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A Field Study of Fastener Tension in High-Strength Bolts

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1. Introduction

1.1. Background

The specifications provided by the Research Council on Structural Connections for the installation of high-strength bolts (1, 2) require that bolts in slip-critical and direct tension connections have an installed tension equal to at least 70% of the specified minimum tensile strength of the bolts. According to these Specifications, the required tension can be attained in one of four ways: turn-of-nut installation, calibrated wrench installation, use of a direct tension indicator, or use of proprietary bolt types that incorporate a design feature intended to directly indicate the load in the bolt. Load-indicating washers are one example of a direct tension indicator and so-called tension control bolts are one example of a type that directly indicates bolt load level.

Studies that report on the actual tension of bolts in joints made up in the laboratory are available, especially for bolts installed by the turn-of-nut or calibrated wrench methods (3). However, studies of bolt tension in field joints are lacking because, until recently, there has not been a method of measuring these forces that was both convenient and reliable. Only in about the last decade have devices been developed that enable bolt tension to be determined in the field in a relatively simple and reliable way (4). These are ultrasonic measurement devices that read the change in bolt length before and after installation. In turn, this change in length can be related to the bolt tension. Although the ultrasonic measurement device is relatively expensive, it gives reliable values of bolt tension and is easy to use in the field.

The ultrasonic measurement device, or "bolt gage", is simply an electronic instrument that delivers a voltage pulse to an acoustic transducer. The transducer emits a very brief burst of ultrasound waves that passes through the bolt, echoes off the far end, and returns back to the transducer. The electronic instrument very precisely measures the time required for this round trip. The change in the measured "transit time" before and after tightening can then be converted into length or into load.

The bolt gage is capable of measuring the tension in a bolt to the nearest 1%. However, there are many factors that affect the accuracy of the ultrasonic device and achieving this level of accuracy is not easy. The variables include such factors as the electric frequency of the device, the bolt size, the change in the velocity of sound (material velocity) due to changes in the elastic modulus and the material density among the bolts, the change in the material velocity due to different residual stress levels in the bolts, the accuracy of the calibration equipment used to determine pretension load from the measured change in time, and measurement of the grip length. The effect of these factors differs from one case to another. In very long bolts, variation in the modulus of elasticity can be one of the main problems. In very short bolts, uncertainty in the grip and thread lengths can dominate. It is argued by Bickford (4) that the ultrasonic measurement device can measure the bolt stretch to the nearest 5% even under unfavorable conditions. It will be shown later in this report that some conditions can result in errors that are larger than 5%.

The first large-scale program set up to measure the tension in installed high-strength structural bolts was carried out as a University of Toronto study that was reported in 1990 (5). Bolt tensions in a total of 231 bolts were obtained. These bolts were in joints located at six different construction sites, included both A325 and A490 bolts (212 and 19 in number, respectively), encompassed industrial buildings, commercial buildings, and bridges, and reflected both turn-of-nut and load-indicating washer installations. The study reported herein can be described as an unofficial extension of the University of Toronto work. It is more limited in scope (only bridges were examined and only the turn-of-nut method of installation using A325 bolts was used), but the study includes a total of 111 bolts and should be a useful addition to the data base of bolt tensions measured in the field.

1.2. Objectives

The objectives of the investigation were to measure the bolt tension in field-installed splice connection bolts, to assess the installation procedure and resulting bolt tensions of field-installed bolts, and to compare these results with code requirements.

1.3. Outline of Work Plan

Three bridges were selected for measurement of bolt tensions. Arrangements were made with the owner, Alberta Transportation, and the erection contractors, Northern Steel Inc. and Empire Iron Works Ltd., to allow and assist in doing the field measurements. The procedure used for measurement of bolt tensions was carried out in three stages.

First, the the ultrasonic measuring device, or "bolt gage", was calibrated in the laboratory using a representative sample of the bolts used in each bridge. The calibration provides an accurate value of the average stress factor and the average temperature factor to be used in the field measurement.

Second, the installation procedure of the bolts at the sites was observed and recorded. Following this, the bolt gage was used to measure the length of the selected bolts before and after loosening the nut. These bolts were then taken back to the laboratory for further examination.

Third, the bolts whose "before" and "after" elongations were established in the field were tested in the laboratory under direct tension to determine their stiffness (load vs. stretch relationship). The measured stiffness is then simply multiplied by the measured change in length to obtain the tension that existed in that bolt in the field condition.

2. Experimental Investigation

2.1. Introduction

The bolts studied herein were used in the girder splices of three composite steel girder bridges. The three bridges, all of which cross the Sturgeon River, are located about 30 km northwest of Edmonton, Alberta (bridges B1 and B2) and 50 km north of Edmonton (bridge B3). All the bolts examined were 22 mm dia. ASTM A325 Type 3 (weathering steel) bolts. The length of these bolts was 70 mm, 82 mm, or 102 mm. (In U.S. customary units, these bolts are 7/8 in. dia. by 2-3/4 in., 3-1/4 in., and 4 in. long).

Bridge B1 is located 14 km west of the city of St. Albert on local road 263 just north of Highway 633, and bridge B2 is 9 km west of St. Albert on local road 262 just south of Highway 633. Bridges B1 and B2 are identical. They are three-span bridges (13 m - 16 m - 13 m) that use five WWF 700 x 151 steel girders. Each girder has two field splices, each one located 4.6 m from the centerline of the bridge.

Bridge B3, which is on the west side of the town of Gibbons, is also a three-span bridge (32 m - 38 m- 32 m). It uses four WWF 1200 x 263 girders. Each girder has four field splices, one 8.0 m on each side of each pier.

The girder splices in bridges B1 and B2 contain 94 bolts each (46 in the web splice, 24 in each of the flange splices.) The splices in bridge B3 contain 152 bolts each (48 in the web and 52 in each of the flanges.) When the bolts in a connection are checked according to the Arbitration Inspection procedure of the Research Council on Structural Connections Specifications, a minimum of 10% but not less than two bolts are to be inspected. Using this as a basis, but including other factors such as the time available before deck formwork was to be placed, it was decided to sample 8 to 10 bolts from each girder splice (generally five from the web and from three to five from the flanges). In bridge B1 three connections of the total of ten in the bridge were used to measure bolt tensions, in bridge B2 six out of ten were examined, and in bridge B3 four out of sixteen splices were used. A total of 111 bolts were inspected for bolt tension. (A few results, a total of seven, were subsequently judged to be invalid, however.)

Bolts are identified by a numbering system that includes the bridge identification, the location of the connection and the girder, and whether the bolt was in the web or in the flange. For example, B1-N2-W3 is a bolt in bridge B1, in the northerly splice connection of girder 2, and is the third bolt sampled in the web. The girders are numbered sequentially from east to west (in case of B1 and B2) or from south to north (in case of B3). The exact location of each bolt is shown in the sketches of Figs. 1, 2, and 3, which correspond to bridges B1, B2, and B3.

Testing was carried out in three stages. A sample of two unused bolts of each length was used to calibrate the bolt gage. This process was repeated for each bridge before starting field measurements. After the field measurements were finished, the field-installed bolts were brought back to the laboratory. A direct tension test was then applied to them in order to determine their stiffness and, using the change in bolt length determined in the field, thereby obtaining the tension. Each of these three steps (calibration, field measurement, and laboratory test) is described in detail following.

2.2. Calibration

The bolt gage used in this study is a PDX-934 model manufactured by Raymond Engineering Inc. (serial number 356) and loaned for the study by Syncrude Canada Ltd. Although the bolt gage measures the time needed for the sound waves to travel through the bolt, it actually displays the length of the bolt. The displayed length is the product of the material velocity and half the transit time. If the change in length is the desired measurement, a similar procedure is followed. A simple equation is used with the bolt gage readings to determine the change in the bolt length due to the tightening procedure (4), as follows:

$$\Delta L = (0.5\, t_2 - 0.5\, t_1)\, M_v \qquad (1)$$

where ΔL is the change in the bolt length

t_1 is the transit time of the signal passing through the unloaded bolt

t_2 is the transit time of the signal passing through the tightened bolt

M_v is the velocity of sound in the unstressed alloy steel bolt at room temperature.

The value of M_v is generally taken as 6 350 m/sec for steel. This is entered into the bolt gage as a constant. However, as soon as the temperature of the bolt is increased or the bolt is stressed, the material velocity decreases proportionally and the bolt gage will read longer bolt lengths which are not only related to tension-induced stretch but also to the velocity changes since M_v changes. For example, a transit time of ten microseconds would correspond to 63.5 mm of length in the case of an unstressed bolt, but the same transit time would correspond to approximately 17.8 mm of length in case of a stressed bolt (4). Instead of changing the material velocity for each temperature and stress level, two correction factors are used in the bolt gage to compensate for the effect of temperature change and stress level. Equation 2.1 is now adjusted as follows (5):

$$\Delta L = \left(0.5t_2 \{1 - (T_2 - T_o)T_f\} - 0.5t_1 \{1 - (T_1 - T_o)T_f\}\right) S_f M_v \qquad (2)$$

where ΔL is the change in length as a result of bolt tension only

T_o is the bolt temperature at the time of calibration

T_1 is the bolt temperature when measuring t_1

T_2 is the bolt temperature when measuring t_2

T_f is the temperature factor (usually around $90x10^{-6}$ mm/mm/ °C)

S_f is the stress factor (normally between 0.25 to 0.33).

The bolt gage reads the length or the stretch directly when the values of M_v, S_f, and T_f are entered in its rear panel. The bolt gage is usually supplied with factory-calibrated values for a certain bolt material. However, for more accurate results these factors should be determined independently by calibrating a representative sample of the tested bolts and re-entering them in the bolt gage. The calibration procedure is detailed in Appendix A, B, and C.

To measure bolt tension, the change in bolt length measured by the bolt gage was the critical measurement in this study. The change in length is affected by the temperature factor only if the temperature changes significantly during the time between measuring the loaded and the unloaded bolt. In this study, the maximum recorded change in the temperature was 6 °C, which is small enough that the factory-calibrated value of the temperature factor, in this case $97x10^{-6}$ mm/mm/ °C, can be accepted. The material velocity and the stress factors obtained from the calibration of the bolts used in each bridge are shown in Table 1. It is to be noted that the bolt gage transducer used in the measurements of bridge B1 was different from the one used in the measurements of bridges B2 and B3, which explains the slight difference between the material velocity obtained from the calibration of the bolt sample from bridge B1 and the sample from bridges B2 and B3.

2.3. Field Measurements

2.3.1. Bridge B1

2.3.1.1. Installation Procedure

The splices in this bridge were bolted up on March 20 and 21, 1991. The steel erector had a three-man crew on the job and the owner had an inspector (employee of Alberta Transportation) who appeared to stay at this site full-time. The bolting crew was properly equipped (air compressor, impact wrench, torque wrench, etc.) Both the bolting crew and the inspector had a Skidmore-Wilhelm hydraulic bolt load indicator. Using the Skidmore, the bolting crew and the inspector established that the turn required for the minimum specified bolt tension was 1/4 to 1/3 past snug. (Each of these figures was reported by different individuals. The specified value for bolts of the length used in the splices of this bridge is 1/3 turn.) The joints into which the bolts were installed were relatively simple and did not require any "compacting". The nuts were run up with the impact wrench and it appeared to the observer that about 1/4 turn was put on after first impact. The operator of the wrench did not stop at what the observer took to be first impact and did not make any mark to indicate the starting point for the subsequent one-third turn (or whatever). The chuck of the impact wrench was marked at suitable intervals so that the amount of rotation after first impact could have been observed. The interpretation of the observers was the bolts were being installed by "sound". The ironworkers said that they used the procedure just described at all bridges, whether small or large.

The inspector used a torque wrench to establish the torque required to start moving the nut of a bolt in the Skidmore. This value was then used to check bolts in the bridge itself. According to the inspector, 90% to 95% of all the bolts in the splices were checked by him. Both the inspector and the crew seemed dedicated to the published value of the minimum specified bolt tension (174 kN or 39 kips in this case), and were just as anxious not to be "over" as they would be to deliver "under". The inspector related that he has been told that it is preferable to be a little under the 39 kip load rather than to be over.

2.3.1.2. Testing

Twenty-four bolts from three different girder splice connections in bridge B1 were sampled. As seen in Fig. 1, eight bolts were inspected in each connection. The surface at each end of the bolt was first ground carefully so that the ends were as flat and parallel as possible. A drop of coupling fluid (80% glycerin and 20% water) was then placed on bolt head in order to reduce acoustic impedance. The transducer was centered on the bolt head and attached to it with a magnet. To spread the couplant into a thin and even layer, the probe was rotated and pressed against the bolt. The probe was used by only one person in order to improve measurement repeatability. The diameter of the transducer is approximately the same as the diameter of the bolt and this minimized any errors that might occur due to replacing the probe on the bolt.

A digital thermometer of 1 ^{o}C accuracy was used to determine the bolt temperature at its preloaded condition (T_2). The bolt number and its temperature were recorded and a reading of the bolt length taken (L_2). The probe was removed and an impact wrench then used to loosen the nut. Once the nut was fully loosened, another length measurement was taken and recorded (L_1). If the time between reading L_2 and L_1 was large enough to encompass a change in bolt temperature, it was remeasured (T_1) and entered into the bolt gage before measuring L_1.

During each length measurement, the transducer was centered on the head as accurately as possible. The fluctuation of the measured length in a certain position of the transducer was observed to be about 0.008 mm. This number translates to 1% of the measured stretch for a 22 mm bolt with 34 mm grip length loaded to 230 kN. This error represents the accuracy of the bolt gage. It should be noted that the transducer had to be removed from the bolt head during loosening of the bolt with the impact wrench. It was believed at that time that repeatable readings could be easily obtained with the ground surface conditions.

After the bolt was removed from the connection, the thickness of the connected parts was measured using a digital caliper that has an accuracy of 0.01 mm. This measurement, which is equal to the grip length of the bolt, was done on both sides of the hole. The grip length (L_G) is important in calibrating the bolt since the stretch is proportional to the grip length. Thus, the accuracy in measuring ΔL (and thus bolt tension) is also proportional to the accuracy in measuring L_G. For example, one mm error in measuring a grip length of 34 mm will cause 3% error in bolt load estimate. After the bolt length in the unloaded condition had been established, that bolt was removed and a new bolt installed as a replacement.

2.3.1.3. Results

The field measurements taken on bridge B1 and the bolt tensions derived from them are shown in Tables 2, 3, and 4. The lengths of the unloaded bolts were re-measured in the laboratory and all but four laboratory measurements were reasonably close to the field measurements. In two of these cases where repeatability could not be attained (B1-N2-F1 and B1-N2-W5), unjustifiably high bolt tensions were calculated. Examination of these bolts showed that shank ends were ground unevenly. This probably was the source of the poor repeatability. It was decided thereafter (bridges B2 and B3) to take more precautions in the bolt length measurements. It is also shown in Tables 2 to 4 that the change in measured thickness of the gripped material did not exceed 4% in one connection.

2.3.2. Bridge B2

2.3.2.1. Installation Procedure

The splices in this bridge were bolted up on March 28 and 29, 1991 by the same ironworker crew and under the supervision of the same inspector who had worked on bridge B1. The bolt installation was observed and recorded by the writers using a video camera.

The method of installation was generally similar to that used on bridge B1, although it was possible to observe more of the bolting-up operation in bridge B2 than had been seen in bridge B1. Typically, each splice had drift pins in about half the holes and bolts loosely placed in the remainder of the splice prior to the preloading of any bolts. After these bolts had been installed to completion, the drift pins were removed and bolts installed in the open holes. Most pins had to be driven out using a sledge hammer and on one occasion a drift pin had to be removed by drilling. As was the situation on bridge B1, the operator of the impact wrench did not stop at the first impact and did not make any mark to indicate the starting point for the subsequent amount of turn past snug. The amount of nut rotation was apparently controlled by the sound of the operation. To the observer, it appeared that about 1/4 or 1/3 turn past snug was applied. The inspector checked all of the bolts in three or four connections in which the process was observed by the authors. This inspection identified one bolt that had not been preloaded at all by the bolting crew and one bolt for which the inspector required more turn of the nut.

2.3.2.2. Testing

In this bridge, 48 bolts taken from six different connections were sampled to obtain their installed bolt tension. Sketches of the locations from which the samples were taken are shown in Fig. 2. The measurement procedure used was similar to that described for bridge B1. However, using the experience gained on that bridge, certain improvements were made in order to improve the reliability of the readings. At least three readings for each measurement of length (L_1 and L_2) were taken. The transducer was centered on the bolt head using a simple cap guide and a special nut with a machined surface was used to help control the grinding process at the free end of the bolt. (This nut was placed on the free threads beyond the structural nut and was a temporary measure only.) Because access to the splices and the necessary equipment was limited on this bridge, the initial measurement of length and temperature of a bolt had to be followed immediately by bolt replacement. Final measurement of length and temperature was taken about two hours after the initial measurements. In this bridge, measurement of the thickness of the connected parts (bolt grip) was taken in only two holes of each splice.

2.3.2.3. Results

The field measurements taken on bridge B2 and the bolt tensions derived from them are listed in Tables 5 through 10. After release of the bolt preload, the length of all bolts was measured both in the field, as described above, and later in the laboratory. In all but three cases, there was satisfactory agreement between these two sets of readings. For another nine bolts, it was found that rotation of the bolt gage probe about the longitudinal axis of a loaded bolt gave substantial differences in recorded bolt length. In some instances, the difference was greater than the change in length between loaded and unloaded conditions. This phenomenon is perhaps the result of bolt curvature, that is, one side of the loaded bolt is shorter than the other. In these cases, the bolt was identified and the smallest set of readings obtained was used to calculate the bolt tension. This is a conservative measure of bolt preload. For all bolts, the average of three readings of length was used in calculating bolt load.

The data in Tables 5 through 10 show that the average value of the difference between any of the three measurements of bolt length and the corresponding mean value is 0.013 mm. This corresponds to about 2% of the change in elongation of a 22 mm dia. bolt with 34 mm grip length loaded to 230 kN. However, the maximum difference reached 0.55 mm, which corresponds to an error of about 8%. The data also show that, for two cases, unjustifiably high bolt tensions are calculated. In summary, of the total of 48 bolts for which bolt tension was determined, two cases were considered to give unreasonable results, three bolts showed different length readings in the laboratory than were obtained in the field, and nine bolts had an apparent curvature in the loaded condition. It was considered that the number of discrepancies from expected results was still too high and further steps were taken to improve the measurement technique for bridge B3.

2.3.3. Bridge B3

2.3.3.1. Installation Procedure

The splices in this bridge were bolted up on April 10 and 11, 1991. The steel erector had eight ironworkers on the site. The inspector on bridges B1 and B2 visited the Bridge B3 site before the installation of the bolts and discussed the purpose of the study with the site inspector and foreman of this job. Installation of bolts on bridge B3 was done slightly differently than the procedure used for bridges B1 and B2.

Prior to installation of any bolts, the two inspectors and the foreman installed three bolts (of the type used in the web splice) in a Skidmore-Wilhelm hydraulic bolt load indicator. They established the torque required to produce a bolt tension of 195 kN, which is 12% greater than the specified minimum tension. Presumably, this measurement of torque was for inspection purposes. They also measured the rotation required to reach this load, and established that this value was a little less than 1/2 turn. The inspectors therefore instructed the foreman to use 1/2 turn of the nut for installation purposes. Installation then proceeded in a manner generally similar to that used for bridges B1 and B2. There was again some difficulty in removing drift pins and one pin had to be drilled out. The markings on the chuck of the impact wrench were clear and it appeared to the observers that the operator turned the nuts of the flange bolts 1/2 turn past snug. The web bolts appeared to have been installed to about 1/3 turn. However, the installation operation was continuous: the operator did not mark the location of the nut (or the chuck) at first impact of the wrench. The procedure was recorded by the writers on video tape.

2.3.3.2. Testing

The location of the power supply limited the investigation of bolt pretension to the splices at one end of the bridge only. A total of 39 bolts were sampled for bolt tension. Sketches of the splices examined and the location of individual bolts within those connections are shown in Fig. 3.

In order to overcome some of the difficulties described earlier in obtaining reproducible readings, almost all bolt lengths in bridge B3 were read without removal of the bolt gage transducer. This required unloading of the bolt by removal of the nut using a spud wrench rather than the impact wrench. This process proved to be more time-consuming, but it is believed to produce better results.

After the transducer was affixed to the bolt head, it was rotated a small amount ($\pm 15°$ relative to the initial location) in order to establish whether or not the length reading was stable. When it was, this location was marked on both the bolt head and transducer for future reference. The bolt temperature (T_2) was measured and entered into the bolt gage and the initial reading of length recorded (L_2). With the transducer remaining in position on the bolt head, the nut was completely loosened and a final reading of the length recorded (L_1). Because only a short period of time elapsed between the two length readings, there was no need to measure the temperature at the time of reading the final length.

2.3.3.3. Results

The field measurements for bridge B3 are shown in Tables 11 through 14. Of 39 bolts sampled, only one showed unreasonable results. Although there were four bolts that showed two sets of readings, the reported measurements were still accurate. The measurements of these bolts were taken in the stable zone where the length displayed by the bolt gage did not switch suddenly from one reading to a much different one.

2.4. Laboratory Tests

2.4.1. Material Properties

Three bolts from each grip length were tested under direct tension in order to determine their material properties. At least one bolt from each size was a new bolt taken from the lot of bolts used in the bridges. The remainder were bolts that had been taken from the bridges in the bolt tension sampling operation. This was done for reasons of economy. As will be seen, the results were independent of whether new or used bolts were tested.

The loading frame shown in Fig. 4 was used to load the bolts. The lower frame rests on the base of the testing machine and captures the head of the bolt-nut assembly as it passes through the bolt plate (Fig. 4). An upper frame passes freely through the bolt plate and is used to capture the nut end of the assembly. When a downward motion of the head of the testing machine is introduced, the upper frame pushes down against the nut and the reaction is taken by the head end of the assembly where it contacts the bolt plate of the lower frame. Some details of the parts of the assembly are shown in Fig. 5. Of course, different arrangements are possible (5).

As the bolt is loaded in tension, the elongation of the bolt was measured by the extensometer shown in Fig. 6. Since both bolt force and elongation are available, both the stiffness of the assembly in the elastic region of response and the entire load vs. deformation response can be obtained. However, these tests were intended primarily to obtain the ultimate tensile strength of the bolts, not the bolt stiffness, and therefore the grip length of the bolts corresponded only approximately to that which would be present in the corresponding bolt sizes in the bridges.

Figure 7 shows the measured load vs. deformation response of the bolts tested in this program. Since it was necessary to remove the extensometer prior to bolt failure, the curves in this figure do not show the ultimate load.

The ultimate tensile load of the nine bolts tested in direct tension are listed in Table 15. The results are consistent within each group and there is little strength variation between the groups.

2.4.2. Load vs. Deformation Relationship for Bolts Tested

Bolt tension can be estimated from the measured change in length using the following equation (4):

$$P = \Delta L \Big/ \left(\frac{L_b}{E A_b} + \frac{L_s}{E A_s}\right) \qquad (3)$$

where P is the bolt tension load,

E is the elastic modulus

ΔL is the measured change in length of the bolt

L_b is the length of the true body plus one-half the thickness of the head

L_s is the length of the exposed threads plus one-half the thickness of the nut

A_b is the area of the body

A_s is the effective stress area of the threads and is defined as:

$$A_s = \frac{\pi}{4}(D - 0.9382\ P)^2 \tag{4}$$

where D is the body diameter

P is the pitch of the threads (mm).

In a similar study, Grgas (5) compared the bolt tension loads estimated using Eq. 3 to those obtained through laboratory tests. He reported that Eq. 3 predicts loads smaller than those measured in the tests by about 5 to 10%.

In order to achieve maximum accuracy in transforming measured lengths to bolt tension, a measured bolt stiffness rather than a calculated bolt stiffness should be used. The load frame used to obtain the bolt tensile strength and described in Section 2.4.1 can also be used to obtain the bolt stiffness. For this specific purpose, bolts were loaded to 260 kN and then unloaded, with force and elongation measurements taken at every 10 kN of unloading. Typical results are shown in Fig. 8.

Table 16 is a summary of the bolt stiffness results. For each bridge, three bolts from each of the flange and web splice bolts were used. These bolts were the same ones sampled in the field tests. In this way, and because the stiffnesses were obtained by unloading the bolts, differences in stiffness between the field condition and the laboratory results are minimized. To introduce an even greater level of accuracy in the stiffness measurements, each bolt examined for pretension in the field could have been subsequently subjected to the laboratory determination of its stiffness. Availability of the bolt gage was limited, however, and it was necessary to sample some bolts as a group. The error associated with assuming the same stiffness for all bolts of the same connection and grip length is believed to be minimal, however; it is probably less than 5%. This is of the same order of difference in stiffness of any two bolts in one group.

The measured bolt stiffnesses for the different grip lengths are shown in Table 16. Good consistency in the measured stiffnesses is noted. The maximum difference between any individual test and the mean value for that group did not exceed 2.5%. It can also be noted that the ratio between the stiffnesses of two bolts of similar grip lengths is close to the ratio between their grip lengths. Thus, if the measured grip length differed slightly from the one used in determining the stiffness, say 34 and 36 mm, then a linear adjustment of the calculated stiffness can be made. However, if the differences in grip length are significant (e.g. 34 and 52 mm), then the linear adjustment procedure should not be used.

3. Test Results

3.1. Bolt Material Properties

The tensile strength of the bolts used in the study reported herein was established as described in Section 2.4.2 As shown in Table 15, a total of nine bolts were tested to obtain their tensile capacity, three from each fastener size. The results are consistent within each group and the three groups have nearly the same strength. Because the differences between the groups are so small, it was decided to treat all bolts as though they had the same tensile strength is 298.6 kN, with a standard deviation of 7.0 kN. Using a tensile stress area[1] of 298 mm^2, this average bolt ultimate tensile strength corresponds to an ultimate tensile stress of 1002 MPa.

The specified minimum tensile strength for ASTM A325 bolts whose diameter is 1 in. or less is 120 000 psi, or 827 MPa (6). Thus, these bolts are, on average, 21% stronger than their specified minimum strength. This is consistent with the results found by others. In the University of Toronto study on measured field pretension of high-strength bolts (5), the figure for A325 bolts was also found to be 21%. In the *Guide to Design for Criteria for Bolted and Riveted Joints* (3), it is reported that A325 bolts through 1 in. diameter exceed their specified tensile strength by an average of 18%.

3.2 Measured Bolt Pretensions

Table 17 gives the relevant statistical information for the bolt pretensions in each bridge and the collective result for all bridges taken together. Given in each case are the number of bolts in the sample and the mean value and standard deviation of the bolt

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[1] Calculations requiring the use of bolt properties (either dimensional or material) were done first using U.S. Customary units and then soft-converted. The bolts used in this project were manufactured according to specifications that use U.S. Customary units.

pretensions. The specified minimum pretension for bolts of this grade and diameter is 174 kN. In all cases, this value was exceeded even at one standard deviation less than the mean. Of course, there were individual bolts that did not reach the specified minimum preload of 174 kN: a total of five bolts in the sample of 104 did not reach the required pretension.

Table 17 provides both overall results for each bridge and the individual results for flange bolts and web bolts in a given bridge. In bridges B1 and B3 the measured pretension in the flange bolts is slightly greater than that in the web bolts. In bridge B2, the pattern is reversed. Considering all bolts in a given bridge, the results obtained for bridges B1 and B3 are nearly identical (mean pretensions of 211.9 kN and 212.3 kN), while the result for bridge B2 is somewhat higher (242.2 kn). It will be recalled that bridges B1 and B2 were identical and were erected by the same crew of ironworkers and with the same inspector. It is a possibility that the process of observation of the erection of Bridge B1 by the writers had a subsequent effect on bolt installation in Bridge B2.

Figures 9 through 12 show the measured bolt pretensions in pictorial form. Figures 9, 10, and 11 are frequency histograms of the bolt pretensions in Bridges B1, B2 and B3, respectively. Figure 12 is a frequency histogram of the bolt pretensions in all bridges taken together.

Another way of looking at the data is to use the ratio of measured bolt pretension to specified minimum bolt pretension. Only the collective results are given here, that is, the results for all bridges taken together. This is shown as the frequency histogram in Figure 13. It shows that the ratio of measured bolt pretension to specified minimum bolt pretension has a mean value of 1.30 for the 104 bolts. The standard deviation is 0.21.

Comparison of the results reported herein and the University of Toronto study (5) is appropriate. As already noted, in this study the ratio of measured bolt pretension to specified minimum bolt pretension is 1.30 (standard deviation 0.21). In the Toronto study, the ratio was 1.16 (standard deviation 0.16) for the total population of 212 A325 bolts. This included both turn-of-nut and load indicating washer installations and both bridge and building sites. If the data obtained only from bridges are addressed, the ratio is 1.24 (standard deviation 0.13) for the 55 A325 bolts in this category. Thus, the data for A325 bolts installed in bridges are in reasonable agreement when comparing the results reported herein and the Toronto results.

It can be hypothesized that bolts installed at bridge sites are likely to be subjected to somewhat more scrutiny during their installation and inspection than are bolts installed in buildings. In the first place, the need for slip-critical connections is probably more recognized in bridges than it is in buildings and therefore more attention might therefore be paid to attainment of the pretension. The lack of adherence to the requirements of the turn-of-nut method of installation that was observed in both this study and the Toronto study (5) would seem to argue against this, however. Perhaps more importantly, the bolt installation in bridges is usually done in an atmosphere that is conducive to good installation: the erection crew at at bridge site is less likely to be competing with other trades for access to connections and use of equipment than they would be at a building site. Furthermore, proper staging or scaffolding for access to the connections receives a high priority at bridge sites. Likewise, it is likely that the inspection process is more likely to be focussed on the bolt installation process at the bridge site than it is at a building because there are fewer inspection processes going on. All of these conjectures are subjective, however. Nevertheless, the Toronto data showed quite clearly that the installed pretension in bolts at bridge sites was significantly higher than it was at building sites. At bridge sites, the ratio of measured bolt pretension to specified minimum bolt pretension was 1.24 (standard deviation 0.15), whereas it was 1.11 (standard deviation 0.15) at non-bridge sites.

Finally, the data can be non-dimensionalized on the basis of the actual strength of the bolts used to obtain the data. As already noted, the average measured tensile strength of the bolts is 297.5 kN. If the measured bolt tensions are divided by this quantity, the mean value of the ratio is 0.75 and the standard deviation of the results is 0.14. Of course, the distribution of the results (not plotted herein) will look the same as the distribution given in Fig. 13. Comparable values for the Toronto study were not reported, although they could be easily calculated from the original data (not currently available to the writers).

3.3. Application of the Results

It is reasonable to conclude from results reported herein and the results of the Toronto study that the measured pretensioned in field-installed bolts are well above the specified minimum values. However, it does not necessarily follow that the situation is therefore satisfactory: it is necessary that the results be examined in light of how the design rules for slip-critical connections have been derived.

The slip resistance of a bolted connection is given by:

$$P_s = m\, n\, T_i\, k_s \tag{5}$$

where P_s = the slip load
m = number of slip surfaces
n = number of bolts
T_i = clamping force provided by a bolt
k_s = slip coefficient

The essential variables that describe the capacity of a slip-critical connection are the slip coefficient of the steel that makes up the joint, k_s, and the clamping force supplied by the bolts, T. Neither of these is deterministic. The approach that was taken by the writers of the RCSC Specifications (2,3) is that the product of these two variables would reflect their mean values and coefficients of variation. As explained in the *Guide* (3), it was also assumed that the distributions of each of these variables could be taken as normal, an assumption reasonably supported by the experimental data. Finally, after the product of the two variables has been obtained, a slip probability level must be selected. As expressed in the *Guide* (eq. 5.26), the slip load is given by:

$$P_s = D\, m\, n\; T_{i\,spec}\, k_{s\,mean} \tag{6}$$

where $T_{i\,spec}$ = specified minimum strength of the bolt material

$k_{s\,mean}$ = mean value of the slip coefficient

Since a designer will simply wish to deal with the mean value of the slip coefficient and with the specified value of the bolt tension (the quantities identified in Eq. 6) and because the real answer lies in use of the actual values of each of these (Eq. 5), the *Guide* provides the additional linkage by way of the multiplier D. This provides the relationship between $k_{s\,mean}$ and k_s, reflects the difference between the bolt pretensions actually attained and the specified bolt pretension (i.e. 70% of the ultimate strength), and incorporates a slip probability level. In Tables 5.2 and 5.3 of the *Guide*, values of D are provided for a variety of combinations of fastener grade, method of installation, mean slip coefficient, and slip probability levels. North American specifications (7, 8) follow the slip load formulation developed in the *Guide*. Specific information is given for the 5% slip probability level, although the designer is permitted to refer to the *Guide* data if another slip probability level can be justified.

Considering all of the above, it is necessary to know the specific parameters associated with the bolt preload data used in the *Guide* in order that the *in situ* bolt pretension data identified in this report can be properly evaluated. The specifics of the data used by the *Guide* are that the mean value and standard deviation of the ratio $T_i / T_{i\,spec}$ were taken as 1.35 and 0.12, respectively, for ASTM A325 bolts installed to 1/2 turn-of-nut. This mean value was obtained as follows -

Bolt material: $\sigma_{u\ actual} = 1.183\ \sigma_{u\ spec}$

Bolt pretension: measured value = $0.80\ \sigma_{u\ actual}$

Thus, the bolt pretension can be expressed as -

$(1.183 \times 0.80)\ \sigma_{u\ spec} = 0.95\ \sigma_{u\ spec}$

Since the RCSC Specification requires that the bolt preload be (at least) $0.70_{u\ spec}$, this means that the mean measured bolt pretension is (0.95 - 0.70)/0.70 = 35% greater than the specified minimum ultimate tensile strength, hence the use of 1.35 in the *Guide* as the mean value of the ratio $T_i / T_{i\ spec}$. (Not shown here is the derivation of the standard deviation of this quantity, which is given as 0.12 in the Guide).

Using the data measured in the study reported herein, a figure corresponding to the 1.35 value used by the *Guide* in its derivation of the design rules can be calculated. Since this study found that

$\sigma_{u\ actual} = 1.21\ \sigma_{u\ spec}$

and that the measured bolt pretensions are 0.76 times the actual bolt material ultimate strength, the measured bolt tensions are 1.21 x 0.76 = 0.92 times the specified strength. The result is that the figure comparable to the 35% value used by the *Guide* is now calculated to be 31%. This is reasonably close to the value that was used by the *Guide* to develop the design rules for slip-critical connections. In other words, the results of this study confirm the selection of the design values given by the *Guide*, even though those were based on laboratory measurement of bolt tensions and not values obtained from field measurements. Once the specific data are available from the Toronto study, they can be used to increase the size of the data base and the numbers recalculated. Since the Toronto results and the results reported here are generally in agreement, it is not anticipated that the conclusion stated above is likely to change.

4. Summary and Conclusions

This study has reported on the installation of bolts in highway bridges at three different sites. Two of the bridges were identical and they were erected by the same crew of ironworkers and inspected by the same representative of the owner. The third bridge was of different configuration and was erected by a different contractor. The inspection was done by a different representative of the owner, although it was noted that the inspector on the first two bridges communicated with the inspector on the third bridge.

Following installation of the high-strength bolts and the observation thereof by the writers, a representative number of bolts were selected for measurement of pretension. This was accomplished by first measuring the length of the installed bolt by means of an ultrasonic bolt gage, removing the bolt and then remeasuring the length of the bolt. The bolt was then calibrated in the laboratory so that its change in length could be related to the pretension that had existed in that bolt. Experience showed that best results were obtained when the transducer of the bolt gage remained on the bolt during the measurement of the change in length.

The following observations and conclusions are reported:

1. Each job site was properly equipped for the installation of high-strength bolts and this equipment included at least one hydraulic bolt calibrator (Skidmore-Wilhelm). Sometimes both the inspector (owner's representative) and the erector (ironworker crew) had a Skidmore-Wilhelm calibrator.

2. The owner's requirement was that the bolts be installed by the turn-of-nut method, as outlined by the RCSC Specification. The installation procedure followed by the erection crews was not in conformance with these requirements in several respects. Specifically, the following items were noted -

(a) The requirement for the amount of turn necessary for installation of a given bolt length appeared to be determined on the basis of the behavior of a sample of bolts installed in the Skidmore-Wilhelm calibrator. Although this literally is not in conformance with the RCSC Specification, it should not result in any reduction in bolt performance.

(b) Notwithstanding how the desired amount of turn was established, the installation of a bolt was one continuous operation, that is, the socket of the impact wrench was placed on the nut and then the nut turned continuously until the operator considered the procedure completed. This means that the location of the socket at the location of snug-tight was not marked or noted and the subsequent rotation of the socket to some location like one-half or one-third turn past snug was not identified. It appeared to the observers that the installation was by "sound".

(c) The inspectors used the torque value observed in the Skidmore-Wilhelm calibrator for inspection purposes, which is a satisfactory procedure. However, in at least the case of bridges B1 and B2, the inspector was of the opinion that the bolt pretension must be neither less than that required (which is correct) nor greater than the required value (which is incorrect). Nevertheless, it was never observed that the inspector rejected a bolt because of a pretension in excess of that required.

3. In this study, pretensions were measured in 104 bolts. These were all ASTM A325 Type 3 bolts, all were 22 mm (7/8 in.) diameter, and all were installed by the turn-of-nut method. The bolts were installed in either web of flange splices of welded plate girders. The measured pretensions and basic bolt properties lead to the following conclusions:

(a) The bolts tested in this study were, on average, 21% stronger than their specified minimum strength. This is consistent with results found by others.

(b) The measured bolt pretension was found to be 1.30 times the specified minimum bolt pretension. The result is consistent with the results found by others for A325 bolts installed in bridges.

(c) The RCSC Specification rules for slip-critical connections are based on the premise that the installed bolt pretension will be 35% greater than specified minimum ultimate tensile strength of the bolt material. This value was established on the basis of bolts installed in joints made up in the laboratory. The result of the field study of bolt pretensions reported herein is that the installed bolt pretension is 31% greater than specified minimum ultimate tensile strength of the bolt material.

The most important finding of this study is that the characteristics of pretension of ASTM A325 bolts installed in bridges are consistent with those characteristics used for the development of the design rules published by the Research Council on Structural Connections. The authors observed that the installation of bolts in this study was not in conformance with several of the requirements for the turn-of-nut method of installation as laid out in the RCSC Specification. Since the results (measured pretensions) were nevertheless satisfactory, it may be observed that installation leads to satisfactory results in spite of a lack of conformance with the requirements of the specification. This fortuitous result is the consequence of the flat load versus deformation response of high-strength bolts installed by the turn-of-nut method, a feature that has been recognized for a long time. However, the lack of conformance with the specification is disappointing and the efforts of the Research Council on Structural Connections to disseminate knowledge of the proper procedures should be continued.

5. References

1. Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts. Research Council on Structural Connections of the Engineering Foundation, November 1985. (Available through the American Institute of Steel Construction, Chicago, Illinois 60611.)

2. Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections of the Engineering Foundation, June 1988. (Available through the American Institute of Steel Construction, Chicago, Illinois, 60611.)

3. Guide to Design Criteria for Bolted and Riveted Joints, Second Edition, by G.L. Kulak, J.W. Fisher, and J.H.A. Struik, John Wiley and Sons, Inc., New York, 1987.

4. An Introduction to the Design and Behavior of Bolted Joints, Second Edition, by J.H. Bickford, Marcel Dekker, Inc., New York, 1990.

5. Field Investigation and Evaluation of the Pretension of High Strength Bolts, by Nick Grgas, M.A.Sc. Thesis, University of Toronto, 1990.

6. Standard Specification for High-Strength Bolts for Structural Steel Joints, ASTM A325-83c, American Society for Testing and Materials.

7. Load and Resistance Factor Design Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, 1986.

8. Limit States Design of Steel Structures, CAN/CSA-S16.1-M89, Canadian Standards Association, Rexdale, Ontario, 1989.

Table 1 Calibration Factors for 22 mm A325 Bolts

	No.	Bridge B1	Bridge B2	Bridge B3
Material velocity (M_v) (m/sec)	1	5955.77	5929.73	5927.75
	2	0.00	5926.96	0.00
	3	0.00	5926.56	0.00
	Avg.	5955.77	5927.75	5927.75
Stress factor (S_f)	1	0.280	0.281	0.278
	2	0.282	0.280	0.275
	3	0.278	0.278	0.272
	Avg.	0.280	0.280	0.275

Table 2 Bolt Tensions - Connection N2 of Bridge B1

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B1-N2-F1	95.771	94.224	50.70	444.0	1,3
B1-N2-F2	95.199	94.409	50.99	227.9	
B1-N2-F3	95.479	94.597	51.30	255.8	
B1-N2-W1	82.741	82.169	33.96	203.0	
B1-N2-W2	82.667	82.085	34.30	208.4	
B1-N2-W3	82.956	82.349	34.25	217.2	
B1-N2-W4	82.677	82.118	34.60	201.8	
B1-N2-W5	82.743	81.813	34.35	333.6	1,3
Temperature	5°C	5°C			

[1] Not repeatable in laboratory

[3] Will be rejected as unreasonably high

Table 3 Bolt Tensions - Connection N3 of Bridge B1

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B1-N3-F1	95.501	94.836	51.23	192.9	
B1-N3-F2	94.762	93.985	50.93	224.0	
B1-N3-F3	95.357	94.539	51.40	237.8	
B1-N3-W1	82.487	81.928	35.60	207.1	
B1-N3-W2	82.847	82.179	34.80	242.5	
B1-N3-W3	82.169	81.679	34.87	178.3	1
B1-N3-W4	82.342	81.996	34.73	125.2	
B1-N3-W5	82.304	81.747	34.40	199.9	
Temperature	4°C	6°C			

1 Not repeatable in laboratory

Table 4 Bolt Tensions - Connection N4 of Bridge B1

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B1-N4-F1	95.418	94.747	50.87	193.1	
B1-N4-F2	95.369	94.605	51.45	222.5	
B1-N4-F3	95.199	94.508	51.80	202.3	
B1-N4-W1	82.278	81.719	34.30	200.2	
B1-N4-W2	82.829	82.166	34.55	239.1	1
B1-N4-W3	82.461	81.821	34.05	227.9	
B1-N4-W4	82.634	82.004	34.26	225.5	
B1-N4-W5	82.799	82.156	34.09	229.0	
Temperature	5°C	5°C			

1 Not repeatable in laboratory

Table 5 Bolt Tensions - Connection N2 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-N2-F1	95.700 95.740 95.717	94.938 94.940 94.940	50.70	225.5	
B2-N2-F2	95.832 95.834 95.893	94.889 94.938 94.912	51.40	275.2	1
B2-N2-F3	95.225 95.263 95.298	94.625 94.628 94.663	52.60	186.5	
B2-N2-W1	82.725 82.715 82.723	82.100 82.083 82.093	33.96	220.4	2
B2-N2-W2	82.527 82.553 82.560	81.923 81.907 81.915	34.40	223.9	
B2-N2-W3	82.682 82.692 82.695	82.042 92.045 92.042	34.20	228.1	1
B2-N2-W4	82.700 82.735 82.743	82.029 82.042 92.037	34.60	245.9	
B2-N2-W5	82.875 82.873 82.885	82.174 82.166 82.169	34.42	251.1	
Temperature	9 - 10°C	9 - 10°C			

1 Not repeatable in laboratory

2 Two sets of readings

Table 6 Bolt Tensions - Connection N3 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-N3-F1	95.700 95.745 95.720	94.793 94.775 94.780	50.99	272.9	
B2-N3-F2	95.247 95.242 95.258	94.785 94.762 94.757	51.15	140.2	1
B2-N3-F3	96.243 96.223 96.238	94.587 94.579 94.585	51.08	480.7	3
B2-N3-W1	82.715 82.723 82.723	82.088 82.075 82.080	33.80	225.4	
B2-N3-W2	83.025 83.025 83.035	82.370 82.377 82.354	34.40	236.9	2
B2-N3-W3	82.883 82.890 82.893	82.255 82.210 82.222	34.50	237.0	
B2-N3-W4	82.481 81.487 82.481	81.971 81.961 81.966	34.10	183.9	
B2-N3-W5	83.472 83.495 83.515	82.151 82.103 82.133	34.15	485.7	3
Temperature	9 - 10°C	9 - 10°C			

1 Not repeatable in laboratory

2 Two sets of readings

3 Will be rejected as unreasonably high

Table 7 Bolt Tensions - Connection N4 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-N4-F1	95.806 95.827 95.827	94.851 94.897 94.889	51.20	273.5	2
B2-N4-F2	95.740 95.735 95.722	95.222 95.212 95.214	51.15	150.0	
B2-N4-F3	96.098 96.096 96.114	94.800 94.800 94.806	51.20	378.1	3
B2-N4-W1	83.157 83.144 83.155	82.385 82.365 82.360	34.00	281.9	
B2-N4-W2	83.182 82.182 83.185	82.385 82.385 82.387	34.00	287.4	1
B2-N4-W3	83.073 83.086 83.081	82.337 82.309 82.301	34.00	275.5	
B2-N4-W4	82.286 82.283 82.281	81.562 81.562 81.570	34.10	259.7	
B2-N4-W5	82.954 82.974 82.974	82.151 82.103 82.133	34.14	303.2	3
Temperature	9 - 10°C	9 - 10°C			

1 Not repeatable in laboratory

2 Two sets of readings

3 Will be rejected as unreasonably high

Table 8 Bolt Tensions - Connection S2 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-S2-F1	96.418 96.418 96.413	95.456 95.441 95.466	51.87	284.3	
B2-S2-F2	95.611 95.618 95.616	94.747 94.750 94.742	51.90	256.7	
B2-S2-F3	95.855 85.857 85.855	94.874 94.856 94.869	52.02	292.9	
B2-S2-W1	83.850 83.871 83.848	83.081 83.055 83.071	34.30	278.4	2
B2-S2-W2	83.231 83.233 83.236	82.431 82.431 82.438	34.40	283.7	
B2-S2-W3	84.351 84.328 84.287	83.500 83.515 83.505	34.20	287.5	
B2-S2-W4	83.182 83.185 83.185	82.365 82.352 82.360	34.60	294.2	
B2-S2-W5	83.182 83.177 83.190	82.398 82.377 82.400	34.42	280.8	2
Temperature	9 - 10°C	9 - 10°C			

2 Two sets of readings

Table 9 Bolt Tensions - Connection S3 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-S3-F1	95.661 95.639 95.654	94.844 94.856 94.846	51.01	233.4	
B2-S3-F2	95.395 95.413 95.405	94.559 94.579 94.577	51.20	242.9	
B2-S3-F3	95.745 95.766 95.768	94.839 94.836 94.831	51.08	269.2	
B2-S3-W1	82.946 92.926 82.939	82.255 82.271 82.266	34.00	238.7	
B2-S3-W2	82.961 82.931 82.916	82.266 82.258 82.266	34.40	241.2	
B2-S3-W3	82.951 82.949 82.951	82.268 82.227 82.263	34.50	250.6	
B2-S3-W4	82.873 82.865 82.880	82.235 82.260 82.260	34.10	220.6	
B2-S3-W5	82.822 82.802 82.807	82.177 82.131 82.126	34.15	236.8	
Temperature	9 - 10°C	4 - 5°C			

Table 10 Bolt Tensions - Connection S4 of Bridge B2

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B2-S4-F1	95.910 95.900 95.903	94.925 94.905 94.907	51.00	288.5	
B2-S4-F2	95.573 95.575 95.585	94.915 94.910 94.907	51.20	194.7	
B2-S4-F3	96.652 96.627 96.670	95.885 95.631 95.987	51.08	237.4	2
B2-S4-W1	82.423 82.431 82.431	81.935 91.948 81.953	34.00	171.1	
B2-S4-W2	83.403 83.416 83.429	82.649 82.603 82.629	34.40	282.8	2
B2-S4-W3	82.992 83.010 83.010	82.299 82.314 82.309	34.00	247.1	2
B2-S4-W4	82.593 82.598 82.598	81.973 81.973 81.979	34.00	220.3	2
B2-S4-W5	84.008 84.031 84.011	83.190 83.238 83.259	34.00	279.2	
Temperature	9 - 10°C	4 - 5°C			

2 Two sets of readings

Table 11 Bolt Tensions - Connection E1 of Bridge B3

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B3-E1-F1	116.279	115.425	64.00	189.1	2
B3-E1-F2	116.967	115.865	64.00	244.2	
B3-E1-F3	115.369	114.323	64.00	231.8	
B3-E1-F4	116.464	115.369	64.00	242.5	
B3-E1-F5	115.733	114.668	64.00	235.8	
B3-E1-W1	83.678	83.030	36.00	205.4	
B3-E1-W2	84.485	83.850	36.00	201.4	
B3-E1-W3	85.082	84.404	36.00	215.1	
B3-E1-W4	85.080	84.455	36.00	198.2	
B3-E1-W5	83.035	82.420	36.00	195.0	
Temperature	8°C	8°C			

Table 12 Bolt Tensions - Connection E2 of Bridge B3

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B3-E2-F1	115.611	113.792	64.19	402.9	3
B3-E2-F2	115.075	114.137	64.19	207.6	
B3-E2-F3	115.380	114.417	64.19	213.3	
B3-E2-F4	114.310	113.830	64.19	106.4	
B3-E2-F5	115.179	114.242	64.19	207.6	
B3-E2-W1	84.160	83.503	36.35	208.7	
B3-E2-W2	83.604	82.931	36.35	213.5	
B3-E2-W3	82.906	82.266	36.35	203.0	
B3-E2-W4	83.558	82.865	36.35	220.0	
B3-E2-W5	83.678	82.956	36.35	228.8	2
Temperature	20°C	20°C			

2 Two sets of readings

3 Will be rejected as unreasonably high

Table 13 Bolt Tensions - Connection E3 of Bridge B3

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B3-E3-F1	114.869	113.944	63.78	204.8	
B3-E3-F2	115.357	114.376	63.78	217.2	
B3-E3-F3	115.628	114.546	63.78	239.7	
B3-E3-F4	115.611	114.536	63.78	238.0	
B3-E3-F5	114.935	113.835	63.78	243.7	2
B3-E3-W1	82.558	81.918	36.43	203.0	
B3-E3-W2	82.746	82.060	36.43	217.5	
B3-E3-W3	N/A	N/A	36.43	NA	
B3-E3-W4	82.718	82.159	36.43	177.3	
B3-E3-W5	82.771	82.199	36.43	181.3	
Temperature	20°C	20°C			

Table 14 Bolt Tensions - Connection E4 of Bridge B3

Bolt No.	Measured Lengths (mm)			Tension (Calculated) kN	Remarks
	Loaded	Unloaded	Grip		
B3-E4-F1	114.978	114.097	65.61	195.3	2
B3-E4-F2	116.296	115.245	65.61	233.0	
B3-E4-F3	115.700	114.554	65.61	253.8	
B3-E4-F4	115.235	114.165	65.61	236.9	
B3-E4-F5	115.329	114.242	65.61	240.8	
B3-E4-W1	83.391	82.733	36.80	208.7	
B3-E4-W2	84.389	83.668	36.80	228.8	
B3-E4-W3	82.718	82.080	36.80	202.2	
B3-E4-W4	83.048	82.469	36.80	183.7	
B3-E4-W5	82.530	81.918	36.80	194.2	
Temperature	28°C	28°C			

2 Two sets of readings

Table 15 Bolt Ultimate Tensile Strengths

Length (mm)	Grip (mm)	Ultimate Load (kN)	Average (kN)	Max. Diff. %	70% P_u (kN)
115.0	65.0	294.8			
115.0	65.0	307.1			
115.0	65.0	299.0	300.3	2.3	210.2
95.0	51.5	300.5			
95.0	51.5	297.4			
95.0	51.5	304.9	300.9	1.3	210.6
82.0	34.0	303.4			
82.0	34.0	297.4			
82.0	34.0	283.2	294.7	3.9	206.3

Table 16 Bolt Stiffnesses

Location	Measured Stiffness kN/m	Average Stiffness kN/mm	Std. Deviation
B1-F (52 mm grip)	1013.09		
	1006.87		
	1004.41	1008.12	4.47
B1-W (34 mm grip)	1251.59		
	1224.09		
	1212.14	1229.27	20.3
B2-F (52 mm grip)	1001.36		
	1018.56		
	1016.19	1012.04	9.32
B2-W (34 mm grip)	1208.02		
	1233.01		
	1231.56	1224.20	14.0
B3-F (65 mm grip)	808.94		
	808.66		
	813.93	810.51	2.97
B3-W (36 mm grip)	1126.77		
	1160.47		
	1173.11	1153.45	23.9

Table 17 Measured Bolt Pretensions

Bridge No	No. of Bolts	Pretension (kN)	
		Mean	Standard Deviation
B1 - Flange -Web - All	8 14 22	219.5 207.5 211.9	20.8 28.4 26.5
B2 - Flange - Web - All	16 28 44	239.0 248.9 239.9	46.7 31.1 37.8
B3 - Flange - Web - All	19 19 38	220.1 204.5 212.3	32.3 14.2 26.1
All Bridges	104	226.2	35.7

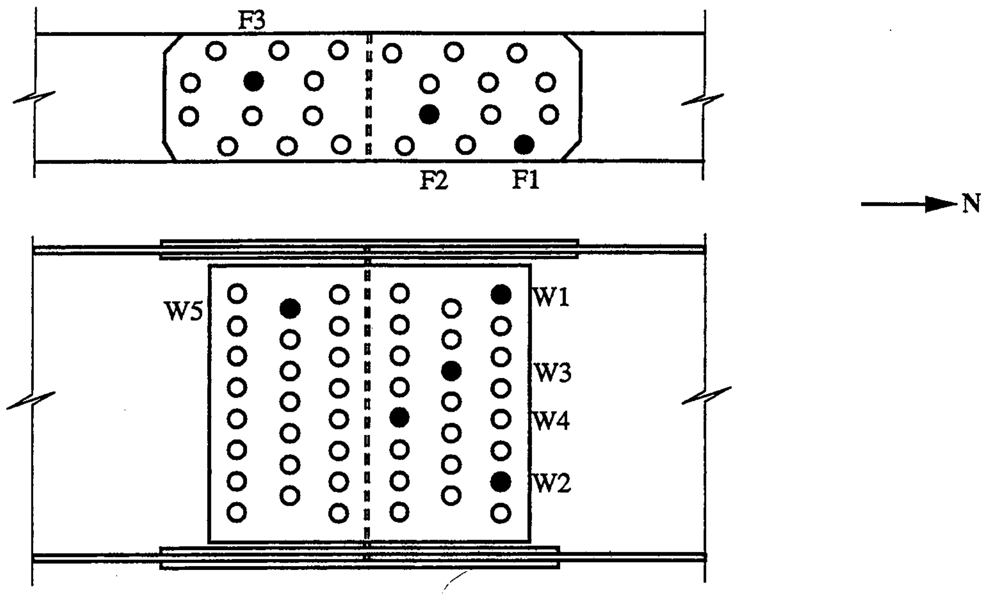

Fig. 1(a) Location of sampled bolts in connection N2 of bridge B1

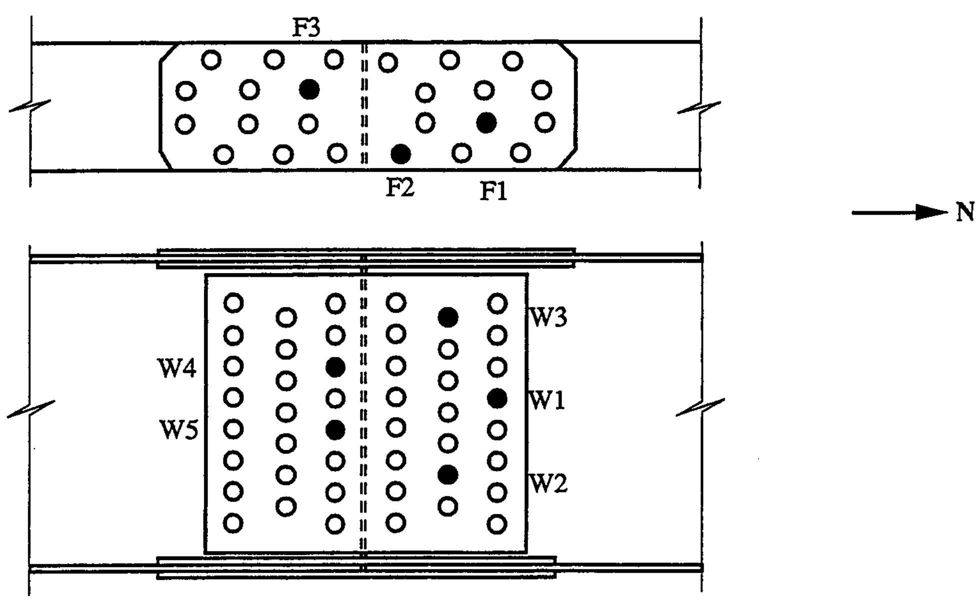

Fig. 1(b) Location of sampled bolts in connection N3 of bridge B1

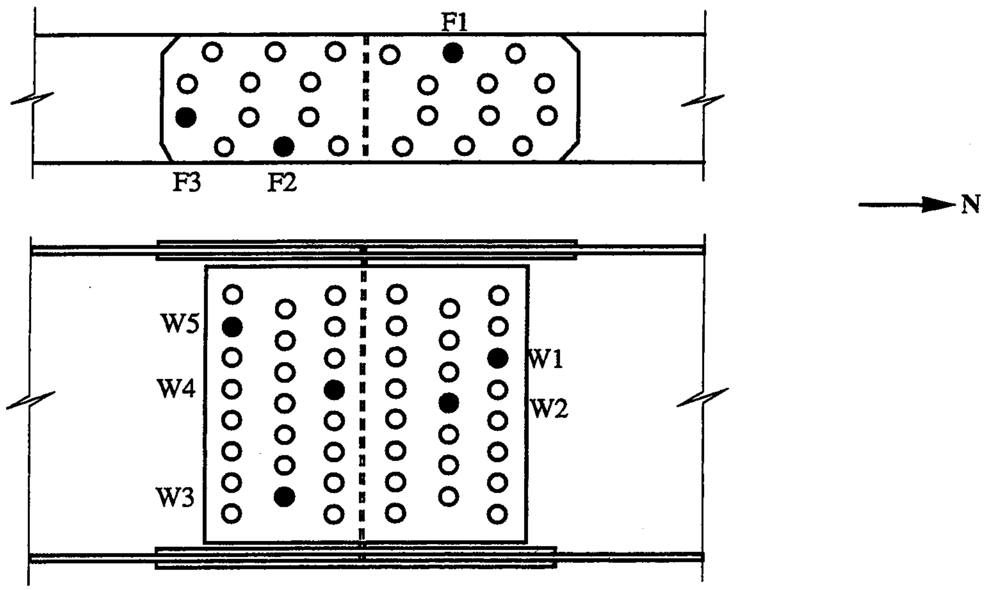

Fig. 1(c) Location of sampled bolts in connection N4 of bridge B1

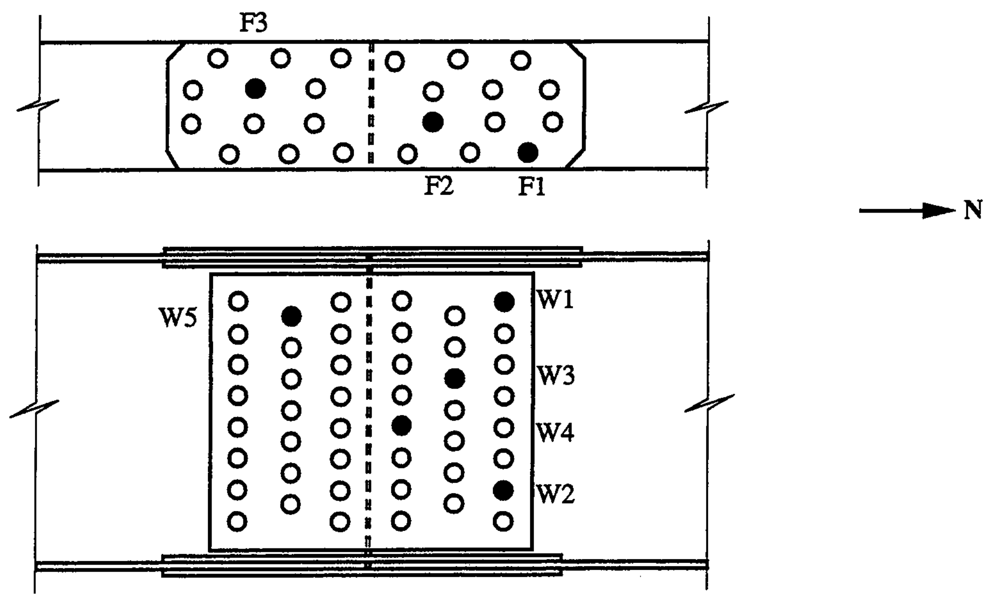

Fig. 2(a) Location of sampled bolts in connection N2 of bridge B2

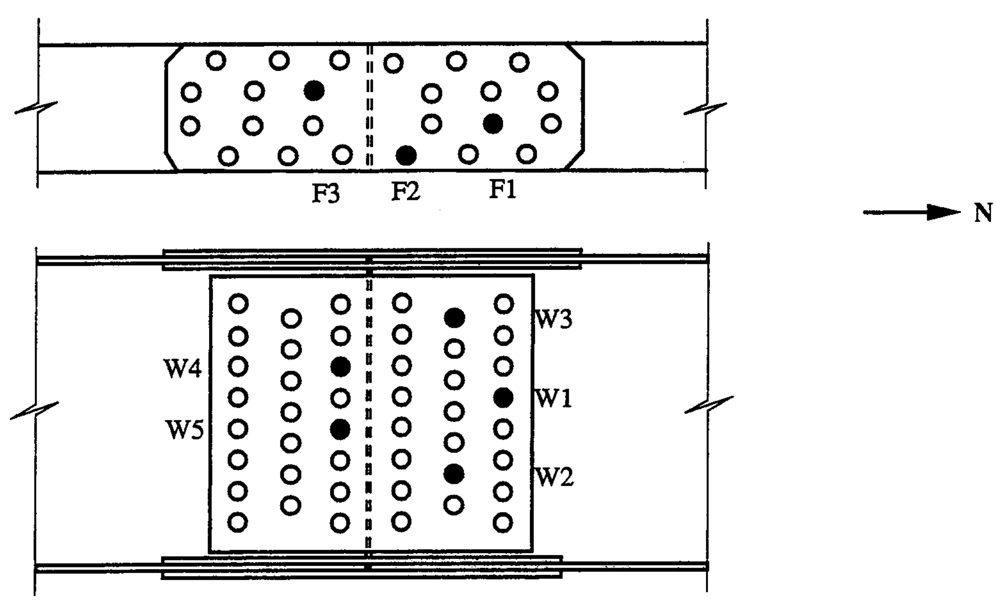

Fig. 2(b) Location of sampled bolts in connection N3 of bridge B2

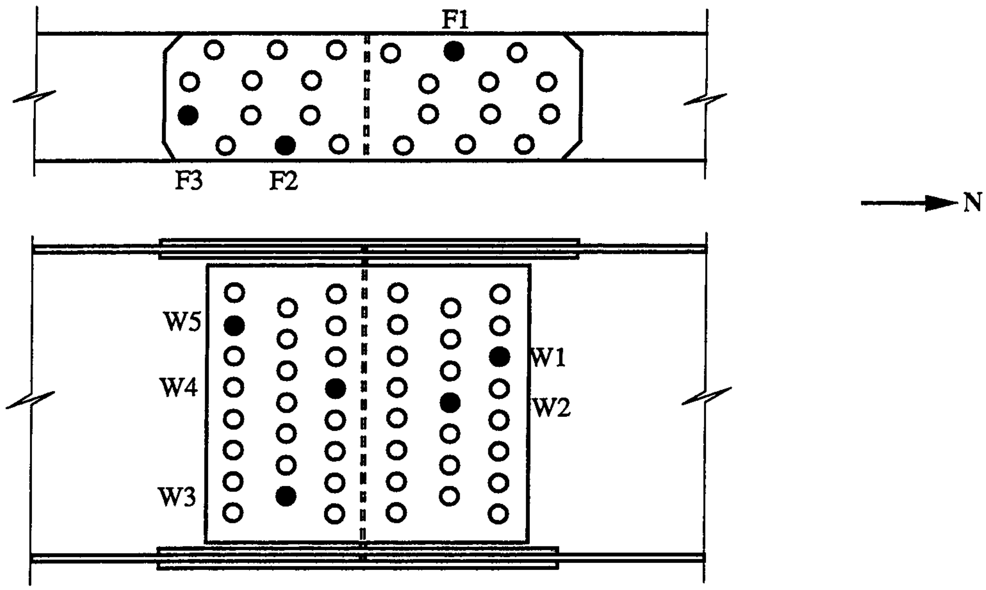

Fig. 2(c) Location of sampled bolts in connection N4 of bridge B2

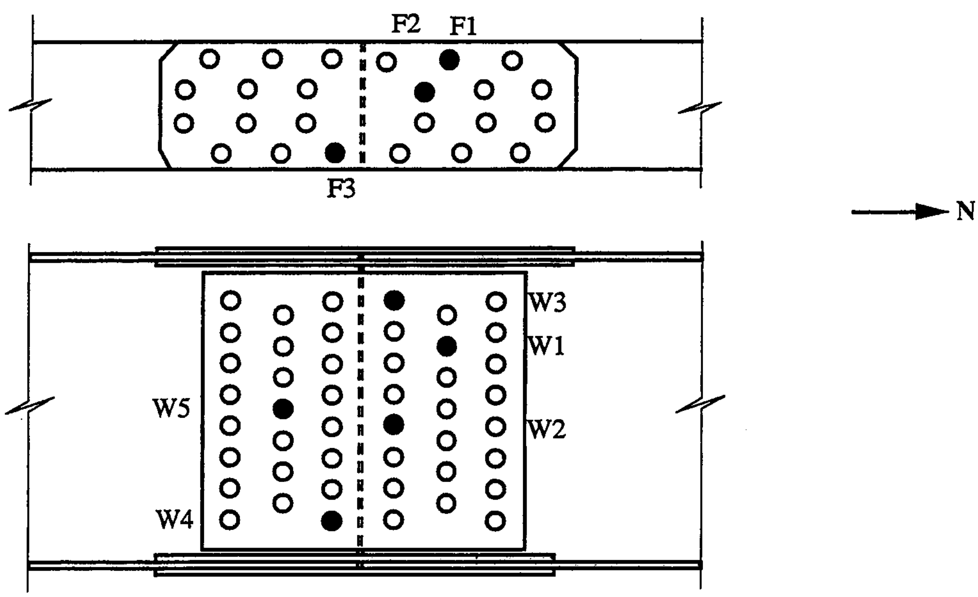

Fig. 2(d) Location of sampled bolts in connection S2 of bridge B2

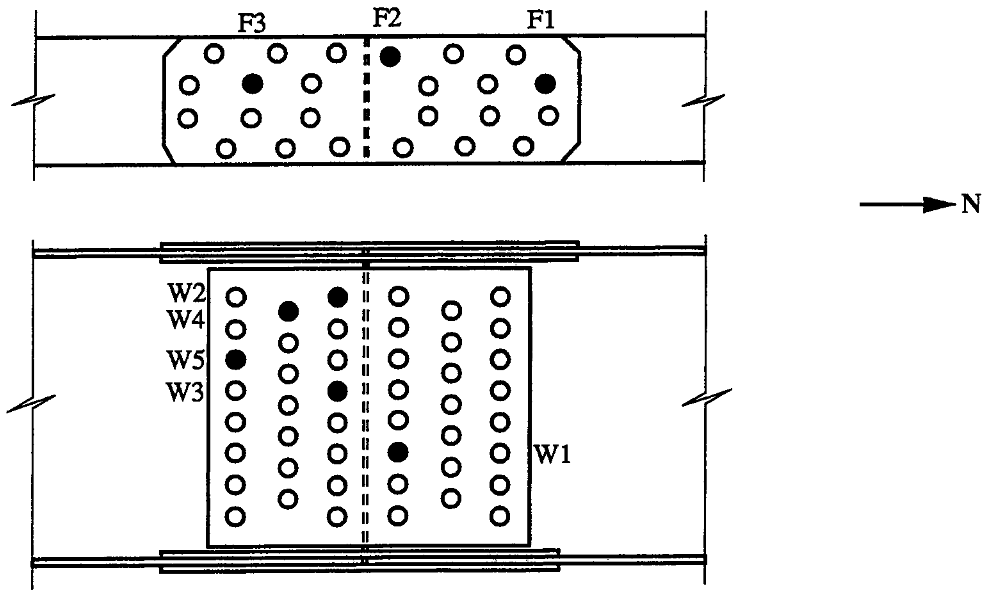

Fig. 2(e) Location of sampled bolts in connection S3 of bridge B2

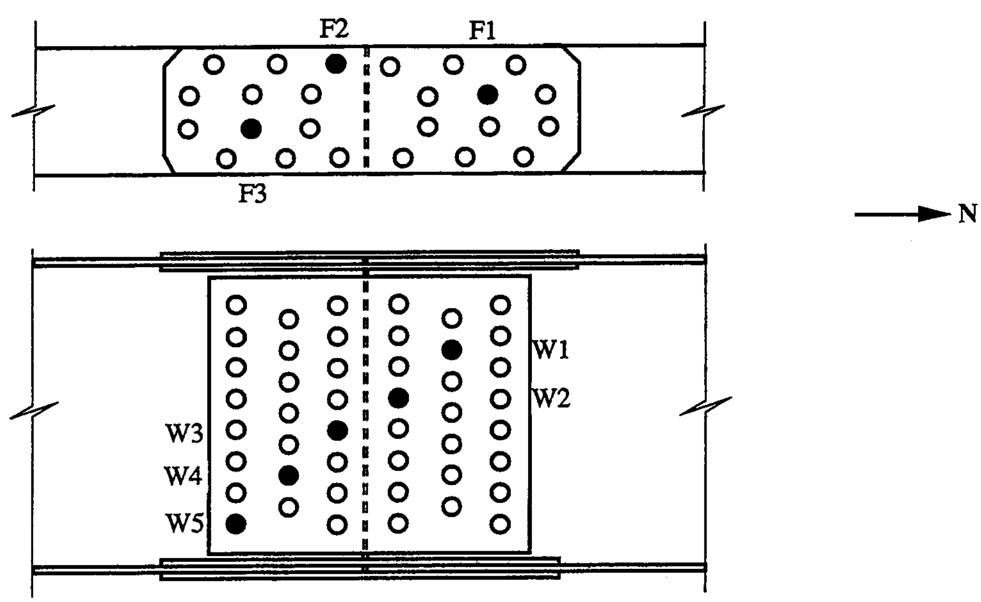

Fig. 2(f) Location of sampled bolts in connection S4 of bridge B2

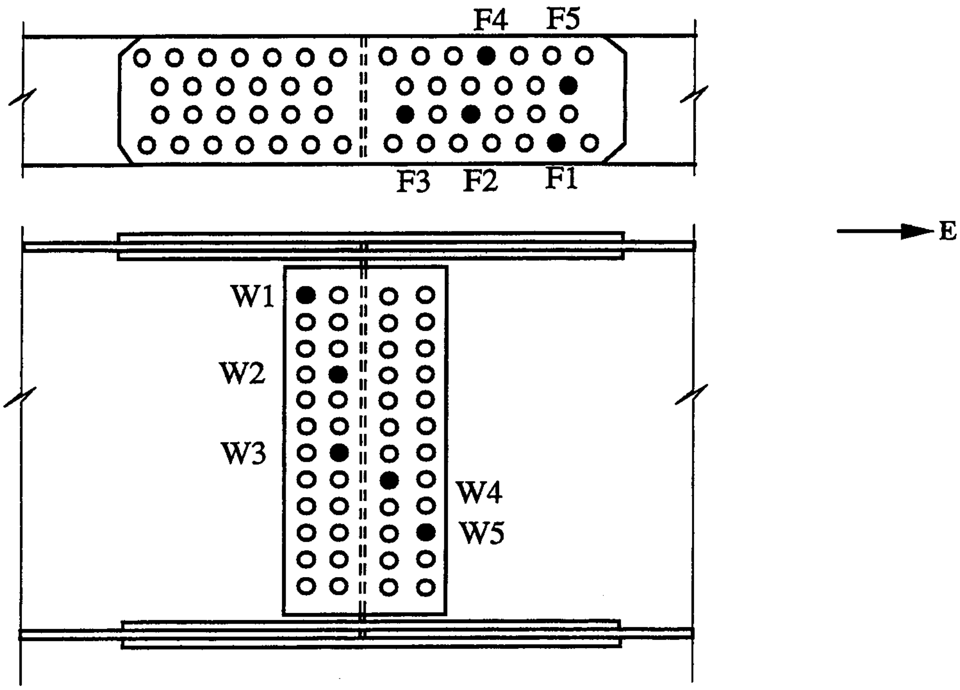

Fig. 3(a) Location of sampled bolts in connection E1 of bridge B3

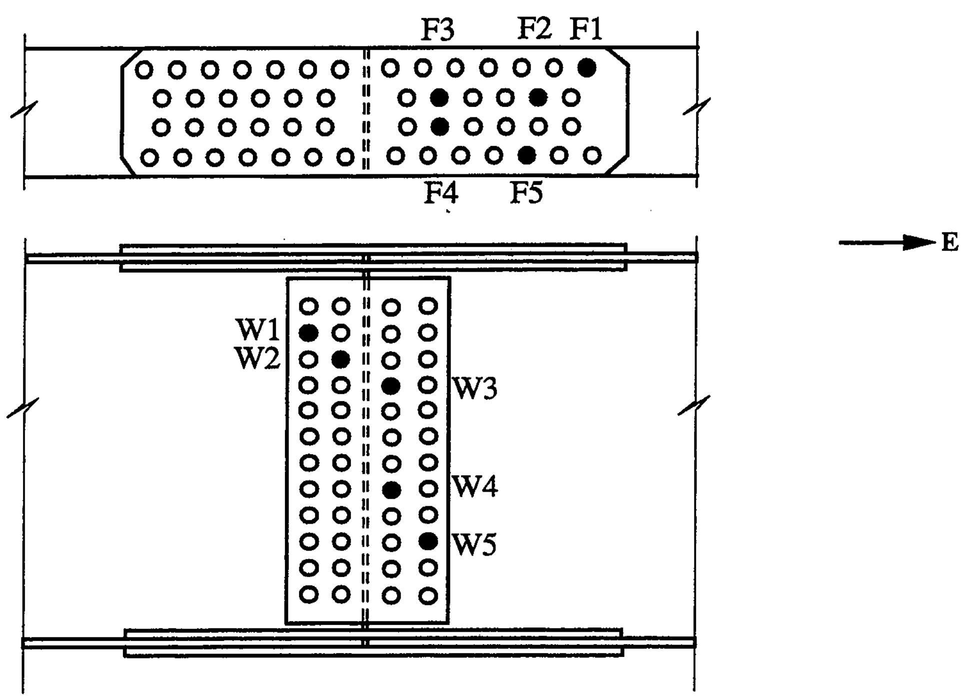

Fig. 3(b) Location of sampled bolts in connection E2 of bridge B3

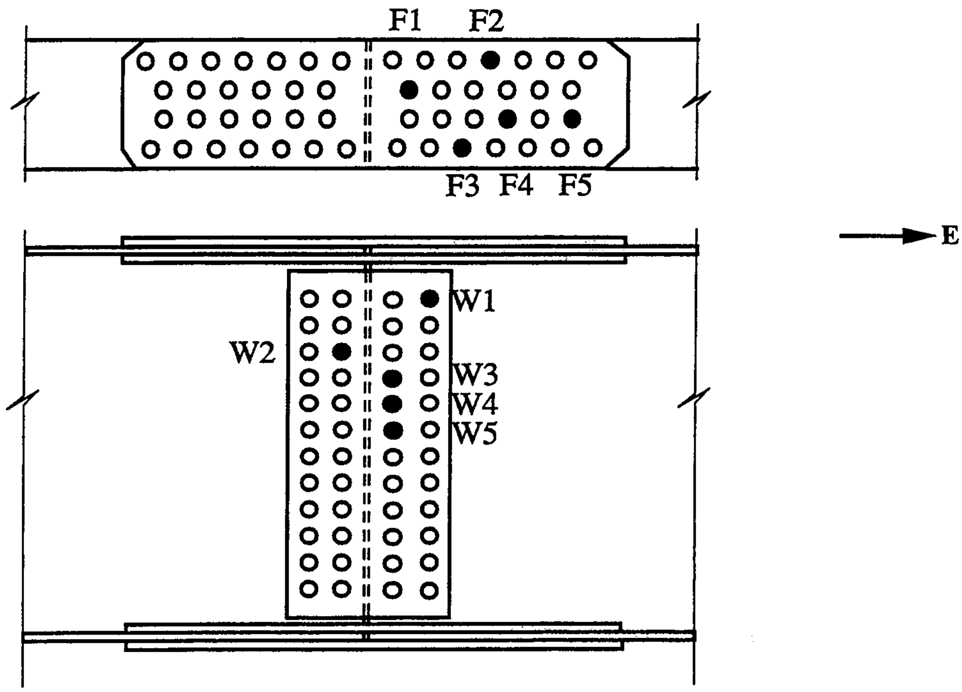

Fig. 3(c) Location of sampled bolts in connection E3 of bridge B3

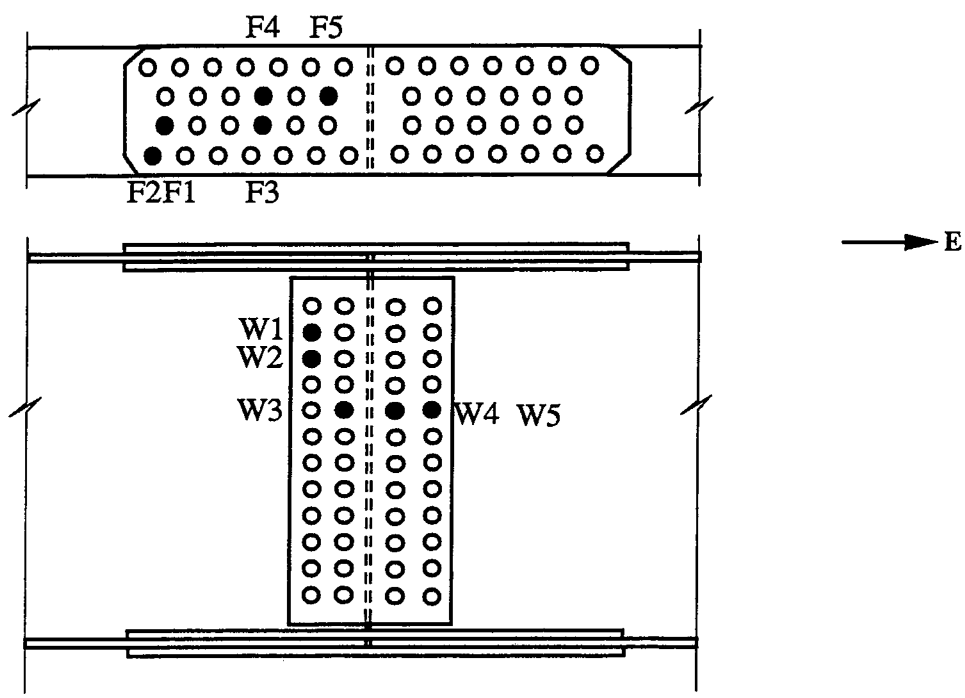

Fig. 3(d) Location of sampled bolts in connection E4 of bridge B3

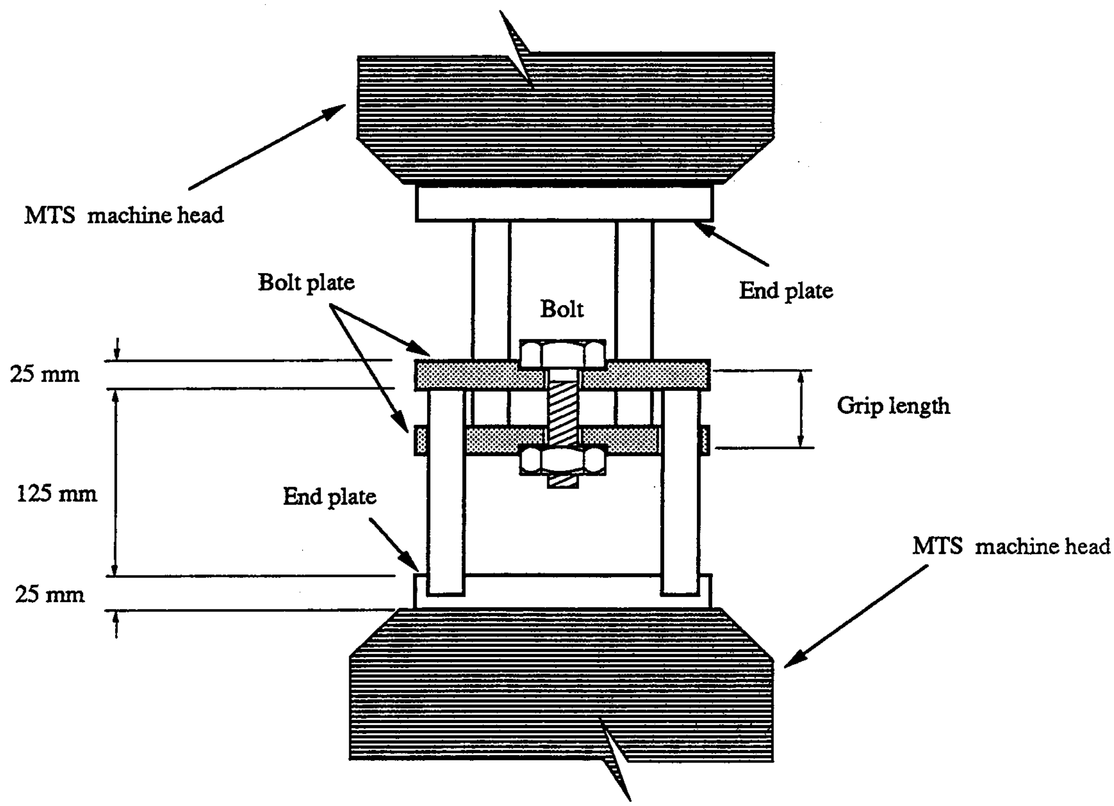

Fig. 4 Tension test set-up for bolts

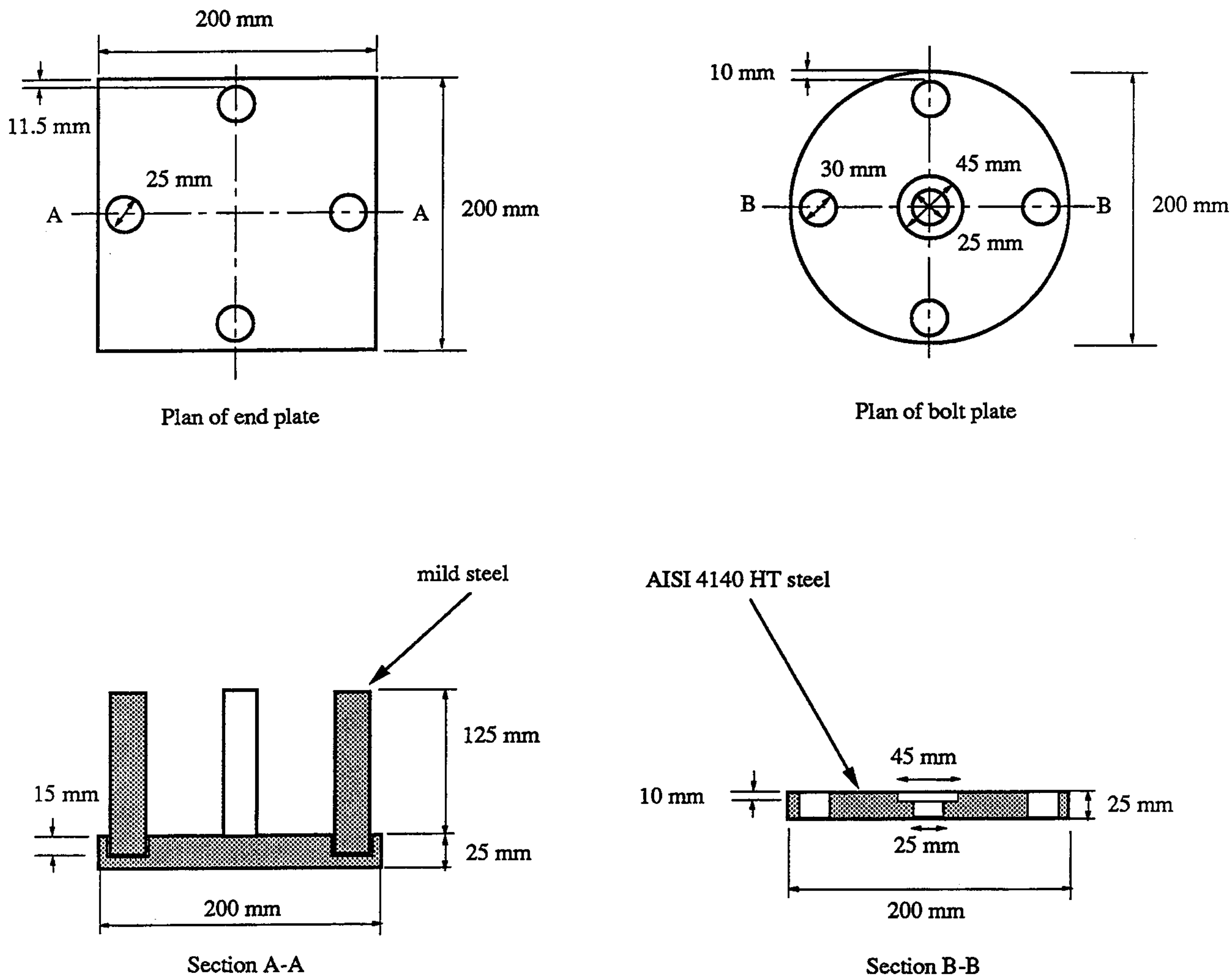

Fig. 5 Details of the grip plates

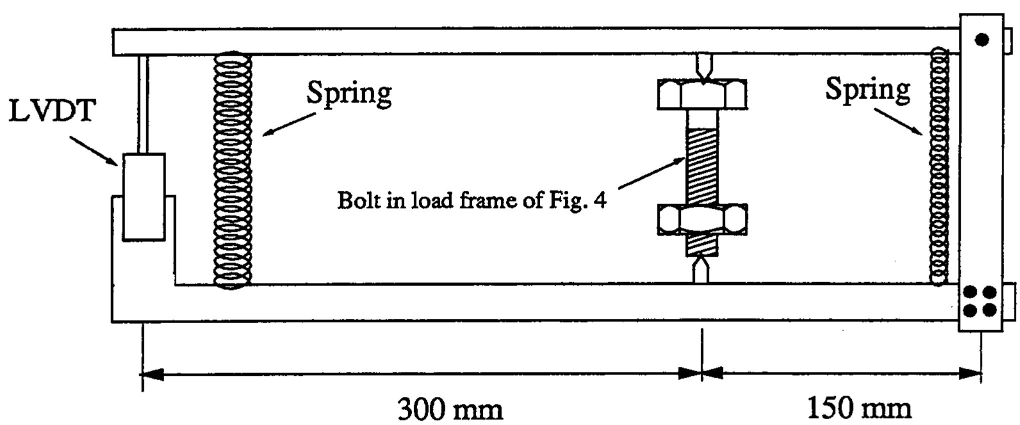

Fig. 6 Extensometer frame

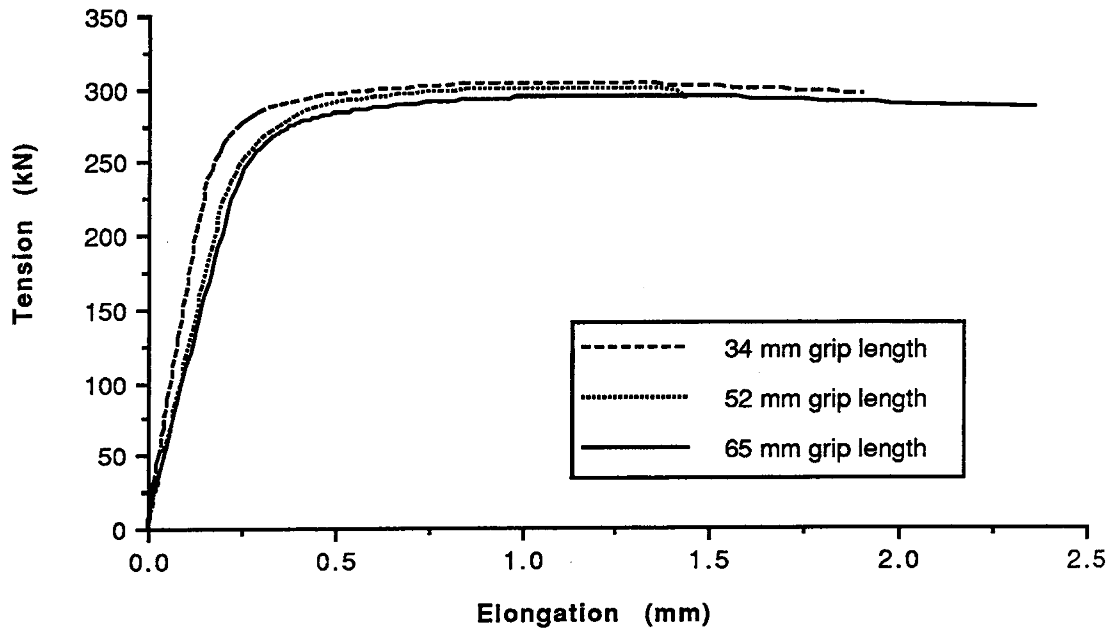

Fig. 7 Bolt tension vs. elongation response

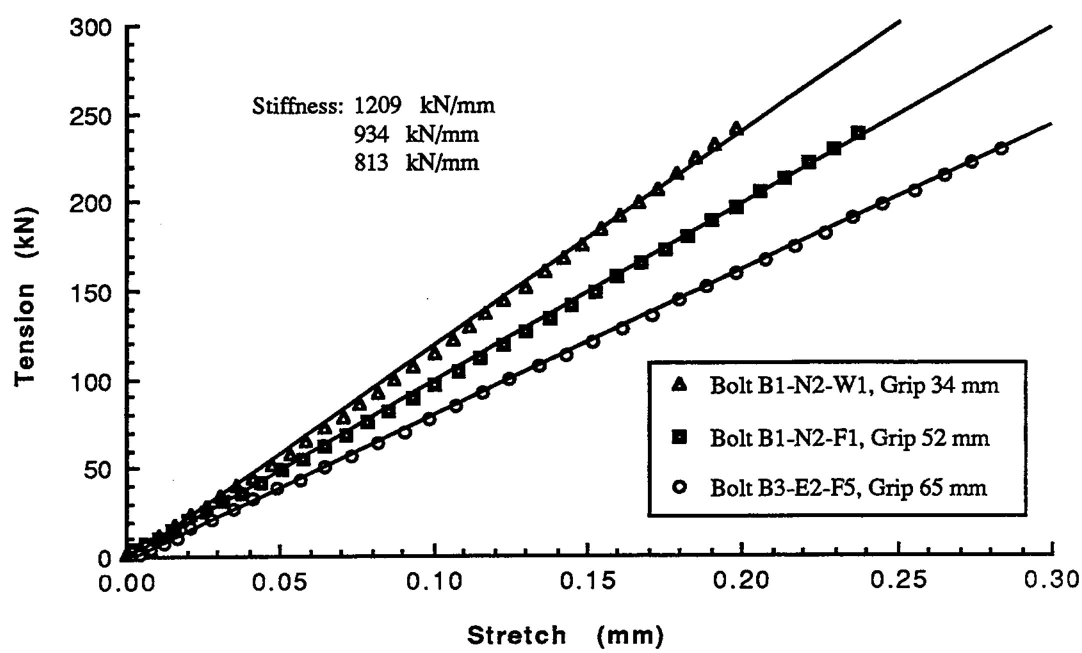

Fig. 8 Typical bolt stiffnesses

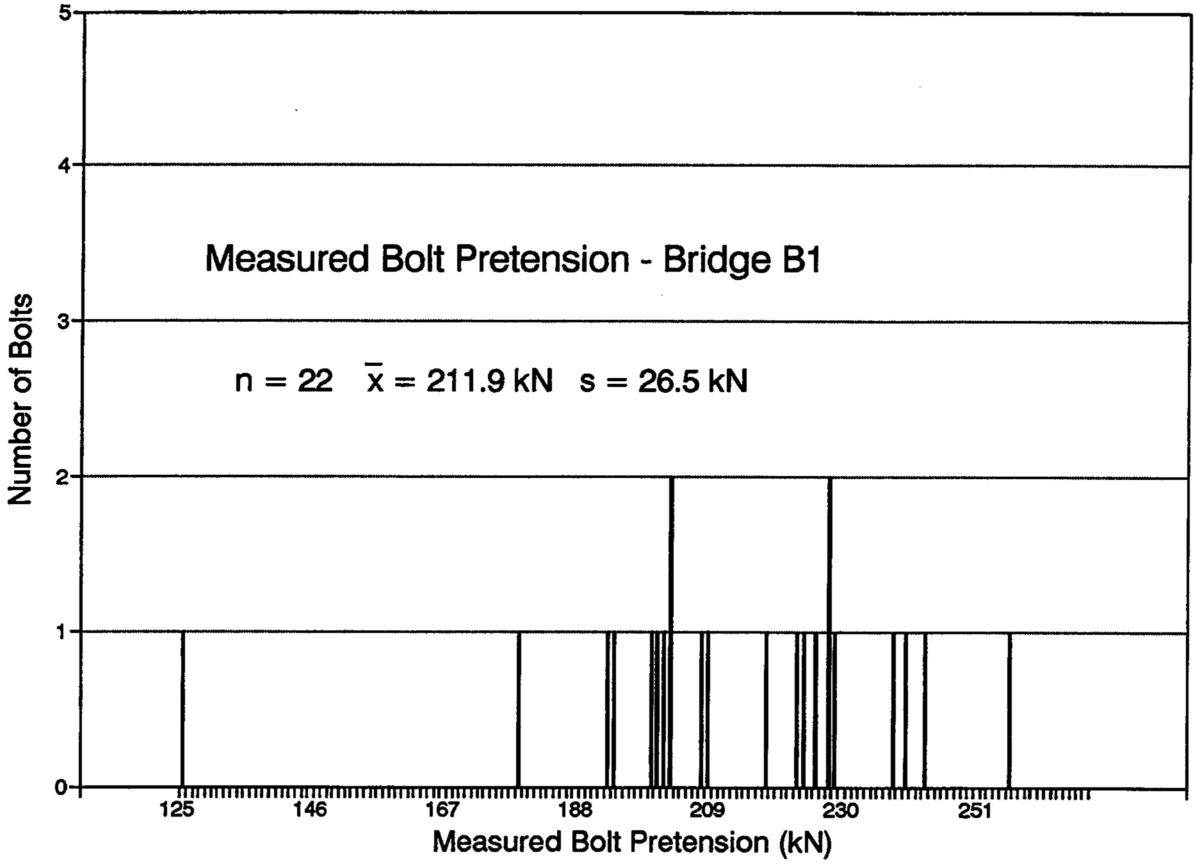

Fig. 9 Measured Bolt Pretension – Bridge B1

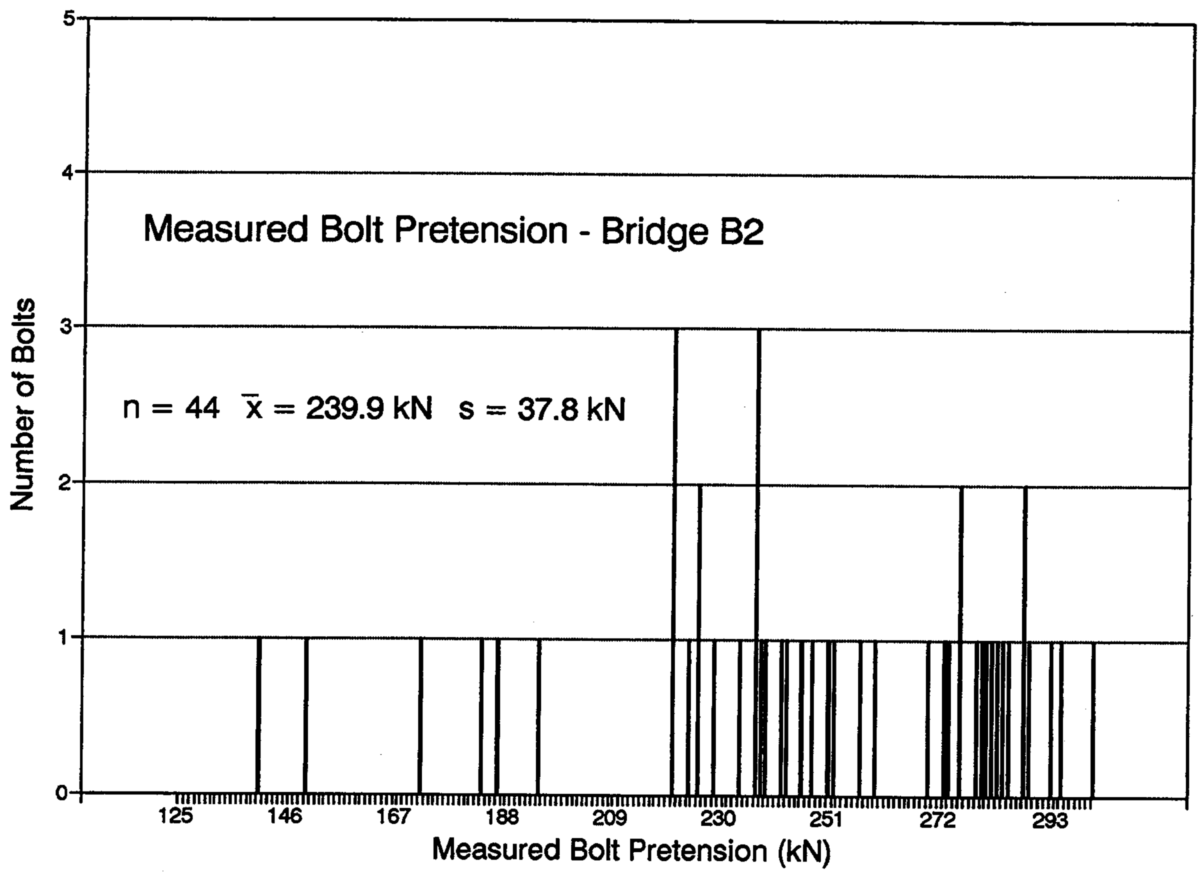

Fig. 10 Measured Bolt Pretension – Bridge B2

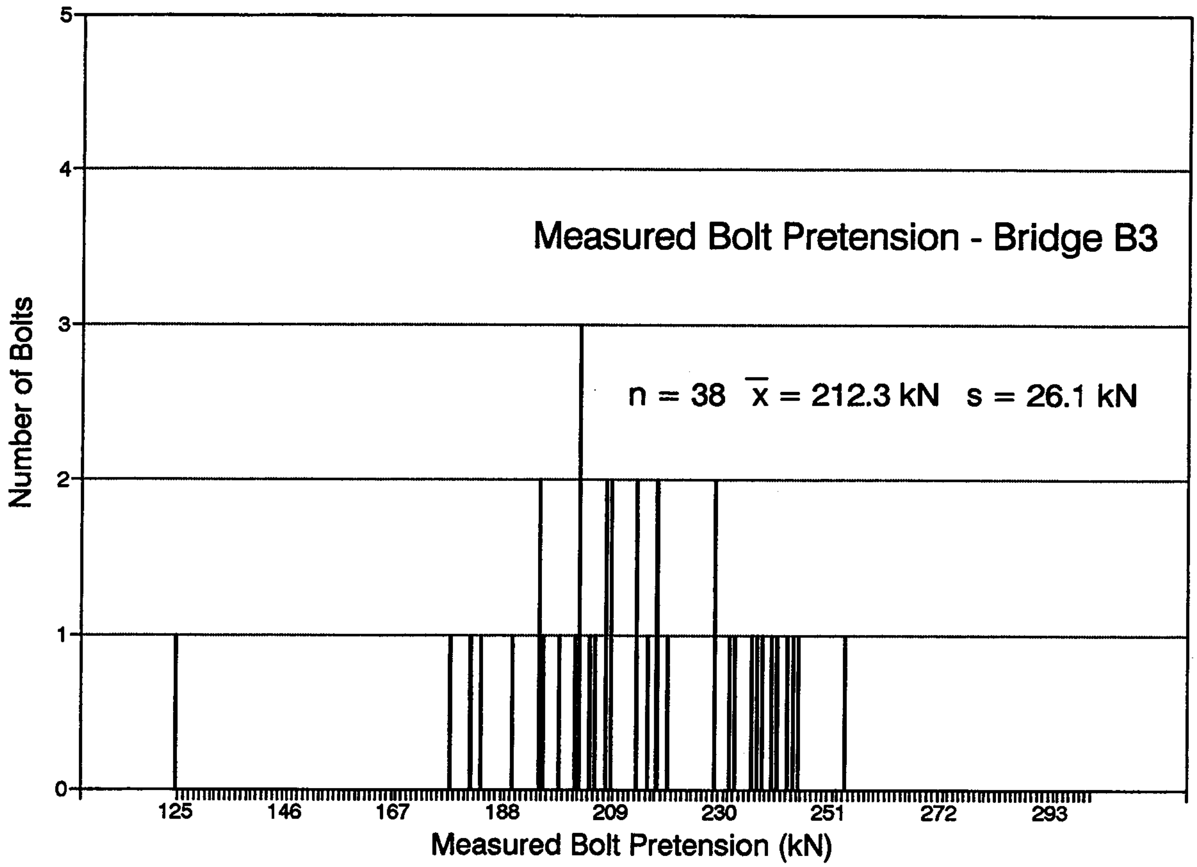

Fig. 11 Measured Bolt Pretension – Bridge B3

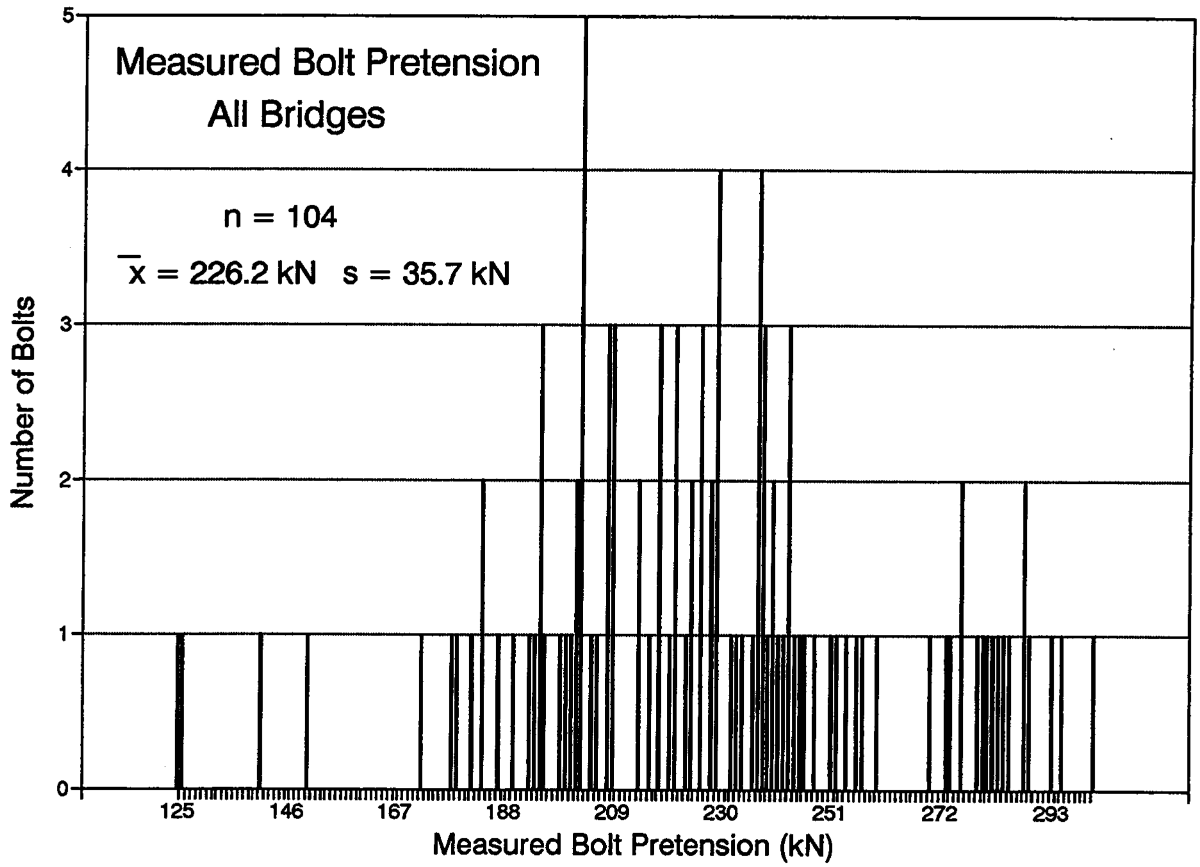

Fig. 12 Measured Bolt Pretension – All Bridges

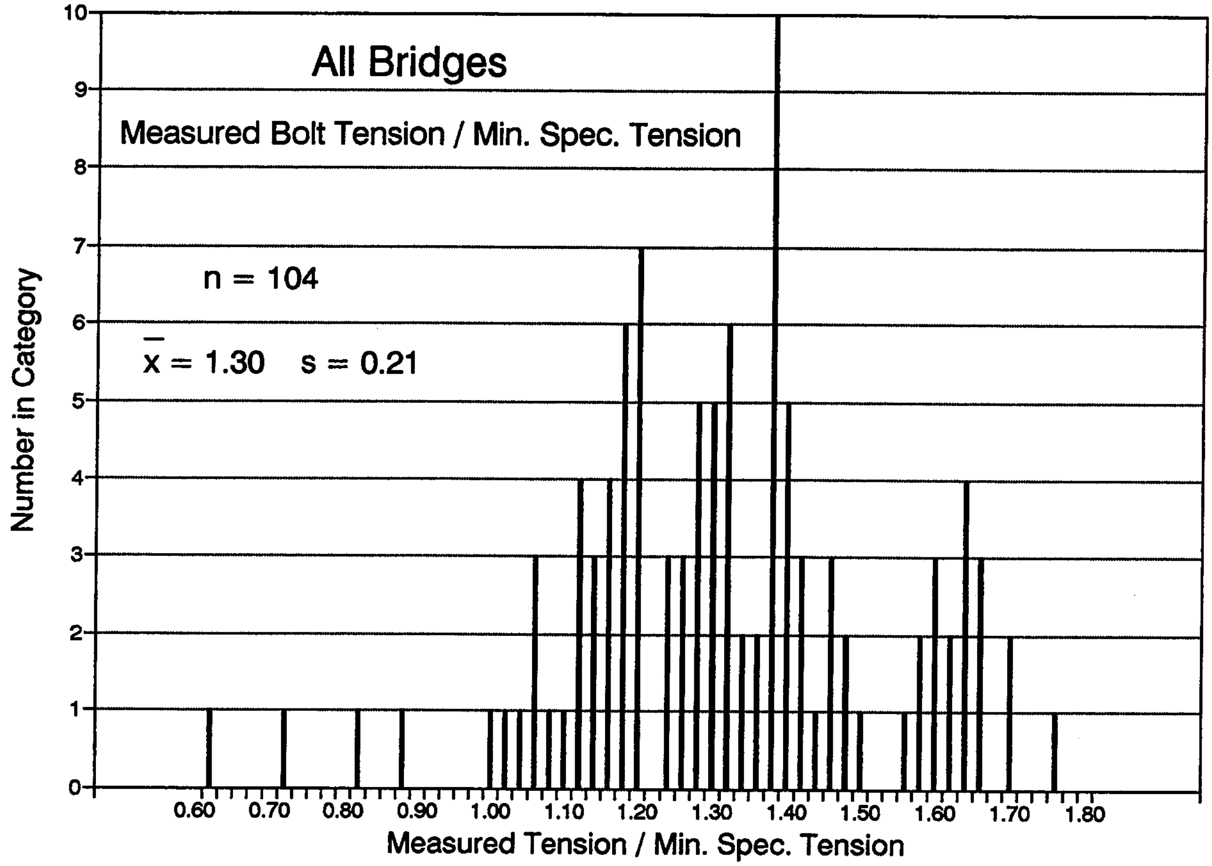

Fig. 13 Measured Bolt Pretension / Min. Spec. Tension

Appendix A: Evaluation of the Temperature Factor

The following steps show how to calibrate the bolt gage for temperature changes.

1. Place the unloaded bolt and the ultrasonic transducer in a temperature chamber.

2. Measure the initial bolt length L_1 and its temperature T_1.

3. Raise, or lower, the bolt temperature to T_2 and wait until it stabilizes.

4. Enter the temperature in keyboard of the bolt gage and take another reading of the bolt length L_2. Since it is required that the bolt gage reflect only changes in the length that are associated with tension in the bolt, adjust the temperature factor until L_2 is changed to L_1.

5. Repeat for different temperature values until the length measured by the bolt gage is constant and equal to L_1 at any temperature.

Appendix B: Evaluation of the Material Velocity

The bolt gage should read the length of an unloaded bolt exactly if the temperature factor and the bolt temperature are entered correctly into the gage. Assuming that the value of the temperature factor is correct, a bolt with machined ends can be used to evaluate the material velocity.

1. Simply measure the length with an accurate calipher and measure it again with the bolt gage.

2. The bolt temperature should be entered in the gage.

3. Adjust the material velocity so that the bolt gage shows the same length as the calipher.

4. Repeat for different lengths and take the average.

It should be noted that in the present study only the change in the bolt length is of interest. Therefore, any error in the determination of the material velocity can be compensated by the stress factor. This can be easily seen by examining Eq. 2.2.

Appendix C: Evaluation of the Stress Factor

The following steps shows how to calibrate the bolt gage for changes in the stress level.

1. Place the unloaded bolt in the set-up shown in Fig. 1(a). Attach the extensometer (shown in Fig. 2) to the bolt.

2. Apply a given load and record the extensometer reading. Unload the bolt and measure the change in length. Measuring the displacement in the unloading stage simulates the field situation and eliminates any errors related to inelastic deformation.

3. Replace the extensometer with the transducer of the bolt gage. Measure the bolt length in the unloaded condition.

4. Enter the factory-calibrated value of the stress factor and the initial measured length in the bolt gage. Switch the reading on stretch.

5. Apply the same load level as in step 2 and measure the stretch of the bolt due to this load.

6. Adjust the stress factor until the bolt gage reads the same stretch as recorded by the extensometer.

7. Repeat for different load levels, different bolts, and different grip lengths. Take the average value.

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