THE EFFECT OF OVERLOAD OF A BOLTED CONNECTION ON ITS FATIGUE LIFE: A CASE STUDY

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Abstract:

This study investigates how overload of a bolted connection affects its fatigue life by using experimental and numerical methods. The structure chosen for this case study is a sub-assembly of the steelwork used to support the skips used in a potash mine shaft under simulated slam forces from payload skips. After exposing the structure to one million cycles at the design slam load and one million cycles at double the design load, it was subjected to one cycle at five times the design load. Then, the structure was carefully investigated for signs of damage and slippage of bolts and other components on one another. In addition, the structure was partially dis-assembled to investigate the presence of local damage, especially in bolts and bolt holes. Moreover, the structure was subjected to further cyclic loading to determine how the overload affected the bolted joint fatigue strength. The results indicated that the overload of bolted connections can dramatically deteriorate its fatigue strength which is in contrast to the well-established findings that tensile overload is typically understood to retard crack growth and increase fatigue performance of individual components. Results also highlight the need for designers to consider possible unintended behaviours in bolted connections.

1 Introduction

Bolted gusset plate connections are commonly used in steel structures to transfer the load between members. Because of their primary function, they are subjected to compressive or tensile loads which could result in normal and shear forces as well as in-plane bending in the gusset plates (Nast et al. 1999). Due to the complexity of gusset plate connections, it is difficult to determine their mechanical strength. Considerable efforts have been expended by researchers to develop a deeper understanding of the behaviour of gusset plates and particularly the stress distribution in common gusset plate configurations (Whitmore 1952). This information was used to develop guidelines for designing gusset plate connections. For instance, Thornton (1984) proposed a block shear tear-out model to confirm the sufficiency of gusset plates under tension. Despite all the efforts directed toward the design of gusset plates, there is still a demand for more detailed guidelines for their design. For example, because of the under-design of several gusset plates, the bridge that carried highway I-35W over the Mississippi River in Minneapolis, Minnesota, collapsed suddenly during afternoon rush hour traffic (Crosti and Duthinh 2013).

There are several factors affecting the strength of the connection that still require more studies. One of these factors is eccentric loading of a connection. For a very long period, the design of an eccentrically loaded connection was carried out on an elastic basis, assuming that rotation of the connection occurred about the geometric centroid of the fastener group. In other words, the problem could be treated as the superposition of a concentric shear component and a shear component due to the torsional moment (Kulak et al. 1987). Since this assumption leads to a conservative design (Higgins 1971), various methods have been proposed to obtain more accurate predictions of the connection strength (Yarimci and Slutter 1963; Wang et al. 2014). However, there are still several ambiguities in regards to design of eccentrically loaded connections (Hsu et al. 2012; Oliphant et al. 2000).

Cyclic loading is another factor which makes the design of a gusset plate connection more complicated (Herter 2012). Many structural connections are subjected to cyclic loading during their service life. A bolted connection can fail at much lower stresses than its yield strength when exposed to cyclic loading (de Jesus et al. 2015; Esmaeili et al. 2015). Several research projects have been conducted to shed light on how different factors affect the fatigue life of a bolted connection (Esmaeili et al. 2014; Novoselac et al. 2014; Benhaddou et al. 2014). However, the strength and reliability of bolted connections when exposed to cyclic loading are not yet fully understood. The fatigue life of a bolted connection depends on several factors, including size of bolt, the number and position of bolts, the amount of bolt preload, occurrence of overload, the gusset plate thickness and surface roughness (Mínguez and Vogwell 2006). Uncertainty regarding the fatigue life of gusset plate connections leads most designers to take a conservative approach to their design.

The presence of a high peak load during constant amplitude cyclic loading is referred to as an overload (Sengül and Çelik 2014). Extensive study conducted on the effect of overload on the fatigue life of individual components has indicated that overload usually leads to crack closure, hence retarding the fatigue crack growth (Lopez-Crespo et al. 2015a; Lopez-Crespo et al. 2015b; MOHANTY et al. 2009; Sahu et al. 2014). However, other than a few case studies on overload of bolted joints (Buckner et al. 2014; Mohammadi and Salimi 2007), the literature suffers from a lack of comprehensive and systematic study on the influence of overloads on the fatigue strength of bolted joints. Further study is needed, particularly in the case of eccentrically loaded connections.

This article presents a case study of overloading of a bolted connection using experimental and numerical methods. The structure for this case study was a sub-assembly of the steelwork in a potash mine shaft under simulated slam forces from payload skips. During an attempt to evaluate the fatigue endurance of the bolted joint by conducting accelerated fatigue testing, the structure was subjected to one cycle of overload. The consequences and influences of the overload on the bolted connection were investigated by exploring the resulting damage and then continuing the accelerated fatigue test.

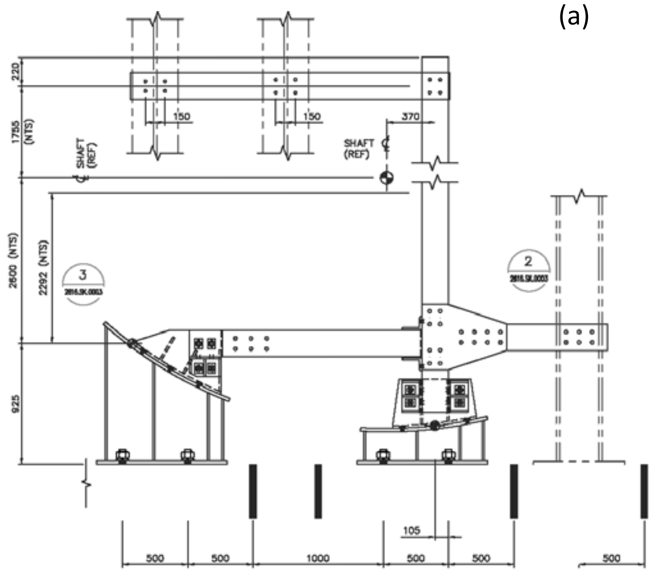
2 The test setup

As can be seen in Figure 1, the sub-assembly consisted of three lengths of HSS sections. Under service conditions, the sub-assembly would be oriented horizontally. However, in the laboratory, the first HSS section was oriented vertically and reached a 5.5 meter height (only partly shown in Figure 1); it was fastened to large steel columns by a horizontal I-beam, representing a point of support under service conditions. The second HSS section extended horizontally from the vertical section with a double gusset plate connection incorporating standard drilled holes and standard bolts. The MTS system applied a cyclic vertical load to this section of the apparatus, representing a horizontal slam load from a high-speed skip. The third section extended horizontally, in the opposite direction of the second section, and was connected to the base as shown. The base plates of the large columns and apparatus base pieces, which represented the circular wall of the mine shaft, were bolted to the strong floor of the laboratory.

Vertical load was applied at the location indicated in Figure 1 using an MTS hydraulic actuator with a capacity of 250 kN. As can be seen, the gusset plate connection was eccentrically loaded. Based on the accelerated fatigue test plan, the structure was subjected to cyclic testing in a series of stages, each stage consisting of one million cycles. The first stage of fatigue testing was conducted at the design load of 42.6 kN, the second stage at twice the design load, and the following stages at 93 kN, 103 kN and 113 kN, respectively, each stage 10% higher than the previous one.

(b)

(a)



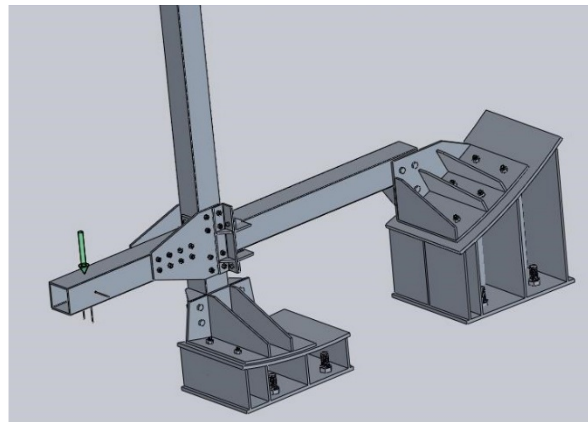


Figure 1: (a) 2D Test setup for the sub-assembly (dimensions in mm) (b) 3D test setup for the sub-assembly

The minimum load for all stages was approximately 1.5 kN. An overload of 215 kN occurred between the second and third stages as the result of a software malfunction in the control system. The movement of the structure at different places was monitored using six linear variable differential transformers (LVDTs), as shown in Figure 1. Two gauges measured vertical displacement at the load-point, two gauges measured lateral displacement on either side of the specimen at the load point, and two gauges measured the horizontal displacement at the top and mid-height of the sample.

3 Finite element analysis

A finite element (FE) analysis was conducted using ABAQUS software. Since the only non-symmetric boundary conditions were present at the supports of the structure and fairly far from the region of interest (i.e., gusset plate connection), the model was assumed to be symmetric and only half of the structure was modelled in order to reduce the computational cost. To simulate the boundary conditions, all degrees of freedom were fixed at the support locations as illustrated in Figure 2(a). The material behaviour was assumed linear elastic with an elastic modulus of $E=200\ GPa$ and Poisson's ratio of $\nu=0.3$.

Since the main goal of the FE analysis was to estimate the share of load carried by each bolt, instead of modelling contact between bolt and plates, a tie constraint was used for computational efficiency. This assumption was considered acceptable as long as the structure remained in a linear elastic state and no slippage occurred in the system. However, in locations away from the bolts where two plates or components were in the vicinity of one another and were either in contact or could come into contact during the analysis, a hard surface contact interaction was modeled.

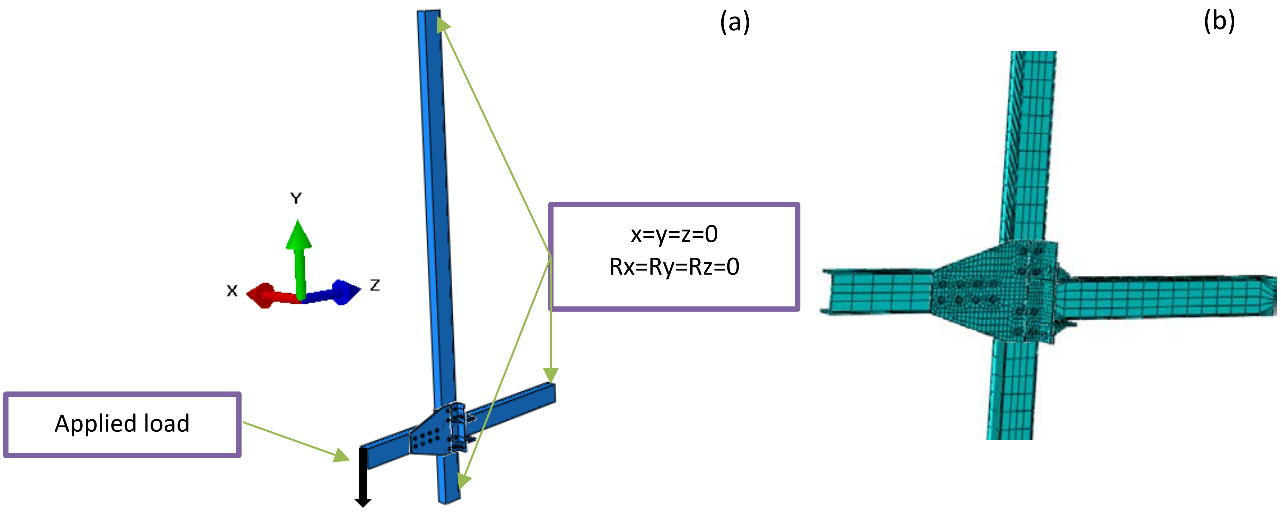


Figure 2: (a) Boundary conditions of the FE model (b) A typical FE mesh

The coefficient of friction between surfaces in contact was assumed to be 0.5, which is the value provided by the Handbook of Steel Construction for the Class B coating used in the bolted connection. A mesh sensitivity analysis was also conducted to determine the effect of mesh density on the results in order to utilize the most efficient mesh size and also to ensure the accuracy of the analysis. A typical mesh used in the modelling is depicted in Figure 2(b).

4 Results and discussion

As mentioned earlier, the displacement of the structure at various locations was monitored using LVDTs. The load-point displacement was considered to be the most important measurement since its trend over the course of the fatigue test can provide important information about the behaviour of the bolted connection. The vertical load-point displacements for the first one million cycles with a maximum load of 42.6 kN are plotted in Figure 3.

Positive values represent downward motion of the sample at the load-point. It is observed that the displacements varied from a minimum of between 0.6 and 0.7 mm at the minimum load in a cycle, to a maximum value of 2.7 to 2.8 mm when the load reached its maximum value in a cycle. The shift in the graph (values rise and range decreases) around 900,000 cycles is associated with re-tightening of the large bolts fastening the supports to the strong floor. Other than this shift, a slow increasing trend in the value of the displacement was observed while the range remained fairly constant. This increasing trend became less noticeable as the number of cycles increased. This trend suggests that the structure at the load-point was slowly moving in the load direction over the course of testing. Since this was the first stage of the fatigue test on the structure after its assembly, the trend may be explained by the release of possible residual stresses created in the structure during assembly. The absence of this increasing trend in the next stage of the test (i.e., 85 kN) confirms this hypothesis.

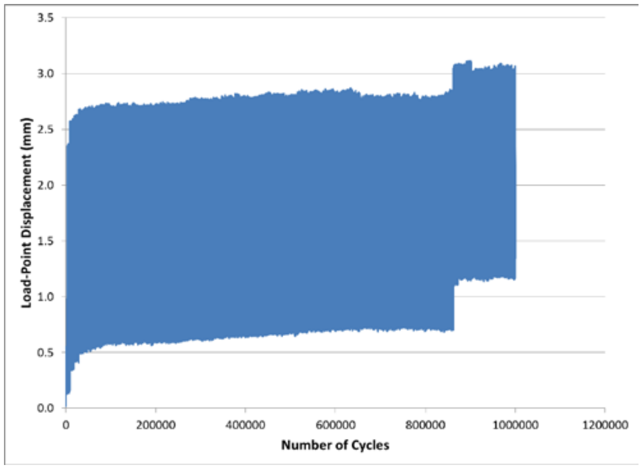


Figure 3: Vertical load-point displacements over one million cycles at the 42.6 kN load level.

After conducting the first two stages of fatigue testing (i.e., 42.6 kN and 85 kN), because of a software malfunction, the structure was subjected to an overload of 215 kN. Figure 4 illustrates the force and displacement data from the MTS machine when the incident occurred. After the overload, a permanent displacement was observed at the load point of the structure. To determine the sources of the observed permanent deflection, a detailed investigation was conducted on the structure. It was found that the permanent deflection was partially due to slippage between the gusset plates and bolts, which was determined by observing broken paint beside some bolts. Slippage also occurred on two clip angles connecting the vertical member to the horizontal members. The location and direction of slipped bolts are shown in Figure 5. Judging by the amount of slippage (broken paint), the slippage of the clip angles is responsible for the major share of permanent deflection detected at the load point.

Four of the bolts on the gusset plate which showed sign of slippage were taken off and investigated for damage to bolt threads and bolt-hole. As shown in Figure 6, there was some sign of local scratches on the bolt shank and damaged threads, confirming that the bolt came in contact with the bolt-hole in the gusset plate. However, there was no sign of cracking on any of the bolts or bolt-holes.

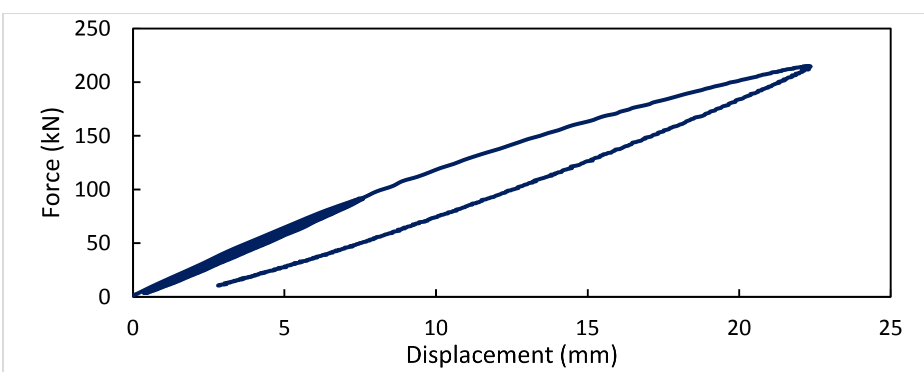


Figure 4: Force-displacement during overload incident

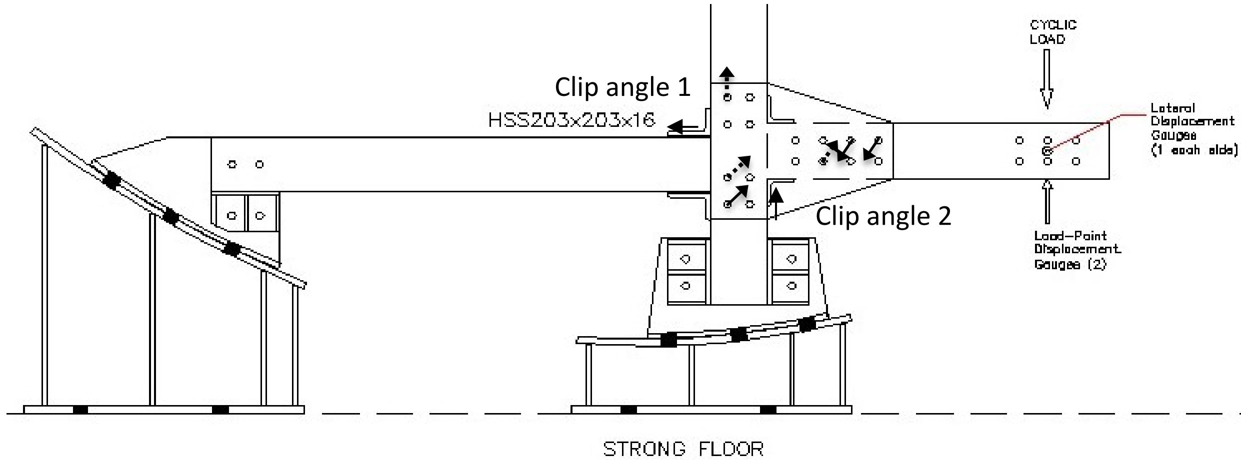


Figure 5: The sources of slippage on the structure; the doted arrows show the direction of movement of bolts on west gusset and the solid arrows show the direction of movement of bolts on the east gusset.

(a) (b)

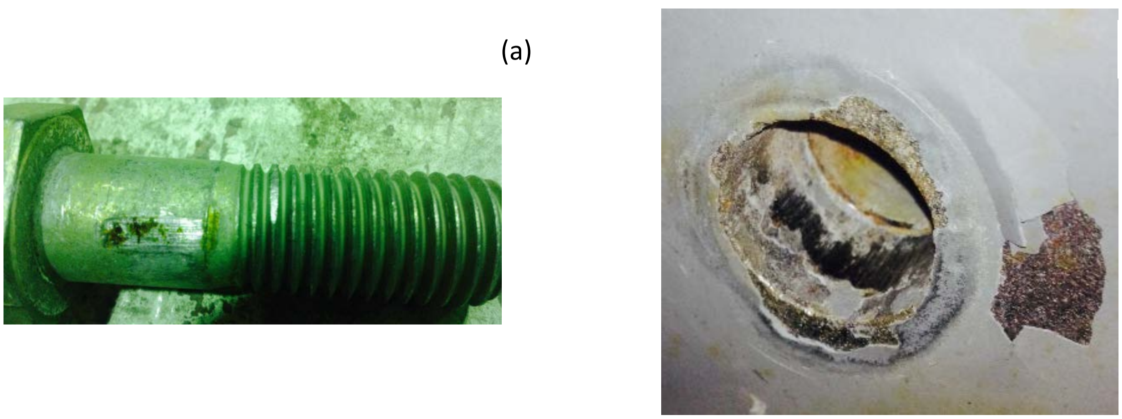
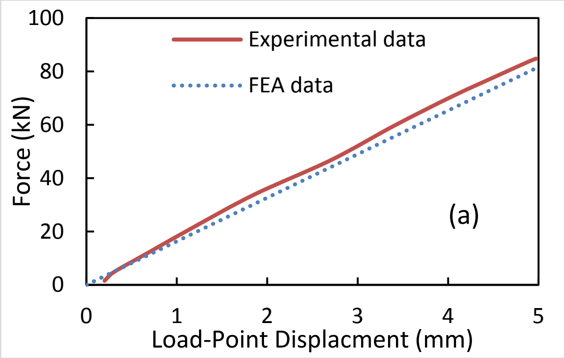


Figure 6: Typical surface damage on a (a) bolt and (b) gusset plate bolt-hole

As mentioned earlier, an FE analysis was conducted to determine the share of load carried by each bolt, thus predicting the critical load required for commencement of slippage. To verify the FE model, the force-displacement curve at the load-point obtained from the FE simulation was compared with that of experimental study (see Figure 7(a)). As can be seen, there is a good agreement between the two curves. The small difference could stem from the simplifying FE assumptions such as model symmetry and using tie constraints instead of contact interaction. The exaggerated deformation of the structure when subjected to 81.6 kN is also depicted in Figure 7(b).

For the sake of simplicity and quick grasp of the results, the percentage of the load carried by each bolt compared to the total load applied to the structure (i.e., 81.7 kN) is shown in Figure 8. According to the Handbook of Steel Construction, for bolts with class B coating, the slip resistance is 56.1 kN. The amount of load applied to the structure during the overload incident was 215 kN. Therefore, for Class B coating, bolts with more than 26% share of loading are prone to slippage during the overload.



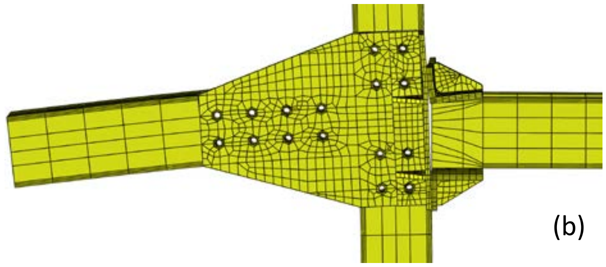


Figure 7: (a) Comparison of FE and experimental results (b) Exaggerated deformation (20 times) of the structure when subjected to 81.6 kN

Based on the FE results, bolts 1, 2, 9, and 12 were exposed to a load close to the slip critical value. Figure 5 shows that after the overload all these bolts except for bolt #2 had slipped, which agrees with FE results. However, as mentioned before, in addition to these bolts (predicted by FE), some other bolts also showed signs of slippage, including bolts 3, 6, and 8. This can be attributed to the fact that slippage of the bolts not predicted by FE analysis was secondary to the initial slippage of those that were predicted. Moreover, further dis-assembly of the structure revealed that the coating of some surfaces in contact was partially worn off, which could have been due to the previous cyclic loadings or the overload itself. Lack of proper coating and/or pre-tension could explain the premature slippage of the mentioned bolts.

After replacing the investigated bolts with new ones, the accelerated fatigue test was continued at 93 kN. During this stage of the test, bolt #19 failed. The load-point displacement for this test is depicted in Figure 9(a). As can be seen, an increase in the range of displacement started at about 900,000 cycles and a shift occurred at 970,000 cycles. The first change (i.e., the gradual increase in the range of displacement) could be associated with loosening of some bolts in the structure (bolts 20 and 24 and their symmetric neighbours, which were not included in the FE analysis). The second change (i.e. the shift) is likely associated with breakage of the bolt. It is noteworthy that there is no sign of change in the values or range of displacement prior to 900,000 cycles and it looks much more uniform than was observed during the two previous stages (i.e., 42.6 kN and 85 kN). This suggests that the bolted joint had settled in as a result of the overload and possibly during the previous stages of cyclic loading.

The fracture surface of the bolt is shown in Figure 9(b). Crack initiation occurred between the 9 and 11 o'clock positions; some ratchet marks are also visible in this region. The smooth fracture roughness from ratcheting marks toward the central region of the bolt indicates steady fatigue crack growth and the fracture region with higher roughness close to outer surface of bolt from 12 to 6 o'clock position is a sign of static fast fracture. After replacing the broken and loosened bolts, the next stage of accelerated fatigue test continued at 103 kN. At this stage, no incident occurred, except for a small increase in the range of the load-point displacement. The next stage of testing was conducted at 113 kN and stopped at 500,000 cycles because of failure of two bolts at the support of the structure. Since these two bolts were not in the region of interest for this study, the details of their fracture are not discussed here. At the time of writing, additional testing is planned.

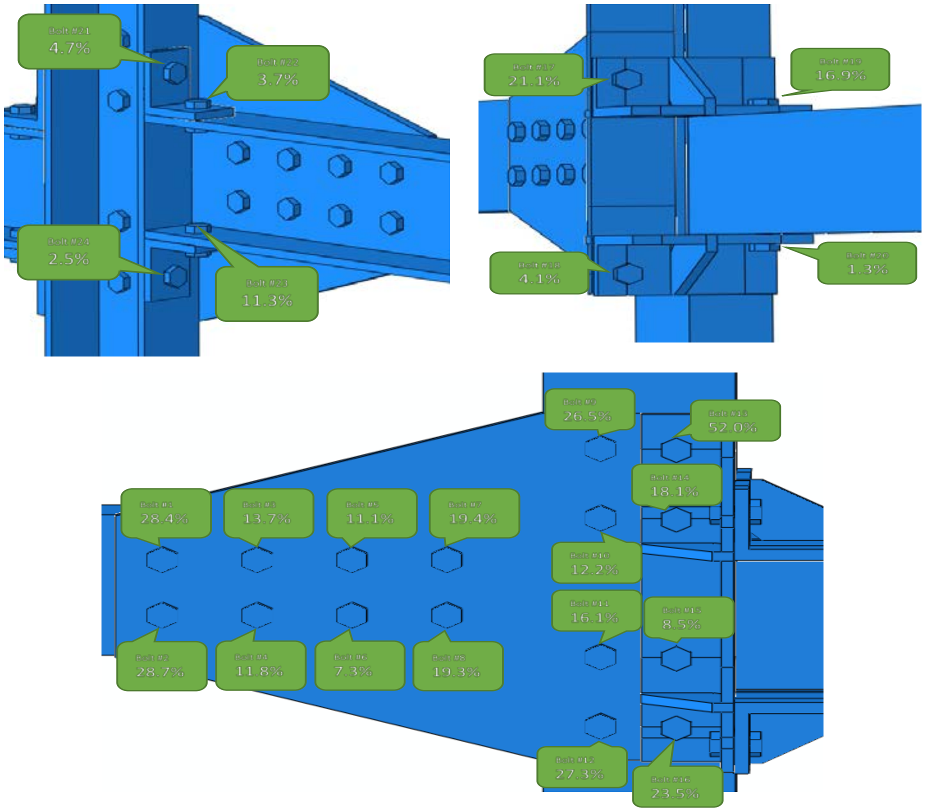


Figure 8: Percentage of the applied load carried by each bolt, as determined by FEA

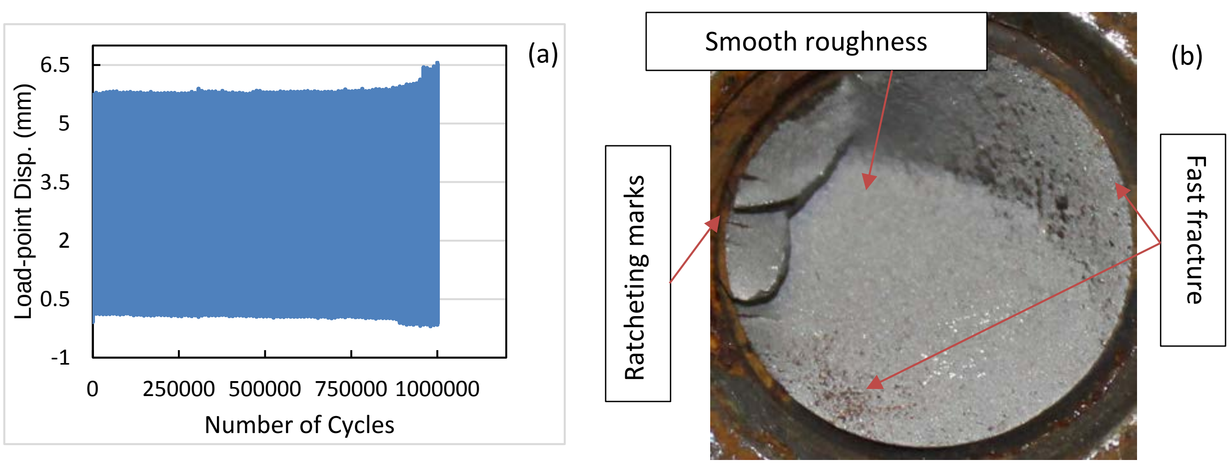


Figure 9: (a) Vertical load-point displacements over one million cycles at the 93 kN load level; (b) Fracture surface of bolt #19

5 Conclusions

During a case study involving accelerated fatigue testing of an eccentrically loaded bolted connection, it was accidentally subjected to one cycle of overload. Investigation of the structure after the overload revealed several signs of damage, especially on the bolts. Moreover, some of the bolts slipped and came in contact with the bolt-hole. The coating on the clip angles was also partially worn off, which can dramatically affect the fatigue life of slip critical joints. Failure of one of the clip angle bolts during the next stage of testing after the overload can be attributed to damage caused by the overload. This was confirmed by replacing the broken bolt with a new one and running the test at higher loads, during which no failure was observed.

By using FE analysis, the share of the transfer load carried by each bolt was estimated. In addition to the bolts predicted by FE analysis to slip, three other bolts also slipped during the overload incident, which was attributed to the following factors: (i) secondary slippage after the initial slippage of the bolts that were predicted to slip; (ii) the symmetry assumption for boundary conditions or other simplifying assumptions; and (iii) accumulated damage from previous cyclic loads or construction defects. However, comparison of the results obtained through FE analysis and the experimental study showed that the FE results were generally accurate while the structure remained in the elastic state and there was no slippage.

It is also instructive to note that, while the fatigue behaviour of the gusset plate connection was the primary design concern and the focus of the experimental study, bolts at several other locations failed first. This highlights the need for designers to give careful attention to the components of connections that may not be considered most critical, particularly identifying the possibility of unintended behaviours. For the current study, a bolted clip angle connection that was intended to function as a simple support for a member not directly along the primary load path for the cyclic loads was subjected to a significant bending component, leading to the eventual fatigue failure of clip angle bolts. It should be noted, though, that this occurred at loads that exceeded the design load by a large margin.

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