1963

# Long bolted joints. Proc. ASCE, Vol. 89 (ST6), December 1963, Publication No. 231 (63-17)

R. A. Bendigo

R. M. Hansen

J. L. Rumpf

① ~~CHS~~
② ~~GLK~~
③ ~~RK~~
④ JWF

any comments

~~Return to JLR~~

TO John Fisher

**LONG BOLTED JOINTS**[a]

Closure by John L. Rumpf

J. L. Rumpf[30]

The authors wish to acknowledge the contribution of Professor Francis to the literature on the behavior of structural connections.[19] Rather than escaping the notice of the authors his work provided a valuable basis for the analytical work[15] carried on simultaneously with the experimental work to which this paper was confined.[a] The analytical solutions have now progressed to computerized form and as such have become successful in reducing the amount of costly physical testing of large joints[21,22]. It is expected that the analytical work will be available in a more readily obtainable form in the near future so that engineers may compare the analytical and experimental results in detail. At this point we will simply state that the correlation has been excellent.

Professor Chesson has tried to correlate tests of the connections of various truss type members[23,24] having fasteners in single shear with the results of the double shear plate splices reported in this paper. While it is true that the truss type connection is more complex in its method of load transfer than the flat plate splice this is not the way to explain what may appear to some to be a large deviation from the results of this paper. Chesson has failed to recognize the effect on the unbuttoning factor of varying the connection geometry (T/S ratio). The T/S ratio may also be expressed as $A_s/A_n$ where the numerator representing the bolt shear area is usually expressed as 1, and where the denominator representing the net area of the plate indicates the relative proportions of the connection. If the net area is very large the plate stresses and strain are negligible and the plate functions practically as a rigid member. As a result the "Mean Curve" shown in Fig. 8 (or Fig. 14) becomes horizontal through an unbuttoning factor of 1.0. On the other hand if the net area is reduced markedly the plate strains become very great thus hastening the premature failure of the end fasteners. In this case the "Mean Curve" takes a steeper slope.

The bolted joints reported by Chesson had $A_s/A_n$ ratios ranging from 1:0.65 to 1:0.88 whereas the "Mean Curve" of Fig. 8 is for joints having a ratio of 1:1.10. Thus it is to be expected that ~~the~~ his plotted points fall below this line as they do. Similarly for the riveted connections ~~where~~ the $A_s/A_n$ ratio ranged from 1:0.65 to 1:0.70 whereas the limited Lehigh University riveted connections had ratios of 1:0.76 to 1:0.78. Use of the analytical method noted in the first paragraph shows Chesson's experimental work to fall close to the predicted values. (See Fig. 9, Ref.22)

Two other items prevent a direct comparison. In the connections reported by Chesson (1) the bolts sheared through the threads and (2) the main members were of higher strength A7 steel. ? "higher strength than A7 steel" perhaps.

It should be noted that the analytical analysis of flat plate joints of Refs. 15 and 22 is not limited to equal inner and outer ply dimensions as stated by Chesson.

## List of References

(Where a reference has been used before in the original paper or the three discussions the same number has been assigned in this closure.)

In order of appearance in closure.


a. December 1963 by Robert A. Bendigo, Roger M. Hansen, and John L. Rumpf (Proc. Paper 3727)

80. Professor and Head, Department of Civil Engineering, Drexel Institute of Technology, Philadelphia, Pa.

19. Francis, A. J., "The Behavior of Aluminium Alloy Riveted Joints", Research Rpt. No. 15, Aluminium Development Association, London, 1953.

15. Rumpf, J. L., "The Ultimate Strength of Bolted Connections", Ph.D. Dissertation, Lehigh University, 1960.

21. Fisher, J. W., "The Analysis of Bolted Plate Splices", Ph.D. Dissertation, Lehigh University, 1964.

22. Fisher, J. W., and Rumpf, J. L., "The Analysis of Bolted Butt Joints", Report 288.17, Fritz Engineering Laboratory, Lehigh University, 1964.

23. Chesson, E., Jr., and Munse, W. H., "Behavior of Riveted Truss-Type Connections," Transactions, ASCE, Vol. 123, 1958, p. 1087.

24. Munse, W. H., and Chesson, E., Jr., "Riveted and Bolted Joints: Net Section Design," Journal of the Structural Division, ASCE, Vol. 89, No. ST1, Proc. Paper 3413, February, 1963.

ERRATA

p. 194, line 20

should read "....ultimate shear strength of 43.5 ksi ~~45.3~~ ksi

Please check

January 20, 1965

Mr. Irvin J. Schwartz
Editor, Technical Publications,
American Society of Civil Engineers
345 East 47th Street
New York 17, New York

Re: Digest for TRANSACTIONS
Paper #3727

Dear Mr. Schwartz:

Enclosed find two (2) copies of the digest of Proceedings Paper #3727 entitled "Long Bolted Joints."

Very truly yours,

John L. Rumpf
Professor and Head
Civil Engineering and Mechanics

JLR:eh

Enclosures (2)

cc: J. W. Fisher

DIGEST

of

<u>Long Bolted Joints</u>

By Robert A. Bendigo,[1] Roger M. Hansen,[2] and John L. Rumpf,[3] M. ASCE

Tension tests of twenty long structural splices of A7 steel plate connected with high strength bolts provide background for parts of the specification of the Research Council on Riveted and Bolted Structural Joints for the assembly of structural connections using A325 bolts. The principal item under investigation was the effect of the length of the joint on the ultimate strength of bearing-type connections but valuable information for the design of friction-type connections was determined also.

The tests illustrated again the phenomenon of premature unbuttoning-type failure of the end fasteners in long connections. Short connections of approximately 16 inches in length failed by shearing of bolts at an average shear stress ~~only~~ of 85% of the strength of a single bolt. For a connection with 16 bolts in line and an overall length of 52.5 inches the average shear at failure was reduced to only 60%. Considering an allowable design shear stress $F_v = 22$ ksi the factor of safety against failure for the short connection is about 3.0 but this decreases to about 2.1 for the long connection. A few comparison tests of riveted connections showed similar behavior. Comparison of several connections with various pitch dimensions shows that the unbuttoning of end fasteners depends on the length of the joint and not on the number of bolts in line, per se.

Keep

John Fisher

Item for further analytical research on bolted joints -

See p. 418, June 1964 Structural Division

A J. Francis discussion of "Long Bolted Joints"

Reduce plate thickness to increase bearing stress thus making "bolt nucleus" more flexible.

This is coupled with greater flexibility in plate if "g" is controll, hence resulting increase may not be beneficial as interaction is nec.)

Questions

What is a practical minimum thickness under present day conditions?

As width increases, thickness can decrease. As width increases more gage lines can be fitted into the width, thus more fasteners, and thus a shorter joint and less unbuttoning.

But narrower members give less wind resistance, less painting surface, greater thickness is better for corrosion resistance, narrower members take up less space architecturally.

JWF:

Please read the attached closure on "Long Bolted Joints" and call me promptly if you have any suggestions. I must have it in to ASCE by the 8th.

The A325 bolts tightened by the turn-of-nut method consistently had tensions of approximately 1.30 times the proof load of the bolt. No detrimental shear behavior of a connection was experienced because of this high initial tension. Joints with tight mill scale faying surfaces slipped into bearing at values of average shear stress in excess of $F_v = 1.33 \times 15 = 20$ ksi for dead and live load plus wind. These tests indicated 0.35 as a reasonable lower limit for the coefficient of slip.

Joints with semi-polished surfaces obtained by removing all mill scale with a power tool slipped at an average shear stress greater than 15 ksi but less than 20 ksi and a coefficient of slip of 0.20.

Keep.

LONG BOLTED JOINTS[a]

Closure by John L. Rumpf

J. L. Rumpf [30]

The authors wish to acknowledge the contribution of Professor Francis to the literature on the behavior of structural connections.[19] Rather than escaping the notice of the authors his work provided a valuable basis for the analytical work[15] carried on simultaneously with the experimental work to which this paper was confined.[a] The analytical solutions have now progressed to computerized form and as such have become successful in reducing the amount of costly physical testing of large joints[21,22]. It is expected that the analytical work will be available in a more readily obtainable form in the near future so that engineers may compare the analytical and experimental results in detail. At this point we will simply state that the correlation has been excellent.

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ERRATA

p. 194, line 20

should read "....ultimate shear strength of 45.3 ksi

Please check.

**AMERICAN SOCIETY OF CIVIL ENGINEERS**
United Engineering Center
345 East Forty-Seventh Street
New York 17, N. Y.

November 8, 1963

This will acknowledge your communication dated Nov. 7, 1963
Please be assured that this matter will be given our careful attention.

*Technical Publications Department*

THIS SIDE OF CARD IS FOR ADDRESS


Mr. John W. Fisher
Department of Civil Engineering
Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

November 7, 1963

288.1

Ref: 16-2-4.B

Mr. William D. French
Assistant Editor
Technical Publications
American Society of Civil Engineers
345 E. 47th Street
New York 17, New York

Dear Mr. French,

Your communication addressed to Mr. Robert Bendigo dated October 28, 1963 reached us on November 6, 1963. Upon receipt of the edited photocopy we have reviewed your editorial changes and have the following comments:

On page 1, the location should be given as New York for reference number 4. References 10 and 11 need the appropriate date and city of publication inserted. Reference 13 has a word deleted from the title.

Several corrections have been made to tables 6, 7, and 8. Several of the sub-headings have been modified. In table 6 we have completed the column labelled grip. The last column of table 8 has been deleted.

On pages 12 and 14 we would prefer that the allowable shear stress be included in the text and not centered in the page.

Mr. William D. French November 7, 1963

The intent of the summary and conclusions is modified if "bearing-type joints" and "friction-type joints" are centered and capitalized. These are intended as sub-headings. We would prefer to see them italicized as the other sub-headings have been.

Several other minor modifications have been made on various pages of the photocopy. The other editorial changes you made are satisfactory.

If we can be of further assistance please advise us. We have certainly appreciated the work you have done in getting this manuscript into final form.

Sincerely yours,

John W. Fisher

JWF/va
cc: L. S. Beedle
J. L. Rumpf

TELEPHONE PLAZA 2-6800
CABLE ADDRESS, CEAS, NEW YORK

WILLIAM H. WISELY
EXECUTIVE SECRETARY

AMERICAN SOCIETY OF CIVIL ENGINEERS

UNITED ENGINEERING CENTER
345 EAST FORTY-SEVENTH STREET · NEW YORK 17, NEW YORK

October 28, 1963
File: 16-2-4.B

Mr. Robert A. Bendigo
Bethlehem Steel Company
Bethlehem, Pennsylvania

Dear Mr. Bendigo:

This is in reference to the paper by yourself, Roger M. Hansen and John L. Rumpf entitled "Long Bolted Joints" which has been accepted for publication by the Society.

A photocopy of the edited paper is enclosed for your review. Form and style markings such as literary references, tabulation arrangement, etc., are readily recognizable. Other editorial changes may be an attempt (for example) to convert your script to the third person style in accord with Society requirements. If we have inadvertently altered your meaning, please feel free to restore the original or possibly rephrase it to perfect that area. The enclosed copy of "The Authors' Guide to the Publications of the Society" should assist you in this final review before publication.

Because of the minor changes involved, and in order to avoid delays, I have released your paper directly to the printer for the December issue of the Journal of the Structural Division. If you wish to adjust the enclosed material, it will be necessary to return the photocopy with any corrections or changes indicated in red on or before November 8, 1963. To spare you the bother of correspondence, your approval will be assumed if no communication is received from you prior to this deadline date.

Thank you for your ready willingness to cooperate.

Very truly yours,

William D. French
Assistant Editor, Technical Publications

WDF:na
Enclosures

July 23, 1963

288.1

Mr. Paul A. Parisi
Manager, Technical Publications
American Society of Civil Engineers
345 E. 57th Street
New York 17, New York

RE: <u>File 16 - 2 - 4.B</u>

Dear Paul:

Enclosed herewith is the corrected copy of our manuscript entitled "Long Bolted Joints" by Robert Bendigo, Roger Hansen and John Rumpf. The manuscript was reviewed and revised where appropriate after receiving your letter of July 8, 1963 with the reviewers comments. We believe the Tables to now be at a minimum if they are to be of use to others.

In compliance with your sheet of "necessary adjustments" we have prepared the following items for your use.

1. A 50-word abstract for Civil Engineering.
2. A 175-word abstract for the information-retrieval program.
3. A list of Figure titles.
4. A set of instructions for aiding in the reproduction.
5. An appendix of Nomenclature.

We wish to place an order for reprints at this time. Will you please furnish us with 700 reprints. Enclosed herewith is a sample cover from a previous ASCE reprint. We would like to have "Fritz Lab. ~~Report~~ <u>No. 231</u>" placed at the top of the cover as illustrated. Reprint

When you are finished with the ink tracing and photographs we would appreciate your returning them to our files.

Sincerely yours

Lynn S. Beedle

JWF/va
Encl:
cc: R. A. Bendigo, R. M. Hansen, J. L. Rumpf, J. W. Fisher

TELEPHONE PLAZA 2-6800
CABLE ADDRESS, CEAS, NEW YORK

WILLIAM H. WISELY
EXECUTIVE SECRETARY

AMERICAN SOCIETY OF CIVIL ENGINEERS

UNITED ENGINEERING CENTER
345 EAST FORTY-SEVENTH STREET · NEW YORK 17, NEW YORK

July 8, 1963

File: 16-2-4.B

Mr. Lynn S. Beedle
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

Dear Lynn:

We are pleased to notify you that the paper by Robert Bendigo, Roger Hansen, and John Rumpf entitled "Long Bolted Joints" has been approved for publication in the Journal of the Structural Division. Prior to processing it further, however, we ask that you consider the suggestions offered by the Division reviewers in the enclosed abstracts of their comments, and the enclosed copies of the paper (which should be returned).

You can make the necessary adjustments directly on the enclosed manuscript. If the changes to a particular page are such that it will be difficult for a non-technical typist to follow, would you please re-type that page?

As we would like to include your paper in the forthcoming Journal, it is necessary that we receive your adjustments by July 26, 1963. If you cannot complete the necessary work by this deadline, please advise us as to the additional time needed. Failing to hear from you by then, we will be obliged to list the paper as "tentatively withdrawn by author, without prejudice when the revised paper is submitted at some later date". By this device, we are able to grant an author unlimited time, subject also to our own schedules, without the annoyance sometimes created by our routine follow-up inquiries. However, it is hoped that you will be able to meet the above deadline.

Very truly yours,

Paul A. Parisi
Manager, Technical Publications

PAP:mm

REVIEWER'S COMMENTS ON THE PAPER "LONG BOLTED JOINTS" BY BENDIGO, HANSEN, AND RUMPF

## Reviewer A

Since other data suggests that a somewhat different picture of behavior could be obtained if other heats of steel from other mills had been used, it would be desirable for the authors to qualify their conclusions to indicate this fact. The application of the data reported without consideration of the other data in the literature could provide structures on the unsafe side insofar as slip of the joint is concerned.

Pg 20 Concl 5 + 6 begin "These tests ---

If this is so then the Research Council is 'way out on a limb with its specification.

## Reviewer B

OK - Correction in order

Specific recommendations would improve value. Figure 3, Col 3, not clearly identified. Ref. 9 and 11 not readily available. Conclusion on $k_s \geq 0.35$ may be questioned.

See pg 20 Concl 5 + 6

Form 6

## NECESSARY ADJUSTMENTS

- ____ The title should have a maximum length of 50 characters, including spaces.
- ____ The paper should be written in the third person.
- By JLR ✓ A 50-word abstract, for use in CIVIL ENGINEERING, is required.
- ____ The paper must be preceded by a Synopsis of about 300 words.
- ____ A set of Conclusions should be appended to bring the paper to a logical close.
- ____ Centered and side headings should be used to divide paper into logical parts.
- JWF ✓ The references should be typed as footnotes on the pages on which they are first cited; the format to be used is illustrated in the enclosed Guide. ✓
- JWF ✓ The first reference-footnote should be given a number in sequence with the separate footnotes used for the present-employment description of the authors. ✓
- ____ The explanatory footnotes should be made a part of the text.
- ____ All centered equations should be numbered in sequence.
- JWF ✓ Mathematics are recomposed from the copy that is submitted. Because of this, equations should be typed as fully as possible, hand written symbols should be drawn carefully, and special symbols are to be properly identified. ✓
- JWF ✓ The letter symbols should be defined where they first appear and summarized alphabetically in an Appendix.
- JWF ✓ Figure captions should be listed separately.
- ____ Illustrations must be drawn in black ink on $8\frac{1}{2}$ in. by 11 in. paper. Because they will be reproduced with a width of between 3 in. and $4\frac{1}{2}$ in., the lettering must be large enough to be legible at this reduced size.
- ____ Extensive notes and descriptions should be removed from the figures.
- ____ Photographs should be submitted as glossy prints.
- ____ A copy of a recent paper is enclosed to show the quality of illustration we would like to receive.
- JWF ✓ Tables should be typed on one side of $8\frac{1}{2}$ in. by 11 in. paper; an original and one duplicate are required.
- ✓ Tables should only include enough data to indicate the general thesis of the paper.

  Now at a min. We want others to be able to verify data

- JLR ✓ To advance the information-retrieval program described in the enclosed reprint form CIVIL ENGINEERING, we need an informative abstract with a maximum length of 175 words and a list of key words.

John F.

In Table 3, Col 3.

Heading should be Length
Under
Head

See my comments on Sheet 2 "Comments"

See Sheet 3 Necessary Adjustments.
You are in a better position to
do some of these items than I am.

Check over my "Key Words"

Call me if any problems arise.

John.

of the two types of problems. For the case where the frame is loaded by the distributed load, all the members in the frame are subjected to combined bending moment and axial force, whereas in the case with the simplified loading condition only axial forces are present in the members at the instant of buckling. In general, the presence of bending moment in the members causes only a small reduction in the buckling load for portal frames considered here. Detailed discussions of the effect of bending moments on elastic frame instability can be found in Refs. 7.3 and 7.4. In the following it will be assumed that the simplified loading condition is applicable in determining the elastic buckling load of the frames.

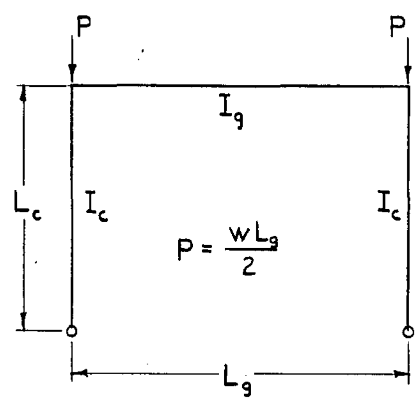


FIG. 7.3

### 7.2.2 Methods of Buckling Analysis

Numerous methods have been developed and effectively used in solving stability problems related to building frames. They resemble very much those methods ordinarily used in analyzing statically indeterminate structures, except that some modification is made to include the bending effect of the axial forces. Essentially there are three different avenues of approaches that are now in common use. These are, 1) the analytical methods including the methods of slope-deflection and four-moment equation, 2) the moment distribution method, equal the tensile capacity of the net section of the main material. Tests of compact bolted joints(6) established the balanced design tension-shear ratio for A7 steel and A325 bolts at 1 to 1.10. In other words, equal ultimate strength will result if the areas of the main material and bolts are chosen so that the average tensile stress on the net section and the average shear stress on the bolts are in the ratio of 1 to 1.10.

In design calculations for riveted and bolted work it is assumed that each fastener carries an equal share of the load. However, analytical work(7,8) has shown that in long connections, with all parts behaving elastically, the end fasteners carry a much larger proportion of the load than do the inner fasteners. But, experimental work with riveted connections(9) has shown that successive yielding of the outer rivets produces a redistribution of load so that at failure a more uniform distribution exists than the elastic analysis indicates. In the case of hot driven rivets, it usually has been assumed that the equalization of load is complete enough that no changes in design procedure need be specified for long connections.

The amount of redistribution of load depends upon the ability of the fasteners to undergo large shear deformations without failing. Because the properties of high strength bolts differ from those of rivets, it was deemed advisable to conduct tests of bolted joints in order to measure the ability of the bolts to redistribute load, and to determine if long bolted connections designed with a tension-shear ratio of 1 to 1.10 still provided a reasonable factor of safety. This work was started in 1958 and provided further background for the formulation of portions of the specifications of the Research Council on Riveted and Bolted Structural Joints.

---

6. "Static Tension Tests of Compact Bolted Joints", by R. T. Foreman, J. L. Rumpf, Transactions ASCE, Vol. 126, Paper No. 3125, p. 228 (1961)
7. "The Partition of Load in Riveted Joints", by C. Batho, Journal of Franklin Institute, Vol. 182, p. 553 (1916)
8. "The Work of Rivets in Riveted Joints", by A. Hrennikoff, Transactions ASCE, Vol. 99, p. 437 (1934)
9. "Tension Tests of Large Riveted Joints", by R. E. Davis, G. B. Woodruff, Davis, H. E., Transactions ASCE, Vol. 105, p. 1193 (1940)

## APPENDIX - NOMENCLATURE

1. The following symbols have been adopted for use in this paper:

$A_n$ = net tensile area of the plate

$A_s$ = bolt shear area (for butt-type splices there are two shear planes)

$F_v$ = allowable shear stress

$K_s$ = slip coefficient

m = number of shear planes

n = number of bolts

$P_s$ = major slip load

T/S = ratio of the tensile stress on the net section of plate to the shear stress on the nominal area of the fasteners $(A_s/A_n)$

$\bar{T}i$ = average initial bolt tension

U = the unbuttoning factor - defined as the ratio of the average bolt shear stress in the connection when the first bolt shears to the ultimate strength of a single bolt of the same lot and of the same grip

$U_L$ = unbuttoning factor for length, L

$\sigma_{min}$ = minimum tensile strength for A325 bolt

$\tau_u$ = the average bolt shear stress in the bolted connections

$\tau_1$ = average shear strength of a single bolt

2. The following terms are used:

| | |
|---|---|
| Gage | The transverse spacing of the bolts |
| Grip | The thickness of the plate material in the connection |
| Pitch | The longitudinal spacing of the bolts |
| Snug | The expression used to describe the tightness of a bolt before beginning the turn of the nut. "Snug" is indicated by the impact wrench when impacting begins |
| Unbuttoning | The sequential failure of fasteners which progresses from the ends of a joint inward |

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## LONG BOLTED JOINTS

50 word ABSTRACT for Civil Engineering

ABSTRACT: Tension tests of long structural splices of A7 steel provide background for portions of the specification of the Research Council on Riveted and Bolted Structural Joints for joints using A325 bolts. Data on the slip resistance and ultimate strength of connections are given.

# INSTRUCTIONS FOR PAPER

1. Figures should be interspaced with the written text. An approximate location of each is noted in pencil.

2. Figure titles are to be set in type as shown on the copies and on the enclosed list.

3. Figures 1, 4 and 5 contain tabulated data which should be set in type. The small tracings heading these Figures are enclosed. (271.168, 271.169 and 271.170)

4. It would be desirable if all Figures except Figs. 6 and 7 appeared in the normal position in which the journal is held. All Figures can be reduced without impairing their understanding.

5. The following is a tabulation of the Figs.

| Figure | Tracing No. |
|---|---|
| 1 | 271.168 |
| 2 | Tables |
| 3 | Tables |
| 4 | 271.169 |
| 5 | 271.170 |
| 6 | Tables |
| 7 | Tables |
| 8 | 271.171 |
| 9 | 271.171 |
| 10 | Photo-Sawed Section Joint D61 |
| 11 | Photo-Sawed Section Joint DR71 |
| 12 | Tables |
| 13 | Tables |
| 14 | 271.172 |
| 15 | 271.173 |
| 16 | 271.174 |
| 17 | 271.175 |
| 18 | 271.175 |

6. Figures 1, 2, 3, 4, 5, 6, 7, 12, and 13 contain tabulated data which should be set in type.

# INSTRUCTIONS FOR PAPER

1. Figures should be interspaced with the written text. An approximate location of each is noted in pencil.

2. Figure titles are to be set in type as shown on the copies and on the enclosed list.

3. Figures 1, 4 and 5 contain tabulated data which should be set in type. The small tracings heading these Figures are enclosed. (271.168, 271.169 and 271.170)

4. It would be desirable if all Figures except Figs. 6 and 7 appeared in the normal position in which the journal is held. All Figures can be reduced without impairing their understanding.

5. The following is a tabulation of the Figs.

| Figure | Tracing No. |
|---|---|
| 1 | 271.168 |
| 2 | Tables |
| 3 | Tables |
| 4 | 271.169 |
| 5 | 271.170 |
| 6 | Tables |
| 7 | Tables |
| 8 | 271.171 |
| 9 | 271.171 |
| 10 | Photo-Sawed Section Joint D61 |
| 11 | Photo-Sawed Section Joint DR71 |
| 12 | Tables |
| 13 | Tables |
| 14 | 271.172 |
| 15 | 271.173 |
| 16 | 271.174 |
| 17 | 271.175 |
| 18 | 271.175 |



6. Figures 1, 2, 3, 4, 5, 6, 7, 12, and 13 contain tabulated data which should be set in type.

## FIGURE TITLES

## FIGURE TITLES

# LONG BOLTED JOINTS

175 word INFORMATIVE ABSTRACT for information-retrieval program

KEY WORDS: Connections, bolts, strength, slip, steel, tests


ABSTRACT: Tension tests of long structural butt splices of A7 steel connected with 7/8" high strength bolts provide background for portions of the specification of the Research Council on Riveted and Bolted Structural Joints for structural joints using A325 bolts. Joints proportioned for a tensile stress of 20 ksi and a shear stress of 22 ksi produced a balance ultimate strength design for short connections approximately 16" long. In longer connections end bolts sheared before all bolts could develop their full shearing strength. In short connections the average shear stress was about 85% of the strength of a single bolt but bolts in a 52.5" (16@ 3-1/2") long connection developed only 60%.

Joints with tight mill scale faying surfaces and bolts tightened by the turn-of-nut method slipped into bearing at a lower limit of slip coefficient of 0.35. When the mill scale was removed with a power tool the slip coefficient was only 0.20.

Limited tests of bolted lap joints provide information on the behavior of bolts in single shear. Several tests of riveted connections give interesting comparative data.

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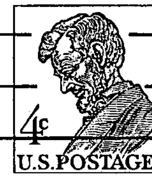


THIS SIDE OF CARD IS FOR ADDRESS

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# LONG BOLTED JOINTS



by

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# LONG BOLTED JOINTS

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SYNOPSIS

Tension tests of long structural splices of A7 steel connected with high strength bolts provide background for portions of the specification of the Research Council on Riveted and Bolted Structural Joints for structural joints using A325 bolts. The influence of the length of the joint on the ultimate strength of bearing-type connections is determined. Data on slip resistance also is obtained for use in designing friction-type connections.

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# LONG BOLTED JOINTS

## INTRODUCTION

When the A325 high strength bolt was first introduced it was used as a 1 for 1 replacement for rivets made of A141 steel(1). Eventually, as the concept of the high strength bolt became accepted, it was feasible to try to refine the design of bolted joints to take advantage of the greater strength of the bolt. Under the guidance of the Research Council on Riveted and Bolted Structural Joints research was conducted at several universities which led to a specification(2) acknowledging the different behavior of friction-type and bearing-type connections.

In the friction-type joint movement of the connected parts under static, impact or cyclic loading is not tolerated because of the detrimental effect on the behavior and configuration of the structure. In this case, slip constitutes failure and working loads must be resisted by friction with a reasonable factor of safety against the occurrence of slip.

On the other hand there are many connections that are erected in a "slipped" position with the bolts bearing against the sides of the holes in the same way that they will bear under service loads. In such a bearing-type connection it is the shearing of the bolts that constitutes failure and allowable stresses are based on the ultimate strength of the bolts. It is in this case that the greater strength of the A325 bolt can be utilized resulting in the use of fewer bolts than hot driven rivets.

In 1956 work was begun at Lehigh University to establish the balanced design shear stress for A325 bolts in bearing-type connections. For balanced design the capacity of the cross sectional area of the bolts in shear should equal the tensile capacity of the net section of the main material. Tests of compact bolted joints(3) established the balanced design tension-shear ratio for A7 steel and A325 bolts at 1 to 1.10. In other words, equal ultimate strengths will result if the areas of the main material and bolts are chosen so that the average tensile stress on the net section and the average shear stress on the bolts are in the ratio of 1 to 1.10.

In design calculations for riveted and bolted work it is assumed that each fastener carries an equal share of the load. However, analytical work(4,5) has shown that in long connections, with all parts behaving elastically, the end fasteners carry a much larger proportion of the load than do the inner fasteners. But, experimental work with riveted connections(6) has shown that successive yielding of the outer rivets produces a redistribution of load so that at failure a more uniform distribution exists than the elastic analysis indicates. In the case of hot driven rivets, it usually has been assumed that the equalization of load is complete enough that no changes in design procedure need be specified for long connections.

The amount of redistribution of load depends upon the ability of the fasteners to undergo large shear deformations without failing. Because the properties of high strength bolts differ from those of rivets, it was deemed advisable to conduct tests of bolted joints in order to measure the ability of the bolts to redistribute load, and to determine if long bolted connections designed with a tension-shear ratio of 1 to 1.10 still provided a reasonable factor of safety. This work was started in 1958 and provided further background for the formulation of portions of the specifications of the Research Council on Riveted and Bolted Structural Joints.

## SCOPE OF PROGRAM

The research program described in this paper consisted of static tension tests of sixteen double shear butt joints with three to sixteen 7/8" diameter A325 bolts in line. Widths and thicknesses of the plate material were varied to maintain the tension-shear ratio at approximately 1 to 1.10. These test results are supplemental to the results reported in Ref. 3.

For comparison three riveted joints of the same type were tested, and tests of four bolted lap joints provided data on the behavior of bolts in single shear.

## BOLTED BUTT JOINTS

Dimensions - The sixteen double shear plate splices, are conveniently grouped in three categories as follows:

D Series, Part a - Constant 4 inch grip, varied width

Part b - Constant width, varied grip

Part c - Constant 8 inch grip, varied width

Nominal dimensions are tabulated in Fig. 1. With one exception (D13A) the pitch was held at 3.5 inches, hence the joint length and the number of bolts in line were directly related. The thick grips were attained by using either 4 or 8 plies of plate.

Plates - The one inch plates for the joints of Part a and c were cut from 24" x 1" universal mill plates rolled from the same heat. The plates of various thicknesses used in Part b joints were 10-1/4" wide universal mill plates from a different heat. Finished widths were achieved by rough burning and then machining to the final dimension. All plates were structural carbon steel meeting the requirements of ASTM A7[7] except for the yield point of the 1" plates. Results of standard coupon tests are summarized in Fig. 2.

Bolts - All bolts were 7/8" diameter with regular semifinished heads. Two quenched and tempered washers and a heavy semifinished hexagon nut completed the bolting assembly. The length of thread, 2" or 2.25", was standard for A325 bolts at the time this research was initiated but longer than the 1-1/2" thread length now specified.[8] The bolts satisfied the proof load (36.05 kips) and minimum tensile strength (53.15 kips) requirements of ASTM A325. Tension-elongation curves were established for each lot of bolts by pulling bolts in direct tension and by inducing the tension by tightening the nut.[3] These curves were used to determine the clamping force of bolts installed in the test joints. Shear tests[9] of single bolts were conducted to provide a comparison with the behavior of the bolts used in the joints. A summary of bolt properties is given in Fig. 3.

Fabrication - Fabrication was done by a local fabricator. The plies of material were clamped together and the four end holes were subdrilled and reamed. Fitted pins were inserted in these holes and the remaining holes were drilled through all plies of the solid material. Thus, all holes were aligned.

Fabrication instructions for preparation of the faying surfaces of the plate called for removal of grease with solvent and hand wire brushing to remove loose mill scale only. This procedure was followed except in the case of Joints D31 to 101 where an ambitious ironworker removed all mill scale with a power tool. The resulting surfaces were quite shiny and hereafter will be designated as semi-polished.

The lamina of plates and the fill plates were welded together near the ends where the testing machine was to grip the specimen.

Assembly - A field erection crew of the fabricator bolted the joints with an impact wrench using the turn-of-nut method[10]. The nuts on Joints D31 to D101 and D701 were tightened 1/2 turn from "snug". In the remaining joints, with grips greater than 5", the nuts were torqued through a 3/4 turn from "snug". The length of all bolts was measured before and after tightening so that the elongation might be used to determine the tension in the bolt from the tension-elongation curves as described in Ref. 3. In all cases bolt threads did not intercept the shearing planes of the test joints.

## BOLTED LAP JOINTS

Although specifications traditionally have assigned to rivets a single shear value equal to one half of that for double shear, it seemed advisable to have some experimental confirmation of this relationship for high strength bolts. Accordingly, four lap joints[11] were obtained by altering duplicate joints originally fabricated for the D Series, Part a. Because of this fabrication procedure three of the lap joints had more net section area than required by the tension-shear ratio of 1 to 1.10. These proportions were considered satisfactory in this case because they insured bolt failures in even the short joints. Nominal dimensions are shown in Fig. 4.

The plate was the same semi-polished kind used for joints D31 to D101. The A325 bolts had regular semifinished hexagon heads and were 5-1/2" long under head. The thread did not extend into the shearing plane of the joint. Quenched and tempered washers under the bolt head and the heavy semifinished nut were used. Bolt properties are listed in Fig. 3.

Because the inherent eccentricity of lap joints causes them to bend, an external bracing system was used to restrain the rotation of the connection during test[11]. The bracing system simulated the effect of the diaphragms often used by designers in conjunction with lap type joints.

## RIVETED BUTT JOINTS

Although some experimental work on long riveted joints had been done previously[6] it was not possible to adequately correlate that work with these tests on bolted joints. For that reason three double shear plate splices were fabricated and machine riveted with hot driven rivets. These connections were designed for the balanced design tension-shear ratio of 1 to 0.75 for A7 plate and rivets made of A141 steel. Nominal dimensions are shown in Fig. 5.

The 1" plate was the same as that used in the bolted joints. Tensile tests of the rivet bar stock gave a yield point of 36.1 ksi and a tensile strength of 57.7 ksi. Double shear tests of single rivets driven at the same time as the rivets in the joints gave an ultimate shear strength of 55.3 ksi.

## TESTING

Instrumentation - Instrumentation of the test specimens included SR-4 strain gages, a mechanical extensometer, and dial gages.

A few SR-4 strain gages on the plates outside of the connection proper were used to detect any eccentricity introduced by improper gripping and to pick up the onset of yielding of the gross section.

The elongation of each pitch was measured along the edges of the plates by a hand held slidebar extensometer. These readings were valid in both the elastic and plastic range. They were used primarily in the experimental confirmation of an analytical solution for the load partition and ultimate strength of bolted joints[12].

Dial gages (0.001") were used to measure the overall elongation of the connection, thereby providing a general picture of joint behavior including slip, yielding and bolt failures. Others were mounted at the ends of the main plates and the splice plates to record the relative movement or slip of the plates, and in the joints with laminated splice plates dial gages (0.0001") were used to detect relative movement of these lamina.

Testing Procedure - The joints were loaded in static tension by a 5000 kip universal testing machine using wedge grips. The testing procedure was similar to that described in Ref. 3 except that in most cases the testing machine was unloaded after one or several bolts had sheared. This permitted examination and disassembly of the intact connection, and provided valuable information that would not have been available had complete destruction of the joint been accomplished.

## TEST RESULTS

Bolted Butt Joints - The results of the tests of 16 bolted butt joints are tabulated in Figs. 6 and 7.

The slip of the joints with semi-polished faying surfaces was usually gradual and occurred in steps as load was applied as shown in Fig. 8, the test curve for joint D91. However, joints with tight mill scale surfaces experienced a sudden and complete slip accompanied by a loud noise. Fig. 9, the test curve for joint D901, illustrates this type of instantaneous slip. In viewing Figs. 8 and 9 it should be noted that the relative movement of the gage points, x, includes plate strains as well as the rigid body displacements of the inner and outer plates.

Joints D31 to D61 failed by excessive tensile stress on the net section. In two cases actual tearing occurred but in the other two it was possible to unload the testing machine after the ultimate load had been reached but before rupture occurred. Although bolt failures did not occur, calculations of average shear stress and examination of the intact joints indicated that bolt failure was imminent. Fig. 10 illustrates this by showing a sawed longitudinal section of joint D61. The high stress on the net section is revealed by the great elongation of the end holes, and the imminent bolt failure is indicated by the large shearing deformation of the end bolts. The inner bolts show practically no deformation. For comparison, Fig. 11 shows a sawed section of riveted joint DR71 with the end rivets sheared. The inner rivets show a measurable deformation.

The longer joints failed by the shearing of bolts. In all cases the load at which the first bolt sheared has been recorded as the ultimate load. However, in several instances the joint was reloaded following the first failure and then other bolts failed in a sequential fashion. The sequential failure of fasteners in long connections has been called "unbuttoning". In some cases a slightly greater "ultimate" load was required to cause the failure of a succeeding bolt as shown in Fig. 8.

<u>Bolted Lap Joints</u> - The results of the tests of bolted lap joints are summarized in Fig. 12. All lap joints slipped in a very gradual fashion, under increasing load, until the bolts came into bearing. Unlike the bolted butt joints no sudden movement or loud noise occurred. Therefore,

for purposes of computation, the slip load has been taken as the first marked change in slope of the load-elongation curve.

Failure of these joints occurred by shearing of the bolts. Because L10 was made with similar dimensions, from the same plate, and from bolts of a strength level comparable to those in butt joint D101, these two connections may be compared directly. L10 failed at 748 kips, almost exactly half of the 1506 kips failure load for D101.

Joints L5 and L7 cannot be compared directly to D51 and D71 because of the different tension-shear ratios.

Riveted Butt Joints - The results of the tests of the three riveted joints are tabulated in Fig. 13. The riveted joints failed by shearing of end rivets whereupon the tests were stopped and the joints kept intact for examination. The general behavior of these joints was very similar to that of the bolted joints but direct comparisons cannot be made because of the different tension-shear ratios.

It should be noted that riveted joints often experience slip despite the customary assumption that hot driven rivets fill the holes completely. In these tests slip amounted to about 0.02 inches, which is about 1/3 to 1/4 of the slip experienced by the bolted joints.

## ANALYSIS OF RESULTS

Ultimate Strength - The allowable shear stress for bolts used in bearing type connections is based upon the ultimate shearing strength of the connection. As explained in the introduction, a prime purpose of this test program was to see if long bearing-type connections designed with a tension-shear ratio of 1 to 1.10 have an adequate factor of safety against failure.

Four joints, D31 to D61 inclusive, failed by tearing of the plate through the net section thus corroborating previous work(3) that established the 1 to 1.10 ratio for balanced design of short bolted joints. However, the longer butt joints failed by shearing of the end fasteners due to the bolts' inability to deform sufficiently to equalize the load among all the fasteners. The longer the joint the lower the average shear stress at the time the first bolt failed.

Because bolts of several different lots and strengths were used, it is best to represent the average shearing stress at failure in non-dimensional form. Since the "premature" failure of end fasteners has often been called "unbuttoning" this non-dimensional quantity is called the unbuttoning factor. The unbuttoning factor is defined as the average bolt shear stress in the connection when the first bolt shears ($\tau_u$) divided by the ultimate shear strength of a single bolt of the same lot and of the same grip ($\tau_1$). Expressed mathematically,

$$U = \frac{\tau_u}{\tau_1} \tag{1}$$

The unbuttoning factor is an efficiency factor that measures the ability of the connection to develop the full strength of all the fasteners.

The unbuttoning factors for all the bolted butt joints (including compact joints(3)) with a tension-shear ratio of 1 to 1.10 are plotted in Fig. 14 using the length of the joint as an argument. Joints D31 to D61 are shown although they failed in tension. Their plotted points represent a lower bound for unbuttoning factors. The lap joints and riveted joints are also plotted although only L10 had a tension shear ratio of 1 to 1.10.

The plotted points define a fairly linear relationship between average shear stress and length of joint. This relation extends from U = 0.98 for joint L2 to U = 0.60 for joint D16, the longest connection tested. Theoretically, joint L2, being statically determinate, should have an unbuttoning factor of 1.0. Joint D16 shows that in joints of that length, 52.5 inches, the end bolts may be expected to fail when only 60% of the joint carrying capacity, based on the strength of an individual bolt, has been realized. The unbuttoning factors for lap joints L5 and L7, having tension-shear ratios of 1 to 1.29 and 1 to 1.55 respectively, fall above the general band of values. This is to be expected because the excess plate area results in less plate strain than in a balanced design joint and the connection approaches the "rigid plate" theory used by the designer when he assumes that each fastener carries an equal share of the load. It has been shown analytically(12) that the ultimate strength of a bolted joint is a function of these plate strains as well as of bolt deformation.

The riveted joints show that the same unbuttoning trend holds for rivets. Rivets made of A141 steel apparently possess a little more deformation capacity and thus are able to redistribute load to a greater extent than A325 bolts. However, it must be remembered that a fewer number of bolts in a shorter connection will sustain the same load as a given number of rivets when design is for bearing type connections.

In order to determine the minimum factor of safety against rupture it is convenient to convert the unbuttoning factor data into terms of the shear strength of bolts that just meet the minimum strength requirements of the A325 specification(8). As a result of previous work(9), the ultimate shear strength of a single bolt is taken conservatively as 68% of the tensile strength. Thus, for a minimum strength bolt in a connection of length, L

$$\tau_u = .68\ \sigma_{min}\ U_L \qquad (2)$$

where $U_L$ is taken from the mean line in Fig. 14. This information is plotted in Fig. 15 for 7/8" and 1" bolts with $\sigma_{min}$ = 115 ksi.

The horizontal lines in Fig. 15 represent the allowable shear stress for A325 bolts in bearing-type connections in buildings, namely: $F_V$ = 22 ksi for dead plus live load only, and $F_V$ = 1.33 x 22 = 29.3 ksi for dead plus live loads in combination with wind or earthquake loads. The vertical distance between the horizontal working stress lines and the sloping ultimate stress lines represents the margin of safety.

For very short joints under dead and live loading the factor of safety against a bolt failure is greater than 3, but at a length of 52.5 inches (16 in line at 3-1/2") this is reduced to 2.1. The minimum factor of safety against a tensile failure of the net section of A7 plate is $\frac{60}{20}$ = 3 if the allowable stress is 20 ksi and if net section efficiencies greater than 1 are not considered. The factor of safety against yielding of the net section is $\frac{33}{20}$ = 1.65.

Under wind loading conditions the factor of safety against a bolt failure in the 52.5" long joint is only 1.50 but even this is greater than the factor of safety against yielding of the net section, $\frac{33}{1.33 \times 20}$ = 1.24.

Effect of Pitch - The pitch was held constant at 3-1/2" (4 bolt diameters) for all joints except D13A where the pitch was 2-5/8", the minimum spacing of 3 diameters allowed by most specifications. Nevertheless, some interesting observations can be made concerning the effect of pitch by comparing D10, D13A and D13. D10 with 10 bolts at a 3-1/2" pitch and D13A with 13 bolts at 2-5/8" had the same overall length of 31-1/2". D13A and D13 had 13 bolts in line but their overall lengths were 31-1/2" and 42" respectively.

The efficiency of these connections may be compared by examining the unbuttoning factor plotted in Fig. 14. Joints D10 and D13A had almost identical unbuttoning factors of about 0.70. This points out that the tendency of end bolts to fail "prematurely" is a function of the length of joint rather than of the number of fasteners per se. One must be careful not to construe this to mean that the carrying capacity of the connection with 10 bolts in line is equal to that of the same length connection with 13 bolts in line, for although both members developed the same unit shearing stress, the latter carried 1.30 as much load.

Comparison of D13A and D13 shows unbuttoning factors of about 0.70 and 0.65 respectively. The shorter joint was more efficient and a 7% increase in load carrying capacity was achieved by decreasing the pitch from 4 to 3 diameters.

The conclusion might be reached that it is desirable to make all joints as short as possible. However, other research(14) has shown that to obtain the greatest strength of the connected material this may not be the best procedure. In some types of connections, such as bridge hangers riveted or bolted to gusset plates, a longer connection may be necessary for the stress in an unconnected web to be developed by fasteners through the flanges and gusset plates because of the phenomenon known as shear lag.

Long Grips - Specifications(13) for riveted construction require that rivets carrying calculated stress shall have their number increased one percent for each 1/16" that the grip exceeds 5 rivet diameters. Nothing in these tests indicated a need for a similar provision for high strength bolts even though grips ran higher than 9 bolt diameters.

Joints D101, D1001 and D10 had 10 bolts in line and grips of 4", 6-3/4" and 8" respectively. The unbuttoning factors plotted in Fig. 14 show no substantial spread and indicate that variation of grip is of negligible importance in determining the ultimate strength of bolted joints.

Slip Resistance - This test program was designed primarily to provide information about the ultimate strength of long bolted joints, but valuable information about slip also was obtained.

Resistance to slip is dependent on the frictional forces that can be developed on the faying surfaces. Although the bolts are not actually acting in shear, it is convenient to regulate the design of friction type joints by specifying a tension-shear ratio(2) or an allowable shear stress(13) for the bolts. The bar graph in Fig. 16 shows the average shear stress per bolt existing at the time major slip occurred. The two horizontal lines cutting across the bars represent the allowable unit shear stresses for friction type joints in buildings(13), i.e. under dead and live loads, $F_v$ = 15 ksi, and under conditions of dead and live load plus wind or seismic loading, $F_v$ = 1.33 x 15 = 20 ksi.

The high clamping force produced by the turn-of-nut method together with tight mill scale faying surfaces produced slip resistance in excess of the specified shear stresses. The joints having the semi-polished faying surfaces with practically no mill scale satisfied the working stress for dead and live load. However, two of them slipped at an average stress less than the increased stress permitted for wind loading conditions. This illustrates the importance of the faying surface in friction type connections.

A better way of expressing the role of the faying surface alone is by means of the slip coefficient, $K_s$, given by the expression

$$K_s = \frac{P_s}{m\ n\ \overline{T}_i} \qquad (3)$$

where $P_s$ = major slip load

$m$ = number of bolt shear planes

$n$ = number of bolts

$\overline{T}_i$ = average initial bolt tension (or clamping force)

In making this computation $\overline{T}_i$ has been obtained from a torqued tension bolt calibration curve using the average elongation of all bolts in a joint(3). The values of $K_s$ are tabulated in Figs. 6, 7 and 12, and the high and low values are also shown on the bar graph in Fig. 16.

The slip coefficient for tight mill scale surfaces ranged from 0.34 to 0.57 with a mean value of 0.46. This mean value coincides with the mean value of slip coefficient for mill scale surfaces as reported previously for compact bolted joints(3). For the semi-polished surfaces the range was from 0.22 to 0.36 with a mean value of 0.29.

In addition to surface condition, frictional resistance depends upon the bolt clamping force. The mean value of $\overline{T}_i$ for all 7/8" bolts used in these test joints was 47.8 kips. Thus, for the type of bolt used, the turn-of-nut method produced tensions equal to 132.5% of the proof load--the specified minimum tension(2). If nuts are tightened by the calibrated impact wrench method it is likely that the wrench will have to be set somewhat above the proof load in order to insure that all bolts reach the specified minimum tension.

The inter-relation of slip coefficient, bolt tension, the allowable shear stress and the cross-sectional area of the bolt is given by the following equation for the factor of safety against slip.

$$F.S. = \frac{\text{Total Slip Resistance}}{\text{Design Slip Resistance}} = \frac{K_s \; m \; n \; \overline{T_i}}{F_v \; m \; n \; A_b} \qquad (4)$$

To illustrate the range of the factor of safety against slip that exists in friction-type joints that are designed for a specific allowable shear stress, Fig. 17 has been plotted from Eq. 4 for $F_v = 15$ ksi and for 7/8" bolts with $A_b = 0.6$ sq. in. The values of $K_s$, shown as sloping lines, represent reasonable lower limits for the surface conditions found in these tests. The value of $\overline{T_i}$ represented by a solid horizontal line at 1.30 PL represents the value of initial bolt tension obtained by the turn-of-nut method in these tests. The dashed horizontal line indicates the proof load, the specified minimum tension. Fig. 18 has been plotted from Eq. 4 for $F_v$ increased 33% as permitted for conditions of dead and live loads plus wind loading.

These two figures illustrate the importance of making sure that mill scale faying surfaces remain intact and unpainted in friction-type connections. Furthermore, they point out the desirability of achieving a high clamping force by a tightening procedure that will develop high initial stress in the bolt. The combination of tight mill scale faying surfaces and bolts tightened by the turn-of-nut method shows a margin of safety against slip, even under wind conditions, of 37%. If all bolts in a connection are tightened just to the proof load the factor of safety against slip may approach unity under wind conditions.

## SUMMARY AND CONCLUSIONS

The following conclusions are based on the results of 20 tests of large bolted joints conducted at Lehigh University and described herein. Certain of these observations are reinforced by the results of previous tests described in Ref. 3. The joints, consisting of plate splices of A7 steel connected with A325 bolts, were proportioned for a balanced design at ultimate strength with a tension-shear ratio of 1 to 1.10. The principal item under investigation was the effect of the length of the joint upon the ultimate strength of the connection. Valuable information also was obtained on the turn-of-nut method, the effect of surface condition on slip resistance, single shear versus double shear, the effect of pitch, and long grip bolts. Tests of several riveted joints permitted a comparison of the behavior of riveted and bolted connections of similar geometry.

### Bearing-Type Joints

1. Joints of A7 steel and A325 bolts proportioned at a tension-shear ratio of 1 to 1.10 produced a balanced design at ultimate strength for connections about 16" or less in length. In longer connections the differential strains in the connected material caused the end bolts to shear before all bolts could develop their full shearing strength. This "premature" failure of end bolts is called unbuttoning.

2. The "unbuttoning factor" (U) is used to measure the efficiency of a connection in utilizing the full shearing strength of all the connectors and is defined as: the average shear stress when the first bolt in a connection fails divided by the ultimate shear strength of a single bolt. These tests show that U decreases in a fairly uniform manner as the length of the connection increases.

Theoretically U should be 1.0 for a statically determinate connection with two fasteners in line and the test of one lap joint (L2) corroborated this with U of 0.98. The tests show that for short compact joints with 3 to 5 bolts in line and lengths up to about 16" the unbuttoning factor is about 0.85. For the connection with 16 bolts at a pitch of 3-1/2", an overall length of 52.5", U decreased to about 0.60 (Fig. 14). In other words a connection of that length utilizes only 60% of the full capacity of all the bolts before an end bolt shears. However, since the allowable shear stress for A325 bolts is based upon the performance of the compact joints[3] it should be noted that the decrease in the unbuttoning factor from 0.85 to 0.60 is a more pertinent relationship.

3. The unbuttoning phenomenon can be expressed in terms of the average shear stress in a connection using A325 bolts that just meet the strength requirements of the ASTM specification (See Fig. 15). The same downward trend occurs as the length of the joint is increased. If a constant value of allowable shear stress is used for design purposes, a reduced factor of safety occurs for longer joints.

These tests show that if the allowable shear stress for bearing-type connections is taken as $F_v = 22$ ksi the factor of safety against failure is 3.0 or greater for joints less than 16" long. However, when the joint becomes 52.5" long the factor of safety for 7/8" or 1" bolts is 2.1. Under wind loading conditions with $F_v = 1.33 \times 22$ ksi this factor is 1.5.

4. The unbuttoning type behavior also occurs in riveted joints. Several tests indicate that A141 rivets possess better deformation capacity than A325 bolts and therefore somewhat better redistribution

of load among the fasteners takes place. Nevertheless a reduction in the factor of safety also occurs as riveted joints become longer. For years American riveted construction specifications have tolerated this reduction in the factor of safety without comment.

These tests showed that for a riveted joint 42" long (13 in line at 3-1/2") the unbuttoning factor was 0.75 whereas a bolted joint of the same length had an unbuttoning factor of 0.68. However, when a bearing-type connection can be used the greater strength of high strength bolts results in a shorter bolted connection, thus nullifying this apparent advantage of rivets.

5. Bolts used in single shear have one-half the load carrying capacity of comparable bolts in double shear provided the shear planes act through the shank of the bolt. Bending of lap type joints should be restrained by diaphragms or other suitable bracing.

6. Pitch distance was held at 4 diameters in all but one case where it was reduced to 3 diameters - the minimum permitted by some specifications. However, comparison of three connections (10 bolts at 3-1/2", 13 bolts at 2-5/8" and 13 bolts at 3-1/2") shows that unbuttoning of end fasteners depends upon the length of the joint and not on the number of bolts in line per se.

7. Specifications for riveted construction[13] require that the number of rivets be increased 1% for each 1/16" that the grip exceeds 5 diameters. Although bolts with grips up to 9 diameters were used in these test specimens the tests indicated no need for a similar provision for high strength bolts.

Friction-Type Joints

1. Bolts tightened by the turn-of-nut method[10] consistently had tensions of approximately 1.30 times the proof load of the bolt thus easily meeting the requirements of specifications[2,13] which call for a minimum tension equal to the proof load.

2. No detrimental shear behavior of a connection was experienced because of this high initial tension in the bolts.

3. Joints with tight mill scale faying surfaces slipped into bearing at values of average shear stress in excess of working stresses for friction-type connections, namely, $F_v$ = 15 ksi for dead and live loads and $F_v$ = 20 ksi for dead and live loads plus wind load[13].

4. Joints with semi-polished faying surfaces obtained by removing all mill scale with a power tool slipped at values of average shear stress in excess of 15 ksi but less than 20 ksi.

5. These tests indicated $K_s$ = 0.35 as a reasonable lower limit for the coefficient of slip for tight mill scale faying surfaces for A7 steel and $K_s$ = 0.20 as a lower limit for semi-polished surfaces with mill scale removed.

6. These tests indicated that for connections with tight mill scale surfaces and bolted by the turn-of-nut method the factor of safety against slip under conditions of dead and live loads plus wind loading may be expected to be about 1.37.

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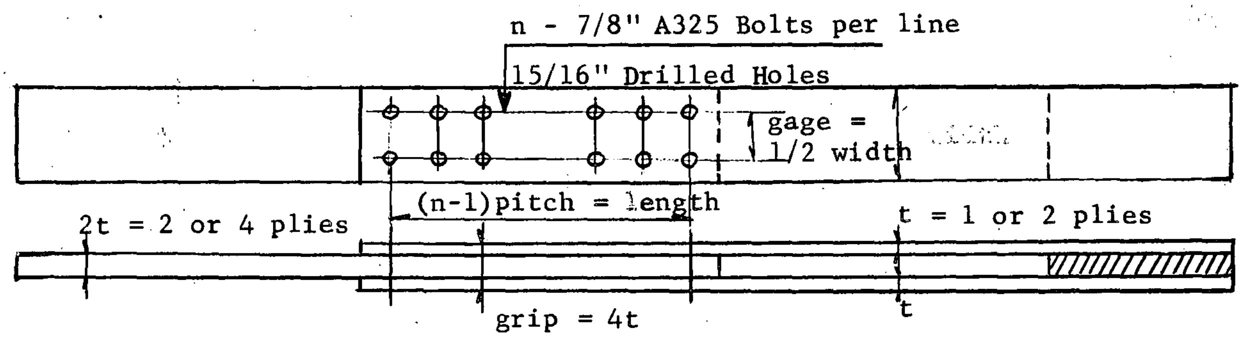



| Mark | n | Pitch (in.) | Length (in.) | Width (in.) | Grip (in.) | Gage / Hole Dia. | Shear Area sq. in. | Net Area sq. in. | T:S |
|---|---|---|---|---|---|---|---|---|---|
| **D-Series - Part a** (Varied - Width) | | | | | | | | | |
| D31 | 3 | 3.50 | 7.00 | 5.84 | 4.00 | 3.11 | 7.21 | 7.93 | 1:1.10 |
| D41 | 4 | 3.50 | 10.50 | 7.16 | 4.00 | 3.82 | 9.62 | 10.58 | 1:1.10 |
| D51 | 5 | 3.50 | 14.00 | 8.48 | 4.00 | 4.52 | 12.02 | 13.22 | 1:1.10 |
| D61 | 6 | 3.50 | 17.50 | 9.80 | 4.00 | 5.23 | 14.42 | 15.86 | 1:1.10 |
| D71 | 7 | 3.50 | 21.00 | 11.13 | 4.00 | 5.93 | 16.83 | 18.51 | 1:1.10 |
| D81 | 8 | 3.50 | 24.50 | 12.45 | 4.00 | 6.63 | 19.23 | 21.15 | 1:1.10 |
| D91 | 9 | 3.50 | 28.00 | 13.78 | 4.00 | 7.34 | 21.64 | 23.80 | 1:1.10 |
| D101 | 10 | 3.50 | 31.50 | 15.10 | 4.00 | 8.04 | 24.04 | 26.44 | 1:1.10 |
| **D-Series - Part b** (Varied - Grip) | | | | | | | | | |
| D701 | 7 | 3.50 | 21.00 | 9.88 | 4.75 | 5.27 | 16.83 | 19.00 | 1:1.13 |
| D801 | 8 | 3.50 | 24.50 | 9.88 | 5.25 | 5.27 | 19.23 | 21.00 | 1:1.09 |
| D901 | 9 | 3.50 | 28.00 | 9.88 | 6.00 | 5.27 | 21.64 | 24.00 | 1:1.11 |
| D1001 | 10 | 3.50 | 31.50 | 9.88 | 6.75 | 5.27 | 24.04 | 27.00 | 1:1.12 |
| **D-Series - Part c** (Varied - Width) | | | | | | | | | |
| D10 | 10 | 3.50 | 31.50 | 8.48 | 8.00 | 4.52 | 24.04 | 26.44 | 1:1.10 |
| D13A | 13 | 2.63 | 31.50 | 10.47 | 8.00 | 5.58 | 31.25 | 34.38 | 1:1.10 |
| D13 | 13 | 3.50 | 42.00 | 10.47 | 8.00 | 5.58 | 31.25 | 34.38 | 1:1.10 |
| D16 | 16 | 3.50 | 52.50 | 12.45 | 8.00 | 6.63 | 38.46 | 42.31 | 1:1.10 |

FIG. 1 NOMINAL DIMENSIONS OF BOLTED BUTT JOINTS

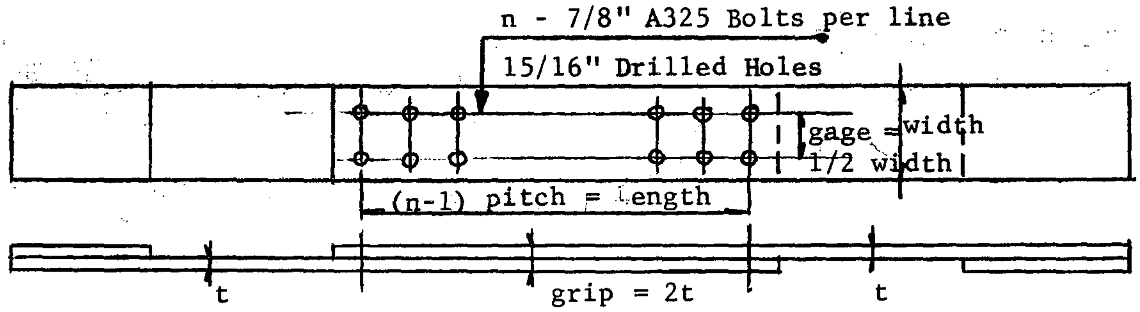
| Test Series | No. of Coupons | Static Yield Point*, ksi | | Yield Point*, ksi | | Ult. Tensile Strength, ksi | |
|---|---|---|---|---|---|---|---|
| | | Mean | Std. Dev. | Mean | Std. Dev. | Mean | Std. Dev. |
| Parts a & c | 22 | 28.2 | 1.35 | 30.0 | 1.29 | 60.0 | 0.80 |
| Part b | 16 | 33.6 | 1.71 | 35.7 | 1.94 | 64.3 | 1.08 |

*Taken at .005 strain because of rising yield zone

FIG. 2 PROPERTIES OF PLATE

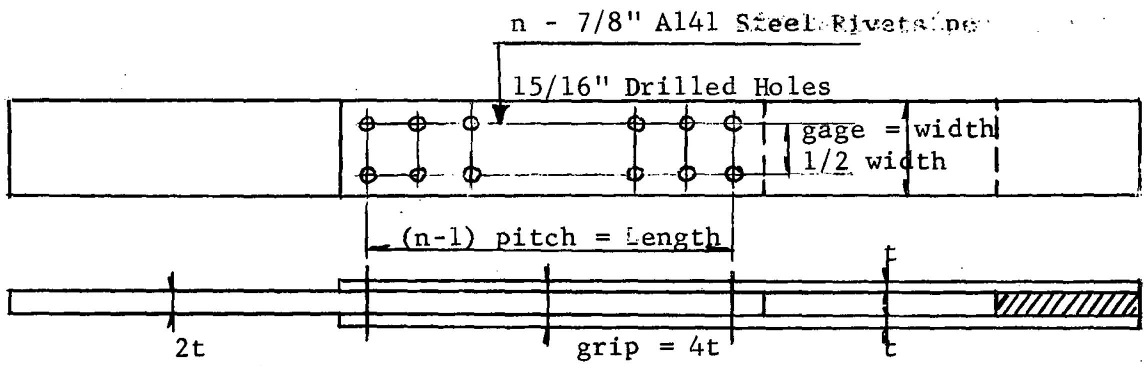
| Used in Joints | Bolt Lot Mark | Length Head inches | Direct Tensile Strength, kips | | Torqued Tensile Strength, kips | | Shear Strength, $\tau_1$, ksi | | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | Mean | Std. Dev. | Mean | Std. Dev. | Mean | Std. Dev. | |
| D31-D101 | D | 5.5 | 56.9 | 0.55 | 51.1 | 0.77 | 85.3 | - | 2" Rolled Thread |
| D701-D1001 | T, U V, W | 6.5, 7.0 7.5, 8.5 | 54.8 | 0.94 | 51.3 | 0.96 | 93.2 | 2.01 | 2.25" Cut Thread, Same heat treat. |
| D10-D16 | C | 9.5 | 53.4 | 0.12 | 45.8 | 0.51 | 90.7 | 2.35 | 2.25" Rolled Thread |
| L2-L10 | L | 3.5 | 55.2 | 0.97 | 49.2 | 1.10 | 83.4 | 1.33 | 2" Rolled Thread |

FIG. 3 PROPERTIES OF BOLTS




| Mark | n | Pitch (in.) | Length (in.) | Width (in.) | Grip (in.) | Gage / Hole Dia. | Shear Area sq. in. | Net Area sq. in. | T:S |
|---|---|---|---|---|---|---|---|---|---|
| L2 | 2 | 3.50 | 3.50 | 5.84 | 2 | 3.11 | 2.40 | 3.96 | 1:1.65 |
| L5 | 5 | 3.50 | 14.00 | 9.80 | 2 | 5.23 | 6.01 | 7.92 | 1:1.32 |
| L7 | 7 | 3.50 | 21.00 | 15.10 | 2 | 8.06 | 8.41 | 13.20 | 1:1.57 |
| L10 | 10 | 3.50 | 31.50 | 15.10 | 2 | 8.06 | 12.02 | 13.20 | 1:1.10 |

FIG. 4 NOMINAL DIMENSIONS OF BOLTED LAP JOINTS




| Mark | n | Pitch (in.) | Length (in.) | Width (in.) | Grip (in.) | Gage / Hole Dia. | Shear Area sq. in. | Net Area sq. in. | T:S |
|---|---|---|---|---|---|---|---|---|---|
| DR71 | 7 | 3.50 | 21.0 | 8.48 | 4 | 4.52 | 16.83 | 13.21 | 1:0.78 |
| DR101 | 10 | 3.50 | 31.5 | 11.12 | 4 | 5.94 | 24.04 | 18.49 | 1:0.77 |
| DR131 | 13 | 3.50 | 42.0 | 13.78 | 4 | 7.35 | 31.25 | 23.81 | 1:0.76 |

FIG. 5 NOMINAL DIMENSIONS OF RIVETED BUTT JOINTS

| See Fig. 1 for nominal dimensions and pattern of joints | UNITS | D31 | D41 | D51 | D61 | D71 | D81 | D91 | D101 |
|---|---|---|---|---|---|---|---|---|---|
| **BOLTS** | | | | | | | | | |
| No. in line | | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| No. of 7/8" A325 Bolts | | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 |
| Nominal shear area (= Actual) | sq. in. | 7.21 | 9.62 | 12.02 | 14.42 | 16.83 | 19.23 | 21.64 | 24.04 |
| **PLATE** | | | | | | | | | |
| Actual (measured) width | in. | 5.83 | 7.16 | 8.47 | 9.77 | 11.13 | 12.42 | 13.76 | 14.99 |
| Actual (measured) thickness | in. | 1.984 | 2.000 | 1.996 | 1.968 | 1.964 | 1.992 | 1.996 | 1.986 |
| Actual gross area | sq. in. | 11.57 | 14.32 | 16.91 | 19.23 | 21.86 | 24.74 | 27.46 | 29.77 |
| Actual net area | sq. in. | 7.85 | 10.57 | 13.17 | 15.54 | 18.18 | 21.00 | 23.72 | 26.05 |
| **T/S RATIO ($A_s/A_n$)** | | | | | | | | | |
| Nominal | | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 |
| Actual | | 1:1.09 | 1:1.10 | 1:1.10 | 1:1.08 | 1:1.08 | 1:1.09 | 1:1.10 | 1:1.08 |
| **WORKING LOAD** | | | | | | | | | |
| Friction Type Joint, $F_V$ = 15 ksi | kips | 108 | 144 | 180 | 216 | 252 | 288 | 325 | 361 |
| Bearing Type Joint, $F_V$ = 22 ksi | kips | 159 | 212 | 264 | 317 | 370 | 423 | 476 | 529 |
| **SLIP LOAD** | kips | 176 | 234 | 348 | 338 | 358 | 560 | 405 | 568 |
| Bolt shear stress | ksi | 24.4 | 24.3 | 29.0 | 23.4 | 21.3 | 29.1 | 18.7 | 23.6 |
| Avg. ext. of bolts | in. | .0249 | .0267 | .0324 | .0313 | .0310 | .0329 | .0368 | .0381 |
| Clamping force per bolt | kips | 49.0 | 49.4 | 49.9 | 49.8 | 49.8 | 50.0 | 50.4 | 50.5 |
| Slip coefficient | | 0.299 | 0.296 | 0.349 | 0.283 | 0.257 | 0.350 | 0.223 | 0.281 |
| **TYPE OF FAILURE** | | plate | plate | plate | plate | all bolts sheared | 1 bolt sheared | 1 bolt sheared | 1 bolt sheared |
| **LOAD AT FAILURE** | kips | 514 | 690 | 850 | 994 | 1126 | 1282 | 1358 | 1506 |
| Bolt shear stress | ksi | 71.3 | 71.7 | 70.7 | 68.9 | 66.9 | 66.7 | 62.8 | 62.6 |
| **EFFICIENCY** | | | | | | | | | |
| g/d | | 3.11 | 3.82 | 4.52 | 5.23 | | | | |
| Theoretical | | 67.8 | 73.8 | 77.9 | 80.8 | | | | |
| Gross section (actual) | | 74.7 | 79.5 | 83.8 | 87.2 | | | | |
| Net section (actual) | | 110.0 | 107.7 | 107.6 | 107.9 | | | | |

FIG 6 TEST RESULTS - BOLTED BUTT JOINTS

| | UNITS | D701 | D801 | D901 | D1001 | D10 | D13A | D13 | D16 |
|---|---|---|---|---|---|---|---|---|---|
| See Fig. 1 for nominal dimensions and pattern of joints | | | | | | | | | |
| BOLTS | | | | | | | | | |
| No. in line | | 7 | 8 | 9 | 10 | 10 | 13 | 13 | 16 |
| No. of 7/8" A325 Bolts | | 14 | 16 | 18 | 20 | 20 | 26 | 26 | 32 |
| Nominal shear area (=Actual) | sq. in. | 16.83 | 19.23 | 21.64 | 24.04 | 24.04 | 31.25 | 31.25 | 38.46 |
| PLATE | | | | | | | | | |
| Actual (measured) width | in. | 9.863 | 9.878 | 9.875 | 9.869 | 8.462 | 10.459 | 10.463 | 12.40 |
| Actual (measured) thickness | in. | 2.410 | 2.632 | 3.04 | 3.354 | 3.984 | 3.984 | 3.984 | 3.984 |
| Actual gross area | sq. in. | 23.77 | 26.00 | 30.02 | 33.10 | 33.71 | 41.67 | 41.68 | 49.40 |
| Actual net area | sq. in. | 19.25 | 21.06 | 24.32 | 26.81 | 26.24 | 34.20 | 34.21 | 41.93 |
| Grip (nominal) | in. | 4.75 | 5.25 | 6.00 | 6.75 | 8.00 | 8.00 | 8.00 | 8.00 |
| T/S RATIO ($A_s/A_n$) | | | | | | | | | |
| Nominal | | 1:1.13 | 1:1.09 | 1:1.11 | 1:1.12 | 1:1.10 | 1:1.10 | 1:1.10 | 1:1.10 |
| Actual | | 1:1.14 | 1:1.10 | 1:1.12 | 1:1.12 | 1:1.09 | 1:1.09 | 1:1.09 | 1:1.09 |
| WORKING LOAD | | | | | | | | | |
| Friction Type Joint, $F_v$ = 15 ksi | kips | 252 | 288 | 325 | 361 | 361 | 469 | 469 | 577 |
| Bearing Type Joint, $F_v$ = 22 ksi | kips | 370 | 423 | 476 | 529 | 529 | 688 | 688 | 846 |
| SLIP LOAD | kips | 720 | 610 | 865 | 1010 | 900 | 792 | 960 | 1242 |
| Bolt shear stress | ksi | 42.8 | 31.7 | 40.0 | 42.0 | 37.4 | 25.3 | 30.7 | 32.3 |
| Avg. ext. of bolts | in. | .0406 | .0549 | .0473 | .0595 | .0762 | .0702 | .0767 | .0671 |
| Clamping force per bolt | kips | 45.5 | 48.0 | 46.5 | 48.5 | 45.8 | 45.3 | 45.9 | 45.0 |
| Slip coefficient | | 0.565 | 0.397 | 0.517 | 0.521 | 0.491 | 0.336 | 0.402 | 0.431 |
| TYPE OF FAILURE | | 2 bolts sheared | 1 bolt sheared | 1 bolt sheared | 2 bolts sheared | 1 bolt sheared | 1 bolt sheared | 1 bolt sheared | 1 bolt sheared |
| LOAD AT FAILURE | kips | 1213 | 1313 | 1497 | 1667 | 1544 | 1988 | 1854 | 2085 |
| Bolt shear stress | ksi | 72.1 | 68.3 | 69.2 | 69.3 | 64.2 | 63.6 | 59.3 | 54.2 |

FIG. 7 TEST RESULTS - BOLTED BUTT JOINTS

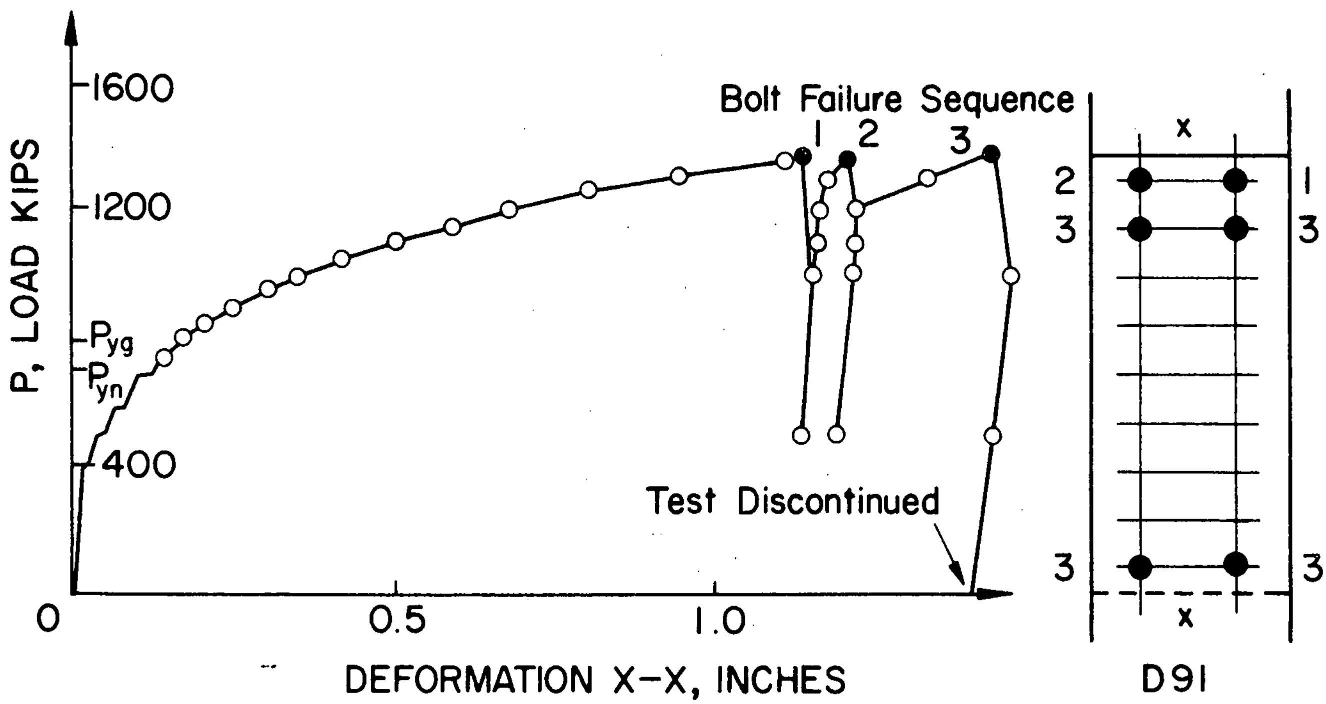



FIG. 8 LOAD-ELONGATION CURVE - JOINT D91

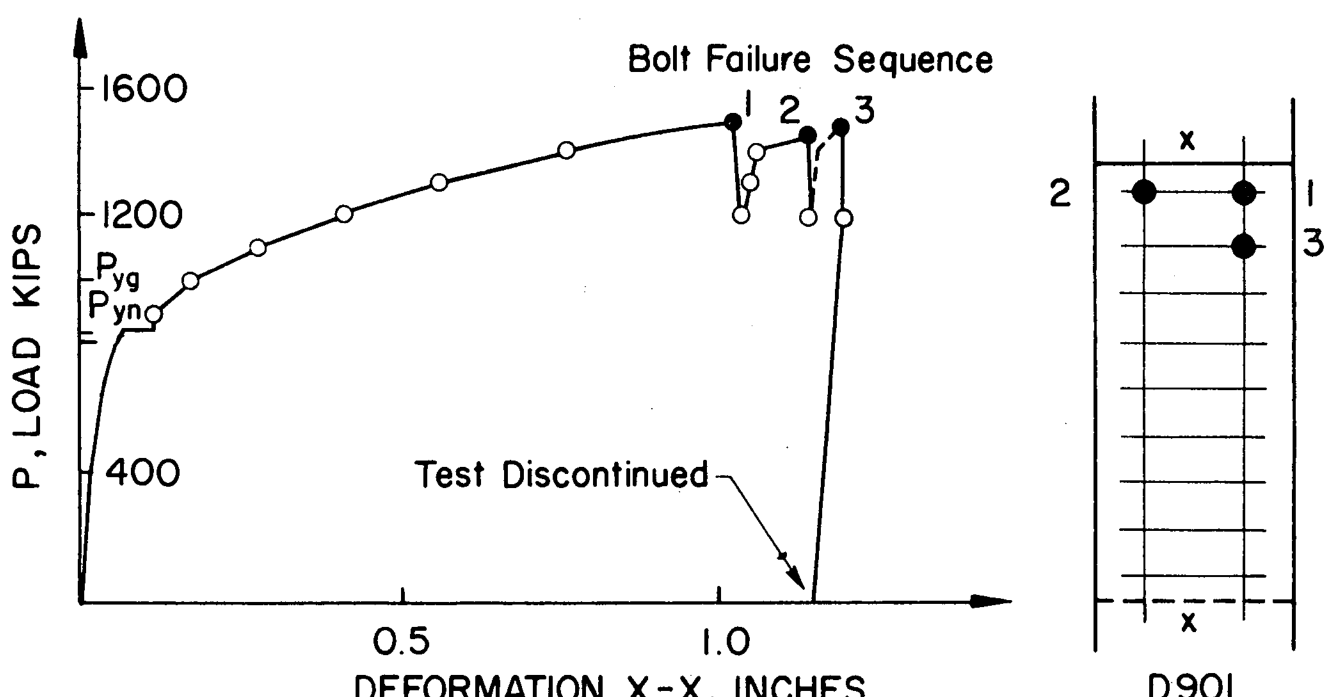



FIG. 9 LOAD-ELONGATION CURVE - JOINT D901

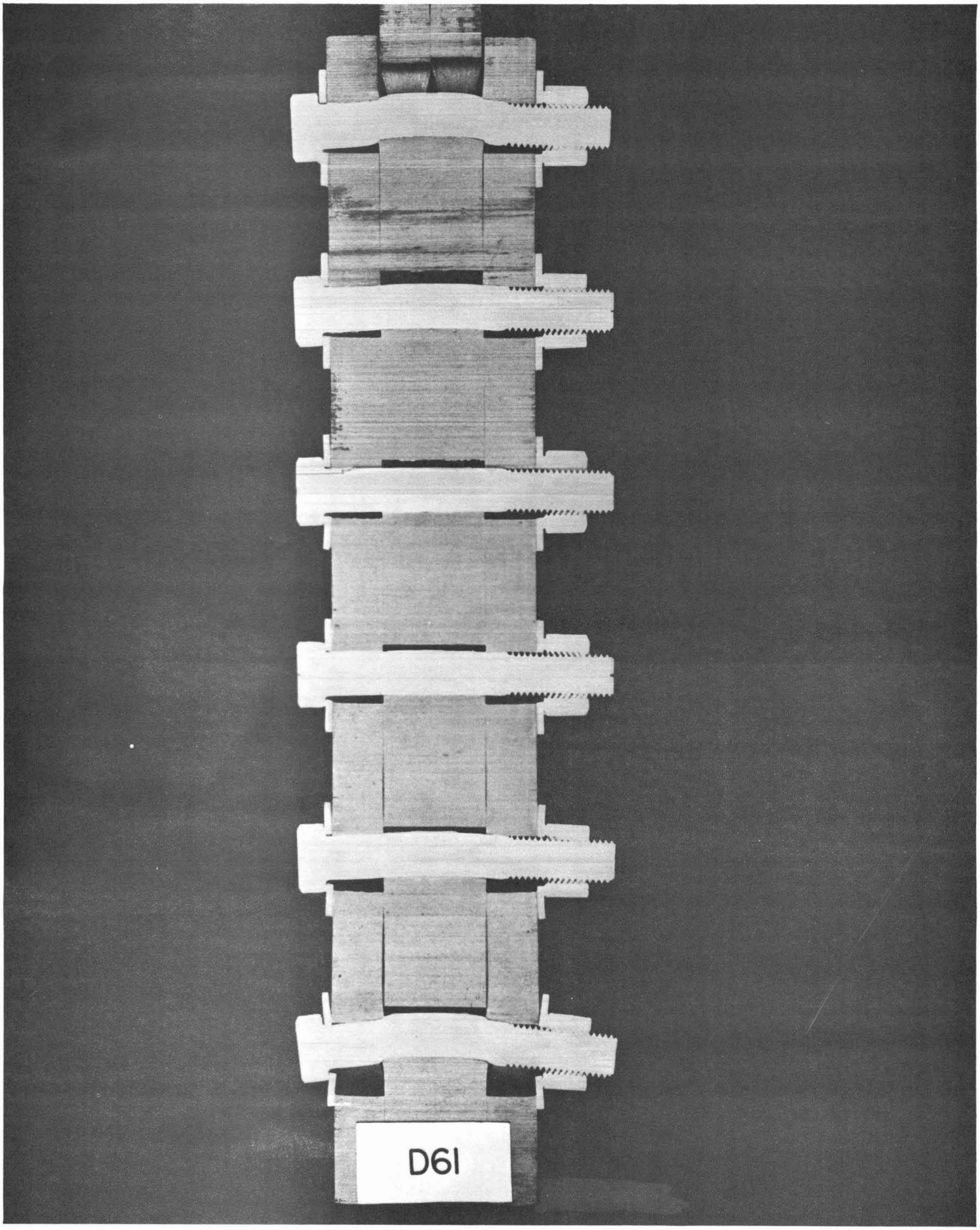



FIG. 10 – SAWED SECTION – JOINT D61

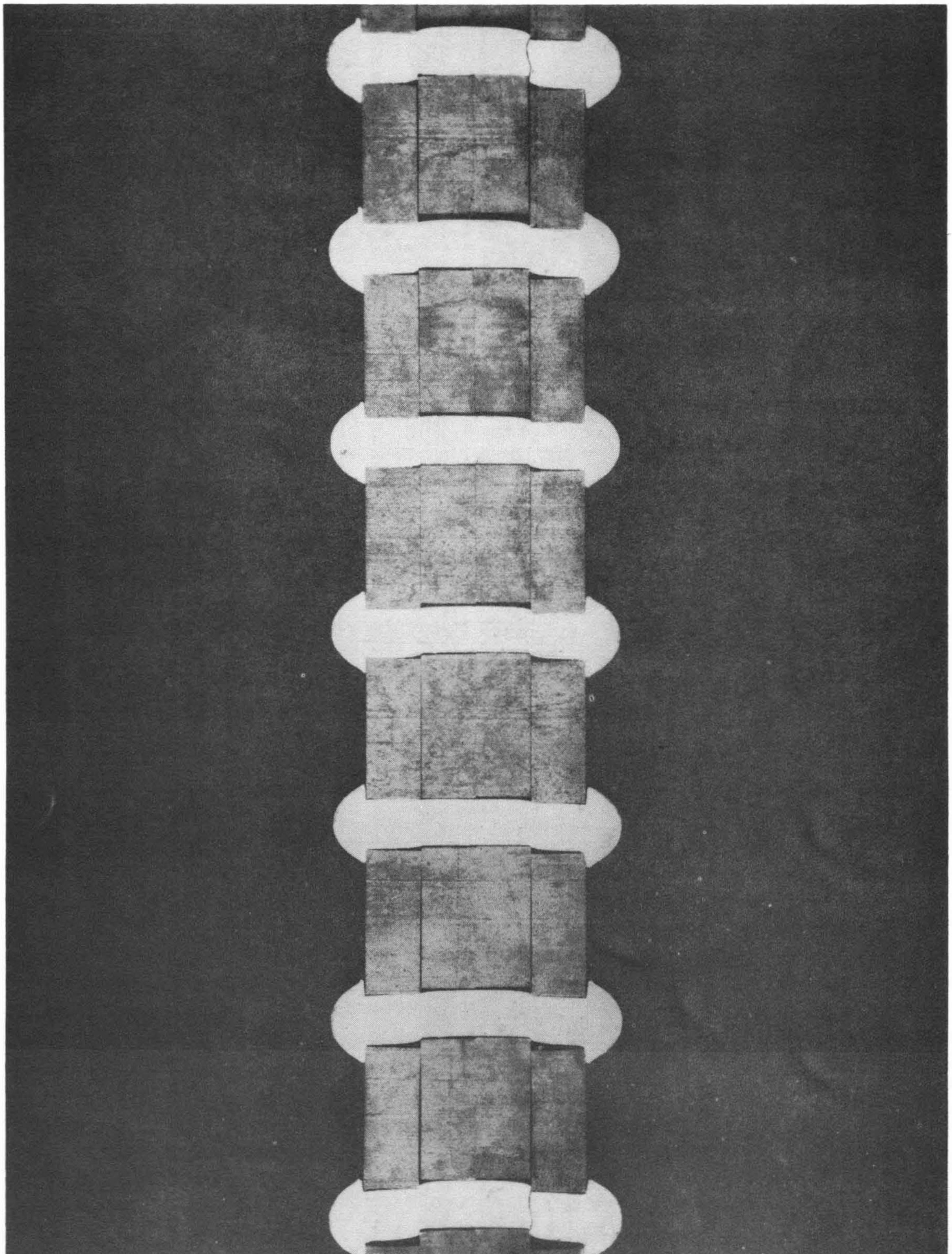


FIG. 11 - SAWED SECTION - JOINT DR71

| | UNITS | L2 | L5 | L7 | L10 |
|---|---|---|---|---|---|
| See Fig. 4 for nominal dimensions and pattern of joints | | | | | |
| BOLTS | | | | | |
| No. in line | | 2 | 5 | 7 | 10 |
| No. of 7/8" A325 Bolts | | 4 | 10 | 14 | 20 |
| Nominal shear area ( = Actual) | sq. in. | 2.40 | 6.01 | 8.41 | 12.02 |
| PLATE | | | | | |
| Actual (measured) width | in. | 5.80 | 9.79 | 15.02 | 15.00 |
| Actual (measured) thickness | in. | 0.993 | 0.984 | 0.993 | 0.993 |
| Actual gross area | sq. in. | 5.76 | 9.63 | 14.92 | 14.90 |
| Actual net area | sq. in. | 3.88 | 7.75 | 13.04 | 13.02 |
| T/S RATIO ($A_s/A_n$) | | | | | |
| Nominal | | 1:1.65 | 1:1.32 | 1:1.57 | 1:1.10 |
| Actual | | 1:1.62 | 1:1.29 | 1:1.55 | 1:1.08 |
| WORKING LOAD | | | | | |
| Friction Type Joint, $F_v$ = 15 ksi | kips | 36 | 90 | 126 | 180 |
| Bearing Type Joint, $F_v$ = 22 ksi | kips | 53 | 132 | 185 | 264 |
| SLIP LOAD | kips | 62 | 107 | 195 | 350 |
| Bolt shear stress | ksi | 25.8 | 17.8 | 23.2 | 29.1 |
| Avg. ext. of bolts | in. | .0428 | .0409 | .0445 | .0435 |
| Clamping force per bolt | kips | 48.6 | 48.4 | 48.7 | 48.6 |
| Slip coefficient | | .319 | .221 | .286 | .360 |
| TYPE OF FAILURE | | all bolts sheared | 1 bolt sheared | all bolts sheared | 1 bolt sheared |
| LOAD AT FAILURE | kips | 197 | 446 | 640 | 748 |
| Bolt shear stress | ksi | 82.1 | 74.2 | 76.1 | 62.2 |

FIG. 12 TEST RESULTS - BOLTED LAP JOINTS

| | UNITS | DR71 | DR101 | DR131 |
|---|---|---|---|---|
| See Fig. 5 for nominal dimensions and pattern of joints | | | | |
| **RIVETS** | | | | |
| No. in line | | 7 | 10 | 13 |
| No. of 7/8" A141 rivets | | 14 | 20 | 26 |
| Nominal shear area (= Actual) | sq. in. | 16.83 | 24.04 | 31.25 |
| **PLATE** | | | | |
| Actual (measured) width | in. | 8.47 | 11.11 | 13.79 |
| Actual (measured) thickness | in. | 1.00 | 1.00 | 1.00 |
| Actual gross area | sq. in. | 16.93 | 22.22 | 27.48 |
| Actual net area | sq. in. | 13.18 | 18.47 | 23.73 |
| **T/S RATIO** ($A_s/A_n$) | | | | |
| Nominal | | 1:0.78 | 1:0.77 | 1:0.76 |
| Actual | | 1:0.78 | 1:0.77 | 1:0.76 |
| **WORKING LOAD** | | | | |
| Bearing Type Joint, $F_v$= 15 ksi | kips | 252 | 361 | 469 |
| **SLIP LOAD** | kips | 444 | 518 | 830 |
| Rivet shear stress | ksi | 26.4 | 21.6 | 26.6 |
| Tension, net section | ksi | 33.6 | 28.0 | 34.9 |
| **TYPE OF FAILURE** | | rivet | rivet | rivet |
| **LOAD AT FAILURE** | kips | 738 | 942 | 1216 |
| Rivet shear stress | ksi | 43.9 | 39.2 | 38.9 |
| Tension, net section | ksi | 55.9 | 51.0 | 51.0 |

FIG. 13 TEST RESULTS - RIVETED BUTT JOINTS

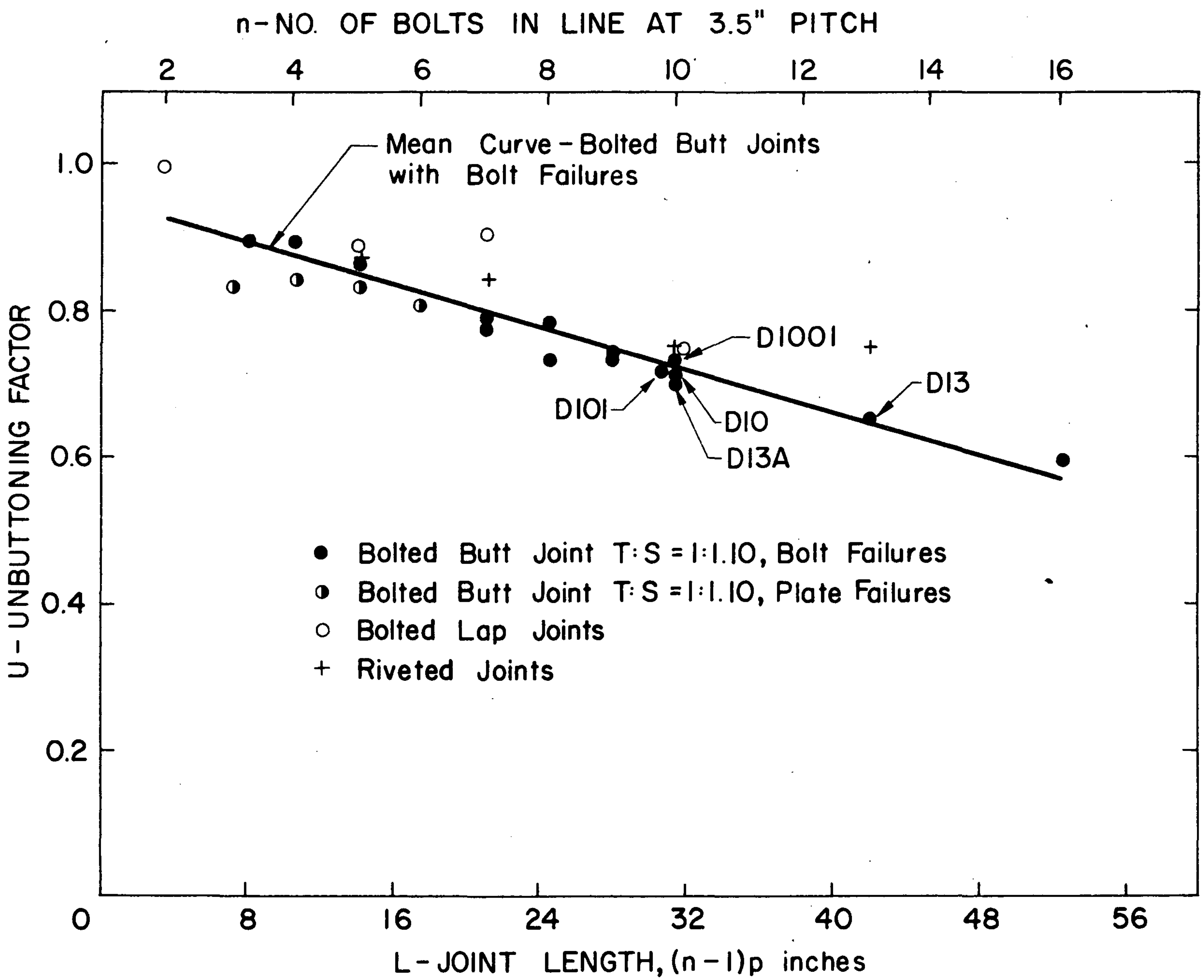



FIG. 14 UNBUTTONING FACTOR

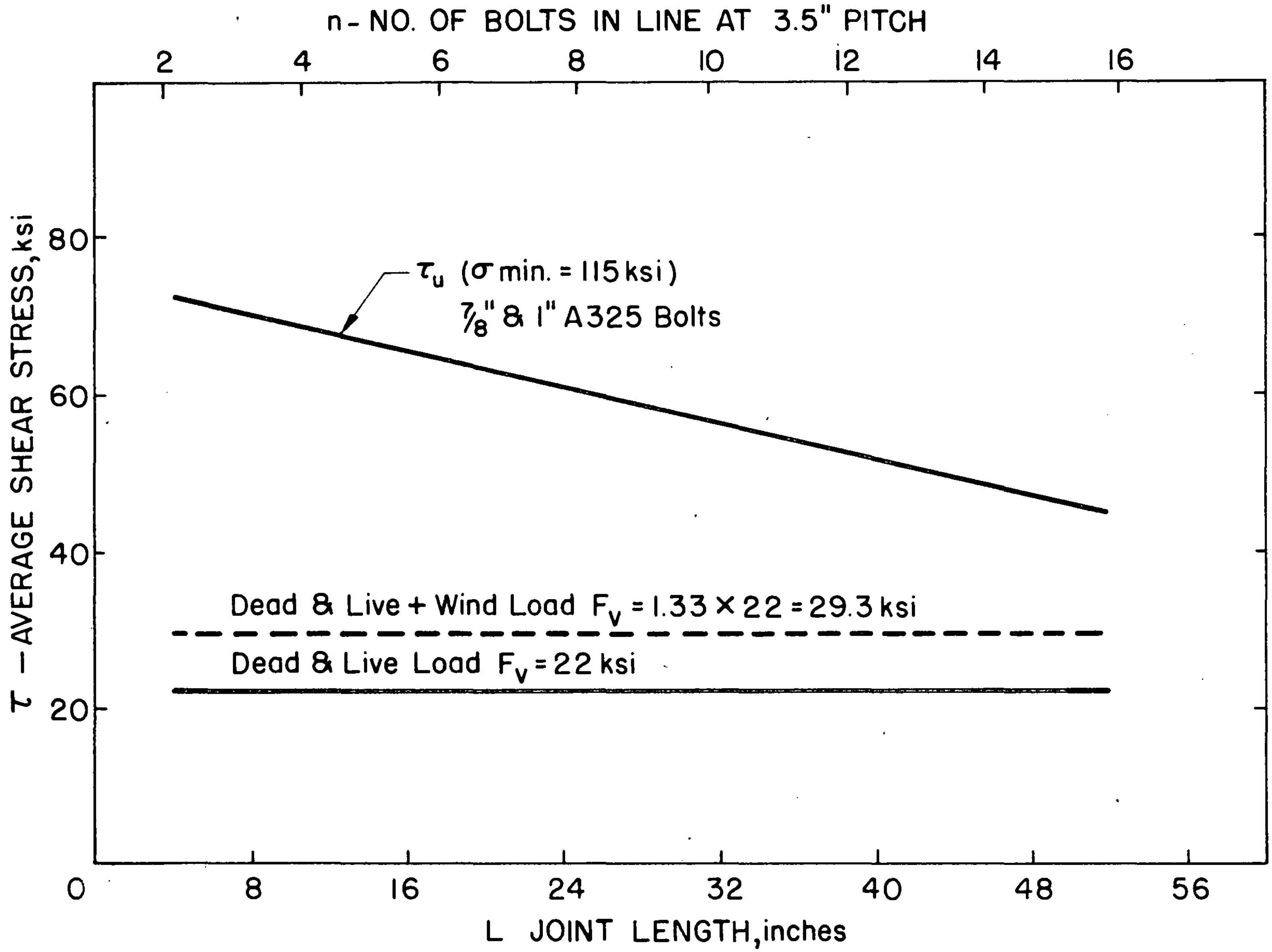



FIG. 15 COMPARISON OF ULTIMATE AND WORKING SHEAR STRESS

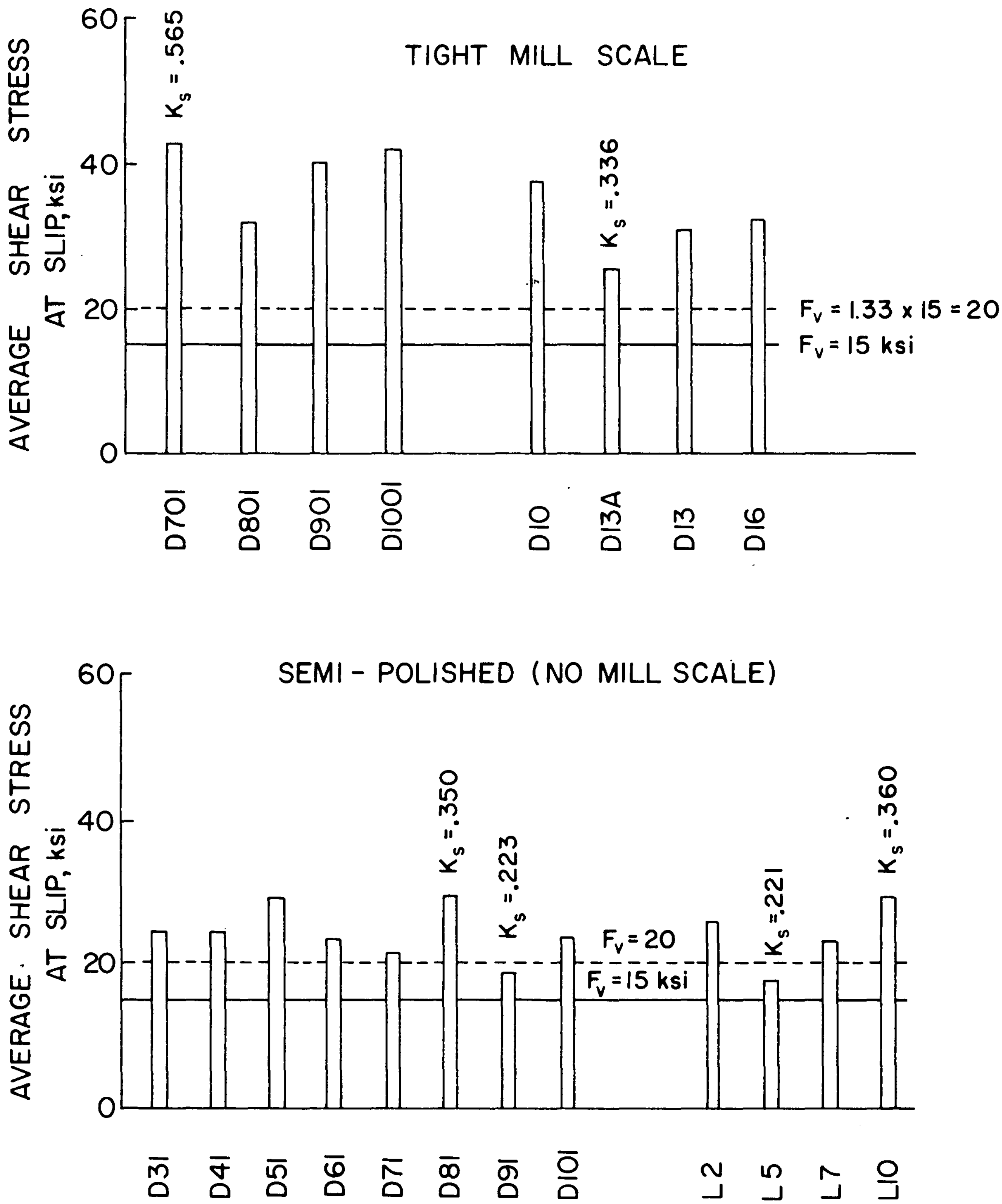



FIG. 16 SLIP RESISTANCE OF BOLTED JOINTS
TIGHTENED BY TURN-OF-NUT METHOD

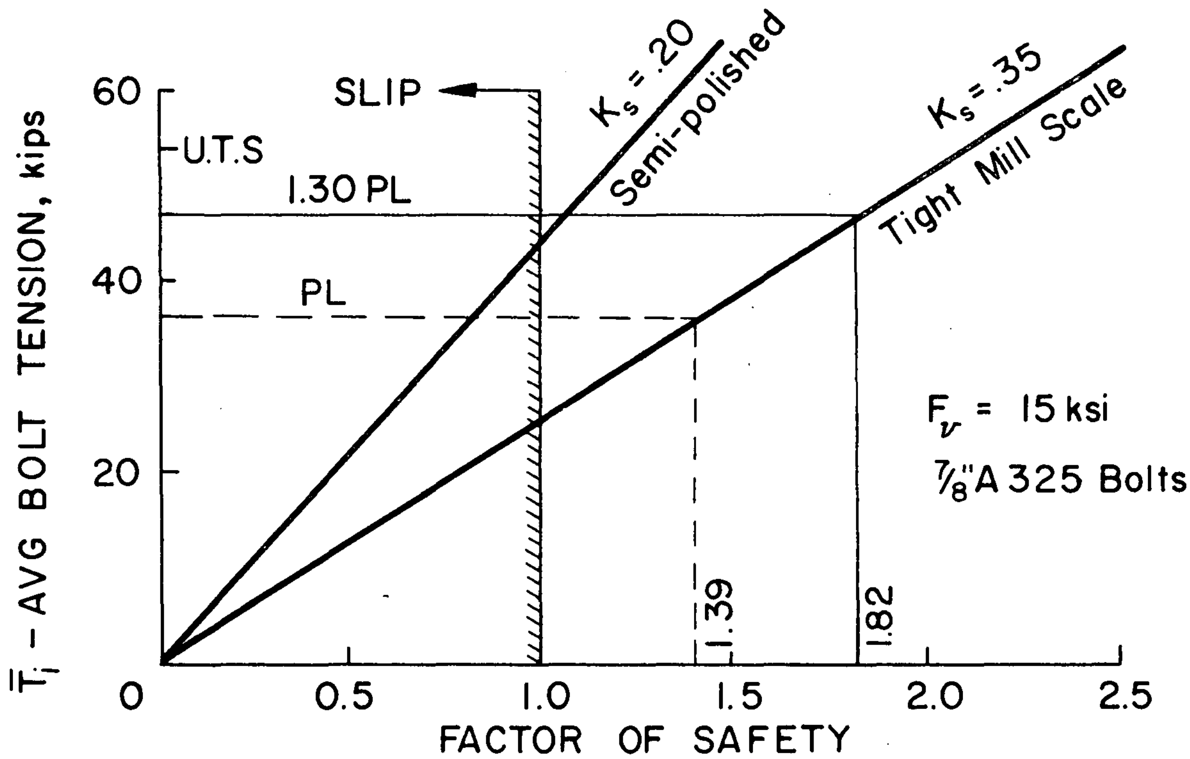



FIG. 17 FACTOR OF SAFETY AGAINST SLIP
DEAD AND LIVE LOAD, $F_V$ = 15 KSI

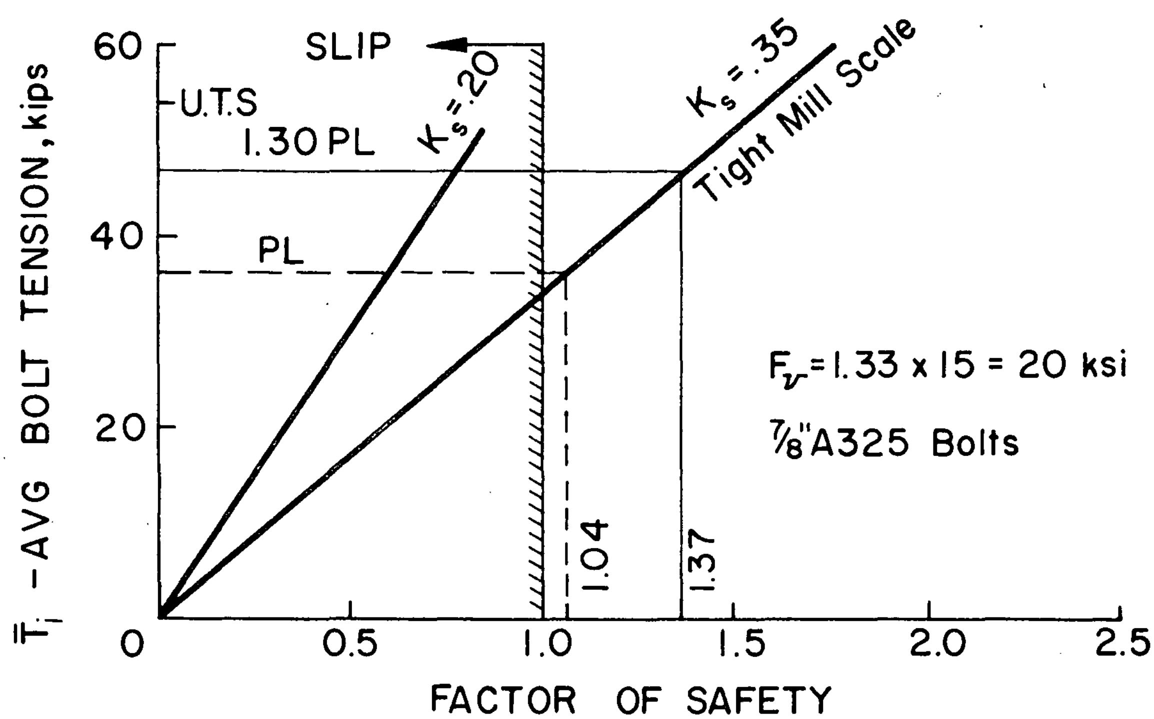



FIG. 18 FACTOR OF SAFETY AGAINST SLIP
DEAD AND LIVE PLUS WIND LOAD, $F_V$ = 20 ksi