>
<td>G40.4</td>
<td>39</td>
<td>0.500</td>
<td>0.144</td>
<td>3,580</td>
<td>49,740 66,650</td>
</tr>
<tr>
<td>$L_2\frac{1}{2}\times2\frac{1}{2}\times1/8$</td>
<td>G40.4</td>
<td>51</td>
<td>0.503</td>
<td>0.136</td>
<td>3,280</td>
<td>47,960 64,350</td>
</tr>
<tr>
<td>$L_1\frac{1}{2}\times1\frac{1}{2}\times3/16$</td>
<td>G40.4</td>
<td>26</td>
<td>0.496</td>
<td>0.206</td>
<td>4,770</td>
<td>46,700 63,400</td>
</tr>
<tr>
<td>$L_1-3/4\times1-3/4\times3/16$</td>
<td>G40.4</td>
<td>35</td>
<td>0.505</td>
<td>0.194</td>
<td>4,480</td>
<td>65,750 61,050</td>
</tr>
<tr>
<td>$L_2 \times 2 \times 3/16$</td>
<td>G40.4</td>
<td>45</td>
<td>0.505</td>
<td>0.184</td>
<td>4,630</td>
<td>69,620 67,950</td>
</tr>
<tr>
<td>$L_2\frac{1}{2}\times2\frac{1}{2}\times3/16$</td>
<td>G40.4</td>
<td>55</td>
<td>0.506</td>
<td>0.193</td>
<td>5,100</td>
<td>52,200 68,900</td>
</tr>
<tr>
<td>$L_2\frac{3}{8}\times2\frac{3}{8}\times1/4$</td>
<td>G40.4</td>
<td>59</td>
<td>0.510</td>
<td>0.259</td>
<td>5,975</td>
<td>45,250 64,750</td>
</tr>
<tr>
<td>$L_1\frac{1}{2}\times1\frac{1}{2}\times1/8$</td>
<td>G40.6</td>
<td>23</td>
<td>0.504</td>
<td>0.141</td>
<td>4,050</td>
<td>57,000 83,200</td>
</tr>
<tr>
<td>$L_1-3/4\times1-3/4\times1/8$</td>
<td>G40.6</td>
<td>32</td>
<td>0.496</td>
<td>0.148</td>
<td>4,280</td>
<td>53,300 83,300</td>
</tr>
<tr>
<td>$L_2 \times 2 \times 1/3$</td>
<td>G40.6</td>
<td>42</td>
<td>0.505</td>
<td>0.138</td>
<td>4,450</td>
<td>63,900 87,150</td>
</tr>
<tr>
<td>$L_2\frac{1}{2}\times2\frac{1}{2}\times1/8$</td>
<td>G40.6</td>
<td>53</td>
<td>0.491</td>
<td>0.130</td>
<td>4,160</td>
<td>65,200 89,100</td>
</tr>
<tr>
<td>$L_1\frac{3}{8}\times1\frac{3}{8}\times3/16$</td>
<td>G40.6</td>
<td>28</td>
<td>0.495</td>
<td>0.205</td>
<td>6,360</td>
<td>62,720 85,750</td>
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<tr>
<td>$L_1-3/4\times1-3/4\times3/16$</td>
<td>G40.6</td>
<td>37</td>
<td>0.506</td>
<td>0.206</td>
<td>6,150</td>
<td>59,000 83,900</td>
</tr>
<tr>
<td>$L_2 \times 2 \times 3/16$</td>
<td>G40.6</td>
<td>47</td>
<td>0.509</td>
<td>0.194</td>
<td>6,350</td>
<td>64,350 87,150</td>
</tr>
</tbody>
</table>

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<table>
<thead>
<tr>
<th>Section</th>
<th>Material No.</th>
<th>Test No.</th>
<th>Dimensions</th>
<th>Yield Stress</th>
<th>Ultimate Tensile Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1½x1¾ x 1/8</td>
<td>G40.4</td>
<td>105</td>
<td>0.492</td>
<td>0.121</td>
<td>3,120</td>
</tr>
<tr>
<td>L1½x1¾ x 3/16</td>
<td>G40.4</td>
<td>108</td>
<td>0.491</td>
<td>0.194</td>
<td>4,740</td>
</tr>
<tr>
<td>L1-3¼x1-3/4 x 1/8</td>
<td>G40.4</td>
<td>111</td>
<td>0.501</td>
<td>0.131</td>
<td>3,590</td>
</tr>
<tr>
<td>L1-3¼x1-3/4 x 3/16</td>
<td>G40.4</td>
<td>112</td>
<td>0.499</td>
<td>0.186</td>
<td>4,120</td>
</tr>
<tr>
<td>L2 x 2 x 1/8</td>
<td>G40.4</td>
<td>123</td>
<td>0.495</td>
<td>0.142</td>
<td>3,500</td>
</tr>
<tr>
<td>L2 x 2 x 3/16</td>
<td>G40.4</td>
<td>134</td>
<td>0.503</td>
<td>0.177</td>
<td>4,730</td>
</tr>
<tr>
<td>L2½ x 2½ x 1/8</td>
<td>G40.4</td>
<td>141</td>
<td>0.500</td>
<td>0.139</td>
<td>3,690</td>
</tr>
<tr>
<td>L2½ x 2½ x 3/16</td>
<td>G40.4</td>
<td>142</td>
<td>0.497</td>
<td>0.200</td>
<td>5,020</td>
</tr>
<tr>
<td>L2½ x 2½ x 1/4</td>
<td>G40.4</td>
<td>143</td>
<td>0.512</td>
<td>0.266</td>
<td>7,150</td>
</tr>
<tr>
<td>L1½x1½ x 1/8</td>
<td>G40.6</td>
<td>195</td>
<td>0.494</td>
<td>0.122</td>
<td>3,850</td>
</tr>
<tr>
<td>L1-3/4x1-3/4 x 1/8</td>
<td>G40.6</td>
<td>196</td>
<td>0.496</td>
<td>0.132</td>
<td>4,180</td>
</tr>
<tr>
<td>L1-3/4x1-3/4 x 1/8</td>
<td>G40.6</td>
<td>197</td>
<td>0.494</td>
<td>0.135</td>
<td>4,160</td>
</tr>
<tr>
<td>L2 x 2 x 3/16</td>
<td>G40.6</td>
<td>207</td>
<td>0.500</td>
<td>0.190</td>
<td>6,100</td>
</tr>
<tr>
<td>L2½x2½x1/8</td>
<td>G40.6</td>
<td>225</td>
<td>0.499</td>
<td>0.131</td>
<td>4,480</td>
</tr>
<tr>
<td>L2½x2½x3/16</td>
<td>G40.6</td>
<td>226</td>
<td>0.507</td>
<td>0.184</td>
<td>6,150</td>
</tr>
<tr>
<td>L2½x2½x3/16</td>
<td>G40.6</td>
<td>227</td>
<td>0.493</td>
<td>0.188</td>
<td>6,280</td>
</tr>
<tr>
<td>L3 x 2½ x 1/4</td>
<td>G40.6</td>
<td>230</td>
<td>0.511</td>
<td>0.251</td>
<td>7,820</td>
</tr>
<tr>
<td>L1-3/4x1-3/4 x 1/8</td>
<td>G40.6</td>
<td>231</td>
<td></td>
<td>AVERAGE VALUES</td>
<td>64,325</td>
</tr>
<tr>
<td>L2½x2½x3/16</td>
<td>G40.6</td>
<td>232</td>
<td></td>
<td>AVERAGE VALUES</td>
<td>66,825</td>
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</tbody>
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**TABLE 3.2**

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
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<th>Thickness (in)</th>
<th>Yield Load (ksi)</th>
<th>Load Stress (ksi)</th>
<th>Ultimate Tensile Load (ksi)</th>
<th>Ultimate Tensile Stress (ksi)</th>
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<td>0.134</td>
<td>3,140</td>
<td>46,700</td>
<td>4,850</td>
<td>72,100</td>
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**TABLE 3.3**

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<th>Stress</th>
<th>Ultimate Tensile Load</th>
<th>Stress</th>
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<td>69,800</td>
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TABLE 3.4

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<th>YIELD LOAD</th>
<th>STRESS</th>
<th>ULTIMATE LOAD</th>
<th>STRESS</th>
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TABLE 3.5

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<th>Section</th>
<th>Mat‘l.</th>
<th>Test Dimensions</th>
<th>Yield Load</th>
<th>Stress</th>
<th>Ultimate Load</th>
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**TABLE 3.6**

*This material is obviously not RB60 Steel*

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AVERAGE VALUES OF \( \phi_2 \)

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<td>1/4</td>
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<td>2.742</td>
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<td>2.522</td>
<td>2.742</td>
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</tr>
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<td>2.522</td>
<td>2.742</td>
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<td>2.522</td>
<td>2.742</td>
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<td>2.522</td>
<td>2.742</td>
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To derive relationship between $\phi_y$ and end distance for end failures.

For data see Figures 4.1 and 4.2

<table>
<thead>
<tr>
<th>End Dist</th>
<th>$\phi_y$</th>
<th>$x = x - \bar{x}$</th>
<th>$\phi = \phi - \bar{\phi}$</th>
<th>$x\phi$</th>
<th>$x^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.750</td>
<td>1.863</td>
<td>-0.500</td>
<td>-1.025</td>
<td>0.5125</td>
<td>0.250</td>
</tr>
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<td>0.875</td>
<td>2.068</td>
<td>-0.375</td>
<td>-0.820</td>
<td>0.3075</td>
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<tr>
<td>1.000</td>
<td>2.467</td>
<td>-0.250</td>
<td>-0.421</td>
<td>0.10525</td>
<td>0.0625</td>
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<td>-0.200</td>
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<td>0.015625</td>
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<td>0.0000</td>
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<td>1.375</td>
<td>3.073</td>
<td>+0.125</td>
<td>+0.185</td>
<td>0.023125</td>
<td>0.015625</td>
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<tr>
<td>1.500</td>
<td>3.374</td>
<td>+0.250</td>
<td>+0.506</td>
<td>0.12650</td>
<td>0.0625</td>
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<tr>
<td>1.625</td>
<td>3.676</td>
<td>+0.375</td>
<td>+0.808</td>
<td>0.30300</td>
<td>0.140625</td>
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<td>+0.500</td>
<td>+0.965</td>
<td>0.48250</td>
<td>0.2500</td>
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<tr>
<td>Σ = 11.250</td>
<td>25.992</td>
<td>0</td>
<td>0</td>
<td>1.885375</td>
<td>0.937500</td>
</tr>
</tbody>
</table>

$\bar{x} = \frac{11.250}{9} = 1.250$  \[ \bar{\phi}_y = \frac{25.992}{9} = 2.888 \]

For $\phi_y = \delta y + a$

$\delta = \frac{\sum x\phi}{\sum x^2} = \frac{1.885375}{0.93750} = 2.011067$

$\phi_y - \bar{\phi}_y = \delta (x - \bar{x})$

$\phi_y = 2.888 - 2.011067 \times 1.25 = 2.011067 \times$

$\phi_y = 2.888 - 2.513033 = 2.011067 \times$

$\phi_y = 2.011 + 0.374$ (Eqn 4.1)
To calculate standard deviation of $\phi_y$ as given by equation 4.1

<table>
<thead>
<tr>
<th>$x$</th>
<th>$\phi_y$</th>
<th>$2.01107 \times$</th>
<th>$\phi_y \times 2.01107$</th>
<th>$\epsilon = \phi_y - \phi_y \times 2.01107$</th>
<th>$\epsilon^2$</th>
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<tbody>
<tr>
<td>0.750</td>
<td>0.374 + 1.509300 = 1.883300</td>
<td>-0.019 300</td>
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<tr>
<td>0.875</td>
<td>0.374 + 1.759383 = 2.133 683</td>
<td>-0.065 683</td>
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<tr>
<td>1.000</td>
<td>0.374 + 2.011067 = 2.385 067</td>
<td>+0.081 933</td>
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<tr>
<td>1.125</td>
<td>0.374 + 2.762 650 = 3.136 650</td>
<td>+0.051 550</td>
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<tr>
<td>1.250</td>
<td>0.374 + 2.513 833 = 2.887 833</td>
<td>+0.002 167</td>
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<tr>
<td>1.375</td>
<td>0.374 + 2.765 217 = 3.139 217</td>
<td>-0.066 217</td>
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<tr>
<td>1.500</td>
<td>0.374 + 3.016 600 = 3.379 600</td>
<td>+0.003 400</td>
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<tr>
<td>1.625</td>
<td>0.374 + 3.267 983 = 3.641 983</td>
<td>+0.054 017</td>
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<tr>
<td>1.750</td>
<td>0.374 + 3.519 367 = 3.893 367</td>
<td>-0.040 367</td>
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</table>

\[ S_{\phi_y} = \sqrt{\frac{\sum \epsilon^2}{n - 2}} = \sqrt{\frac{0.023 005 443 434}{9 - 2}} = \sqrt{0.003 286 491 91} = 0.057 327 933 \]

To check correlation of equation 4.1

\[ \text{correlation coefficient} = r = \frac{\Sigma X \phi}{\sqrt{\Sigma X^2 \Sigma \phi^2}} \]

\[ \Sigma X = 0.937 500 \quad \Sigma \phi^2 = 3.814 620 \]

\[ r = \frac{1.885 375}{\sqrt{0.937 500 \times 3.814 620}} = 0.997 0 \]

Table A.11 of reference (12) gives $r = 0.798$ for $\gamma = n - 2 = 7$ at 1% level of significance.

The correlation of Eqn 4.1 is excellent.
TO COMPARISON 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.

| ENO | 2 | n | $\sum (\hat{x}_i - \bar{x})^2$ | $\nu$ | $\frac{\sum (\hat{x}_i - \bar{x})^2}{\nu}$ | $\log \hat{\sigma}^2$ | $(n-1) \log \hat{\sigma}^2$
<table>
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<td>0.00219567</td>
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<td>0.001065,229</td>
<td>3.0273312</td>
<td>9 + 0.0822735</td>
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<tr>
<td>0.875</td>
<td>6</td>
<td>0.011417,750</td>
<td>5</td>
<td>0.101,389,500</td>
<td>2.01653</td>
<td>7 + 0.082,845</td>
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<tr>
<td>1.6</td>
<td>5</td>
<td>0.0531,620</td>
<td>4</td>
<td>0.013,225,000</td>
<td>2.129,66</td>
<td>8 + 0.498,440</td>
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<td>1.255</td>
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<td>4</td>
<td>0.007,918,200</td>
<td>3.898,63</td>
<td>12 + 3.597,520</td>
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<td>3.918,025</td>
<td>9 + 2.81,4075</td>
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<td>2.772,447</td>
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<td>2.249,479</td>
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<td>2</td>
<td>0.527,489,43</td>
<td>35</td>
<td>80</td>
<td>16.950,847</td>
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$$\frac{3}{2} - \frac{\sum (\hat{x}_i - \bar{x})^2}{\sum (n-1)} = 0.527,489,43 - 0.015,984,5$$

$$\log \hat{\sigma}^2 = -2.203,7015 = -1.796,298,5$$

$$\chi^2 = 2.30,12 \times [\log \hat{\sigma}^2 - \sum (n-1) \log \hat{\sigma}^2]$$

$$= 2.30,12 \times [-1.796,298,5 - 13 + 0.950,847,8]$$

$$= 2.30,12 \times 5.7713 = 13.289$$

FOR $\alpha = 0.05$, $\chi^2 = 13.367$ AT 10% LEVEL OF SIGNIFICANCE

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

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TO PREDICT CONFIDENCE INTERVAL OF VALUES
OF $\Phi_Y$ AS GIVEN BY EQUATION 4.1

FROM REFERENCE (12), PAGE 181

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

$$S_{\phi_i} = 0.057327933 \sqrt{1 + \frac{1}{9} + \frac{(\Phi_i - \bar{\Phi})^2}{0.937500}}$$

MINIMUM VALUE OF $\Phi = 0.75$

$$S_{\phi_{0.75}} = 0.057327933 \sqrt{1 + 0.11111 + \frac{(0.75 - 1.25)^2}{0.937500}}$$

$$= 0.057327933 \times 1.37778$$

$$= 0.057327933 \times 1.173787771$$

$$S_{\phi_{0.75}} = 0.067291 = S_{\phi_{1.75}}$$

FOR 10% LEVEL OF SIGNIFICANCE IN A ONE-SIDED TEST SEE TABLE 3 OF "STATISTICAL MANUAL" BY E.L. CLOW, F.A. DAVIS AND H.W. NARFIE.<br>

$L = 1.915$ FOR $\mu = 9.2 - 7$

THUS $\Phi_{0.75} = 1.8823 - 1.915 \times 0.067291$

$$= 1.8823 - 0.0952$$

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

$n = \Phi_{1.75} - 3.8934 = 0.0952$

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MEAN VALUE OF \( t = 1.25 \)

\[
\Phi_{1.25} = 0.057327933 \sqrt{1 + 0.11111} = 0.057327933 \times 1.054092555 = 0.060428947
\]

FOR 10% LEVEL OF SIGNIFICANCE

\[
\Phi_{0.15} = 2.71832 - 1.415 \times 0.060428947 = 2.71832 - 0.08550696
\]

IF IN PLACE OF THIS VALUE OF \( \Phi_{0.15} \) WE USE \( \Phi_{0.095} \)

THE VALUE OF \( t \) THUS INCLUDED IS

\[
t = \frac{0.0952}{0.060428947} = 1.5754
\]

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

THUS IF \( t = 0.0952 \) IS USED OVER THE RANGE OF \( y \) FROM 0.750" TO 1.750" THE LEVEL OF SIGNIFICANCE WILL RANGE FROM 10% MAXIMUM TO 8.3% MINIMUM.

\[
\Phi_y = 2.011y + 0.279 \quad (60 \text{°} 4.5)
\]
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.

Awarded the Bronze Medal for first place standing in Civil Engineering by Queen's University.

Awarded an Athlone Fellowship for the study of Concrete Technology, in the United Kingdom by the United Kingdom Government.
2.

1951 September, entered the Civil Engineering Department of City and Guilds College, Imperial College, University of London, London, England.


1958 Appointed Tower Engineer in the Engineering Department of Canadian Bridge Company, Walkerville, Ontario, Canada.

1965 Appointed Chief Engineer, Dosco Industries Limited, Canadian Bridge Division, Walkerville, Ontario, Canada.

Technical Societies

Member of the Association of Professional Engineers of Ontario.

Member of the Engineering Institute of Canada.