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Long bolted joints of structural steel, draft #2, October 1961

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Power Division

2:30 p.m. Gold Room

Presiding: Marcel P. Aillery, Chairman, Exec. Committee

2:30 Towers and Foundations for Extra-High-Voltage Transmission Line


3:30 The Peixota-Furnas-Belo Horizonte 345/400-kv Line, South America’s First EHV Transmission Line

ARTHUR G. VILLEPIQUE, Civil Engr., Ebasco Services, Inc., New York, N. Y.

4:00 Design of Extra-High-Voltage Steel-Tower Transmission Lines, Niagara Adirondack Tie Line

W. O. PETERSEN, Civil Engr., Chas. T. Main, Inc., Boston, Mass.

Structural Division

2:30 p.m. Georgian Room

Presiding: Emerson J. Ruble, Chairman, Exec. Committee

Research Session

2:30 Use of Epoxy Resins in Rail Joint Bars


2:45 Transfer of Moment from Slabs to Columns

NORMAN W. HANSON, Portland Cement Assoc., Chicago, Ill.

3:00 Friction Welding of Steel

T. H. HAZLETT, Prof., Dept. of Industrial Eng., Inst. of Eng. Research, Univ. of Calif., Berkeley.

3:15 Testing of Welded Structural Joints of a Two-Dimensional Truss Built from Steel Tubes

J. C. BOUKAMP, Asst. Prof., Dept. of Civil Eng., Univ. of Calif., Berkeley.
3:30 Hinge Formation in Concrete Connections
C. P. SIESS, Dept. of Civil Eng., Univ. of Ill., Urbana.

3:45 Fatigue Strength of Butt Welds in High-Strength Steel
W. H. MUNSE, Prof., Dept. of Civil Eng., Univ. of Illinois, Urbana.

4:00 Fatigue Strength of Fillet Welds in High-Strength Steel

4:15 Role of Field Testing in Evaluating Connections of Precast Members
JACK R. JANNEY, The Engineers Collaborative, Des Plaines, Ill.

4:30 Effect of Welded Studs on Fatigue Behavior of Plates and Beams
J. E. STALLMEYER, Dept. of Civil Eng., Univ. of Illinois, Urbana

4:45 Tapered Beams and Columns

5:00 Concrete Connections
C. W. WASHA, Dept. of Eng. Mechanics, Univ. of Wisconsin, Madison.

Surveying and Mapping Division
2:30 p.m. Gold Room Foyer

Design Surveys
2:30 Chesapeake Bay Bridge Tunnel Survey

3:00 Certificates of Special Knowledge in Photogrammetric Engineering
JOHN H. WICKHAM, JR., Exec. Director, Assoc. of Professional Photogrammetrists, Jenkintown, Pa.
10:30 Automatic Highways—Future Possibilities

ALBERT A. ATWELL, Transportation Policy Group, Washington, D. C.

Power Division

9:00 a.m. Gold Room

Presiding: A. J. Michael, Member, Committee on Session Programs

9:00 General Civil Engineering Features of Indian Point Atomic Power Plant

T. R. GALLOWAY, Chief Structural Engr., Consolidated Edison Co., New York, N. Y.

9:30 Internal Construction—Containment and Shielding for Indian Point Atomic Power Plant


10:00 Design Features of Fuel Handling and Waste Treatment at Indian Point Atomic Power Plant

W. B. WHITE, Boiler Div. Engr., Consolidated Edison Co., New York, N. Y.

Soil Mechanics and Foundations Division

9:00 a.m. Ivy Suite

Presiding: Thomas M. Lepes, Chairman, Exec. Committee

9:00 Stabilization of Excavations by Freezing


9:15 Discussion

WILLIAM A. O'LEARY, Director, Div. of Sewage Disposal, N.Y.C. Dept. of Public Works, New York, N. Y.
9:30 Stabilization of New Haven Long Wharf Area for Industrial Development

JAMES D. PARSONS, Assoc. Partner, Moran, Proctor, Mueser & Rutledge, New York; and HENRY A. PFISTERER, Consulting Engr., New Haven, Conn.

9:50 Discussion

JOHN LOWE, III, Assoc. Partner, Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

10:00 Foundation Difficulties and Failures

JACOB FELD, Consulting Engr., New York, N. Y.

10:30 Discussion

EDWARD E. WHITE, President, Spencer, White & Prentis, New York, N. Y.

Structural Division

9:00 a.m. Grand Ballroom

Presiding: T. R. Higgins, Member, Exec. Committee

Structural Connections—Steel

9:00 Recent Trends in Structural Bolt Connections, Design and Specifications

EDWARD R. ESTES, JR., Chief Engr., Florida Steel Corp., Tampa, Fla.

9:30 Long Bolted Joints of Structural Steel


10:00 Tensile Behavior of Large Riveted and Bolted Truss-Type Connections

E. CHESSON, JR., Asst. Prof., and WILLIAM H. MUNSE, Prof., Civil Eng. Dept., Univ. of Ill., Urbana.

10:30 Net-Section Design of Riveted and Bolted Tension Connections

WILLIAM H. MUNSE, Prof., and E. CHESSON, JR., Asst. Prof., Civil Eng. Dept., Univ. of Ill., Urbana.
CALENDAR OF EVENTS

Annual Business Meeting: Wed., Oct. 18, 9:00 a.m.

Conditions of Practice Sessions: Mon., Oct. 16, 11:00 a.m.; Tues., Oct. 17, 11:00 a.m.; Thurs., Oct. 19, 11:00 a.m.; Fri., Oct. 20, 11:00 a.m.

Division Sessions

Air Transport: Mon., Oct. 16, 9:00 a.m., 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.

City Planning: Mon., Oct. 16, 9:00 a.m., 2:30 p.m.; Tues., Oct. 17, 2:30 p.m.; Thurs., Oct. 19, 2:30 p.m.

Construction: Mon., Oct. 16, 9:00 a.m.; 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.

Engineering Mechanics: Mon., Oct. 16, 9:00 a.m., 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.; Thurs., Oct. 19, 2:30 p.m.; Fri., Oct. 20, 2:30 p.m.

Highway: Tues., Oct. 17, 2:30 p.m.; Wed., Oct. 18, 2:30 p.m.; Thurs., Oct. 19, 9:00 a.m., 2:30 p.m.

Hydraulics: Wed., Oct. 18, 2:30 p.m.; Thurs., Oct. 19, 9:00 a.m.; Fri., Oct. 20, 9:00 a.m.

Pipeline: Thurs., Oct. 19, 2:30 p.m.; Fri., Oct. 20, 9:00 a.m.

Power: Wed., Oct. 18, 2:30 p.m.; Thurs., Oct. 19, 9:00 a.m.; Fri., Oct. 20, 9:00 a.m., 2:30 p.m.

Sanitary Engineering: Mon., Oct. 16, 9:00 a.m., 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.

Soil Mechanics and Foundations: Thurs., Oct. 19, 9:00 a.m., 2:30 p.m.; Fri., Oct. 20, 9:00 a.m., 2:30 p.m.

Structural: Mon., Oct. 16, 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.; Wed., Oct. 18, 2:30 p.m.; Thurs., Oct. 19, 9:00 a.m., 2:30 p.m.; Fri. Oct. 20, 9:00 a.m., 2:30 p.m.

Surveying and Mapping: Wed., Oct. 18, 2:30 p.m.; Thurs., Oct. 19, 9:00 a.m.

Waterways and Harbors: Mon., Oct. 16, 9:00 a.m., 2:30 p.m.; Tues., Oct. 17, 9:00 a.m., 2:30 p.m.; Wed., Oct. 18, 2:30 p.m.

(Continued on inside back cover)
CALENDAR OF EVENTS
(Continued from back cover)

Conferences
Student Chapter: Mon., Oct. 16, 2:30 p.m.

Luncheons
General Membership: Mon., Oct. 16
General Membership: Tues., Oct. 17
Awards: Wed., Oct. 18
General Membership: Thurs., Oct. 19

Evening Social Events
Kick-off Party: Mon., Oct. 16, 5:30 p.m.
Dinner-Dance and Reception: Wed., Oct. 18, 6:30 p.m.
United Engineering Center — Open House: Thurs., Oct. 19, 5:15 p.m.

Excursions
Field Trip to World’s Fair and New York City Arterial Improvements: Tues., Oct. 17, 9:00 a.m.
Harbor Tour Around Manhattan: Tues., Oct. 17, 2:30 p.m.
Field Trip to Indian Point Atomic Power Plant: Thurs., Oct. 19, 1:00 p.m.

Ladies Entertainment and Tours
See separate program.
LONG BOLTED JOINTS OF STRUCTURAL STEEL

by

JOHN L. RUMF

For oral presentation before the
Structural Division of the
American Society of Civil Engineers
at the Annual Meeting, October 1961
LONG BOLTED JOINTS OF STRUCTURAL STEEL
by John L. Rumpf

1. The Problem of Long Connections

Let us consider a structural connection with several fasteners in line with the load. When the connected parts and the fasteners are stressed elastically, the end fasteners carry a high percentage of the applied load. This has been shown analytically by Bethe, Bleich and Bremlhoff to mention a few of those who have studied this problem of load partitions in a riveted joint.

As the load on the connection is increased the more highly stressed end fasteners deform plastically and thereby effect a redistribution of load among the other fasteners. The deformation of the fasteners is needed to satisfy compatibility conditions. For example, the first pitch in a connected member will have a large elongation whereas the corresponding pitch in the splice material will have a small elongation. The fasteners adjacent to this pitch must deform to accommodate this difference in elongations. The amount of redistribution is a function of the fasteners' ability to deform without fracturing.

2. Some Previous Research with Long Riveted Connections

It has been found experimentally that ordinary low carbon rivets are able to deform enough to permit distribution so that at ultimate load each rivet carries an equal share of the load provided the joint is not too long.

*Professor of Civil Engineering, Drexel Institute of Technology, Phila., Pa. Formerly Research Instructor, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.*
Davis, Woodruff and Davis, in their tests of large riveted joints in conjunction with the building of the Oakland Bay Bridge, found that in some long joints the rivets did not have enough deformation capacity to effect complete redistribution. In these joints, and rivets sheared before the ultimate strength of all the rivets was utilized. Nevertheless, Davis, Woodruff and Davis felt that the redistribution was close enough to being complete that no alteration in rivet design procedure need be made.

Notice that the philosophy of rivet design has been one based upon ultimate strength. The assumption made that each rivet carrying an equal share of the load is not true in the working stress range and only approaches the truth when a load factor of about 3 carries the joint to its ultimate capacity.

3. Advent of the High Strength Bolt

When the A325 high strength bolt came upon the structural scene following World War II, it was used as a 1 for 1 replacement for the A191 rivet. This usage was adopted despite the facts that the bolt material was stronger and the method of carrying working load quite different - friction vs shear and bearing for the riveted joint.

Eventually, after the bolt had become fairly well accepted it appeared possible to try to refine the design of bolted joints. Accordingly, in March of 1960, the Research Council for Riveted and Bolted Structural Joints issued a specification that recognized two types of bolted connections - the "friction type" and the "bearing type".
In the friction-type joint no slip can be tolerated until some acceptable load factor beyond the working load. Frictional resistance depends upon the condition of the contact surfaces and the tension in the installed bolt. The Council chose to design such a joint using an allowable shear stress even though no bolt shear stress actually exists when the joint is transferring load by friction. The allowable shear stress was chosen the same as the allowable rivet stress because that value had proven safe in practice over the last ten years, and it agreed with research work.

4. Lehigh University Research Program

In the bearing type joint the higher strength of the A325 can be utilized. Beginning in 1957 an investigation has been conducted at the Fritz Laboratory at Lehigh University to determine the ultimate behavior of such joints, and thus establish the proper allowable working stress. The research project is sponsored by the Pennsylvania Department of Highways, the Bureau of Public Roads and the American Institute of Steel Construction. The Research Council acts in an advisory capacity.

The first phase of the project dealt with the static tensile strength of large but compact bolted joints. It was determined that balanced design at ultimate load existed when the joint was proportioned with a tension-shear ratio of 1 to 1.10. That is, at any given load, the calculated tensile stress on the net section of the main member and the calculated average shear stress on the bolts are in the proportion of 1 to 1.10. The results of this work were reported in the Journal of the Structural Division in June 1960.
5. Problems with Long Bolted Joints

Being aware of the phenomenon of "premature" failure of end rivets in long joints, the questions naturally arose: How will long bolted joints behave? Do the bolts, being of higher strength material, lack the deformation capacity to effect equalization of bolt stresses? Must a modification of allowable stress be specified for long bolted joints?

To answer these questions a second phase of the Lehigh University program was begun and the results will be reported here today.

6. Description of Test Series

A total of 16 large splices of A7 steel plate and 7/8" diameter A325 bolts were tested under static tensile loading. Fig. 1. The number of bolts in line varied from 3 to 16. In order to hold the tension-shear ratio to 1 to 1.10 the width or the thickness of the plate material was varied. This is illustrated in Fig. 2, which shows the salient features of the test specimens.

All bolts were installed in drilled 15/16" holes that were perfectly aligned. Washers were used under both head and nut. Bolts were tightened with an impact wrench by the turn-of-nut method. Those with grips of less than 5" received one-half (½) of a turn from snug, those 5" or over 3/4 of a turn. This method of tightening produced tensions of over 130% of the bolt proof load.

The large plate elements were obtained by laminating 4 to 8 plates of 9/16" to 1" thickness. This A7 steel ranged in ultimate strength from 60 to 66 ksi.
7. Progress of a Test

It is interesting to follow the progress of a test by means of a load-elongation curve. (Fig. 3) Let us use for an illustration joint B61 with 9 bolts in line. The ordinate of the graph is the load applied to the connection by the testing machine and the abscissa is the total elongation in the distance x to x.

As load is applied initially the joint transfers load by friction and also it behaves in an elastic fashion. Eventually the applied load exceeds the static frictional resistance and slip occurs. In this case slip took place in a number of steps but in other cases the joint slipped into bearing at one instant. The bolts now transfer load by shear and bearing and some friction. Further loading produces inelastic behavior in portions of the connection and eventually the deformation capacity of one of the end bolts is exceeded and it fails. In this case one bolt sheared at 1396 k and the testing machine was partially unloaded in order to inspect the connection from close-up. On reloading a second bolt sheared at 1347 k. The connection was unloaded again and then on re-application of load 4 bolts sheared at a load of 1372 k. Note that this is higher than the load which caused the first bolt to shear. However, in all of this work, the load which caused the first bolt to shear has been taken as the ultimate load. Hence, many of the tests discontinued after the first shear failure although a few were carried to complete destruction.

The sequential failure of bolts has been called "unbuttoning".

Figure 4 shows rather clearly how differential elongations in the plates can cause a bolt to fail due to excessive deformation. In this photo one sees a sawed section of joint B61. This joint actually failed by overstressing the net section in tension. Notice the plate nailing.
After the test, the joint was sawed longitudinally through one line of bolts exposing the very large deformations of the end bolts. The inner bolts appear to be almost straight.

8. Slip Behavior

Although the primary objective of these tests was to evaluate the ultimate strength of long joints, useful information on slip was obtained. Fig. 5 shows the slip results of all Lehigh University test specimens designed with a tension-shear ratio of 1 to 1:10. The height of the bar represents the average bolt shear stress at the time of slip. All of those joints had dry, tight mill scale contact surfaces except the joints of D series Part a. In that case an overeager iron worker removed loose mill scale by using a power grinder. Such treatment resulted in semi-polished surfaces that were quite shiny and reflective.

Notice that all of those joints, regardless of the surface condition, slipped above the allowable working shear stress of 15 ksi specified by the AISC for "friction type" joints. Furthermore, all joints with tight mill scale faying surfaces slipped above the allowable working shear stress of 22 ksi specified for bearing type joints. The experience at Lehigh University has shown that with the high clamping force obtained by the turn-of-nut method the latter stress should be satisfactory for both types of joints, friction and bearing.

Fig. 6. Another, and more realistic way of expressing resistance to slip is by means of a slip coefficient. For the tight mill scale surfaces, this ranged from .31 to .51, whereas for the semi-polished surfaces it spread from .21 to .34.
9. Ultimate Strength

As stated before, the main objective of these tests was to determine the effect of the length of the joint on its ultimate strength. To represent this effect a non-dimensional quantity called the "unbuttoning factor" has been used. (Fig. 7) The "unbuttoning factor" is defined as the average bolt shear stress in the large joint when the first bolt fails divided by the ultimate shear stress on a single bolt of the same lot and of the same grip. One sees that this is an efficiency factor that measures how well equalization of load on the bolts has taken place.

(Fig. 8) Let us plot the "unbuttoning factor" for each test joint against the length of that particular joint. Since a pitch of 3\(\frac{1}{4}\) inches was used in almost all cases the length has been expressed in terms of the number of 3\(\frac{1}{4}\) pitches. Theoretically, when 2 bolts are in line the bolt forces are statically indeterminate and the unbuttoning factor should be 1.0. The experimental work shows an unbuttoning factor of 0.9 with 3 bolts in line. Some of the joints failed in tension, so the value for \(U\) represents a lower limit only. At the extreme length of 16 bolts in line, or 52\(\frac{1}{2}\)" between end bolts, the "unbuttoning factor" is down to .60. This shows qualitatively how in a long joint the end fasteners fail by excessive deformation before the inner bolts are helping to carry much of the load.

It is useful to put this information in terms of the average shear stress in connections fastened with minimum strength A325 bolts (Fig.9). Such a plot helps us decide what the allowable shear stress should be for the bearing type connection. If we propose to hold a constant load factor, then we should use a working stress that varies with the length of the
joint as represented by the dashed line. In this case the load factor was chosen as 3.

On the other hand, we may decide to set the allowable stress at a constant value as done in the 1960 specifications of the Research Council. The majority of structural connections are of the shorter lengths and tests on such connections have shown balanced design at a tension-shear ratio of 1 to 1.10. Therefore, we may set the allowable stress for A7 plate at 20 ksi and for A325 bolts 22 ksi. The latter stress is represented by the solid line. Up to about 4 bolts in line the load factor is 3 or greater but for longer joints it decreases. At 16 bolts in line the load factor is down to about 2.1. By use of the present specification we are accepting a reduced factor of safety on the infrequent but very important long structural joints. Designers are urged to approach the long connection with caution and possibly shorten it by using fewer bolts of a larger diameter.

Another way of examining the results of these tests is to measure the ability of the bolts to develop the strength of the connected material. In Fig. 10, the height of the bars represents the net section tensile stress divided by the minimum ultimate tensile stress of A7 steel, 60 ksi. The bars are arranged with the longest joint to the right. In the short joints the bolts developed a net section stress greater than 60 ksi, whereas with 16 bolts in line only 82% of that minimum stress was developed. In other words "balanced design" did not exist.
10. **Analytical Solution**

Along with the experimental work an analytical solution for the ultimate strength was attempted. A solution for this type of connection entails one equation of static equilibrium involving a unknown bolt forces and \((n-1)\) compatibility equations relating plate and bolt deformations. To solve these equations the deformations must be related to force. This was done by experimentally calibrating single bolts and typical pitches of plate. The resulting calibration curves were practically linear at low load but were non-linear in the upper regions. Thus determination of the ultimate strength of the joint required solution of a system non-linear equations. The solution was accomplished by a graphical iteration procedure.

*Results of analytical work for joints of D Series - Part a are compared with the experimental results on a unbuttoning plot.* (Fig. 11) The red crosses represent the analytical solution. Good correlation is shown.

It is interesting to examine the forces on the individual bolts as determined by the analytical solution. Fig. 12 shows the bolt forces in D91 as a percentage of the equally distributed bolt force assumed in design. The end bolts \(R_1\) and \(R_9\) pick up a greater percentage of the load as the plate yields in the end pitches. As load is increased the end bolts yield and therefore pick up load at a decreased rate. Percentage wise the end bolts then carry a smaller proportion of the total load whereas \(R_2\), \(R_3\) and even \(R_4\) pick up load during the redistribution process. However, at the ultimate load when \(R_1\) fails it is still carrying 135\% of the force assumed in design calculations whereas \(R_3\), in the middle of the joint, is only carrying 50\%.
11. **Effect of Pitch**

In all but one of the test joints the pitch was 3½" or 4 bolt diameters. This is greater than the minimum pitch of 3 bolt diameters stated by many specifications. In one connection, D13A, the pitch was held at three diameters or 2 5/8". To observe the effect of pitch on the ultimate strength of bolted connections it is interesting to compare joints D10, D13A and D13 (Fig. 13) D10 with 10 bolts at a 3½" pitch and D13A with 13 bolts at 2 5/8" had the same overall length of 31½". D13A and D13 had 13 bolts in line but their overall lengths were 31½" and 42" respectively.

To compare the efficiency of these connections let us return to the unbuttoning factor graph. (Fig. 14) In this case the joints being discussed are shown in red. Joints D10 and D13A had almost identical unbuttoning factors of .708 and .701. This simply points out that the tendency of end bolts to fail "prematurely" is more a function of the length of the joint than of the number of fasteners per se. One must be careful not to construe this to mean that the carrying capacity of the 10 bolt in line connection is equal to that of the 13 bolt in line connection. The latter carries 13/10 as much load.

Comparing D13A and D13 we see unbuttoning factors of .701 and .654. The shorter joint is more efficient and a 7% increase in load carrying capacity was achieved by decreasing the pitch from 4 diameters to 3 diameters.

12. **Comparison with Rivets**

As this program developed and the unbuttoning trend for bolts became defined an attempt was made to compare it qualitatively to the work on rivets done by Davis, Woodruff and Davis. Difficulty was experienced
because many of the riveted test specimens had another variable, namely pattern. Thus, it was decided to test a few riveted joints using the same plate and the 2 line pattern as for the bolted joints. Three joints were designed at a tension shear ratio of 1 to 0.75 with 7, 10 and 13 A141 rivets in line. Results of a short compact joint were also available.

Examining the unbuttoning factor graph again (Fig. 15) we see the riveted joints represented by green points. The same general trend is noted although the rivet appears to have a little more ability to deform, redistribute load, and develop a higher average shear stress before end rivets shear.

Once again one must not gain the wrong idea from this plot. For example a joint with 13 A141 rivets in line is not stronger than a joint with 13 A325 bolts. The latter actually is about 50% stronger.

13. Summary

Let us summarize the important findings of this test program.

a. In bearing type joints - either erected in bearing or where slip into bearing can be tolerated - one can legitimately design A325 bolts for a shear stress based upon the ultimate strength of such a connection. If a short compact joint of A7 steel is proportioned at a tension shear ratio of 1 to 1.10 a balanced ultimate strength design results. Using a load factor of 3 the allowable shearing stress is 22 ksi.

b. In long joints the differential strains in the connected material will force end bolts to shear before equalization of load among all the bolts has taken place. This premature failure of end bolts has been dubbed "unbuttoning". It has been shown that with 16 bolts in line, designed on the basis of 22 ksi shear stress, the load factor to produce failure is only 2.1. Engineers are cautioned to pay attention to
long joints especially in the determination of loading and the permitting of occasional overloads.

c. Whenever a given number of bolts is required to transmit load the efficiency of the bolts is enhanced by holding to small pitch distances.

d. Lest anyone be unduly concerned over the "unbuttoning" problem of bolts it has been shown once again that the same situation exists and has existed in riveted joints. No special requirements have covered this in past and present specifications. Furthermore use of the higher strength A325 bolt will result in fewer fasteners and shorter connections.

e. The slip behavior of these test joints showed that for dry, tight mill scale surfaces and bolts tightened by the turn-of-nut method a coefficient of slip of .35 is probable. In terms of average bolt shear stress this is well above the AISC Specification value of 15 ksi for friction type joints.

f. Last, a point not mentioned before, there was no evidence in these tests of a need for a rule, as presently exists in rivet specifications calling for additional bolts whenever the grip is over 5".
Sawed Section thru D61
A second theoretical program was introduced to study the effect of pitch on riveted joints. Using the plate calibration work, several hypothetical joints were analyzed by theoretical means. Additional test specimens were fabricated and tested as part of the general research program. The results of theoretical and experimental work were arranged to show comparative degrees of validity.

4.21 Description of Hypothetical Joints

Five hypothetical riveted joints were analyzed to obtain information about the effect of pitch. Four joints had thirteen rivets in line and one had ten rivets in line. All were designed at T/S equal to 1/0.75. Using the plate calibration specimens described these hypothetical riveted joints were similar to the series of theoretical bolted joints.

The joints were assumed to be fastened with 7/8 inch diameter four-inch grip ASTM-A141 rivets installed in 15/16 inch diameter holes. The properties of these DR lot rivets were as follows:

- Ultimate tensile strength: 57.7 ksi
- Ultimate shear strength: 52.9 ksi

A similar notation system, illustrated in Fig. 4.1, was
\[ \frac{T}{S} = \frac{1}{1.10} \]
D· SERIES
TEST JOINTS

Part a
8 joints

Part b
4 joints

Part c
4 joints

Slide 271.153
271.19 Fig 2
Bolt Failure Sequence

Elongation x-x, inches

Test Discontinued

D 91

Slide 271.154
27.1.19 Fig. #3
Unbuttoning Factor \( U = \frac{\tau_{\text{avg.}}}{\tau_1} \)
Average Shear Stress ksi

N, Number of Pitches at 3.5"

\( \tau_{\text{ultimate}} \)

\( \tau_{\text{working}} = 22 \text{ ksi} \)

Slide 271.159
27/6/19 Fig 9
ABILITY OF BOLTS TO DEVELOP 60 ksi ON NET SECTION OF PLATE

Bar chart showing the ability of bolts to develop 60 ksi on net section of plate. The x-axis represents the number of bolts in line (3 to 16), and the y-axis represents the ratio of net stress to 60 ksi. Symbols indicate bolt failure and plate failure.

\[
\frac{T}{S} = \frac{1}{1.10}
\]
Fig. 4.16
JLR Dissertation
Bolt Force as % of Uniform Distribution
Slide No. 271.143
4. BEHAVIOR OF LONG RIVETED JOINTS

4.1 Application of Theory

With slight modifications, the semi-graphical analysis described may also be used for riveted joints. A bolt and rivet behave in a similar manner except that the rivet is assumed to entirely fill the hole and no slip is possible. Thus, the value of \( c \) in Eq. 2.2 would be zero. Actually, the rivet in some cases does not entirely fill the hole and slip may take place.

Because the effects of slip are regarded as nonessential to ultimate strength behavior, the same equations apply to riveted joints as did bolted joints and analysis proceeds accordingly.

4.2 Theoretical and Experimental Work

The balanced design tension-shear ratio for A7 plates fastened by A141 rivets is 1/0.75. A hypothetical joint can be devised using the plate calibration specimen PC9b and thirteen 7/8 inch rivets in line that will have T/S equal to 1/0.76.
9 at $3\frac{1}{2} = 31\frac{1}{2}$

12 at $2\frac{5}{8} = 31\frac{1}{2}$

12 at $3\frac{1}{2} = 42''$