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Large Bolted Joints

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ABSTRACT

The results of analytical and experimental studies of constructional alloy (A514) steel bolted joints are used to evaluate the performance of A514 steel members. The examination shows that A514 steel joints using high strength bolts do not produce desirable behavior in their members if the elements of the joint are designed according to current (1968) practice. The study shows that the ratio of net area to gross member area and the ratio of net area to total bolt shear area need to be considered in the design of A514 steel tension members.

It is shown that satisfactory member behavior for A514 steel joints can be obtained by using different fabrication techniques and proper care in detailings of the joints. Some type of upset end, such as an increase in plate width or thickness in the region of the joint can be used to provide the required net to gross member area to insure yielding of the gross cross-section before the ultimate load of the joint is reached.

BEHAVIOR OF LARGE A514 STEEL BOLTED JOINTS

by Geoffrey L. Kulak[1] and John W. Fisher[2]

INTRODUCTION

The purpose of this paper is twofold. First, the behavior of constructional alloy (ASTM A514) steel tension members which use high strength bolted splices is examined. This examination shows that such members may not meet commonly accepted performance criteria when current design procedures are used. The second purpose of the paper grows out of the first, that is, a solution to the problem of inadequate member performance is presented. This solution recognizes the need for consistency with the design methods for high strength bolted tension splices in other, lower strength steels. Since the behavior of bolted tension splices has now been examined[3,4,5,6] for the full range

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[3]Bendigo, R. A., Hansen, R. M., and Rumpf, J. L., "Long Bolted Joints", Journal of the Structural Division, ASCE, Vol. 89, No. ST6, December 1963

[4]Fisher, J. W., Ramseier, P. O., and Beedle, L. S., "Strength of A440 Steel Joints Fastened by A325 Bolts", Publications, IABSE, Vol. 23, 1963

[5]Sterling, G. H., and Fisher, J. W., "A440 Steel Joints Connected by A490 Bolts", Journal of the Structural Division, ASCE, Vol. 92, No. ST3, June 1966

[6]Kulak, G. L., and Fisher, J. W., "A514 Steel Joints Fastened by A490 Bolts", Report No. 317.8, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania, June 1967

of steels commonly available for bridges and buildings, specific design recommendations can be made.

The fact that equality of load cannot exist among the fasteners in a tension splice containing more than two fasteners in line has been known for a considerable time.[7] Early research focussed on the elastic region of behavior, however, and it was assumed that, as the splice material becomes inelastic, fastener load did approach equality. In fact, the non-linear behavior of the component parts as the joint approaches its ultimate load means that the end fasteners receive the highest load and there is a gradual decrease in fastener load as one proceeds toward the center of the joint. The general mathematical development of this phenomenon[8] and the particulars of the case under present consideration[9] have been previously presented and will not be reviewed here.

[7] Arnovlevic, I., "Inansprunchahme der Anschlussnieten Elastischer Stabe", <u>Zeitschrift fur Architekten und Ingenieure</u>, Vol. 14, Heftz 1909

[8] Fisher, J. W., and Rumpf, J. L., "Analysis of Bolted Plate Splices", <u>Journal of the Structural Division</u>, ASCE, Vol. 91, No. ST5, October 1965

[9] Kulak, G. L., "The Analysis of Constructional Alloy Bolted Plate Splices", Ph.D. Dissertation, Lehigh University, Bethlehem, Pennsylvania, 1967

Although the primary objective of this paper is to examine the behavior of bearing-type bolted joints, information is also presented on the slip resistance of A514 steel bolted connections.

CHARACTERISTICS OF CONSTRUCTIONAL ALLOY STEEL

Steels of ASTM A514 were first introduced in 1952 when United States Steel Corporation offered its proprietary steel "T-1".[10] Other manufacturers subsequently either were licensed to produce this steel or produced their own proprietary steels of similar characteristics. In 1964, most of these steels were covered by ASTM Specification A514.[11] The mechanical properties of A514 steel are summarized in Table 1.

The high yield strength alloy steels were developed to meet the need for a constructional steel, primarily in plate form, which had a yield point in the order of 90,000 psi, good low temperature toughness, and good weldability. The attainment of these qualities is largely a reflection of two factors; a microstructure of tempered martensite and a low carbon content.[10]

[10] Bibber, L. C., Hodge, J. M., Altman, R. C., and Doty, W. D., "A New High-Yield Strength Alloy Steel for Welded Structures", Transactions, ASME, Vol. 74, 1952

[11] "Standard Specification for High-Yield-Strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding", ASTM Designation A514-64, 1967 Book of ASTM Standards, Part 4, Philadelphia, Pennsylvania 1967

Tempered martensite is characteristically tough and this toughness is most pronounced at low levels of carbon. Furthermore, this mirco-structure permits the attainment of the very high strength desired at this low carbon level. The principal alloying elements used are manganese, molybdenum, boron, chromium, nickel, and vanadium, titanium, or zirconium. The latter is included to maintain the high yield strength characteristics of the material in the face of tempering at high temperature.

The fundamental difference in behavior between A514 and other grades of structural steel can be seen by examining Figure 1. The curves shown are indicative of the minimum stress-strain patterns which may be expected from actual testing of specimens. These curves show that constructional alloy steel is characterized by a very low ultimate v. yield stress ratio. This can be as low as about 1.07 while the ratio for A440 and A36 steels is about 1.46 and 1.67 respectively. It will be shown that this can be a significant feature in the behavior of constructional alloy steel tension members. In addition, this steel, like most other alloy steels, does not exhibit any well-defined yield point.

Examination of the stress-strain curves also shows that, while the total energy absorbed by each of the various grades is not greatly different, the elongation of a structural carbon steel is about twice that of A514. The lesser amount of ductility of A514 and its low ultimate to yield stress ratio may be of concern. A certain amount of engineering opinion exists that believes there should be as large a spread as possible between yield and ultimate.[12] If this opinion is well-founded, there is a paradox in the fact that constructional alloy steels have met with wide acceptance and been successfully used for some fifteen years.

Part of the answer lies in the applications for which A514 has been used over this period. It is quite clear that the primary use for which A514 was intended was in the construction of pressure vessels.[13] The important criterion here is the amount of energy absorbed and a large amount of discussion is given over to this point in the early literature.[12]

The problems with respect to structural engineering are somewhat different, however. It will be shown that member ductility is considered to be of considerable importance in tension members. It will also be shown that for a wide range of practical cases A514 steel members may not provide sufficient ductility if designed according to current practice.

[12]Bibbler, L. C., "The Suitability of Quenched and Tempered Steels for Pressure Vessel Construction", Welding Journal, Vol. 34, 1955

[13]Doty, W. D., "Properties and Characteristics of a Quenched and Tempered Steel for Pressure Vessels", Welding Journal, Vol. 34, 1955

REVIEW OF CURRENT DESIGN PHILOSOPHY

Before examining the characteristics of constructional alloy steel members designed according to current practice, it is pertinent to review the design of structural steel tension members in general. The design of the members and their connections must be carried out in accordance with an inter-related philosophy. The basis of design and the desired member behavior has been clearly set forth.[14,15] The philosophy that has been established considers that the limit of usefulness of a tension member is given by the load at which contained plastic flow commences in the gross section of the member. Beyond this point, "significant and relatively uncontrolled" elongation occurs.[14] Thus, the factor of safety in the member is against contained plastic flow.

The philosophy of design of connections in tension members has recently been reviewed.[15] In evaluating the behavior of any structure, it is considered desirable that the system have capacity for distortion or geometrical adjustment before failure

[14] Tall, L., et al, "Structural Steel Design", Ronald Press Company, New York, New York, 1963

[15] Fisher, J. W., and Beedle, L. S., "Criteria for Designing Bearing-Type Bolted Joints", Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October 1965

by fracture. In an axially loaded structure, this means that the connections should be proportioned so that yielding takes place in the gross cross-section of the member before the joint fails. (This joint failure can be either by tearing of the plates through the net section or by fastener shear). Thus, although the individual member will be beyond its defined limit of usefulness, the criterion for satisfactory behavior of the structure or assemblage of which the member is a part demands further deformation capacity. The criteria proposed[15] for mechanically fastened axially loaded structures of carbon or high strength steels are based on this requirement.

It appears then, that whatever allowable stresses are established for A514 steel tension members, the same general philosophy should apply. The factor of safety will be against contained plastic flow and the design procedure and/or allowable stresses must ensure yielding through the gross section before failure.

CHARACTERISTICS OF A514 STEEL MEMBERS DESIGNED ACCORDING TO CURRENT PRACTICE

Basis of the Examination

As stated, the mathematical analysis of bolted tension splices of constructional alloy steel will not be presented herein. It is appropriate, however, to examine the basis upon which the analytical studies have been made.

1. All analytical work, except that which related directly to test specimens, has been carried out on the basis of plate and fastener material of minimum strength. Thus, a lower bound to the solution of the problem has been obtained.

2. The analytical studies have been based on a consideration of a single line of fasteners. Previous research has shown that the behavior of multiple, identical lines of fasteners is directly proportional to the number of such lines present.[4,16] The behavior of multiple, staggered lines of fasteners has not been examined.

3. The examination has been made on the basis of 7/8 in. diameter fasteners placed at 3-1/2 in. pitch. The general strength characteristics of A514 steel bolted joints have been shown to be independent of these variables, per se.[9]

It should also be noted that the mathematical solution is such that the use of an electronic digital computer is a

[16] Foreman, R. T., and Rumpf, J. L., "Static Tension Tests of Compact Bolted Joints", <u>Transactions</u>, ASCE, Vol. 126, Part II, 1960

necessity if joints of any practical size are to be analyzed. Considering this, as well as the fact that certain physical control tests are needed, it was not the intention to develop an analytical solution suitable for design purposes. Rather, after suitable experimental verification, it was to be used as the basis for evaluation of the behavior of A514 steel members under present design methods and to suggest new design procedures and revised allowable bolt shear stresses.

<u>Location of the Plate Failure-Fastener Failure Boundary</u>

The first step in any examination of the ultimate strength of a bolted joint must be to determine the mode of failure. The boundary between those joints failing by fastener shear and those failing by tearing of the plates is shown in Fig. 2 for A514 joints using A490 bolts. It is plotted here as the average fastener shear stress at ultimate joint load vs. joint length. The dashed horizontal line extending across the figure at a shear stress value of 91.5 ksi represents the "ideal" joint, that is, one in which all the fasteners carry an equal load. In this case, the parameter A_n/A_s would have to be equal to infinity. A_n is the net plate area of either the main or the lap plates and A_s is the total fastener shear area. Shown between this limiting line and the other limit, the failure mode boundary, are joint strength curves for selected values of A_n/A_s. A similar representation is shown in Fig. 3 for the case of A325 fasteners in A514 steel. These boundaries are plotted again in Figs. 4 and 5 for A490 and A325 bolts, respectively. Here the plots are made on A_n/A_s vs. Joint Length axes.

Characteristics of A514 Steel Joints

If A514 steel joints fastened using high strength bolts are designed according to current practice, it is unlikely that proportions will be such that failure will occur in the fasteners. Using an allowable stress of 60 ksi in the plate material[17] in combination with the current allowable stress of 32 ksi for A490 bolts used in buildings,[18] an A_n/A_s ratio of 0.53 results. As seen in Fig. 4, this A_n/A_s value intersects the plate failure - fastener failure boundary at a joint length of about 85 inches. In other words, at these stress levels, joints would have to be longer than 85 in. before the fasteners become the critical element.

The behavior of A325 bolts in A514 plate is very similar to that just described for the A490 - A514 case. The A_n/A_s ratio for A514 joints using A325 bolts will be 0.37, based again on an allowable stress of 60 ksi in the plate material and the current allowable shear stress of 22 ksi for A325 bolts used in buildings.[18]

[17] T-1 Constructional Alloy Steel, United States Steel Corporation, ADUSS 01-1205, Pittsburgh, Pennsylvania, June 1966

[18] Specifications for Structural Joints Using ASTM A325 or A490 Bolts, Approved by the Research Council on Riveted and Bolted Structural Joints, September 1966

It can be seen in Fig. 5 that at this A_n/A_s value, plate failure is the governing failure mode throughout the range of joint length investigated.

Recently, higher allowable shear stresses have been suggested for both A490 and A325 bolts when used in bearing-type connections.[15] Using these values, 40 ksi for A490 bolts and 30 ksi for A325 bolts, the lengths over which plate failure governs are reduced. In the case of A490 bolts, plate failure will govern up to a joint length of about 60 inches. The figure is about 65 in. when A325 fasteners are used. Additionally, the use of these higher allowable stresses reduces the variation in joint strength and, thus, the factor of safety, with joint length.

The effect of joint length and of changes in the relative plate and fastener shear area proportions can also be seen in Figures 2 and 3. The importance of joint length in the determination of ultimate joint strength has been shown by previous investigators.[8,19] These studies showed that the end fasteners carried the greatest load and in many of the experimental studies the amount of this inequality of fastener loads was enough that the test could be stopped when an end fastener had failed. This phenomenon was expected to occur to a lesser degree in A514

[19] Fisher, J. W., Kulak, G. L., and Beedle, L. S., "Behavior of Large Bolted Joints", Highway Research Record No. 147, Highway Research Board, Washington, D. C., 1966

steel joints. Because of the much higher yield strength of this material as compared to those grades previously investigated, a more uniform distribution of load should occur. The strength curves in Figs. 2 and 3 show that this is the case. For a given value of A_n/A_s, the average shear stress in the fasteners undergoes a gradual, almost linear, decrease.

The effect of joint length upon the behavior of individual fasteners within a joint can be seen in Figure 6. Here, the shear stresses in the fasteners of a 25-bolt joint are shown. The fasteners are A490 bolts and the A_n/A_s ratio is 0.60. This represents an extreme case - the joint length is long (84 in.) and the A_n/A_s ratio chosen puts the specimen only slightly above the failure mode boundary. The degree of load inequality among the fasteners could be expected to be relatively high in such a joint. This is borne out by the values shown in Figure 6. The shear stress in the end fasteners of this joint is 91.5 ksi while that in the centerline bolt is only 59.8 ksi.

Because of its greater ductility, the A325 bolt allows more favorable distribution of load among the fasteners than does the A490 bolt. In a similarly extreme case, long joint length (84 in.) and low A_n/A_s value (0.45), the shear stress in the end fasteners is 64.8 ksi and that in the centerline bolt is 49.0 ksi when A325 bolts are used.

The A_n/A_s ratio of a mechanically fastened joint can be thought of as a "modulus of rigidity". At a given joint length, an increasing A_n/A_s ratio means an increasingly more uniform distribution of load among the fasteners. As has already been pointed out, the ideal case of equal load distribution among fasteners occurs only at the value of $A_n/A_s = \infty$. This represents, then, a perfectly rigid joint. For any lesser value of A_n/A_s, the fasteners carry unequal loads.

It would be desirable, of course, to make the fasteners work at as uniform a stress level as practical. Two methods are available in helping to achieve this. One way of raising the A_n/A_s ratio is to provide more plate area at a given joint length. In effect, this means that allowable plate stresses are reduced while a given allowable shear stress is maintained in the fasteners. Examination of the behavior of either A325/A514 or A490/A514 joints will show that the load increase in such a step is markedly disproportionate to the increased amount of plate required to achieve it. For example, if an A490/A514 joint 70 in. long and having an A_n/A_s ratio of 0.62 has the plate area increased 61 percent, the load that can be carried increases by only 18 percent.[9]

A second way of producing an increase in the A_n/A_s ratio is to make the fasteners work at a higher stress level.

This produces no increase in material cost and the benefits in increased load carrying capability, even if small, can be accepted without question of economics. What must be examined now is the resulting factor of safety at any suggested higher fastener shear stress. Naturally, the amount of any such increase in fastener shear stress must satisfy a desired minimum factor of safety and provide sufficient strength so that satisfactory member performance is obtained.

Another possibility would be to use a combination of these two approaches. In particular, since the increased plate area is needed only in the vicinity of the joint and not throughout the length of the member, it may be feasible in some cases to provide this by means of some type of upset end. This approach is examined subsequently in this paper.

Problems Resulting from Current Design Practice

As already outlined, it is considered desirable that tension members yield across their gross cross-section before they fail, either through the net cross-section or in the fasteners. Since the use of current design practice and stress levels will generally result in joints whose potential failure is by fracture of the plates, an examination of the behavior of such joints is pertinent.

For plate failure type joints, and using minimum specified yield and ultimate stresses for A514 steel,[11] this means that the net area of the member (A_n) should be equal to or greater than 87 percent of the gross area (A_g). No explicit expression can be set up so that this requirement will be generally met. Involved in the computation of the gross and net area values are complications such as the hole pattern, method of hole formation, shape of cross section, etc. However, if the examination is restricted to consideration of members composed of plates of constant thickness containing drilled, non-staggered holes, some indication of the effect of the A_n/A_g requirement can be obtained. For this type of member, it has been shown that the minimum allowable spacing of fasteners measured perpendicular to the line of the load (gage) is about seven times the fastener diameter.[6] If this requirement is met, the gross cross section of the member will reach yield at or before failure occurs at the net section in the joint.

A staggered hole pattern is commonly used to increase joint efficiency and the situation may not be so extreme as that described above. Care must be taken in detailing so that the net to gross area requirement is still met, however. For example, if two lines of bolts were used in a non-staggered pattern, the plate width would have to be 14 fastener diameters.

If a staggered pattern were used, the plate width would have to be a minimum of seven fastener diameters. The "stagger" or pitch would then have to be chosen such that the net section formed through the first two fastener holes also equals or exceeds the A_n/A_g requirement of 0.87. Similar examples could be given for three or more fastener lines.

It should be noted that whether or not yield is reached is also dependent upon the disposition of the plate material used. For a given net area, the ratio of net to gross area will be higher for higher values of the width to thickness ratio of the section. In this examination, the practical lower limit of the width to thickness ratio as applied to either the main plate or to the combined lap plates and considering only a single line of bolts, is taken as unity.

Whether the fastener pattern is staggered or not, the same member area will have to be provided. The most economical joint, in terms of the volume of plate material, will be that which has the least joint length, however. Other considerations such as practical limits on the plate width, will enter into the design as well. Other ways of helping to overcome the difficulties in joint design peculiar to A514 steel members are discussed below.

Before discussing the effects of changes in stress levels which might help in meeting the desired member behavior, the A_n/A_g value of 0.87 used in the preceeding discussion should be critically examined. As noted, this figure was arrived at on the basis of the minimum allowable strength properties of A514 steel. These are 100 ksi yield and 115 ksi ultimate.

The as-delivered properties may be somewhat different than these minimum specified values, however. A large number of standard tensile coupon tests on T-1 steel showed an average value of yield of 118.4 ksi and an average ultimate strength of 127.0 ksi.[12] These stress levels would require the A_n/A_g to be at least 0.93. Although these figures came from a reasonably large sample, it may be argued that they do not represent A514 steel as currently produced. Recent tests on A514 plate have shown similar results, however.[6] The A_n/A_g requirement based on the yield and ultimate strengths of standard bar coupons taken from A514 plate can be expected to be 0.90 or greater.

In the absence of tests of full-size members, it is reasonable to base the A_n/A_g requirement on the results of plate - with - holes coupon tests. It has often been noted that the introduction of a hole into a plate has a strengthening effect. In setting up an A_n/A_g requirement on this basis, the ultimate strength of a plate - with - holes coupon should be compared to the yield strength of a standard bar coupon of the same material. (The ultimate strength of the connection is based upon behavior within the joint while the desired yielding occurs within the member where no holes are present). Figures from recent tests[9] show a A_n/A_g requirement here of about 0.86. Thus, the value of 0.87, as already used in this presentation, seems to represent a reasonable approach.

A DESIGN CRITERIA FOR BOLTED A514 STEEL MEMBERS

It has been shown that current design practice applied to joints in A514 steel members results in proportions such that the factor of safety in the member will generally be against plate failure. It was further shown that if the joint is fabricated by removing material from the main member in the form of holes, considerable care must be taken so that yield is reached in the gross cross section before the plates tear through the net section.

In attempting to improve the behavior of bolted constructional alloy steel tension members, it seems advisable to tie in any new proposals to the work that has been done in bolted joints of other steels. For example, there would have to be some marked increase in economy or behavior to justify the use of allowable bolt stresses that are different from those suitable for use in other grades. The stress levels that have been suggested as reasonable are 30 ksi in A325 bolts and 40 ksi in A490 bolts.[15] These recommendations covered the use of these fasteners in A36 and A440 steels.

If an allowable shear stress of 40 ksi is used for A490 bolts, it was pointed out that plate failure is the governing failure mode up to joint lengths of about 60 inches. Examination of the strength of joints of this proportion ($A_n/A_s = 0.67$) and lengths greater than 60 inches shows that the factor of safety varies from 2.02 at 60 inches to 1.98 at a joint length of 84 inches. The latter is only fractionally less than the figure of 2.0 suggested as a desirable minimum level.[15] (Factor of safety for fastener type joints is taken as the average shear stress in the bolts at the time of joint failure divided by the allowable bolt shear stress). Joints of this proportion also produce the desired yielding of the gross cross section before the ultimate load is reached.

The suggested allowable shear stress of 30 ksi for A325 bolts gives joint proportions which mean that plate failure is the governing failure mode for joints less than about 65 inches. The A_n/A_s ratio here is 0.50. The factor of safety against fastener failure is 2.02 at a joint length of 65 inches and at 84 inches it is 1.98. Again, these proportions produce satisfactory member behavior in the fastener failure region, that is, the gross section will have reached yield before failure.

These suggested allowable bolt stresses, used in conjunction with an allowable plate stress of 60 ksi, result in satisfactory joint and member behavior for joints in which the factor of safety is against fastener shear. They provide a high enough factor of safety, a factor of safety reasonably constant with length, and satisfactory behavior in the member itself. However, the most practical range of joints, those under about five feet in length, still remain as plate failure type joints and failure will ordinarily be reached before yielding starts in the gross section of the member.

Examination of Fig. 4 shows that it would take an increase in the A_n/A_s ratio to about 0.80 to ensure fastener failure for all lengths of A514 steel joints fastened by A490 bolts. If the allowable shear stress in the fasteners is maintained at 40 ksi, this can be accomplished by a decrease in the suggested allowable tensile stress of the plate material from 60 ksi down to 50 ksi. The minimum factor of safety is 2.14. However, member behavior resulting from these proportions is satisfactory only for the longer joints. At the shorter joint lengths, the requirement that yield be reached in the gross section prior to joint failure is not achieved. For example, at the time of failure of a joint with seven bolts in line (3-1/2 in. pitch) the stress in the gross cross section is only 90 ksi. It is not until the joint length reaches about 40 in. that the yield requirement is met.

The decrease in the allowable plate stress to 50 ksi has a similar effect upon the behavior of A514 joints in which A325 bolts are used. With an allowable fastener shear stress of 30 ksi, the A_n/A_s ratio now is 0.60. This provides a satisfactory margin above the theoretical failure mode boundary value of 0.54 at zero joint length. Joints of this proportion will have a factor of safety that is against fastener failure for all joint lengths. The factor of safety is 2.11 at a joint length of 84 inches. However, as was the situation with joints using A490 bolts, these proportions do not ensure the desired member behavior. Only for joints longer than about 60 in. does yielding occur in the gross cross section at or before the ultimate load of the joint is reached.

It is apparent that both the A_n/A_s and the A_n/A_g ratios need to be considered in the design of A514 steel tension members. The use of higher allowable bolt shear stress values, while appropriate, satisfies only the A_n/A_s requirement and satisfactory member behavior is still not assured. The net to gross area requirement must be met in detailing the joint. This has been discussed briefly above. An alternative means of meeting this requirement would be to provide some type of upset end, such as an increase in plate width or thickness, in the region of the joint. This means simply that more material would be provided in the region of the connection. The net section area of the upset end would be established at 87 percent or more of the gross area of the main member. Allowable stresses could be set at 60 ksi in the plate (main member) and at 40 ksi in A490 bolts and 30 ksi in A325 bolts. The use of upset ends would then be required for joints up to 60 in. long when A490 bolts were used and up to 65 in. long when A325 bolts were used. Joints of greater lengths than these could be fabricated in the usual manner, that is, holes for the fasteners would be removed from the main member cross section. For convenience, the figure of five feet might be established as the "cut-off" point for both fastener types.

If this approach were used, the suggested stress levels appear to be optimum. Although either an increase in allowable shear stress in the fasteners or a decrease in the allowable tension stress in the plate would reduce the length over which such joint reinforcement would be required, the benefits are small. The factor of safety against shear failure in the fasteners is probably at a minimum desirable value at the 40 ksi (A490 bolts) and 30 ksi (A325 bolts) levels. A decrease in allowable stress in the gross section of the member from 60 ksi to 50 ksi would mean that 16 percent more plate material would be required over the length of the member. The reduction in member stress does reduce the length required for joints in which upset ends are needed to 40 in. when A490 bolts are used and to 60 in. when the fasteners are A325 bolts. Although extra fabrication cost must be included, the material saved by this reduction in allowable member stress occurs only over a relatively short length, that of the joint.

The additional cost involved in providing some type of upset end is difficult to evaluate. In certain cases, however, it may be fairly inexpensive. Since constructional alloy steel is weldable, the situation may often arise in which shop fabrication will be done by welding and field connections made using high strength bolts. If the member were composed of one or more plies of plate, the upset end could be provided by welding plates of the same thickness but of greater width to the ends of the main member. When rolled shapes rather than plates are used, the provision of upset ends will be more complicated. They can be provided in essentially the same manner, however.

The principal reason for the difficulties arising in plate failure type joints and their members is the low spread between the yield and tensile strengths of A514 steel. The situation would be considerably improved if the minimum specified tensile strength were increased from the present level of 115 ksi to, say, 125 ksi while the minimum specified yield strength is kept at 100 ksi. This change would reduce the A_n/A_g requirement from 0.87 to 0.80 and the minimum allowable fastener spacing would now be about 4-1/4 times the fastener diameter. The minimum allowable spacing presently suggested in building codes is typically three fastener diameters.[20]

A change in material properties is probably an impractical solution from the point of view of the manufacturer. Moreover, metallurgists consider that it is already a difficult problem to provide a quenched and tempered steel with a yield - tensile ratio less than about 0.90.

EXPERIMENTAL VERIFICATION OF THE THEORY

A considerable amount of experimental work has been carried out in connection with this theoretical assessment of constructional alloy steel members. A series of eight tests of large A514 steel joints using A490 bolts was completed in late

[20] Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings, American Institute of Steel Construction, New York, New York 1963

1966.[6] A pilot series of four smaller joints was also reported at that time. More recently, a test program of A514 joints using A325 bolts as the fasteners has been completed.[21] Here, six small joints and four large joints were tested.

These tests verified the validity of the theoretical studies upon which this paper has been based. They showed that the theoretical predictions are reliable, both for obtaining the ultimate load of a joint and for obtaining the distribution of load among the fasteners at loads less than ultimate. The maximum discrepancy between theoretical and actual values of ultimate joint load was about 6 percent.

SLIP BEHAVIOR OF A514 JOINTS

Although the main purpose of this paper is to present an assessment of the ultimate strength characteristics of A514 steel joints, the test program described above also provided information on the slip characteristics of such joints. As an aid in the design of friction-type A514 joints, this information is included here also.

[21] Fisher, J. W., and Kulak, G. L., "Tests of Bolted Butt-Splices", Report No. 317.11, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania, December 1967

The slip behavior of bolted joints is customarily defined on the basis of a "slip coefficient" (K_s). This is defined as

$$K_s = \frac{P_s}{m\ n\ T_i}$$

where P_s is the slip load, m is the number of faying surfaces, n is the number of bolts, and T_i is the average clamping force per bolt. On this basis, slip coefficients were computed for 21 joints described above. These test joints covered a wide range of length and joint width. They included the two fastener types (A490 and A325 bolts) and the A514 plate came from four different rollings.

Variations in the magnitude of the slip coefficient appeared to be random, that is, they were independent of joint width or length, or magnitude of clamping force. The mean value of K_s for the 21 joints was 0.33 with a standard deviation of 0.04. The value of slip coefficient most commonly specified[20] for joints with clean mill scale is 0.35. It should be noted that all of the A514 plate used in these tests was blast cleaned with No. 50 chilled steel grit. The resulting polishing of the surface probably results in a conservative value of the slip coefficient.

SUMMARY AND CONCLUSIONS

An examination of the behavior of mechanically fastened constructional alloy steel tension members has been made. This examination showed that the application of current design procedure results generally in joints whose potential failure mode is by fracture of the plates. Under usual fabrication techniques, this leads to unsatisfactory member behavior in that failure occurs through the net section of the joint before yielding starts on the cross section of the main member. A number of possible ways of overcoming this situation were explored.

The conclusions reached as a result of this study can be itemized as follows:

1. An accurate theoretical solution is available for predicting the ultimate load of bolted, butt splices of constructional alloy steel. The same theoretical development can be used to provide plate or individual fastener loads at levels less than ultimate.

2. Constructional alloy steel joints using high strength bolts do not produce desirable behavior in the members in which they are contained if the elements of the joint are designed according to current stress levels.

3. The use of higher allowable bolt stresses, in line with those suggested as a result of studies of mechanically fastened joints of other grades of steel, is suitable for use in A514 steel.

4. Satisfactory member behavior for a large and important class of joints can only be obtained by proper care in detailing of the joint. Specifically, the ratio of net to gross member area must be not less than about 0.87.

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TABLE 1 MECHANICAL PROPERTIES OF A514 STEEL

	Plates	Structural Shapes
Yield Strength, Ext. under load, min, psi	100,000	100,000
Tensile Strength, psi	115,000/135,000	115,000/140,000
Elongation in 2 in. min, %	18	18
Reduction of Area, min, %	3/4 in. and under-40 over 3/4 in. - 50	3/4 in. and under-45 over 3/4 in. - 55

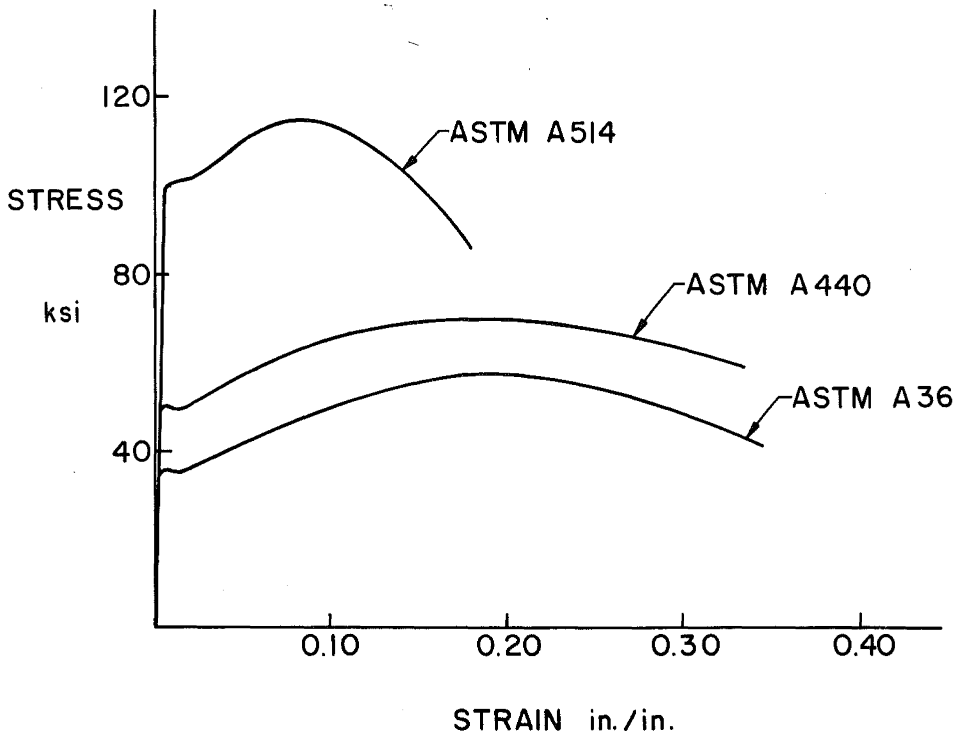

Fig. 1 Typical Stress - Strain Curves of Various Steels

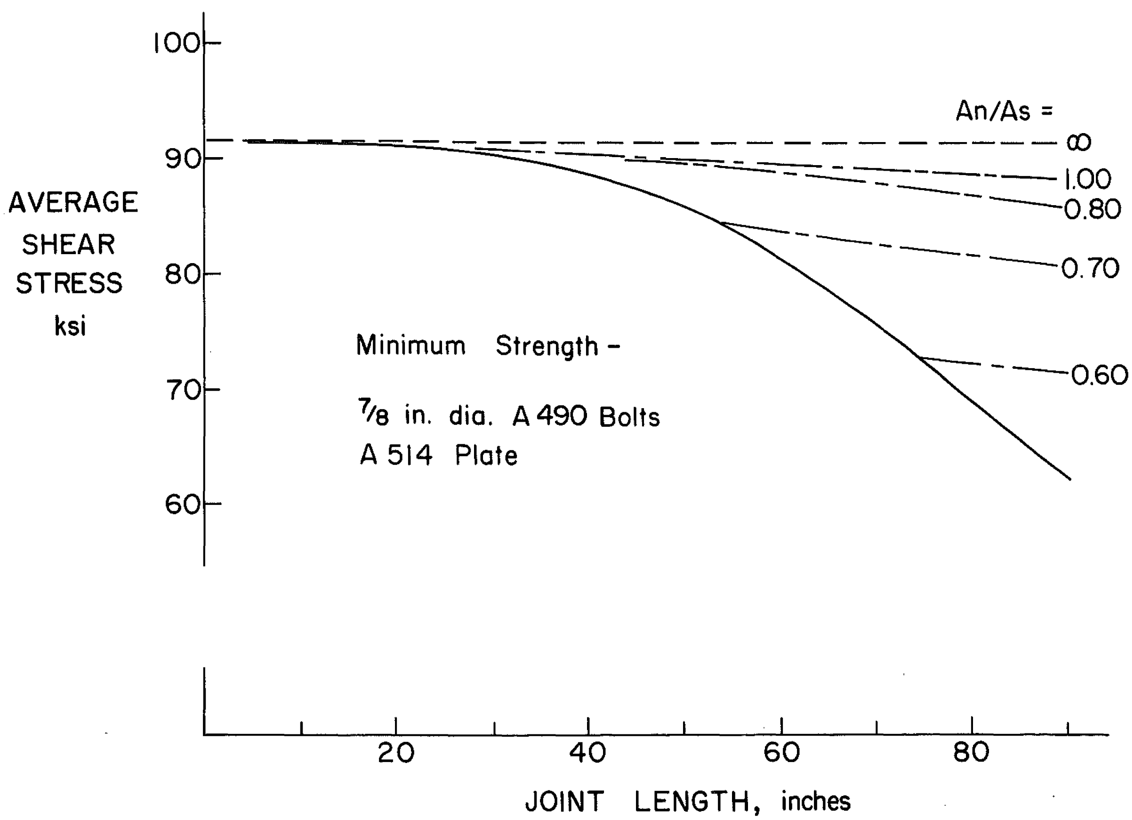

Fig. 2 Failure Mode Boundary (τ vs. L) - A490 Bolts

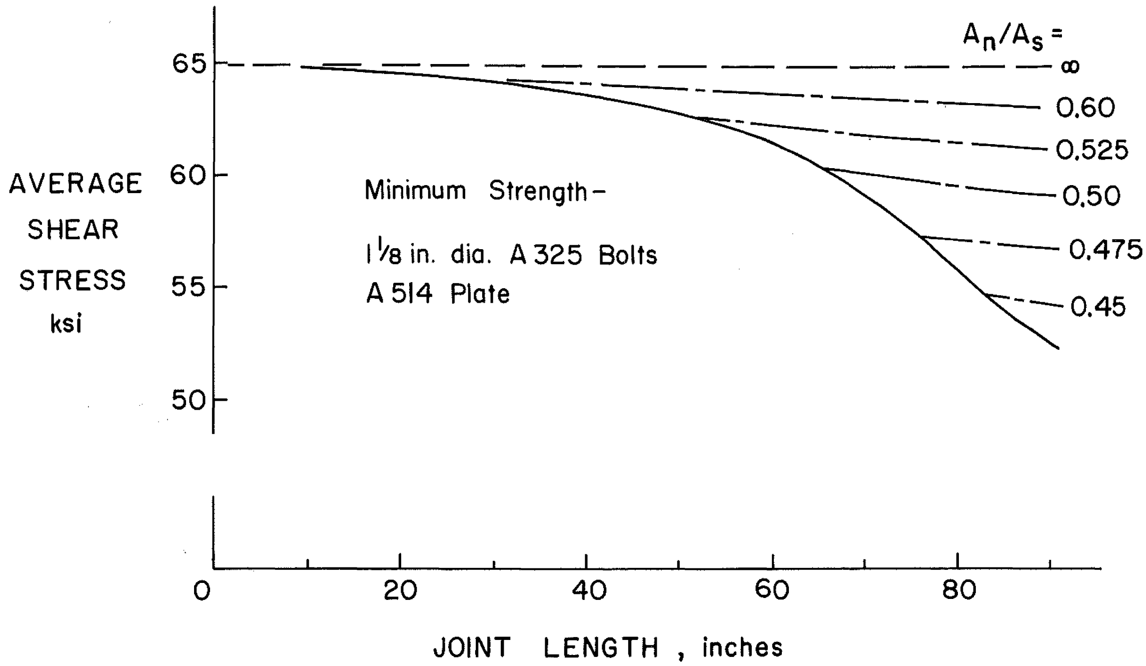

Fig. 3 Failure Mode Boundary (τ vs. L) - A325 Bolts

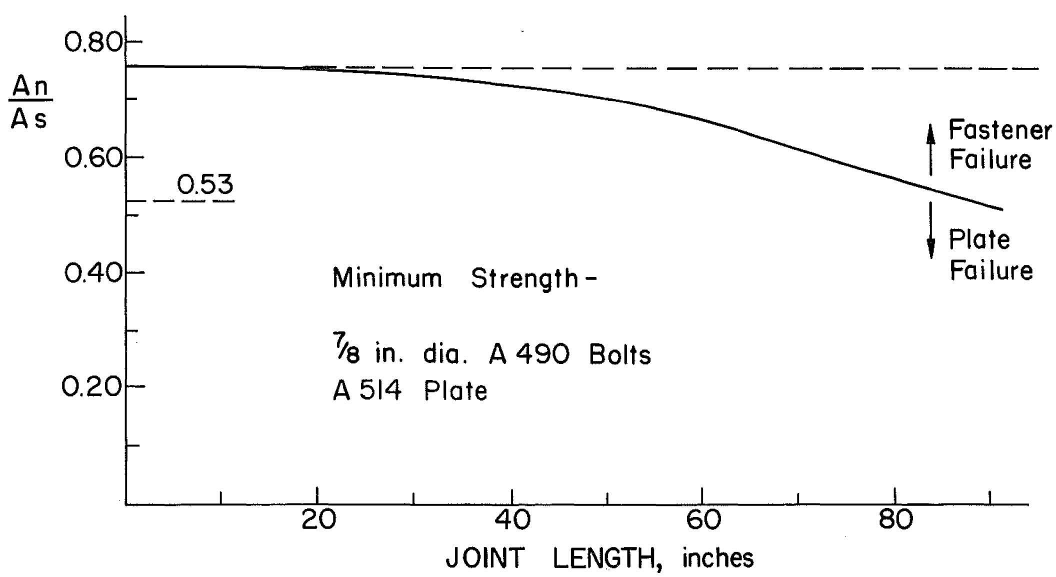

Fig. 4 Failure Mode Boundary (A_n/A_s vs. L) - A490 Bolts

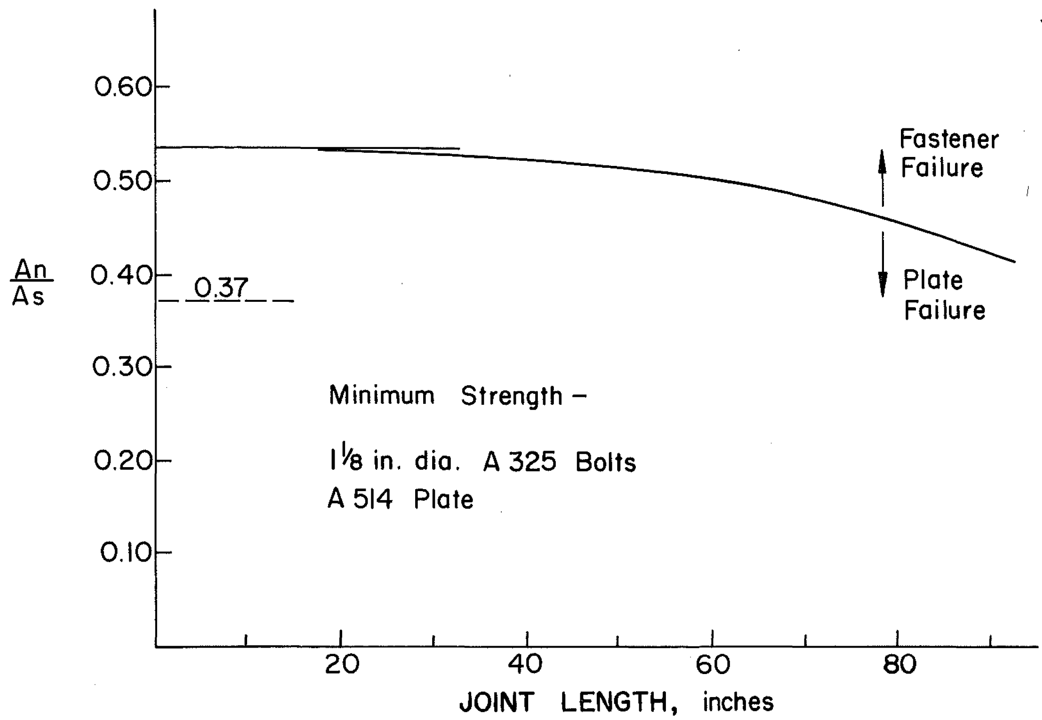

Fig. 5 Failure Mode Boundary (A_n/A_s vs. L) - A325 Bolts

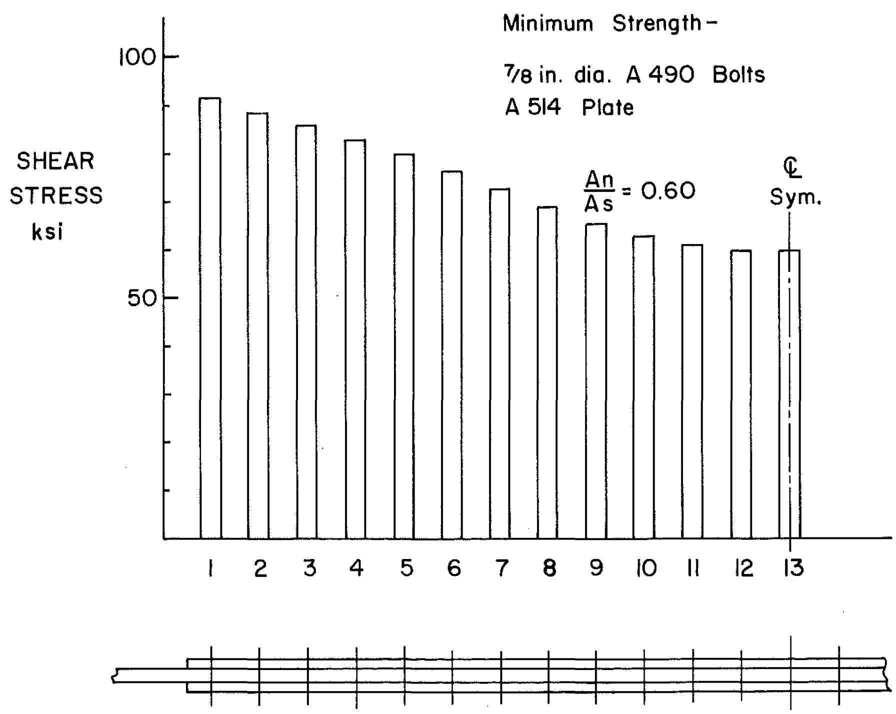

Fig. 6 Load Distribution to Fasteners of 25-Bolt Joint - A490 Bolts