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Light gage steel connections with high-strength, high-torqued bolts

George Winter

Light Gage Steel Connections with High-Strength, High-Torqued Bolts

Les assemblages de toles minces au moyen de boulons de haute résistance à grande torsion de serrage

Leichtmetall-Verbindungen mit hochwertigen, hochverdrillten Schrauben

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A. Introduction

The usual methods for designing bolted connections of conventional steel structures must be modified for application to thin-walled, light-gage steel construction. This is so because the ratios of bolt diameter to steel thickness in light-gage construction are generally much larger than those customary in conventional construction. In a previous paper [1] the writer has published the results of 547 tests of light-gage steel connections with ordinary "black" bolts. The results could be expressed in four simple equations for determining failure loads which, when used with an appropriate factor of safety, can serve as a safe basis for design.

It is the purpose of the present paper to report and evaluate the results of 476 additional tests made on similar connections, but with high-strength, high-torqued bolts of the kind which has come into use in the U. S. A. during the last five years. These tests were undertaken in order to investigate in what manner such connections would perform differently from those made with ordinary bolts, and what possible advantage could be gained from using these special bolts in light-gage, thin-walled steel construction.

B. Results of Previous Tests on Connections with Ordinary Bolts

As was indicated in the previous paper [1], four distinct kinds of connection failure were observed which are designated by I, II, III, and IV on figure 1. In addition, some specimens failed in a combination of two of these four types, such as the specimen designated by I—III on that figure. The four types of failure and the equations which determine the loads at which these failures occur are as follows:

Type I. Longitudinal shearing of the sheet along two almost parallel planes whose perpendicular distance equals the bolt diameter. This failure occurs for relatively short edge distances e (see fig. 2a) and takes place at a shear stress τ_s on the two shear planes approximately equal to

$$\tau_s = 0.7\,\sigma_y \tag{1}$$

where σ_y = yield stress of sheet steel. Consequently, the load P_f at which this failure will occur can be determined with sufficient accuracy from

$$P_f = 1.4\,e\,t\,\sigma_y \tag{1a}$$

where t = sheet thickness.

Type II is somewhat similar to Type I but occurs for long edge distances e. In the initial stages the bolt simply cuts through the sheet which causes shearing along two somewhat inclined planes. During this stage the sheet material piles up in front of the bolt. When the edge distance has been reduced in this

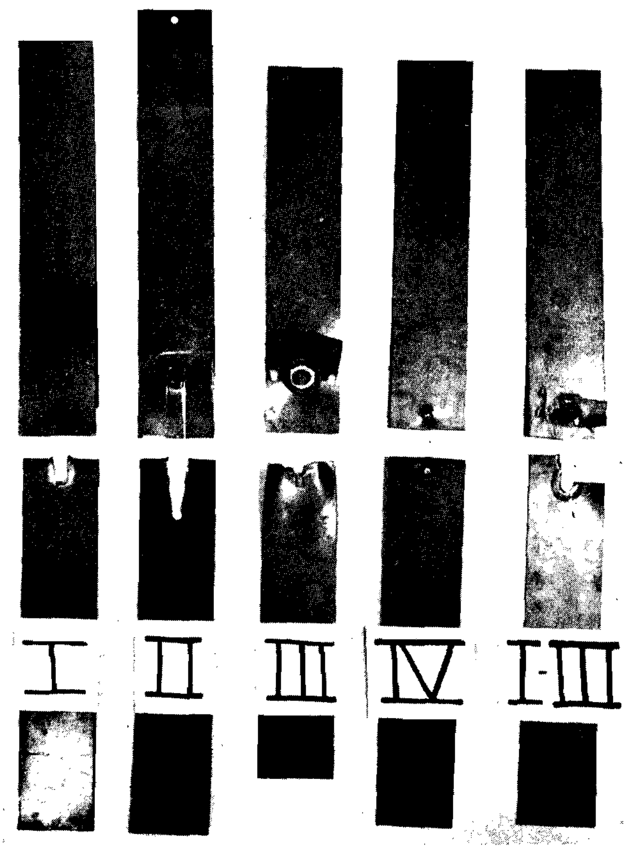

Fig. 1. Types of failure.

manner, the last stage of this process becomes identical with that of Type I. It was found that this type of failure depends on the bearing stress σ_b and that a conservative expression for this bearing stress is

$$\sigma_b = 4.9\,\sigma_y \tag{2}$$

where d = bolt diameter. Consequently, this type of failure is predicted by the expression

$$P_f = 4.9\,d\,t\,\sigma_y \tag{2a}$$

where d = bolt diameter. Equating the right sides of eqs. (1a) and (2a) it is found that eq. 1a governs as long as $e/d \leqq 3.5$, and that eq. (2a) merely sets a fixed upper limit for eq. (1a), when e/d exceeds 3.5.

Type III. Transverse tearing of the sheet at the section reduced by the bolt hole. It was found that the stress on the net section at which this type of failure occurs can be obtained from

$$\sigma_{net} = (0.10 + 3.0\,d/s)\,\sigma_t \leqq \sigma_t \tag{3}$$

where s is as indicated on figure 2a or, for multi-bolt connections, the center-line distance perpendicular to the direction of force between adjacent bolt holes, and σ_t = ultimate tensile strength of sheet steel. Consequently, for this type, the failure load per bolt is

$$P_f = (s - d)\,t\,\sigma_{net}. \tag{3a}$$

It is seen from eq. (3) that $\sigma_{net} < \sigma_t$ when $s/d > 3.33$. This indicates that for large s/d-ratios there is a definite effect of stress concentration which is not fully compensated by plastic flow and which reduces the strength of the connection below that given by the tensile strength of the steel. Eq. (3) was found to be satisfactorily accurate for single-shear connections and somewhat conservative for double-shear connections.

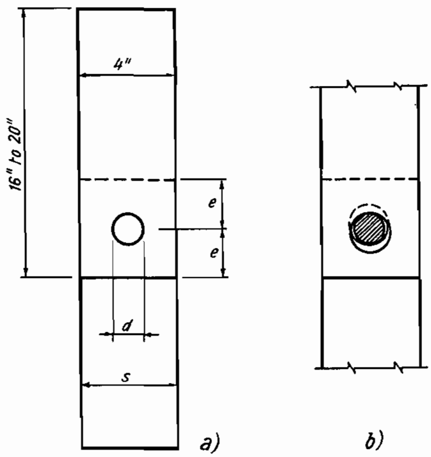

Fig. 2. Test specimens.
a) Dimensions and designations. b) Initial alignment.

Type IV. Shearing of bolt. It was found that shearing of the bolts occurs when the shear stress τ_b on the area at the root of the thread is approximately

$$\tau_b = 0.60\,\sigma_{tb} \tag{4}$$

where σ_{tb} = ultimate tensile strength of the bolt. Consequently, for this type of failure the load per bolt is

$$P_f = 0.6\,\sigma_{tb}\,A_{root} \tag{4a}$$

where A_{root} is the cross-sectional area of the bolt at the root of the thread. (In all tests the bolts were threaded over their full length so that shearing occurred in the threaded part.) Eq. (4) was found to be satisfactorily accurate for double-shear connections and somewhat conservative for single-shear connections.

Connection Slip. In these tests the hole diameter exceeded the bolt diameter by $^1/_{32}$ in. for the $^1/_4$ in. and $^3/_8$ in. bolts, and by $^1/_{16}$ in. for bolts with diameters from $^1/_2$ in. to 1 in. which corresponds to customary practice. By means of autographic slip recorders it was found that if normal tightening was applied to these bolts, slip equal to the hole clearance (designated as "clearance slip") would occur at loads which represented small fractions of the failure load. Once bearing is established by such clearance slip, further slip occurs only when the material begins to deform noticeably (designated as "deformation slip"). To judge the practical effect of such deformation slip, a somewhat arbitrary criterion of $^1/_{16}$ in. was adopted as representing a permissible limit of such slip in many, though by no means all, practical applications. A study of all pertinent tests showed that if eqs. (1) to (4) are used for design purposes with a safety factor of 2.5, a deformation slip of $^1/_{16}$ in. at the design load will be exceeded in only 2.4% of all cases. For a safety factor of 2.25 this percentage increases to 8.7%, and for a safety factor of 2.0 it increases to 17.4%. It is seen, therefore, that in such connections with ordinary, "hand-tight", "black" bolts a certain amount of connection slip is inevitable at design loads. While such slip is harmless in many applications, there are cases, such as knee braces of frames, splices of arch ribs, etc., where even such a limited amount of slip may be objectionable.

C. Purpose of Tests with High-Strength, High-Torqued Bolts

Such bolts and their use in structural connections [2] differ in two respects from those of ordinary black bolts:

a) The material from which these bolts are made has about twice the tensile strength of ordinary bolts. (According to the ASTM Standard A307—53T the minimum tensile strength of ordinary machine bolts is 55,000 psi, whereas according to ASTM A325—53T the minimum for high strength (quenched and tempered) bolts is 120,000 psi. for bolts up to $^3/_4$ in., and 115,000 psi for $^7/_8$ in. and 1 in bolts).

b) The nuts of high-strength bolts are torqued to prescribed amounts which results in a minimum bolt tension of 90% of the proof load of the bolt [2]. The proof load corresponds approximately to the proportional limit of the bolt material, and is defined as the load producing a tensile stress of 85,000 psi for bolts up to $^3/_4$ in., and 78,000 psi for $^7/_8$ in. and 1 in. bolts. (This stress is referred to the "stress area" of the bolt which is an imaginary area slightly larger than the root or net area, but considerably smaller than the gross area.) Accordingly, it was the purpose of these tests to investigate:

1. Whether the large connection friction (caused by (b), above) has any beneficial effect on the ultimate load of the connection in the case when such ultimate load is determined by failure of the sheet material (failure Types I, II, and III, above).

2. To what degree the higher tensile strength (see (a), above) results in an increased shear strength of the connection (failure Type IV, above).

3. To what degree the large connection friction can be utilized to reduce or prevent that connection slip which, as has been discussed, cannot entirely be eliminated with ordinary bolts.

As a relatively minor side issues it was also desired to investigate:

4. Whether the sheet failure formulas (eqs. (1), (2), and (3) are reliably applicable to the relatively thick sheets (8, 10, and 12 ga.). In the previous tests [1], for these thicknesses, shear failure of the bolts had often prevented sheet failure to develop, so that information on failure of Types I, II, and III was somewhat inadequate for these large thicknesses.

D. Scope, Type, and Method of Tests

The connection friction mentioned in 1. and 3., above, obviously depends not only on the bolt tension, but also on the character of the faying surfaces, since this character determines the magnitude of the coefficient of friction. For this reason, three types of faying surfaces were investigated: bare steel as received from the steel mill, galvanized steel, and shop painted steel. The latter two surface conditions are the ones which are used almost exclusively in light-gage, cold-formed steel construction. The tests on bare steel were made for two reasons: On the one hand, in high-strength bolted conventional steel construction the use of bare faying surfaces is recommended in certain situations to increase friction [2]. It was desired to investigate whether such an approach, even though of doubtful practicability, would be beneficial in light-gage construction. Also, for the purposes numbered 1. and 4., above, it was essential that tests be made on the same steels which had been used in the previous investigation [1], which steels were bare.

All tests were made on specimens of the type shown on figure 2a, both in single shear (single sheets) and in double shear (one single sheet connected to two sheets, one on each side). Sheet thicknesses ranged from 20 ga. (nominally 0.036 in.) to 8 ga. (nominally 0.164 in.) and bolt diameters from $^1/_4$ in. to 1 in.

The table below gives the ranges of physical properties of the various types of sheet steel used.

Table 1. Strength Properties of Sheet Steels

Steel	Lower yield point, psi minimum	Lower yield point, psi maximum	Tensile strength, psi minimum	Tensile strength, psi maximum
Bare, normal strength	26,000	36,600	41,150	49,500
Bare, high strength	46,750	59,500	68,000	76,975
Painted	36,480	46,200	54,660	63,750
Galvanized	41,320	45,500	53,650	66,330

Elongation in 2 in. ranged from 26 to 42 percent.

Tensile tests of bolts were made only on bolts of those diameters for which shear failures were obtained in the connection tests. The average results of five tests of each of these diameters are given in table 2.

Table 2. Tensile Strength of Bolts

Diam. in.	Strength psi. on root area	Strength psi. on stress area
$^1/_4$	171,400	145,000
$^3/_8$	137,600	123,000
$^1/_2$	155,800	139,000

It is seen that these values exceed the specified minimum value of 120,000 psi. on the stress area.

Nuts were tightened with calibrated torque wrenches to the following values

Table 3. Torques

Diam., in.	Torque, ft-lbs.
$^1/_4$	11
$^3/_8$	37.5
$^1/_2$	95
$^5/_8$	190
$^3/_4$	335
1	750

These torques are about 5% larger than those recommended [2] to obtain the minimum required bolt tension.

In addition to loads, the only other measurements which were made were those of connection slip. For this purpose, an autographic slip gage was used which has been described previously [1].

Specimens were assembled in such a manner that the bolt was so located in regard to the oversize holes as to permit a maximum amount of connection slip before bearing of the sheets was established on the bolt (see fig. 2b).

E. Ultimate Strength, Sheet Failure (Types I, II, III)

Two different kinds of behaviour, with minor variations, were observed in these tests, which are illustrated by the two load-slip graphs of fig. 3. In most of the tests the holding friction was overcome and the connection slipped into bearing at a load noticeably below the ultimate strength of the connection, as evident from the graph of fig. 3a. However, in about one-tenth of the tests the largest load reached was that which caused initial slip to occur (fig. 3b). In these cases, even though the load usually showed some increase after bearing between bolts and sheet was established, sheet failure occurred at a load below the initial slip load. Since this section of the present report deals with connection strength as governed by sheet failure, only those tests will be included which failed in the first of the two described modes, that is, where initial slip into bearing occurred at a load lower than the ultimate load of the connection.

Type I and *Type I–III* failures are plotted on figs. 4 and 5, for single and double shear, respectively. For reasons evident from the preceeding discussion of eqs. (1) to (2a), only specimens with $e/d \leqq 3.5$ are included in these figures. The solid line represents eq. (1a), and the two broken lines correspond to values 20% larger and smaller respectively, than those given by eq. (1a).

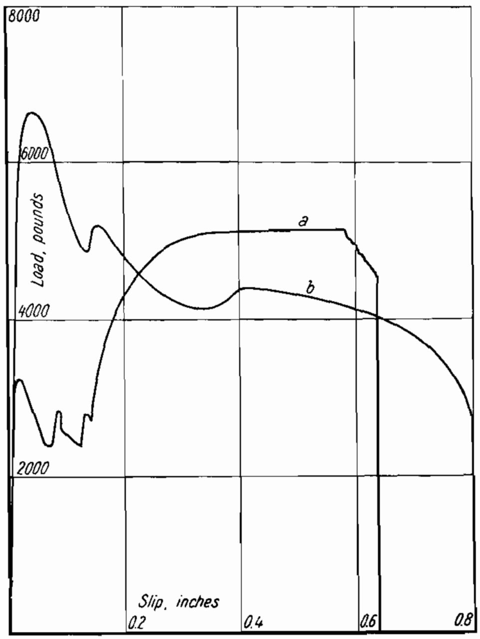

Fig. 3. Typical load. Slip autographs.

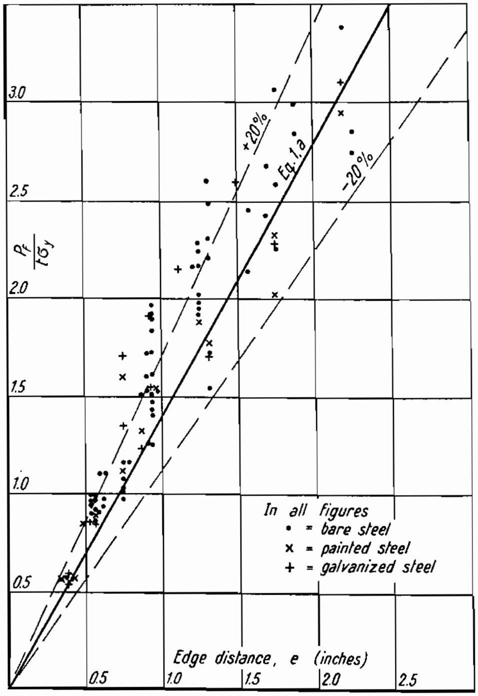

Fig. 4. Type I and I—III failures, single shear.

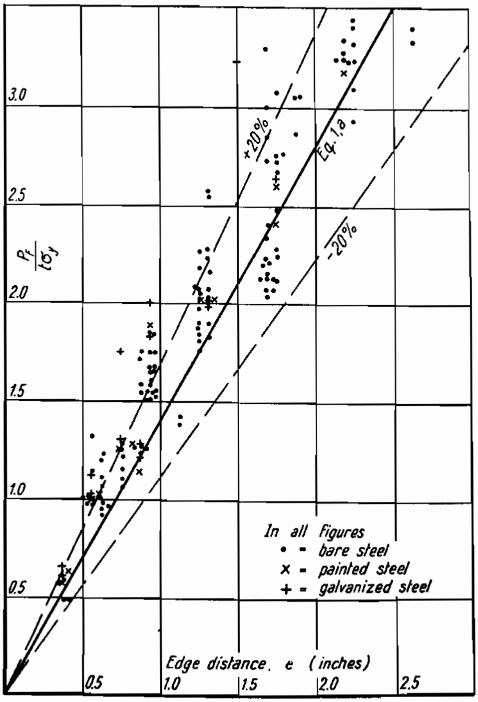

Fig. 5. Type I and I—III failures, double shear.

It is seen eq. (1a) predicts the strength in a satisfactory and somewhat conservative manner.

Type II and *Type II–III* failures are plotted on fig. 6. This figure is drawn in terms of e/d, so that Type I and Type I–III failures also can be shown on that figure. For the latter two types only the ranges of test results are shown, since plotting of all individual tests on one figure would have been impossible. The inclined solid straight line up to $e/d = 3.5$ again represent eq. 1a, the horizontal one eq. (2a) and the broken lines the $\pm 20\%$ limits. The change of behaviour at about $e/d = 3.5$, previously discussed, is evident from the figure. It is also seen that in spite of the rather wide scatter eq. (2a) represents the test results for $e/d \geqq 3.5$ reasonably well and conservatively.

Type III, Type III–I and Type III–II failures are plotted on figs. 7 and 8, for single and for double shear. The figures also give the lines corresponding to eqs. (3) and (3a) and the $\pm 20\%$ limits. It is seen that these equations

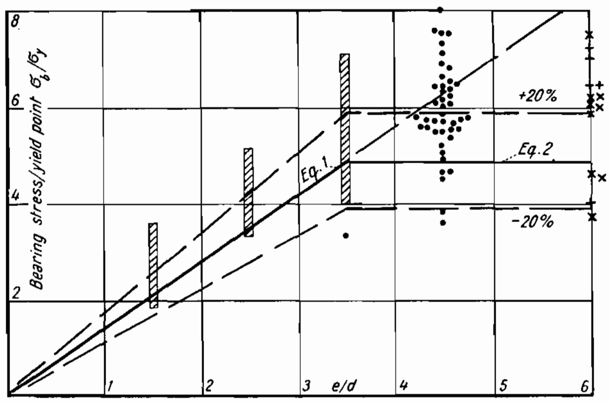

Fig. 6. Type I, I—III, II and II—III failures.

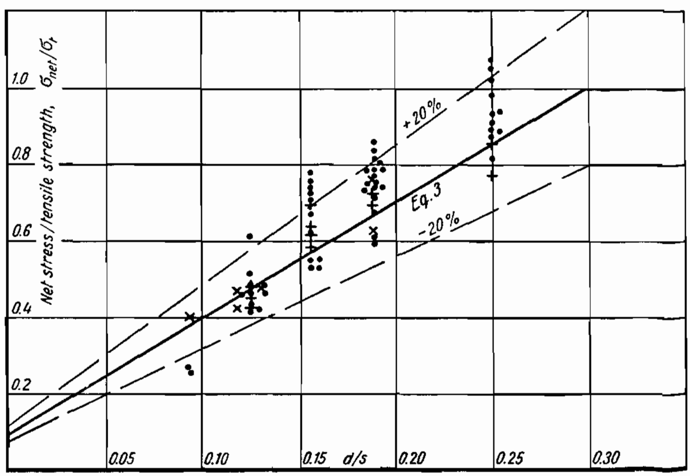

Fig. 7. Type III failures, single shear.

It is seen eq. (1a) predicts the strength in a satisfactory and somewhat conservative manner.

Type II and *Type II–III* failures are plotted on fig. 6. This figure is drawn in terms of e/d, so that Type I and Type I–III failures also can be shown on that figure. For the latter two types only the ranges of test results are shown, since plotting of all individual tests on one figure would have been impossible. The inclined solid straight line up to $e/d = 3.5$ again represent eq. 1a, the horizontal one eq. (2a) and the broken lines the $\pm 20\%$ limits. The change of behaviour at about $e/d = 3.5$, previously discussed, is evident from the figure. It is also seen that in spite of the rather wide scatter eq. (2a) represents the test results for $e/d \geqq 3.5$ reasonably well and conservatively.

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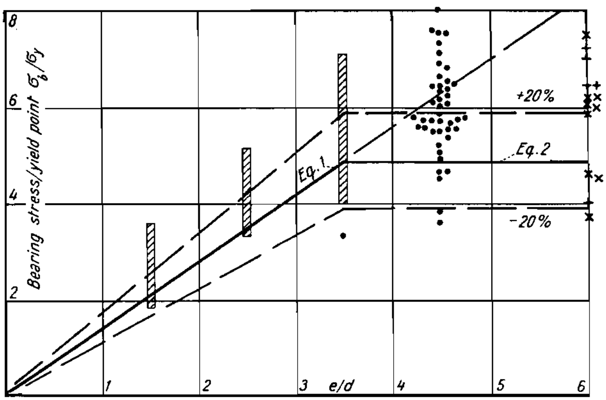
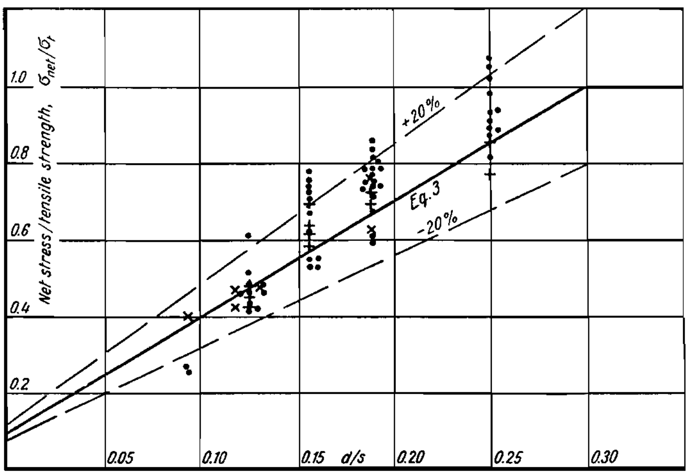


Fig. 6. Type I, I—III, II and II—III failures.



represent the results of the single shear tests very satisfactorily and that they give values which are somewhat too low for the double-shear tests.

It is seen, then, that the equations for sheet failure, derived from the earlier tests with ordinary bolts, represent satisfactorily the results of the present tests with high-strength, high-torqued bolts. A more detailed comparison of figs. 5 to 9 with the corresponding figures of the earlier paper [1] will show that, on the average sheet failure loads obtained with high-strength bolts were somewhat larger than those obtained with ordinary bolts, but by an amount not exceeding about 10%. It is doubtful whether, with the considerable scatter evident from the figures, this difference is significant and reliable.

The investigation included 56 tests with 8 ga. sheets (nominally 0.164 in.), 84 tests with 10 ga. (0.134), and 96 tests with 12 ga. (0.105 in.). The results of those of these tests which failed in the sheets showed the same degree of agreement with eqs. (1) to (3) as the tests on thinner sheets. This supplements the somewhat insufficient information on thicker sheets of the previous investigation [1] and indicates that these equations are satisfactorily confirmed for these larger thicknesses.

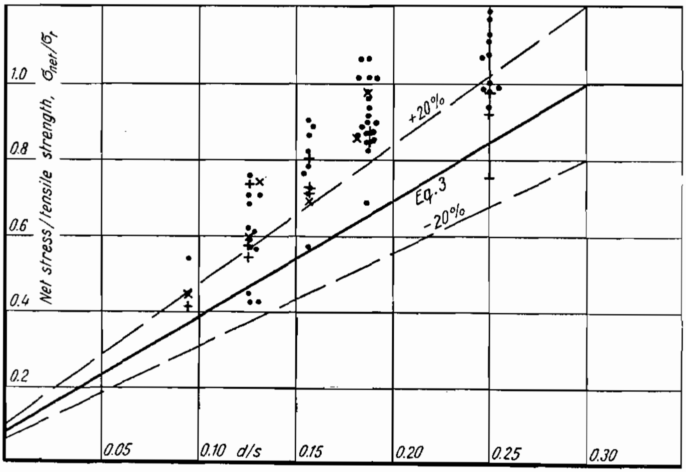


Fig. 8. Type III failures, double shear.

F. Ultimate Strength, Bolt Failure in Shear (Type IV)

In view of the high strength of these bolts, and the relatively small thickness of the connected sheets, shear failures were obtained only on bolts of $^1/_4$, $^3/_8$, and $^1/_2$ in. diameter. These test results are plotted on fig. 9, which also shows the straight line representing eq. (4) and the $\pm 20\%$ limits. It is seen that that equation, originally obtained from shear failures of ordinary bolts, represents the shear strength of high-strength bolts in a satisfactory manner.

From this one can draw the important conclusion that the shear strength of high-strength bolts, which is about twice that of ordinary bolts, can be utilized advantageously in such cases where, with ordinary bolts, the strength of the connection would be limited by bolt shear. This is generally the case for relatively small bolts used to connect members formed of relatively thick and/or high-strength steel. Eqs. (4) and (4a) can be used for determining the shear strength of high-strength bolts with the same degree of assurance as for the ordinary bolts from whose test performance they had been derived originally.

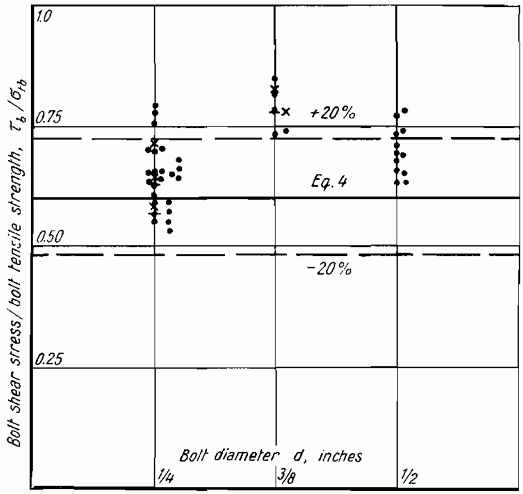


Fig. 9. Type IV failures.

G. Slip Loads

In light-gage steel construction it is generally not possible to achieve the close fit which can be obtained in conventional steel construction by using turned bolts in closely fitted reamed or drilled holes. As was pointed out, in view of the necessary hole clearances and of the relatively low and uncontrolled bolt tension, a certain amount of connection slip cannot be avoided with certainty when using ordinary, handtight bolts. It was one of the objects of the present investigation to determine to what extent the use of high-strength, high-torqued bolts results in improving this behaviour.

The friction force which resists connection slip depends, in addition to bolt tension, on the coefficient of friction between the connected surfaces. For this reason three types of surfaces were included in the programm: a) Specimens painted with a red oxide primer, applied by dipping, draining and baking for 10 minutes at 350° to 400° F. b) Galvanized specimens, with the coating of not less than $1^1/_4$ oz. per sq. ft. applied by continuous galvanizing. c) Bare steel specimens the details of whose surface conditions are discussed below in connection with the behaviour of these specimens.

Since all specimens were assembled with maximum hole clearance (see fig. 2b), definite connection slip of the order of twice the hole clearance was recorded by the autographic slip gage (see fig. 3a). The load at which such slip into bearing occurred is designated as P_s, the slip load.

Fig. 10 shows the upper and lower limits of the slip loads for painted specimens, separately for the single shear and the double shear specimens. It is seen that these loads fall into a reasonably narrow range and that they are not significantly lower for single than for double shear. This is so because the single shear specimens, like the double shear specimens, actually develop friction on two surfaces, namely on the interface between the two connected sheets and on the contact surface of washer and sheet.

Fig. 11 gives the same information for the galvanized specimens. The same observations can be made. Comparison of figs. 10 and 11 shows that the galvanized specimens slipped at somewhat lower loads than the painted ones.

Fig. 12 gives the same information for the bare steel specimens. It will be noted that the scatter, as expressed by the difference between upper and lower limits, is much greater for these than for the coated specimens. This is understandable from the manufacturing history of these steels, to wit: the thicker gages (8 ga. to 14 ga. inclusive) were hot rolled, pickled, and oiled, the thinner gages cold rolled and oiled. This is normal rolling procedure. In consequence, not only is there a difference between surface conditions of

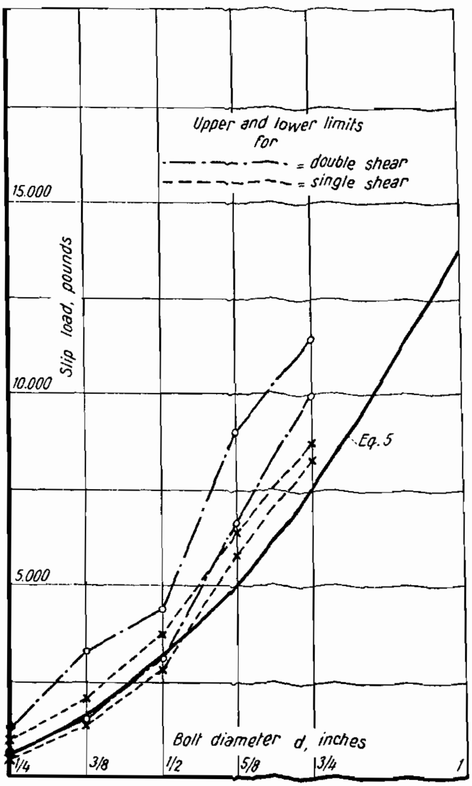


Fig. 10. Slip loads, painted steel.

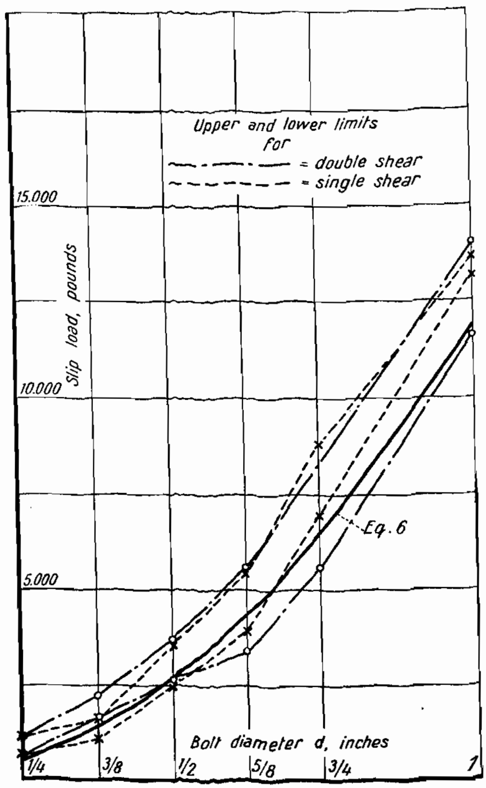


Fig. 11. Slip loads, galvanized steel.

hot versus cold rolled steel. More important, depending on the degree of preservation of the thin oil film at time of testing, various degrees of interface lubrication were inevitably obtained in these tests. Since exactly this same situation would obtain in construction (in such rare cases where bare steel would be used), no attempt was made to remove the oil before assembling the specimens.

It is seen, for all three types of faying surfaces, that the slip load increases consistently with bolt diameter, as would be expected. Considering a) that the applied torques were about 1.05 times the required minimum torques, b) that these minimum torques result approximately in a stress equal to 90% of the proof stress on the stress area (85,000 psi. for the smaller bolts and 78,000 psi. for the larger ones, a difference which can be neglected for the present purposes), the total tension force in the bolt should be approximately proportional to its stress area. If it is further assumed that for each type of faying surface, particularly for the coated specimens, the coefficient of friction is reasonably constant, the slip loads for the various diameters should also be approximately proportional to the stress areas. For practical purposes the stress area, which is not given in handbooks and does not correspond to a real or measurable dimension, is an inconvenient quantity. Since that area differs

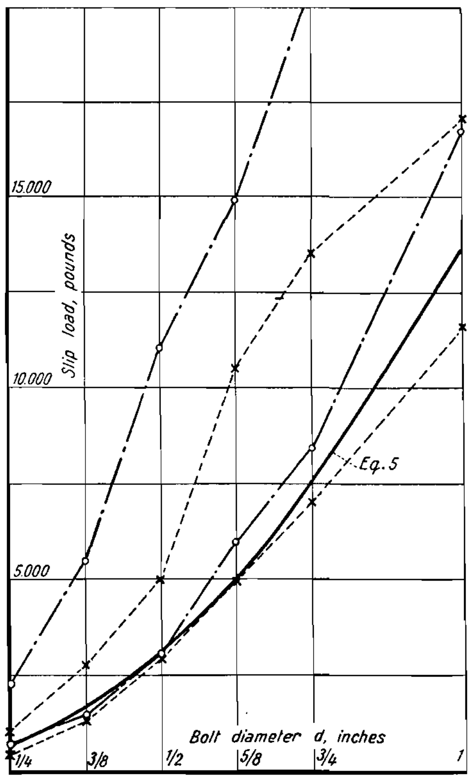


Fig. 12. Slip loads, bare steel.

only insignificantly from the root area (particularly in relation to the scattering evident from the test results), this line of reasoning suggests an attempt at correlating the slip load P_s with the root area A_{root}, or with the square of the root diameter d_{root}. In view of the scattering, a conservative expression which would follow roughly the lower bound of the zone of test results appears desirable.

Such a study has been made and resulted in the following two expressions for the slip loads:

For painted and for bare steel,

$$P_s = 25{,}000\, A_{root} = 19{,}600\, d^2{}_{root}. \tag{5}$$

For galvanized steel,

$$P_s = 21{,}500\, A_{root} = 16{,}900\, d^2{}_{root}. \tag{6}$$

The curves of eqs. (5) and (6) are shown on figs. 11 to 12 and are seen to be located satisfactorily close to the lower bounds of the zones of test results.

If, for reasons outlined above, it is assumed that on the average the tensile stress on the root area in these bolts was

$$1.05 \times 0.90 \times (85{,}000 + 78{,}000)/2 = 77{,}000 \text{ psi.},$$

then, considering that slip occurred in all cases along two surfaces, the approximate coefficients of friction are, for painted and bare specimens $0.5 \times \times (25{,}000/77{,}000) = 0.16$ and for galvanized specimens $0.5 \times (21{,}500/77{,}000) = = 0.14$. These values, which represent approximate lower limits, are not out of line with other information on friction coefficients.

The following practical conclusions can be drawn from the results of this evaluation of the measured slip loads:

1. If in a connection which was designed for ordinary, handtight bolts, high-strength, high-torqued bolts of the same diameter are substituted, connection slip at design loads will not occur provided that the connection has been designed with a factor of safety larger than, for painted or bare steel

$$0.6 \times 55{,}000/25{,}000 = 1.32$$

and for galvanized steel

$$0.6 \times 55{,}000/21{,}500 = 1.54.$$

This follows from eqs. (4a), (5), and (6) and from the fact that the specified minimum value of σ_{tb} for ordinary bolts is 55,000 psi. Since such low factors of safety are not used in connection design, it can be said that direct substitution of high torqued for ordinary bolts will eliminate connection slip.

2. As was pointed out in the last part of Section II of the present paper, for connections with ordinary bolts a factor of safety of about 2.5 appears desirable in order to limit connection slip to reasonable values [1]. It follows from 1., above, that a smaller factor of safety appears adequate for connections designed for high-strength, high-torqued bolts.

3. If it is desired to design connections with high-strength high-torqued

bolts in a manner which will assure adequate safety against failure and some margin against connection slip at design loads, a factor of safety of about 1.7 to 2.0 may be appropriate if applied to eqs. (1a), (2a), and (3a) (which refer to sheet failure), provided the design load is 20 to 25% smaller than the slip load as given by eqs. (5) or (6). In this case the shear resistance of the high-strength bolts will never control because, for these bolts, the slip load (eqs. (5) or (6)) is only of the order of one-third of the shear strength (eq. (4a)).

4. Conversely, the very large shear strength of these bolts can be utilized completely only in such connections where some slip at or below design loads is permissible.

5. Since slip loads for bare steel are frequently as low as for painted, and not significantly higher than for galvanized steel, there is no advantage in using bare steel for such bolted connections. This is due to the character of the surface of sheet and strip steel, in contrast with the dry, mill scale surface of hot-rolled sections and plates which develops a larger friction than painted surfaces.

Acknowledgement

Most of the tests were carried out, and the calculations for their evaluation made by Emin Veral, graduate research assistant, whose careful and competent work contributed greatly to this investigation. Others who assisted in this work are R. J. Brungraber, instructor; T. N. Chernak, graduate student; and E. S. Yueh, student. — This investigation is part of an extensive research project on light-gage, cold-formed steel structures sponsored at Cornell University by the American Iron and Steel Institute. To the members of the Institute's technical subcommittee (M. Male, chairman, F. E. Fahy, vice-chairman, B. L. Wood, consulting engineer) the author is indebted for valuable cooperation.

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2. Specifications for Assembly of Structural Joints Using High Strength Steel Bolts, Research Council on Riveted and Bolted Structural Joints, February 1954.

Summary

Bolted connections in cold-formed, light-gage steel construction perform somewhat differently from those of hot-rolled construction, in view of the small ratio of sheet thickness to bolt diameter. Previously published results by the author on such connections with ordinary bolts are briefly reviewed and four equations are given, referring to the four possible modes of failure; they permit the strength of such connections to be computed.

Next, results are presented in detail of 476 additional tests made with high-strength, high-torqued bolts of the kind increasingly used in steel construction in the U. S. A. It is shown that: a) the quoted four equations also apply to connections with such high-strength bolts; b) the high shear strength of such bolts often makes it possible to reduce the required number of bolts; c) in view of the large friction, connection slip at design loads can be eliminated by using such bolts; and d) a lower factor of safety is warranted for such connections than when using ordinary bolts.

Résumé

Les assemblages à boulons dans les charpentes en tôles minces se comportent en partie differemment de ceux des charpentes en profils normaux, par suite du rapport plus faible de l'épaisseur de la tôle au diamètre du boulon. On donne un résumé des résultats de 574 essais avec boulons ordinaires, publiés récemment ailleurs par l'auteur. Quatre formules correspondant aux quatre modes de destruction de ces assemblages permettent de calculer leur résistance.

On donne ensuite les résultats de 476 essais additionnels, qui ont été faits avec des boulons d'acier à grande résistance serrés à grande torsion, ainsi qu'ils sont employés de plus en plus généralement aux Etats-Unis pour les charpentes ordinaires. On voit que: a) les quatre formules citées ci-dessus s'appliquent également aux assemblages de ce genre; b) en vue de la grande résistance au cisaillement de tels boulons il est souvent possible d'en réduire le nombre; c) la grande résistance de frottement permet d'éliminer tout glissement d'assemblage sous poids normal; d) en calculant ces assemblages un coefficient de sécurité plus petit est admissible lorsqu'on utilise ces boulons au lieu des boulons ordinaires.

Zusammenfassung

Schraubenverbindungen im Leichtstahlbau verhalten sich teilweise anders als die im gewöhnlichen Stahlbau infolge des viel kleineren Verhältnisses der Blechdicke zum Durchmesser. Es folgt eine Zusammenfassung vor kurzem veröffentlichter Resultate des Verfassers von 574 Versuchen an Verbindungen mit gewöhnlichen Schrauben. Vier Formeln, die sich auf die vier Arten der Verbindungszerstörung beziehen, gestatten die Berechnung solcher Verbindungen.

Sodann werden die Resultate weiterer 476 Versuche gegeben, die mit hochwertigen, hochverdrillten Schrauben ausgeführt wurden, von derselben Art wie sie in den Vereinigten Staaten zunehmend im Stahlbau Verwendung finden. Es wird gezeigt a) daß die angeführten vier Formeln auch für solche Verbindungen gelten; b) daß die hohe Scherfestigkeit solcher Schrauben es oft erlaubt, deren erforderliche Anzahl zu vermindern; c) daß der hohe Reibungswiderstand es ermöglicht, mit solchen Schrauben Verbindungsverschiebungen unter Entwurfsbelastung zu verhindern; d) daß mit solchen Schrauben ein kleinerer Sicherheitsgrad zulässig ist als mit gewöhnlichen Schrauben.