Behaviour of Welded Steel Wire Mesh as Surface Support

by

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Abstract

An effective ground support is a requirement for the design of underground excavations in rock. In order to design a ground support system, it is necessary to understand the mechanical behaviour of each ground support system component. The focus of this thesis is the behaviour of welded steel wire mesh as surface support. A critical review of the results obtained from various testing rig configurations has been undertaken. This provided the basis for a series of numerical experiments to reproduce the mechanical behaviour of welded steel wire mesh as observed in the laboratory tests. The mechanical behaviour of welded wire mesh was captured using the 3D-Distinct Element Method (3D DEM). Finally, a series of parametric investigation demonstrated the influence of testing rig configuration on the performance of the mesh. This has significant implications in the design of testing rigs to quantify the performance of mesh.
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Chapter 1

1 Introduction

1.1 Problem Definition

An effective ground support is a requirement for the design of underground excavation in rock. Use of an inadequate or inappropriate ground support system can lead to falls of ground and have a significant impact on the safety of mining personnel and equipment. For example, in the United States, the majority of ground-fall injuries at underground mines were caused by smaller rocks falling from the immediate roof (Batchler et al. 2018). Understanding how to provide an effective ground support system first requires understanding of the mechanical behaviour and failure mechanisms of each ground support component (reinforcement and surface support) involved. However, research has focused primarily on reinforcement component and less on the surface support component’s role in providing an appropriate ground support system. The surface support component is used to confine the smaller broken rock masses between reinforcement components and distribute its load between the reinforcement components (Hadjigeorgiou & Potvin 2011b). Therefore, the surface support component provides an important role in the overall ground support system. Understanding the behaviour of the surface support component is a prerequisite to the design of a ground support system.

The most commonly used surface support for underground hard-rock mines in North America and Australia is welded steel wire mesh. Despite the prevalence, there seems to be a lack of guideline for mesh design. In practice, mesh design primarily relies on empirical design charts associated with a rock mass classification system, such as the Q-system (Potvin & Hadjigeorgiou 2016), and experience. Since empirical approaches do not explicitly account for the mesh’s role in its design of ground support and its interactions with different ground support components, they are intended to be used as a first pass design tool, during the early stages of a project. The visual inspections of ground support in underground mines can provide some understanding of the ground support performance as well. However, it is impossible to measure the performance.

The quantifiable measurements of mesh performance are load and displacement capacities. The load and displacement capacities of mesh are quantified using a series of laboratory tests. While the tests may provide quantifiable design capacities and some understanding of mechanical behaviour of mesh under static and dynamic loading conditions, the results obtained from each laboratory test is limited to particular loading
mechanisms and boundary conditions. A lack of standardized testing rigs and procedures makes the comparison of laboratory test results difficult.

The use of numerical modelling can complement the results of laboratory tests. Numerical modelling can be used to further investigate parameters that influence mesh performance during the tests. There are some considerable challenges associated with successfully representing the mechanical behaviour of mesh in numerical models. These challenges are addressed in this thesis through a series of numerical modelling experiments.

1.2 Significance

The design of a ground support system requires an understanding of the behaviour of each component. An improved understanding of the performance of welded steel wire mesh as a surface support in underground mines will benefit the daily mine operation. Quantifying the performance of mesh can result in improved design, which helps to maintain the safety of personnel and equipment. An improved design can also minimize over-designing which often impacts the operation cost, as it results in the use of extra ground support components, extra labour, and production delays.

1.3 Objectives

This thesis aims to contribute to our understanding of the behaviour of mesh as surface support in underground mines. The thesis has three specific objectives:

- to review the performance of mesh under various laboratory testing rig configurations, and identify the limitations associated with each arrangement
- to develop numerical models to capture the mechanical behaviour of mesh in a laboratory setup
- to demonstrate the impact of testing rig configuration on the mesh performance using the developed numerical model

This has significant implications in the interpretation of results obtained from the various testing rigs used worldwide.

1.4 Methodology

The methodology followed in this thesis is summarized below:

1. **Data collection**: available reports on static laboratory testing rigs for mesh were collected, and their testing parameters and recorded test results were compiled into a database.
2. **Construction of numerical model:** the numerical modelling of mesh using beam structural elements in 3D DEM was examined. The numerical model was calibrated to match the mesh behaviour under a laboratory test configuration.

3. **Analysis:** the developed model was utilized to investigate whether it can capture the mechanical behaviour of mesh as shown under the particular laboratory test configuration. Numerical parametric investigations were conducted using the developed model.

4. **Interpretation:** the results of the numerical experiments were used to examine the impact of different testing parameters on mesh performance.

### 1.5 Thesis Structure

The thesis consists of the seven chapters as outlined below:

**Chapter 1: Introduction** – the problem is defined, and the objectives of the thesis are outlined. The methodology and structure of the thesis are outlined.

**Chapter 2: Welded Wire Mesh Surface Support** – available types of surface support component are discussed, and various factors impacting the performance of surface support component are examined. Empirical and analytical approaches as design guidelines for welded wire mesh in underground mine excavations are reviewed.

**Chapter 3: Performance of Welded Wire Mesh In-Situ** – field observations and internal documentation of ground support system performance under static and dynamic loading conditions at a Canadian hard-rock mine are outlined. Observed interactions between the mesh, the loading condition, and the boundary condition are discussed.

**Chapter 4: Testing of Welded Steel Wire Mesh** – comparative study of different configurations of laboratory testing rigs and their observed results are discussed, and the influence of testing parameters on the mesh performance is reviewed.

**Chapter 5: Numerical Modelling of a Laboratory Test of Welded Steel Wire Mesh** – the calibration of the beam structural elements in the 3D DEM to simulate the mesh surface support is discussed. The captured failure mechanism, load transfer, and the load displacement response of welded wire mesh under laboratory testing conditions are discussed.

**Chapter 6: Numerical Parametric Investigations** – the influence of different testing parameters on the performance of mesh using the developed numerical model is examined.
Chapter 7: Conclusions – contributions of the thesis are summarized, and the limitations associated with the proposed approach of numerical modelling of mesh are discussed. The recommendations to improve the current state of the developed numerical model are presented.
2 Welded Wire Mesh Surface Support

2.1 Introduction

This chapter reviews surface support as an essential component of a ground support system in underground mines. Different types of surface support components used in underground mines are described. Important parameters that can potentially influence the performance of each surface support component are reviewed. Analytical and empirical approaches as design guidelines for welded wire mesh in underground mines are also reviewed.

2.2 Surface Support

In the province of Ontario, Canada, a ground support system is required as an essential part of a mine design package under the ‘Regulation 854 Mines and Mining Plants of Occupational Health and Safety Act’ (Government of Ontario 1990). The ground support system is a combination of reinforcement components (e.g. rock bolts and cable bolts) and surface support components used to control the rock. It is also designed to ensure the stability of the surrounding rock mass in excavation. Hadjigeorgiou & Potvin (2011b) defined surface support as “a technique in which elements such as shotcrete or steel mesh … are applied to excavation surfaces externally to the rock mass.” Figure 2.1 shows the conceptual schematic diagram of the mesh surface support in an underground excavation under a static loading condition.

![Conceptual schematic diagram showing the retainment of the bulged broken rock mass by the mesh surface support in an underground excavation under static loading.](image)

Figure 2.1 Conceptual schematic diagram showing the retainment of the bulged broken rock mass by the mesh surface support in an underground excavation under static loading.
In general, there are two basic types of loading conditions that can be induced by ground movement: static and dynamic. Figure 2.2 shows the conceptual schematic diagram illustrating how mesh is statically loaded under the weight of broken rock mass. An excavation in the rock causes the ground stress to be concentrated at the periphery of the excavation. Once the concentrated stress exceeds the strength of the surrounding rock, the ground breaks. Gravity results in broken rock to fall down or slide along adjacent blocks. The main objective of surface support under a static loading condition is to contain the dead-weight of these broken rocks between the rock bolts loaded under gravity and induced ground stresses (Hadjigeorgiou & Potvin 2011b). Under a dynamic loading condition where a sudden rockburst can occur, the demand of kinetic energy should additionally be considered.

![Conceptual schematic diagram showing the close-up view of the retainment of the bulged broken rock mass by the mesh surface support.](Image)

**Figure 2.2** Conceptual schematic diagram showing the close-up view of the retainment of the bulged broken rock mass by the mesh surface support.

In underground mines, a combination of different surface support components is used along with reinforcement components (e.g. rock bolts and cable bolts) to retain the broken rock mass under the static and dynamic loading conditions. As an example, Figure 2.3 shows various types of surface support components used along with reinforcement components in underground mines.

There are several factors that determine the selection of good combination of ground support components, such as the ground conditions. For example, in the location where the ground is experiencing dynamic loading condition, the additions of mesh strap and mesh plate are used, as shown in Figure 2.3b. The compatibility of different ground support components should also be taken into consideration when choosing ground support components. The compatibility of ground support components and the detailed description of each surface support components are discussed in the next section.
2.3 Surface Support Components

In this section, the various types of surface support components used in underground mines, such as reinforcement plate, steel wire mesh plate, steel wire mesh strap, steel wire mesh, and shotcrete, are described. A general review of the parameters that can influence the behaviour of each surface support component is also discussed.

2.3.1 Reinforcement Plates

There are different types of reinforcement plates that are currently available commercially, as shown in Figure 2.4, depending on the type of reinforcement component used (e.g. a friction plate for a friction bolt).

Figure 2.3 Examples of various types of surface support components being used in a Canadian underground hard-rock mine: (a) reinforcement plate, wire mesh plate, and wire mesh, and (b) wire mesh strap.

Figure 2.4 Different types of reinforcement plates, after Hadjiigeorgiou & Potvin (2011b).
The reinforcement plate works with the reinforcement components to provide a clamping force to the steel wire mesh on the wall of a drift, as previously shown in Figure 2.3. It is often the case that each reinforcement component supplier provides a specific configuration of reinforcement plate for their specific type of reinforcement component. As such, it is possible for an operating mine to have a number of different types of reinforcement plates used, which in turn can result in installation errors.

Another parameter that can impact the performance of the reinforcement plate is the type of nut and its seat. The nuts are used to actively clamp the reinforcement plate to the surface of a drift. A nut and seat combination is commonly used with a threaded-rebar and is dependent on the type of reinforcement component used. Some reinforcement components do not require the use of a nut, as they instead make use of other devices. For example, the installation of a friction bolt (i.e. Split Set Stabilizer) and expandable bolt (i.e. Swellex) have a welded steel ring flange that acts as a “nut”, as shown in Figure 2.5. Consequently, an additional external nut is not required to hold the reinforcement plate against the wall of a drift.

![Image of a welded steel ring flange of an installed friction bolt.](image)

**Figure 2.5** An example of a welded steel ring flange of an installed friction bolt.

The push-on plate has built-in steel “teeth” around the reinforcement component hole that latch on to the threaded portion of a rebar without the use of an external nut, as shown in Figure 2.6. This allows for the push-on plate to be installed relatively quickly. These built-in holding mechanisms, however, provide a weaker holding capacity when compared to an external nut and seat combination.
Some of the other parameters of a reinforcement plate that can influence the plate performance include:

- Grade of steel
- Plate thickness
- Plate size
- Plate shape
- Reinforcement component hole diameter

The relationship between the performance and the parameters should be carefully reviewed. For instance, a thicker reinforcement plate is expected to behave stiffer and have a higher load capacity than thinner reinforcement plates (Gray 1998). If the interaction between the reinforcement plate, with sharp plate edges, and the wire strands of mesh is stiff, the mesh wires can potentially be cut off under large static and dynamic loading conditions (Gray 1998; Simser 2007). In terms of the plate shape, the domed plate with a spherical nut and seat provide good contact for irregular rock surfaces (Hadjigeorgiou & Potvin 2011b), as it allows for wider installation angles of the reinforcement component in comparison to a flat plate with a flat nut and seat combination. Kang et al. (2015) conducted a series of compression tests on various domed plate configurations. The test results showed an appropriate combination of shape and size of the reinforcement plate is essential in achieving high load-bearing capacity and deformation of the plate. A plate with larger dimensions can help to distribute the applied load onto the wire mesh better as the load can be transferred through more wires. Additionally, the diameter of the reinforcement component hole should be adequately sized to fit the specific diameter of reinforcement component and its nut. The performance of the
reinforcement plate is dependent on the combination of the previously stated parameters. As such, a poor combination under a particular ground condition can lead to unexpected plate failure caused by either rupture of the hole or premature collapsing of the plate. Different interfaces of various surface support components and how they behave under different ground conditions are reviewed in Chapter 3.

2.3.2 Mesh Strap and Mesh Plate

The mesh strap, as shown previously in Figure 2.3b, is commonly a band of #0-gauge (8 mm) wire mesh used to strengthen the overlaps existing between two steel wire meshes (Hadjigeorgiou & Potvin 2011b). The mesh strap is often used to help clamp the mesh overlap and to control the bulging of the mesh. The mesh plate is often a square #0-gauge wire mesh used to strengthen the interface between the reinforcement plate and the steel mesh wires clamped underneath the reinforcement plate, as shown previously in Figure 2.3a. The mesh plate can help to control the breakage of steel wires loaded underneath the reinforcement plate by distributing the applied load over a wider area.

2.3.3 Shotcrete

Shotcrete as a surface support in underground mines comes in both wet and dry forms of mixed cement, sand, and fine aggregate concretes (Hadjigeorgiou & Potvin 2011b). It is pneumatically applied onto the wall of a drift either manually or through mechanical means. Often, shotcrete, as shown in Figure 2.7, is used as a “glue” in underground mines to hold heavily fractured rock masses and to control deterioration of the excavation (Hadjigeorgiou & Potvin 2011b).

![Figure 2.7](image)

**Figure 2.7** Examples of the application of shotcrete in underground mines: (a) shotcrete applied on the back of a drift, and (b) shotcrete arches.

This practice is often conducted in the areas where highly concentrated stress is expected, as there is a higher risk of heavy deterioration of excavation. Shotcrete is also applied to areas where there are important
infrastructures (e.g. refuge stations, sub-station, loading bay, etc.) This thesis does not include a detailed review of the performance of shotcrete.

2.3.4 Wire Mesh

Welded wire mesh and chain link mesh are shown in Figure 2.8a and Figure 2.8b, respectively. These are the two main types of mesh that are the most popular choices of surface support for hard-rock mines in North America.

![Figure 2.8 Two main types of wire mesh used as surface support in the underground mines in North America: (a) welded wire mesh, and (b) chain link mesh.](image)

The welded wire mesh is a sheet of steel wires where longitudinal wires and cross-cut wires are welded at the intersecting points. Some of the main mesh properties that can influence the performance of the welded wire mesh include (Thompson 2004):

- Grade of steel
- Mesh aperture (i.e. wire spacing)
- Wire gauge (i.e. wire thickness)
- Weld quality
- Tensile and shear strength of wire

In North America, the typical mesh aperture is 100 x 100 mm, and the typical wire gauges are #4, #6 and #9 (5.8, 4.9, and 3.7 mm diameter, respectively) (Hadjigeorgiou & Potvin 2011b). The chain link mesh is a sheet of steel wires that are intertwined where the wires intersect. The main advantage of chain link mesh is that it can exhibit a larger displacement capacity than the welded wire mesh (Hadjigeorgiou & Potvin 2011b). This characteristic in the behaviour of chain link mesh makes it more appropriate in ground conditions where a large deformation occurs. The chain link mesh can exhibit a higher load capacity than
the welded steel wire mesh, which is often associated with the use of high-tensile steel wire for its mesh construction (Morton et al. 2007).

The mesh aperture and the wire gauge can both influence the mesh behaviour under an applied load. For example, a stiffer response is expected when the welded wire mesh has a smaller mesh aperture (i.e. smaller wire spacing) and a lower wire gauge (i.e. larger wire diameter). The selection of the mesh parameters can also impose some operating constraints. For instance, a thicker wire diameter will increase the total weight of the mesh screen. This can present an issue in the case of a narrow-vein cut-and-fill operation where wire mesh is installed manually with hand-held tools as it will increase the difficulty of the installation. Additionally, a larger mesh aperture can unintentionally allow the smaller broken rocks to sieve through. Larger size apertures, however, can allow for a more effective installation of shotcrete over the mesh. Therefore, the selection of the mesh parameters can be dependent on the required trade-offs between mesh performance and other operational constraints.

2.4 Design of Welded Wire Mesh

In this section, some of the available analytical and empirical approaches that provide design guidelines for mesh are reviewed.

2.4.1 Analytical Approaches

2.4.1.1 Coates

Coates (1981) introduced an analytical method that calculates the maximum probable static pressure applied by the broken rock and the maximum axial tensile stress on mesh (Figure 2.9). The maximum probable vertical pressure \( P_v \) applied by the broken rock on mesh can be calculated with the following simplified equation (Coates 1981):

\[
P_v \leq 0.727 \gamma s
\]

where \((s)\) represents the bolting spacing \([m]\), \((\gamma)\) represents the unit weight of the broken rock \([N/m^3]\), and \((P_v)\) represents the maximum vertical pressure applied by the dead-weight of broken rock \([Pa]\).

The maximum probable tensile force \( T \) that could occur in mesh is calculated by Coates (1981):

\[
T = \frac{P_v s^2}{8h}
\]

where \((s)\) represents the bolting spacing \([m]\), \((T)\) represents the maximum probable tensile force \([N/m]\), and \((h)\) represents the probable sag of the membrane \([m]\).
Equations (1) and (2) were derived using the arching theory. The following assumptions are made, after Coates (1981):

- The width of the arch is equal to the bolting spacing, “S”.
- The failure depth, “z”, is less than the bolting spacing. This was assumed to maintain bolt tension.
- The sag of the mesh occurs in a vertical direction, and the applied vertical load, “P_v”, is uniform over the mesh.
- The internal frictional angle of rock is greater than 45 degrees, which makes the ratio of the horizontal to vertical stress less than 0.33 (based on arching theory).
- The two-dimensional section is an appropriate representation of the three-dimensional mesh.
- The failure mechanism is assumed to be tensile failure of the steel wires.

The equations proposed by Coates (1981) are applied to a simple example. In the example, a bolting spacing, “S”, of 1 m by 1 m and a rock density of 2700 kg/m³ are assumed. Table 2.1 shows the calculated maximum probable vertical pressure (P_v) as induced by the broken ground using Coates (1981)’s equation (1). It also shows the comparison of the calculated weights of the six different volume estimations for the broken ground acting on mesh, as shown in Figure 2.10 and Figure 2.11.

Figure 2.10 Different broken ground volume estimations: (a) pyramidal, (b) cubical, and (c) triangular prismatic shapes. The mesh surface is represented by the grey shaded area.
Figure 2.11 Different broken ground volume estimations: (d) conical, (e) hemispherical, and (f) paraboloidal shapes. The mesh surface is represented by the grey shaded area.

These volume estimations, however, are calculated based on the assumptions that the height of the failure depth, “z”, is equal to the bolting spacing (except for the hemispherical shape, due to its geometrical limitation. In this case, half of the bolting spacing is assumed) and that there is no sagging of mesh. In all cases, uniform loading is assumed on mesh.

Table 2.1 The maximum probable vertical pressure comparison between Coates (1981)’s solution and the six broken rock mass volume estimations, in an ascending order of volume estimation.

<table>
<thead>
<tr>
<th>Broken ground volume estimation (m$^3$)</th>
<th>Maximum probable vertical pressure, $P_v$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coates (1981)</td>
<td>--</td>
</tr>
<tr>
<td>Conical</td>
<td>0.26</td>
</tr>
<tr>
<td>Hemispherical</td>
<td>0.26</td>
</tr>
<tr>
<td>Pyramidal</td>
<td>0.33</td>
</tr>
<tr>
<td>Paraboloidal</td>
<td>0.39</td>
</tr>
<tr>
<td>Triangular prismatic</td>
<td>0.50</td>
</tr>
<tr>
<td>Cubical</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The comparison shows that the calculated maximum probable vertical pressure from Coates (1981)’s equation falls in between the values obtained from the triangular prismatic and the cubical volume estimations. These simplified loading estimations, however, do not represent the complex loading mechanisms observed in underground mines.

Table 2.2 shows a range of calculated maximum probable tensile forces based on a 5.6 mm diameter welded wire mesh with a tensile strength of 450 MPa. The analysis also assumes that the maximum probable sag of membrane is 25 % of the bolting spacing. The calculated results show that, as the bolting spacing increases, the maximum probable vertical pressure induced by the weight of the broken rock increases. This is expected since the maximum probable vertical pressure estimated by Coates (1981) is dependent on two parameters: unit weight of the broken rock and bolting spacing. Table 2.3 shows a range of calculated maximum probable tensile forces based on different maximum probable sag percentages.
Table 2.2  The maximum probable vertical pressure and maximum probable tensile force on mesh with varying bolting spacing, based on the analytical equations by Coates (1981).

<table>
<thead>
<tr>
<th>Bolting spacing (m)</th>
<th>Maximum probable vertical pressure (kPa)</th>
<th>Maximum probable tensile force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>15.40</td>
<td>6.16</td>
</tr>
<tr>
<td>0.9</td>
<td>17.33</td>
<td>7.80</td>
</tr>
<tr>
<td>1.0</td>
<td>19.26</td>
<td>9.63</td>
</tr>
<tr>
<td>1.1</td>
<td>21.18</td>
<td>11.65</td>
</tr>
<tr>
<td>1.2</td>
<td>23.11</td>
<td>13.86</td>
</tr>
<tr>
<td>1.3</td>
<td>25.03</td>
<td>16.27</td>
</tr>
</tbody>
</table>

The calculated results show that, as the bolting spacing increases, the maximum probable vertical pressure induced by the weight of the broken rock increases. This is expected since the maximum probable vertical pressure estimated by Coates (1981) is dependent on two parameters: unit weight of the broken rock and bolting spacing.

Table 2.3  The maximum probable vertical pressure and maximum probable tensile force on mesh with varying maximum probable sag percentage on 1.0 m bolting spacing, based on the analytical equations by Coates (1981).

<table>
<thead>
<tr>
<th>Maximum probable sag (%)</th>
<th>Maximum probable vertical pressure (kPa)</th>
<th>Maximum probable tensile force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>19.26</td>
<td>24.07</td>
</tr>
<tr>
<td>20</td>
<td>38.51</td>
<td>48.14</td>
</tr>
<tr>
<td>30</td>
<td>57.77</td>
<td>72.21</td>
</tr>
<tr>
<td>40</td>
<td>77.02</td>
<td>96.28</td>
</tr>
<tr>
<td>50</td>
<td>96.28</td>
<td>120.35</td>
</tr>
<tr>
<td>60</td>
<td>115.54</td>
<td>144.42</td>
</tr>
<tr>
<td>70</td>
<td>134.79</td>
<td>168.49</td>
</tr>
</tbody>
</table>

The maximum probable tensile force that can occur in mesh also increases with the increase in bolting spacing. This correlation is anticipated since, as the bolting spacing increases, the mesh allows for larger displacement, and the corresponding tensile force in the mesh is then higher. This relationship can be further illustrated by reviewing the calculated results, which show that there is a significant increase in the maximum probable tensile force with the increase in the maximum probable sag percentage.

Even though the maximum probable sag percentage parameter makes a significant impact on the maximum probable tensile force, it is thus far unclear which sag percentage is appropriate for use in the equations presented by Coates (1981). The equations presented by Coates (1981) do not account for other important factors such as wire gauge, mesh aperture, bolting arrangement, etc. which can influence the performance of the mesh as discussed previously. Consequently, due to its simplified problem boundary and its
limitations, the theoretical maximum tensile force obtained from such an approach is not used for design purposes.

### 2.4.1.2 Tannant’s Approach

An analytical approach was developed by Tannant (1995) to predict the theoretical peak load based on both uniform loading (3) and point loading (4) of a single wire fixed at both ends. Figure 2.12 shows the schematic diagrams of these two loading scenarios.

\[
L_p = \frac{2NT}{\sqrt{1 + \left(\frac{s}{4d_p}\right)^2}}
\]  \hspace{1cm} (3)

\[
L_p = \frac{2NT}{\sqrt{1 + \left(\frac{s}{2d_p}\right)^2}}
\]  \hspace{1cm} (4)

where \(L_p\) represents the peak load of the mesh [N], \(N\) represents the number of wires on direct load, \(T\) represents the tensile load of each wire [N], \(s\) represents the reinforcement spacing [m], and \(d_p\) represents the peak displacement of the mesh [m].

The implication of using equations proposed by Tannant (1995) is shown with a simple example. In the example, #6-gauge (4.88 mm nominal diameter) welded wire mesh is used in the calculation. Tannant (1995) discussed diamond bolting pattern as his primary example in his analysis and stated 4 as the “N” factor. To align with his example, the value of 4 is used for this analysis as well. The peak tensile load of each wire is assumed to be 10.3 kN. The peak displacement is assumed to be 25 % of the bolting spacing. The calculated results (Table 2.4) from the equations presented by Tannant (1995) show point loading condition results in lower peak load values when compared to uniform loading conditions. It also shows that while peak displacement varies with bolting spacing, the peak load values do not change. Instead, the tensile load of the wire is the sole parameter that controls the peak load, which is dependent on wire gauges. This result is because the ratio between the bolting spacing and the peak displacement (which is assumed to be 25 % of the bolting spacing) is constant for all values of bolting spacing. The equations presented by Tannant (1995), similar to Coates (1981), require pre-assigning the assumed peak displacement in order to
estimate the corresponding peak load. This is difficult in practice as peak displacement values are not always readily available.

**Table 2.4** The maximum tensile load and vertical peak load for #6-gauge welded wire mesh.

<table>
<thead>
<tr>
<th>Bolting spacing (m)</th>
<th>Peak tensile load (kN)</th>
<th>Point-loading peak load (kN)</th>
<th>Uniform loading peak load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
<tr>
<td>0.9</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
<tr>
<td>1.0</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
<tr>
<td>1.1</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
<tr>
<td>1.2</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
<tr>
<td>1.3</td>
<td>10.3</td>
<td>36.85</td>
<td>58.27</td>
</tr>
</tbody>
</table>

2.4.1.3 Summary of the Analytical Approaches

In conclusion, the analytical approaches presented by Coates (1981) and Tannant (1995) can contribute towards obtaining an estimate of mesh performance. However, these approaches use simplified assumptions. Consequently, they are not used as ground support design tools.

Neither of the approaches can explicitly capture the failure mechanisms of mesh and can only be applicable for simple cases where the currently present dead-weight of broken rock is applied to the mesh (i.e. no change in the static load or dynamic load over time). The analytical approaches used by Coates (1981) and Tannant (1995) also require further information, such as the peak displacement, to be applicable.

2.4.2 Empirical Approaches

Empirical approaches are recommendations or rules that are based on past experiences. There are different empirical rock mass classification systems used for the choice of ground support based on a wide range of engineering applications. Most of the original rock mass classification systems were aimed towards civil engineering applications. Civil engineering applications often consider a longer serviceable life span, public users, and relatively stable ground stress conditions. This also influences the choice of the type of ground support recommendations made by each empirical rock mass classification system. For the purpose of this thesis, the Q-system rock mass classification is examined in more detail as a “great majority” of mines in North America and Australia rely primarily on the Q-system for their selection of the initial ground support patterns and standard ground support practices (Potvin & Hadjigeorgiou 2016).

Barton et al. (1974) introduced the rock mass quality (Q) with respect to the stability of tunnels. Based on the initial analysis of 200 tunnel case studies, a correlation between the amount and the type of permanent
support used in tunnel design was made. The rock mass quality is described as a function of the following six parameters (Barton et al. 1974):

- Rock Quality Designation (RQD)
- Number of joint sets (Jn)
- Roughness of the weakest joints (Jr)
- The degree of alternation or filling along the weakest joints (Ja)
- Water inflow parameter (Jw)
- Stress Reduction Factor (SRF)

The rock mass quality categorizes the rock mass based on the stress condition as well as the type and quality of joints in the rock mass. Barton et al. (1974) noted that most of the data were obtained from Scandinavian case studies. Even though the original tunnel support recommendations by Barton et al. (1974) included the use of wire mesh, wire mesh is no longer accepted in a more recent version by Grimstad & Barton (1993).

Grimstad & Barton (1993) developed a design chart, as shown in Figure 2.13, based on the additional tunnel case studies and made a correlation between the installed ground support and the rock mass quality introduced by Barton et al. (1974).

![Figure 2.13 Rock mass classification – permanent support recommendation based on Q and NMT. Note extensive use of S(fr) as permanent support, after Grimstad & Barton (1993); reproduced from Potvin & Hadjigeorgiou (2016).](image-url)
While it was developed to align with the Norwegian Method of Tunneling (NMT), emphasis was put on the use of wet steel fibre reinforced sprayed concrete “S(fr)” and rock bolting as final tunnel support components (Grimstad & Barton 1993). The original ratings for the Stress Reduction Factor (SRF) were updated to provide a higher range to account for the extreme cases of high stress and hard massive rock. This was done to correlate Q values to the “modern rock support” or steel fibre reinforced sprayed concrete for tunneling, which emerged in the late 1970s (Grimstad & Barton 1993). It was stated that the changes were driven due to their “recent experiences from tunnels under high stresses in hard rock” which included less bolting, but more extensive use of S(fr) (Grimstad & Barton 1993). Nevertheless, this practice does not necessarily correspond to the practices being used in underground hard-rock mines in North America where the use of reinforcement component and steel mesh is more prominent for underground excavations.

Potvin & Hadjigeorgiou (2016) discussed a comprehensive benchmarking review of current ground support design approaches implemented at various underground hard-rock mine sites in Australia and Canada. Figure 2.14 shows the new empirical ground support guidelines presented by Potvin & Hadjigeorgiou (2016) based on the review of current ground support standards from various mine sites in North America and Australia.

Figure 2.14 Ground support guidelines for mine drives of 4 to 6 m spans, after Potvin & Hadjigeorgiou (2016).
The new guidelines focus on four design variables in mine drives’ ground support: bolting pattern (in bolt density), type of surface support, thickness of surface support (in the case of reinforced shotcrete), and the coverage of the ground support down the wall. These variables were collected from 65 mines’ ground support standards during the benchmarking study and were correlated to the $Q_{74}$ values from Barton et al. (1974). $Q_{74}$ was chosen over $Q_{93}$ because the review of GCMPs from Australia and Canada has shown that most mines have not adopted the new SRF$_{93}$ guidelines (Potvin & Hadjigeorgiou 2016).

A review of the 141 case studies representing the ground support standards in each geomechanical domain clearly shows a trend for lower bolt density with better quality of rock mass. The database also shows that when the excavation span was < 5 m, reinforced shotcrete was not frequently used. When $Q_{74}$ was < 1.0, the majority of cases (65 %) have used reinforced shotcrete or a combination of reinforced shotcrete and mesh. When $Q_{74}$ was > 10.0, the majority of cases (86 %) used mesh only. Potvin & Hadjigeorgiou (2016) noted that when $Q_{74}$ was between 1.0 and 10.0, productivity considerations, not only the ground conditions, have a great influence in the selection of the type of surface support. Potvin & Hadjigeorgiou (2016) made a generalization that when $Q_{74}$ was > 1.0, mesh with a higher density provided adequate ground support more so than the reinforced shotcrete. When $Q_{74}$ was < 1.0, the reinforcement component and surface support wall coverage was often extended closely to the floor (< 1 m). When $Q_{74}$ was between 1.0 and 4.0, the majority of cases showed that the ground support coverage stopped around the mid-drift (1 to 3 m from the floor). When $Q_{74}$ was > 4.0, the ground support coverage only went to the shoulders (> 3 m from floor).

One of the advantages of Potvin & Hadjigeorgiou (2016)’s new guidelines is the removal of the ESR value that had been inconsistently applied. In the original Q-system, the ESR parameter was used to reflect the type of infrastructure (e.g. permanent or temporary openings). Potvin & Hadjigeorgiou (2016) discussed that there are two issues associated with such an approach: the original Q database had a very limited set of mine cases and the ESR values, which required subjective interpretation and were selected inconsistently. Another advantage is that the new guidelines were developed based on Ground Control Management Plans (GCMPs) from mines, which contain extensive information about ground conditions and ground support knowledge that are constantly updated as the mine and the ground conditions change. Therefore, the participating guidelines are considered to be a “good reflection of successful ground support practices” at mines (Potvin & Hadjigeorgiou 2016).

The ground support guidelines are limited in part as they do not account for either squeezing ground or rockburst ground conditions, do not support dynamic or yielding bolts, and are only applicable for mine drives with 4 to 6 m span. The guidelines also do not propose when specific ground support components (e.g. wire gauges of mesh, wire spacing of mesh, type of mesh, type of reinforcement component, etc.) are required in different ground conditions.
The choice of the type of ground support component being used at a mine can be highly influenced by local experience and ground conditions. Potvin & Hadjigeorgiou (2016) provided examples that, in Australia, larger and heavier mesh sheets are used since the mesh installation is often performed at the same time as primary bolting using a jumbo drill. A common wire diameter of 5.6 mm diameter mesh sheet with 100 x 100 mm aperture is used (Potvin & Hadjigeorgiou 2016). While, in North America, the mesh installation is generally done manually using hand-held equipment, and, therefore, smaller and lighter sheets are used. Common wire diameters of 3.8 mm (#9-gauge), 4.9 mm (#6-gauge), and 5.8 mm (#9-gauge) mesh sheets with 100 x 100 mm aperture are used.

2.5 Summary

This chapter described the types of surface support components used along with reinforcement components to provide overall ground support to underground excavations. Available analytical and empirical approaches for the design of mesh were also reviewed. It was clearly shown that the analytical approaches by Coates (1981) and Tannant (1995) could provide some understanding of the influence on the mesh performance. However, it was evident that, due to the simplified nature of the approach, they are not used in practice as a method of providing design guidelines for mesh. Empirical approaches provided ground support guidelines that are based on the correlation between rock classification and actual past practices. Potvin & Hadjigeorgiou (2016) successfully developed new empirical design guidelines for ground support systems in underground mines that include wire mesh as a surface support. The new guidelines eliminated the use of ESR parameter, given it was shown to be inconsistently applied with the Barton et al. (1974) support charts. These empirical guidelines are meant to be used as a first pass design in the early stages (i.e. during the feasibility stage and the early mine development).

In-situ observations are another approach which can be used to better understand the mesh behaviour. Chapter 3 provides a detailed discussion of a field investigation that was conducted to understand the mesh behaviour at an underground hard-rock mine in Canada.
Chapter 3

3 Performance of Welded Wire Mesh In-Situ

3.1 Introduction
The best indicator of an effective design of ground support is its performance under field conditions. This chapter aims to provide some insight on the performance of ground support through a simple visual analysis of field observation performed during a 4-month field investigation at a Canadian underground mine. In particular, this chapter reviews the behaviour of welded wire mesh under both static and dynamic loading conditions, as well as the observed interactions between the mesh, loading condition, and boundary condition.

3.2 Field Investigation
3.2.1 Types of Loading on Ground Support
The welded wire mesh is applied to the excavation surface to retain broken rock between rock bolts (Hadjigeorgiou & Potvin 2011b). A critical factor in determining the behaviour of welded steel wire mesh is the loading mechanism. There are two general types of loading conditions that can be induced in underground mines: static and dynamic.

Static loading condition is driven by both simple gravity (e.g. dead-weight of broken rock mass) and mine-induced stress (Bawden 2011). Therefore, the design of the ground support under a static loading condition should account for the load and displacement which gradually accumulate over time (Bawden 2011). In contrast, dynamic loading is driven by a sudden release of stored energy in the rock mass, which can cause a rockburst. Rockburst is defined as a sudden, violent seismic event which causes injury to persons or damage to underground workings (Hedley 1992).

Hedley (1992) classified rockburst into three types: strain-burst, pillar-burst, and fault-slip burst. The strain-burst is a sudden and violent failure of rock caused by high stress concentration at the edge of mine openings. The pillar-burst is a sudden and violent, complete failure of rock pillar that occurs when the pillars “become small and overloaded.” The pillar-burst can cause a seismic event depending on the intensity of the energy released during the failure. Fault-slip rockburst is caused by a sudden slippage along a geological weakness plane. This type of rockburst can cause damage to an excavation due to the energy released from a remote location away from the excavation. Hedley (1992) noted that the fault-slip rockburst
is the “same mechanism as for an earthquake”. Production/development blasts can trigger slip movements of geological faults which then can lead to a fault-slip rockburst.

Dynamic loads can cause damage and therefore significantly impact the longevity of the ground support. However, it is difficult to establish the remaining capacity of ground support after a rockburst. Dynamic loading can also result in a significant bulging of broken ground behind the mesh, which then causes the mesh to be loaded under the static loading over time by the dead-weight of the broken rocks.

During the field investigation performed between May – August 2018, two general modes of static loading were observed on the ground support:

- Uniform static loading
- Discrete static loading

Uniform static loading (Figure 3.1a) is associated with dead-weight of small pieces of broken rock that are created by stress induced fractures leading to the bulging of broken, weak material (e.g. weak ore veins) or by high stress induced cracks around the excavation. In the case of the uniform static loading condition, the applied static load from the dead-weight of the broken rock is more uniformly distributed over the surface of the mesh. Under this load, the mesh bulges and bends along the wider mesh area. In contrast, the discrete static loading (Figure 3.1b) is associated with dead-weight of larger slabs of broken rock wedges, where the applied static load can be localized at discrete points of the welded wire mesh. Under this load, the mesh will bend sharply against the localized and discrete points. In a highly stressed area or in a weak surrounding rock mass, there is a potential for additional upward slabbing to build up. This can apply additional static load to the already existing dead-weight of the broken rock.

Figure 3.1  Two general modes of static loading observed in the field: (a) uniform static loading (from the mine’s internal documentation), and (b) discrete static loading.
These modes of static loading can either occur on the side walls or the back (roof) of a drift. Since the main driving force behind the static loading is gravity, the direction of applied static load on the mesh can differ depending on where the bulged rock mass is behind the mesh. For example, Figure 3.2a shows that the general direction of the applied load, as assumed by the displacement of the broken rock, on the side wall of a drift is sloped downward. In contrast, Figure 3.2b shows a vertical applied static load of the broken rock mass on the roof of a drift. Understanding these different modes of loading condition can help identify any concentrated high load and its load transfer onto the rest of the ground support system.

![Location: side wall](image1)

![Location: back/roof](image2)

**Figure 3.2** Examples of the directions of static loading of bulged broken rock mass behind the mesh: (a) from the side wall of a drift, and (b) from the roof of a drift.

### 3.2.2 Degradation and Failure of Ground Support

In further understanding the critical role that different loading conditions play on the mesh behaviour, the degradation of the ground support is examined. In underground mines, the newly installed supports are typically designed and installed to provide adequate support that befits the ground condition at a particular time in the mine’s life-cycle. However, as the mining progresses, the ground condition changes by further accumulating stress and stress induced fractures. Consequently, this leads to increased applied load on the ground support. With the increase in the applied load, the support performance of ground support will be reduced due to degradation, and it will eventually lead to a failure. However, if the degradation in support capacity does not exceed a critical performance level, the ground support is considered to have reduced performance, but is not considered to have failed (Hadjigeorgiou 2016). However, those ground supports that degrade beyond its critical performance level will eventually fail. The definition of “critical performance level” can be varied depending on the operating tolerance of different mine sites.

Hadjigeorgiou (2016) distinguished two different types of ground support failures. The first failure is caused by the failure of a particular ground support component in the system. This type of failure can cause
redistribution of the load onto the rest of the support system and cause successive component failures. However, Hadjigeorgiou (2016) stated that there is an “inherent degree of redundancy in the choice and design of support systems” which makes it possible that a component failure does not always lead to the failure of the support system. The second type of failure is caused by a component not meeting its performance criteria (e.g. service life). According to Hadjigeorgiou (2016), the service life of ground support can be defined by the operating life of each excavation of which the ground support is used in. The duration of the operating life of an excavation can vary depending on the type of excavation (e.g. production drift, or main access ramp) and mine sites.

Hadjigeorgiou (2016) noted that all ground supports are susceptible to degradation over time but are exposed to a varying degree. Table 3.1 presents the factors that cause or accelerate the degradation process of ground support in underground mines. Hadjigeorgiou (2016) illustrated that the degradation of ground support can be better understood by examining the degradation process in terms of the increased demand and the loss of ground support capacity.

<table>
<thead>
<tr>
<th>Causes for degradation of support, after Hadjigeorgiou (2016).</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Increased demand</strong></td>
</tr>
<tr>
<td>Change in stress</td>
</tr>
<tr>
<td>Rock mass degradation</td>
</tr>
<tr>
<td>Increase in excavation dimensions</td>
</tr>
<tr>
<td>Mine induced seismicity</td>
</tr>
<tr>
<td>Blast loading</td>
</tr>
</tbody>
</table>

Simser (2007) suggested that degradation and failure will manifest at the point of the weakest link that exists in a ground support system. In the next section, the Simser (2007)’s finding on the weakest link is reviewed in comparison to a field investigation undertaken at a Canadian hard-rock underground mine.

### 3.2.3 Weakest Links between Different Ground Support Components

The loading condition is a critical factor on the mesh behaviour. In order to understand the overall performance of ground support system, the crucial connecting interfaces (i.e. links) that exist between different ground support components should be reviewed (Charette & Bennett 2017; Simser 2007). The interfaces of different ground support components can be examined through a comprehensive review of “weakest link”. Simser (2007) illustrated that failure in a ground support system will occur at the weakest link that may exist between different ground support components. In other words, a localized failure of the ground support system at the weakest link can lead to a failure of the support before reaching the ground support system’s design capacity. The overall capacity of the ground support system, therefore, is dependent on the strength of the weakest link. This concept illustrates that the success of the ground support system is not merely measured by the sum of the working capacities of individual components (Simser 2007).
Simser (2007) noted during the simple underground observations of some Canadian Shield hard-rock mines that the weakest link in a ground support system could exist at three different contact locations:

- The contact between the rock bolt and rock bolt plate
- The rock bolt plate and wire mesh
- The overlap between two wire meshes

These three contact locations were referenced when conducting the visual inspection at the Canadian hard-rock underground mine. The findings from the visual inspections are outlined below, along with the comparative analysis of the findings to Simser (2007) and Charette & Bennett (2017).

3.2.3.1 Contact between Rock Bolt and Rock Bolt Plate

In general, welded wire mesh is pinned to the wall and the back of a drift by the rock bolts at some discrete points defined by the bolt spacing and bolting pattern. The failure at the contact between a rock bolt and its plate can create an opening in the mesh allowing for deterioration of the surrounding rock. Simser (2007) stated that “one of the more common resin rebar failure modes in hard brittle ground is the plate ripping over the bolt nut or the nut breaking at the bolt threads”, as shown in Figure 3.3.

**Figure 3.3** Examples of rock bolt nut ripping through the plate: (a) 125 mm² rebar plates after dynamic loading, after Simser (2007) and (b) bulking caused by schistose layers delaminating under load, after Simser (2007).

Simser (2007) illustrated through some of the earlier preliminary drop-tests conducted at the Noranda Technology Centre (NTC) on 22 mm diameter rebar and showed that this issue could be controlled by increasing the rock bolt plate’s thickness. Simser (2007) noted that the use of yielding rock bolt can also reduce the premature failure of the rock bolt plate under dynamic load, since the yielding rock bolt absorbs more energy than a stiff rebar. Figure 3.4 shows some of the observations based on the Canadian hard-rock mine’s internal documentation. Contrary to what Simser (2007) noted, the Figure 3.4 shows that a certain
combination of ground support components and ground condition can cause the rock bolt to break near the toe instead of the plate at the collar.

Figure 3.5 shows some examples of a typical “pressure cross” on the rock bolt plate. The pressure cross is a good indicator that the plate has taken the load and collapsed to a certain degree. These visual inspections found that there are more possible variations on the location of weakest links, depending on the different parameters of ground support components, than that were identified by Simser (2007).

![Figure 3.4](image1)  
**Figure 3.4** Examples of rock bolts breaking at toe, while the contact between the rock bolt and the rock bolt plate was intact (from the mine’s internal documentation).

![Figure 3.5](image2)  
**Figure 3.5** Examples of the plate taking loads and displaying “pressure cross” (from the mine’s internal documentation).

Charette & Bennett (2017) investigated the link between rock bolt and rock bolt plate by examining the load transfer on a simple support system during a laboratory impact test. The test (Figure 3.6) involved a set of bolts equipped with strong flat plate and a set of a strong but still deformable domed plate.
The reinforcement system using a deformable domed plate absorbed 5 kJ more than the stiff system using a strong flat plate. The elongation of the rock bolt was reduced by about 20 mm. Charette & Bennett (2017) noted that it is crucial for the rock bolt plate to restrict the nut from going through the plate hole by providing some deformable capability at the bolt head, and for the collapse of the plate to correspond to the ultimate strength of the rock bolt. Both Simser (2007)’s field observations and Charette & Bennett (2017)’s laboratory observations illustrated the importance of achieving a good link between different ground support components (e.g. surface support and rock bolt) to achieve the best ground support system.

### 3.2.3.2 Contact between Rock Bolt Plate and Wire Mesh

It is possible that the mesh fails at the plate. In this case, the shear contact strength is mainly controlled by the friction created between the plate surface and the mesh wires. A weak shear contact strength between the plate and the mesh wires allows for the mesh wires to slip underneath the plate, thus causing additional displacement on mesh. Factors that influence the contact shear strength may include:

- The pre-tension loads on the plate
- The surface area of the rock bolt plate
- The roughness of the rock bolt plate and the mesh surface
- The number of wires directly loaded underneath the plate

The number of mesh wires directly loaded underneath the rock bolt plate can be increased either by reducing the spacing of mesh wires (i.e. aperture) or increasing the surface area of the rock bolt plate. Often, when there is a failure at the contact between the rock bolt plate and the steel wire mesh, the rock bolt plate’s
sharp edge cut through the steel wire mesh, as shown in Figure 3.7a. Simser (2007) suggested that an additional component, such as a mesh plate, can greatly reduce the potential for this type of failure.

Figure 3.7 Examples of: (a) rock bolt plate cutting through welded wire mesh, after Simser (2007), and (b) mesh plate cutting through the welded wire mesh.

Figure 3.8 shows that it is still possible for the rock bolt plate with additional #0-gauge mesh plate to cut through the welded wires of mesh under certain dynamic loads. These contrasting observations demonstrate the difficulties in determining the best ground support components for a particular ground condition.

Figure 3.8 Photos showing an example of the rock bolt plate and mesh plate cutting through the welded wire mesh (from the mine's internal documentation).

3.2.3.3 Contact at Wire Mesh Overlap

On several occasions it was observed that the bulging of rock mass behind the mesh can result in opening of the mesh. This is because the mesh overlaps are often merely held by the strength of the mesh wires.
along the vicinity of overlaps and a couple of rock bolts at some discrete points on the mesh (depending on the bolt spacing and bolting pattern).

Figure 3.9 shows an example of failure at the mesh overlap. A 0.5 Mn seismic event caused a rock-burst and #6-gauge welded wire mesh ruptured along the overlap. Similarly, in another case (Figure 3.10) #7-gauge welded wire mesh ruptured at the mesh overlap once the rock bolt and its plate ruptured through the mesh wires.

Figure 3.9  Broken mesh overlap due to dynamic loading (from the mine’s internal documentation).

Figure 3.10  Status of a ground support system after 2.4 Mn rock-burst event (from the mine’s internal report).
In some cases (Figure 3.11 and Figure 3.12), the use of #0-gauge wire mesh straps successfully aided in taking the initial dynamic impact from a rockburst event.

![Figure 3.11](image1)

**Figure 3.11** The photos showing the ground support system after 1.4 Mn seismic event due to a nearby production blast: (a) the ore vein tried to squeeze out, and (b) the ground support failed eventually (from the mine’s internal documentation).

![Figure 3.12](image2)

**Figure 3.12** The photos showing the status of a ground support system after a 2.2 Mn seismic event impacted on the wall of an advancing drift. The support retained approximately 12 to 15 t of broken materials: (a) general overview after the event, and (b) a close-up view of the ground support system.

Table 3.2 presents a summarized review of observations. The limited review shows that the ground support system mainly ruptured at the mesh-to-mesh overlap and the plate-to-mesh contact. The study also illustrates that the potential rupture at the rock bolt-to-plate contact and the plate-to-mesh contact can have significant influence on the overall performance of a ground support system.
### Table 3.2 Summary table of the ground support system’s performance in relation to historical events collected from the personal observations and the mine’s internal documentations.

<table>
<thead>
<tr>
<th>Case #</th>
<th>Grounds support</th>
<th>Rupture location</th>
<th>Additional support at the location of rupture?</th>
<th>Related event</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>#6-gauge welded mesh, 7/8” dia.</td>
<td>Rock bolt to rock bolt plate interface</td>
<td>X</td>
<td>#0-gauge mesh plate</td>
</tr>
<tr>
<td></td>
<td>resin rebar</td>
<td>Plate to mesh interface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>MM31 mechanical bolt, 3/4” dia.</td>
<td>Rockburst</td>
<td>X</td>
<td>MM31 mechanical bolt, 3/4” dia. resin rebar, #7-gauge welded mesh</td>
</tr>
<tr>
<td></td>
<td>resin rebar</td>
<td></td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>MM31 mechanical bolt, 3/4” dia.</td>
<td>X</td>
<td>X</td>
<td>#0-gauge mesh plate</td>
</tr>
<tr>
<td></td>
<td>resin rebar</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>--</td>
<td>X</td>
<td>#0-gauge mesh strap &amp; mesh plate</td>
<td>Rockburst</td>
</tr>
<tr>
<td>5</td>
<td>3/4” dia. dynamic bolt, #6-gauge</td>
<td>--</td>
<td>X</td>
<td>#0-gauge mesh strap &amp; mesh plate</td>
</tr>
<tr>
<td></td>
<td>welded wire mesh</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.3 Window Damage Analysis (WDA)

Most mining operations record the performance of ground support following falls of ground. Examples of the valuable lessons learned from these were presented in the previous section. The above review of different types of loading conditions as well as the concept of “weakest link” illustrated that simple visual inspection can provide useful information on the performance of ground support system under a particular ground condition. Nevertheless, as they provide only an image of the post-event status of the ground support system, the role of degradation in ground support is not explicitly considered. Therefore, the present method of recording ground support performance collects incomplete information.

Hadjigeorgiou (2016) showed a conceptual representation of degradation and failure of ground support, as shown in Figure 3.13, in order to better understand the ground support performance. The diagrams plot the anticipated or actual exposure time of ground support under particular ground condition on the x-axis and the performance indication on the y-axis. The conceptual representation diagrams show the progress of ground support degradation under both static and dynamic loading conditions over time.
The process of quantifying the impact of degradation on the ground support performance is challenging. However, this is important to be able to ensure the long-term ground support performance during its service life.

For this purpose, a systemic Window Damage Analysis (WDA) can provide additional guidelines on conducting a more consistent visual inspection which can be used as a method to capture degradation of ground support. The Window Damage Analysis is a qualitative back-analysis tool that can allow for a consistent and structured way of conducting visual investigation on the performance of the ground support system at a particular location of interest (“window”) over a set period of time. For the purpose of this thesis, the WDA was conducted at active locations around the mine in an attempt to capture the performance of the ground support system and its interaction with the rock bolts. The objective was to explore how a WDA could provide useful information.

Prior to conducting WDA, the following four key steps were taken:

a) An area of interest was selected, and the window was drawn, as shown in Figure 3.14.

b) A reference point was selected.

c) A procedure for collecting and storing database was developed.

d) Collected information was analyzed to produce meaningful observations.

Two bottom corners of a “window” were outlined with a white spray paint to allow for consistent observation of the area (Figure 3.14). An active development heading was chosen as the area of interest because the ground support system was expected to take loads over time by the changes in the induced stress regime. Also, the designated window was at an area where there was no planned excavation during the review period.

Figure 3.13 Conceptual representations of degradation over time: (a) due to progressive loss of performance, and (b) due to major impact load, after Hadjigeorgiou (2016).
Figure 3.14 The photo showing an example of a “window” on a side wall of an active mine excavation at a Canadian hard-rock mine.

A reference point was chosen and marked on the opposite wall of the drift (facing the window) with a white spray paint to ensure the consistency in the viewing angle and the distance from the window. Prior to heading underground to mark the window and the reference point, a detailed procedure was developed for collecting and storing data. The database included, but does not have to be limited to, the documentation of any near-by mining activities that may influence the performance of the ground support system (e.g. production/development blasting, drilling, etc.), dated photos, determining the period of interest for the review, and the location of the study area. The date and the time of the photos taken were recorded in order to connect any known ground movement/mine activities to the performance of the ground support system. In order to ensure the high quality of photos, multiples of the same photos were taken. During the study, it was noted that unplanned changes to the physical environment at the location can occur due to:

- fine dust (often associated with newly blasted heading or recently travelled heading by heavy equipment) reflecting the light of the camera flash
- axillary infrastructure (e.g. newly installed ventilation tube, electrical wires, etc.) installation
- parked equipment blocking the line-of-sight of the window from the reference point
- unsafe access to the study window due to active seismic activities or falls of ground, or high traffic

These factors can impose significant limitations in obtaining quality photos for the analysis. The back-analysis involved comparing a number of images of the window at different time-frames to observe the
changes to the ground support system. This work was reviewed in reference to near-by mining activities and ground movement recorded in the seismic monitoring system. The comparison study of the two images from different time-frames showed three localized minor damages to the welded steel wire mesh (marked with “yellow circles” in Figure 3.15).

![General overview of the window and localized damages at three locations.](image)

**Figure 3.15** General overview of the window and localized damages at three locations.

Figure 3.16 and Figure 3.17 are close-up views of the two of the three damage locations. The close-up images show that the damage was minor and was associated with one or two wire strands. A preferential direction in the broken wire strands was observed, which ruptured away from the newly blasted face.

![Close-up views of the localized damage at the location #1: (a) status before damage, and (b) status after damage (after 1 ½ months).](image)

**Figure 3.16** Close-up views of the localized damage at the location #1: (a) status before damage, and (b) status after damage (after 1 ½ months).
Therefore, the preferential direction and the locations of the minor damage (near the back and shoulder of the drift) can indicate that these localized damages were likely associated with fly-rocks from a near-by blast.

Another example of a window damage analysis is shown in Figure 3.18. In this example, the mesh screen was damaged due to equipment scraping the side wall while travelling through the drift. In this case, there was no significant visual change to the ground support system and the welded wire mesh besides a minor bending of the wire strands.
Figure 3.19 and Figure 3.20 show another example of the Window Damage Analysis where the ground support system was induced by a 2.2 Mn rockburst event. In this case, there was a notable visual change to the ground support system.

Figure 3.19  The photo of the ground support system taken prior to the seismic event.

Figure 3.20  The photo of the ground support system taken after a 2.2Mn rock-burst event.

Figure 3.21 shows that the mesh plate and the mesh strap both have bent under the dynamic load. This illustrates that the ground support system had taken significant load. However, it was not possible to quantify the amount of the load and the displacement that the ground support system received.
Figure 3.21 The photos of close-up views of the bent mesh strap and mesh plate: (a) prior to the seismic event, and (b) after the seismic event.

Figure 3.22 and Figure 3.23 show another example of a ground support system that had a noticeable visual change after a near-by development blast. In this case, the study area was not under active ventilation, and therefore the photo had to be taken from a distance and with limited light exposure on the wall.

Figure 3.22 The photo showing another example of the window damage analysis on a side wall of a drift.
The photos showing the close-up views of the ruptured mesh overlap: (a) before the blast, and (b) after the blast.

Overall, five WDA cases were collected, and three out of the five cases showed no significant visual changes to the ground support system. However, the other two cases, which were associated with rockburst events, showed significant changes to the ground support system. The result, summarized in Table 3.3, illustrates that, in this particular hard-rock mine, it was hard to capture the static changes of the ground support systems using the simple visual inspection. The practice of reporting the state of the ground support system over a certain study period can provide some insights into its performance. However, the method has some practical limitations and difficulties in quantifying the performance of the ground support elements.

Table 3.3  Summary table of the five window damage analysis cases.

<table>
<thead>
<tr>
<th>Case #</th>
<th>Rupture location</th>
<th>Rupture type</th>
<th>Related event</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mesh overlap</td>
<td>Minor (breakage of one or two wire strands)</td>
<td>Round-blast</td>
</tr>
<tr>
<td>2</td>
<td>Grade line</td>
<td>Minor (bent wire strands)</td>
<td>Heavy-equipment</td>
</tr>
<tr>
<td>3</td>
<td>--</td>
<td>No damage</td>
<td>Production blast</td>
</tr>
<tr>
<td>4</td>
<td>Shoulder and wall</td>
<td>Major (bent mesh strap and washer, bulking of welded wire mesh)</td>
<td>Rock-burst</td>
</tr>
<tr>
<td>5</td>
<td>Mesh-mesh overlap</td>
<td>Major (opened overlap)</td>
<td>Near-by development blast</td>
</tr>
</tbody>
</table>

Some of the limitations associated with the WDA are that:

- It is a two-dimensional analysis. Therefore, it requires a third viewing axis to capture the displacement along the third dimension (i.e. in/out of the page).
- The quality of the analysis is highly dependent on the skill-level of an individual who is responsible for collecting the images.
• It requires a clear direct line-of-sight over a period, which is not always available in a highly active and congested area due to axillary infrastructure, addition of shotcrete, dust, and parked equipment.
• It cannot quantify the working displacements and loads that are being experienced by the ground support systems at any given time. Instead, it provides an approximate visual state of the system.

The above implementation of the WDA only included visual observation on the wall and the shoulder of drifts. The challenge lies in demonstrating the value of the technique and providing a cost-effective implementation.

3.4 Summary

This chapter presented observations made at a Canadian underground hard-rock mine specific to mesh behaviour. In the field, different types of loading conditions, from static to dynamic, were reviewed and were further categorized into uniform and discrete point loading conditions. The chapter further explored the concept of “weakest link” and in doing so provided additional insight into Simser (2007)’s observations regarding the failed cases of ground support system. The contrasting observations demonstrated the difficulties of determining the best ground support components for a particular ground condition. Lastly, by undertaking Window Damage Analysis of select locations at the mine, additional guidelines on conducting a more consistent visual inspection over time were considered.

In reviewing these variables, the field observation illustrated the importance of accounting for these parameters and how they interact with each other to impact the mesh performance in the ground support system. While it was evident that the field investigation can lead to some qualitative understanding of how the mesh behaves, the simple nature of the method contained some practical limitations.

This chapter demonstrated the advantages of a monitoring strategy of support behaviour. This, however, is only one of the measures that can be implemented at a mine site. Other options include instrumentation and monitoring of individual elements. The motivation for utilizing the present approach was to identify the type of loading as opposed to the magnitude. This is necessary in the design of laboratory investigation for quantifying the performance of mesh.

To better quantitatively capture the capacities of the mesh performance and how these parameters interact with each other, Chapter 4 discusses various laboratory testing rigs to see the influence of different test parameters on the mesh performance.
4 Testing of Welded Steel Wire Mesh

4.1 Introduction

In Chapter 3, a collection of visual field observations of the parameters that impact ground support performance at a Canadian underground hard-rock mine were discussed. Although visual inspections can contribute to an understanding of the interactions of various ground support components, the performance of mesh in particular is difficult to measure and quantify.

There have been several attempts to provide a quantitative indicator of mesh performance in the laboratory. The development of impact testing rig for ground support has been discussed elsewhere, Hadjigeorgiou & Potvin (2011a), and is outside the scope of this thesis. This work focuses on the static and quasi-static testing of mesh in a laboratory setting. This chapter provides a comprehensive review of various static mesh testing rig configurations, and their observed results and interpretations.

4.2 Working and Ultimate Capacities of Welded Wire Mesh

As Potvin & Hadjigeorgiou pointed out, the load and displacement capacities of the welded wire mesh can be divided into two main categories, as shown in Figure 4.1: working capacity and ultimate capacity. The working capacity is defined as the high point on the loading response curve prior to the initial failure of mesh is achieved and is followed by a significant drop in the load. Once the load drops, it can increase again and exceed the working capacity before resulting in the ultimate mesh failure (i.e. no more increase in the load).

![Figure 4.1 Conceptual load and displacement graph of a laboratory test of welded wire mesh showing the working and ultimate capacities, after Potvin & Hadjigeorgiou (2018).](image)
The ultimate capacity is defined as the point where the highest point of the loading response curve is achieved before the ultimate mesh failure and is followed by a significant drop in the load. Contrary to the working capacity, the load does not surpass the ultimate capacity once it drops. There are no universal or standardized definitions. Consequently, even though testing rigs can be used to quantify the working and ultimate load and displacement capacities of the welded wire mesh, there is no standardized way to construct and conduct such a test. Therefore, each testing rig is constructed differently, which can create inconsistencies in the obtained results. In effect, the results are then subjective to each particular testing rig configuration. Even within the same testing facility, there are different test parameters that produce variations specific to each laboratory experiment of welded wire mesh. For these reasons, this chapter reviews the influence of different parameters governing the load and displacement capacities of the welded steel wire mesh through various testing rigs and their observed results.

4.3 Laboratory Testing Rigs of Mesh

There are numerous variations of testing rig configurations developed over the years to investigate the mesh performance. The performance of wire mesh as a surface support component in the ground support system has received less attention (Ortlepp 1983). The assumption was made that the steel wire mesh was simply a passive retaining support for the dead-weight of small broken rock mass between reinforcement components.

4.3.1 The Ortlepp Rig

One of the first static testing rigs was developed by Ortlepp (1983) in South Africa. The testing rig consisted of a 1.1 m x 1.1 m square heavy steel frame with “a peripheral clamping arrangement” (Figure 4.2). The load was applied at the center of the mesh through “four steel plates covering an area of 0.42 m x 0.52 m”.

Figure 4.2 Wire mesh testing rig, after Ortlepp (1983).
Even though a range of different types of mesh samples were tested, there was a limited number of tests conducted for each mesh type. There was limited information provided on the description of each mesh type and their test results. The “diamond mesh”, as introduced by Ortlepp (1983), is believed to be the same mesh type as the chain link mesh, as previously described in Chapter 2, based on its description of “inter-linking construction”. Ortlepp (1983) suggested that the diamond mesh cannot be made of high-tensile wire or with an aperture larger than 100 mm. This is different than high-tensile wire chain link mesh currently commercially available (Fischer et al. 2017).

Table 4.1 summarizes the “load and displacement at failure” for 3.2 mm wire diameter welded wire mesh samples and 2.5 to 4.0 mm wire diameter diamond (stainless and galvanised) mesh samples. The description of “load at failure”, as presented by Ortlepp (1983), was not clear as to whether it referred to the maximum load prior to the initial failure of the mesh or the highest load prior to the ultimate mesh failure. A load and displacement comparison diagrams showed no drop in the load, this was interpreted that the tests were not conducted beyond the initial failure of mesh. The welded wire mesh samples showed the lowest failure load and deformation when compared to other types of mesh samples that were tested.

Table 4.1  The failure load and displacement values from the test results of welded wire mesh and diamond mesh samples, after Ortlepp (1983).

<table>
<thead>
<tr>
<th>Type of mesh</th>
<th>Wire gauge # (wire diameter in mm)</th>
<th>Wire spacing (mm)</th>
<th>Failure load (kN)</th>
<th>Failure displacement¹ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded</td>
<td>(3.2)</td>
<td>100 x 100</td>
<td>20.0</td>
<td>106.4</td>
</tr>
<tr>
<td>Welded</td>
<td>(3.2)</td>
<td>100 x 100</td>
<td>19.0</td>
<td>106.4</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>19.5 (0.5)</strong></td>
<td><strong>106.4</strong></td>
</tr>
<tr>
<td>Diamond (stainless)</td>
<td>(2.5)</td>
<td>55 x 55</td>
<td>71.3</td>
<td>212.6</td>
</tr>
<tr>
<td>Diamond (stainless)</td>
<td>(2.5)</td>
<td>65 x 65</td>
<td>66.8</td>
<td>185.4</td>
</tr>
<tr>
<td>Diamond (stainless)</td>
<td>(2.5)</td>
<td>65 x 65</td>
<td>64.1</td>
<td>185.4</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>67.4 (3.0)</strong></td>
<td><strong>194.5 (12.8)</strong></td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(3.2)</td>
<td>55 x 55</td>
<td>60.0</td>
<td>--</td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(3.2)</td>
<td>50 x 50</td>
<td>58.8</td>
<td>151.7</td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(3.2)</td>
<td>50 x 50</td>
<td>57.7</td>
<td>151.7</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>58.8 (0.9)</strong></td>
<td><strong>151.7</strong></td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(4.0)</td>
<td>105 x 105</td>
<td>51.0</td>
<td>147.3</td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(4.0)</td>
<td>105 x 105</td>
<td>44.5</td>
<td>147.3</td>
</tr>
<tr>
<td>Diamond (galvanised)</td>
<td>(4.0)</td>
<td>100 x 100</td>
<td>50.0</td>
<td>--</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>48.5 (2.9)</strong></td>
<td><strong>147.3</strong></td>
</tr>
</tbody>
</table>

¹ The failure displacements of different mesh samples were extracted from load - displacement results from Ortlepp (1983) diagrams of different types of mesh (after Ortlepp 1983)
² Mean and standard deviation (s.d.)

Furthermore, Ortlepp (1983) noted the importance of the strength of wire intersections stating that “failure of the strands invariably occurred at a mesh intersection or ‘cross-over’.” In effect, “different degrees of stress-concentration and impairment of material properties coexisted,” depending on the type of cross-over
used (e.g. welded, woven, knotted, or linked). The test also showed that the inter-linking construction of “diamond mesh” caused the “least impairment of the potential strength of the wire”.

Figure 4.3 compares the failure load and displacement values based on the test results from Ortlepp (1983). The comparison shows that diamond mesh had significantly higher failure load and displacement compared to the welded wire mesh. It would appear that diamond mesh with the thinnest wire diameter (2.5 mm) provided the highest load and displacement at failure. The direct comparison between diamond mesh samples can be misleading, as the samples had different wire gauges as well as different wire spacings. Therefore, a significant correlation cannot be made between the mesh performance and wire gauge parameter based on Ortlepp (1983)’s test results.

![Failure load and displacement of welded wire mesh and diamond mesh, based on the results from Ortlepp (1983).](image)

**Figure 4.3** Failure load and displacement of welded wire mesh and diamond mesh, based on the results from Ortlepp (1983).

### 4.3.2 Ontario Ministry of Labour

A static testing of wire mesh was conducted in an operating underground mine in Canada (Pakalnis & Ames 1987). This in-situ testing arrangement (Figure 4.4) involved bolting the drift wall with 1.2 m and 1.8 m long 16 mm threaded-both-end reinforcement component in a 1.2 x 1.2 m staggered pattern; this represented the “most common practice in Ontario mines” at the time (Pakalnis & Ames 1987).

The mesh samples were installed using wooden washers topped with 100 x 100 mm steel reinforcement plates. The 305 x 305 mm, 95 mm thick steel plate (with rounded edges) was used to incrementally pull the steel mesh from the center using a pulling chain. The pulling apparatus was anchored to the opposite side of the drift wall. It can be argued that such a test arrangement can allow for the mesh performance to be influenced by the deformation of the pulling apparatus anchored on the opposite side.
In this in-situ test, four different types of welded wire mesh and five different types of chain link mesh were investigated. In general, mesh failure occurred around the reinforcement components instead of at the pulling plate (Pakalnis & Ames 1987). It was observed that the dimension of the loading plate influences the maximum mesh capacity. A larger dimension loading plate would result in a more distributed load on the mesh and consequently would reduce the tensile stress in the wire strands. The validity of this observation is discussed in detail in Chapter 6 using numerical modelling. Table 4.2 summarizes the “maximum test load and displacement” results from the in-situ tests.

Table 4.2 The maximum test load and displacement values from the in-situ pull test results of welded wire mesh and chain link mesh samples, after Pakalnis & Ames (1987).

<table>
<thead>
<tr>
<th>Mesh Type</th>
<th>Wire gauge # (wire diameter in mm)</th>
<th>Wire spacing (mm x mm)</th>
<th>Max. test load (kN)</th>
<th>Max. test displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded wire mesh</td>
<td>#4 (5.8)</td>
<td>102 x 102</td>
<td>36</td>
<td>292</td>
</tr>
<tr>
<td>Welded wire mesh</td>
<td>#6 (4.9)</td>
<td>102 x 102</td>
<td>32.9</td>
<td>286</td>
</tr>
<tr>
<td>Welded wire mesh</td>
<td>#9 (3.7)</td>
<td>102 x 102</td>
<td>18.7</td>
<td>273</td>
</tr>
<tr>
<td>Welded wire mesh</td>
<td>#12 (2.7)</td>
<td>102 x 51</td>
<td>13.8</td>
<td>311</td>
</tr>
<tr>
<td>Mean (s.d.)³</td>
<td></td>
<td></td>
<td>25.4 (9.3)</td>
<td>291 (14)</td>
</tr>
<tr>
<td>Chain link (BM¹)</td>
<td>#11 (3.0)</td>
<td>51 x 51</td>
<td>28.9</td>
<td>413</td>
</tr>
<tr>
<td>Chain link (GAW²)</td>
<td>#11 (3.0)</td>
<td>51 x 51</td>
<td>17.3</td>
<td>349</td>
</tr>
<tr>
<td>Chain link (WG³)</td>
<td>#9 (3.7)</td>
<td>51 x 51</td>
<td>15.6</td>
<td>400</td>
</tr>
<tr>
<td>Chain link (BM¹)</td>
<td>#9 (3.7)</td>
<td>51 x 51</td>
<td>36.5</td>
<td>425</td>
</tr>
<tr>
<td>Chain link (GAW²)</td>
<td>#9 (3.7)</td>
<td>51 x 51</td>
<td>32.0</td>
<td>438</td>
</tr>
<tr>
<td>Mean (s.d.)⁴</td>
<td></td>
<td></td>
<td>26.1 (8.2)</td>
<td>405 (31)</td>
</tr>
</tbody>
</table>

¹ Bare metal (BM)
² Galvanized after welding (GAW)
³ Woven galvanized wire (WG)
⁴ Mean and standard deviation (s.d.)
The load-displacement graphs showed that the “maximum test load”, as denoted by Pakalnis & Ames (1987), corresponded to the initial mesh failure prior to a significant drop in the load. Therefore, “maximum test load” and “maximum test displacement” correspond to the working capacity of mesh, defined in Section 4.2. However, since the tests were not continued after the initial drop in load, it is not clear if the ultimate capacity of mesh was reached.

Pakalnis & Ames (1987)’s in-situ test results showed that the maximum test load increased as the wire diameter of the mesh increased. In general, the chain link mesh had a higher maximum test load than the welded wire mesh under the same wire gauge. Furthermore, all chain link mesh samples displayed higher maximum test displacements than welded mesh samples. Pakalnis & Ames (1987) suggested that the maximum test displacement is more dependent on mesh type than wire diameter, and concluded that the heavy gauge welded wire mesh should be “more suitable for providing additional ground support at lower displacement and improving the overall support system”.

Figure 4.5 compares the maximum test load and displacement from Pakalnis & Ames (1987)’s test results of welded wire mesh and chain link mesh samples. Welded wire diameter (#4-gauge) resulted in the highest maximum test load. The welded wire mesh samples with #4-gauge and #6-gauge wires showed significantly higher maximum test load compared to the welded wire mesh samples with #9-gauge and #12-gauge wires. However, there was no significant difference in terms of the maximum test displacement between the welded wire mesh samples.

![Graph](image)

**Figure 4.5** Maximum test load and displacement of welded wire mesh and chain link mesh, based on results from Pakalnis & Ames (1987).

Chain link mesh had significantly higher maximum displacement than welded wire mesh. The maximum test load, however, did not vary much from the welded wire mesh samples. Chain link mesh showed significant variation in the maximum test load and displacement values. This was due to the use of different types of chain link mesh: bare metal, galvanized after welding, and woven galvanized wire. This variation
of chain link mesh types as well as the limited data sample of the provided load and displacement made it difficult to make a correlation between the mesh performances in relation to other testing parameters, such as wire gauge.

Based on the limited observations, Ortlepp (1983)’s chain link mesh samples with 2.5 mm diameter (i.e. thinnest) provided the highest load and displacement at failure. In contrast, Pakalnis & Ames (1987)’s test results showed the opposite as the welded wire mesh samples with the #4-gauge wires (i.e. thickest) provided the highest maximum test load and displacement. These conflicting observations illustrate that there are other parameters (e.g. wire spacing, construction of mesh, type of mesh steel, etc.) that may influence the performance of mesh.

A comparison of test results by Ortlepp (1983) and Pakalnis & Ames (1987) show the influence of testing rig configuration on the performance of mesh. For example, the maximum test load of the 3.7 mm diameter welded wire mesh samples (18.7 kN) from Pakalnis & Ames (1987)’s in-situ test result was slightly lower than the failure load of the 3.2 mm diameter welded wire mesh samples (19-20 kN) from Ortlepp (1983)’s laboratory test. Ortlepp (1983)’s observation of the comparison was different from the general observed trend from Pakalnis & Ames (1987)’s test results where a thicker wire diameter resulted in a higher maximum test load. Nevertheless, the comparison of the two testing facilities of Ortlepp (1983) and Pakalnis & Ames (1987) was made based on a limited available data.

4.3.3 Tannant

Tannant (1995) investigated the performance of mesh under a static loading condition at the Geomechanics Research Centre (GRC) in Sudbury, Ontario, Canada. A full-scale laboratory pull test, shown in Figure 4.6, was used to test three different bolting configurations (1.2 x 1.2 m diamond pattern, 1.2 x 1.5 m diamond pattern, and 1.2 x 1.2 m square pattern) and three different mesh wire gauges (#9, #6, and #4) for welded wire mesh.

![Figure 4.6](image-url)  
**Figure 4.6** Different bolting configurations for welded wire mesh, after Tannant (1995).
A total of 81 pull tests were performed with an average of three to six pull tests for each testing arrangement (Tannant 1995). Tannant (1995)’s test arrangement included a 0.3 x 0.3 m, 13 mm thick flat steel loading plate (with rounded edges and corners) pulled by an electric winch at the center of the mesh at the rate of 18 to 22 mm/s. The mesh was bolted to the lower part of the test frame by 19 mm diameter rock bolts and 127 x 127 mm, 9.5 mm thick flat steel plate and a nut. A typical mesh aperture size of 102 x 102 mm was used. The rock bolts were pre-tensioned by tightening the nuts with a torque wrench to generate between 1 and 3 tonnes of bolt tension on each reinforcement component. The area of mesh enclosed between the rock bolts varied between 0.72 m$^2$ and 1.44 m$^2$. A rotary potentiometer measured the displacement of the pulling plate at the center, and a load cell measured the load on the mesh. Table 4.3, Table 4.4, and Table 4.5 summarize the “peak load and displacement” results from the laboratory test of welded wire mesh samples with different wire gauges and bolting configurations.

### Table 4.3  Peak load and displacement of the test results using 1.2 x 1.2 m square bolting pattern, after Tannant (1995).

<table>
<thead>
<tr>
<th>Mesh area (m$^2$)</th>
<th>Mesh gauge$^1$ (diameter in mm)</th>
<th>Bolt load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.44</td>
<td>#9 (3.77)</td>
<td>30</td>
<td>13.3</td>
<td>340</td>
</tr>
<tr>
<td>1.44</td>
<td>#6 (4.88)</td>
<td>30</td>
<td>26.0</td>
<td>405</td>
</tr>
<tr>
<td>1.44</td>
<td>#4 (5.72)</td>
<td>30</td>
<td>34.9</td>
<td>426</td>
</tr>
<tr>
<td>Overall mean (s.d.)$^2$</td>
<td></td>
<td></td>
<td>24.7 (8.9)</td>
<td>390 (37)</td>
</tr>
</tbody>
</table>

$^1$ The steel has a minimum ultimate tensile strength of 550 MPa (after Tannant 1995)

$^2$ Overall mean and standard deviation (s.d.) – for 1.2 x 1.2 m square bolting pattern

### Table 4.4  Peak load and displacement of the test results using 1.2 x 1.2 m diamond bolting pattern, after Tannant (1995).

<table>
<thead>
<tr>
<th>Mesh area (m$^2$)</th>
<th>Mesh gauge$^1$ (diameter in mm)</th>
<th>Bolt load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.72</td>
<td>#9 (3.77)</td>
<td>10</td>
<td>17.2</td>
<td>128</td>
</tr>
<tr>
<td>0.72</td>
<td>#9 (3.77)</td>
<td>20</td>
<td>15.0</td>
<td>118</td>
</tr>
<tr>
<td>0.72</td>
<td>#9 (3.77)</td>
<td>30</td>
<td>14.8</td>
<td>105</td>
</tr>
<tr>
<td>Mean (s.d.)$^2$</td>
<td></td>
<td></td>
<td>15.7 (1.1)</td>
<td>117 (9)</td>
</tr>
<tr>
<td>0.72</td>
<td>#6 (4.88)</td>
<td>10</td>
<td>28.4</td>
<td>165</td>
</tr>
<tr>
<td>0.72</td>
<td>#6 (4.88)</td>
<td>20</td>
<td>26.4</td>
<td>125</td>
</tr>
<tr>
<td>0.72</td>
<td>#6 (4.88)</td>
<td>30</td>
<td>24.3</td>
<td>114</td>
</tr>
<tr>
<td>Mean (s.d.)$^2$</td>
<td></td>
<td></td>
<td>26.4 (1.7)</td>
<td>135 (22)</td>
</tr>
<tr>
<td>0.72</td>
<td>#4 (5.72)</td>
<td>10</td>
<td>39.8</td>
<td>205</td>
</tr>
<tr>
<td>0.72</td>
<td>#4 (5.72)</td>
<td>20</td>
<td>44.7</td>
<td>192</td>
</tr>
<tr>
<td>0.72</td>
<td>#4 (5.72)</td>
<td>30</td>
<td>38.2</td>
<td>152</td>
</tr>
<tr>
<td>Mean (s.d.)$^2$</td>
<td></td>
<td></td>
<td>40.9 (2.8)</td>
<td>183 (23)</td>
</tr>
<tr>
<td>Overall mean (s.d.)$^3$</td>
<td></td>
<td></td>
<td>27.6 (10.5)</td>
<td>145 (34)</td>
</tr>
</tbody>
</table>

$^1$ The steel has a minimum ultimate tensile strength of 550 MPa (after Tannant 1995)

$^2$ Mean and standard deviation (s.d.)

$^3$ Overall mean and standard deviation (s.d.) – for 1.2 x 1.2 m diamond bolting pattern
Table 4.5  Peak load and displacement of the test results using 1.2 x 1.5 m diamond bolting pattern, after Tannant (1995).

<table>
<thead>
<tr>
<th>Mesh area (m²)</th>
<th>Mesh gauge¹</th>
<th>Bolt load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.90</td>
<td>#9 (3.77)</td>
<td>10</td>
<td>16.4</td>
<td>189</td>
</tr>
<tr>
<td>0.90</td>
<td>#9 (3.77)</td>
<td>30</td>
<td>11.2</td>
<td>106</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>13.8 (2.6)</strong></td>
<td><strong>148 (42)</strong></td>
</tr>
<tr>
<td>0.90</td>
<td>#6 (4.88)</td>
<td>10</td>
<td>26.9</td>
<td>192</td>
</tr>
<tr>
<td>0.90</td>
<td>#6 (4.88)</td>
<td>30</td>
<td>23.6</td>
<td>140</td>
</tr>
<tr>
<td><strong>Mean (s.d.)²</strong></td>
<td></td>
<td></td>
<td><strong>25.3 (1.7)</strong></td>
<td><strong>166 (26)</strong></td>
</tr>
<tr>
<td>0.90</td>
<td>#4 (5.72)</td>
<td>10</td>
<td>38.3</td>
<td>216</td>
</tr>
<tr>
<td>0.90</td>
<td>#4 (5.72)</td>
<td>30</td>
<td>30.6</td>
<td>159</td>
</tr>
<tr>
<td><strong>Mean (s.d.)³</strong></td>
<td></td>
<td></td>
<td><strong>34.5 (3.9)</strong></td>
<td><strong>188 (29)</strong></td>
</tr>
<tr>
<td><strong>Overall mean (s.d.)³</strong></td>
<td></td>
<td></td>
<td><strong>24.5 (8.9)</strong></td>
<td><strong>167 (37)</strong></td>
</tr>
</tbody>
</table>

¹ The steel has a minimum ultimate tensile strength of 550 MPa (after Tannant 1995)
² Mean and standard deviation (s.d.)
³ Overall mean and standard deviation (s.d.) – for 1.2 x 1.5 m diamond bolting pattern

Tannant (1995) defined the "peak load" as the first major peak in the load prior to "a significant drop in load…invariably caused by failure of a mesh wire”. This definition is in close alignment with the definition of working capacity load in Section 4.2. The smaller load drops in the loading response were generally associated with the slippage of the mesh wires underneath the reinforcement plate (Tannant 1995).

Table 4.6 shows the mean peak load and displacement from Tannant (2004)’s tests of chain link mesh. A limited comparison of the peak load and displacement of both mesh types show that the significantly higher peak load and displacement were observed for chain link mesh over welded wire mesh under the same wire gauge, bolting spacing, and bolting pattern.

Table 4.6  Peak load and displacement of a chain link mesh using a 1.2 x 1.2 m diamond bolting pattern, based on the test results from Tannant (2004).

<table>
<thead>
<tr>
<th>Mesh gauge¹</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#9 (3.77)</td>
<td>34.4</td>
<td>417</td>
</tr>
</tbody>
</table>

Figure 4.7 compares the peak load and displacement from Tannant (1995)’s test results of the welded wire mesh under three different bolting configurations. The comparison shows that the diamond bolting pattern at a close bolting spacing of 1.2 x 1.2 m provided the lowest peak displacement, or stiffest loading response; while the 1.2 x 1.2 m square bolting pattern provided the highest peak displacement.

Figure 4.8 compares the peak load and displacement from Tannant (1995)’s test results of welded wire mesh under three different wire gauges. The comparison shows that the peak load was the highest for the thinnest wire mesh (i.e. #4-gauge wire mesh). This general observation aligned with the observation of the welded wire mesh performance in relation to mesh wire gauges from Pakalnis & Ames (1987)’s test results.
Tannant (1995)’s test results showed that the mesh wire gauge has the dominant control on the mesh’s load capacity, while the bolting pattern did not greatly affect the peak load of the mesh.

Figure 4.7 Peak load and displacement grouped based on different bolting patterns, based on the tests results from Tannant (1995).

Figure 4.8 Peak load and displacement grouped based on different mesh wire gauge, based on the test results from Tannant (1995).

Tannant (1995) discussed that the loading stiffness of welded wire mesh varied significantly depending on the direction of pull in relation to the orientation of wires. For example, the mesh with a diamond bolting pattern, at a close bolting spacing, provided a stiffer loading response than the mesh tested using a square
bolting pattern or with wider bolting spacing. Tannant (1995) further discussed that different “mesh failure modes” can be observed depending on the type of bolting pattern. For example, the mesh tested using a diamond bolting pattern generally failed by tensile failure of the mesh wires either near the edge of the reinforcement plates or the pulling plate (Tannant 1995). In contrast, failure of the mesh was, in a square bolting pattern before the wires broke.

Table 4.7 compares the peak load and displacement from Tannant (1995)’s test results and the maximum test load and displacement from Pakalnis & Ames (1987)’s test results of welded wire mesh using a 1.2 x 1.2 m diamond bolting configuration. In both cases, mesh wire spacing of 102 x 102 mm was used. Tannant (1995)’s test results were based on three different bolt pre-tension loads: 10, 20, and 30 kN, while the bolt tension load used for Pakalnis & Ames (1987)’s tests is unknown. The bolt tension loads can be an important parameter to consider since Tannant (1995)’s test results showed that the bolt tension loads of < 30 kN were unable to prevent the wires from sliding under the reinforcement plates. This slippage of mesh wire ultimately produced a higher peak displacement and overall softer loading response (Tannant 1995).

Table 4.7 Comparison between Tannant (1995)’s peak load and displacement and Pakalnis & Ames (1987)’s maximum test load and displacement from the tests of welded wire mesh using a 1.2 x 1.2 m diamond bolting pattern.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak/maximum test load (kN)</td>
<td>Peak/maximum test displacement (mm)</td>
</tr>
<tr>
<td></td>
<td>#4</td>
<td>#6</td>
</tr>
<tr>
<td>Pakalnis &amp; Ames (1987)</td>
<td>36.0</td>
<td>32.9</td>
</tr>
</tbody>
</table>

The comparison shows that the maximum test load from Pakalnis & Ames (1987) fall outside of Tannant (1995)’s peak load ranges. For example, the maximum test load of the #4-gauge wire mesh from Pakalnis & Ames (1987)’s test result was lower than the peak load range provided by Tannant (1995)’s #4-gauge wire mesh test results. The Pakalnis & Ames (1987)’s maximum test loads from the #6- and #9-gauge wire meshes were higher than Tannant (1995)’s peak load ranges of #6- and #9-gauge wire meshes. The maximum test displacement from Pakalnis & Ames (1987)’s tests for all three mesh wire gauges were significantly higher than the peak displacement ranges provided by Tannant (1995).

Table 4.8 summarizes the general testing rig configurations used by Pakalnis & Ames (1987) and Tannant (1995). Based on the limited comparisons of the available data, both testing configurations had similar general loading and boundary conditions, and therefore were expected to obtain similar results. However, the previous comparison illustrated that there are variations in the load and displacement results. This may illustrate that there can be other testing parameters which influence the performance of the mesh other than the loading plate dimension, bolting pattern, and bolting spacing.
Table 4.8 Comparison table of general testing rig configurations between the tests from Pakalnis & Ames (1987) and Tannant (1995).

<table>
<thead>
<tr>
<th></th>
<th>Loading plate</th>
<th>Bolting pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pakalnis &amp; Ames</td>
<td>305 mm square 9.5 mm thick steel plate with</td>
<td>1.2 x 1.2 m diamond</td>
</tr>
<tr>
<td>(1987)</td>
<td>rounded edges</td>
<td></td>
</tr>
<tr>
<td>Tannant (1995)</td>
<td>300 mm square 13 mm thick, flat steel plate with</td>
<td>1.2 x 1.2 m diamond</td>
</tr>
<tr>
<td></td>
<td>rounded edges</td>
<td></td>
</tr>
</tbody>
</table>

4.3.4 Western Australian School of Mines (WASM)

A static testing rig for mesh was constructed and tested by Thompson et al. (1999) in Australia at the Western Australian School of Mines, Curtin University of Technology (WASM). Figure 4.9 shows the testing rig setup for welded wire mesh.

![Mesh testing facility](image)

The welded wire mesh samples used for the test consisted of 5.6 mm diameter steel wires, with a mesh wire spacing of 100 x 100 mm. Several testing parameters were varied during the tests, including:

- Bolting pattern
- Bolting spacing
- Loading frame orientation
- Loading frame dimension
- Mesh sample orientation
- Reinforcement plate orientation

Two different square steel loading frame dimensions (750 x 750 mm and 1050 x 1050 mm) were used. The welded wire mesh was held down to the concrete floor with four 10 mm thick, 200 x 200 mm square steel
reinforcement plates and reinforcement components. The reinforcement components were loaded with a pre-tension load of approximately 45 kN. An electronic load cell was used to measure the applied force to the welded wire mesh. The displacement was measured by counting the number of revolutions of the capstan, as a single rotation resulted in “10 mm of vertical displacement” (Thompson et al. 1999).

Table 4.9 and Table 4.10 show the maximum test load and displacement extracted from Thompson et al. (1999) and Thompson (2004) s’ load and displacement comparison diagrams. Thompson et al. (1999)’s test results of different testing configurations showed significant variation in the load and displacement response of the mesh samples within the same mesh type, bolting spacing, and loading frame dimension. This variation in the load and displacement can be contributed to the different test orientation arrangement of the loading frame and bolt plates in relation to the orientation of the mesh wires.

Table 4.9  The maximum test load and displacement comparisons of different loading frame dimensions with 1.5 x 1.5 m bolting spacing, based on the load and displacement extracted from Thompson et al. (1999) and Thompson (2004).

<table>
<thead>
<tr>
<th>Bolting spacing (m)</th>
<th>Loading frame dimension (mm x mm)</th>
<th>Maximum test load (kN)</th>
<th>Maximum test displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>21.6</td>
<td>329.7</td>
</tr>
<tr>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>32.9</td>
<td>340.1</td>
</tr>
<tr>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>20.5</td>
<td>169.7</td>
</tr>
<tr>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>17.9</td>
<td>180.1</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>23.2 (5.7)</td>
<td>254.9 (80.2)</td>
</tr>
<tr>
<td>1.5 x 1.5</td>
<td>1050 x 1050</td>
<td>33.1</td>
<td>169.7</td>
</tr>
<tr>
<td>1.5 x 1.5</td>
<td>1050 x 1050</td>
<td>19.8</td>
<td>100.1</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>26.5 (6.7)</td>
<td>134.9 (34.8)</td>
</tr>
<tr>
<td>Overall mean (s.d.)²</td>
<td></td>
<td>24.3 (6.2)</td>
<td>214.9 (88.8)</td>
</tr>
</tbody>
</table>

¹ Mean and standard deviation (s.d.)
² Overall mean and standard deviation (s.d.) – for 1.5 x 1.5 m bolting spacing

Table 4.10  The maximum test load and displacement comparisons of different loading frame dimensions with 2.0 x 2.0 m bolting spacing, based on the load and displacement extracted from Thompson et al. (1999) and Thompson (2004).

<table>
<thead>
<tr>
<th>Bolting spacing (m)</th>
<th>Loading frame dimension (mm x mm)</th>
<th>Maximum test load (kN)</th>
<th>Maximum test displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0 x 2.0</td>
<td>750 x 750</td>
<td>25.3</td>
<td>499.4</td>
</tr>
<tr>
<td>2.0 x 2.0</td>
<td>750 x 750</td>
<td>17.4</td>
<td>396.5</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>21.4 (4.0)</td>
<td>448.0 (51.5)</td>
</tr>
<tr>
<td>2.0 x 2.0</td>
<td>1050 x 1050</td>
<td>23.7</td>
<td>300.0</td>
</tr>
<tr>
<td>2.0 x 2.0</td>
<td>1050 x 1050</td>
<td>17.9</td>
<td>340.1</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>20.8 (2.9)</td>
<td>320.1 (20.1)</td>
</tr>
<tr>
<td>Overall mean (s.d.)²</td>
<td></td>
<td>21.1 (3.5)</td>
<td>384.0 (74.9)</td>
</tr>
</tbody>
</table>

¹ Mean and standard deviation (s.d.)
² Overall mean and standard deviation (s.d.) – for 2.0 x 2.0 bolting spacing
Figure 4.10 shows schematic diagrams of different testing arrangements for a 1.5 x 1.5 m bolting spacing and 750 mm loading frame dimension. Table 4.11 summarizes the maximum test load and displacement of the test results for the different testing arrangements. The comparison shows that the welded wire mesh samples that were tested using a diamond bolting pattern (e.g. test configurations C and D) showed significantly lower maximum test displacement in comparison to the mesh samples tested using a square bolting pattern (e.g. test configurations A and B). This observation coincided with the observation from Tannant (1995)’s test results. The comparison also shows that the orientation of the loading frame, in relation to the bolting pattern, can influence the response to loading (e.g. test result with the configuration A shows a lower maximum test load than the test result with the configuration B). However, it was not possible to explore further due to the limited available data.

Table 4.11 Maximum test load and displacement comparison of different test configurations for a 1.5 x 1.5 m bolting spacing and 750 x 750 mm loading frame dimension, based on the test results from Thompson et al. (1999).

<table>
<thead>
<tr>
<th>Testing configuration</th>
<th>Bolting spacing (m)</th>
<th>Loading frame (mm x mm)</th>
<th>Maximum test load (kN)</th>
<th>Maximum test displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>21.6</td>
<td>329.7</td>
</tr>
<tr>
<td>B</td>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>32.9</td>
<td>340.1</td>
</tr>
<tr>
<td>C</td>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>20.5</td>
<td>169.7</td>
</tr>
<tr>
<td>D</td>
<td>1.5 x 1.5</td>
<td>750 x 750</td>
<td>17.9</td>
<td>180.1</td>
</tr>
</tbody>
</table>

Figure 4.10 Different testing configurations for the tests using 1.5 x 1.5 m bolting spacing and 750 x 750 mm loading frame dimensions, after Thompson et al. (1999); after Thompson (2004).
Figure 4.11 compares the maximum test load and displacement from Thompson et al. (1999)’s tests of 5.6 mm diameter welded wire mesh under two different bolting spacing and two different loading frame dimensions. The comparison shows a significant variation in the maximum test load and the displacement of the mesh samples within the same bolting spacing and loading frame dimensions. This variation in the results is caused by different orientations of the loading frame in relation to the orientations of the mesh wires and bolting pattern. The influence of the loading frame orientation on the mesh performance is further discussed in Chapter 6 through a use of numerical modelling.

Figure 4.11 Maximum test load and displacement of 5.6 mm diameter welded wire mesh samples, based on results from Thompson et al. (1999).

4.3.5 National Institute for Occupational Safety and Health (NIOSH)

Dolinar (2006) constructed a static testing rig for mesh (Figure 4.12) in the Mine Roof Simulator (MRS) at the National Institute for Occupational Safety and Health (NIOSH)-Pittsburgh Research Laboratory. The #8-gauge welded steel wire mesh samples with a 102 x 102 mm mesh wire spacing, which are used in coal mines in the United States, were used for most tests.

Figure 4.12 Static testing rig, after Dolinar (2006).
In the test reported by Dolinar (2006), a 305 x 305 mm loading plate dimension was used with rounded-edges to load the mesh sample from the center of the mesh, with a pulling rate of 0.85 mm/s. Four 19 mm diameter reinforcement components were used to pin down the mesh to the steel frame. Two different dimensions (152 x 152 mm or 203 x 203 mm) of flat reinforcement steel plates with the thickness of 9.5 mm were used in the test. 25.4 x 25.4 mm wooden and steel load-bearing surfaces were used under the mesh at all rock bolts. For all tests, a square bolting pattern with a bolting spacing of 1.2 x 1.2 m was used to simulate a typical installation in underground coal mines in the United States (Dolinar 2006). Table 4.12 summarizes the peak load and the displacement of the test results for #8-gauge welded wire mesh samples.

Table 4.12  Peak load and displacement of #8-gauge welded wire mesh samples using a 1.2 x 1.2 m bolting spacing, based on the test results from Dolinar (2006).

<table>
<thead>
<tr>
<th>Bearing surface</th>
<th>Reinforcement plate dimension (mm x mm)</th>
<th>Bolt tension load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>152.4 x 152.4</td>
<td>25.6</td>
<td>12.4</td>
<td>472.4</td>
</tr>
<tr>
<td>Steel</td>
<td>152.4 x 152.4</td>
<td>38.4</td>
<td>12.9</td>
<td>462.3</td>
</tr>
<tr>
<td>Wood</td>
<td>152.4 x 152.4</td>
<td>25.6</td>
<td>14.6</td>
<td>487.7</td>
</tr>
<tr>
<td>Wood</td>
<td>152.4 x 152.4</td>
<td>38.4</td>
<td>15.9</td>
<td>424.2</td>
</tr>
<tr>
<td>Wood</td>
<td>152.4 x 152.4</td>
<td>51.1</td>
<td>9.3</td>
<td>287.0</td>
</tr>
<tr>
<td><strong>Mean (s.d.)</strong></td>
<td></td>
<td><strong>13.0 (2.2)</strong></td>
<td><strong>426.7 (72.9)</strong></td>
<td></td>
</tr>
<tr>
<td>Wood</td>
<td>203.2 x 203.2</td>
<td>25.6</td>
<td>24.3</td>
<td>500.4</td>
</tr>
<tr>
<td>Wood</td>
<td>203.2 x 203.2</td>
<td>38.4</td>
<td>24.3</td>
<td>482.6</td>
</tr>
<tr>
<td>Wood</td>
<td>203.2 x 203.2</td>
<td>51.1</td>
<td>10.9</td>
<td>274.3</td>
</tr>
<tr>
<td><strong>Mean (s.d.)</strong></td>
<td></td>
<td><strong>19.9 (6.4)</strong></td>
<td><strong>419.1 (102.6)</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Overall mean (s.d.)</strong></td>
<td></td>
<td><strong>15.6 (5.4)</strong></td>
<td><strong>423.9 (85.4)</strong></td>
<td></td>
</tr>
</tbody>
</table>

1 Mean and standard deviation (s.d.)
2 Overall mean and standard deviation (s.d.) – for 1.2 x 1.2 m bolting spacing

Dolinar (2006) defined the peak load as “the maximum load just prior to a significant drop in load” or “initial peak load”. This corresponds to the working capacity defined in Section 4.2. Dolinar (2006) defined the “yield load” as the transition point where there is a significant change from the elastic behavior to a plastic behavior, as shown in Figure 4.13.

The definition of “screen stiffness” was defined based on the slope of a line drawn from the peak load to the 25 % of the peak load (Dolinar 2006). The “displacement offset” was defined as the amount of deformation required for the mesh to start producing resistance against the applied load (Dolinar 2006).

Dolinar (2006) observed that the dominant failure mode of the mesh was at the weld and that 98 % of the 104 wire failures occurred along the eight wires that are directly loaded with the reinforcement plates.
Figure 4.13  Load and displacement of welded wire mesh showing key parameters used to evaluate mesh performance, based on the test results from Dolinar (2006).

For the square bolting pattern, the applied load transferred from the cross-wires of the center load plate to the eight wires that are directly restrained under the reinforcement plates, shown in Figure 4.14. Therefore, the eight wires directly restrained under the reinforcement plates were critical for the stability of mesh (Dolinar 2006).

Figure 4.14  Schematic diagram of square bolting configuration of mesh showing the directions of load transfers on the mesh wires, after Dolinar (2006).

Dolinar (2006) noted that the “saw-toothed” behavior after the yield point in the loading response was caused either by the failure of one of the eight wires or the slippage of the wires under the reinforcement
plate. Dolinar (2006) stated that with the absence of wire slippage, the peak load will be less because the wire breakage will occur earlier on. Dolinar (2009) provided additional peak load and displacement values from the tests of welded wire mesh samples, summarized in Table 4.13 and Table 4.14.

**Table 4.13** Peak load and displacement of welded wire mesh samples using a bolting spacing of 1.4 x 1.4 m, based on the test results from Dolinar (2009).

<table>
<thead>
<tr>
<th>Mesh wire gauge</th>
<th>Bolt tension load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>111.2</td>
<td>7.7</td>
<td>327.7</td>
</tr>
<tr>
<td>9</td>
<td>111.2</td>
<td>8.8</td>
<td>363.2</td>
</tr>
</tbody>
</table>

**Table 4.14** Peak load and displacement of welded wire mesh samples using a bolting spacing of 1.2 x 1.2 m, based on the test results from Dolinar (2009).

<table>
<thead>
<tr>
<th>Mesh wire gauge</th>
<th>Bolt tension load (kN)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>20.0</td>
<td>16.1</td>
<td>525.8</td>
</tr>
<tr>
<td>4</td>
<td>35.6</td>
<td>21.6</td>
<td>472.4</td>
</tr>
<tr>
<td>4</td>
<td>66.7</td>
<td>29.3</td>
<td>490.2</td>
</tr>
<tr>
<td>4</td>
<td>111.2</td>
<td>23.5</td>
<td>381.0</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td>58.4</td>
<td>22.6 (4.7)</td>
<td>467.4 (53.4)</td>
</tr>
<tr>
<td>6</td>
<td>20.0</td>
<td>14.0</td>
<td>495.3</td>
</tr>
<tr>
<td>6</td>
<td>35.6</td>
<td>19.2</td>
<td>482.6</td>
</tr>
<tr>
<td>6</td>
<td>66.7</td>
<td>22.2</td>
<td>408.9</td>
</tr>
<tr>
<td>6</td>
<td>111.2</td>
<td>22.0</td>
<td>358.1</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td>58.4</td>
<td>19.4 (3.3)</td>
<td>436.2 (55.9)</td>
</tr>
<tr>
<td>8</td>
<td>20.0</td>
<td>9.3</td>
<td>492.8</td>
</tr>
<tr>
<td>8</td>
<td>35.6</td>
<td>11.7</td>
<td>370.8</td>
</tr>
<tr>
<td>8</td>
<td>66.7</td>
<td>10.6</td>
<td>345.4</td>
</tr>
<tr>
<td>8</td>
<td>111.2</td>
<td>9.0</td>
<td>284.5</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td>58.4</td>
<td>10.2 (1.1)</td>
<td>373.4 (75.7)</td>
</tr>
<tr>
<td>9</td>
<td>20.0</td>
<td>9.1</td>
<td>485.1</td>
</tr>
<tr>
<td>9</td>
<td>35.6</td>
<td>10.9</td>
<td>378.5</td>
</tr>
<tr>
<td>9</td>
<td>66.7</td>
<td>9.5</td>
<td>355.6</td>
</tr>
<tr>
<td>9</td>
<td>111.2</td>
<td>8.4</td>
<td>337.8</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td>58.4</td>
<td>9.5 (0.9)</td>
<td>389.3 (57.2)</td>
</tr>
<tr>
<td>10</td>
<td>20.0</td>
<td>7.8</td>
<td>447.0</td>
</tr>
<tr>
<td>10</td>
<td>35.6</td>
<td>9.0</td>
<td>340.4</td>
</tr>
<tr>
<td>10</td>
<td>66.7</td>
<td>5.8</td>
<td>294.6</td>
</tr>
<tr>
<td>10</td>
<td>111.2</td>
<td>5.2</td>
<td>279.4</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td>58.4</td>
<td>7.0 (1.5)</td>
<td>340.4 (65.5)</td>
</tr>
<tr>
<td>Overall mean (s.d.)²</td>
<td>13.7 (6.7)</td>
<td>401.3 (76.8)</td>
<td></td>
</tr>
</tbody>
</table>

¹ Mean and standard deviation (s.d.)

² Overall mean and standard deviation (s.d.) – for 1.2 x 1.2 m bolting spacing
Figure 4.15 compares the peak load and the displacement from Dolinar (2009)’s tests of welded wire mesh using a 1.2 x 1.2 m bolting spacing. The comparison shows a clear trend of higher peak loads for the tested mesh samples with a lower wire gauge (i.e. thicker wire).

![Graph showing peak load and displacement for different wire gauges](image)

**Figure 4.15** Peak load and displacement of welded wire mesh samples using a 1.2 x 1.2 m bolting spacing, based on the test results from Dolinar (2009).

This observation is consistent with other results stated by Pakalnis & Ames (1987)’s and Tannant (1995). There is, however, a significant variation in the peak load and the displacement of the tested mesh samples using the same wire gauge. This difference was caused by a variation in bolt tension load. In general, the mesh sample tested with higher bolt tension load resulted in lower peak displacement.

### 4.3.6 WASM

Morton et al. (2007) described a large-scale static testing rig for mesh built by the Western Australian School of Mines (WASM). The testing rig consisted of a reaction beam and a frame to support the wire mesh sample. A 300 x 300 mm, 35 mm thick hardened steel plate with a spherical seat was used to apply the load to the center of the mesh with a loading rate of 0.067 mm/s. Two different types of boundary constraints were used: lacing and shackle. The lacing boundary condition involved using a 6-mm wire rope to lace the edge wires of the mesh to the steel frame. Morton (2009) stated that the rope was tensioned manually, and the pre-tension load was “dependent on the type of mesh and the number of people assisting in the sample setup.” The initial tensions applied with the rope were not consistent and influenced the amount of displacement which occurred prior to the mesh resisting the load. The shackle boundary condition involved clamping the edge wires of the mesh with a high tensile bar, eye nuts, and D shackles passing through steel frame (Morton 2009). Two types of mesh that were tested: welded wire mesh and chain link mesh. The welded wire mesh used 5.6 mm galvanized steel wires with 100 x 100 mm wire
spacing. The chain link mesh used 4 mm high tensile wires. For both mesh types, the sheet size of 1.3 x 1.3 m was used.

A total of ten tests were conducted using the lacing boundary condition (eight tests using the standard welded mesh, and two tests using the chain link mesh), and thirteen tests were conducted using the fixed boundary condition (ten tests using the standard welded mesh, and three tests using chain link mesh). Table 4.15 and Table 4.16 summarize the rupture, the peak load, and the displacement from Morton et al. (2007)’s tests of welded wire mesh and chain link mesh samples.

Table 4.15  Rupture and peak load and displacement of welded wire mesh samples, based on the test results from Morton et al. (2007).

<table>
<thead>
<tr>
<th>Mesh type</th>
<th>Boundary condition</th>
<th>Rupture load (kN)</th>
<th>Rupture displacement (mm)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>46.2</td>
<td>242</td>
<td>46.2</td>
<td>242</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>46.7</td>
<td>249</td>
<td>46.7</td>
<td>249</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>35.9</td>
<td>222</td>
<td>38.9</td>
<td>241</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>34.1</td>
<td>209</td>
<td>38.4</td>
<td>253</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>28.1</td>
<td>216</td>
<td>41.4</td>
<td>236</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>44.0</td>
<td>228</td>
<td>44.0</td>
<td>228</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>45.4</td>
<td>239</td>
<td>45.4</td>
<td>239</td>
</tr>
<tr>
<td>Welded</td>
<td>Lacing</td>
<td>33.4</td>
<td>222</td>
<td>35.4</td>
<td>249</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>39.2 (6.7)</td>
<td>228 (13)</td>
<td>42.1 (3.9)</td>
<td>242 (8)</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>45.0</td>
<td>173</td>
<td>45.0</td>
<td>173</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>44.1</td>
<td>192</td>
<td>44.1</td>
<td>192</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>37.9</td>
<td>151</td>
<td>37.9</td>
<td>151</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>40.9</td>
<td>182</td>
<td>43.4</td>
<td>195</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>38.5</td>
<td>150</td>
<td>38.5</td>
<td>150</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>46.4</td>
<td>181</td>
<td>46.4</td>
<td>181</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>44.9</td>
<td>188</td>
<td>44.9</td>
<td>188</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>40.7</td>
<td>195</td>
<td>40.7</td>
<td>195</td>
</tr>
<tr>
<td>Welded</td>
<td>Fixed</td>
<td>29.7</td>
<td>209</td>
<td>29.7</td>
<td>209</td>
</tr>
<tr>
<td>Mean (s.d.)¹</td>
<td></td>
<td>40.9 (4.9)</td>
<td>180 (19)</td>
<td>41.2 (4.9)</td>
<td>182 (19)</td>
</tr>
</tbody>
</table>

¹ Mean and standard deviation (s.d.)

The test results demonstrated that the chain link mesh samples showed a higher peak load and displacement than the welded mesh samples. Mesh retained by shackle showed lower peak displacements than the lacing boundary condition. For the lacing boundary condition, more displacement occurred prior to the mesh responding to the load. Most of the welded wire mesh samples failed either through the weld or in the heat affected zone. Furthermore, the failures occurred along the mesh boundary where the wires were under direct loading. The failure of the wires continued to alternate between two sides of mesh along the boundary.
In contrast, the chain link mesh samples failed on the edge of the loading plate. It was noted that the load dropped completely due to plate movement leading up to the initial failure.

**Table 4.16** Rupture and peak load and displacement of high tensile wire chain link mesh samples, based on the test results from Morton et al. (2007).

<table>
<thead>
<tr>
<th>Mesh type</th>
<th>Boundary condition</th>
<th>Rupture load (kN)</th>
<th>Rupture displacement (mm)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chain link</td>
<td>Lacing</td>
<td>137.1</td>
<td>343</td>
<td>137.1</td>
<td>343</td>
</tr>
<tr>
<td>Chain link</td>
<td>Lacing</td>
<td>120.8</td>
<td>311</td>
<td>120.8</td>
<td>311</td>
</tr>
<tr>
<td>Mean (s.d.)</td>
<td></td>
<td><strong>129.0 (8.2)</strong></td>
<td><strong>327 (16)</strong></td>
<td><strong>129.0 (8.2)</strong></td>
<td><strong>327 (16)</strong></td>
</tr>
<tr>
<td>Chain link</td>
<td>Fixed</td>
<td>145.6</td>
<td>310</td>
<td>145.6</td>
<td>310</td>
</tr>
<tr>
<td>Chain link</td>
<td>Fixed</td>
<td>155.0</td>
<td>285</td>
<td>155.0</td>
<td>285</td>
</tr>
<tr>
<td>Chain link</td>
<td>Fixed</td>
<td>147.2</td>
<td>292</td>
<td>147.2</td>
<td>292.3</td>
</tr>
<tr>
<td>Mean (s.d.)</td>
<td></td>
<td><strong>149.3 (4.1)</strong></td>
<td><strong>296 (11)</strong></td>
<td><strong>149.3 (4.1)</strong></td>
<td><strong>295.8 (10.5)</strong></td>
</tr>
</tbody>
</table>

Mean and standard deviation (s.d.)

Figure 4.16 compares the peak load and displacement from Morton et al. (2007)’s tests of welded wire mesh and chain link mesh samples. Chain link mesh samples provided significantly higher peak load and displacement when compared to the welded wire mesh. The comparison also shows that mesh samples tested with the lacing boundary condition provided higher peak displacement when compared to the mesh samples tested with the fixed boundary with D shackles. In general, the peak load did not vary much between the two different boundary conditions.

![Figure 4.16](image_url)  
**Figure 4.16** Peak load and displacement of welded wire mesh and chain link mesh, based on the test results from Morton et al. (2007).

Morton et al. (2007)’s tests of welded wire mesh and chain link mesh illustrate the importance of boundary condition of the testing rig on the mesh performance. However, Morton et al. (2007)’s peripheral boundary
conditions do not necessarily reflect actual constraint condition in underground mines where mesh is constrained by rock bolts.

### 4.3.7 Wollongong

Shan et al. (2014) conducted a full-scale pull test (Figure 4.17) on two types of welded steel mesh in a laboratory setting. A dome plate was used to pull up with a pulling rate of 0.4 mm/s to simulate static loading. The load was measured by a 100 kN load cell and the displacement was measured by the Linear Variable Differential Transducer (LVDT). Two types of mesh were tested.

The Type A mesh, representing the roof mesh, consisted of a 1.35 x 3.6 m mesh section constructed with 5 mm diameter steel wires (both transverse and longitudinal). The diameter of the longitudinal reinforcing wires under the reinforcement plates was increased to 7 mm (Shan et al. 2014). The Type B mesh, representing the rib mesh, consisted of a 1.5 x 4.0 m mesh section constructed with 4 mm diameter steel wires (both transverse and longitudinal) without the reinforcing wires (Shan et al. 2014).

In both cases, a square bolting pattern was used with 1.0 x 1.0 m bolting spacing. The mesh was bolted down to the “strong floor” with reinforcement components pre-tensioned to 240 Nm, in order to limit the slippage of the mesh (Shan et al. 2014).

![Figure 4.17 Full-scale test setup, after Shan et al. (2014).](image-url)

For the mesh samples, four Type A and one Type B samples were tested. Figure 4.18 compares the load and displacement from Shan et al. (2014) tests of welded wire mesh. The smaller drops in the load, or the “saw tooth” on the load-displacement curve were caused by the slippage of the mesh under the
reinforcement plates, while the significant drop at the peak load was caused by wire failure (Shan et al. 2014).

![Diagram of load and displacement pull tests of welded wire mesh samples](image)

**Figure 4.18 Load and displacement pull tests of welded wire mesh samples, after Shan et al. (2014).**

The results showed that Type A mesh, with a larger wire diameter produced a larger peak load (48 kN) instead of 21 kN for Type B mesh. This corresponds to the higher strength of wire from the three-point bend tests of single wires. Shan et al. (2014) noted that the difference in the peak displacement was small. For the Type A mesh, all of the wire failure occurred near the loading dome. In contrast, all of the wire failures occurred near the reinforcement plates for the Type B mesh. This suggests that the additional wires at the reinforcement plates allowed for the higher load to be taken by the mesh. Shan et al. (2014) also conducted a set of numerical modelling experiments to simulate laboratory test results. These are discussed in Chapter 5.

### 4.3.8 CSIR

There are other static testing rigs constructed and used to test mesh performance. Watson et al. (2017) conducted a new laboratory testing rig to investigate the performance of welded wire mesh. The pyramidal-shaped loading frame was used to represent “a block of rock that has rotated and is loading the support element along its edge” (Watson et al. 2017). Watson et al. (2017) stated that this testing configuration gives “a more realistic laboratory test”. This was different from the previous testing rigs, which used a flat or domed steel loading plate.

### 4.3.9 IGO

Whiting (2017) conducted in-situ static testing of two different welded wire mesh configurations using a hydraulic loading mechanism developed on-site. The different welded wire mesh configurations included:
a standard mesh (5.6 mm wire diameter, and 100 x 100 mm wire spacing) and a “seismic mesh” (mesh sheet with 5.6 mm wire diameter and 100 x 100 mm wire spacing mesh spacing with additional double 6.3 mm diameter wires at bolt/plate locations). A 300 x 280 mm steel plate was used to pull the mesh at various locations on mesh. The test result, as summarized in Table 4.17, showed that the seismic mesh configuration typically provided 24 % greater average peak load than the standard mesh. This is similar with Shan et al. (2014)’s the test results of the Type A welded wire mesh samples which also used additional reinforcing wires at the bolt/plate locations.

Table 4.17 Rupture and peak load and displacement of two different welded wire mesh configurations, based on the in-situ test results from Whiting (2017).

<table>
<thead>
<tr>
<th>Mesh type</th>
<th>First strand rupture load (kN)</th>
<th>First strand rupture displacement (mm)</th>
<th>Peak load (kN)</th>
<th>Peak displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard mesh</td>
<td>16-54</td>
<td>250-300</td>
<td>30-62</td>
<td>400-500</td>
</tr>
<tr>
<td>Seismic mesh</td>
<td>32-54</td>
<td>250-350</td>
<td>44-67</td>
<td>380-500</td>
</tr>
</tbody>
</table>

4.3.10 NIOSH

Batchler et al. (2018) constructed a laboratory static pull test of a 2.4 x 3.6 m, #8-gauge welded wire mesh. The mesh was retrained on the steel frame with a 1.22 x 1.22 m square bolting pattern and was simultaneously loaded at six loading locations. During the laboratory tests, seven different test scenarios were examined, including loading rate (1.3, 5.1, and 12.7 cm/min) and loading surface geometry (flat steel plate or bell). Batchler et al. (2018) experiments showed that different amount and type of “load shed events” (e.g. wire slip, weld break, or wire break) were observed, depending on the loading rate applied on the mesh. However, Villaescusa (1999) stated that the variation on the type of mesh failure can also be associated with the mesh quality. Despite the fact that some observations on the mesh performance can be made from one particular laboratory test results, it is yet challenging to truly grasp which parameter in fact impacted the performance of the mesh.

4.3.11 Discussion on the Testing Rigs

The review of the testing rigs demonstrated a wide variation in the testing rig configurations and mesh configurations. Depending on a combination of different testing rig configuration and mesh configuration, the measured mesh performance (through load and displacement) varied significantly. Figure 4.19 and Figure 4.20 compare the maximum test load and the displacement plotted against wire diameter of welded wire mesh of various testing rigs.
Figure 4.19 Maximum load plotted against the mesh wire diameter of welded wire mesh from multiple testing rigs.

Figure 4.20 Maximum displacement plotted against the mesh wire diameter of welded wire mesh tested from various testing rigs.

Wire diameter was selected over wire gauge, as the unit of comparison of the load and displacement of different testing rigs, because the corresponding wire diameter to specified wire gauge varied slightly depending on the region (e.g. Australia or North America). The diagrams show that even within the mesh samples, with similar wire diameters, the maximum test load and displacement can vary. This variation can
be associated with either the loading plate or the frame dimension, the bolting pattern (diamond or square), the bolting spacing, and the boundary condition (peripheral or rock bolt and plates), etc.

Figure 4.21 and Figure 4.22 compare the maximum test load and displacement plotted against the wire diameter of chain link mesh testing rigs. Similar to the welded wire mesh, there are significant variations within similar wire diameters of the chain link mesh samples tested.

**Figure 4.21** Maximum load plotted against the mesh wire diameter of chain link mesh from various testing rigs.

**Figure 4.22** Maximum displacement plotted against the mesh wire diameter of chain link mesh from various testing rigs.
Table 4.18 presents a list of static testing facilities. The short list provides a glimpse of the significant endeavor to quantify the performance mesh around the world. Despite these hand-full existence of testing rigs, the absence of a universal standardized way of testing and reporting of test data exist.

**Table 4.18 Static testing rigs.**

<table>
<thead>
<tr>
<th>Testing rig #</th>
<th>Rig type</th>
<th>Ownership</th>
<th>Testing facility</th>
<th>Location</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Static</td>
<td>Rand Mines Limited</td>
<td>--</td>
<td>Johannesburg, South Africa</td>
<td>Ortlepp (1983)</td>
</tr>
<tr>
<td>2</td>
<td>Static</td>
<td>Laurentian University</td>
<td>Geomechanics Research Centre (GRC)</td>
<td>Sudbury, Canada</td>
<td>Tannant et al. (1995), and Tannant (2004)</td>
</tr>
<tr>
<td>3</td>
<td>Static</td>
<td>WMC Resources Limited – St Ives Gold</td>
<td>Junction Gold Mine, Kambalda</td>
<td>Western Australia</td>
<td>Thompson et al. (1999), and Thompson (2004)</td>
</tr>
<tr>
<td>4</td>
<td>Static</td>
<td>The National Institute for Occupational Safety and Health</td>
<td>NIOSH-Pittsburgh Research Laboratory (PRL)</td>
<td>Pittsburgh, PA, United States</td>
<td>Dolinar (2006), Dolinar (2009), and Batchler et al. (2018)</td>
</tr>
<tr>
<td>5</td>
<td>Static</td>
<td>Western Australian School of Mines</td>
<td>WASM Static Test facility</td>
<td>Kalgoorlie, Western Australia</td>
<td>Morton et al. (2007), and Morton (2009)</td>
</tr>
<tr>
<td>6</td>
<td>Static</td>
<td>University of Wollongong</td>
<td>Rail and Pavement Laboratory</td>
<td>New South Wales, Australia</td>
<td>Shan et al. (2014)</td>
</tr>
</tbody>
</table>

Table 4.19 compares the boundary condition of various static testing rigs. Based on this limited data, the boundary dimension ranged from 1.1 x 1.1 m to 2.0 x 2.0 m. The most testing rigs used the boundary dimensions of 1.0 x 1.0 m and 1.2 x 1.2 m.

Table 4.20 compares the loading condition of various static testing rigs. It shows that the loading plate dimension ranged from 300 x 300 mm to 1050 x 1050 mm. Static loading rates also ranged from 0.07 to 22 mm/s. The most testing rigs used 300 x 300 mm and 305 x 305 mm loading plate dimensions.

Table 4.21 compares the type of mesh tested in various testing rigs. The typical mesh aperture used during the tests ranged from 100 x 100 mm to 102 x 102 mm, mesh dimension ranged from 1.3 x 1.3 m to 3.2 x 3.2 m, and wire gauge ranged from #4 to #12. Testing rigs in Australia mainly used #5-gauge (5.6 mm diameter) wire mesh, while a range of wire gauges were used by the testing rigs in North America. However, this is an incomplete database as several suppliers undertake in-house tests and are not reported to the public.
Table 4.19 Boundary condition of various static testing rigs.

<table>
<thead>
<tr>
<th>Testing rig #</th>
<th>Boundary type</th>
<th>Window/bolting pattern dimension (m x m)</th>
<th>Frame/bolt type</th>
<th>Bolt plate &amp; washer</th>
<th>Bolt pre-tensioned load (kN)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Peripheral clamping</td>
<td>1.1 x 1.1</td>
<td>Heavy steel frame</td>
<td>--</td>
<td>--</td>
<td>Ortlepp (1983)</td>
</tr>
<tr>
<td>2</td>
<td>Diamond and square bolting</td>
<td>1.2 x 1.2, and 1.2 x 1.5</td>
<td>19 mm dia.</td>
<td>9.5 mm thick, 127 x 127 mm flat steel plate</td>
<td>10.0 to 29.9</td>
<td>Tannant et al. (1995), and Tannant (2004)</td>
</tr>
<tr>
<td>3</td>
<td>Diamond and square bolting</td>
<td>1.0 x 1.0, 1.5 x 1.5, 1.5 x 2.0, and 2.0 x 2.0</td>
<td>M20 anchor bolts</td>
<td>10 mm thick, 200 x 200 mm square steel plate</td>
<td>~ 45</td>
<td>Thompson et al. (1999), and Thompson (2004)</td>
</tr>
<tr>
<td>4</td>
<td>Square bolting</td>
<td>1.2 x 1.2</td>
<td>19.05 mm dia. bolt</td>
<td>Flat grade 4, 9.53 mm thickness 152 x 152 mm or 203 x 203 mm with wooden or without washer</td>
<td>25.9 to 51.8</td>
<td>Dolinar (2006), Dolinar (2009), and Batchler et al. (2018)</td>
</tr>
<tr>
<td>5</td>
<td>Peripheral clamping, lacing, and fixed shackle</td>
<td>1.3 x 1.3</td>
<td>Steel frame fixed with shackles</td>
<td>high tensile threaded bar, eye nuts and “D” shackles</td>
<td>--</td>
<td>Morton et al. (2007), and Morton (2009)</td>
</tr>
<tr>
<td>6</td>
<td>Square bolting</td>
<td>1.0 x 1.0</td>
<td>--</td>
<td>--</td>
<td>240 Nm torque</td>
<td>Shan et al. (2014)</td>
</tr>
<tr>
<td>Testing rig #</td>
<td>Loading plate/frame</td>
<td>Dimension (mm)</td>
<td>Location</td>
<td>Direction of motion</td>
<td>Loading rate (mm/s)</td>
<td>Measuring accuracy</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------------------------------------------------------------------------------</td>
<td>----------------</td>
<td>----------</td>
<td>---------------------</td>
<td>--------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>1</td>
<td>Articulated arrangement of four steel plates</td>
<td>420 x 520</td>
<td>Centre</td>
<td>Pushed down</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>13 mm thick square flat steel plate with rounded edge</td>
<td>300 x 300</td>
<td>Centre</td>
<td>Pulled up</td>
<td>18 – 22</td>
<td>±0.2 kN and ±0.6 mm</td>
</tr>
<tr>
<td>3</td>
<td>Square steel angle section loading frame</td>
<td>750 x 750, 1050 x 1050</td>
<td>Centre</td>
<td>Pulled up</td>
<td>“increments of 5 mm or 10 mm”</td>
<td>--</td>
</tr>
<tr>
<td>4</td>
<td>Square flat plate with rounded edge</td>
<td>305 x 305</td>
<td>Centre</td>
<td>Pulled up</td>
<td>0.85</td>
<td>±0.09 kN and ±0.25 mm</td>
</tr>
<tr>
<td>5</td>
<td>35 mm thick, flat hardened square steel plate and Curved based hardened steel plate</td>
<td>300 x 300</td>
<td>Centre</td>
<td>Pulled up</td>
<td>0.07</td>
<td>--</td>
</tr>
<tr>
<td>6</td>
<td>Dome plate</td>
<td>--</td>
<td>Centre</td>
<td>Pulled up</td>
<td>0.4</td>
<td>±0.2 kN and ±0.6 mm</td>
</tr>
</tbody>
</table>
Table 4.21 Summary comparison table of welded wire mesh samples of various testing rigs.

<table>
<thead>
<tr>
<th>Testing rig #</th>
<th>Type of mesh</th>
<th>Typical aperture (mm)</th>
<th>Mesh dimension (m)</th>
<th>Nominal wire gauge (diameter in mm)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Welded mesh</td>
<td>100</td>
<td>3.2 x 3.2</td>
<td>--</td>
<td>Ortlepp (1983)</td>
</tr>
<tr>
<td>2</td>
<td>Steel welded wire</td>
<td>102</td>
<td>1.4 x 1.4, and 1.4 x 1.7</td>
<td>#9 (3.77), #6 (4.88), #4 (5.72)</td>
<td>Tannant et al. (1995), and Tannant (2004)</td>
</tr>
<tr>
<td>3</td>
<td>Steel welded wire</td>
<td>100</td>
<td>2.4 x 3.0 to 6.0</td>
<td>#5 (5.6)</td>
<td>Thompson et al. (1999), and Thompson (2004)</td>
</tr>
<tr>
<td>4</td>
<td>Steel welded wire</td>
<td>102</td>
<td>1.5 x 1.5</td>
<td>#8 (4.11)</td>
<td>Dolinar (2006), Dolinar (2009), and Batchler et al. (2018)</td>
</tr>
<tr>
<td>5</td>
<td>Steel welded wire</td>
<td>100</td>
<td>1.3 x 1.3</td>
<td>#5 (5.6)</td>
<td>Morton et al. (2007), and Morton (2009)</td>
</tr>
<tr>
<td>6</td>
<td>Steel welded wire</td>
<td>--</td>
<td>1.36 x 3.6, and 1.5 x 4.0</td>
<td>5 mm with 7 mm reinforcing wires, 4 mm</td>
<td>Shan et al. (2014)</td>
</tr>
</tbody>
</table>

4.4 Summary

Several testing rigs were constructed at different times to quantify the load and the displacement capacities of mesh. While each testing rig provided some quantifiable results on the load and the displacement capacities of mesh, the absence of standard in the test configurations led to varying results and made it difficult to effectively compare the testing rigs. It is noted that all laboratory tests use a vertical load or push, using regular shape loading plates. As the field observations demonstrated the actual loading mechanisms are more complex. Furthermore, it was difficult to compare the performance of different types of mesh as there were a number of mesh and testing parameters that were not used consistently across the testing rigs (e.g. mesh wire spacing, type of steel, etc.). Tannant (1995)’s and Thompson et al. (1999)’s investigations on the influence of different bolting spacing, bolting pattern, orientation of loading frame in relation to the mesh wires and bolts showed significant difference in the loading response of the welded wire mesh. The recent works by Shan et al. (2014), Watson et al. (2017), Whiting (2017), and Batchler et al. (2018) showed different attempts to develop a laboratory testing configuration that provide a more realistic representation of the actual ground condition in underground mines.
The detailed review of various laboratory tests of mesh illustrated that the mesh behavior is determined by several factors. However, it is difficult to quantify the impact the different parameters have on the mesh performance. Numerical modelling can be a more robust tool in examining the mesh performance under different testing conditions. The next chapter discusses the use of numerical modelling to investigate the influence of testing parameters on the mesh performance.
Chapter 5

5  Numerical Modelling of a Laboratory Test of Welded Steel Wire Mesh

5.1 Introduction

In Chapter 4, a comprehensive review of various laboratory testing rigs used for testing mesh was undertaken. Laboratory tests can contribute to an understanding of the relative mesh performance under given testing rig and mesh configurations. However, it was evident that there were considerable variations in the testing rigs and the mesh configurations used in each test. Quite often each individual test captured a single loading mechanism and a specific boundary condition. Consequently, this makes the direct comparison of the test results and the extrapolation of the in-situ mesh performance difficult.

This chapter illustrates how numerical modelling can provide a tool to examine the impact of different testing parameters on mesh performance. To these purposes, this chapter explores whether numerical modelling can in fact reproduce the failure mechanism, load transfer, and the load-displacement response of a static laboratory testing configuration of mesh. This can subsequently lead to further numerical experiments which can identify the interaction of testing parameters and support performance. This chapter reviews the historical use of numerical modelling on mesh, as well as the test configuration and test results from one particular laboratory testing rig, and describes the numerical models developed to reproduce the mechanical behaviour of mesh as observed in the laboratory test.

5.2 Numerical Modelling

The concept of using numerical modelling for ground support for underground mines is not new (Lorig & Varona 2013). In order to have confidence in the results, the numerical model should simulate the observed mechanical behaviour and failure mechanism of the ground support under a particular ground condition. However, in general, when modelling ground support, the mesh is rarely accounted in numerical models.

Gadde et al. (2006) examined the use of Itasca’s Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D) numerical modelling to investigate the behaviour of welded wire mesh. Gadde et al. (2006) used elastic-perfectly plastic material behaviour on large strain mode. The mesh geometry was constructed using a network of beam structural elements and pile elements. The steel properties used in the model are listed in Table 5.1.
Table 5.1  Mesh properties used in FLAC3D model, after Gadde et al. (2006).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (psi)</td>
<td>$29 \times 10^6$</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Yield strength (ksi)</td>
<td>65</td>
</tr>
<tr>
<td>Strain at the point of failure (%)</td>
<td>10</td>
</tr>
<tr>
<td>Cross-sectional area and moment of inertia</td>
<td>Wire gauge dependent</td>
</tr>
</tbody>
</table>

Gadde et al. (2006) validated the numerical modelling method by comparing the modelling results with the laboratory test data from two different sources. One of the data was obtained from the personal communication with Dolinar in 2006, which tested a 5 x 5 ft, #8-gauge wire mesh with 4 x 4 inch mesh apertures (Gadde et al. 2006). In the laboratory test, the fasteners, as shown in Figure 5.1, were used to attach the mesh to bearing plates.

![Wire mesh and fasteners](image)

**Figure 5.1  An example of wire fasteners, after Gadde et al. (2006).**

Gadde et al. (2006) suggested that the use of fasteners allowed for the exact known number of wires to be restrained under the bearing plate and restricted the slipping of wires, similar to the assumption made in the numerical model. Figure 5.2 shows the load and displacement comparison diagram of the numerical result and the actual test results. It shows that the numerical model ran until the “monotonic increase in the load with deformation” ended (Gadde et al. 2006).

Gadde et al. (2006) suggested that this was the point where the “mesh has suffered bagging failure” or “15 inches” of mesh deformation. The complexity associated with the failure mechanism beyond this point limited the modelling from continuing further.
The second set of data was obtained from Tannant (1995). Gadde et al. (2006) used the #9- and #6-wire gauges for comparison purposes. Figure 5.3 compares the load and displacement of the modelling and actual laboratory test result. Gadde et al. (2006)’s numerical results showed reasonable fit to Tannant (1995)’s actual laboratory test data.

Subsequent to the verification process of the modelling approach, Gadde et al. (2006) simulated a small-size and a full-size welded wire mesh as shown in Figure 5.4. The model showed that the majority of rock load is carried by the wires passing under the bearing plates with very little axial load seen in the rest of the wires. Furthermore, the model showed a good match to load displacement curves from laboratory results.
Parametric studies were conducted using a “few common input design variables”, which are the wire gauges, the height of rock load, the distribution of rock load, and the number of wires fixed under the bearing plates. The result of the parametric studies showed that “a higher mesh stiffness and load capacity could be obtained by ensuring the maximum possible number of wires are held tight by the plates”.

**Figure 5.4.** Load-displacement distributions in (a) small- and (b) full-size sheet for identical rock loads, after Gadde et al. (2006).

Shan et al. (2014) investigated the performance of welded steel mesh in underground coal mines through full-scale laboratory tests and numerical modelling using FLAC3D. First, the three-point bend tests on two different diameters of two individual wires were conducted and simulated using numerical model. The individual wires were simulated using the beam structural element in FLAC3D. Figure 5.5 shows the load and displacement comparison diagram of the numerical and laboratory test results. The diagrams showed a close match of the load and displacement response of the laboratory test results with the numerical results.

**Figure 5.5.** Load and displacement graph of laboratory and numerical test results of three-point bend test of the 5 mm and 7 mm diameter steel wires, after Shan et al. (2014).

Consequently, a full-scale laboratory test of mesh was numerically modelled (Figure 5.6). A network of beam elements was created by linking the beam elements at each node. Domed pull plate was simulated by
increasing the number of loading nodes as the displacement increased (Shan et al. 2014). The simulation was stopped when the first wire breakage occurred (Shan et al. 2014).

![FLAC3D 5.00](image)

**Figure 5.6.** Numerical test results: (a) FLAC3D numerical modelling, and (b) the load and displacement curve comparing the numerical and laboratory test results, after Shan et al. (2014)

The results showed that the load at the first wire failure matched closely to the laboratory test. However, Shan et al. (2014) noted that the stiffness of the curve from the numerical modelling was greater than the stiffness of the curve in the laboratory test, since the model did not allow for the slippage of mesh.

Bertrand et al. (2008) discussed the numerical simulation of dynamic loading on double-twisted hexagonal mesh in the context of rockfall-protection in civil engineering using the Discrete Element Method (DEM). For this purpose, Bertrand et al. (2008) used Itasca’s PFC3D to simulate mesh using remote interactions between spherical particles. Figure 5.7 illustrates that spherical particles can be used to generate load on mesh. This work, however, does not represent impact loading observed in underground mines.

![Numerical representations of mesh testing under dynamic loading](image)

**Figure 5.7** Numerical representations of mesh testing under dynamic loading: (a) shape of mesh at time $t = 40$, and (b) mesh at failure, after Bertrand et al. (2008).
Based on the limited investigation of the available historical numerical modelling records, the previous focus of numerical simulation of mesh behaviour has been based on continuum modelling approaches (discussed more in detail in Section 5.2.1). The resulting output of the models also presented limited information on the load transfers and the failure mechanisms of the mesh under a load. A comprehensive representation of mechanical behaviour should show how the load transfers on the steel mesh wires during different loading stages, and how mesh behaves upon failure.

5.2.1 Continuum and Discontinuum Numerical Modelling Approaches

To capture the mechanical behaviour of mesh in numerical models, the right choice of the modelling approach should be used. Numerical modelling that attempts to simulate mesh using continuum numerical approaches is not appropriate, because it cannot explicitly reproduce the non-linear anisotropic behaviour of the rock mass. However, the numerical representation of steel mesh as a surface support should implement the model that reflects a “real” support situation with broken rock mass. Lorig & Varona (2013) suggested that the rational decision on the selection of an appropriate numerical modelling approach for ground support design is based on the identification of the failure mode of rock mass and ground support. In this regard, a discontinuum numerical model simulates the interactions between deformable or rigid discrete blocks, as shown in Figure 5.8 (Itasca Consulting Group Inc. 2017). The approach “allows finite displacements and rotations of discrete bodies; including complete detachment”, and “recognizes new interactions automatically as the calculation progresses” (Itasca Consulting Group Inc. 2017).

Lorig & Varona (2013) provided a table summarizing recommended numerical analysis approaches for each mode of rock mass failure (Table 5.2). Discontinuum modelling approaches can better simulate the
structurally controlled instabilities and the failure mechanisms in the rock mass. The discontinuum modelling approach can also explicitly represent the interaction between the rock mass and the ground support elements. However, limited work has been done in the past using the discontinuum modelling approach to capture the mechanical behaviour of mesh. The following section introduces this thesis’ ongoing work to capture the mechanical behaviour of mesh using the discontinuum modelling approach.

<table>
<thead>
<tr>
<th>Analysis approach</th>
<th>Instability mode</th>
<th>Brittle rock failure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Empirical</strong></td>
<td>Weak rock shear failure</td>
<td>Hoek squeezing chart (Hoek &amp; Marinos 2000), Chern Criteria (Chern et al. 1998)</td>
</tr>
<tr>
<td><strong>Analytical</strong></td>
<td>Structurally controlled instability</td>
<td>Plasticity based closed-form solutions (Duncan-Fama 1993; Carranza-Torres &amp; Fairhurst 1999)</td>
</tr>
<tr>
<td><strong>Numerical modelling</strong></td>
<td>Brittle rock failure</td>
<td>Limit equilibrium (Hudson &amp; Harrison 2000) (e.g. UNWEDGE)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 5.3 Numerical Representation of the Mechanical Behaviour of Welded Wire Mesh in the 3D DEM

For the purpose of this thesis, the 3D Discrete Element Method (3D DEM) software package, 3DEC version 5.2 from Itasca Consulting Group Inc. (2016), was used. The construction of welded steel wire mesh made use of beam structural elements to represent individual steel wires. Each beam structural element is assumed as a “straight segment of uniform bisymmetrical cross-sectional properties” between two discrete nodes (Itasca Consulting Group Inc. 2016). There are twelve active degrees-of-freedom (Figure 5.9), and a number of independent parameters, which define the mechanical state of the beam element. For each displacement (translation and rotation) there is a corresponding generalized force (force and moment) in x-, y-, and z- axes (Itasca Consulting Group Inc. 2016). In Figure 5.9a, the translation is denoted by the notations: $w_{1,2}$, $u_{1,2}$, and $v_{1,2}$, and rotation is denoted by the notations: $\theta_{x1,2}$, $\theta_{y1,2}$ and $\theta_{z1,2}$. In Figure 5.9b, the force is denoted by the notations: $F_{x1,2}$, $F_{y1,2}$, and $F_{z1,2}$, and moment is denoted by the notations: $M_{x1,2}$, $M_{y1,2}$ and $M_{z1,2}$. For each beam element, these parameters are estimated at the two nodal points. Each beam element has a local coordinate system which is also defined at two nodal points. The forces generated in
the beam structural elements are applied to the “lumped mass” at each nodal point which move in response to unbalanced forces (force and moment), Figure 5.10. Slip between beam elements and block elements is modelled in the same manner as the block interaction along a discontinuity in 3DEC.

![Figure 5.9](image)

**Figure 5.9** Schematic mechanical representations of beam structural element: a) beam coordinate system and 12 active degrees-of-freedom of the beam element, after Itasca Consulting Group Inc. (2016), and b) sign convention for forces and moments at the ends of a beam, after Itasca Consulting Group Inc. (2016).

In the 3D DEM, block elements can be used to represent intact rock or other solid materials. The interface between the block element and beam structural element is represented with springs as illustrated in Figure 5.10. It is noted, however, that there is no remote interaction between the structural elements and the block elements as in the PFC3D model used by Bertrand et al. (2008).

![Figure 5.10](image)

**Figure 5.10** Schematic mechanical representation of the interface between the beam element and block element in the 3D DEM, reproduced based on the lumped mass representation of structural element by Itasca Consulting Group Inc. (2016).

The calculation method utilized by the 3D DEM for the numerical studies is explicit. This method employs a “time-marching” or “time-stepping” algorithm where the calculation steps begin from an initial location on the model and propagate to the rest of the model over the model cycle (Figure 5.11). An advantage of using 3D DEM is that it can better capture the mechanics of the rock mass behaviour and explicitly simulate
the interaction between the rock mass, the test frame, and the surface support elements. This is advantageous in further work involving the interaction between actual rock mass instead of a flat loading plate.

Figure 5.11 Explicit calculation cycle, after Itasca Consulting Group Inc. (2017).

5.4 Numerical Modelling of a Laboratory Static Load Test of Welded Wire Mesh Using the 3D DEM

5.4.1 Test Description

In order to explore the applicability of the 3D DEM to simulate loading on mesh, it was necessary to identify a laboratory testing rig as a reference. To this purpose, the large-scale static load test facility (Figure 5.12), built by the Western Australian School of Mines (WASM) Rock Mechanics Department, was chosen. The details of the test were described by Morton et al. (2007) and Morton (2009).

Figure 5.12 WASM static test facility, after Morton et al. (2007).
The WASM static test facility was chosen as the case study for its simple loading mechanism and boundary condition that can be more easily introduced in a numerical model. In the numerical test, the loading mechanism and boundary condition were the critical factors. The loading mechanism of the laboratory test, Figure 5.13a, involved a mechanical jack that applied a downward axial load to a 300 x 300 mm flat steel plate at the center of the mesh sample with a rate of approximately 4 mm per minute (Morton 2009). The loading steel plate was oriented in the same alignment as the steel wires on the mesh. The mesh sample was restrained to the metallic sample frame through a series of fixed steel “D shackles”, shown in Figure 5.13b.

![Loading plate](image1.png)  ![Fixed D shackle](image2.png)

**Figure 5.13** The loading and boundary condition from the laboratory test: (a) 300 x 300 mm flat steel loading plate, and (b) fixed D shackle boundary condition, after Morton (2009).

This boundary condition provided a rigid anchorage to the mesh. The overall deformation of the mesh was measured using four Linear Voltage Displacement Transducers (LVDTs) placed at the center of each of the four quadrants on the mesh sample. For the purpose of this work, only the welded steel wire mesh samples were studied. The laboratory test results showed combinations of three different failure mechanisms of welded steel wire mesh:

- Tensile failure of wire
- Failure of the wire through the Heat Affected Zone (HAZ)
- Failure at the weld

Morton (2009) suggested that the weld failure, Figure 5.14a, is associated with shear failure of the weld. The failure of the wire through the HAZ, Figure 5.14b, occurs when there is a failure of wire at the weld due to the reduction in the cross-sectional area of the wire. The tensile failure occurs when the weld strength is greater than the tensile strength of the wire, Figure 5.14c.
Figure 5.14 Three different welded wire mesh failure mechanisms: (a) weld failure, (b) failure of the wire through the HAZ, and (c) tensile wire failure, after Morton (2009).

The laboratory test results of the welded steel wire mesh samples showed that the steel wire failed first along the boundary on “one of the four directly loaded wires”, as shown in Figure 5.15a. The laboratory test result showed that the directly loaded wires initially experienced the highest tensile load as the applied load transferred to the mesh wires, as represented with grey shaded area in Figure 5.15b.

Figure 5.15 An example of the ruptured welded steel wire mesh sample: (a) laboratory test, after Morton (2009), and (b) diagram of broken wire sequence, after Morton (2009).

Failure of mesh wires propagated only along two sides of the mesh at right angles, instead of symmetrically on all four sides, as shown in Figure 5.15b, Morton (2009). This was because the initial failure of the mesh wires resulted in the loading plate rotating and producing a non-uniform loading on the mesh. The side of the mesh boundary where the initial failure of mesh wire occurred varied between different mesh samples. One of the critical factors which could have impacted the choice of the side of the initial mesh failure is the variability in the quality of mesh samples. This was suggested previously by Villaescusa (1999).

For the purpose of the numerical modelling, the tested welded wire mesh sample (MT025) was chosen. The detailed welded wire mesh sample configuration used for the numerical experiment is summarized in Table 5.3.
Table 5.3  The detailed configuration of the MT025 welded steel wire mesh laboratory static test sample, after Morton et al. (2007).

<table>
<thead>
<tr>
<th>MT025 welded wire mesh sample configuration</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesh type</td>
<td>Welded steel wire</td>
</tr>
<tr>
<td>Wire spacing (mm)</td>
<td>100 x 100 grid</td>
</tr>
<tr>
<td>Wire diameter (mm)</td>
<td>5.6</td>
</tr>
<tr>
<td>Sample mesh dimension (m)</td>
<td>1.3 x 1.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Laboratory test result</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak load (kN)</td>
<td>46.38</td>
</tr>
<tr>
<td>Peak displacement (mm)</td>
<td>181</td>
</tr>
<tr>
<td>Rupture mechanism</td>
<td>Tensile failure of wire</td>
</tr>
<tr>
<td>Position of rupture</td>
<td>Loaded wire on manufacturers boundary</td>
</tr>
</tbody>
</table>

Figure 5.16 shows the load and displacement response of one of the welded steel wire mesh samples in the laboratory test. The load-displacement response of the mesh sample shows a gradual increase in the load until the load drops at the peak upon mesh failure.

5.4.2 Numerical Model Geometry of Welded Wire Mesh

The use of 3DEC to reproduce the experiment described in Section 5.4.2. has been described by Karampinos et al. (2018). This section expands on the work of Karampinos et al. (2018).

Figure 5.17 shows a schematic diagram of the welded wire mesh geometry in the numerical model. A 1.3 x 1.3 m sheet of welded steel wire mesh was numerically modelled using a network of 50 mm long beam elements. Each of the 100 mm long steel wires was represented with two 50 mm long beam elements to facilitate the bending of each wire between the two weld points. The sheet of welded steel wire mesh was restricted around the boundary with block elements, which explicitly represented the metallic sample frame and the fixed D shackles. The motion of the block elements was fixed in all axes through a velocity
boundary condition to simulate a rigid test frame. The slippage at the contact between the beam structural elements and the block elements was modelled in the same manner as the block interaction along a discontinuity. A 300 x 300 mm block element was used to simulate the loading plate at the center of the mesh sample. The loading plate was fixed along Y- and X- axes through a velocity boundary condition. The load was applied to the beam structural elements through a velocity boundary condition along the z-axis. The applied load and the vertical displacement were measured at the center of the loading plate and plotted for monitoring and analysis purposes.

![Diagram of numerical model geometry](image)

**Figure 5.17** Schematic diagram of the numerical model geometry.

### 5.4.3 Material Properties in the 3D DEM

A detailed review of the material properties used to simulate the laboratory static load test of welded wire mesh in the 3D DEM models is discussed in this section. The material properties are divided into four main categories:

- Block properties
- Beam structural element properties
- Contact properties between blocks
- Contact properties between beam elements and blocks

#### 5.4.3.1 Block and Beam Structural Element Properties

The block elements were used to simulate the boundary constraints of the laboratory testing rig. In order to simulate a rigid metallic frame, a simple isotropic and elastic constitutive block model was used with very high bulk and shear moduli. A very high elastic strength was assigned to prevent any deformation and
translation of the metallic test frame. The block’s elastic properties include: density, bulk modulus, and shear modulus. The block properties were assigned based on the typical steel properties, and the steel density of 8050 kg/m³ was assumed. It is recognized that the density of the steel will change depending on the alloying constituents.

Beam structural elements were used to simulate the welded wire mesh samples. In this regard, the beam structural element properties were used to assign the material properties of mesh wires. The beam structural element is isotropic and linearly elastic, as default. However, an axial strain limit can be assigned to allow axial failure of the beam element. The material properties can be further categorized into either geometrical or material properties. The cross-sectional area and moment of inertia are the geometrical properties, and they are defined by the geometric characteristics of the beam element (e.g. beam’s thickness and cross-sectional shape). The geometrical material properties were estimated based on a 5.6 mm diameter steel wire. The steel properties are Young’s modulus, density, Poisson’s ratio, axial strain limit, and axial tensile yield strength. These are defined independently of the geometrical characteristics of the beam element. Young’s modulus, density, and Poisson’s ratio were obtained based on the typical steel properties. Axial tensile yield strength was obtained based on the axial tensile strength (450 MPa) of the steel wire as assumed by Morton (2009).

5.4.3.2 Contact Properties

The constitutive model for the contact between blocks was assumed to be an elastic-perfectly plastic Coulomb slip model. The Coulomb slip model properties include: normal stiffness, shear stiffness, friction angle, cohesion, dilation angle, and tensile strength. The constitutive model for the contact between the beam element and the block is the same as the Mohr-Coulomb slip constitutive model of the block joint model. The contact properties define the behaviour of the contact interface between the beam structural elements and block element. These can be further categorized into normal/shear elastic stiffness, and normal/shear strength parameters. Strength parameters include contact tensile strength, contact cohesive shear strength, and contact friction angle. The choice of appropriate contact properties depends on the objective of each numerical model. For example, high contact normal and contact shear stiffness were assigned at the interface between blocks to maintain the stiff contact between the blocks representing the metallic sample frame and the blocks representing the D shackles around the test boundary.

5.4.3.3 Properties Assigned in the Initial Base-Model

A base-model was developed based on the model geometry, as defined in the Section 5.4.2, and the material properties as summarized in this section. Table 5.4 summarizes the material properties used in the base-model.
Table 5.4  The material properties of the block and beam structural elements used in the initial base-model for a laboratory static load test of welded wire mesh using the 3D DEM.

<table>
<thead>
<tr>
<th>Block element properties</th>
<th>Beam structural element properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Bulk modulus (Pa)</td>
<td>1.67 x 10$^{10}$</td>
</tr>
<tr>
<td>Shear modulus (Pa)</td>
<td>7.69 x 10$^9$</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Young’s modulus (Pa)</td>
<td>2 x 10$^{11}$</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Tensile yield force (kN)</td>
<td>11.08</td>
</tr>
<tr>
<td>Cross-sectional area (m$^2$)</td>
<td>2.46 x 10$^{-5}$</td>
</tr>
<tr>
<td>Moment of inertia (m$^4$)</td>
<td>4.83 x 10$^{-11}$</td>
</tr>
</tbody>
</table>

Contact properties

| Joint normal stiffness (Pa/m) | 100 x 10$^9$ |
| Joint shear stiffness (Pa/m)  | 10 x 10$^9$  |
| Normal stiffness (Pa/m)       | 6 x 10$^6$ |
| Shear stiffness (Pa/m)        | 6 x 10$^6$ |
| Cohesive shear strength (N/m) | 7 x 10$^{20}$ |
| Tensile strength (N/m)        | 2 x 10$^{20}$ |
| Friction angle (degrees)      | 85          |

5.4.4 Verification Numerical Model of Uniaxial Pull Test of a Single Steel Wire

Prior to the construction of a full-scale welded wire mesh test, a simple uniaxial pull test of a single steel wire was constructed in the 3D DEM in order to verify the appropriateness of the numerical representation of tensile strength of a single steel wire. The numerical model consisted of two connected 50 mm length beam elements to represent a single steel wire, shown in Figure 5.18a. The axial load and the displacement of the top block were recorded and shown in Figure 5.18b.

![Numerical modelling of uniaxial pull test of a steel wire: a) force diagram of the pull test, and b) load-displacement response of the pull test.](image-url)

The boundary condition consisted of the bottom block fixed in all directions (stationary). A velocity was applied to the top block to pull the attached beam elements in z-direction to simulate the tensile loading of a steel wire. The contact stiffness and strength for the contact between the beam structural elements and the
block were assigned to be very high in order to maintain stiff contact during the experiment. The material and contact parameters of the beam elements used in the initial numerical test are summarized in Table 5.5.

**Table 5.5** Initial input parameters for a numerical test of a simple uniaxial pull test of a single steel wire.

<table>
<thead>
<tr>
<th>Beam structural element properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (Pa)</td>
<td>$2 \times 10^{11}$</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Tensile yield force (kN)</td>
<td>11.08</td>
</tr>
<tr>
<td>Cross-sectional area (m$^2$)</td>
<td>$2.46 \times 10^{-5}$</td>
</tr>
<tr>
<td>Moment of inertia (m$^4$)</td>
<td>$4.83 \times 10^{-11}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Contact properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic properties</td>
<td></td>
</tr>
<tr>
<td>Normal stiffness (Pa/m)</td>
<td>$6 \times 10^7$</td>
</tr>
<tr>
<td>Shear stiffness (Pa/m)</td>
<td>$6 \times 10^8$</td>
</tr>
<tr>
<td>Plastic properties</td>
<td></td>
</tr>
<tr>
<td>Cohesive shear strength (N/m)</td>
<td>$6 \times 10^6$</td>
</tr>
<tr>
<td>Tensile strength (N/m)</td>
<td>$6 \times 10^8$</td>
</tr>
<tr>
<td>Friction angle (degrees)</td>
<td>85</td>
</tr>
</tbody>
</table>

In this model, a block element is used to represent two stiff steel blocks to pin and pull a single steel wire. The elastic, isotropic constitutive model was used with very high elastic strength (bulk modulus and shear modulus). The bulk and shear moduli are related to Young’s modulus and Poisson’s ratio. The block properties used in the model are summarized in Table 5.6.

**Table 5.6** The material properties of the block elements used in the uniaxial pull test/tensile test of a single steel wire.

<table>
<thead>
<tr>
<th>Block element properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Bulk modulus (Pa)</td>
<td>$1.67 \times 10^{10}$</td>
</tr>
<tr>
<td>Shear modulus (Pa)</td>
<td>$7.69 \times 10^7$</td>
</tr>
</tbody>
</table>

The numerical result confirmed that the perfect-plastic deformation of the steel wire was observed when the axial load on the steel wire reached the tensile axial yield force of 11.08 kN.

**5.4.5 Full-Scale Model Calibration**

In this section, a series of numerical tests were performed to examine the impact of the elastic contact properties, the maximum strain limit, and the maximum bending moment limit.

**5.4.5.1 Beam-to-Block Contact Normal and Shear Stiffness**

A range ($6 \times 10^4$ to $6 \times 10^8$ Pa/m) of different beam-to-block normal and shear contact stiffnesses on the load and displacement response of mesh were examined. Figure 5.19 and Figure 5.20 compare the recorded
load and displacement responses at the loading plate to investigate the numerical fit of the model to the laboratory test’s loading response.

**Figure 5.19** Load and displacement comparison diagram of different contact normal stiffness with the fixed contact shear stiffness of $6 \times 10^6$ Pa/m.

To examine the influence of each individual contact stiffness parameter (normal or shear), the other contact stiffness was fixed at $6 \times 10^6$ Pa/m, as base-case. For example, when the contact normal stiffness was varying, the contact shear stiffness was kept constant at $6 \times 10^6$ Pa/m. The analysis showed that significant variability exists in the load and displacement response by changing the normal and shear contact stiffnesses. For the case when a contact normal stiffness of $6 \times 10^5$ and a contact shear stiffness of $6 \times 10^6$ were assigned, the closest fit of the curve was observed up to the load of about 16.3 kN, shown in Figure 5.19. However, the curve did not follow the experimental trend beyond this threshold.

**Figure 5.20** Load and displacement comparison diagram of different contact shear stiffness with the fixed contact normal stiffness of $6 \times 10^6$ Pa/m.

An additional comparison analysis was conducted on the combinations of different normal and shear contact stiffness, Figure 5.21. The result showed that when a normal contact stiffness of $6 \times 10^6$ and shear contact stiffness of $2 \times 10^6$ were assigned, they displayed a similar load-displacement response as when a normal contact stiffness of $6 \times 10^5$ and shear contact stiffness of $6 \times 10^6$ were assigned. This observation
illustrated that different combinations of normal and shear stiffness can give similar load-displacement responses. The case of when the normal contact stiffness was selected as $6 \times 10^6$ and the shear contact stiffness was assigned as $4 \times 10^6$ was selected for further analysis, as it closely intersected the peak load of the laboratory test load-displacement response.

![Figure 5.21 Load and displacement comparison diagram of different normal and shear contact stiffness combinations.]

**5.4.5.2 Maximum Strain Limit**

In order for the welded steel wire mesh to break, a tensile strain limit was assigned to each steel wire. A range of different strain limits ($0.001 - 0.0023$) was investigated to observe the load-displacement responses, Figure 5.22. This range was carefully selected after monitoring the strain response in real time from the initial model. The result showed that as the assigned strain limit increased, the peak load of the mesh sample increased. However, this did not influence the stiffness of the loading response.

![Figure 5.22 Load and displacement comparison diagram of different axial strain limit values.]

A simple observation based on the load and displacement responses showed that a strain limit of 0.0017 was the most appropriate, as the numerically measured axial load dropped, as expected, at the measured maximum load of the laboratory test. Nevertheless, a detailed analysis showed that the strain limit was
reached before the axial yield had occurred, which was not an accurate representation of the loading mechanism. Therefore, after further calibration processes, a strain limit of 0.0023 was chosen, despite its higher peak load in comparison to the laboratory test result. The material properties and the load-displacement response diagram of the first calibrated model are shown in Table 5.7 and Figure 5.23, respectively. The colour contour plots in Figure 5.24 show the measured axial tensile load on the mesh wires and the vertical displacement of the mesh. For the colour contours, FISH codes were written in order to explicitly show the breakage of individual steel wires upon failure (i.e. when the tensile strain limit of a wire is reached).

![Load and displacement comparison diagram](image)

**Figure 5.23** Load and displacement comparison diagram of the numerical result with 0.0023 axial strain limit and the laboratory test result.

**Table 5.7** Detailed test configuration used for numerical model with 0.0023 axial strain limit.

<table>
<thead>
<tr>
<th>Beam Structural Element Properties</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (Pa)</td>
<td>$2 \times 10^{11}$</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Tensile yield force (kN)</td>
<td>11.08</td>
</tr>
<tr>
<td>Cross-sectional area (m$^2$)</td>
<td>$2.46 \times 10^{-3}$</td>
</tr>
<tr>
<td>Moment of inertia (m$^4$)</td>
<td>$4.83 \times 10^{-11}$</td>
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<tr>
<td>Strain Limit</td>
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<table>
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<th>Contact Properties</th>
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<tr>
<td>Elastic Properties</td>
</tr>
<tr>
<td>Normal stiffness (Pa/m)</td>
</tr>
<tr>
<td>Shear stiffness (Pa/m)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plastic Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive shear strength (N/m)</td>
</tr>
<tr>
<td>Tensile strength (N/m)</td>
</tr>
<tr>
<td>Friction angle (degrees)</td>
</tr>
</tbody>
</table>
1. Applied load: 23.06 kN, displacement: 141.27 mm

2. Applied load: 66.84 kN, displacement: 208.78 mm

3. Applied load: 35.42 kN, displacement: 208.80 mm

Figure 5.24 Axial load on mesh wires (left) and vertical displacement of the mesh (right) colour contours of three-dimensional DEM initial result of welded steel wire mesh.
In the first calibrated numerical modelling result, the failure of the welded steel wire mesh occurred at the loading plate. However, in the laboratory tests, the failure of the welded mesh occurred along the mesh boundary. Consequently, further calibration was necessary in order to capture the failure mechanism as observed during the physical laboratory test.

5.4.5.3 Maximum Bending Moment Limit

A limit was set to the generated moment, with the objective to capture the bending failure mechanism shown in Figure 5.25. The limit was set based on the maximum bending moment obtained using the analytical solution of a simple three-point load test.

![Figure 5.25 Schematic representation and force diagram of a three-point bending experiment: (a) schematic representation of a beam, and (b) force diagram showing the deflection (S), applied load (P), and axial load (tensile and compression) on steel wire.]

The maximum bending moment (My) of a Ø5.6 mm steel wire was calculated using the analytical flexure formula (Gere & Goodno 2008):

\[
\sigma_{ts} = \frac{M_y c}{I} \quad (5)
\]

\[
I = \frac{\pi d^4}{64} \quad (6)
\]

where, \(\sigma_{ts}\) represents the maximum stress occurred at the surface furthest away from the neutral axis of the beam [N/m²], \(M_y\) represents the maximum moment generated [Nm], \(c\) represents the furthest distance from the neutral axis [m], and \(I\) represents the moment of inertia [m⁴]. Based on the analytical formula, the maximum bending moment limit of a Ø5.6 mm steel wire was calculated to be 7.8 Nm.

Then, the theoretical yield load and yield displacement were calculated using the following formulas (Gere & Goodno 2008):

\[
P = \frac{4L \sigma_{ts}}{L} \quad (7)
\]

\[
S = P \frac{L^3}{48EI} \quad (8)
\]
where \( P \) represents the load applied to cause the bending of the wire [N], \( L \) represents the length of a wire [m], \( E \) represents the elastic modulus [Pa], \( S \) represents the maximum deflection [m], and \( d \) represents the diameter of the wire [m]. An elastic modulus of 200 GPa and a wire length of 100 mm were assumed in the calculation. The resulting theoretical yield load and displacement were calculated to be 312 N and 0.7 mm, respectively.

In order to observe the appropriateness of the representation of the bending mechanism in the 3D DEM, a numerical model of a three-point load test, Figure 5.26, was constructed and compared to the analytical solution. The input parameters used in the model are summarized in Table 5.8.

![Figure 5.26 Schematic diagram of numerical model of three-point load experiment using 3D DEM.](image)

**Figure 5.26 Schematic diagram of numerical model of three-point load experiment using 3D DEM.**

<table>
<thead>
<tr>
<th>Beam Structural Element Properties</th>
<th>Contact Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment limit (Nm)</td>
<td>Elastic Properties</td>
</tr>
<tr>
<td>Elastic modulus (Pa)</td>
<td>Normal stiffness (Pa/m)</td>
</tr>
<tr>
<td>Moment of inertia (m^4)</td>
<td>Shear stiffness (Pa/m)</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>Plastic Properties</td>
</tr>
<tr>
<td>Density of steel (kg/m^3)</td>
<td>Cohesive shear strength (N/m)</td>
</tr>
<tr>
<td>Tensile yield force (kN)</td>
<td>Tensile strength (N/m)</td>
</tr>
<tr>
<td>Cross-sectional area (m^2)</td>
<td>Friction angle (Degree)</td>
</tr>
</tbody>
</table>

A single Ø5.6 mm steel wire was constructed using two beam structural elements, similar to the ones used in the uniaxial pull-test experiment as discussed previously. A velocity was applied on the loading block at the center node to simulate the static loading condition. The boundary condition consisted of two support blocks fixed along all three directions. For the contact properties, a high normal stiffness and tensile strength and a low cohesive shear strength and friction angle were assigned in order for the beam structural element to slip along the x-axis. The yield load and yield displacement values from the analytical flexure formulas were closely matched with the numerical model as shown in Figure 5.27.
5.4.6 Calibrated Model of Welded Wire Mesh in the 3D DEM model

A calibration process between the load and displacement curve from the laboratory and the numerical results led to a revised model. The resulting load-displacement response, compared to the previous response, illustrated that the behaviour of the loading response curve became softer, as shown in Figure 5.28.

Figure 5.27 a) Load displacement curve for the modelled three-point load test; b) magnitude of the bending moment at the central node; and c) Modelled axial load on the beam elements for different magnitudes of the applied load.

Figure 5.28 Load and displacement comparison diagram of numerical modelling result with- and without- bending moment limit.
Consequently, the contact stiffness of the beam elements was calibrated further to provide a better fit to the laboratory test loading response. Figure 5.29 shows a load and displacement comparison revised diagram of the latest calibrated model and the laboratory test result. The input parameters used in the calibrated model are summarized in Table 5.9. The colour contour plots showing the axial load and displacement on the beam elements at different loading stages are illustrated in Figure 5.30.

Table 5.9 Input parameters for the calibrated numerical model of welded steel wire mesh.

<table>
<thead>
<tr>
<th>Beam Structural Element Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (Pa)</td>
<td>$2 \times 10^{11}$</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>8050</td>
</tr>
<tr>
<td>Tensile yield force (kN)</td>
<td>11.08</td>
</tr>
<tr>
<td>Cross-sectional area (m$^2$)</td>
<td>$2.46 \times 10^{-5}$</td>
</tr>
<tr>
<td>Moment of inertia (m$^4$)</td>
<td>$4.83 \times 10^{-11}$</td>
</tr>
<tr>
<td>Axial Strain Limit</td>
<td>0.025</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Contact Properties</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Elastic Properties</td>
<td></td>
</tr>
<tr>
<td>Normal Stiffness (Pa/m)</td>
<td>$6 \times 10^7$</td>
</tr>
<tr>
<td>Shear Stiffness (Pa/m)</td>
<td>$6 \times 10^6$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plastic Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive shear strength (N/m)</td>
<td>$7 \times 10^{20}$</td>
</tr>
<tr>
<td>Tensile strength (N/m)</td>
<td>$2 \times 10^{20}$</td>
</tr>
<tr>
<td>Friction angle (degrees)</td>
<td>85</td>
</tr>
</tbody>
</table>

![Figure 5.29 Load and displacement comparison diagram of the calibrated numerical model and the laboratory test result.](image-url)
Figure 5.30 Axial load on mesh wires (left) and vertical displacement of mesh (right) colour contours of three-dimensional DEM calibrated result of welded steel wire mesh.

1. Applied load: 16.47 kN, displacement: 133.55 mm

2. Applied load: 43.51 kN, displacement: 176.94 mm

3. Applied load: 43.53 kN, displacement: 176.95 mm
5.5 Comparison between the Laboratory Result and Numerical Model

The results from the calibrated numerical models of the welded steel wire mesh were further compared to the laboratory test result. In both cases, the applied load was first transferred from the loading plate to the welded steel wire mesh along the directly loaded wires, illustrated as red arrows in area 1 in Figure 5.31b. The load was then transferred to the wires along the mesh boundary, illustrated as blue dotted arrows in Figure 5.31b. As the mesh continued to take the load, and as the mesh wires were further tensioned, the wires in area 2 started to load. Subsequently, the first failure of the mesh occurred at the mesh boundaries on one of the four directly loaded wires. The numerical results showed that the load transfer was successfully captured.

![Figure 5.31 Schematic diagram showing the load transfer on welded steel wire mesh: (a) laboratory result, after Morton 2009, and (b) calibrated numerical modelling result.](image)

5.6 Summary

This chapter investigated the use of 3D DEM numerical modelling to simulate welded steel wire mesh. For an initial loading model, 3D DEM was used successfully to capture the mechanical behaviour of mesh as observed in laboratory tests using beam structural elements.

The calibrated model explicitly simulated the interaction between the welded wire mesh and the loading plate. The modelling results indicated that there are numerous different material and contact parameters that govern the performance of welded steel wire mesh. Furthermore, the results showed that the use of the bending moment limit was critical for capturing the mechanical behaviour of the mesh under the examined
conditions. The numerical modelling allowed for flexible investigation of the parameters controlling the behaviour of the mesh, such as different loading conditions, using the calibrated model. As a result, the investigation provided further insight into better understanding the behaviour of welded wire mesh under different test configurations. It is acknowledged that the constructed numerical models reproduce the testing rig configuration and the results obtained in a series of laboratory experiments. Like the laboratory tests, they may not necessarily fully capture field conditions.
Chapter 6

6    Numerical Parametric Investigations

6.1    Introduction

In Chapter 5, the 3D DEM was used to successfully simulate the mechanical behaviour of mesh as observed in the static laboratory test of welded wire mesh using beam structural elements. This included the failure mechanism, load transfer, and the load-displacement response of the welded wire mesh. The developed calibrated model provided the confidence to further investigate how different testing parameters impact the mesh behaviour.

In this chapter, a series of numerical tests were conducted to examine the influence of different loading plate and mesh dimensions, and the loading plate orientations on the mesh behaviour using the developed calibrated model.

6.2    Loading Plate

The loading plate size and shape are important considerations for testing rig configurations. Under static conditions, the loading plate attempts to simulate the static loading of the rock mass on the mesh. The loading plate in laboratory tests typically applies a single vertical axial load on the surface of the mesh with a specific plate dimension. However, it was shown in Chapter 3 that the surface area of the mesh which is imposed on by the broken rock mass can significantly vary. In this section, the influence of different loading plate dimensions and orientations on the mesh performance were investigated using the developed calibrated model from Chapter 5. In all cases, the material properties of the mesh sample and the test frame were kept the same as in the calibrated model. While the loading condition varied, the boundary conditions were kept constant for all cases.

6.2.1    Loading Plate Dimensions

In this section, the impact of different loading plate dimensions on the mesh performance is reviewed. In all cases, the loading plate was assumed to be square and the plate orientation was aligned parallel to the mesh wires, as previously shown in Figure 5.13. The fixed mesh dimension of 1.3 x 1.3 m was used for all cases. The colour contour plots showing the axial load and displacement on the beam elements at different loading stages are illustrated in Figure 6.1, Figure 6.2, and Figure 6.3. For example, Figure 6.1a shows the test result of mesh dimension of 1.3 x 1.3 m with loading plate dimension of 100 x 100 mm. The load is
applied to the beam structural elements through the loading plate at the center of the mesh. As the load increased, the elements at the mesh boundaries were subjected to a combined bending and axial load. The load is continuously applied until the first failure of element occurred. At an applied load of 32.97 kN, axial load at the wires that are directly loaded from the loading block reached its maximum plastic strain limit and failed. The elements at the boundaries of the model are subjected to a combined bending and axial load. After failure, there is a significant drop in the load and displacement. The other models exhibited similar load transfer as the example illustrated previously.

Figure 6.1  Axial load and vertical displacement of beam elements at initial wire failure: a) 100 x 100 mm, and b) 200 x 200 mm loading plate dimensions.
c) 300 x 300 mm loading plate dimension. Applied load: 43.51 kN, displacement: 176.94 mm

d) 400 x 400 mm loading plate dimension. Applied load: 46.75 kN, displacement: 166.87 mm

e) 500 x 500 mm loading plate dimension. Applied load: 58.14 kN, displacement: 153.41 mm

Figure 6.2  Axial load and vertical displacement of beam elements at initial wire failure: c) 300 x 300 mm, d) 400 x 400 mm, and e) 500 x 500 mm loading plate dimensions.
Figure 6.3  **Axial load and vertical displacement of beam elements at initial wire failure:** f) 600 x 600 mm, and g) 700 x 700 mm loading plate dimensions.

In all cases, the numerical models captured similar load distributions and failure mechanisms as observed in the calibrated model. First, the load was applied to the beam structural elements through the loading plate and was redistributed along the beam structural elements. The beam structural elements at the model boundaries were subjected to a combined bending and axial load. As the load continued to be applied on mesh through the loading plate, the beam structural elements at the boundary continued to bend and the axial load was concentrated on the elements at the boundary. This caused the elements at the boundary to eventually fail in tension as demonstrated in the figures above. After the initial wire failure, there was a significant drop in the load on its loading response. This is shown in Figure 6.4.

The series of numerical tests also show that for the mesh tested with a 100 x 100 mm loading plate dimension (applied load: 32.97 kN, displacement: 207.32 mm; recorded at the initial wire failure), there
was a lower applied load when the initial wire failure occurred compared to the test result of mesh with a 700 x 700 mm loading plate dimension (applied load: 79.60 kN, displacement: 131.04 mm; recorded at the initial wire failure). However, the recorded displacement at the initial wire failure was higher for the mesh tested with a smaller loading plate. A higher peak load and a lower peak displacement was observed when a larger loading plate dimension was used. Despite the higher peak load observed with a larger plate dimension, the axial load on the wires directly below the loading plate decreased (i.e. yellow colour to green colour). These observations demonstrate how the applied load transfer, which occurs through the loading plate onto the mesh wires, changes as the loading plate dimension changes.

The decrease in the axial load on the mesh wires that are directly below the loading plate happens as the loading plate dimension increases. This can be explained by examining the number of wires that transfer the load from the loading plate. The increase of the loading plate dimension caused the additional mesh wires to be directly loaded by the loading plate. This results in a lower load to be taken by each of the wires at the loading plate and wider spread loading distribution. Consequently, the bending of the wires at the boundary occurred at a much slower rate. This resulted in a higher peak load (i.e. higher mesh capacity). Figure 6.4 compares the load and displacement of different loading plate dimensions.

![Figure 6.4 Axial load and displacement comparison diagram of the different loading plate dimensions measured at the loading plate. The mesh size was 1.3 x 1.3 m.](image)

As the loading plate dimensions increased, a stiffer loading response is observed. While the mesh dimensions were kept constant, the increase in the loading plate dimension caused the distance between the loading plate and the test boundary to decrease. This decrease in the distance caused the overall load transfer rate, from the loading plate to the wires at the boundary, to increase. Figure 6.4 illustrates how the load
transfer rate on the wires increased with a loading plate dimension, while less load was transferred along each wire, causing higher peak load and lower peak displacement. The numerical results clearly illustrated that the peak load, peak displacement, and the stiffness of the mesh behaviour are significantly influenced by the loading plate dimensions of the testing rig configuration. The analysis of the results also demonstrated how numerical modelling can be used to better understand the mesh performance by investigating the load transfer and the failure mechanisms of mesh. The implication of this work is that even using the same testing rig and boundary condition, the loading plate size can cause significant changes to the mesh performance given the lack of standardization in testing rigs as discussed in Chapter 4.

Figure 6.5 compares the peak loads at the initial wire failure of different loading plate dimensions against the loading plate area. The observed trend shows a positive linear relationship with the applied load measured at the loading plate and the loading plate area. Figure 6.6 compares the peak displacement at the initial wire failure of different loading plate dimensions against the loading plate area. Based on the limited data points, the observed trend shows a negative linear relationship with the displacement of mesh measured at the loading plate and the loading plate area.

![Figure 6.5 Axial load and loading area comparison diagram of different loading plate dimensions. Linear regression line with $R^2 = 0.988$.](image)

Theses observed trends, however, are based on the numerical test results of mesh under ideal test conditions (i.e. no mesh defects and perfectly symmetrical boundary condition with constant static loading at the center of the mesh). In practice, however, there are various uncertainties associated with conducting such an experiment, which can lead to significant variations in the test results.
Figure 6.6  Displacement of mesh and loading area comparison diagram of different loading plate dimensions. Linear regression line with $R^2 = 0.927$.

The behaviour of the loading response of the numerical test results of welded wire mesh were compared with the loading response of Thompson (2004)’s laboratory test results of welded wire mesh. Figure 6.7 shows schematic diagrams showing the geometric arrangements of some of Thompson (2004)’s laboratory test results that were compared.

Figure 6.7  Geometric arrangements of a laboratory test of mesh using a 1.5 x 1.5 m bolting spacing: (a) 75 x 75 mm loading plate dimension, and (b) 105 x 105 mm loading plate dimension, after Thompson (2004).

The load and displacement responses of Thompson (2004)’s laboratory test results (Figure 6.8) show that the mesh with the 1050 x 1050 mm loading plate dimension responded more stiff and produced a higher peak load. This is in agreement with Thompson (2004)’s laboratory test results.
6.2.1.1 Failure Progression of the Mesh Wires

A series of numerical models were used to investigate the failure progression of the mesh wires. In general, there was a slight numerical variation on the location of the initial wire failure along the boundary (e.g. initial wire failure was slightly off centered with 100 x 100 mm and 200 x 200 mm loading plate dimensions, and the initial wire failure occurred at the “top” side with 300 x 300 mm loading plate dimension). In practice, it is not possible or realistic to predict the exact strand of wire that will fail at the initial failure of the mesh.

For example, Figure 6.9 shows load colour contour plots at different loading stages for the testing of mesh using a 500 x 500 mm loading plate dimension. The colour contour plots show that the failure progression of the beam structural elements initially alternated between the two adjacent sides of the mesh at a right angle, as shown in the loading stages in Figure 6.9b – Figure 6.9e. Then, the failure of the beam structural elements progressed between the other sides of mesh that are adjacent to each other, as shown in the loading stages in Figure 6.9f – Figure 6.9i.
Figure 6.9  Axial load on beam structural elements at different loading stages showing the mesh failure progression for the test of mesh using a 500 x 500 mm loading plate dimension.

Figure 6.10 shows displacement colour contour plots at different loading stages for the testing of mesh using a 500 x 500 mm loading plate dimension. There are no significant changes observed to the beam structural elements as the wire failures occur rather quickly beyond the first wire failure (within few modelling steps, i.e. less than 100).
Figure 6.10 Displacement on beam structural elements at different loading stages showing the mesh failure progression for the test of mesh using a 500 x 500 mm loading plate dimension.

Figure 6.11 shows the axial load and the vertical displacement at different loading stages (a – i) of the test of mesh using a 500 x 500 mm loading plate dimension.
6.2.2 Different Loading Plate Orientations

In this section, the impact of different loading plate orientations on the mesh behaviour is reviewed. For this purpose, two different loading plate orientations were considered, as shown in Figure 6.12. For each loading plate orientation, three loading plate dimensions were investigated: 100 x 100 mm, 300 x 300 mm, and 500 x 500 mm. A fixed mesh dimension of 1.3 x 1.3 m was used for all cases.

Figure 6.12 Laboratory setting of different loading plate orientations: a) square orientation, and b) diamond orientation, after Morton (2009).

In this numerical experiment, the loading plate with a square orientation (Figure 6.12a) had its side edges aligned parallel to the wire strands of the mesh. The loading plate with the diamond orientation (Figure 6.12b) had side edges aligned 45 degrees to the wire strands of the mesh. Figure 6.13 shows the axial load color contour plots comparing the peak loads from the test of mesh using the two different plate orientations. Figure 6.14 shows the displacement color contour plots comparing the peak displacements from the test of mesh using the two different plate orientations.
Beam elements, axial force (N)

<table>
<thead>
<tr>
<th></th>
<th>-3000</th>
<th>-6000</th>
<th>-9000</th>
<th>-11084</th>
</tr>
</thead>
</table>

a) Square orientation: 100 x 100 mm, 300 x 300 mm, and 500 x 500 mm loading plate dimensions (left to right). Applied load (left to right): 32.97 kN, 43.51 kN, and 58.14 kN.

b) Diamond orientation: 100 x 100 mm, 300 x 300 mm, and 500 x 500 mm loading plate dimensions (left to right). Applied load (left to right): 31.28 kN, 40.86 kN, and 50.85 kN.

Figure 6.13 Axial load of beam elements at initial wire failure with different loading plate dimensions and orientations: (a) square orientation, and (b) diamond orientation.

In all cases, the numerical models captured similar load distributions and failure mechanisms as observed in the calibrated model. The load was applied to the beam structural elements through the loading plate and was redistributed along the beam structural elements. The beam structural elements at the model boundaries were subjected to a combined bending and axial load. As load continued to be applied on the mesh through the loading plate, the beam structural elements at the boundary continued to bend. The axial load was concentrated on the elements. This caused the elements at the boundary to eventually fail in tension as demonstrated in the figures above. As a result of the initial wire failure, there was a significant drop in the applied load response, as shown in Figure 6.15.
The test results showed that the initial wire failure was observed at the mesh boundary for all scenarios. Similar higher peak load and lower peak displacement were observed with a larger plate dimension, as previously noted in the Section 6.2. The initial wire failure in all scenarios occurred along the mesh boundary. However, there were slight numerical variations on the initial wire failure locations. The limited test results showed that in general, with the square-oriented loading plate, the location of the initial wire failure occurred on one of the center wires at the mesh boundary. However, with the diamond-oriented loading plate, the location of the initial wire failure occurred on one of the wires slightly off-centered at the mesh boundary.

Figure 6.15 compares the axial load and displacement of different loading plate dimensions and orientations. The diagram illustrates that the test results with a square-oriented loading plate generally produced a higher peak load at the initial wire failure when compared to the test with a diamond-oriented
loading plate. The stiffness of the loading response was also greater for the square-oriented loading plate cases when compared to the diamond-oriented plate cases. Based on the limited test results, it was observed that the difference in the peak displacements between the square-oriented and diamond-oriented loading plate was greater with a smaller loading plate dimension (100 x 100 mm), while a larger loading plate dimension (500 x 500 mm) showed greater difference in the peak loads. The difference can be attributed to the fact that the diamond-oriented 500 x 500 mm loading plate touched the mesh wire at the corners of its loading plate (while other loading plate dimensions did not). This can be further explained in future work.

Figure 6.15 Axial load and displacement comparison diagram of the different loading plate orientations and loading plate dimensions.

6.2.2.1 Load Transfer

Figure 6.16 shows that, in both loading plate orientation scenarios, the redistribution of the applied load from the loading plate onto the directly loaded wires occurred in the area “1”. Figure 6.16 also shows that the axial loads on the directly loaded outer wires (as represented with red arrows in area “1”) by the diamond-oriented loading plate were significantly lower than the same wires loaded by the square-oriented loading plate. The distance between the directly loaded outer wires and the boundary condition was longer with the diamond-oriented loading plate, which resulted in a lower load on the outer wires. In contrast, the loads on the wires in area “2” increased with the diamond-oriented loading plate as the loading plate directly loaded more wires (8 by 8 wires) than the square-oriented loading plate (6 by 6 wires). There are more wires loaded directly in the diamond-oriented plate as opposed to the square oriented plate. However, the load concentration at the boundaries is higher in the square oriented loading plate case.
Figure 6.16 Modelled axial load on beam structural elements before the initial showing load transfer on the mesh steel wires: (a) square-oriented loading plate, and (b) diamond-oriented loading plate.

Figure 6.17 shows the vertical displacement of mesh colour contour plots with different loading areas on the mesh. The plots show a slightly broader area of displaced mesh at area 1 for the square-oriented loading plate than the diamond-oriented loading plate. Whereas, area 2 shows a broader area of displaced mesh for the diamond-oriented loading plate.

Beam elements, vertical displacement (m)

Figure 6.17 Modelled displacement on beam structural elements before the initial showing load transfer on the mesh steel wires: (a) square-oriented loading plate, and (b) diamond-oriented loading plate.
6.3 Different Mesh Dimensions

In this section, the influence of different mesh dimensions on the mesh performance were numerically investigated using the developed calibrated model from Chapter 5. In all cases, the material properties of the mesh sample and the test frame were kept the same as in the calibrated model. For this analysis, three different mesh dimensions were investigated: 1.3 x 1.3 m, 1.1 x 1.1 m, and 0.9 x 0.9 m. While the mesh dimensions varied, the loading conditions and the boundary conditions were kept constant.

Figure 6.18 compares the load and displacement recorded at the initial wire failure of three test configurations of different mesh and loading plate dimensions. In general, the tests conducted using a 100 x 100 mm loading plate produced the highest peak displacements. In contrast, the tests conducted using a 500 x 500 mm loading plate generated the highest peak load. Within the same loading plate dimension, smaller mesh dimension provided the highest peak load, while the largest mesh dimension provided the highest peak displacement.

Figure 6.18 Load and displacement comparison diagram of three different mesh and loading plate dimensions recorded at the initial wire failure.

Figure 6.19, Figure 6.21, and Figure 6.23 compare the loading responses, until the first failure of mesh wire, of different mesh dimensions with fixed 100 x 100 mm, 300 x 300 mm, and 500 x 500 mm loading plate dimensions, respectively. Figure 6.20, Figure 6.22, and Figure 6.24 compare colour contour plots of axial load and vertical displacement of the beam structural elements at the initial wire failure.

Dolinar (2006) defined stiffness as the slope of a line drawn from the peak load to 25% of the peak load. Tannant (2004) defined stiffness as the slope of a line drawn from the peak load to 50% of the peak load. Based on both definitions, the loading responses of different mesh dimensions demonstrate that the smaller mesh dimension (0.9 x 0.9 m) generally provided stiffer responses, compared to the larger mesh dimension (1.3 x 1.3 m).
Figure 6.19  Load-displacement comparison diagram of different mesh dimensions with fixed 100 x 100 mm loading plate dimension.

Figure 6.20  Axial load and vertical displacement of the beam elements for 100 x 100 mm loading plate dimension showing the initial wire failure. Applied load & displacement (top to bottom): (41.43 kN, 182.52 mm), (36.47 kN, 195.97 mm), and (32.97 kN, 207.32 mm).
Figure 6.21 Load-displacement comparison diagram of different mesh dimensions with fixed 300 x 300 mm loading plate dimension.

Figure 6.22 Axial load and vertical displacement of the beam elements for 300 x 300 mm loading plate dimension showing the initial wire failure. Applied load & displacement (top to bottom): (59.10 kN, 146.29 mm), (49.59 kN, 162.92 mm), and (43.51 kN, 176.94 mm).
Figure 6.23 Load-displacement comparison diagram of different mesh dimensions with fixed 500 x 500 mm loading plate dimension.

Figure 6.24 Axial load and vertical displacement of the beam elements for 500 x 500 mm loading plate dimension showing the initial wire failure. Applied load (top to bottom): (80.48 kN, 112.64 mm), (65.25 kN, 134.41 mm) and (58.14 kN, 153.41 mm).
The numerical results follow the same trend as observed in a laboratory test. The peripheral boundary condition, as used in the numerical tests, attempts to represent the bolting spacing in practice. In this regard, the numerical modelling results of different mesh dimensions were compared to Thompson (2004)’s test results of different bolting spacing using a constant loading plate dimension. Figure 6.25 and Figure 6.26 compare load and displacement curves from Thompson (2004)’s laboratory test results of different bolting spacing configurations using two different loading plate dimensions.

![Figure 6.25 Load and displacement comparison diagram of different bolting spacings with the constant 750 x 750 mm loading plate dimension, after Thompson (2004).](image1)

![Figure 6.26 Load and displacement comparison diagram of different bolting spacings with the constant 1050 x 1050 mm loading plate dimension, after Thompson (2004).](image2)

The diagrams show that a stiffer loading response and a greater peak load were observed from the test with the tighter bolting spacing. This observation was clearly shown in both scenarios (Figure 6.25 and Figure 6.26) to a different degree. In contrast, the peak displacement was greater for the test with the wider bolting spacing.
spacing. The general trend from the numerical modelling results is in agreement with the trend reported from Thompson (2004)’s laboratory test results.

6.4 Summary

This chapter investigated the influence of different testing rig configurations on the behaviour of mesh using the successfully calibrated numerical model from Chapter 5. The numerical results showed that the loading plate dimension, as a function of the mesh dimensions, is a critical parameter in the performance of mesh. The numerical simulation demonstrated that variability in the loading plate and mesh dimensions clearly influence the peak load, peak displacement, and stiffness response of the mesh performance.

Further investigation into the mesh dimensions showed that the use of larger loading plates can provide better performance for the same quality of mesh sample under the constant boundary conditions. For practical purposes, the mesh dimensions are defined by the bolting pattern, and the loading plate dimensions are defined by the loaded rock mass on mesh.

The work discussed in this chapter also illustrated that the laboratory test configuration can significantly influence the performance of mesh samples. Therefore, it may not be appropriate to compare the test results from one laboratory testing rig to another. The numerical modelling showed some advantages over the physical laboratory tests for parametric studies of mesh as it allowed for easier manipulation of the parameters.
Chapter 7

7 Conclusions

7.1 Summary

Welded wire mesh is part of the ground support in underground mines. There are however limited data available on the performance of mesh. Laboratory tests are used to provide a quantitative indicator of mesh performance. For this purpose, a comprehensive review of various static laboratory mesh testing rigs was conducted. This included reviews of different testing rig configurations, mesh configurations, and observed test results. To this end, the load and displacement of the static working capacity and the ultimate capacity were defined as fundamental quantifiable measurements of mesh performance. The quantifiable mesh performance measurements of each testing rig examples were then discussed in relation to the corresponding testing rig configurations and mesh configurations. These comparative reviews demonstrated that there exist various testing rig configurations for testing mesh, but that each testing rig captures limited loading mechanisms and boundary conditions. The observed results of these reviews showed that the testing configurations and the selected testing parameters significantly influenced the loading response of mesh samples.

Field observation showed that complex loading mechanisms are not reproduced in testing rigs. The attempts to standardize a method of conducting the visual inspection in field, as well as the strengths and the challenges associated with the approach, were discussed.

After reviewing a number of configurations, a static laboratory testing rig configuration of mesh was chosen for the numerical modelling. Its testing configuration and observed results were discussed in detail. The welded wire mesh sample used in the laboratory test was numerically simulated using beam structural elements in 3D DEM. The numerically modelled laboratory test was then calibrated; it showed a close initial loading response alignment with the laboratory test. The calibrated numerical model provided a tool to manipulate and investigate some of the critical test parameters controlling the mesh behaviour. The numerical parametric studies on some of the critical parameters of mesh behaviour showed that different test configurations, such as the loading plate and mesh dimensions, had significant influence on the mesh performance. The results illustrated a critical role that numerical modelling can have in developing guidelines on mesh design. Therefore, the numerical modelling can be a better choice to provide more
robust tool to examine the mesh performance compared to other available approaches for mesh design guidelines.

7.2 Contributions

This thesis provided a comprehensive review of testing rigs used to determine the performance of mesh. It was clearly shown that all rigs only capture a simple loading mechanism. An in-situ investigation revealed that there are several and considerably more complex loading mechanisms. Furthermore, it was also observed that the testing rigs have considerable variations in the testing configurations and procedures. This thesis developed a 3D model of welded wire mesh and demonstrated that it can capture the mechanical behaviour of a loading and boundary condition. This calibrated model was used to further investigate the influence of the variations in testing rig configurations on the obtained results. This work is of significance towards the design of a standardized testing procedure.

7.3 Limitations

The limitations of this thesis are largely associated with numerical modelling. This has illustrated that the obtained and available results on mesh performance are strongly influenced by testing rig configuration. A limitation of the developed numerical model was that the preferential failure location, limited to only two sides of the mesh, was not captured as observed in the laboratory test. This was due to the model’s assumption of an ideal symmetrical loading scenario. The preferential failure observed in the laboratory test could have been caused by the mesh defect associated with the weld quality, a non-uniform boundary condition using the D shackles, or rotation of the loading plate observed upon the initial wire failure. Furthermore, the current numerical model does not capture any variation in the mesh quality. In practice, however, there is no reliable tool available to capture the variation in the mesh quality.

Furthermore, the numerical model does not capture the weld failure of mesh. However, the numerical model does allow the displacement and rotation of each mesh wire to interact at its nodes in three-dimensional space. Furthermore, the model captures the axial loading and bending of a steel wire, and consequently the failure through tensile axial loading of wire.

7.4 Recommendations & Path Forward

The resulting thesis’ work can improve the understanding of how mesh can be incorporated in the design data as surface support in underground mines. The review of the laboratory tests and the numerical parametric investigations indicate that the mesh behaviour is determined by several factors; some of which are controlled by each laboratory testing rig’s configurations. Therefore, it is recommended to compare the
resulting performance of mesh in relation to the testing rig configurations when reviewing the performance of a type of mesh from a particular testing rig.

Furthermore, the use of numerical modelling is a cost and time-efficient way of investigating the mesh performance that accounts for a range of factors. It provides a set of controllable numerical tools for its users to potentially help in better understanding how different ground support components work together to provide overall support in underground mines. The current model, however, captures a limited loading condition and boundary condition, as it is based on a specific laboratory configuration. However, to demonstrate the ground support performance in actual field conditions, the complete ground support system should be modelled. For this purpose, it is recommended to simulate the mesh behaviour under a laboratory test that accounts for the influence of reinforcement component.

Once the model successfully captures the mechanical behaviour of ground support components and their interactions, it is recommended that more complex loading mechanisms are incorporated into the model. This could be done by simulating the loading of rock using block elements. It is necessary, however, to overcome some challenges associated with the current model. One challenge with the current model is that it does not capture the “remote interactions” between blocks and beam structural elements. This means that the newly formed contact between the blocks of rock and the mesh cannot be represented.
List of References


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