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DEVELOPMENT OF DOUBLE SHEAR TESTING OF TENDONS

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ABSTRACT: More than 25 years of uninterrupted research on ground support technology for underground mines has been undertaken at the University of Wollongong. This research has resulted in significant findings on tendon characteristics and strength properties. The paper focuses on the development of a fourth generation of cylindrically shaped shear test apparatus for assessing tendon performance in shear. This shear apparatus is known as the MK-IV Double Shear Box or Naj Aziz Double Shear Box (NADSB), and is based on the experience gained from the development of previous versions of rectangular double shear boxes. The new NADSB is circular in shape and is fitted with a truss system, which permits friction free shear testing of tendons across joint planes. A series of double shear tests were carried out on a number of cable bolts commonly used in Australian mines, both plain and indented wires, under varied pretension loads. The results were compared with similar test results using rectangular shaped double shear apparatus, with and without friction across joint faces. The significance of wire surface roughness and increased initial pretension loads are discussed and conclusions made, suggesting that indented wires are inferior in shear compared with plain cable bolts. The general test procedure of the NADSB is described and different concrete reinforcement technics are reported. The influence of external and internal confinement of the concrete medium blocks in circular double shear box contributed to consistent test results with a minimum of lateral and axial cracks occurring in the host medium.

INTRODUCTION

Load transfer property testing of tendons has been studied over several decades with much of the early research focused on pull and shear testing of solid rock bolts and hollow bolts (like Swellex and split set tube bolts), as these elements were used extensively at this time. It was not until the 1970’s that the use of cables for ground reinforcement developed, and as such highlighted the need to extend load transfer property testing to include shear strength of cable bolts. Initially the discarded hoisting ropes were used for ground support in metal mines, particularly in stoping wall support and then late in 1980’s the use of cable tendons started in coal mines. A summary of the historically significant studies undertaken on shear behaviour of tendons is reported in Table 1 (Jalalifar, *et al*., 2006).

The early methods of shear testing were carried out using direct shear testing machines under constant load conditions. Bjurstron (1974) and Ludvig (1983) used the standard constant normal load direct shear rigs to undertake general studies on the load transfer mechanism of tendons. This was further extended to cable bolt shear tests at both 45º and 90º.to the sheared surface. With the exceptions of a few, these methods were mostly examined the pre-failed performance of the tendon with respect to the characteristics of the various tested parameters, such as medium strength, grout type, loading rates. Only a few tests allowed the tendon to be loaded to its final failure, and these were made with lower capacity bolts and cables which were reported by Bjurstron (1974). Others undertook studies on cable bolt shear, including Dobe (1996) and Gores, *et al*.,(1996).

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In Australia tendon elements are normally sheared laterally under constant normal load conditions. Notable studies was that of Dight (1982) who reported on shear testing of steel wire strands in the direct shear machine, initially developed by William (1980) but was later modified by Dight (1982) to handle cable shearing under constant normal load. The tested cable was first grouted in a plastic tube and the plastic sleeved cable was then grouted in 65 mm steel tube using cementitious based grout. Shearing was carried out perpendicular to the axis of the cable. Others undertaking shear testing in guillotine style include the works of Bigby (2005), Thomas (2012) and Aziz *et al* (2015).

Table 1: Chronology of tendon shear performance investigation and development since 1972 (modified from Jalalifar, 2006)

Author	Base of the method	Advantages	Disadvantages
Dulascka (1972)	Development of plastic hinge after max. Moment	Prediction of shear force by bolt	Non static equilibrium condition in shear joint
Bjurstrom (1973)	Equilibrium forces acting on the system	Estimation of shear resistance: due to dowel, reinforcement and friction effect,	Mode of failure in surrounding materials was neglected
Hass (1976)	Single shear test	Test were performed on real rocks	Non-uniform stress distribution along the shear joint
Azuar (1977)	Single shear test	Different bolt angles were considered	Influence of friction effect could not properly considered
Hibino (1981)	Single shear test	Pretensioning was applied	Pretensioning and bolt's inclination could not considered properly
Hass (1981)	Single shear test	Real rocks with different bolt angles were considered	Pretensioning was not applied
Dight (1982)	Theoretical analysis	The prediction of dowel effect and hinge point was considered	Neglecting the bolt behaviour in elastic range, poor effect of normal stress on joint
Egger and Fernandz (1983)	Single shear test	Different bolt angles was applied	Pretensioning was not applied
Ludvige (1983)	Single shear test	Different bolt angles was applied	No fully grouted bolt was tested
Schubert (1984)	Equilibrium forces acting on the deformed system		
Real rocks was tested	Pretensioning was not considered		
Yashinaka (1987)	Direct shear test	Different bolt angles was considered	Pretensioning could not apply
Spang and Egger (1990)	Single shear test	Real rocks was tested, max bolt contribution and displacement was predicted	Limited in: grout types, annulus thickness, rock strength and pretensioning
Egger and Zabuski (1991)	Single shear test	Prediction of bolt failure at a combination of axial and shear	No joint confinement and bolt pretensioning was considered
Holmberge (1991)	The equilibrium of forces acting on the deformed bar	Bolt behaviour was analysed in both elastic and plastic stages	The effect of grout was disregarded
Ferrero (1995)	Single shear test	The plastic stage of the system was considered	In-capability of the method to show the effect of pretensioning
Pellet and Egger (1995)	Theoretical analysis	Both elastic and plastic stages was analysed	The effect of grout material was neglected
Dube, 1996	A laboratory study on cable bolts subjected to combined tensile and shear loads.	evaluating both parameters in one test	Operation difficulties
Goris *et al*. (1996)	Single shear test	Perpendicular bolts was analysed	Non-equilibrium load distribution on the shear joint, Max. Displacement was up to 46 mm
Grasselli (2005)	Double shear test	Symmetric situation around the shear joint	Bolt pretensioning was not considered
Mahoni, *et al*. (2005)	Single shear test	Lengthy bolt-grout-concrete anchorage	-
Aziz *et al* (2005)	Double shear test	Symmetric situation around the shear joint, pretension effect, bolt profile, any grout, bolt and hole diameter	The size of the shear box is small for large bolt diameters and strong steel bolts
Mckenzie and king(2015)	Single shear test	Shearing of cable with no concrete face contact in an integrated test machine	Cable pretension capable of defining cable debonded.

			Labour intensive yields credible results

CHRONOLOGY OF DOUBLE SHEAR TESTING METHODS

Rectangular Double Shear Rigs (MKI, MKII and MKIII)

The earliest reference to the use of double shear testing rigs was a paper detailing on the work carried out by the rock mechanics group of the University of Wollongong, by Aziz, *et al.,* (2003) at the fourth Coal Operators' Conference. The 600 mm long shear box is known as MKI Double Shear box or simply MKI DS- box, as shown in Figure 1. This box was relatively small in dimension and was found to be unsuitable for shear testing of large capacity rock bolts and cable bolts in low strength concrete. Currently DS-MKI is used for shear testing of fibre glass rods and smaller diameter mild steel rib bolts as reported by Aziz, *et al* (2015 and 2016) and recently by Khaleghaparast, *et al*., (2019) used it for the static and dynamic tests.

To accommodate shear testing of larger capacity rock bolts and cable bolts, two new versions of double shear testing rigs were subsequently developed, as shown in Figure 2; (a) The rectangular DS-MKII box consisting of two 300 mm long outer cubic boxes and a 450 mm long middle central cuboid box with 300×300 mm^2 cross-sectional area. The overall length of 1050 mm has opposing concrete joint faces in contact with each other and therefore the applied shear force is spent on overcoming the combination of the shear failure load and the friction force of the sheared medium joint faces, and (b) DS-MKIII box, a modified MKII DS Box, with opposing concrete joint faces not in contact with each other and therefore the measured shear resistance force is spent only on shearing the cable strand wires, in other words, it is a frictionless shearing box. Further information on double shear test boxes MKII and MKIII was reported by Aziz*, et al* (2010, 2014, 2015, a and b, 2016) and Resekh, *et al.,* (2016).

Figure 1: MKI double shear rig

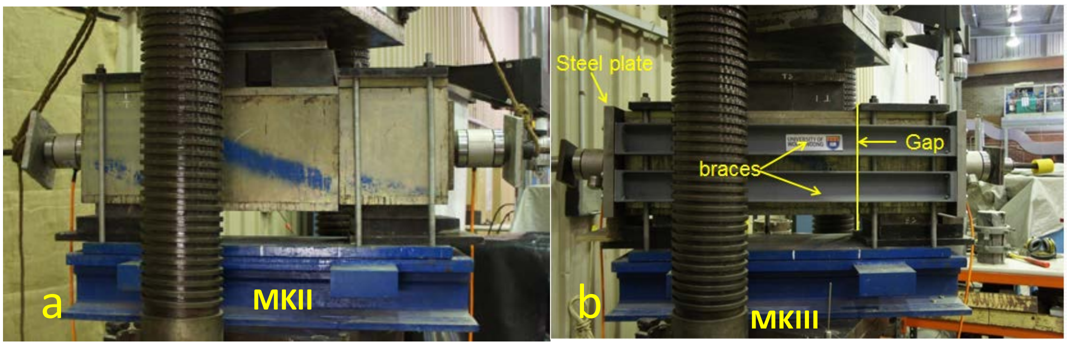

Figure 2- Double shear test rigs (a) MKII and (b) MKIII.

The level of shear force spent on overcoming the friction force between joint faces was determined using the following mathematical equation model based on the combination of Mohr Coulomb criterion and Fourier series scheme (Aziz, *et al.,* 2015a).

$$\tau_p = \left(\frac{a_0}{2} + \sum_{n=1}^{3}\left[a_n \cos\left(\frac{2n\pi \frac{T}{2\pi}\cos^{-1}\left[\frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_3^2}}{24a_3}\right]}{T}\right)\right]\right)\tan(\varphi) + c \qquad (1)$$

Where τp is the shear stress, Σ is the shear load, C is cohesion, a_n is Fourier Coefficient, n is the number of Fourier Coefficient, which varies between 0 and 3 and T is the shearing vertical travel/displacement.

Aziz*, et al.,* (2016) verified the effect of the equation with experimental test results. Excessive fractures in the rectangular shaped concrete medium contribute to the inconsistency in test results. Typical axial cracks are shown in Figure 3.

Figure 3: Cracked concrete block after shearing and zone of crushing

MK-IV CIRCULAR DOUBLE SHEAR BOX (NAJ AZIZ DOUBLE SHEAR BOX)

Figure 4 shows the general view of a new cylindrically shaped Double Shear Testing Rig (MKIV-DSB), and now called “Naj Aziz DS Box. (NADSB) The 300 mm diameter steel circular clamps permit the application of external confinement to the cylindrical concrete medium. The outer DS cylinder sides rest on support cradles to provide a stable positioning during the shearing stage, allowing the longer central part of the box to shear vertically down with a minimum of lateral movement. With the use of four steel trusses and 30 mm thick reinforced side plates, shown in Figure 4b, the arrangement permits a minimum of contacts occurring between the concrete blocks side faces during shearing, thus preventing part of shearing forces being spent on overcoming the medium joint sides rubbing friction. However, for higher axial cable pretension loads, the two 30 mm thick steel plates are further reinforced laterally with; (a) welding a 30 mm thick and 100 mm bar across top half and (b) a 20 mm thick and 200 mm square plates inserted on the cable to minimise their inward bending as shown in Figure 4 b. The whole assembly is mounted on outer base cradle half cylinders, fastened and secured to the carrier base frame using eight 20 mm diameter threaded bars. The concrete steel clamp is 17.5 mm in thickness, fastened to the lower half of the full clamp with three bolts pre side. Figure 4d shows the view of post shear test of the shear box.

The preparation of the **NADSB** assembling differs from the rectangular shear box. 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 wood frame for concrete pour as shown in Figure 5. During casting of the concrete and production of the central hole for cable installation, a conduit wrapped with 8 mm PVC tube is held vertically along the mould to precast a rifled hole through the centre of 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 similar fashion as reported by Aziz, *et al.,* (2017) in ACARP project report C24012. Once a few days of casting the concrete blocks are removed from cardboards and mounted on the double shear. The lower semicircular sides steel frames of the 300 mm long side blocks are placed on steel support cradles resting on the carrier base frame. The middle section of the double shear box set-up rests temporarily on either wood blocks and lately on a purpose built semicircular table with retractable legs.

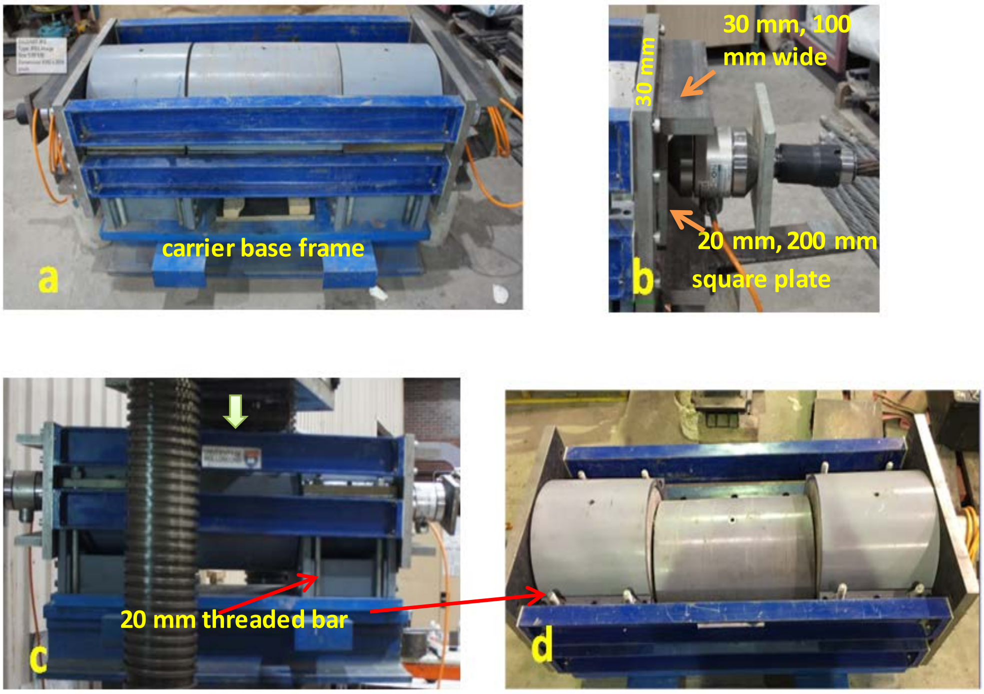

Figure 4: MKIV DSB / NADSB; a) Assembled rig with the central block resting temporarily on wood or purpose built plate; b) Axial load retaining side reinforcement; c) Assembled rig mounted on compression testing machine; d) Post-test assembled box with sheared down central block

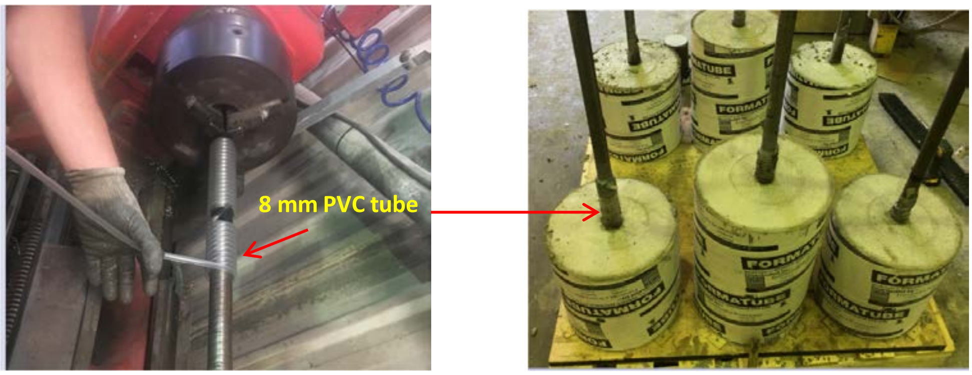

Figure 5: Preparation of the cylindrical concrete blocks and casting of rifled holes in concrete block using PVC flexible tubes wrapped around the central steel rod

Similar to the DS-MKIII rectangular shear box set up, the NADSBuses the truss system/braces around the double shear assembly for frictionless shearing as shown in Figure 4c. The truss system consists of four 1100 mm long 9 mm thick open channel steel braces connected between two 30 mm thick side steel plates. Next the cable bolt is inserted into the central axial hole, which is followed by mounting a suitable capacity load cell on each protruding side of the cable in the assembled concrete blocks and tensioned to a predetermined axial pretension load, using a “Blue Healer” tensioner. Tensioning of the cable is retained by the barrel and wedge retainers. When assembled, gaps of almost 5 mm are left between concrete blocks, thus the adjacent sheared concrete faces are kept apart, eliminating contact between the sheared faces. There will be no joint face surface friction force

Once the cable is pretensioned, cementitious grout is injected from the vertically pre-cast radial holes on top of each concrete block into the hole annulus space around the cable strand. After grout curing time, the double shear assembly is placed on the carrier base frame consisting of a parallel pair of rail track sections welded on a 30 mm thick steel plate. The outer 300 mm side cube blocks of the double shear apparatus are mounted on 20 mm thick steel cradles set on 100 mm steel blocks. Next the central 450 mm long block will be freed to be vertically sheared down using a 500 t capacity hydraulic universal testing machine. The recommended rate of shearing is in the order of 1 mm/min for the maximum 100 mm vertical displacement.

The MK-IV shearing box can be modified to study the cable deboning. This is achieved by adding a long section of circular steel clamp on one side of the double shear box by dispensing with one load cell, enabling encapsulation of the longer cable bolt length. The extent of the axial load build up on the bolt during shearing process can still be monitored by the second load cell on the other side of the shear apparatus. The addition of the extra concrete column length on one side of the box may not influence the truss system performance in Joints frictionless tendon shearing.

RESULTS AND ANALYSIS

Table 2 lists details of several cable bolts being tested in the NADSB apparatus. All tested cables were SUMO cable bolts, plain and indented wire strands. Figure 6 shows the load displacement profiles of both types; with the peak shear load of plain strand cable being typically higher than the indented type and at greater displacement. However, the shear failure load of each wire in the strand is inconsistent compared with the plain strand.

Table 2: Summary of double shear test results using DS MKIV rig (Naj’s DS Box)

Test No.	Cable	Test date	UTS (t)	Pt (t)	Peak Shear load/side (kN)	Peak Axial load L (kN)	Peak Axial load R (kN)	Shear displ. (mm)	Concrete reinforced internally with Steel tube
1	SUMO-P	15-05-18	65	15	400	408	451	71	No
2	SUMO-P	25-05-18	65	15	377	275	294	43	yes
3	SUMO-P	06-06-18	65	2	451	422	442	86	No
4	SUMO-P	26-06-18	65	2	483	270	250	59	yes
5	ID- SUMO	10-07-18	63	15	315	366	360	61	No
6	ID-SUMO	18-07-18	63	2	378	280	286	68	No
7	ID-SUMO	02-08-18	63	15	240	174	173	44	yes
8	ID-SUMO	10-08-18	63	2	310	193	189	47	MIX
9	SUMO-P	20.09.18	65	2	453	285	286	61	Yes

P: Plain, ID: Indented cable wires, Cable dia. 28 mm. Sample 8 had one side block not reinforced

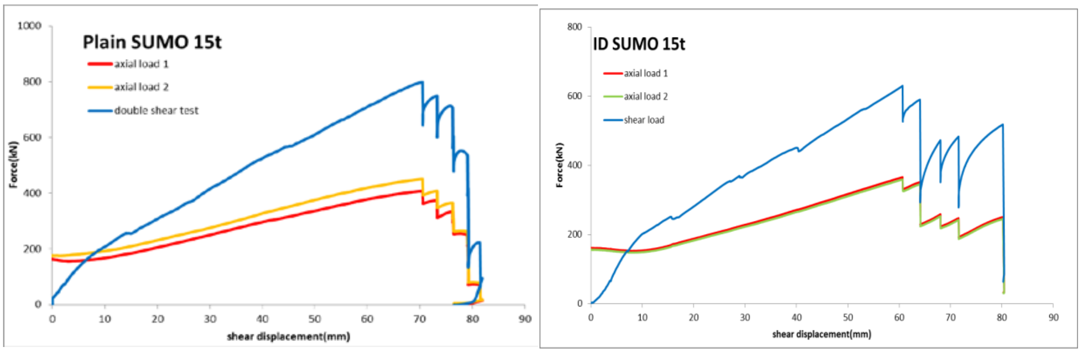

Figure 6: Shear testing of plain and indented cable bolts in 40 MPa concrete subjected to 15 t pretension load

With regard to changes to test environment as listed in Table 2, the load-displacement and axial pretension build up load in plain and indented Sumo cables are shown in Figure 6. Both tests were carried out in concrete blocks externally reinforced with same steel clamps. These two cables were initially pretensioned to 15 t and are listed as test 1 and 5 in Table 2. The level of displacement profiles in both shearing process were close, particularly when defining the first peak shear load. In addition to wire indentation and in reality there are three other factors that may test results. These include the strength of the medium (concrete), the effectiveness of medium confinement, and applied axial pretension loads on bolts.

1. Medium confinement: Without effective confinement, internally, externally or combined, the true shearing of the tendon has been found to be difficult to assess as the host medium would end up being cracked radially and axially with the failed cable being subjected to more of tensile failure rather than shear, particularly at the hinge points in the vicinity of sheared joint planes, as shown in Figure 3.

Figure 7a shows the cable installed in concrete blocks reinforced internally with steel tube (dia: 165 mm, wall thickness: 3 mm). The internal confinement of the concrete block contribute to increased concrete strength and stiffness, which minimises early concrete deformation around the tendon close to sheared joint faces, reducing concrete deformation depth at the hinge points by as much as 50%. These result into reduced vertical cable displacement, which would be less than that occur with un-reinforced concrete medium. Figure 7b shows the load displacement profile of internally reinforced concrete of an indented cable. Clearly there were early strand wires failures prior to the strand wires total peak shear load failures. This phenomenon is reported in various indented cables irrespective of the test condition with regard to block shape and different block test environment Aziz, *et al*., (2016), and is the subject of further research.

2. *Cable bolt indentation:* Comparative tendon shear studies carried out using different methods of testing as reported in various published papers(Aziz, 2015, 2017, 2017 and 2018) and in ACARP Project C 24012 (Aziz, *et al.,* 2017) revealed the following:

- Indented cables are, in general lower in ultimate tensile strength than their counter part plain wire cables. It is understood that such loss of strength may be attributed to cable wires weight loss during indentation process as reported by Aziz, *et al,* (2017, 2014), however, localised wires stress concentration on the wire .has been found to be in higher impact than the cable wires weight loss, as reported by Aziz, *et al.,* (2019)

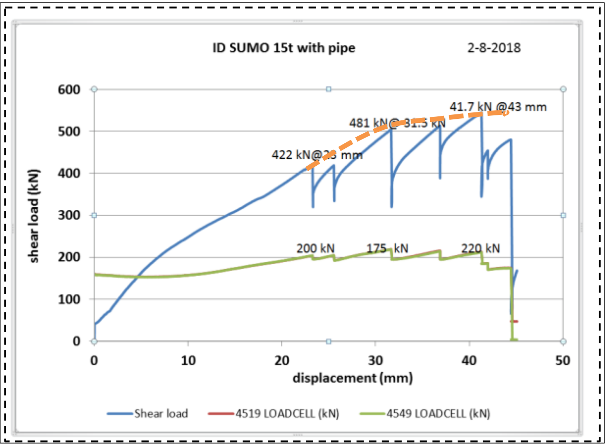

Figure 7a: internally reinforced concrete medium. External clamp not show

Figure 7b: ID Sumo Cable bolt tested for shear in Concrete blocks in MKIV shear box with internal reinforcement

- Vertical shear displacement at peak shear failure load in indented cable is less than the plain wired cable.
- Unlike plain wire failure, where the load decreases gradually with the snapping of each wire in the cable strand, the wire failure in the indented cable may not necessarily follow gradual and sequential load failures. As can be seen from Figure 7b, the initial wire snapping in the strand may not cause the ultimate peak failure load of the cable; on the contrary, there is a gradual increase in successive failure loads in the strand with greater peak failure loads. This phenomenon is currently being investigated with the view that the orientation of the initial failed wires undergoing relatively excessive bending during shearing.

3. Concrete strength and confinement: Irrespective of the test method, the concrete strength and its confinement play a significant role in tendon shearing. It will be difficult to undertake shear testing of cables, particularly stronger cables with the tensile strength (failure loads) greater than 50 t in lower concrete strength of less than 40 MPa unless it is adequately confined. Three ways can be used to increase the medium /strength;

- external confinement of the concrete with steel clamps,
- internal reinforcement of the composite medium by placing moulding steel tubes in the concrete;
- a combination of (a) and (b) techniques and shows all three types of reinforcements with 40 MPa concrete.

Reinforcement of the cylindrically shaped double shear concrete mould alone may not stop axial and radial cracking of concrete blocks. These cracks, when formed initially, may not be large but the shearing of cable will cause the crack to widen and crush, which will influence the cable behaviour in shear with increased shear displacement. The sheared cable may fail in tensile shear instead of shear. Figures 3 and 7a show two exposed reinforcements and in the first one, shown in Figure 3 the concrete confinement has an external confinement, while those shown in Figure 7a are both internal and external confinements. Without the internal confinement/ reinforcement, cracks will occur along the full length of the concrete in both rectangular as well as in circular concrete blocks, however, the extent of axial cracks in circular concrete are small with little influence on cable shear load and the width and size of the deformation or crushing zone because of the evenly distributed and effective external confinement of steel clamps. It should be noted that the internal reinforcement was also found to be effective in rectangular shaped medium as demonstrated in the recent study by Khaleghparast *et al*., 2019) in small diameter solid rock bolts.

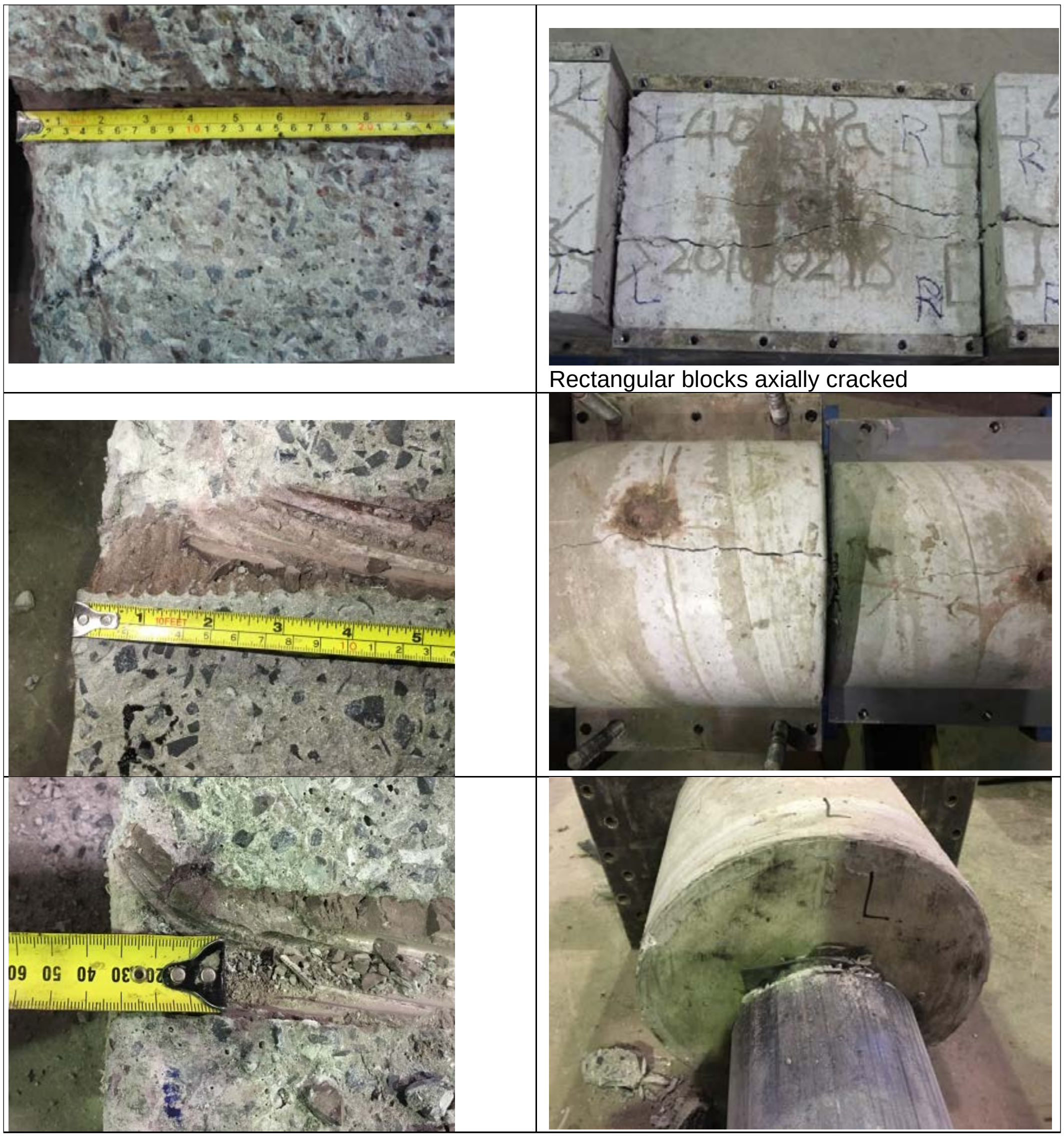

Figure 8: changes in deformation/crushing zones at the hinge point, during shearing in different concrete mould shapes and reinforcements

COMPARING TEST RESULTS BETWEEN DS MKII, MKIII AND MKIV (NAJ AZIZ DSBOX)

Table 3 lists test results from testing Sumo cables using different shear boxes of MKII, MKIII and MKIV. Both Sumo plain and indented cable were tested in externally confined 40 MPa concrete with pretension loads as indicated in the table. Testing with MKII meant the existence of contact joint faces friction, in which part of applied shear load was spent in overcoming friction. Testing using MKIII and MKIV boxes required no joint face friction force. The following points are noted from Table 3 and load-displacement figures reported by Li, *et al* (2017)

- The external steel confinement generally strengthens the integrity of the host medium during shearing, however circular clamps or confinements is more effective method than the rectangular type because of uniform lateral confining loads applied all around the concrete.

- The shear failure loads tested in the circular NADSB (MKIVDSB) are generally lower in comparison with test results from both MKII and MKIII boxes. This is because all-round confinement load is uniform and strengthens concrete stiffness. The higher shear load values, particularly when testing with MKII box is attributed to the additional shearing force needed to overcome the contact frictional forces.
- The shear load values determined when using the NADSB are relatively closer to Megabolt Single Shear Test results (MSST) as reported by Aziz *et al* in ACARP report C24012(2017). These values occur in testing cables that are of indented types and do not debond.
- Excessive crushing and deformation of the concrete at and near cable hinge points may lead to increased shear displacement resulting in to greater shear force values indicating that the sheared section of the cable wires are in near tensile failure rather than in shear, as shown in Figure 8. This may have some benefit for ground reinforcement particularly in softer formation with excessive lamination and shearing.

Table 3: Test data on Sumo plain and indented cables tested in MKII, MKIII and MKIV double shear boxes(NADSB), all externally clamped. Cable diameter- 28 mm.

Apparatus type	Cable type	UTS (t)	Pretension (t)	Peak shear load per side (KN)	Shear load at 70% peak shear load (KN)	Peak axial load (L) (KN)	Peak axial load (R) (KN)	Shear displacement (mm)
MKII	Plain SUMO	65	10	1318/2= 659	461	365	418	78.99
		65	25	1422/2= 711	498	342	412	58.76
	ID SUMO	63	10	976/2= 488	342	280	290	60.58
		63	25	828/2= 412	289	272	295	32.8
MKIII	Plain SUMO	65	0	886/2= 443	N/A	432	432	100*
		65	15	852/2= 426	N/A	433	433	88.2
	ID SUMO	63	0	815/2= 408	N/A	364	360	93.4
		63	15	767/2= 383	N/A	376	380	85.7
MKIV (NADSB)	Plain SUMO	65	2	904/2=452	N/A	422	442	86
		65	15	800/2=400	N/A	408	451	71
	ID SUMO	63	2	756/2=378	N/A	280	286	68
		63	15	630/2=315	N/A	366	360	61

* No cable failure

CONCLUSIONS

MKIVDSB (Naj Aziz DSB) method is an effective and reliable system for determining the shear strength of tendons. The circular shape of the steel clamp allows a uniform application of confinement around the concrete perimeter contributing to reduced depth of deformation or cracking zone, enabling the tested tendon to fail in quasi shear rather than in tension.

The cylindrically shaped moulds can be effectively confined internally thus reducing the formation of wider axial cracks that would influence tests conditions. The internally reinforced concrete medium allows shear testing of cable in stronger strength confinement. Internal reinforcement of the concrete medium was found also to be effective in rectangular shaped medium, thus rendering the concrete shape factor as irrelevant

The test findings from the NADSB study agreed with single shear testing results undertaken by Aziz, *et al.,* (2017) in the ACARP C 24012 report.

The NADSB/MKIVDSB can be modified by dispensing with one load cell and replacing it with a long section of circular steel clamp, enabling encapsulation of the longer cable bolt length for cable bolt debonding studies. The extent of the axial load in the bolt during the shearing process is still monitored by the second load cell on the other side of the shear apparatus.

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