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A study on behavior of steel joints that combine high-strength bolts and fillet welds

Heui-Yung Chang [*1] and Ching-Yu Yeh [2a]

[1] *Department of Civil and Environmental Engineering*, *National University of Kaohsiung*, *700*, *Kaohsiung University Rd*., *Kaohsiung 811*, *Taiwan*
[2] *Department of Civil Engineering*, *National Cheng Kung University*,*1*, *University Rd*., *Tainan 701*, *Taiwan*

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Abstract. In recent years, considerable attention has been paid to the research and development of high-strength steel plates, with particular emphasis on the enhancement of the seismic resistance of buildings and bridges. Many efforts have also been undertaken to improve the properties of high-strength bolts and weld materials. However, there are still different opinions on steel joints that combine high-strength bolts and fillet welds. Therefore, it is necessary to verify the design specifications and guidelines, especially for newly developed 1,400-MPa high-strength bolts, 570-MPa steel plates, and weld materials. This paper presents the results of literature reviews and experimental investigations. Test parameters include bolt strengths, weld orientations, and their combinations. The results show that advances in steel materials have increased the plastic deformation capacities of steel welds. That allows combination joints to gain their maximum strength before the welds have fracture failures. When in combination with longitudinal welds, high-strength bolts slip, come in contact with cover plates, and develop greater bearing strength before the joints reach their maximum strength. However, in the case of combinations with transverse welds, changes in crack angles cause the welds to provide additional strength. The combination joints can therefore develop strength greater than estimated by adding the strength of bolted joints in proportion to those of welded joints. Consequently, using the slip resistance as the available strength of high-strength bolts is recommended. That ensures a margin of safety in the strength design of combination joints.

Keywords: combination joints; high-strength bolts; slip and bearing conditions; fillet welds; crack angles

1. Introduction

High-strength bolts and welds are two important connecting elements in modern steel construction. In addition, recently developed high-strength steel does more than improve the mechanical properties of the connecting elements. It also calls detailed attention to the engineering applications. Öztekin (2015) studied the failure risk and reliability indices of distances describing bolt placement for high-strength steel connections by using the geometric properties, material properties, and design actions as random variables in a Monte Carlo simulation. Additionally, Yang and Lei (2017) conducted a constant-amplitude fatigue test on M20 and M30 high-strength bolts with 40-Cr material and established a design method for analyzing the stress concentrations and fatigue fractures of high-strength bolts in grid structures with bolt-sphere joints. Moreover, Nah and Choi (2017) performed high-strength clamping tests under laboratory conditions (with temperatures varying from 10° C to 50° C at 10°-C intervals). Nah and Choi also tested in outdoor environments for six years (with temperatures ranging from 11℃ to 34℃). Then, they combined the laboratory and outdoor results to revise their equations. As a result, the change in torque coefficient was modified as 0.2% for every 1℃, and the increment of tension was adjusted to 1.89% for every 1℃.

There is great interest in studying the behavior of steel connections, including the slippage of high-strength bolts. Sabbagh *et al*. (2013) conducted cyclic loading tests on six-beam column assemblies comprising cold-formed steel (CFS), curved flange beams, a support column, and a through plate. Their purpose was to describe the finite-element (FE) procedures for simulating the hysteretic-moment rotation behavior and failure deformations of bolted-CFS moment connections. In addition, Brunesi *et al*. (2014) presented FE modeling procedures for bolted top-and-seat angle components and connections for potential use in seismic-moment resistance frames. Special attention was placed on the top-and-seat angle components, which control the global response of the joints in terms of failure mechanisms, thereby limiting the displacement ductility capacity and dissipation energy capabilities of the entire resistance system. Their FE modeling approach gave detailed consideration to the influence of friction, the pretensions of bolts, prying, and the relative slippage of components through highly nonlinear contact elements.

There is also interest in applying high-strength steel plates and weld materials. Lian *et al*. (2017) carried out cyclic tests and shake-table tests on a 1:2 long scaled Y-

*Corresponding author, Ph.D., Associate Professor,
E-mail: hychang@nuk.edu.tw
[a] Graduate Student

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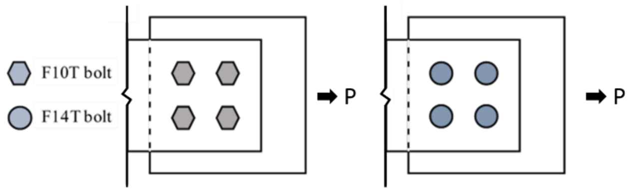
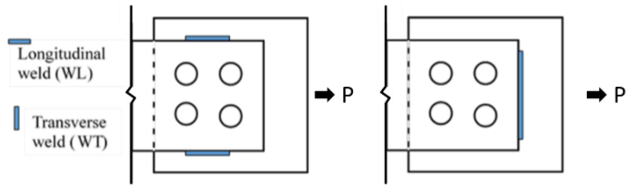
shaped eccentrically braced frame that was fabricated with high-strength steel members and conventional-strength steel links. The results indicate that the links can dissipate energy via shear deformation when experiencing seismic loads. Moreover, the maximum plastic rotation may increase to 0.08 rad and become more than that for the shear links in American Institute of Steel Construction (AISC) provision AISC 341-10. Kang *et al*. (2018) investigated the effects of the plate width-to-thickness ratio, column-slenderness ratio, and axis compression-force ratio on the ultimate load and deformation capacities of SM570 steel bridge box piers subjected to cyclic loading. Their analytical investigation generated new formulas and recommendations for evaluating the ultimate load and deformation capacities of steel-bridge piers, from SS400 and SM490 steel to SM570 steel.

However, there are still different opinions on steel joints that combine high-strength bolts and fillet welds. Manuel and Kulak (2000) indicated that the orientations of fillet welds and the bearing conditions of high-strength bolts can play key factors in determining the extent of load-sharing in combination joints. That caused the AISC to set restrictions on the application of high-strength bolts with welds (AISC 2005). Specifically, slip-critical types of high-strength bolts can only share loads with longitudinal welds in a joint. In the latest version (AISC 2016), the available bolt strength is determined based on the deformation compatibility. However, Eurocode 3 explains matters differently in its Design of Steel Structures – Parts 1-8 (CEN 2012). It states in its 2012 design of joints (preloaded Class 8.8 and 10.9) that bolts in connections designed as slip-resistant at the ultimate limit state (Category C in 3.4) may be assumed to share loads with welds. The Architecture Institute of Japan (AIJ) gives similar design recommendations (AIJ 2012).

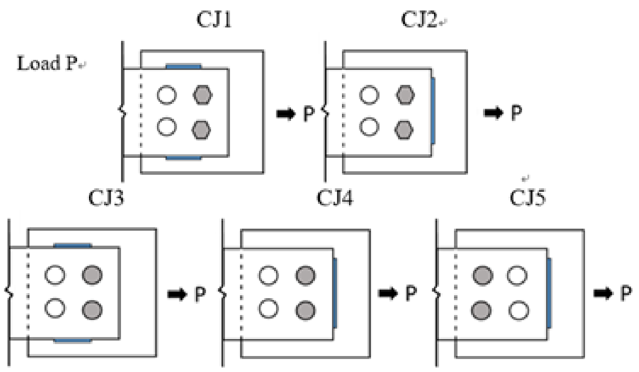
Consequently, it is necessary to verify the design specifications and guidelines, especially for the recently developed 1,400-MPa high-strength bolts, 570-MPa steel plates, and weld material. Therefore, this paper presents the results of detailed investigations into steel joints that combine high-strength bolts and fillet welds. In the first part of the paper, a concise but comprehensive literature review is provided on the historical development of design specifications and guidelines. In the second part, experiments are conducted with the aid of advanced measurements. That is, the pretensions of high-strength bolts and the losses are monitored using bolt-strain gauges. The measurements help confirm in more detail the extent of pre-tension loss and the bearing conditions of bolts in combination joints. The relative displacements of steel plates and high-strength bolts are measured using remote optical sensors. That allows studying the deformation capacities of steel welds and joints. The results obtained will help provide a better understanding of the behavior of high-strength bolts, fillet welds, and combination joints. That will also help advance applications for the seismic upgrading of existing steel structures.



(a) Bolted joints



(b) Welded joints



(c) Combination joints (CJ)

Fig. 1 Types of joint specimens

2. Literature review

2.1 Design standards

When retrofitting existing steel joints, one may replace high-strength bolts and/or add welds. To do so, design standards provide methodologies to evaluate the strength of steel joints that combine slip-critical high-strength bolts and welds (i.e., combination joints), as shown in Fig. 1.

Considerable attention has been paid to comparing the small amount of fillet welds and the limited deformation capacity with fracture failures. Manuel and Kulak (1998, 2000) indicated that, due to a lack of weld-deformation capacity, high-strength bolts cannot slip and provide less strength. This is especially true when bolts are used with transverse fillet welds on the same shear plane. They concluded that the methodology provided by the AISC specification (1999) may not appropriately evaluate the strength of combination joints. After that, Sato (2000) tested the joints of high-strength bolts and longitudinal fillet welds in an as-built condition. The AISC specification was then modified and set limitations (AISC 2005). Specifically, the slip-critical type of high-strength bolts can share load with fillet welds on a shear plane, but only if the final tightening of the high-strength bolts is done before the welds are deposited. Additionally, in combination with longitudinal welds, the available bolt strength cannot be greater than 50% of the bearing type. Moreover, in the latest version (AISC 2016), the available strength of high-strength bolts is determined based on the deformation compatibility with the fillet welds used on the same shear plane.

Masuda *et al*. (2001) conducted a series of tests on bolted joints, welded joints, and combination joints. The results showed that combination joints probably have a test-to-design strength ratio (or factor of safety) of more than 1.5. That provided experimental evidence supporting a recommendation made by the Architecture Institute of Japan (AIJ 2012). That is, the slip-critical type of high-strength bolts can share load with fillet welds on a shear plane provided the final tightening of the high-strength bolts is done before the welds are done. In addition, in combination with transverse and/or longitudinal welds, the slip-critical strength must equal the available bolt strength. Eurocode 3 (2012) was then established with design standards similar to those in the AIJ recommendation. As explained in its Section 3.9.3 on hybrid connections, preloaded Class 8.8 and 10.9 bolts in connections designed as slip-resistant for the ultimate-limit state (Category C in 3.4) may be assumed to share loads with welds, provided that the final tightening of the bolts is carried out after the welding is complete.

The tightening of high-strength bolts before or after welding is not considered a main issue in modern steel construction. That is because the construction uses steel plates with a thickness of at least 6 mm. That plate thickness eliminates the effects of welding heat, so no deformed plates affect the tightening of the high-strength bolts. As a result, except for special cases with extremely thin steel plates, it does not make any difference whether the final tightening of high-strength bolts is before or after the welds are deposited. Nevertheless, the available strength of high-strength bolts remains a key issue, especially for the application of high-strength bolts together with fillet welds for the seismic upgrading of existing steel structures. Therefore, the AISC specification focuses on the deformation compatibility of high-strength bolts and fillet welds used on the same shear plane. The AIJ recommendation suggests that the slip resistance of high-strength bolts should be taken as the available strength.That ensures a margin of safety for the strength design of combination joints.

2.2 Experimental studies

The evaluation of test data resulted in different conclusions for previous studies. That affected the development of design standards and guidelines, even though the studies had many similarities in terms of their test programs and specimen details. To illustrate these points, we re-analyzed the above studies referred to by the AISC and AIJ, and we compared the studies in a uniform way, as follows.

2.2.1 Test parameters and findings

Table 1 shows the specimen details of previous work. As can be seen, the variation in steel strength and plate thickness is relatively small, and the combination is considered compatible. A joint specimen is composed of two lap plates connected to a main plate by means of high strength bolts and/or fillet welds (see Fig. 1). As also can be seen there, the leg size is small, when compared to the plate thickness. Each joint is tested in direct tension using a material testing machine system. The load applied is directly measured using the internal load cell. The relative displacement of the lap plates with respect to the main plates can also be measured using the added diagauges. The strength and deformation capacities of fillet welds can vary significantly depending on strength of weld material and total amount of welds (i.e., the combination of weld length and leg size). The small amounts of fillet welds have fracture failures at small deformations, and determine the strength of combination joints. The test stops after the fracture of fillet welds. The strength values and deformation capacities of combination joints are then compared in detail to each other.

Table 1 Specimen details from previous studies

(a) Steel plates		$*F_y$	$*F_u$	Lap plate	Main plate
Manuel and Kulak (1998)	A572 Gr. 50	345 MPa	450 MPa	19 mm	36.5 mm
Sato (2000)	A572 Gr. 50	345 MPa	450 MPa	19 mm	38 mm
Masuda *et al*. (2001)	SM490A	325 MPa	490 MPa	19 mm	25 mm

* F_y = yield strength; F_u = tensile strength

(b) High-strength bolts		F_u	Size	Clearance	$**\mu$	Layout
Manuel and Kulak (1998)	A325	840 MPa	M20	1.5 mm	0.33	2×2
Sato (2000)	A325	840 MPa	M20	1.6 mm	0.33	2×2
Masuda *et al*. (2001)	S10T	1,000 MPa	M16, M20	1.5 mm	0.45	1×1, 1×2, 1×3

** μ = slip coefficient

(c) Fillet welds		F_u	Leg	WL length	WT length
Manuel and Kulak (1998)	E48018-1	482 MPa	6 mm	140 mm × 4	260 mm × 2
Sato (2000)	E48018-1	482 MPa	6 mm	140 mm × 4	-
Masuda *et al*. (2001)	YGW11	592 MPa	5 mm	50 mm × 4, 100 mm × 4	50 mm × 2, 100 mm × 2

Research by Manuel and Kulak (1998)

Manuel and Kulak (1998) performed a series of tests using tension-lap splices that had bolts and welds on the same shear plane. Each joint contained four 20-mm diameter ASTM A325 bolts in combination with fillet welds having 6-mm nominal leg size (E48018-1 filler metal). The fasteners were placed in standard holes with a clearance (△) of 1.5 mm. In addition, ancillary tests were carried out to determine the load vs. deformation characteristics of the fastener components. The ultimate strength of combination joints was then predicted by using the load versus the deformation characteristics (i.e., the deformation compatibility) of the individual components of the joints and by making an estimate of the frictional resistance present. The following results were obtained:

(1) Longitudinal welds + bolts (various conditions of bolt bearing and bolt pretension); predicted load/actual test load = 0.98 (8 cases)
(2) Transverse welds + bolts (various conditions of bolt bearing and bolt pretension); predicted load/actual test load = 1.02 (6 cases)
(3) Both longitudinal and transverse weld + bolts (various conditions of bolt bearing and bolt pretension); predicted load/actual test load = 1.09 (6 cases)

For the total 20 tests, the ratios of predicted ultimate load to test ultimate load has a mean of 1.03 with a standard deviation of 0.06.

Research by Sato (2000)

Following the work by Manuel and Kulak (1998), Sato (2000) tested the joints that combine high strength bolts and longitudinal fillet welds in the "as-built" condition. The fasteners used in the test joint consisted of four 20 mm A325 bolts, placed in 20.6 mm holes, with 6 mm longitudinal welds deposited using E48018-1 welding electrode. The plate material used for the test specimens was ASTM A572 Grade 50 steel. The bolts in the test joints were pretensioned using the turn-of-nut method. To determine the pretension force, the turn-of-nut method was calibrated using bolts from the same lot as those used in the test joints. The mean pretension measured in this way was 183 kN per bolt. The longitudinal fillet welds were deposited after the bolts had been pretensioned. All specimens were nominally the same, except for the bearing condition of the bolts, which was random. For the total 19 tests, the strength ratio has a mean of 1.02 with a standard deviation of 0.06.

Research by Masuda et al. (2001)

Masuda *et al*. (2001) also performed a series of tests using tension-lap splices that had bolts and welds on the same shear plane. Each connection contained one to three Japanese Industrial Standard (JIS) S10T bolts that were 16 or 20 mm in diameter and combined with fillet welds of 5-mm nominal leg size (JIS Z 3312 YGW11 wire). The fasteners were placed in standard holes with a clearance of 2 mm. By using the same experimental setup and specimen details, the values for connection strengths and overall deformations were also tested for slip-critical bolted connections and fillet-welded connections. The strength values of the slip-critical and fillet-welded connections were added and compared with those of bolted-welded connections. The following results were obtained:

(1) Longitudinal welds + bolts (two types of bolts; one, two and three bolts in a line; two weld lengths; standard and small pitches; bolt pretension); predicted load/actual test load = 1.10 (23 cases)
(2) Transverse welds + bolts (two types of bolts; one or two bolts in a line; two weld lengths; bolt pretension); predicted load/actual test load = 1.06 (8 cases)
(3) Both longitudinal and transverse weld + bolts (two types of bolts; one or two bolts in a line; two weld lengths; standard and small pitches; bolt pretension); predicted load/actual test load = 1.19 (19 cases)

For a total of 50 tests, the strength ratio had a mean of 1.13 with a standard deviation of 0.10.

2.2 Strength ratios

Table 2 gives examples illustrating a re-analysis of joint strengths. From the total of 20 cases reported by Manuel and Kulak (1998), only the eight preloaded ones were

Table 2 Strength ratios of work by Manuel and Kulak (1998)

(a) Test-to-calculated strength ratios

Specimen	Test strength (kN)			Strength analysis	
	(1)	(2)	(3)	(4) = (1)+(2)	(3)/(4)
	Bolt	Weld	Combination	Calculated	S.R.
NPT-1	459	1383	1676	1842	0.91
NPT-2	459	1381	1685	1840	0.92
PPT-1	459	1478	2111	1937	1.09
PPT-2	459	1522	1965	1981	0.99
NPL-1	459	1224	1776	1683	1.06
NPL-2	459	1199	1706	1658	1.03
PPL-1	459	1206	2418	1665	1.45
PPL-2	459	1257	2428	1716	1.41

(b) Test-to-design strength ratios

Specimen	Test strength (kN)			Strength analysis	
	(1)	(2)	(3) = (1)+(2)	(4)	(4)/(3)
	Bolt	Weld	Combination	Test	S.R.
NPT-1	354	718	1071	1776	1.56
NPT-2	354	718	1071	1706	1.57
PPT-1	354	718	1071	2418	1.97
PPT-2	354	718	1071	2428	1.83
NPL-1	354	515	869	1676	2.04
NPL-2	354	515	869	1685	1.96
PPL-1	354	515	869	2111	2.78
PPL-2	354	515	869	1965	2.79

selected for further study. The specimens were designated with three capital letters and one number. The first letter was "P" or "N", which represented the bearing condition of the high-strength bolts at the beginning of the testing. That is, a "P" bolt was in a location in which no slip was possible ("Positive bearing"), while an "N" bolt was in a position such that maximum slip could take place ("Negative bearing"). The authors used "P" or "N" for the second letter. This told us whether a bolt was "Pretensioned" or "Not pretensioned". The last letter was "T" or "L", having been selected for "Transverse fillet weld" or "Longitudinal fillet weld". The number was "1" or "2". The "1" was for the first specimen for the same bearing and pretension conditions, while the "2" was for the second such specimen. The designations in the table are the same as in the original work, with "N" never appearing as the second letter.

The test strengths of combination joints were compared with the sums of bolt slip strengths and weld fracture strengths, as shown in Table 2(a). There were four specimens with transverse welds, and three of them had a strength ratio (S.R.) below 1.0. On the other hand, there were four specimens with longitudinal welds, and all of them had a strength ratio greater than 1.0. The test-to-design strength ratios were also compared and are given in Table 2(b). The nominal values of weld lengths and leg sizes were used for the calculations of the design strengths. The ratios were greater than 1.0 in all eight cases.

According to the AISC-LRFD specifications, the bolt design strength (R_{db}) can be calculated as follows

$$R_{db} = \emptyset R_{nb} \tag{1}$$

$$R_{nb} = \mu D_u h_f T_b n_{s1} n_{s2} \tag{2}$$

where

- $\emptyset$ = resistance factor (= 0.75);
- R_{nb} = nominal slip resistance of a high-strength bolt;
- μ = slip coefficient, where μ = 0.33 and 0.50 for clean mill-scale steel and blast-cleaned surfaces, respectively;
- D_u = ratio of the bolt pretension to the specified minimum (= 1.13);
- h_f = hole factor, where h_f = 1.0 for a regular-size hole;
- T_b = bolt pretension (= 0.7 bolt tensile strength);
- n_{s1} = number of shear faces (=2 for double shear);
- n_{s2} = number of bolts (see Table 1(a))

In addition, the weld design strength (R_d) can be calculated as follows

$$R_{dw} = \emptyset R_{nw} = \emptyset \sum r_{nw} \tag{3}$$

$$r_{nw} = F_w A_w \tag{4}$$

$$F_w = 0.6 F_{Exx} [1.0 + 0.5(sin\theta)^{1.5}] \tag{5}$$

$$A_w = 0.707 wl \tag{6}$$

Table 3 Strength ratios of previous studies (Manuel and Kulak 1998, Sato 2000, Masuda *et al*. 2001)

Reference	Test-to-calculatedS.R.		Test-to-design S. R.	
	*B+WT	B+WL	B+WT	B+WL
Manuel and Kulak (1998)	0.91-1.09	1.03-1.45	1.56-1.97	1.96-2.79
Sato (2000)	-	1.05-1.29	-	1.92-2.29
Masuda *et al.* (2001)	0.85-1.01	0.73-1.01	2.63-3.20	2.46-3.43

*B = high strength bolts; WT= transverse fillet welds; WL = longitudinal fillet welds

where

- $\emptyset$ = resistance factor (= 0.75);
- r_{nw} = nominal shear strength of a fillet weld;
- F_w = weld strength;
- A_w = effective weld area;
- F_{Exx} = tensile strength of weld material;
- θ = crack angle (= 45°);
- w = weld-leg size;
- l = weld length.

In this study, the design strengths were calculated in a way slightly different from that for the AISC-LRFD specifications. That is, the calculations did not consider the ratio of bolt pretension to the specified minimum (D_u = 1.13). The design strengths were calculated using the equations with the bolt numbers in Table 1(b) and tensile strength of weld material, weld-leg size and weld length in Table 1(c).

Table 3 summarizes re-analysis results for previous studies (Manuel and Kulak 1998, Sato 2000, Masuda *et al*. 2001). As can be seen in the table, in some cases, the test-to-calculated strength ratios became smaller than 1.0. This may have been true whether the high-strength bolts were used in combination with transverse or longitudinal fillet welds.

It is also clear from the table that the test-to-design strength ratios were greater than 1.0 for all previous work. Moreover, the design strength of combination joints was determined by taking the sum of the slip resistance of high-strength bolts and the fracture strength of fillet welds. In other words, using the slip resistance as the available strength of the high-strength bolts ensured a margin of safety when designing the strength of combination joints.

3. Experimental investigation

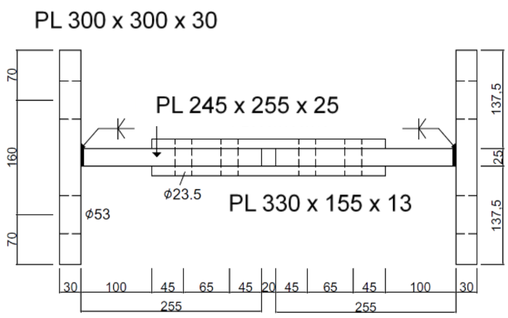
3.1 Specimen details

3.1.1 Joint types

As illustrated in Fig. 1, there were three types of joint specimens, that is, bolted joints, welded joints, and combination joints. The test parameters were the bolt strengths (i.e., F10T and F14T bolts) and the weld orientations (i.e., longitudinal welds [WL] and transverse welds [WT]). Each combination joint had a row of two bolts. In addition, four of the combination joints had the bolts placed far away from the central line. The other one had the bolts placed close to that line. Such arrangements allowed for more observation of the effects of the boundary conditions on the bolt pretensions and joint behavior. The two bolted joints, two welded joints, and five combination joints were all tested in the same manner. That made it easy to make comparisons and observations.

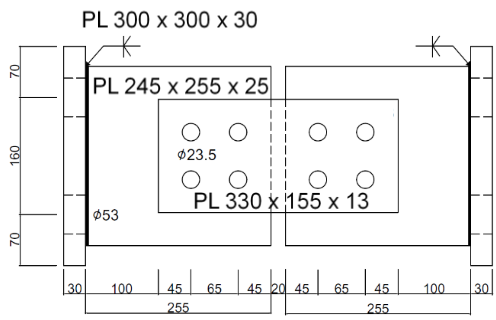
3.1.2 Steel plates

Fig. 2 shows the details of a joint specimen. As can be seen, the specimen was composed of three steel plates. Two lap plates (PL 330 × 155 × 13) were connected to one main plate (PL 245 × 255 × 25) with high-strength bolts and/or fillet welds. The main plates were then welded to a base plate (PL 260 × 260 × 30). The base plate was used to secure the specimen in the testing machine. In addition, a total of eight bolts were arranged in two rows and equally spaced in two parts of the joint specimen. That arrangement allowed the testing of the slip strength of four bolts with two shear planes. The interfaces were treated using a standard sandblasting process.



(Unit: mm)

(a) Side view



(Unit: mm)

(b) Top view

Fig. 2 Specimen details

Table 4 Analysis of steel plates' tensile strengths

	Yield of gross section	Fracture of net section
(a) Section areas	4,030 mm^2	2,808 mm^2
(b) Steel strengths	450 MPa	570 MPa
(c) Tensile strengths	1,813.5 kN	1,600.6 kN

Table 4 summarizes the calculations of the tensile strengths for the steel plates. The plate thickness and width of the two cover plates were 13 mm and 155 mm, respectively. See Fig. 2(a). In addition, there were two bolts in a row. Refer to Fig. 2(b). The bolts had a diameter of 22 mm, and the standard holes had a diameter of 23.5 mm. The areas of the gross section and net section were 4,030 mm^2 and 2,808 mm^2, respectively. For the SM570 steel that was used, the nominal values of the yield strength and tensile strength were 450 MPa and 570 MPa, respectively. It was then possible to calculate the tensile strengths of the connecting steel plates by taking the product of a section area and a steel strength. Consequently, the gross section had a nominal yield strength of 1,813.5 kN, while the net section had a nominal fracture strength of 1,600.6 kN.

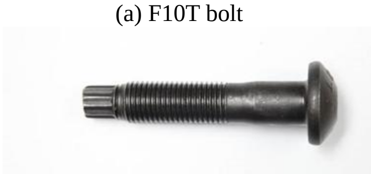
3.1.3 High-strength bolts

Fig. 3 shows the tested F10T and F14T high-strength bolts, which had a diameter of 22 mm and length of 95 mm. As shown, the bolts had different bolt heads, shanks, and threads. Additionally, Table 5 compares the mechanical properties and strength values for JIS F10T and F14T high-strength bolts (AIJ 2012). The nominal values for tensile strength were 1,000 MPa and 1,400 MPa for the F10 and F14T, respectively. The effective areas were 303 mm^2 for an M22 F10T bolt and 316 mm^2 for an M22 F14T bolt. In addition, the ratio of bolt pretension to tensile strength was kept at 0.75 for the F10T bolts. However, the ratios for the F14T bolts were close to 0.75 but varied slightly depending on the bolt size and effective area.

Super-high-strength bolts (SHTB®) with a tensile strength of 1,400 MPa (i.e., F14T bolts) were developed and successfully applied to steel high-rise buildings in Japan beginning in 2001 (Uno *et al*. 2008). Until the end of the last century, delayed fracture limited the application of high-strength bolts with tensile strengths of more than 1,260 MPa. To develop the F14T bolts, the chemical components of steel were modified by adding molybdenum (Mo) and vanadium (V) to help reduce the effect of delayed fracture in high-strength bolts. Moreover, the technique of optimum design was applied to reduce the effect of stress concentration.

The tested F10T and F14T high strength bolts were manufactured in Japanese standards. In detail, the F10T high-strength hexagon-head bolts were made in Taiwan and

(a) F10T bolt



(b) F14T bolt

Fig. 3 High-strength bolts

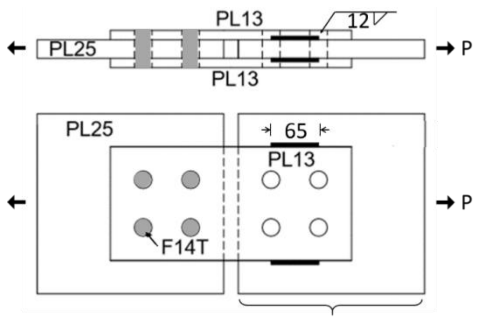
Table 5 Mechanical properties of F10T and F14T high-strength bolts

	F_y (MPa)	F_u (MPa)	Elongation (%)	Area reduction (%)
F10T	≧ 900	1,000~1,200	≧ 14	≧ 45
F14T	≧ 1,260	1,400~1,490	≧ 14	≧ 45

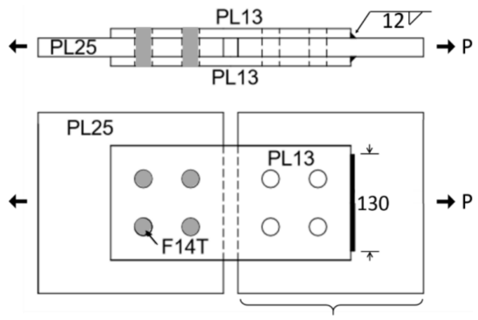
M22	Effective areas (mm^2)	Bolt pretension (kN)	Slip strength (kN)	Shear strength (kN)
F10T	303	209	171	439
F14T	316	299	260	639

satisfied the JIS B1186 standard. The F14T high-strength round-head bolts were imported from Japan and were granted a general approval by the Minister of Construction (now called as the Minister of Land, Infrastructure and Transport) in 1999. For reference, the tension control twist-off type bolts are denoted as shear-torque type S10T bolts, as to distinguish from friction-type F10T bolts. The S10T bolts are round-head and have the same physical properties with F10T bolts. The tension control twist-off type F14T bolts are called shear-torque type super high strength bolts in Japan (AIJ 2012).

Table 5 gives the recommended values for pretensions, slip strengths, and shear strengths. The values of the slip strengths and shear strengths were computed and compared for two shear planes with a slip coefficient of 0.45 for sand-blasted clean surfaces. That enabled a comparison of the slip strength values for F10T and F14T bolts, with the obtained strength ratio being 1.43.



(a) Longitudinal weld (WL)



(b) Transverse weld (WT)

Fig. 4 Welded joints

3.1.4 Fillet welds

The fracture strengths and deformation capacities were tested for two types of steel-welded joints. As depicted in Fig. 4, both the WL and WT had a leg size of 12 mm. In addition, the total amount of weld material was kept the same for each orientation. Specifically, the WL specimen had four 65-mm-long welds, and the WT specimen had two 130-mm-long welds.

3.2 Tests and measurements

The strength and deformation capacities of joints were tested, measured, and evaluated in a manner similar to that of previous work. That is, a joint was first attached to a universal machine. A load cell was then installed in the machine to determine the joint strength. Additionally, a pair of diameter gauges were installed on both sides of the joint to find out the relative displacements of the cover plates and middle plates. That allowed made it possible to determine the joint deformations. After that, the joint was tested by applying a tensile force in the machine.

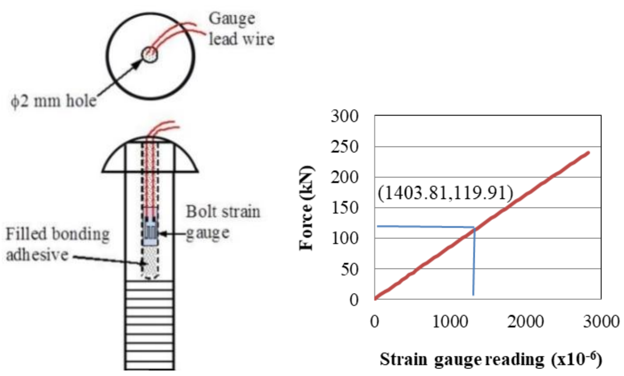
Fig. 5 shows the experimental measurements of bolt pretensions and weld deformations. Pre-installed strain gauges helped control the pretension of high-strength bolts before the testing. See Fig. 5(a). The gauges also helped monitor the loss of bolt pretension during the testing. A remote optical 3D measurement system was used to track the displacements of the steel plates and high-strength bolts. Specifically, Northern Digital Inc. (NDI) markers were numbered and added to the joint specimen with two pairs of diameter gauges. Refer to Fig. 5(b). That allowed measurements to determine the deformation capacities of the fillet welds and the slip displacements of the high-strength bolts.

3.3 Results of joint tests

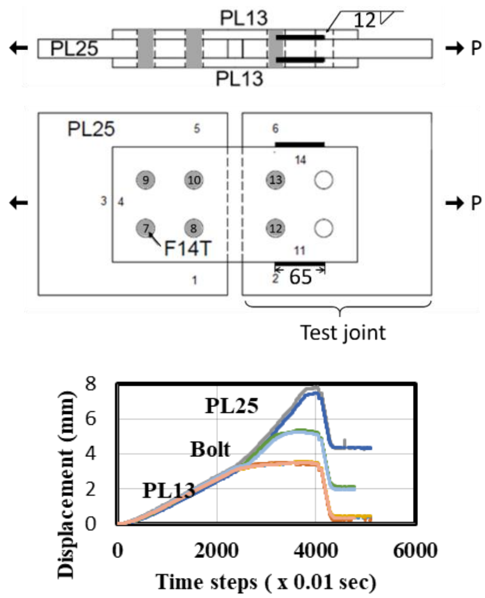
First, the tests were used to evaluate the strength and deformation capacities of four bolted joints and two welded joints. Next, the strength values of the bolted and welded joints were proportionally added to estimate the strengths of the combination joints. To prevent the effect of plate yielding, the tension force was controlled to keep it under 1,600 kN. Accordingly, only two F10T or F14T high-strength bolts were used together with longitudinal welds or transverse welds in the combination joints. Then, the strengths of the combination joints were verified through testing and compared with the calculated strengths.

3.3.1 Bolted joints

Table 6 summarizes the strengths of the tested bolted joints. There were two tests: one for 2 × 2 JIS F10T high-strength bolts and one for JIS F14T high-strength bolts. The bolts were tested by following JIS guidelines. The F10T bolts had a diameter of 22 mm. Therefore, the slip-critical strengths needed testing with pretensions ranging from 187.37 kN to 254.08 kN. For that reason, we set the pretension at 225.63 kN per bolt for the first F10T bolt test



(a) Installation and calibration of a bolt strain gauge



(b) NDI markers (Numbers 1-22) and displacement measurements

Fig. 5 Experimental measurements

and increased it slightly to 235.44 kN for the second test. See Table 6(a). Each of the two tests of the F14T bolts had a pretension of 329.61 kN per bolt. The pretension ratio between the F14T and F10T bolts approximated 1.4. As mentioned in Section 3.1.2, the materials of the F14T and F10T bolts had nominal tensile strengths of 1,400 MPa and 1,000 MPa, respectively. In other words, the pretensions of the high-strength bolts increased in proportion to the material strengths. For ease in comparing them with combination joints, the tested strengths of the joints with 2 × 2 high-strength bolts were used to estimate the strengths of joints with 2 × 1 bolts. See Table 6(b).

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Table 6 Strengths of tested bolted joints (force unit: kN)

(a)

2 × 2	Joint strength	Bolt pretension	Slip strength	*Slip coeff.
F10T	1,004.65	225.63	251.16	0.56
	1,013.00	235.44	253.25	0.54
F14T	1,214.10	329.61	303.53	0.46
	1,300.05	329.61	325.01	0.49

* Slip coefficient = slip strength / bolt pretension / 2 (number of faying surfaces)

(b)

F10T		F14T	
2 × 2	2 × 1	2 × 2	2 × 1
1,008.84	504.42	1,257.08	628.54

3.3.1 Bolted joints

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The pretensions of the high-strength bolts and their losses were monitored by the strain gauges pre-installed in the bolts, as illustrated in Fig. 5(a). It was found that the F14T bolts can lose pretension to a slightly greater extent than the F10T ones. Specifically, for the F14T bolts, the extent of pretension loses ranged from 7% to 9%. In contrast, the degree of pretension loss ranged from 2% to 5% for the F10T bolts. To compensate for this loss of pretension, it is recommended that 10% more pretension be applied to high-strength bolts on construction sites (AIJ 2012). Nevertheless, the extent of pretension loss in the tested F10T and F14T bolts was considered acceptable. This is because the bolt strain gauges controlled the initial bolt-pretension values with reasonably good precision.

The joint strengths and bolt pretensions were then used to calculate the slip strength and coefficient per bolt, as shown in Table 6(a). The slip-critical strengths were tested for the 2 × 2 high-strength bolts using a joint specimen with two faying surfaces. The AIJ recommends a slip coefficient of 0.45 for sand-blasted clean surfaces. The two tests of the F10T and F14T high-strength bolts had slip coefficients of 0.55 and 0.48, respectively, on average. Those tested slip coefficients were greater than 0.45, and therefore satisfied the AIJ recommendation.

3.3.2 Welded joints

Table 7 summarizes and compares the joint strength and deformation capacities of two welded joints. As shown, both the WL and WT had a leg size of 12 mm. In addition, the WL specimen had four 65-mm-long welds, while the WT specimen had two 130-mm-long welds. In other words, the total amounts of weld material were the same for the two welded joints. All the welds were made using KFX-81TN wire, which is designed for welding 590-MPa-grade steel with 100% CO2 gas. The welded joints developed the maximum strength right before the welds fractured. As illustrated by Table 7, the joint with the WT had slightly greater strength when compared with that of the WL. Specifically, the WT specimen had a strength of 955.25 kN, while the WL specimen was at 897.40 kN. Therefore, the strength ratio between the WT and WL specimens was 1.06.The deformations at the maximum strengths were also measured and compared. The joint with the WT had greater deformation than that of the WL, as shown in the table. That is, the WT specimen had deformation of 3.26 mm, and the WL specimen had 1.87 mm. That meant the deformation ratio between the WT and WL specimens was 1.74.

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Table 7 Maximum strength and deformations of welded joints

	Weld leg size	Weld length	Maximum strength	Maximum deformation
WL	12 mm	65 mm × 4	897.40 kN	1.87 mm
WT	12 mm	130 mm × 2	955.25 kN	3.26 mm

3.3.2 Welded joints

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3.3.3 Combination joints

Table 8 compares the strengths of tested combination joints with their calculated strengths. To calculate the strengths for the combination joints, the tested strengths of bolted joints were added in proportion to those of welded joints. In Case CJ4, two F14T bolts were close to the center of the joint specimen, as illustrated in Fig. 1(c). However, in Case CJ5 and the others, the two bolts were placed near the base plates. In addition, comparing the strength values of Cases CJ4 and CJ5 made it possible to confirm the small influence of bolt locations. There were two 65-mm longitudinal welds for one cover plate, and there were a total of four welds for two cover plates in a joint, as shown in Fig. 4(a). Accordingly, in Case CJ3, the high-strength bolts were used in combination with the four welds. As indicated by Table 8(a), one of the four longitudinal welds in Case CJ3 had a fracture failure at an early stage. That caused the test-to-calculated strength ratio in Table 8(b) to go below 1.0. In the other four cases, the combination joints had strength ratios equal to or greater than 1.0, regardless of the bolt type or weld orientation. Moreover, the steel welds for all of the combination joints had fracture failures only after the joint developed its maximum strength.

Table 9 shows the deformation capacities of joints that had combinations of F14T high-strength bolts and fillet welds. For comparison, the deformation capacities of welded joints and bolted joints are also given there. The deformation capacities of the combination joints showed the effects of weld orientations. As first indicated via Table 7, a joint with four WLs had a smaller deformation capacity. Moreover, the early fracture of a single weld reduced the joint strength. See Case CJ3 in Tables 8(a) and (b). That

Table 8 Test strength of combination joints and the ratio to calculated strength

(a) Test strength

Case	Details of combination joints		Joint strength
	High strength bolts	Fillet welds	
CJ1	1 × 2 F10T	WL	1,860.25 kN
CJ2	1 × 2 F10T	WT	1,613.90 kN
CJ3	1 × 2 F14T	WL	1,482.80 kN*
CJ4	1 × 2 F14T (near the center line)	WT	1,588.05 kN
CJ5	1 × 2 F14T (away from the center line)	WT	1,606.35 kN

* Early fracture at 1 of 4 welds

(b) Strength ratio (SR)

	Tested strengths of joints			Calculation	
	(1)	(2)	(3)	(4)	(5)
Case	**CJ	BJ	WJ	(2)+(3)	SR = (1)/(4)
CJ1	1,860.25	504.42	897.40	1,401.82	1.33
CJ2	1,613.90	504.42	955.25	1,459.67	1.11
CJ3	1,482.80	628.54	897.40	1,525.94	0.97
CJ4	1,588.05	628.54	955.25	1,583.79	1.00
CJ5	1,606.35	628.54	955.25	1,583.79	1.01

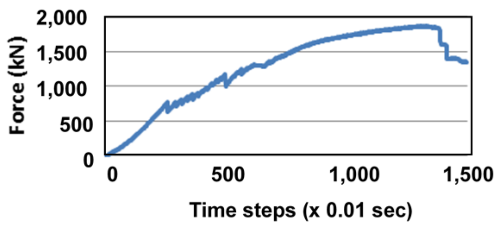
** CJ: combination joint; BJ: bolted joint; WJ: welded joint

Table 9 Comparisons of joint deformations

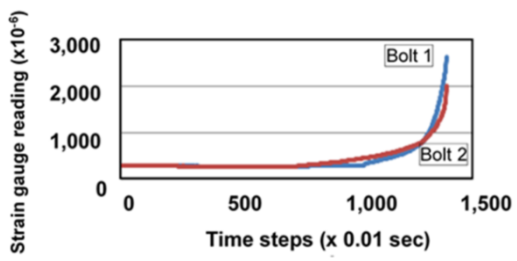
*CJ		WJ	BJ
CJ3: 2F14T + WL	0.54 mm	1.87 mm	0.49 mm
CJ4: 2F14T + WT	5.14 mm	3.26 mm	0.49 mm
CJ5: 2F14T + WT	5.07 mm	3.26 mm	0.49 mm

* Deformations at weld fracture for combination joints (CJ) and welded joints (WJ); Deformations at bolt slippage for F14T bolted joints (BJ)

also reduced the deformation capacity of the combination joint. Despite that, the CJ3 joint had a deformation capacity greater than the slip deformation capacity of the F14T high-strength bolts. In other words, the bolt slipped before the weld fractured. That means that the high-strength bolts both slipped and shared load before the fracture of the fillet welds in the combination joint. In addition, the CJ4 and CJ5 joints had deformation capacities greater than the slip deformation capacities of the F14T high-strength bolts. These two joints used F14T high-strength bolts in combination with transverse fillet welds. Therefore, the combination joints developed greater deformation capacities than the welded joints. While the CJ4 joint had two F14T bolts located close to the center of the joint specimen, the CJ5 joint had the bolts placed far away from the center. Nevertheless, as mentioned for Table 8(a), the bolt location barely affected the joint strength. That also had a small influence on the joint deformation capacity.

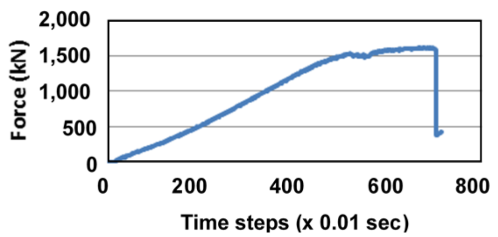


(a) Joint force (CJ1: 2F10T + WL)

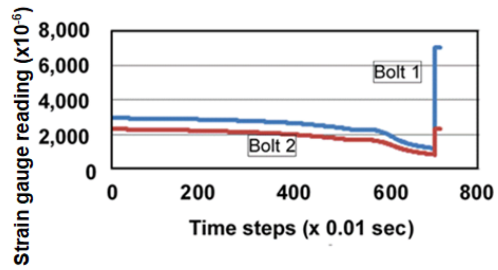


Strain	Initial value	Min value	Pretension loss
Bolt 1	3016 (step 0)	2817 (step 672)	(3016-2817) / 3016 = 7%
Bolt 2	3007 (step 0)	2739 (step 673)	(3007-2793) / 3007 = 9%

(b) Bolt strain (CJ1: 2F10T + WL)



(c) Joint force (CJ2: 2F10T + WT)



(d) Bolt strain (CJ2: 2F10T + WT)

Fig. 6 Strength of combination joints and strain of high-strength bolts

The pretensions of high-strength bolts were controlled and monitored using bolt-strain gauges. The strain values decreased as a bolt slipped and the pretensions went down. In Section 2.2.2, the ASIC-LRFD formulas are listed and used to determine the slip resistance for high-strength bolts. As Eq. (2) explains, the slip resistance of a high-strength bolt is in proportion to the pretension. Accordingly, when a bolt started to slip, the strain value decreased and showed a loss of pretension. The strain values began to increase after the bolts came in contact with the plates and developed bearing strength.

Fig. 6 gives examples showing the effects of weld orientations on the strength of combination joints and the strain of high-strength bolts. In the combinations with the longitudinal welds (for example, Case CJ1), the bolts slipped, came in contact with the plate, and developed greater bearing strength. Then, the combination joint developed its maximum strength. See Figs. 6(a) and (b). Therefore, a combination joint can develop more strength than estimated by adding the strength of bolted joints in proportion to the strength of welded joints. Note that raw data, not calibrated data, was used to obtain the results in Figs. 6(b) and (d) for two bolts. This is because only raw data can show the differences between bolts at the start of testing.

Table 10 shows the loss of pretension and bolt behavior at the maximum strengths of combination joints. In the combinations with transverse welds (for example, CJ2), the strain values of the two bolts decreased greatly, as seen in Fig. 6(d). In other words, the combinations with the transverse welds caused a greater loss in the bolt pretensions and slip resistances. However, the combinations also caused the crack angles of the transverse welds to change.

Table 10 Behavior of high-strength bolts at maximum combination-joint strengthsand loss of pretension (%)

	Bolt 1	Bolt 2
CJ1: 2F10T + WL	Bearing (7%)	Bearing (9%)
CJ2: 2F10T + WT	Slipping (59%)	Slipping (63%)
CJ3: 2F14T + WL	Bearing (8%)	Bearing (11%)
CJ4: 2F14T + WT	Slipping (59%)	Slipping (72%)
CJ5: 2F14T + WT	Slipping (34%)	Slipping (46%)

Fig. 7 shows the crack angles of transverse fillet welds before and after being used in combination with high-strength bolts. In Section 2.2.2, the ASIC-LRFD formulas are listed and were used to determine the fracture strengths of the fillet welds. Additionally, as Eq. (5) shows, the strengths of fillet welds vary with the crack angles. Those changes in crack angles provided additional weld strength. That compensated for the decrease in the slip resistance of high-strength bolts due to the loss of pretension. The combination joints can therefore develop strength greater than estimated by adding the strength of a bolted joint in proportion to that of a welded joint.

4. Conclusions

Recently developed high-strength steel improves the mechanical properties of steel plates, welds, and high-strength bolts. Additionally, the steel calls close attention to the engineering applications. However, there are still different opinions on the available strength of high-strength bolts used in combination with fillet welds on the same shear plane. To address this issue, this paper reviewed the historical development of design standards and presented the results of experimental verification for newly developed 1,400-MPa high-strength bolts, 570-MPa steel plates, and weld materials. It was found that the evaluations of test data led previous studies to draw different conclusions and affected the development of design standards, even though the studies had similar test programs and specimen details. Moreover, there are difficulties determining the available strengths of high-strength bolts based on the deformation compatibility with fillet welds used in combination. That is because the behavior of high-strength bolts may vary with the orientation of the fillet welds used in combination. It is also because the welds can have different crack angles and fracture strengths before and after being used in combination. Consequently, using the slip resistance as the available strength of high-strength bolts is recommended. That ensures a margin of safety in the strength design of combination joints.

(a) Welded joint (crack angle = 45°)

(b) Combination joint (crack angle > 45°)

Fig. 7 Crack angles of transverse welds before and after using with high-strength bolts

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