Canadian Conference - Pacific Conference on Earthquake Engineering 2023

Vancouver, British Columbia

June 25th – June 30th, 2023

Inelastic Cyclic Response of Bolts in Low Ductility Braced Frames

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ABSTRACT

Conventional Construction (Type CC) Concentrically Braced Frames (CCBF) are widely used as steel Seismic Force-Resisting System (SFRS) in Canada. CCBFs are often the structure of choice in low and moderate seismic regions. In general, CCBFs are preferred since the design process involves waiving the capacity-based procedures, among other simplifications. That is, there is no localized yielding element; energy dissipation can occur in any of its structural components. Despite the aforementioned design convenience of CCBFs, some aspects of their seismic behaviour remain poorly understood. This includes the potential inelastic response of the structural steel bolts in the brace connections during seismic events.

Grade A325 and A490 ¾″ and 1″ bolts were tested in shear under reversed cyclic loading using a modified ASTM F606 single shear test. Initially, the bolts were loaded monotonically, and the overstrength was found to be at least 10% that of the design standard. Following this, the bolts were subjected to a reversed-cyclic loading until failure in shear. Results show a 5-12% reduction of the ultimate shear resistance of the bolts under cyclic loading when compared with monotonic loading.

This reversed cyclic data was then used in comparison with a component-based numerical model to represent the inelastic bolted brace connections in CCBFs. Specifically, the shear force-deformation behaviour of the bolt and the accuracy of the *Richard* equation were evaluated, and recommendations were made as how to model this fastener accounting for its cyclic inelastic response. Having this improved bolt model will allow the advancement of the component-based numerical model for CCBFs.

Keywords: Low ductility, concentrically braced frame, structural bolt, cyclic shear test, earthquake-resistant design, component-based model

INTRODUCTION

In Canada, the Seismic Force Resisting System (SFRS) is a crucial component of a structure's design, as it provides adequate resilience against seismic forces and their impacts. The National Building Code of Canada (NBCC) recognizes a variety of seismic force-resisting systems, including moment-resisting frames (MRF), concentrically braced frames (CBF), and plate walls [1]. Among these systems, the Canadian Steel Design Standard (CSA S16:19) classifies CBFs into ductile and non-ductile groups [2]. Ductile CBFs use a capacity-based design procedure [2], which involves identifying the specific elements within the structural system that are allowed to yield during seismic events, while the remainder stays elastic. This approach contributes to preventing premature failure of structural members by designating the braces to yield and buckle first. The second type of CBF, also referred to as Type CC (conventional construction) concentrically braced frames (CCBF), is frequently used in low to moderate seismicity regions [3]. These systems, in addition to not requiring capacity-based design calculations, are generally simpler to design and stiffer than ductile systems. Moreover, as it lacks a localized yielding component, energy dissipation is allowed to occur across any of its structural components [2]. This causes a problem as any part within the SFRS can fail, compromising the structural integrity of the system. Although CCBFs are a common and convenient option for designers, they have received less research attention than ductile systems. Some aspects of their seismic behaviour are not yet fully understood at a fundamental level, e.g., the inelastic response of the structural steel bolts in the brace connections during seismic events [4].

In the late 1950s, high-strength bolts were introduced in structures and quickly replaced rivets, the leading fastener at the time [5]. This was mainly due to severe problems encountered with rivets, including the need for heat and extensive labour for installation, as well as inconsistent strength [6]. In contrast, high-strength bolts offer uniform strength and easier installation. Bolts and rivets were then believed to have one-to-one specifications [5]. In 1958, Rumpf conducted initial tests on bolts and rivets, aiming to study their behaviour and draw comparisons under different conclusions. They concluded that the shearing deformation of the rivet at ultimate load was approximately the same as that observed in the bolt, approximately 6 mm [7]. Building upon the work of these researchers, further testing to optimize the design of high-strength bolts was carried out [8]. Moore tested 1500 bolts of various grades, diameters, and lengths. They recommended an increase in the resistance factor for bolts in tension, shear with the threads excluded, and shear with the threads not excluded from the shear plane. The recommended resistance factors are 0.90, 0.85, and 0.80, respectively [9].

Over the years, as the focus shifted toward design provisions and seismic codes, researchers such as Li [10] aimed to study the bolt slippage effects on transmission towers. Since bolt characteristic studies are limited to static analysis, Li investigated the mechanical properties of typical bolt joints in transmission towers under both cyclic and monotonic loads. The data gathered was then used in numerical simulations of transmission towers subjected to seismic response. It was concluded that ignoring the dynamic effect of bolt slippage in numerical models will result in an incorrect estimation of the load-bearing capacity, as well as the failure mode of the transmission towers, hence overestimating their dynamic resistance capacity. Overall, the combined efforts of these researchers have significantly contributed to our understanding of high-strength bolts and their behaviour in various conditions, providing valuable insights for future research and design considerations.

Rudman et al. [11,12] conducted a series of experiments to study the ductility of CCBF connections, with a particular interest in seismic events. Six full-scale Type CC I-Shape braces and their bolted connections were tested under reversed-cyclic loading. The test configurations were chosen due to their prevalence in high-rise and industrial buildings. Their findings revealed that the braces and connections were susceptible to various forms of damage, including brace buckling, plate yielding, block shear failure, gusset plate buckling and fracture, and bolt fracture [11,13]. Following the work of Rudman et al., Wang et al. [14] developed a high-fidelity finite element numerical model to confirm the results from the tests and to investigate further the ductility of the Type CC braces. The models of Wang demonstrated that using the bolt bearing deformation as a fuse element did not improve the ductility of CCBFs [15]. Despite the valuable data obtained from the experiments of Rudman et al., the testing of full connections did not provide a clear indication of the specific contribution of the bolts to resisting cyclic forces. It was also challenging to incorporate the shear resistance properties of bolts into the numerical models, such as Wang's model [4], because of the lack of information of these fasteners when loaded cyclically in shear. Therefore, to better understand the shear behaviour of bolts during seismic events, it was essential to test bolts in isolation, rather than in an integrated full connection configuration.

In continuation of this line of work, the study documented herein aims at creating a database for the shear capacity and behaviour of high-strength bolts subjected to reversed-cyclic load. Conducting these tests can provide a precise understanding of the shear behaviour of bolts during seismic events, enabling researchers to accurately calibrate numerical models. Isolating bolts also provides a clear picture of their behaviour, which can be challenging to determine in full connection tests.

TESTING PROGRAM

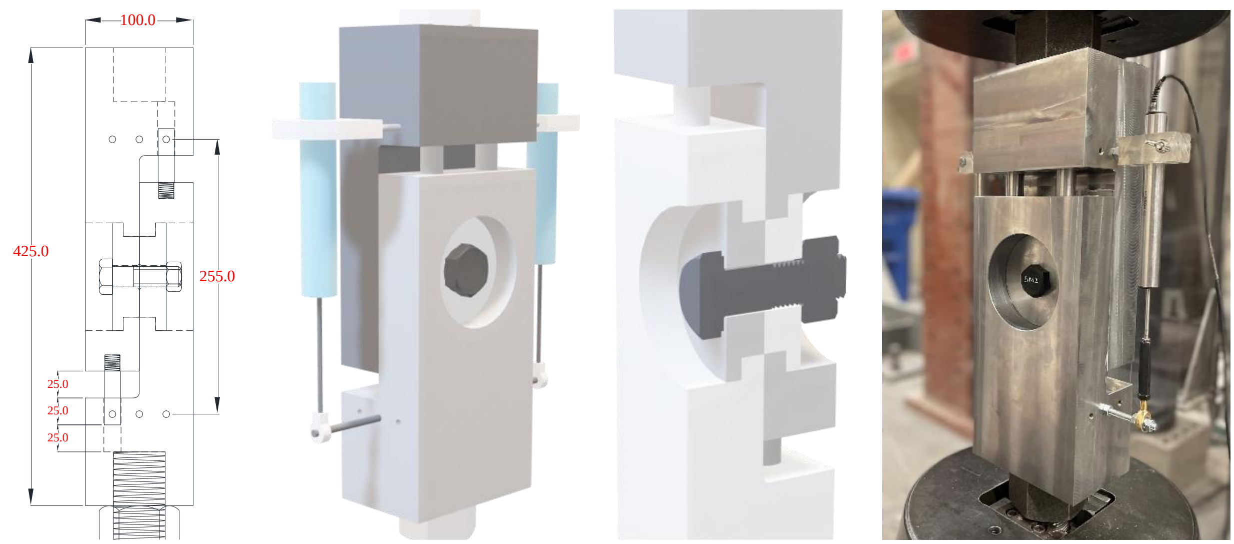
The aim of this study was to investigate the shear behaviour and ductility of individual bolts under reversed-cyclic loading conditions to enhance the understanding of their performance in seismic events. The test specimens included the most widely used bolt grades and sizes in the North American construction industry [16], which is summarized in Table 1. The testing apparatus was modified from that of ASTM F606 single shear test. The modifications made are documented in detail in the test set-up description section.

Grade A325 and A490 high-strength steel bolts of diameter ¾″ and 1″ were tested in shear under reversed-cyclic loading. Two types of shear planes are considered, i.e. through the shank and through the threads. Installing the bolts considered both the snug tight and pretensioned assembly. The length of the bolts were consistent with the CISC handbook recommendations [2]. The turn-of-nut was the method used to pretension the bolts. The test specimens’ specifications are summarized in Table 1. Six specimens were tested for each type, three for each bolt grade, totaling 60 bolts. Each size and grade of bolt specimen were obtained from the same lot.

The tests were performed in the Structural Engineering Laboratory at McGill University. Figure 1 shows the testing apparatus fixed in the 1 MN MTS load frame. The force measurements were captured by the MTS internal load cell, and two external LVDTs were installed to record the displacement data.

Table 1. Bolt Test Specimen Properties

Diameter [″]	Length [″]	Shear Plane	Installation	Loading Conditions	
3/4	2 3/4	Shank	Snug Tight	Monotonic	Cyclic
3/4	2 3/4	Shank	Pretensioned		Cyclic
3/4	2 1/4	Threads	Snug Tight	Monotonic	Cyclic
1	3 1/4	Shank	Snug Tight	Monotonic	Cyclic
1	3 1/4	Shank	Pretensioned		Cyclic
1	2 3/4	Threads	Snug Tight	Monotonic	Cyclic



(a) *(b)* *(c)* *(d)*

Figure 1. Bolt Shear Test Apparatus: (a) Overall Dimensions (mm), (b) 3D model, (c) 3D model cutout, (d) Full Laboratory Setup

The test apparatus was developed from the ASTM F606 single shear test by including several modifications. These included additional vertical space between the jig ends to compensate for the cyclic movement, and four pins to keep the jigs aligned and prevent any rotational movement that may occur during testing. Sacrificial inserts were used to protect the jigs from being damaged due to bolt bearing deformations. The bolts holes were drilled 1/16″ larger than the bolt diameter to conform with industry standards. To allow for multiple tests without damage, high tensile strength 4140 Tool Steel was used to manufacture the jigs and pins. The sacrificial inserts were produced in grade 350W to embody what is commonly used in the field. Owing to bolt plate bearing damage after each test, each pair of inserts was reused by rotating to an undamaged section before the next test until all the sides were unusable.

All tests were displacement-controlled. Monotonic tests were performed at a rate of 1 mm per min, and each monotonic test lasted about 5 to 10 minutes depending on the size and grade of the bolt. The yield displacement (δ_y) of each bolt was calculated from the monotonic tests according to the ATC-24 Guidelines (Applied Technology Council) [17]. The yield shear force (τ_y) was assumed to be 60% of the yield force (F_y) stated by the ASTM F3125. To characterize the elastic range, a linear regression was used to estimate the linear portion of the monotonic tests results. Using the obtained line equation, the displacement at zero force was denoted by x_o and displacement at τ_y was denoted by x_d. The yield displacement was then calculated as the difference between x_o and x_d. An illustration for the calculation process is shown in Figure 2.

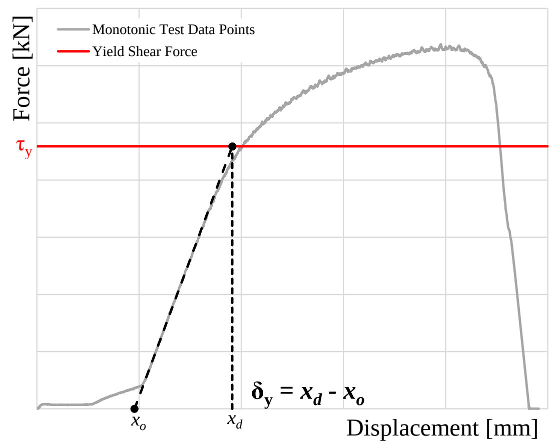


Figure 2. Calculation of yield displacement

The yield displacement was then used to establish the cyclic loading protocol adapted from the seismic testing protocol guidelines suggested by FEMA 461 (Federal Emergency Management Agency) [18]. Since the bolt hole was 1/16″ (approximately 1.6 mm) larger than the bolt diameter, the friction zone was considered to be equal in size to the gap between the bolt hole and bolt diameter. Assuming that the bolt was placed in the middle of the hole, the friction phase amplitude in each direction of movement was half of this distance, that is 0.8 mm. The next phase brought the bolt to its yield displacement obtained from the monotonic tests. Both of these phases were run at a rate of 10 mm per min for 6 cycles each. For the subsequent phases, the amplitude was increased by 0.5 δ_y, and were loaded for 2 cycles at a rate of 12.5 mm per min as suggested by the ASTM F606 shear test. The loading pattern followed a sine curve, and the tests continued until the bolt specimen fractured. The cyclic loading protocol is summarized in Table 2 and Figure 3.

Table 2. Cyclic Shear Test Loading Protocol

Number of cycles	Amplitude [mm]	Rate [mm/min]
6	0.8	10
6	0.8 + 1.0 δ_y	10
2	0.8 + 1.5 δ_y	12.5
2	0.8 + 2.0 δ_y	12.5
2	0.8 + 2.5 δ_y	12.5
2	0.8 + 3.0 δ_y	12.5
2	0.8 + 3.5 δ_y	12.5
2	0.8 + 4.0 δ_y	12.5
2	0.8 + 4.5 δ_y	12.5
2	0.8 + 5.0 δ_y	12.5

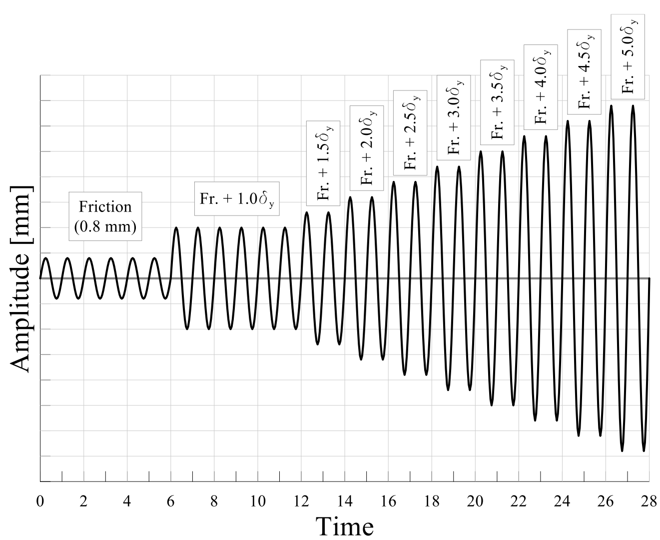


Figure 3. Cyclic Shear Test Loading Protocol

TEST OBSERVATION AND RESULTS

The test results are summarized in Table 3. The cyclic maximum shear force was consistently 88% lower than the monotonic shear force for the A325 grade bolts. In contrast, the ratio of the cyclic shear force to monotonic shear force of the A490 grade bolts varied from 89% to 95%. The pretensioned bolts had a larger ultimate shear resistance than their snug tight counterparts. It is worth noting that the recorded displacement was the total bolt displacement and the bearing suffered by the plate; this is due to the complexity associated with isolating the two values from one another. The overstrength of the bolts ranged from 9% to 23% except for the 1″ A490 shank, which was 2% less than the standard shear strength.

Table 3. Summary of Test Results

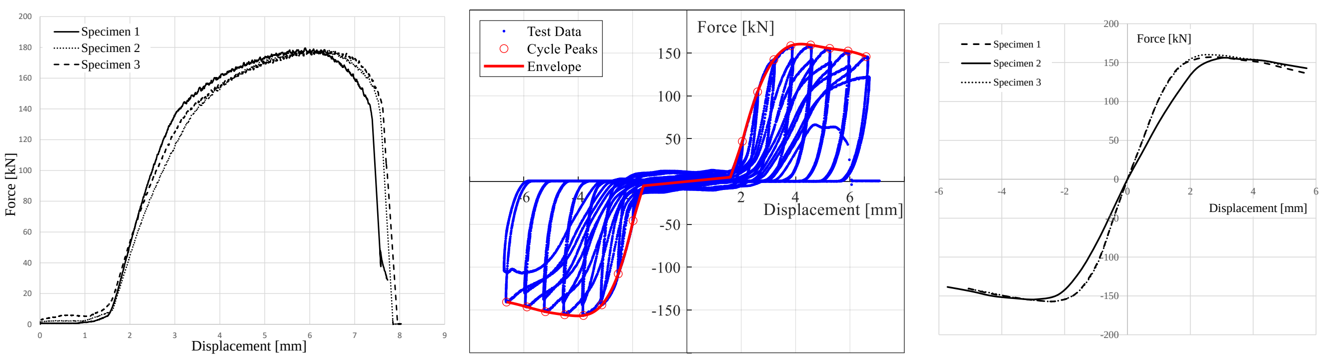
Grade	Diam. [″]	Shear Plane	Inst.	Loading Condition	Bolt ID	Yield Shear Force, τ_y [kN]	Yield Displ., δ_y [mm]	Max. Shear Force, τ_u [kN]	Fracture Displ. [mm]
A325	3/4	Shank	Snug Tight	Monotonic	A325_34_SM	129.5	1.40	178.9	7.79
			Snug Tight	Cyclic	A325_34_SC			157.4 -155.7	7.14 -6.97
			Pretension	Cyclic	A325_34_SCP			169.5 -166.9	6.93 -6.64
A325	3/4	Threads	Snug Tight	Monotonic	A325_34_TM	91.1	1.65	133.8	8.26
			Snug Tight	Cyclic	A325_34_TC			116.6 -114.6	6.13 -5.91
A325	1	Shank	Snug Tight	Monotonic	A325_1_SM	229.4	2.15	321.3	10.12
			Snug Tight	Cyclic	A325_1_SC			283.8 -285.8	8.87 -9.05
			Pretension	Cyclic	A325_1_SCP			289.7 -281.7	8.46 -7.82
A325	1	Threads	Snug Tight	Monotonic	A325_1_TM	160.4	1.75	240.0	8.02
			Snug Tight	Cyclic	A325_1_TC			219.3 -217.9	6.19 -5.90
A490	3/4	Shank	Snug Tight	Monotonic	A490_34_SM	183.0	1.65	212.7	7.06
			Snug Tight	Cyclic	A490_34_SC			197.5 -196.3	6.44 -6.48
			Pretension	Cyclic	A490_34_SCP			207.3 -204.5	7.02 -6.70
A490	3/4	Threads	Snug Tight	Monotonic	A490_34_TM	128.0	2.00	146.8	5.93
			Snug Tight	Cyclic	A490_34_TC			141.9 -139.4	5.14 -4.59
A490	1	Shank	Snug Tight	Monotonic	A490_1_SM	323.1	3.15	348.1	10.06
			Snug Tight	Cyclic	A490_1_SC			316.8 -312.0	8.92 -7.91
			Pretension	Cyclic	A490_1_SCP			320.1 -311.0	8.43 -7.79
A490	1	Threads	Snug Tight	Monotonic	A490_1_TM	226.1	2.20	271.1	6.86
			Snug Tight	Cyclic	A490_1_TC			255.2 -255.9	6.35 -6.07

A visual overview of the damage under various testing conditions is provided in Figure 4. The bolts mostly failed as expected through the designated shear plane. However, during the shank cyclic tests, 7 bolts of grade A490 (two ¾″, two 1″ and three 1″ pretension specimens) failed in tension in the threads. This can be observed in Figure 4 (d); the load was applied to the shank as can be seen by the deformation in the shear plane, but the bolt failed through the threads due to a component of force that developed in tension as the fastener experienced high shear deformations. Further research is recommended to explore this phenomenon.

(a) *(b)* *(c)* *(d)*

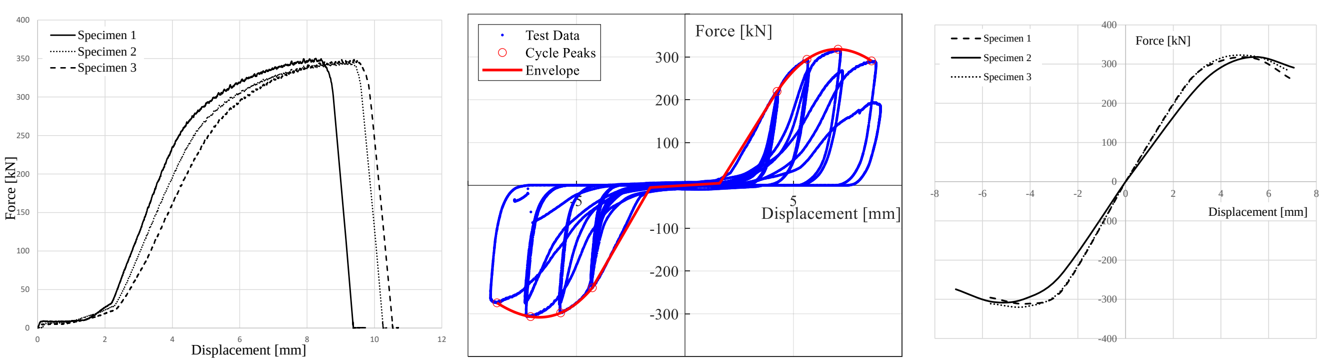
Figure 4. Bolt Test Damage: (a) A325 3/4" Threads Monotonic, (b) A325 3/4" Shank Cyclic, (c) A490 1" Shank Monotonic, (d) A490 1" Shank Cyclic Pretensioned

The test data for the A325 ¾" bolts and the A490 1" bolts, respectively, are summarized in Figure 5 and Figure 6, both with the shear plane through the shank. The flat portion at the beginning of Figure 5 (a) and Figure 6 (a) represents the movement of the bolt in the bolt hole before the load bearing begins (snug tight installation). This value is around 1.6 mm for both, which suggests that the bolts were not centered, but rather at the bottom of the hole. The bolts were installed with more care for the cyclic tests to ensure that they were centered in their holes. From the same graphs, one can observe that the Grade A325 bolts have more ductility than the Grade A490. Figures 5 (b) and 6 (b) include the cyclic test data points with the backbone of the peak shear forces. For each amplitude of the cyclic protocol, the second cycle peak shear force does not reach the first cycle peak shear force. A reduction of stiffness can also be observed in the second cycle when compared with the first cycle. Figure 5 (c) and Figure 6 (c) display the envelope curves for all the cyclic test specimens without the friction zone. The curves are all consistent. While the monotonic test curves show quite a sudden failure, the cyclic test curves have a post-ultimate decay in shear resistance.



(a) *(b)* *(c)*

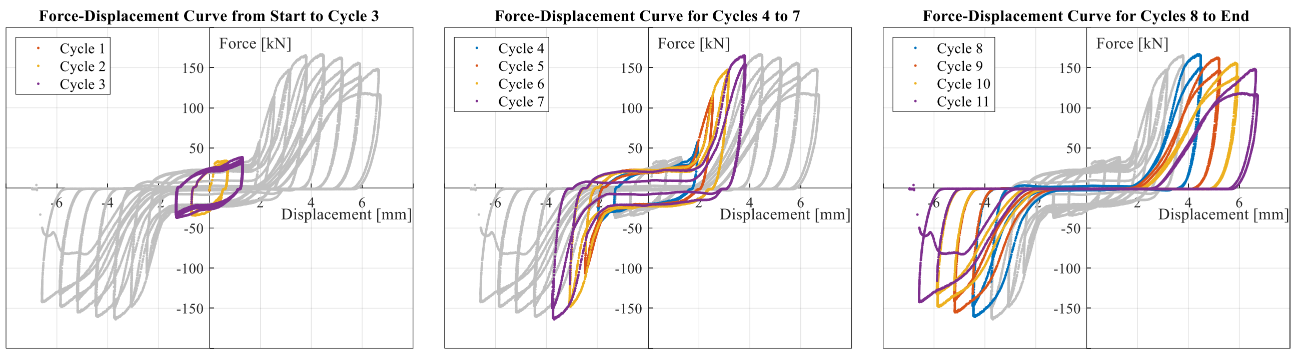
Figure 5. Force - Displacement Curves for A325 3/4" Bolt Shear Test through Shank, Snug Tight Installation: (a) Monotonic Tests, (b) Cyclic Tests and Envelope, (c) Cyclic Tests Envelope Curves without friction portion.



(a) *(b)* *(c)*

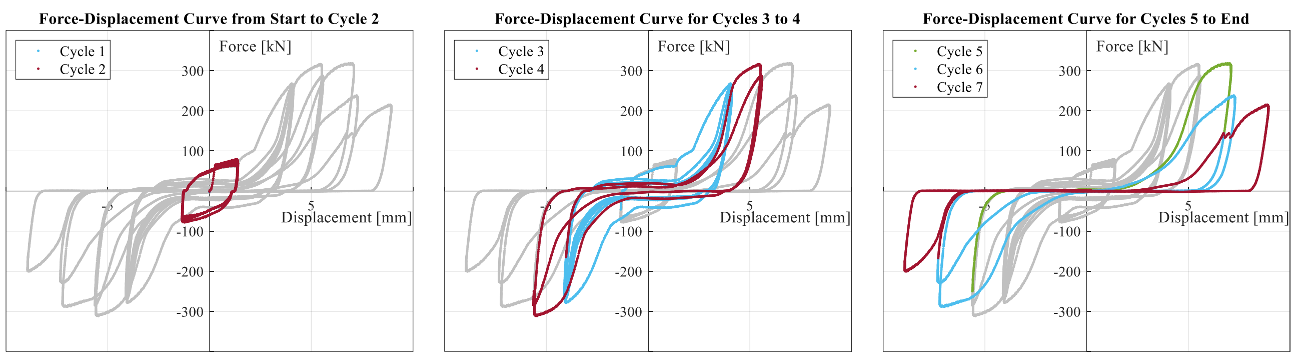
Figure 6. Force - Displacement Curves for A490 1" Bolt Shear Test through Shank, Snug Tight Installation: (a) Monotonic Tests, (b) Cyclic Tests and Envelope, (c) Cyclic Tests Envelope Curves without friction portion.

The different cycles of the pretensioned bolt tests are displaced in Figure 7 and Figure 8. The first cycles shown in Figures 7 (a) and 8 (a) display the pretension force-displacement loops. The pretension force corresponds to the slip critical resistance from CSA S16:19. In the next few cycles, the bolts maintain their slip critical resistance, as can be seen in the middle portion of Figure 7 (b) and Figure 8 (b). However, the bolts lose all their slip critical resistance toward the last cycles of the protocol, where it is assumed the bolts have lost their pretension force due to plastic elongation under high shear deformations. This is an important behaviour to consider when numerically modeling pretensioned bolts, since the change in energy dissipation is quite significant in the slip zone.



(a) *(b)* *(c)*

Figure 7. Step Force - Displacement Curves of A325 3/4" Pretensioned Bolts: (a) Friction Cycles, (b) Middle Cycles, (c) End Cycles



(a) *(b)* *(c)*

Figure 8. Step Force - Displacement Curves of A490 1" Pretensioned Bolts: (a) Friction Cycles, (b) Middle Cycles, (c) End Cycles

Comparison with Richard Equation

Eq. (1) shows the *Richard* equation, as described by Weigand [19], which depicts the shear force-deformation bolt behaviour. More details on the parameter estimation can be found in Wang [14].

$$R_{bolt} = \frac{(K_{i,bolt} - K_{p,bolt})\Delta}{\left[1+\left|\frac{(K_{i,bolt} - K_{p,bolt})\Delta}{R_{v,bolt}}\right|^{2}\right]^{\frac{1}{2}}} + K_{p,bolt}\Delta \qquad (1)$$

where R_{bolt} is the shear force in kN, Δ is the shear deformation in mm, $K_{i,bolt}$ is the bolt initial stiffness in kN/mm, $K_{p,bolt}$ is the bolt plastic stiffness in kN/mm, and $R_{v,bolt}$ is the bolt shear capacity in kN.

The *Richard* equation was plotted with the envelope of the reversed-cyclic bolt test results (friction zone removed) in Figure 9. The curve is in clear agreement with the test data in the elastic zone. However, as can be seen by the graphs, the equation predicts an increasing hardening behaviour as opposed to the softening observed in the test data once the maximum shear resistance was attained. The equation should be revisited to capture a more accurate bolt behaviour for more reliable numerical models.

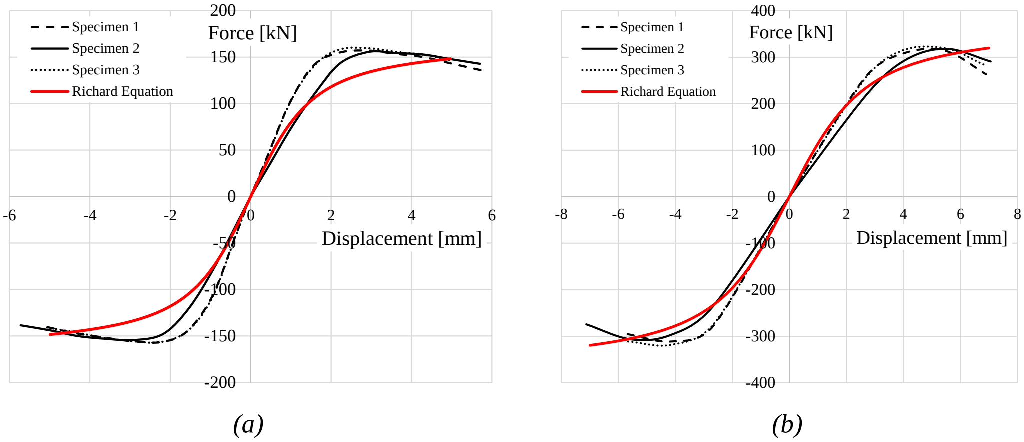


Figure 9. Comparison of Bolt Test and Richard Equation for: (a) A325 ¾" shank, (b) A490 1" shank

CONCLUSION

In this study, the behaviour of high-strength steel bolts under reversed-cyclic loading was studied. A325 and A490 ¾" and 1" bolts were subjected to monotonic and reversed-cyclic loadings. The loads were applied in such a way that the shear plane was through either the shank or through the threads. The results showed that the cyclic shear ultimate shear resistance is 5% to 12% lower than monotonic shear resistance. It was also observed that the *Richard* equation correlates well with the test data in the elastic zone; however, in the hardening zone the equation predicts a continuing increase in shear resistance, which is in contrast with the declining trend of the test data. This suggests the need for revisiting of the bolt behaviour equation to accurately capture the performance of the bolts under reversed-cyclic shear loading.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Natural Sciences and Engineering Research Council of Canada and the Faculty of Engineering at McGill University for their generous financial support. The authors would also like to extend their sincere gratitude to the lab technical staff at the Structural Engineering Laboratory of McGill University for their invaluable contributions to this study.

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