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ORIGINAL PAPER

Angle Shear Testing of 15.2 mm Seven Wire Cable Bolt

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Abstract

This paper focuses on the experimental study of shear testing of 15.2 mm, 25 t capacity seven wire cables at zero, 30° and 45° angles using two different shear testing facilities at the University of Wollongong (UOW) and the University of Southern Queensland (USQ) in Toowoomba. A circular double-shear rig MK-IV was used for testing cable perpendicular to the sheared joint faces (zero angle of orientation), while testing the cable at 30° and 45° was carried out using a larger-size rectangular-shaped rig. Testing was carried out based on the double-shear testing methodology wherein cable bolts were fully encapsulated using Stratabinder HS inside of three concrete blocks representing host rocks. This study was part of the tri-universities-funded ACARP project C27040 awarded jointly to the University of New South Wales, University of Wollongong and University of Southern Queensland. The objective of the experimental testing programme was to provide the essential information for the development of numerical models that included not only the technical parameters, but also the behavioural outcomes from various tests with respect to the angles of testing and their effect on the nature of cable failure, be it pure shear, tensile shear or shear tensile, cable pretension and the credibility of the effectiveness of the Barrel and Wedge (B&W) anchorage system were evaluated. Laboratory facilities at both UOW and USQ were used in the study. The prepared double-shear samples were then positioned inside of compression testing machines and were subjected to shear testing. The values of shear load and displacement were recorded for various inclinations angles. It was found that increased angle of shear contributes to increased stiffness of the cable in shear with other parameters being equal. Subroutine codes were developed in UDEC and 3DEC to simulate shear behaviour of cable bolts installed in angles for different pretension loads. The numerical simulations indicated that UDEC and 3DEC can simulate the general shear behaviour of cable bolts reasonably well for various inclination angles and pretension values.

Highlights

- The historical information that sets the record on the early development and application of cable bolt on ground reinforcement along with the relevant literature review was presented,
- The results of large-scale experimental study on shear behaviour of cable bolts were presented and discussed,
- The results of large-scale experimental studies on shear behaviour of cable bolts installed in angles for various

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pretension values were presented and discussed in detail at a level that has not been investigated in previous geotechnical projects,
- The capabilities of distinct element code in modelling shear behaviour of cable bolts were studied for different geotechnical conditions by developing original subroutine programs, and
- Parametric studies were carried out to investigate the effects of relevant parameters on shear behaviour of cable bolts.

Keywords Cable bolt · Shear strength properties · Pretension load · Installation angle · Numerical simulation

1 Introduction

A significant number of researches reporting on shear testing of tendons have been cited in various publications for the past five decades. Much of the early research, with few exceptions, was focused on solid rock bolting, using direct single-shear rig method. Dulacska (1972) initiated single-shear testing of dowels, and she was then followed by Bjurstrom (1974), Hass (1976) and Azuar (1977) with studies on the application of angled rock bolts to jointed rocks reinforcement; others then followed (Fuller and Cox 1978; Dight 1982; Egger and Fernand 1983; Ludvig 1984; Ferrero 1995).

The first use of cables for rock reinforcement was in South Africa in 1964 and was closely followed in Australia in 1968 by Robin Friday, a civil engineer working at the North Mine at Broken Hill Consolidated limited. It involved installing an array of five smooth 12.5 mm cables, cement grouted in downholes to pre-reinforce a block in the floor which was to eventually be undercut to form a crown pillar (Fuller 1983). Clifford (1974) reported on rock bolt support at New Broken Hill Consolidate Limited (Fuller, private communication). Fuller and Cox (1978) work, especially in the design procedure for rock mass reinforcement being based on the concept that reinforcement should limit displacements at structurally weak region of the rock mass. Their theoretical approach included consideration of bolt orientation at different angles.

Stillborg (1984) PhD thesis from Lulea, Sweden, on shear studies used 15.2 and 38 mm diameter cable bolts subjected to single-shear testing in 40 MPa concrete at different angles of orientation. Stillborg found out that testing of 15.2 mm cables at 45° orientation to the shear plane gave a more effective resistance to shear than cables oriented at 90° to the shear plane. Few tests were then reported in the 1990s, which allowed cable bolts to be loaded to final failures. These include the works of Windsor (1992), on shearing of double-embedded cable bolts and Dobe (1996) on the study on the behaviour of cable bolts the behaviour of cable bolts subjected to combined tensile and shear loads; and Goris et al. (1996) on shear behaviour of cable bolt supports in horizontally bedded deposits, and Aziz et al. (2003) reported on double-shear testing of rock bolts in both 20 and 40 MPa concrete with the direction of shearing force acting perpendicular to the axis of the bolt (0° orientation). Grasselli (2005) reported on angled shear testing of solid rock bolts in a large size 60 MPa double-shear concrete rig. Others undertaking further studies on single- and double-shear tests and in general load transfer mechanisms of tendons include Hyett et al. (1992a, b, 1994, 1995a, b, 1996), Jalalifar (2006), Jalalifar et al. (2006), Jalalifar and Aziz (2010) and more as reported by Hutchinson and Diederichs (1996).

The intensity of focus on research on cable bolting at UoW began again in earnest in 2009 with double-shear testing of 28 mm diameter TG plain cable bolt (Jennmar product with yield strength of 564 kN) using a newly constructed larger-size double-shear apparatus by Craig and Aziz (2010) and Aziz et al. (2014). The double-shear testing methodology is a symmetric test, whereby the shear strength properties and behaviour of cable bolts due to the strata movement are evaluated for various pretension loads and host rock strengths. The testing equipment is design in a way that can record the evolution of axial load during shearing due to the cable deformation. This initial study paved the way for a prolonged research on cable bolt shearing as reported by various research papers (Aziz et al. 2016a, b; Mirzaghorbanali et al. 2017; Li et al. 2017; Rasekh et al. 2016). Also tested was the single-shear testing of cable bolt using the British Standard single-shear (BS 2009) method by Aziz et al. (2015a). A much larger-size version, the Megabolt Integrated Single-Shear Test Rig (MISSTR) was reported initially by McKenzie and King (2015) on testing of several Megabolt cables bolts. The MISSTR was later used by Aziz et al. (2018, 2019) for the study of the shear behaviour of various cable bolts marketed in Australia. This project was funded by the Australian Coal Association Research Program (ACARP) (Aziz et al. 2017). The work on shear testing of cable bolts was carried out mainly by shearing perpendicular to a cable bolt axis installed perpendicular across a sheared joint face. The focus of the ACARP (C 42012) project study, in addition to the shear characterisation of the tested cables was a comparative study of bonding and debonding of cables with respect to cable strand wire surface. The study found that plain wire cables were readily debonded in comparison with indented cables for the same grout encapsulation length. All tests were undertaken with the direction of applied shearing load perpendicular to the cable axis, or the cable being installed perpendicular to the sheared joint face.

In April 2017, a two-day industry workshop on ground support was held at UNSW attended by key ACARP's Underground Research Committee members, Industry researchers, consultants, bolting companies' suppliers. At the workshop, a number of critical issues were identified that would shape future research directions for effective ground control in mines. One of the aims of the workshop was better understanding of the roof failure mechanisms by developing a numerical modelling tool that will enable a broad range of factors to be evaluated in terms of their contribution to the failure mechanisms of underground workings. In particular, there was a need for the development of numerical modelling approach to better understand the effect of cable bolt performance on roof failure mechanisms in varying rock mass conditions. Accordingly, this paper reports on both the laboratory study on angle shear testing of three selected cable bolts at 0°, 30°, 45° respectively and the development and application of the numerical modelling simulation to meet the desired objective of this study. The focus of the distinct element numerical modelling exercise was to obtain a better understanding of the load transfer mechanisms of cable bolts with different installation angles and pretension loads (according to the experiment setups), and to perform sensitivity analysis of critical factors.

Findings from this study shed light on the effects of installation angle on shear load transfer mechanisms of cable bolts for various pretension values. The experimental study was associated with the numerical simulation, conducting a comprehensive sensitivity analysis on parameters related to the cable bolts, grout and installation practice.

2 Experimental Procedure

The essential objective of this study was to examine angled shear strength characteristics of 15.2 mm 7 wire, 25 t capacity cable bolts at zero, 30°, 45° orientation, two different double-shear apparatuses were used for the study. The choice of the method was based on the angle of the cable inclination with respect to sheared joint faces. The circular-shaped double-shear apparatus Naj Aziz Circular Double-Shear Rig, known as MK-IV DS rig, was used to evaluation the shear behaviour of the tested cable at 0°, where the cable is laid perpendicular to the direction of shear. However, testings of the cables at 30°, 45° were made using larger-size double-shear apparatus, adopted from the double-shear testing method similar to that used initially by Grasselli (2005) for testing solid rock bolts.

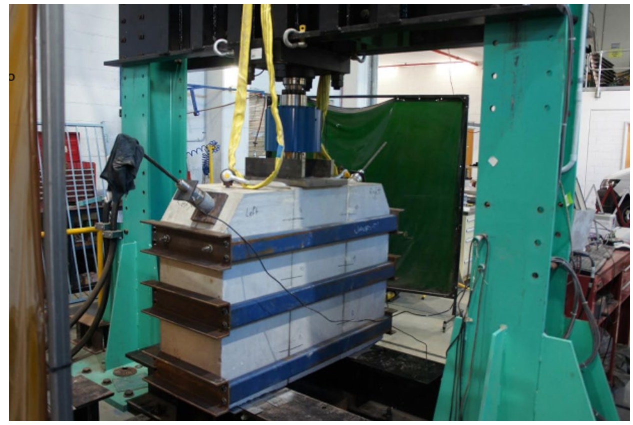


Fig. 2 General view of the initially proposed assembled double-shear testing concrete with predetermined angle of 30° and 45° to the sheared joint face

2.1 Double-Shear Testing of Cable Perpendicular to the Joint Face

This test involved using the MK-IV Double Shear Box, known as the Naj Aziz Double Shear Box. The rig is a cylindrical version of the double-shear testing devices developed over the years at the University of Wollongong (Aziz et al. 2003, 2015a, 2016b, 2019). In this model, efforts were made to eliminate the effect of friction between the middle block and side blocks using the Lateral Truss System (LTS) as reported previously by Aziz et al. (2019). The LTS system consisted of two sets of lateral restraints attached to side plates (5) in Fig. 1. The spacing between concrete blocks is 5 mm. In the study, only one cable will be subjected to shearing perpendicular to the sheared joint faces. Further discussion on test results and analysis will be reported later in the paper.

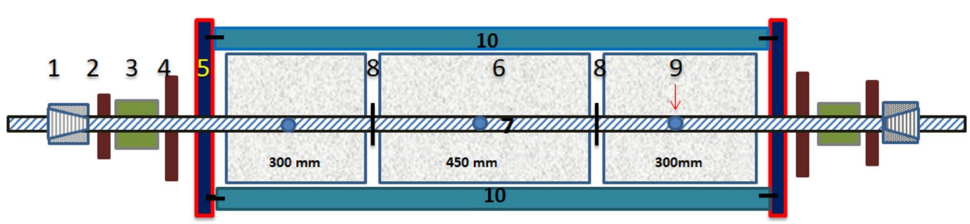


Fig. 1 Double-shear rig (MK-IV)

1: Barrel & Wedge, 2: Outside plate, 3: Load cell, 4: Inside plate, 5: LTS Side steel plate, 6: Concrete block, 7: Bolt (cable bolt), 8: Ring Packers (10 mm), 9: Grouting hole, 10: LTS

2.2 Double Shear Testing of Cables at Inclined Angle

The second method of testing involved the testing of cables at 30° and 45° inclinations. This new arrangement consisted of shear testing two 15.2 mm cables in larger size rectangular concrete blocks as shown in Fig. 2, and is similar to testing of solid rock bolts as reported by Grasselli (2005).

Figure 3 shows the schematic drawing of the proposed angled set up for shear testing of cables at 30° and 45° respectively to joint faces in concrete blocks. This paper covers both the experimental study of the shear testing of 15.2 mm, 25 t capacity and seven wire cable bolts, which are used for modelling simulation reported later in the paper.

2.3 Concrete Blocks Construction

Double shear circular concrete blocks casting: The circular concrete blocks are cast in 300 mm diameter Formatube cardboard cylinders. Two 300 mm and one 450 mm cardboard lengths are cut and assembled in a specially prepared wooden frame for a concrete pour as shown in Aziz et al. (2019). During casting of the concrete and the production of the rifled central hole for cable installation, a steel bar wrapped with 8 mm PVC tube is held vertically along the mould to precast a rifled hole through the centre of the concrete blocks. Once the concrete was poured it was left to set and harden, the steel conduit as well as the PVC tube are removed in a similar fashion as reported by Aziz et al.

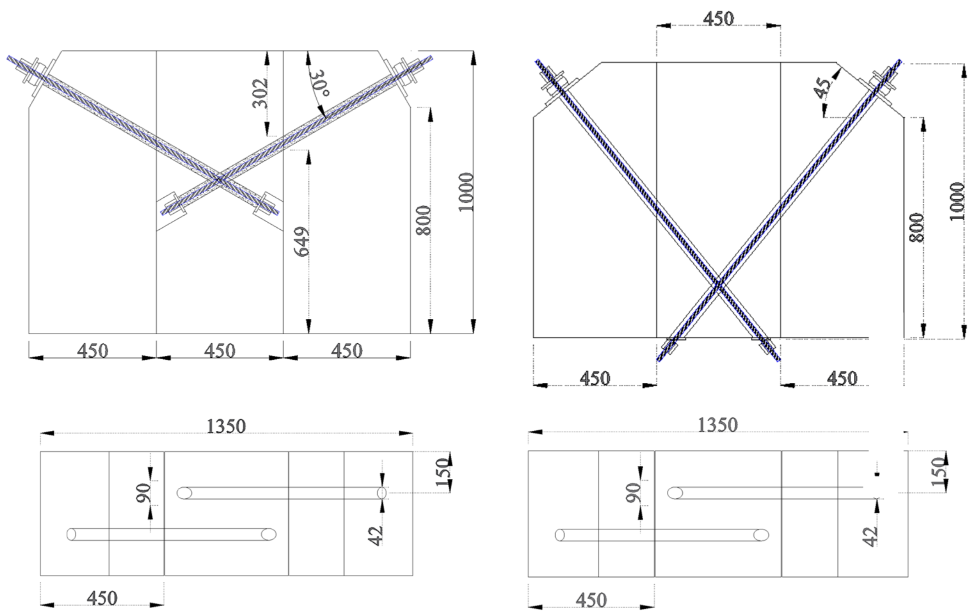


Fig. 3 General view of the initially proposed assembled double-shear testing concrete with the angle of cable installation at 30° and 45° to joint faces (dimensions in mm)

Fig. 4 Preparation of the cylindrical concrete blocks and casting of rifled holes in concrete block using PVC flexible tubes wrapped around the central steel rod

(2017) in ACARP project report C24012. After a 28-day curing time, the concrete blocks are removed from the cardboard and mounted on the double shear circular frames for shear loading. Figure 4 shows the process of concrete casting and central hole creation.

Concrete blocks construction for 30° and 45° angle shearing: Three water-resistant, 15 mm thick marine plywood frame moulds, were constructed for casting concrete blocks for double-shear testing at both 30° and 45° angles of bolt inclination. Figure 5 shows a typical set of moulds used for the preparation of blocks for the 45° orientation.

Each concrete block has the dimensions of $450 \times 450 \times 1000$ mm^3. Due to the size of the samples, external confinements were replaced by internal rebar frames cast inside each concrete block sample to resist tensile cracks of the concrete as shown in Fig. 5a. The tensile cracks may occur due to the load transfer from the cable to the blocks if the concrete was not reinforced internally. The cracking may reduce the overall shear strength and increase the peak shear displacement of the system as the sheared cable begins to follow the cracked voids. In 30° angle, both cables protrude from the sidewalls of the middle block, while in 45°, cable ends reach the bottom end of the middle block as illustrated in Fig. 3. This technically does not affect the test procedure, even though, pretensioning of bolts would be more challenging due to the uplifting effect of the pretension force applied on the middle block.

To cast angled holes in the concrete blocks, two steel conduits, of appropriate diameter, were wrapped with an 8 mm PVC tube and inserted diagonally in the mould holes in a similar fashion to that shown in Fig. 3. Two cables were mounted in each set of the cast moulds at different lateral locations so that the space between them was maintained constant at 100 mm, thus leaving the cable anchored at 150 mm from concrete sides (Fig. 3). The location of the cast holes in the concrete blocks varied depending on the bolt inclination/orientation with respect to the sheared joint faces. Depending on the inclination of the hole, the lower end of the installed cable, with its B&W, would protrude from beneath the central concrete block, in the case where the hole inclination was 45°. For 30° encapsulated inclination, the cable end with its B&W will end up housed on the side socket of the centre block. A 3D printer was used to print plastic cups for the purpose of creating side socket holes in the cast concrete. Once concrete holed blocks were cast, they were then held together and laterally reinforced by three sets of Lateral Truss System (LTS). Each set of the reinforcement truss system consisted of a 9 mm thick, 100 mm × 60 mm rectangular steel tube butted on either side to 10 mm thick 90 mm × 150 mm open steel channels. The real aim of the LTS was to counteract the forces generated by cables being pretensioned and the subsequent double-shear loading of the central block. This applied counter force resisted the applied pretension forces in squeezing the central block, thus minimising the influence of contact friction between joint faces. Also, the LTS, shown in Fig. 6, was (a) to prevent individual blocks from twisting laterally and (b) to prevent the central block from lifting upwards. The later point (b) proved difficult to prevent at high pretension loads; however, the central block uplift during cable pretensioning up to 50 kN was possible with the addition of a 10 mm thick and 150 mm wide open steel channel (green) placed over the three blocks during cables pretensioning, and grouting stage as shown in Fig. 6. Sets of LTS were not strain gauged in this research study. It is recommended to instrument lateral support systems in future studies, investigating the load variation during shearing.

The bolt inserted in each hole was fitted on the lower side with a small load-bearing plate and then secured by a B&W. Each cable was held firmly from the cable top end to hold the B&W in place at the lower end. This protruding topside cable end was fitted with a steel plate and a 75 t capacity load cell to be followed with a B&W and gently pretensioned. The same procedure was used in the installation of the second cable on the opposite side of the three blocks. During shearing, each cable was simultaneously pretensioned separately using two identical Blue–Heeler tensioners to avoid blocks twisting. Once the cables were pretensioned, it was followed by injecting grout (Stratabinder HS grout) from the top short holes to grout cable bolts down into each instrumented cable bolt hole. Any leakage from the packers was checked and the cable tightening level was maintained for the duration of the grout hardening.

3 Testing Process

A total of six tests were carried out on the 15.2 mm cable bolt. One test was made using the standard MK-IV double-shear rig, one test was made at a 30°-angle, and the other four tests were carried out on cable bolts with an orientation of 45°.

3.1 Double Shear Testing of 7 Wires 15.2 mm Cable Bolt at 0° Inclination

Figure 7 shows the circular double-shear rig for testing cable at 0° orientation and Fig. 8 shows the load–displacement profile of the tested cable, together with the axial load generated during the shearing process. The shear load per sheared joint plane was 157.0 kN (314 kN = 157 kN per sheared side). The axial load (pretension) at peak load was 53 kN. Further comparative analysis of this test with other angled shear cables will be addressed in the discussion section.

Fig. 5 **a** structure of reinforced casting mould for 45° cable bolt inclination. **b** Concrete cast in the mould

Fig. 6 **a** Schematic drawing of assembled DS box with constraining mechanisms. **b** readily assembled rig of the large double-shear box

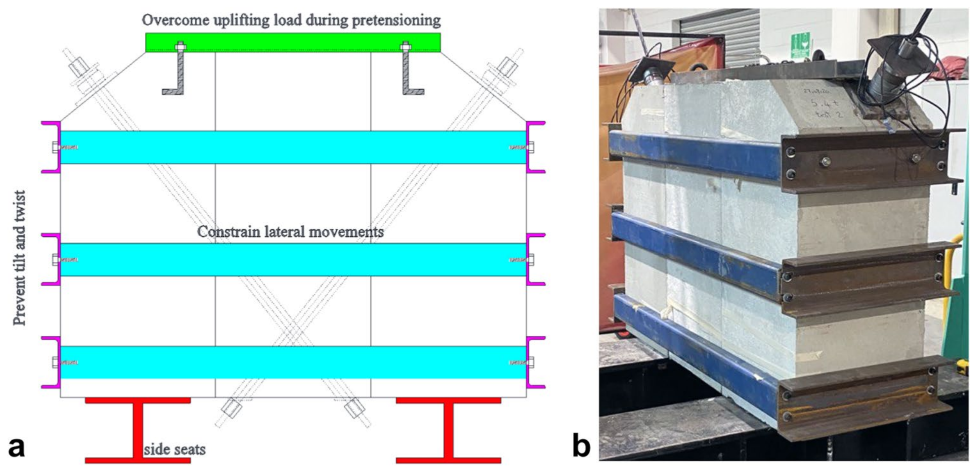
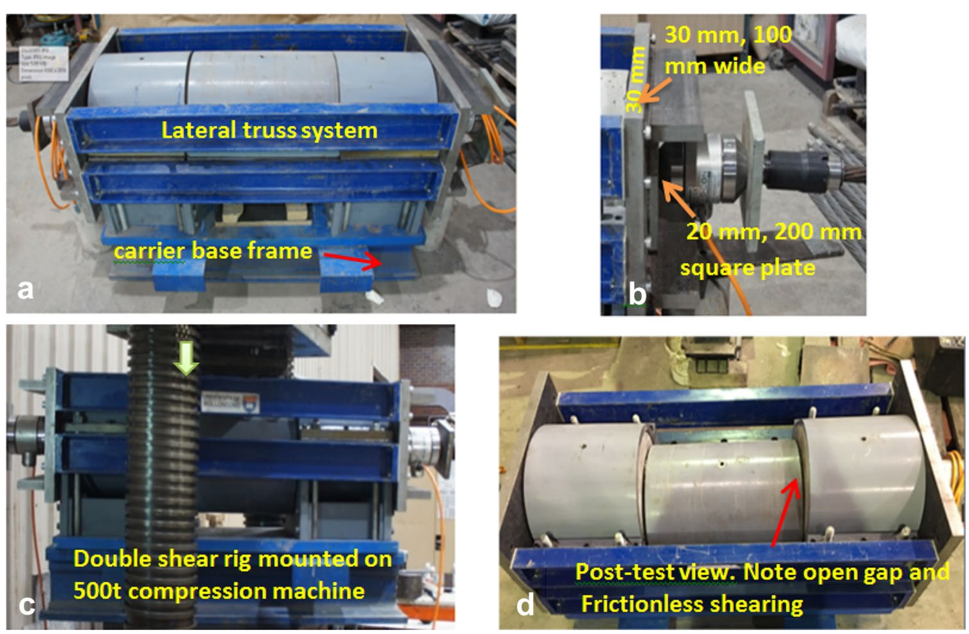


Fig. 7 Circular double-shear MK-IV teting rigs for testing 15.2 mm cable bolt at 0° of inclination



3.2 Double Shear Testing of 7 Wires 15.2 mm Cable Bolt at 30° Inclination

Figure 3 shows the schematic drawing of the cable installation at 30° angle of installation in DS blocks. The 30° inclination arrangement is different from 45°, where B&W anchors do not reach the bottom of the central block.

The peak shear load at cable failure of 508 kN occurred at a shear displacement of 27 mm. During the early stage of the system loading, the compression machine malfunctioned for the first few minutes. The fluctuation in axial load–displacement during this early stage of shearing, shown particularly in the axial load–displacement profiles shown in Fig. 9 was stabilised and the testing continued uninterrupted until reaching the cable peak load of 508 kN.

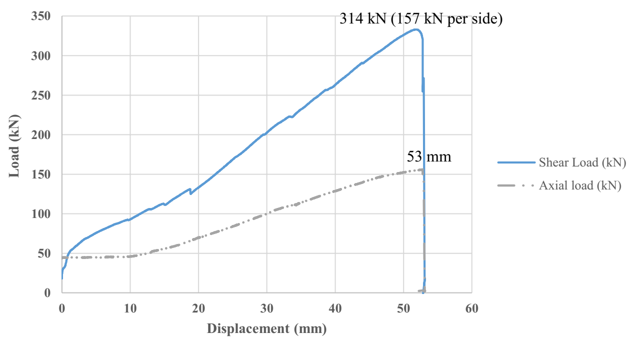


Fig. 8 Load–displacement profile of the testing 15.2 mm cable bolt cable, together with the axial load generated during the shearing process. Tests were made in the MK-IV double testing rig

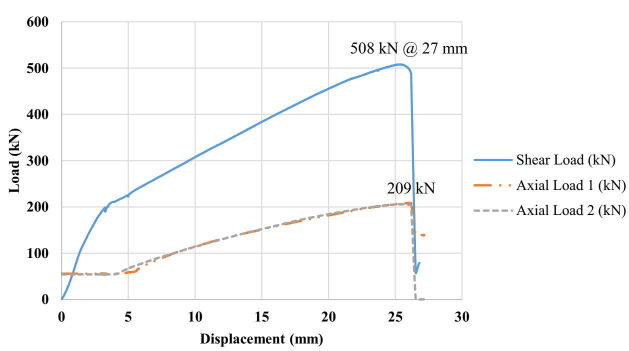


Fig. 9 Load–displacement profiles of testing 15.2 mm Cable bolt at 30°

3.3 Double Shear Testing of 7 Wires 15.2 mm Cable Bolt at 45° Inclination

Four tests were carried out at a 45° inclination as shown in Table 1. The first test was carried out in the 60 t capacity compression loading frame. Figure 10 shows the assembled 40 MPa concrete blocks. Both cables pretensioned at 20 kN. The lower level for the initial cable pretension load was necessary because the pretensioned force of the cable bolts was not in line with the counteracting opposite load generated by the LTS force, thus pulling the central block upwards slightly and causing it to shift. The lateral truss system consisted of three unit sets of 9 mm thick closed box rectangular steel channels (Blue colour side straps) bolted on each end to 9 mm thick and 150 mm wide open steel channels.

Once pretensioned, the double-shear block set-up assembly, weighing around 1.4 t was mounted on the compression machine loading plates with its outer concrete blocks resting on H-shaped steel channels, leaving the middle block free to be sheared down. During the initial phase of the first load application on to the central block, the 300 mm^2, 25 mm thick load steel bearing plate placed beneath the ram over the central concrete block caused a slight crushing of its top surface (Fig. 10a). This was soon

Table 1 Shear test results of 15.2 mm cables

Test	Angle (°)	Cable axial pretension (kN)	Shear load (kN)	Shear displacement (mm)	Comment
1	0	50	314 (157 per each side)	52	One cable used for double-shear test, hence the peak load per joint face is 157 kN
2	30	50	508	25	Cable wires failure mostly in tension with cone and cup ends
3	45	50	572	21	Centre block top surface damaged during early shear loading stage. The first test in the angle shear program did not involve anchoring and bolting outer concrete blocks to the LTS
4	45	0	581	39	Grout leakage during cable encapsulation and loss of pretension load during transportation from Wollongong (NSW) to Toowoomba (QLD)
5	45	50	501	28	Cable wire failure mostly in tension
6	45	40	485	24	Cable wire failure mostly in tension

Fig. 10 45 mm diameter cables encapsulated at 45-degree angle being ready for shear testing. **a** 300 mm^2 loading base plate. **b** 500 mm × 400 mm loading base plate

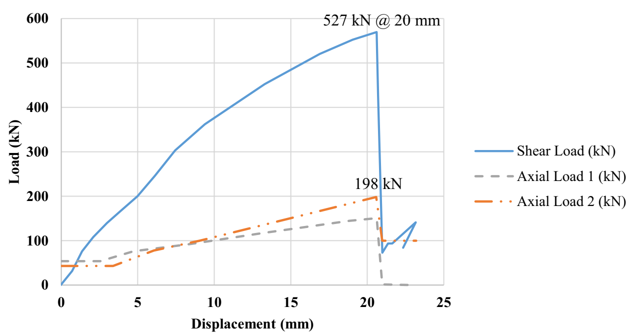


Fig. 11 Load–displacement profiles of 15.2 mm diameter cable bolts at 45-degree inclination

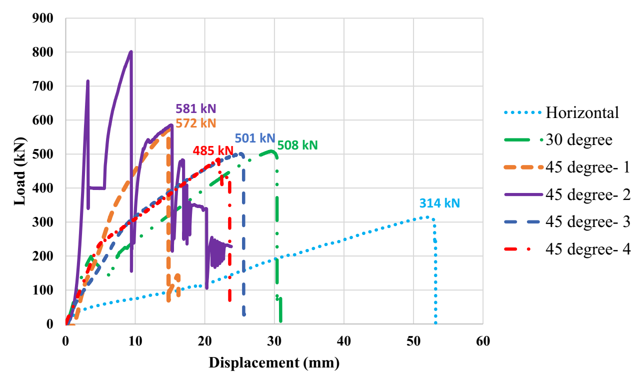


Fig. 12 Load–displacement profiles of testing 15.2 mm diameter cable bolts at different orientations (angles)

overcome by pouring a layer of plaster of Paris gypsum solution over the crushed surface, trowelled smooth, followed by placing a 30 mm thick steel plate large enough to cover the whole surface area of the middle concrete (Fig. 10b), thus allowing the applied load to be distributed evenly on the top side of the centre block. Information retrieved during the loading process from both the vertical ram and from load cells on the angled bolts was processed and displayed on a PC monitor screen. The loading rate was maintained at 1 mm/min. Figure 11 shows the load–displacement graphs of the applied load and axial forces generated for both bolts. The axial load variation may have been caused by the initial crushing of the centre block at surface.

4 Discussion

Figure 12 shows the load–displacement graphs of all 15.2 mm cable bolts tested at angles 0°, 30° and 45°. The effect of the cable's inclination and load transfer mechanisms in all six tests are clearly evident. Only one test out of the six 15.2 mm cable type tested (four at 45°, one at 30° and one at 0°) was carried out at the USQ Toowoomba laboratory, others were carried out at the UOW laboratory. While testing campaign was carried out in USQ and UOW laboratories using different compression testing machines, all experiments were carried out based on the same testing methodology incorporating fully calibrated load cells and computerised data taker. Test results revealed that the higher the bolt angle of inclination to the sheared joint plane, the greater the ultimate failure load of the cable bolt. In other

Fig. 13 Centre block shear displacement in which booth cable have failed symmetrically. Note the snapped and smeared leaked grout between concrete joint faces

words, the ultimate breaking load of the cable bolt increases with the increase in the angle of inclination. This is due to the fact that the applied shearing force tends to pull the cable wires more axially rather than in shear with increase in the angle of inclination. It is of interest to note that all cables tested in this research study failed in combination of shear and tension. Pure shear failure may occur in guillotine box shearing wherein cable bolt is only loaded in shear direction. With the exception of the first test in which the top of the middle concrete block was damaged slightly, none of the blocks were found to end up cracked or split. In many cases, both cables snapped fully with excessive vertical movement of the central block as shown in Fig. 13. The shear testing was continued until the cable bolt failure upon which the experiment was terminated. It should be stated that the peak failure load per each cable side was recorded as 157 kN for the test, wherein the cable was installed perpendicular to the shearing surface.

The shear failure load per cable in the first test at 45° inclination was in the order of 286 kN (total load for both sides 572 kN). This level of spent force was greater than the tensile failure load of the cable of 250 kN (25 t). It is important to note that two cables are sheard in double-shear testing arrangement as shown in Fig. 3. Therefore, the total nominated tensile capacity for the system is in the order of 500 kN for both sides. This increase in loading force is likely to be attributed to; (a) the inefficiency of the two 100 mm wide single sheet Teflon strips instead of double sheets, sandwiched between joint faces; (b) that the lateral truss reaction force was not in line with the applied encapsulated cables force axis against the vertical shearing of the centre block, and (c) the additional force needed to overcome the butted joints face friction. However, this additional lateral friction force between the concrete sides may not reach fully the estimated 30% of the applied shear force, based on the Mohr–Coulomb Fourier series mathematical model as report by Aziz et al. (2016b) and by dynamic testing reported by Khaleghparast et al. (2020), but will be of a significant amount proportional to the size of Teflon strips.

It is worth noting that similar tests were reported in Stillborg's PhD thesis where shear testing of 15.2 mm cable bolts at a 45° inclination in a single-shear test and without sheared joint faces coming in contact with each other were undertaken. The average shear value reported by Stillborg was 206 kN (20.6 t). The sheared cable was encapsulated in 600 mm^3 granite rock (UCS of 234 MPa) embedded in a 60 MPa concrete with Elastic modulus of 38,730 MPa (Stillborg 1984).

The failure pattern of various failed wires in the 15.2 mm cable testing was mostly in tensile/ shear as shown in Fig. 20. At 45° of shear testing in USQ (Toowoomba), all wires were snapped from both cables. All wire failures in both cable strands are clearly from the result of the excessive level of centre block downward travel during the shearing process, as shown in Fig. 13. The pattern of observed broken wires was typically in tensile shear as shown in Fig. 14.

Comparing all six test results at different orientations, shown in Fig. 12, the following was inferred;

- The shear displacement, at peak shear load failure, increases with reduced angle of orientation,
- In shear testing of the cable at a zero angle of inclination, the shearing was carried out on a single cable, not two as would be the case with angled shearing. The shear load–displacement is higher because of the nature of double-shear testing with short side concrete blocks, and the fact

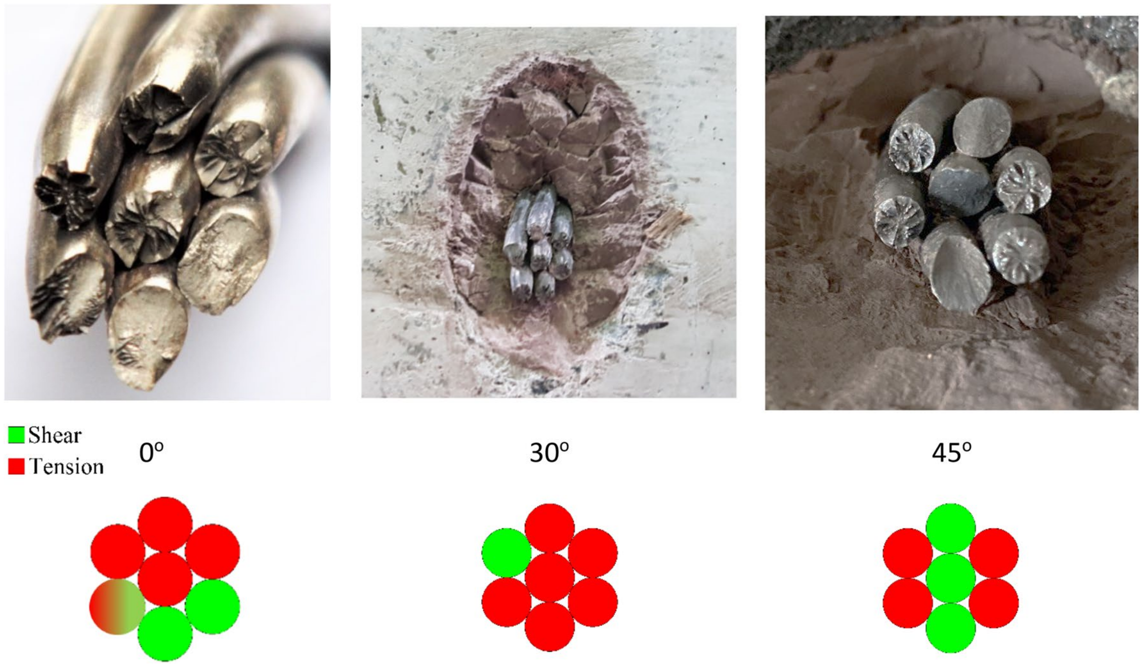


Fig. 14 Various snapped view of 15.2 mm cables tested for shear at 45-degree inclination

that the cable will undergo elongation at each hinge point in the vicinity of both joint planes. Recent research by Aziz et al. (2020) has found that the level of displacement is less with internally reinforced concrete medium and with fully secured ends.

- The blue load–displacement graph (4) shown in Fig. 12 of the 15.2 mm diameter cable bolts tested in USQ (Toowoomba) at 45° orientation indicated that the first of the two peak loads was due to the forces required to separate cemented joint faces from each other, caused by the leaked grout from the seal packers sandwiched between the sides of three blocks during cable grouting process *as* shown in Fig. 13 in which the leaked grout smeared across joint faces. The third peak shear load is attributed to the snapping of the cable at 571.6 kN.
- The brown graph (3) is the first test carried out at the UOW at 45° inclination. It has a peak failure load of 572 kN, which is almost identical to that obtained and shown in the blue graph. This near equal shear failure load demonstrates the consistency of the cable shear value at 45°.
- The purple graph (3) is shear values at 30° cable orientation carried out at the UOW,
- The orange graph (4) is obtained from the standard double-shear testing using the Naj Aziz circular double-shear rig with the direction of shearing perpendicular to the cable axis 'zero angle' as shown **in** Fig. 7.
- The red (5) and green (6) graphs are additional graphs at 45° tested lastly at the UOW**.**
- In general, the increased angle of orientation at 30° and 45° caused cable bolts to fail in tensile shear as shown in Fig. 14, which is evident from the snapped failed cable strand wires. It is important to mention that as the angle of installation increased, the contribution of tensile failure was more pronounced.
- The peak shear load failure of the majority of15.2 mm cables at 45° inclination has demonstrated the consistency and reliability of the test results, which adds to the credibility of the experimental study undertaken.

5 Numerical Modelling

The numerical simulation approach to mimic the laboratory testing was developed. UDEC and 3DEC—as members of ITASCA family—were chosen for the 2D and 3D modelling. In general, they are mostly suited for use in cases where the presence of the discontinuity is taken into consideration. Furthermore, different material and joint models alongside various support elements have been embedded into the software and are ready to use. The useful feature of the software is the ability to utilize deformable and rigid blocks, which in turn can increase modelling efficacy. With built-in programme language, FISH, the software is capable of being expanded to include user-defined features, constitutive models, and capability.

5.1 Modelling Concept

To simulate cable bolts in UDEC and 3DEC, the reinforcement element comprises Local and Global concepts contingent upon the phenomena they are portraying in the numerical model. The Local reinforcement elements are representative of the localized damage area in the vicinity of the intersection of the cable and the discontinuity plane. This situation only takes place when the bolt is fully encapsulated in a strong and stiff confining medium (grout and rock). These kinds of systems usually possess a high level of resistance to axial and shear movement because the bond strength is so high that two hinges form. Beyond these hinges, the rest of the system including the reinforcing element, grout, and rock, is intact and undamaged. On the other hand, the Global elements try to simulate a less strong bond and a less stiff surrounding medium. This means a substantial length of the reinforcement is affected by the movement and the load build-up. The damage is not localized here. In another words, the aforementioned case occurs when there is debonding along the cable or bolt. Hence, in the cases where barrel and wedges are used and/or debonding is not observed, a Local concept is used and, for the other cases, a Global logic is used.

Model variables for different cable bolt types are assigned using single-shear (ACARP-C42012) and pull-out (ACARP-C24018) tests. Table 2 introduces typical variables and the proposed method of acquiring these variables.

The numerical simulation study is intended to evaluate the capabilities of local and global reinforcing elements in simulating cable bolts shear load transfer mechanisms for various installation angles and to carry out parametric study.

5.2 Base Model Simulations

Several preliminary and calibration models were developed to simulate shear load transfer mechanisms of various cable bolts with different pretension loads based on single-shear (ACARP-C42012) and double-shear testing (Mirzaghorbanali et al. 2017) methodologies. Original subroutine codes were developed to model the geometry and boundary conditions of single- and double-shear testing models in 2D, and 3D as shown in Fig. 15. The lengths and width of the single models were 1900, 250 mm, and the sizes of the double-shear models were 1050, 300 mm, respectively. Portions of the left cylindrical block (single shear) and side blocks (double shear) were constrained using pin supports in the *X* and *Y* directions. The middle or side block was subjected to

Table 2 Typical input variables of reinforcement element

Keyword	Definition	Unit	Reference
r astiff	Axial stiffness	[force/length]	Pull out test
r length	Active length (half)	[length]	Single-shear test
r sstiff	Shear stiffness	[force/length]	Single-shear test
r str	Axial failure strain		Pull out test
r uaxial	Ultimate axial capacity—pull-out	[force]	Pull out test
r ushear	Ultimate shear capacity	[force]	Single-shear test

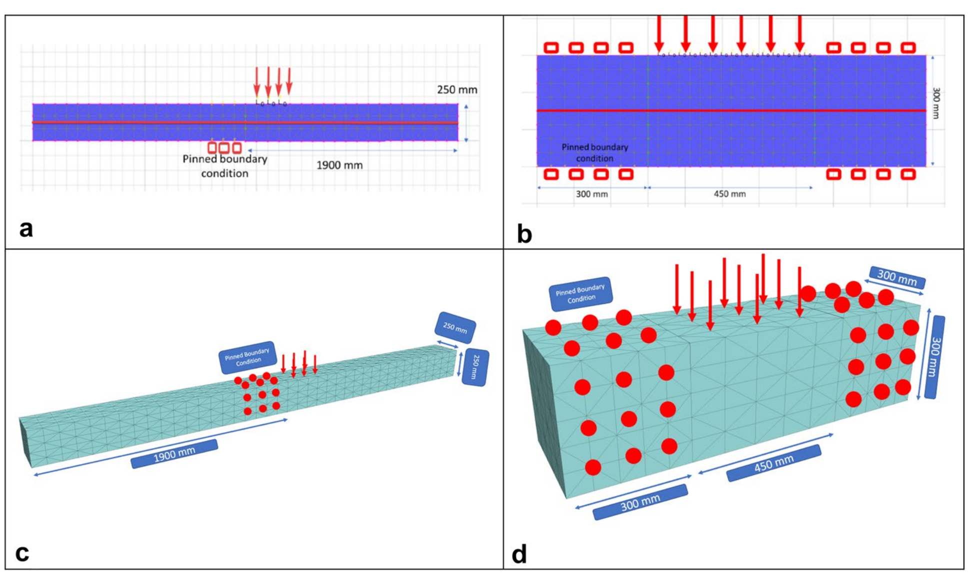


Fig. 15 Geometry and boundary conditions of base models: **a** single shear testing in 2D. **b** Double-shear testing in 2D. **c** Single-shear testing in 3D. **d** Double-shear testing in 3D

Table 3 Typical input variables and properties of cable bolts used for the numerical simulations

Cable	Tensile strength (kN)	r_ushear (N)[a]	r_sstiff (N/m)	r_uaxial (N)	r_astiff (N/m)	r_str	r_length (m)
MW9S (15 tonne)	620	5.00E+05	1.16E+07	4.70E+05	5.90E+07	1.75E−01	3.10E−02
ID SUMO (0 tonne)	630	4.60E+05	9.38E+06	4.90E+05	4.70E+05	1.75E−01	5.60E−02
Garford (0 tonne)	530	4.40E+05	9.30E+06	5.00E+05	4.00E+07	1.44E−01	6.08E−02

[a]Refer to Table 2 for variables definition

Table 4 Typical input variables and properties of concrete

Uniaxial compressive strength (MPa)	Density (kg/m^3)	Bulk modulus (N/m^2)	Shear modulus (N/m^2)
40	2500	1.651e^{10}	1.239e^{10}

vertical loads and the values of shear load and displacement were calculated using the special code developed as part of this research study. Typical input variables and model properties of cable bolts and concrete are given in Tables 3 and 4.

Typical results of the typical 2D (UDEC) and 3D (3DEC) numerical simulations for the single-shear and double-shear testing are shown in Fig. 16. In these figures, the shear load (kN) is shown against shear displacement (mm). As shown in Fig. 16a and b, UDEC can simulate the general shear behaviour of cable bolts. However, discrepancies are noted for capturing the peak shear load and displacement for some of the cases such as the Garford Cable with 0 t pretension (Fig. 16c). In addition, it appears that the Global concept can simulate the trend of shear behaviour with reasonable accuracy for the Plain Superstrand cable, where debonding occurred during large-scale single shearing. Nevertheless, the peak shear load and displacement were not precisely simulated using the global concept (Fig. 16d).

Results suggest that double-shear tests (where cable bolt debonding is prohibited by the means of barrel and wedge) can be reasonably simulated using UDEC based on the Local logic. The results of numerical simulations are in good agreement with the experimental data of Plain Superstrand (0 t pretension), ID SUMO (0 and 15 t pretension) and MW9S cables (7.5 t of pretension). Nevertheless, the peak shear displacement has not been accurately modelled for the Plain Superstrand cable with 16 t of pretension values. This can be related to the fact that cable failure may not co-occur in the left and right sides in the double-shear testing methodology.

It is concluded from the results of the numerical simulations (Fig. 16e and f) that 3DEC is similar to UDEC and can simulate the general trend of cable bolt shearing behaviour based on the large-scale single shearing methodology. However, the peak shear load and displacement may not be exactly captured in all cases. 3D simulations can be further extended to study azimuth and plunge for various cable bolts across a shear surface which were not part of this research study.

5.3 Simulations of Cable Bolts Installed at Various Angles

Experimental tests carried out in this research study were simulated using the subroutine codes developed in base modelling section. It is noted that the model properties and coefficients remained the same as those in previous section, and only the model geometry, the boundary conditions, cable bolt installation angle and the pretension load value varied, based on the experimental conditions.

Typical model geometry and boundary conditions for 2D and 3D numerical simulations are shown in Fig. 17. The boundary conditions on the side block are designed in a way that represents each experimental test and varies from test to test. Roller supports on the sides allow vertical directions while top and bottom rollers only allow lateral and horizontal movements.

Typical results of 2D and 3D numerical simulations for cable bolts installed in angles are shown in Fig. 18. In the below figure, shear load (kN) is shown against shear displacement (mm).

In general, a reasonably good fit in terms of peak load and displacement was achieved for 2D simulations. Nevertheless, the results of numerical simulations are lower than those of the experimental data for the elastic and early stages of plastic deformations for the 15.2 mm cable. This can be ascribed to the concrete and grout damages due to the cable shearing, for which a portion of the shearing energy is dissipated. Here, it is worth stating that, due to high inclination angle at 45 ^{o} test, the imperfection of the samples and loading condition, both cables did not fail simultaneously, which accounts for the irregularities in the test graphs. Consequently, since the numerical solution does not take this phenomenon into consideration, some disparities were witnessed. In addition, it is important to note that the numerical simulations were carried out based the Local and Global reinforcing elements that do not simulate each strand individually. Grout

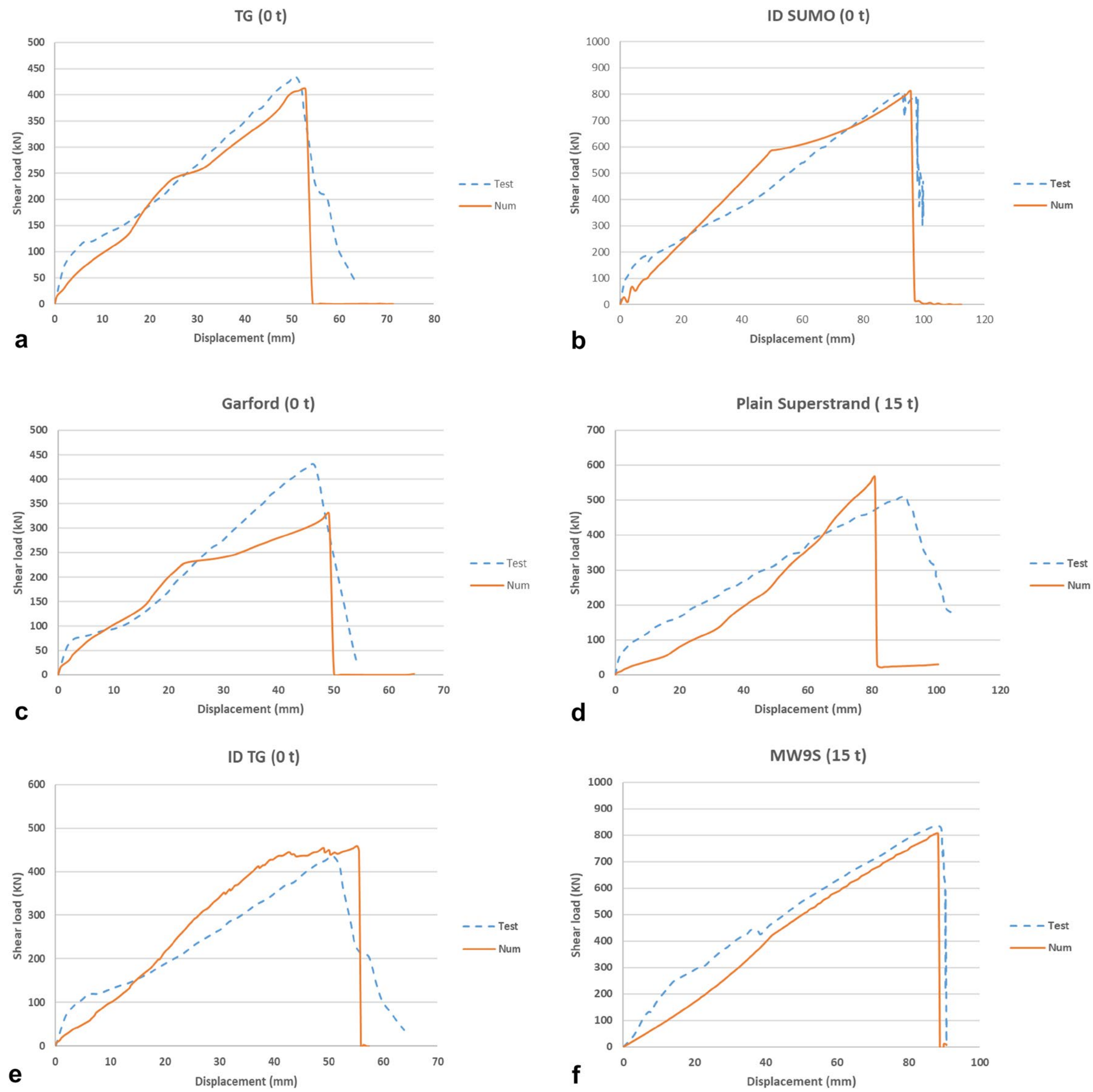


Fig. 16 Typical base model results and comparison with the experimental data: **a** ID TG—single shear—2D, **b** ID SUMO—double shear—2D, **c** Garford—single shear—2D, **d** plain superstrand—single shear—global concept with debonding—2D, **e** ID TG—single shear—3D, and **f** MW9S—double shear—15 t pretension—3D

mechanical properties were remained the same during shearing, but the load transfer mechanisms between the rock-grout and grout-reinforcing elements interfaces varied depending on the relevant constitutive models. These factors may have affected the results of numerical simulations.

5.4 2D and 3D Sensitivity Analysis

In this section, a series of typical graphs analyse the performance of each variable in the Local and Global reinforcement model that was used in this study. Figure 19 presents typical results of sensitivity analysis for 2D simulations.

It appears that an increase in axial stiffness results in an increase in the ultimate loading capacity of the system, which has also been verified previously in laboratory tests. In the shear case, however, these changes were more dramatic, and they tended to affect the loading path quite a lot in a way that the increase in shear stiffness results in the increase of the ultimate loading capacity of the system as well as a stiffer behaviour in the initial loading stages. In both cases, the final displacement was unaffected by changes in stiffness. It is inferred that an increase in axial load capacity (Fig. 19a) results in an increase in ultimate resistance of the system, which is in direct agreement with the experimental work. Similarly, the same is true for the shear case;

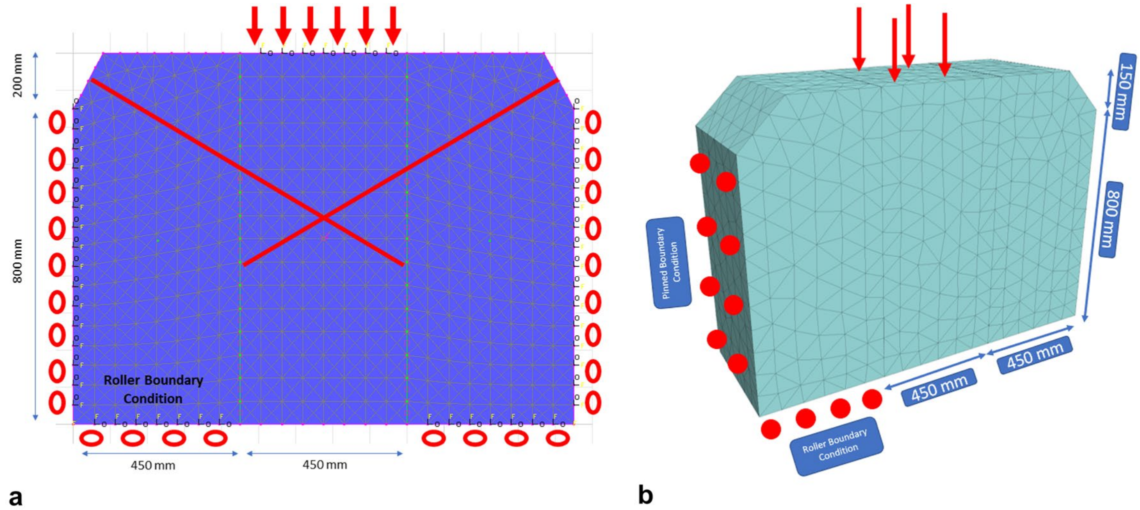


Fig. 17 Typical geometry and boundary conditions of cable bolts installed at angles: **a** Double-shear test with 30-degree inclination in 2D, and **b** boundary condition of double-shear test with 45° installation angle in 3DEC

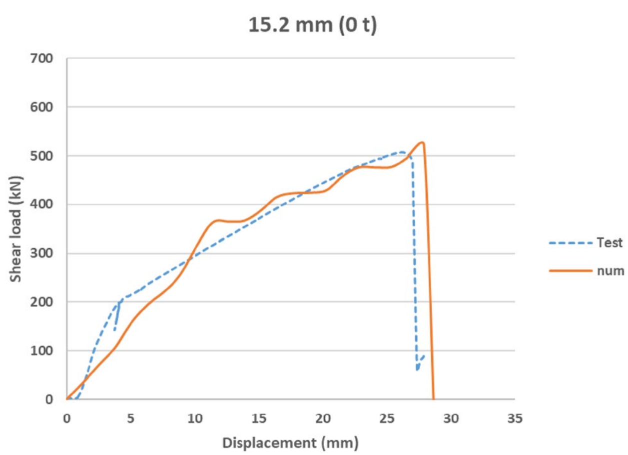


Fig. 18 Typical model results for inclined cable bolts and comparison with the experimental data (15.2 mm cable 30°—3D)

however, in this case there exist some increase in the final vertical displacement as well as the final resistance load, which again conforms to the current understanding of cable bolting concepts. In the case of active length (the distance between the hinge points), it is clear that an increase in this variable resulted in an increase in both ultimate load and final vertical displacement. This is in accordance with the experimental results too, because in a hypothetical situation in which two similar tests can be carried out with two different distances between hinge points, the one with a longer active length has more material engaged, thus more resistance to failure and more displacement to be absorbed. A similar behaviour was also observed in the ultimate axial strain case in which an increase in this variable causes a higher system resistance load and the final displacement. This conformed with the experimental tests and common sense as a cable pull-out test with higher axial failure strain can potentially result in higher flexibility and load-bearing capacity of the system. The effect of cohesion in the normal direction, which is one of the main variables representing the grout annulus, was analysed using the Global concept. In the case of normal strength, the model was not sensitive to higher numbers and the final result was not affected. However, in the lower band, an increase in the final vertical displacement and a decrease in the final resistant load was evident which complies with common sense, because, as grout is weakened, these two behaviours are anticipated. Figure 19b represents the model's sensitivity to yield tensile stress and Young's modulus of cable, respectively. In the case of yield stress, a logical response was observed from the model as higher values resulted in higher load capacity and higher vertical displacement at failure. Furthermore, in the case of Young's modulus, an increase in shear resistance of the system alongside a decrease in failure displacement (meaning stiffer system behaviour) was evident, which could be justifiable.

Figure 20 presents the typical results of the sensitivity analysis for 3D simulations where the shear force (N) was drawn against shear displacement (m). Figure 20a shows the sensitivity of the model to shear stiffness. It is seen that an increase in shear stiffness results in a significant increase in the ultimate loading capacity of the system, which was also verified previously in laboratory tests. This agrees with the results of the 2D numerical simulations. Figure 20b illustrates the effect of the ultimate shear strain. As shown, an increase in this value resulted in an increase in the ultimate resistance load of the system and an increase in the vertical displacement at failure.

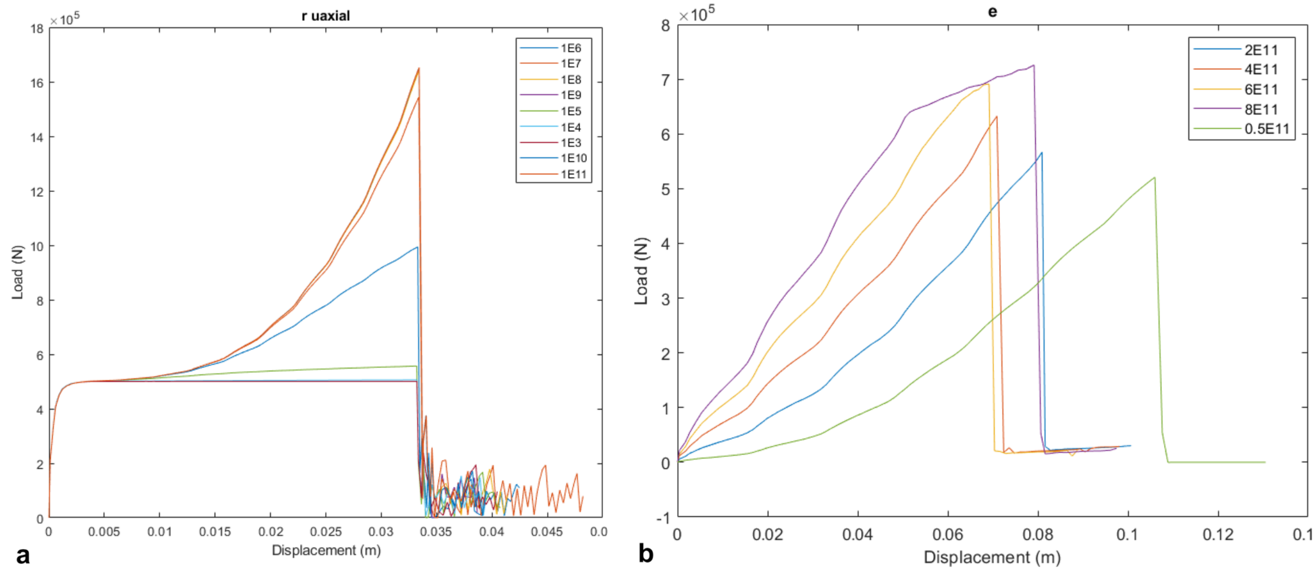


Fig. 19 Typical sensitivity analysis results in 2D: **a** Effect of ultimate axial force limit, and **b** effect of Young's modulus of cable

Figure 21 illustrates the effect of pretension load on different installation angles in UDEC. As is shown, the increase in pretension load resulted in a decrease in both the peak shear load of the system and the displacement at failure. This means the resistance capacity of the system was reduced when pretension load up to 15 t was applied. This agrees with the results of experimental studies carried out as part of ACARP project C42012 and other published results (Aziz et al. 2015a, 2015b, 2015c, 2016b, 2018, 2019; Mirzaghorbanali et al. 2017; Khaleghparast et al. 2020). In addition, the peak shear strength increased with increase in installation angle. The opposite trend is observed for the peak shear displacement where an increase in the installation angle reduced the peak shear displacement. This agrees with the results of the experimental studies carried out in this project.

The output results of the models showed a close approximation with the experimental efforts and illustrated a high potential for understanding the underlying logics in cable bolting. In addition, the availability of more tests, especially in case of de-bonded cables, could potentially help improve the accuracy of the proposed numerical model and trust in its output results. The following are some of the findings from this lab-scale numerical endeavour:

1. Discretion should be exercised in choosing the right modelling tool in UDEC and 3DEC. If debonding is unlikely to happen or its value is negligible, local logic is recommended. If the mentioned assumptions are not true, Global logic is recommended. Similarly, if the shear contribution of the cable is negligible or not of importance, cable logic can be utilized instead of rock bolt logic.
2. Although in most cases, the mechanical system is undetermined, the sum of reaction forces on supports located on the lateral premises of the model can be assumed to be as much as the applied load to the model (minus the energy loss).
3. The loading rate can be neglected, provided that it is in a relatively acceptable range. This means that, as long as the inertia force remains in quasi-static boundaries and does not represent dynamic loading, the final results will not be affected by loading rate. This is of great importance because it can shorten the modelling time drastically. In this study, the maximum loading rate was limited to 0.1 m/s. For shear rates higher than this limit, further research studies are suggested.
4. The mesh size does not play a crucial role in the models as long as the geometry of the model is discretised in a uniformly shaped manner. This again results in higher efficiency of the modelling approach and avoids lengthy simulations. A 1:20 ratio is proposed as a reasonable balance between accuracy and efficiency. Smaller mesh size does not result in meaningful changes in precision.
5. Confinement will result in a stiffer system which means the damaged zone would be concentrated in the shear plane vicinity (i.e. shear surface). This means the distance between hinge points will be decreased. An example of this is the single-shear test where the level of confinement is relatively high and could potentially affect the result. Also, this can be achieved by increasing the grout strength and using stronger reinforcing elements.
6. In pretensioning, the pretension load could be integrated into the model through changes in active length. Models suggest a linear relationship between active length and

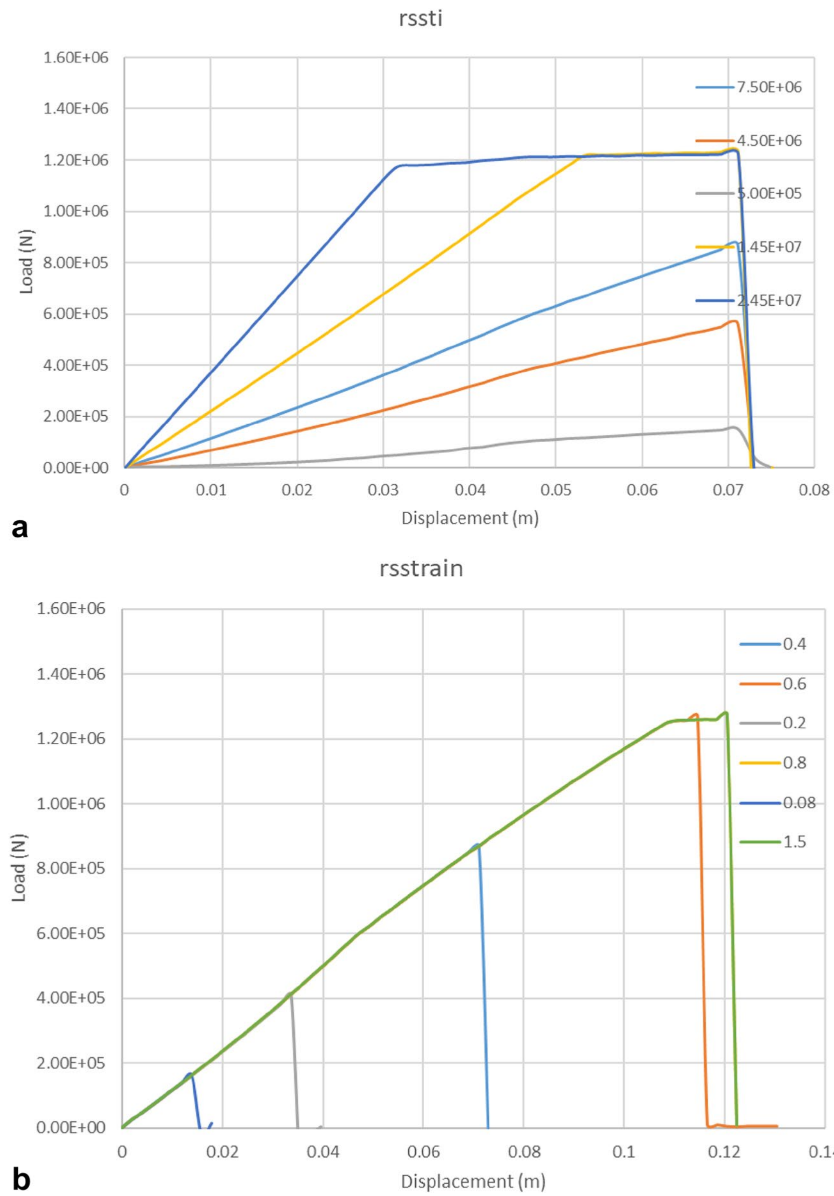


Fig. 20 Typical sensitivity analysis results—3D: **a** Effect of shear stiffness, and **b** Effect of ultimate shear strain

pretension load in which pretension load of 15 t unconfined conditions (i.e., double-shear tests), reduces the multiplier from 4 to 3, and in confined conditions (i.e., single-shear tests) from 2 to 1.

7. Multiple simulations have suggested that plasticity models for the surrounding material could be safely ignored in favour of efficiency as long as the damage is concentrated in the proximity of shear planes, or the damage is deemed relatively controlled.
8. The results of the simulations indicate that the shear strength of cable bolts installed in angles, decreases with an increase in the pretension value. The same trend is observed for peak shear displacement.
9. For the same value of pretension load, the peak shear load increases with an increase in the installation angle. The peak shear displacement, however, decreases as installation angle increases.

6 Conclusions

The behaviour 15.2 mm diameter cable bolts subjected to double-shear loading at different angles are examined and evaluated. Comparison of test results demonstrated the influence of the cable installation angle with respect to the direction of shearing. Cable tensioning was difficult to evaluate

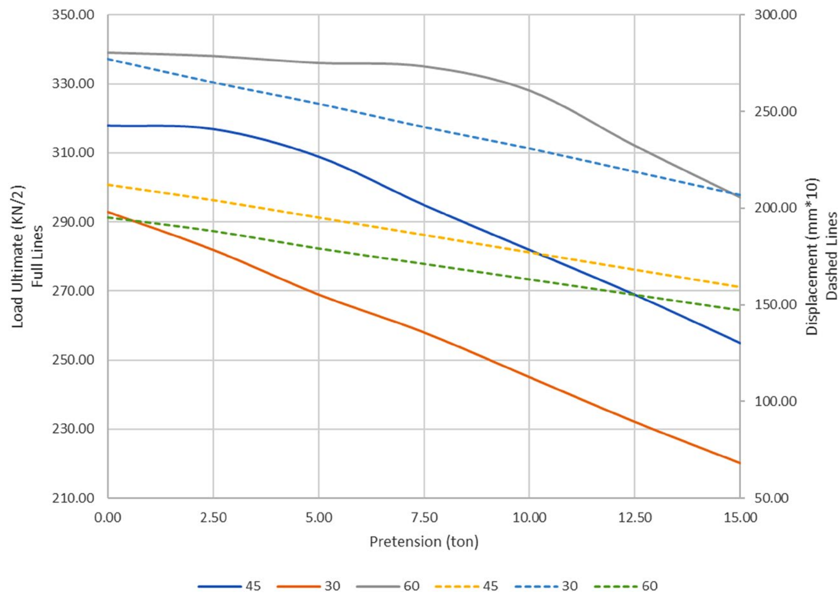


Fig. 21 Effect of pretention on load and displacement

with the existing LTS arrangement because of the unforeseen consequences of the concrete blocks upward movement during excessive pretension loading and the whole concrete block system failure. Accordingly, the pretension loads were kept to around 50 kN.

From the study, it was found that the higher the bolt angle of inclination to the sheared joint plane, the greater was the ultimate failure load of the cable bolt. This was expected, as the greater the angle of inclination with respect to the direction of shearing force, the less likely all cable wires fail in shear. It is noted that there was no rotation of cable bolts in the experiments.

It was inferred from the results of numerical simulations that UDEC and 3DEC can simulate the general trend of cable bolt shear behaviour for different pretension loads and installation angles in two- and three-dimensional frameworks.

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