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ORIGINAL PAPER

A Critical Review on Development of Laboratory Testing Technologies of Tendons Applied to Stabilize Underground Structures

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Abstract
The paper reports on different methods and techniques developed, over the past several decades, to characterize axial and shear strength of various tendons used for ground reinforcement in mining and civil engineering operations. Most of axial loading methods are carried out by pull testing, with very few tests reported on push load tests. Based on an extensive literature survey of more than 80 scientific documents, published between 1970 and 2023, it was shown that a significant number of apparatuses have been developed for testing tendons axially with a limited number being developed for shear testing. The majority of axial testing is undertaken by pull testing tests. The type of tensile strength test selected is dependent on the test purpose and tendon type. The short encapsulation pull test is used mostly for testing solid rebar bolts, while double embedment can be used for the pull testing of both solid rebar and cables, preventing unwinding. A new reverse pull test rig is now available to undertake both the static and dynamic testing of tendons. The single shear and double shear test methods are discussed concerning the different purposes of testing and tendon tool design for the different ground conditions. Several factors and parameters, which influence the effectiveness of bolting, concerning rock reinforcement, are discussed. These factors or parameters include: the medium type and strength, the host medium shape and size of confinement, and the frictional resistance. Particular emphasis is placed on frictionless shear testing of joint faces using the Lateral Truss System (LTS) that prevents shearing faces from coming in contact with each other. The findings from the double shear test indicate that the type of medium, shape and size, and the sheared joint surface of the medium along the applied loading rate all played a prominent role in the determination of the shear test results to achieve effective reinforcement design of the reinforcement system, based on sound calculated parameters for effective ground reinforcement designs and construction. Finally, shear testing of tendons at various angles with respect to rock formation discussed is angle shear testing of tendons installed in large concrete blocks. The paper provides an opportunity for future tests to include the testing and further assessment of tendons studied dynamically to consider reinforcement in seismically active ground.

Highlights

- The various tendon testing facilities, reported over the last decades, have been critically reviewed.
- Several factors affecting the effectiveness of a tendon reinforcement system have been discussed.
- In shear testing, particular attention should be paid to the frictional effect of the joint planes on the test results.
- Pull test of cables should be carried out using the double embedment method to prevent unwinding.
- The findings should provide the opportunity of advancing research going forward to different levels.

Keywords Pull-out test · Shear testing · Rock bolt · Cables · Rock engineering · Rock reinforcement

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Abbreviations

LTC	Load transfer capacity
GFRP	Glass fiber-reinforced polymer
DEPT	Double embedment pull test
SEPT	Single embedment pull test
LSEPT	Laboratory short encapsulation pull test
MHC	Modified Hoek cell
CRP	Constant radial pressure
CRS	Constant radial stiffness
SSPT	Split-pipe pull test
PVC	Polyvinyl chloride
SEDESSS	Short encapsulation double embedment steel split set
ASTM	Stands for the American Society for Testing and Materials
LVDT	Linear variable differential transformer
RPTM	Reverse pull-out test machine
SST	Single shear test
BSSST	British Standard Single Shear Test
MISSR	Megabolt integrated single shear rig
DST	Double shear test
LTS	Lateral truss system
UCS	Uniaxial compressive strength
AST	Angle shear test
CST	Combination shear and tensile
MAST	Multi-axis substructure testing

1 Introduction

Rock mass stabilization during and after the underground excavations has always been a paramount concern for engineers due to the ever-changing nature of the geological formations and fluctuating ground conditions (Varelija and Hartlieb 2024). In this regard, various rock supports (e.g., timber, steel arcs, and concrete liners) have been widely utilized in underground operations to apply a reactive force to the surface of an excavation (Galvin 2016). However, Lang (1962) introduces a new idea in the domain of ground support called “rock reinforcement”. Rock reinforcement is a sub-group of rock improvement, comprising all methods employed for boosting the overall rock mass stability and limiting its movement (Jodeiri Shokri et al. 2024). This idea implies that a reinforcement system should help the rock mass to support itself. For this aim, different tools and approaches can be used. A good approach is to help the unstable rock masses around the excavation boundary maintain their load-bearing capacity via reinforcing elements (Moosavi and Bawden 2003). In this context, the element refers to one of the effective and popular ground control tools called ground reinforcement tendon or simply tendon (Rastegarmanesh et al. 2022). Generally, tendons can be classified into three main groups: cable bolts, rock bolts, and ground anchors (Windsor 1997). The length and Load Transfer Capacity (LTC) of cables are more than rock bolts. Rock bolts and cables are commonly used to address surface and near-surface instabilities. Whereas ground anchors target deep-seated instabilities. The cables' length typically ranges between 3 and 15 m, while a rock bolt’s length is generally less than 3 m. Besides, the length of ground anchors ranges from 10 to 30 m (Galvin 2016). However, in comparison with cables and rock bolts, the application of ground anchors in underground excavations, especially underground coal mining, is not considerable (Yang 2019).

According to the available records, in 1872, steel bars were first used in a slate quarry located in North Wales, UK (Lang 1961). Later on, in 1918 and 1927, rock bolts were employed in coal and metal mines located in Germany and the United States, respectively (Lang et al. 1979). Another recorded application of tendons dates back to 1921 in Poland where reinforcement bars were used to secure the roof strata in Mir Mine (Farah and Aref 1986). On the other side, tendon supports were first used in civil engineering in 1934 during the construction of the Cheurfas Dam in Nigeria (Farah and Aref 1986). In Australia, tendon supports, in the form of basic roof bolts, were first used around 1948 at Elrington Colliery near Cessnock in New South Wales (Howarth and Renwick 1992; Gray 1998). Shortly, rock bolts were utilized in the Snowy Mountains project in 1949 near the border of New South Wales and Victoria (Bolstad et al. 1983). Since the 1960s, fully grouted cables have been utilized in underground mining for ground stabilization (Aziz et al. 2018b). Around 1970, cable bolts were introduced to surface mining and underground metalliferous mining. Afterward, in the early 1980s, cable bolts were first used to reinforce coal mine roadways as a secondary support system (Craig and Aziz 2010). Throughout the last decades, the tendon support system has become popular among underground operators, various kinds of cables and rock bolts have been developed, and their details can be found in the available literature (Windsor and Thompson 1993; Thompson et al. 2012). Aziz (2014) roughly estimated that more than 500 million tendons were used worldwide to stabilize rock masses. Obviously, the volume of the used tendons has to be increased much more than the estimated number by this date. The reasons for such a popularity include (Aydan et al. 1987; Marence and Swoboda 1995; Ferrero 1995; Anzanpour et al. 2021; Varelija and Hartlieb 2024):

- Their application is not limited by the shape of underground openings.
- They can be integrated with other types of supports (e.g., shotcrete and wire mesh).
- Easy transportation and storage compared to other systems.

- Their installation after excavation is faster than other support systems.
- Their application leads to saving space in the underground openings.
- Both active and passive support can be provided.
- More economical than other support systems.

As shown in Fig. 1, in situ installed rock reinforcement systems undergo both tensile and shear loading modes with ground movement caused by either human excavations and/or seismic activities (Hutchinson and Diederichs 1996). In this regard, the basic function of the tendon is to convey load from the unstable block close to the excavation boundary to the firm rock strata beyond the unstable region (Thompson and Villaescusa 2014). Hence, the strength of a tendon support system relies on its ability to enhance the tensile and shear strength needed to maintain the LTC of a rock mass (Peter et al. 2022). At the early stages of rock stabilization using tendons, the rock support principles had primarily evolved based on experience rather than on knowledge (Aziz and Jalalifar 2007). However, it should be noted that being dependent on judgment alone when designing a ground reinforcement system can lead to either over-design or under-design (Varelija and Hartlieb 2024). Thus, to diminish the confusion and enhance knowledge, throughout the last few decades, the axial and shear performances of tendon support systems have been examined experimentally, numerically, and also analytically (Ghorbani et al. 2020; Peter et al. 2022; Jodeiri Shokri et al. 2024). According to the literature, various kinds of laboratory-based facilities have been developed worldwide for testing the performance of the reinforcement system's components and the interaction between them under axial and shear loading modes. The axial performance of a tendon is primarily controlled by the tendon–grout interface. This can be assessed by pull-out or push-out testing, either in the field or the laboratory. However, the shear behavior of tendons can only be conducted in the laboratory using either single or double shear apparatus (Aziz et al. 2018b). The available test rigs may vary from different aspects, including, for example, test set-up size and capacity, embedment length (bond length), embedment type (e.g., either single or double), tendon type, host medium type (e.g., steel tubes, concrete sample, or rock samples), confinement type, and host medium size or shape. These variations determine the capabilities and limitations of the previously presented apparatuses. Thus, the main aim of this paper is to provide an overview of the available literature and classify and compare the tendon testing technologies in static mode from different aspects. The results of this study will help identify the strengths and weak points/limitations of the presented laboratory-based test set-ups. It will also facilitate the modification of laboratory testing technologies to enhance their capability, to simulate in-field conditions.

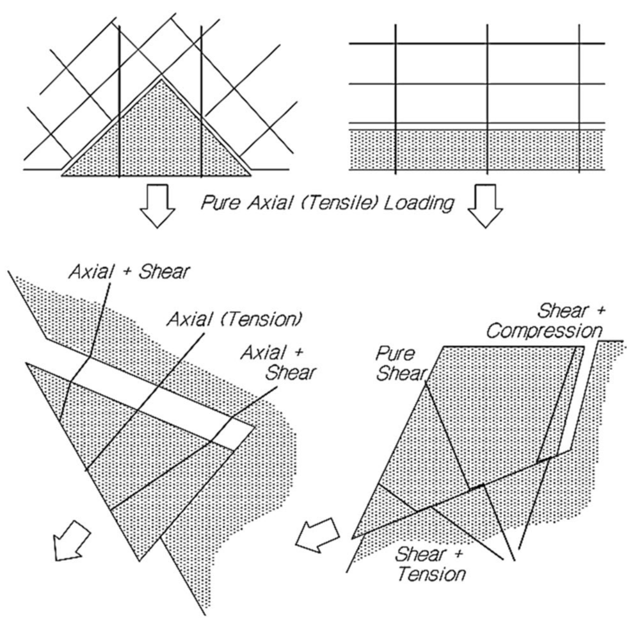


Fig. 1 A schematic view of different loading modes on tendons in the field condition (Hutchinson and Diederichs 1996)

2 Axial Loading of Tendon Support System

For examining the axial behavior of tendons and their bonding to the encapsulation material, push and pull testing has been carried out over the last few years. Pull testing, in general, is a widely used and accepted method, while push testing is usually frowned upon, as it does not reflect the reality of bolting applications found in situ. Push testing is normally carried out in the laboratory and involves pushing the encapsulated length of the tendon out of the confining host medium. Alternatively, a common method of evaluating the real situation is pull testing, in which the tendon is pulled out of the host medium. Based on the reviewed literature, the earliest attempts to introduce a device for examining tendons in tension date back to the 1970s (Fig. 2). Since then, the number of research studies dedicated to this field has significantly increased with more test rigs being built. Moreover, it is shown that the majority of the studies focused on the pull testing of tendons rather than push testing due to some fundamental drawbacks associated with push testing. In this regard, a comparison between push and pull testing methods has been undertaken and illustrated in Table 1. Additionally, as the distribution of the research across various countries within the reviewed literature indicates, Australia stands out as a leading contributor in developing apparatuses for axial testing of tendons. In the next sections, the developed technologies for assessing the axial behavior of tendon supports have been classified and reviewed over several decades in various countries.

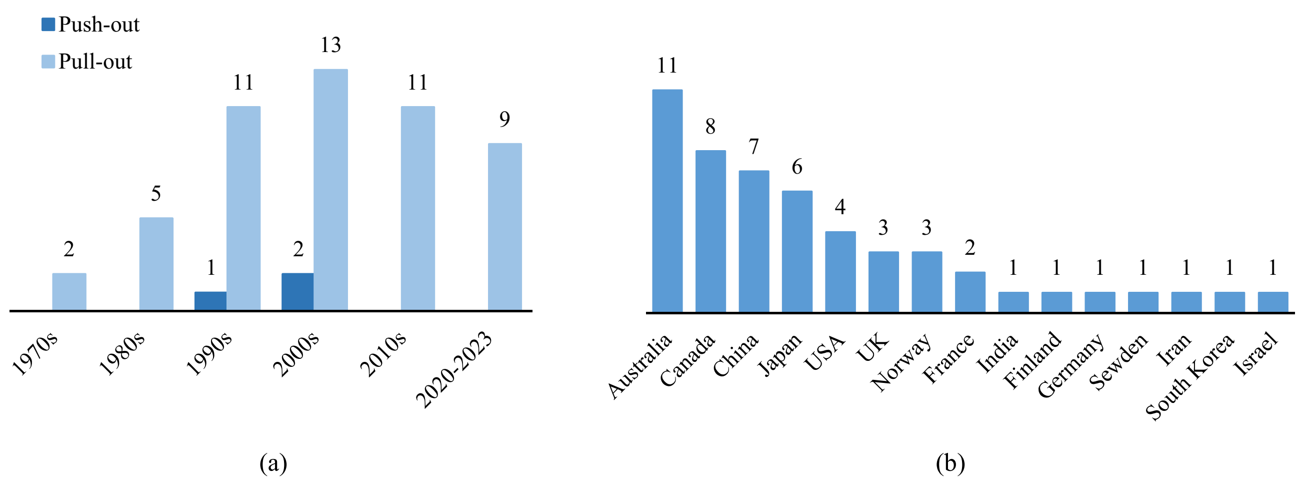


Fig. 2 **a** The trend of axial testing technologies over the past decades, and **b** the number of developed test devices in various countries

Table 1 A comparison between push-out and pull-out methods

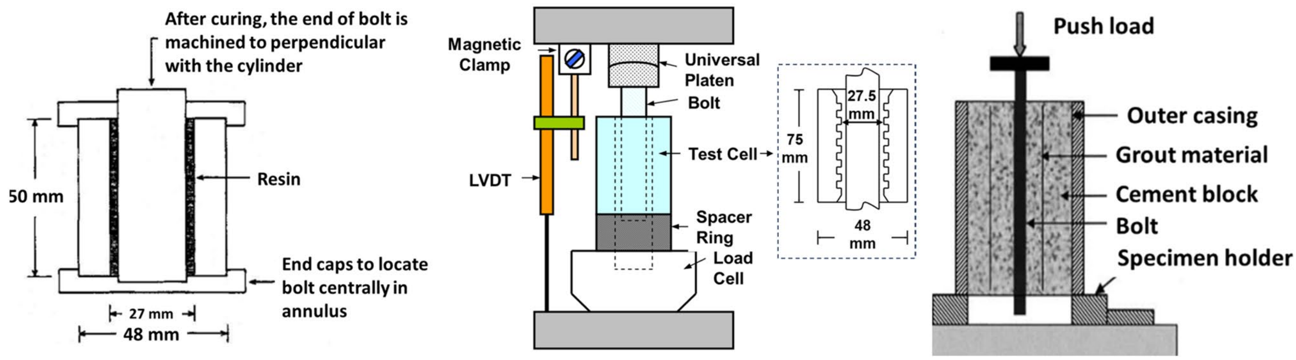
Push-out method	Pull-out method
The method does not represent the actual field condition as the tendon is in compression	Since the tendon is in tension, it can reflect a true load transfer mechanism similar to in situ conditions
The applied load is relatively high as an additional force is required to overcome the forward pushing force of the accumulated damaged encapsulation material particles in front of the tendon end	The method can be replicated in situ
	Less force is needed to pull the tendon out. As the damaged encapsulation material particles are left behind the pulled-out tendon end
The pushed tendon may cause the encapsulation material to dilate laterally with extra force	The pulled-out load may lead to a reduction in the tendon diameter with less force being generated laterally. While the state in the push test is in the reverse order
It is not easy to test cable bolts and glass fiber-reinforced polymer (GFRP) bars	Both cables and rock bolts can be tested
	It can be carried out statically as well as dynamically

2.1 Push-Out Testing Technologies

In push testing, an axial load is applied to the head of a fully grouted rock bolt until the bolt is pushed out of the hole into a hollow specimen (Karanam and Dasyapu 2005). Fabjanczyk and Tarrant (1992) reported on this method of testing to evaluate the load transfer characteristics of rock bolts by pushing them out of 50-mm-long steel tubes with internal thread (1 mm deep) (Fig. 3a). In this method, only rock bolts that were not rusted or stained with oil or paint should be tested. Later on, Aziz and Webb (2003) extended this study and engaged similar techniques using an encapsulation length of 75 mm (50% greater than the original version) to accommodate wider spaced rib profile bolts (Fig. 3b). Also, the test cell incorporated machined steel contained internal grooves for better interlocking between the encapsulation material and the steel tube. However, in the proposed apparatuses, the bolts were encapsulated directly into the steel tubes. According to Hyett et al. (1992), when the confining material is strong, more load is required to debond the tendon along the tendon–grout interface. Therefore, the application of steel tubes as the confining material overestimates the stiffness of the real field host medium, which might be misleading. On the other hand, the push-out rig designed by Karanam and Dasyapu (2005) used a concrete block as the host medium, and a steel casing was used for external confinement (Fig. 3c). Notwithstanding, the application of this equipment was also limited to the rock bolt with a diameter of 19.05 mm to prevent any possible bolt buckling during the tests.

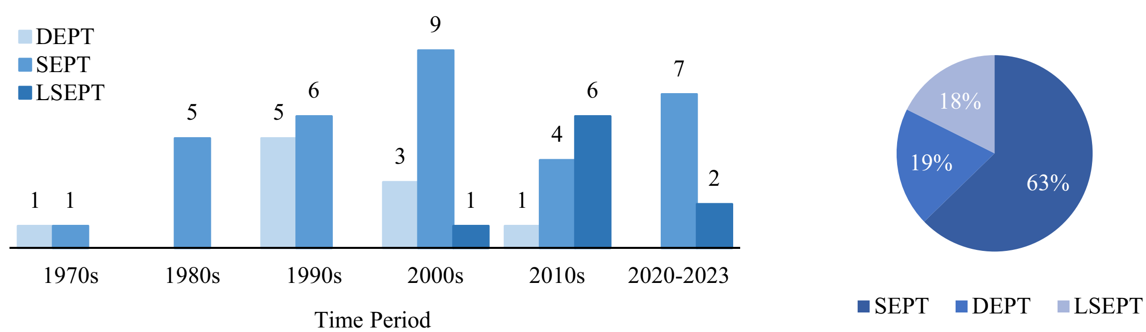
2.2 Pull-Out Testing Technologies

The percentage of different pull-out testing technologies is shown in Fig. 4. As can be seen, from the 1970s, researchers commenced to develop test devices based on the Double Embedment Pull Test (DEPT) and Single Embedment Pull Test (SEPT) methods. In DEPT method, the full length of the tendon is encapsulated in the double embedment within various encapsulation types and lengths (Thomas 2012). On the other hand, SEPT is a simpler way for testing tendons compared to DEPT, in which there is only one encapsulated section (Rastegarmanesh et al. 2022). In the next decades, the application of the



(a) (Fabjanczyk and Tarrant 1992) (b) (Aziz and Webb 2003) (c) (Karanam and Dasyapu 2005)

Fig. 3 The structure of apparatuses introduced for push-out testing



(a) (b)

Fig. 4 **a** Classification of test methods based on the type and year and **b** percentages of the different pull-out testing technologies

SEPT method considerably increased compared to DEPT. As shown, since the 1990s, the popularity of the DEPT techniques has gradually reduced. Therefore, it is obvious that most of the proposed rigs (about 63%) are designed for conducting SEPT. However, the SEPT method also comprises some limitations, especially in prevention of cable rotation. Thus, a new method, called the Laboratory Short Encapsulation Pull Test (LSEPT), has been applied to develop testing facilities since 2000s. The LSEPT system consists of two parts including anchor and embedment sections and the anchor part aims to restrict the cable rotation during the tests (Clifford et al. 2001). All in all, the available testing technologies have their own strengths and limitations which have been addressed and explained in the following sections. The basic features of the pull-out test devices introduced throughout the last decades are listed chronologically in Table 2. As shown, they have been categorized based on the types of host medium, confinement, and tested tendon.

The first difference between the testing methods is in the type of host medium where strength plays a major role in the LTC of tendons (Aziz et al. 2019). As shown in Fig. 5a, around 27% of the test designs steel pipes with diameters close to the borehole size have been used as the host medium. Meanwhile, the Modified Hoek Cell (MHC) and cement mortar medium have also been employed in some of the previous test arrangements. However, they may not be suitable as being representative of the real rock mass condition. In this regard, concrete specimens, which are artificial rock samples, have been widely used, (45%), to resemble rock mass conditions in pull testing. It is worth mentioning that one of the main reasons real rock samples were considered less favorable by the researchers, might be attributed to the difficulty of preparing such samples for the required size for testing purposes.

Another important issue is to provide a boundary condition, by simulating in situ condition using external confinement (Han et al. 2022). This helps to minimize crack propagation in specimens and maintain their rigidity until the end of the test (Benmokrane et al. 1995a). Once samples are cracked radially and laterally, their resistance to the pull-out load will be diminished (Anzanpour 2022). Despite this fact, it was found that almost one-third of the rigs did not use any form of medium confinement, because

Table 2 Features of the developed test arrangements for static pull-out testing of tendons

Developer	Type	Host medium		Medium confinement		Tested tendon
		Type	Shape	Type	Method	
Fuller and Cox (1975)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Oland and Callahan (1978)	SEPT	Concrete	Rectangular	–	–	Rock bolt
Natau and Wullschlaeger (1983)	SEPT	Concrete	Cylindrical	Steel tube	CRS	Cable bolt
Stillborg (1984)	SEPT	Concrete	Rectangular	–	–	Cable bolt
Stimpson (1984)	SEPT	Rock	Rectangular	–	–	Rock bolt
Farah and Aref (1986)	SEPT	Concrete	Cylindrical	Steel tube	CRS	Cable bolt
Aydan et al. (1987)	SEPT	Rock	Cylindrical	Triaxial cell	CRP	Rock bolt
Hutchins et al. (1990)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Goris (1990)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Hyett et al. (1992)	DEPT	Steel tube	Cylindrical	Steel or PVC tube	CRS	Cable bolt
Maloney et al. (1992)	SEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Yan (1992)	SEPT	Concrete	Rectangular	–	–	Rock bolt
McSporran (1993)	DEPT	MHC	Cylindrical	MHC	CRP	Cable bolt
Goto et al. (1993)	SEPT	Concrete	Rectangular	–	–	Rock bolt
Maruyama et al. (1994)	SEPT	Concrete	Rectangular	–	–	Rock bolt
Benmokrane et al. (1995a)	SEPT	Concrete	Cylindrical	Steel tube	CRS	Rock bolt
Benmokrane et al. (1995b)	SEPT	Concrete	Cylindrical	–	–	Cable/rock bolt
SCT (1996)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable/rock bolt
Satola and Hakala (2001)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable/rock bolt
Ito et al. (2001)	SEPT	Concrete	Rectangular	–	–	Cable/rock bolt
Clifford et al. (2001)	LSEPT	Rock	Cylindrical	Biaxial cell	CRP	Cable bolt
Aoki et al. (2002)	SEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Aoki et al. (2003)	SEPT	Concrete	Rectangular	–	–	Cable bolt
Wecker (2003)	SEPT	Rock	Cylindrical	Biaxial cell	CRP	Rock bolt
ASTM F432-19 (2004)	SEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Moosavi et al. (2005)	SEPT	MHC	Cylindrical	MHC	CRP	Rock bolt
Hassell et al. (2006)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
Oh and Kim (2007)	SEPT	Concrete	Rectangular	–	–	Rock bolt
BS 7861-2 (2009)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable bolt
BS 7861-2 (2009)	SEPT	Rock	Cylindrical	Biaxial cell	CRP	Cable/rock bolt
Ivanović and Neilson (2009)	SEPT	Concrete	Cylindrical	PVC tube	CRS	Rock bolt
Thomas (2012)	LSEPT	Rock	Cylindrical	Split steel tube	CRS	Cable bolt
Blanco-Martín (2012)	LSEPT	Rock	Cylindrical	Biaxial cell	CRP	Rock bolt
Holden and Hagan (2014)	LSEPT	Concrete	Cylindrical	–	–	Cable bolt
Aziz et al. (2016)	DEPT	Steel tube	Cylindrical	Steel tube	CRS	Cable/rock bolt
Li et al. (2016)	SEPT	Concrete	Rectangular	–	–	Rock bolt
Chen et al. (2016)	LSEPT	Concrete	Cylindrical	Split steel tube	CRS	Cable bolt
Thenevin et al. (2017)	LSEPT	Rock	Cylindrical	Biaxial cell	CRP	Cable/rock bolt
Tistel et al. (2017)	SEPT	Concrete	Rectangular	–	–	Rock bolt
Wang et al. (2018)	SEPT	Steel tube	Cylindrical	Steel tube	CRS	Rock bolt
Chen et al. (2018)	LSEPT	Concrete	Cylindrical	Split steel tube	CRS	Cable bolt
Yu et al. (2019)	SEPT	Concrete	Cylindrical	–	–	Rock bolt
Chong et al. (2021)	SEPT	Cement mortar	Cylindrical	Polyethylene pipes	CRS	Rock bolt
Høien et al. (2021)	SEPT	Steel tube	Cylindrical	Steel tube	CRS	Rock bolt
Anzanpour et al. (2021)	LSEPT	Concrete	Cylindrical	Split steel tube	CRS	Cable/rock bolt
Rastegarmanesh et al. (2022)	LSEPT	Concrete	Cylindrical	Split steel tube	CRS	Cable bolt
Han et al. (2022)	SEPT	Cement mortar	Cylindrical	PVC tube	CRS	Cable bolt
Shi et al. (2022)	SEPT	Concrete	Rectangular	–	–	Rock bolt

Table 2 (continued)

Developer	Type	Host medium		Medium confinement		Tested tendon
		Type	Shape	Type	Method	
Klar et al. (2023)	SEPT	Concrete	Cylindrical	Steel or PVC tube	CRS	Rock bolt
Li et al. (2023)	SEPT	MHC	Cylindrical	MHC	CRP	Rock bolt
Wang et al. (2023)	SEPT	Cement mortar	Rectangular	–	–	Rock bolt

of the rig's design simplicity (see Fig. 5b). On the other hand, other testing technologies used different kinds of confinement to provide two types of boundary conditions for the host medium, including Constant Radial Pressure (CRP) and Constant Radial Stiffness (CRS). The CRS can be provided via a biaxial cell, triaxial cell, or MHC. When the tests are performed under the CRP mode, the confining pressure is kept constant. In contrast, steel tubes or PVC pipes can be used to conduct the tests under the CRS mode, where rigid steel encloses the sample (Thenevin et al. 2017). As shown in Fig. 6a, it was found that, among the reviewed literature, 43% of test designs were used to test the axial performance of cables; in contrast, only 41% used rock bolts for similar studies. Only 16% of the rigs were designed to test both cables and solid rock bolts. As also shown in Fig. 6a, 75% of the testing facilities employed cylindrically shaped host medium, as it is much easier to provide uniform lateral confinement in cylindrical samples than rectangular samples. For the rectangular shape, the majority of tested tendons were rock bolts and all of them were based on the SEPT method. Besides, 7.7% of the rectangular samples were made of cement mortar and 92.3% of aggregate.

2.2.1 Double Embedment Pull Test (DEPT)

The features and limitations of previously developed DEPT rigs are presented in Table 3. As shown, the conventional DEPT apparatuses consisted of two cylindrical steel sections. It allows the study of the impact of embedment lengths on both sides of the discontinuity. Besides, the encapsulation lengths of these rigs varied from short bond length (100 mm) (Fuller and Cox 1975) to large bond length (2000 mm) (Satola and Hakala 2001). Moreover, in some rigs, the embedment lengths of these two sections are different. For instance, Fuller and Cox (1975) introduced the Split-Pipe Pull Test (SSPT), an apparatus that provided a range of bond lengths from 100 to 700 mm. In this rig, the length of embedment of two sections was different, leading to failure at the shorter length side during the tests.

On the other hand, in the experimental set-up presented by Hutchins et al. (1990), equal encapsulation length led to an equal chance of failure on both sides, which made the laboratory measurement difficult and invalid. Therefore, there was no failure along the interface, especially for the grout/rock interface, unless two different bond lengths were used. Another common issue among DEPT rigs is rigid lateral confinement (Fuller and Cox 1975; Hyett et al. 1992; Satola and Hakala 2001). Since the confining pressure provided by the thick steel tubes was far higher than the rock mass, the obtained results were usually greater than the real values achieved in the field. The stiff confinement overestimated the host medium strength. To address this issue, Hyett et al. (1992) modified the setup proposed by Hutchins et al. (1990) and used three different types of confinement, including steel, Aluminum, and PVC pipes, to examine the influence of radial wall stiffness of the confining pipe. Later on, McSporran (1993) proposed a modification of Hyett et al. (1992) pull test and used a modified Hoek cell to add different levels of radial confining pressure. However, the size of the Hoek cell limited the embedment length to 250 mm.

Another issue was tendon rotation during pull testing. Fuller and Cox (1975) eliminated the tendon rotation by securing the two embedment sections tightly together. In the rig developed by BS 7861-2 (2009), a pin between the two sections was used to prevent cable rotation. Aziz et al. (2016) used a rectangular 10-mm-thick steel channel, inserted on the split steel medium ensured non-rotation of the anchored cable during the test.

2.2.2 Single Embedment Pull Test (SEPT)

This system is widely used for pull testing of solid rebar bolts rather than cable bolts, as in this method the cable rotation has not been properly addressed. Oland and Callahan (1978), to examine the bond between the rock bolt and grout, designed a test set-up for pulling out the rebar from 152-mm concrete cubes. There are some issues with the proposed set-up. First, its application was limited to 19.05-mm-diameter rock bolts. Another issue is that their test set-up was not able to provide appropriate confinement to avoid concrete cracking during the tests. Therefore, the test outputs were not reliable as the samples cracked laterally. Natau and Wullschlaeger (1983) proposed a test machine for conducting large-scale pull-out tests (Fig. 7a). 4580 mm of the cable was anchored in a 596.4-mm-diameter concrete cylinder and

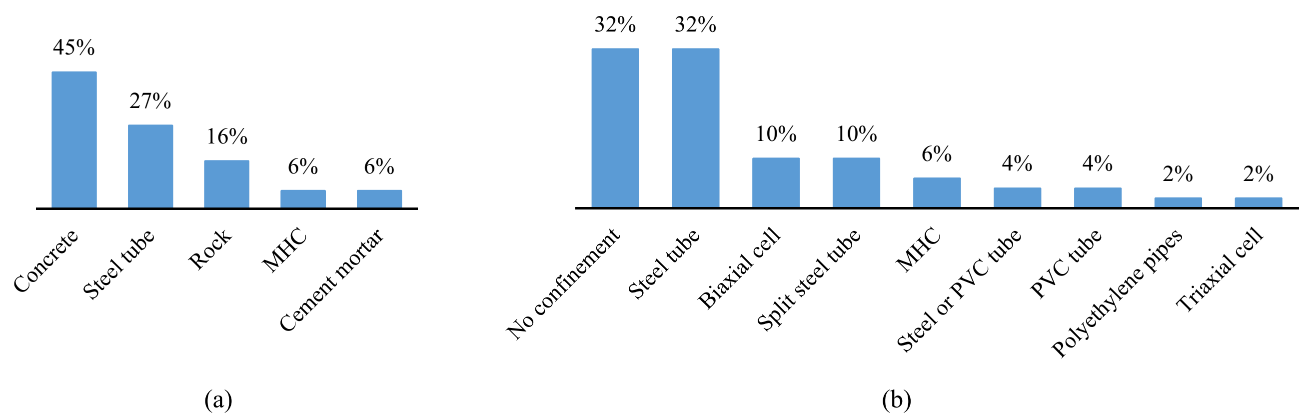
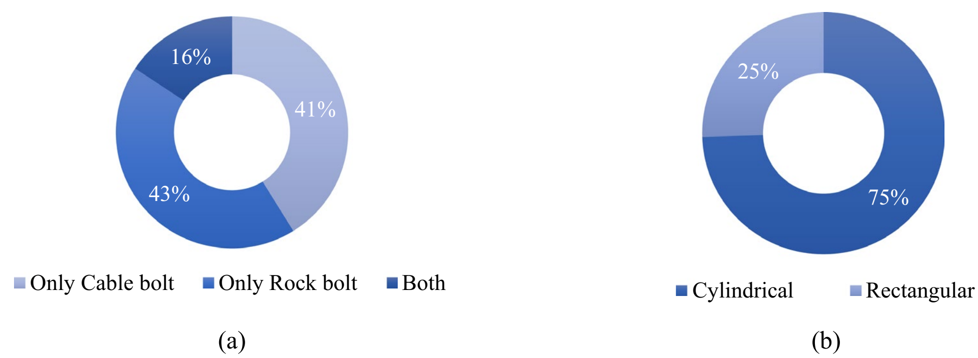


Fig. 5 Percentages of different types of **a** host mediums and **b** confinements employed in the previous testing facilities

Fig. 6 Percentages of different types of **a** tendons and **b** host medium shapes used in the previous testing facilities



a constant confinement pressure was provided by a 6.3 thick steel tube. The limitation of this study was that the concrete specimens were built in a way to only mimic the rock masses with elastic behavior.

Stillborg (1984) used $1000 \times 500 \times 300$-mm^3 concrete to simulate the rock mass in pull testing (Fig. 7b). Although it was a large-scale rig that enabled the researchers to test larger capacity cables in greater embedment length (up to 25 times the cable diameter), there was no external confinement for concrete medium, leading to cracking of some samples during the tests. Moreover, there was no rotation restriction for the free end of the cable bolt, which might considerably affect the outcomes. In the pull-out apparatus developed by Stimpson (1984), the effect of confining pressure was also ignored. In their experimental set-up, the rock bolt was encapsulated in a rock sample artificially built by gluing together several rock pieces. The capacity of the loading system was only 200 kN, which might be suitable for low-strength rock bolts.

Farah and Aref (1986) apparatus is equipped with a steel frame consisting of a special grip and an adjustable cage to test cables at various embedded lengths. Thus, once the test assembly was mounted on the loading system, the cage held the concrete sample, and the cable was pulled out through the bottom of the sample. Another test set-up designed in the 1980s belongs to Aydan et al. (1987). In this apparatus, a 200-mm-long cylindrical rock specimen (Oya tuff) with 120 mm of diameter was used as the host medium, which was laterally confined by a triaxial cell. However, due to the small size of the testing system, only low-strength rock bolts could be tested.

Maloney et al. (1992) introduced a simpler test configuration called SEPT for pull testing of cables with only one encapsulation section (Fig. 8a). Therefore, in the SEPT method, the encapsulated cable is pulled against a barrel and wedge. However, it was reported that the free section (unconstrained part) could lead to cable rotation. Yan (1992) developed a test set-up that was not strong enough to test

Table 3 Developed conventional DEPT apparatuses

<table>
<tr><th>Developer</th><th>Features and Comments</th><th>Apparatus Structure</th></tr>
<tr><td>Fuller and Cox (1975)</td><td>

Split-Pipe Pull Test

- The rig consisted of two cylindrical steel sections with different embedment lengths.
- A range of bond lengths from 100 mm to 700 mm were provided.
- To eliminate the tendon rotation during the tests, two embedment sections were tightly secured.
- The rigid steel section acted as a constant external confinement, overestimating the results.
- Only cable bolts were tested.

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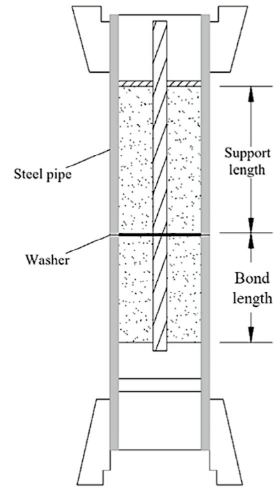

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<tr><td>Hutchins et al. (1990)</td><td>

Double Embedment Setup

- The test set-up consisted of two cylindrical steel embedment sections with equal lengths of encapsulation.
- Equal encapsulation length led to an equal chance of failure on both sides, which made the laboratory measurement difficult and invalid.
- The cables were encapsulated in stiff steel tubes, which overestimated the medium strength.
- In the vicinity of the threaded pulling head, stress concentration occurred due to excessive confinement.
- Only cable bolts were tested.

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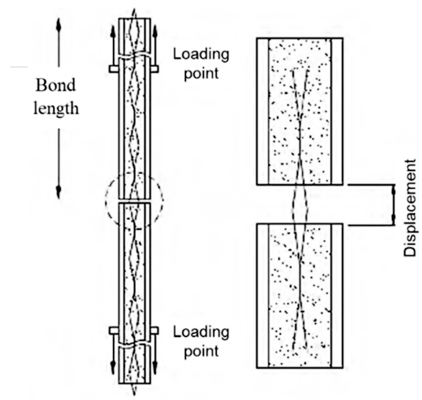

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<tr><td>Goris (1990)</td><td>

Long Double-Encapsulation Pull Test

- The test set-up consisted of two steel pipes with different lengths of encapsulation.
- The bottom steel pipe was the segment to be tested.
- To prevent slippage of the cable end encapsulated in the upper pipe, a barrel & wedge anchor were used.
- The cables were encapsulated in stiff steel tubes, which overestimated the medium strength. Thus, the load-displacement curves were not exactly what reinforcement systems experience in a large rock mass.
- Only cable bolts were tested.

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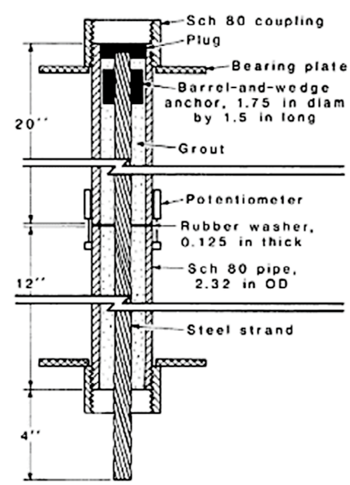

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larger capacity rock bolts. In addition, the bolt was installed in a $63.5 \times 152.4 \times 152.4\text{-mm}^3$ concrete sample without external confinement (Fig. 8b). In the test configuration presented by Goto et al. (1993), the anchor bolt pulled out of the concrete beam of dimensions $5d \times 24.5d \times 51d$, where d is the diameter of the bolt (Fig. 8c). In the design, the unbounded length of the anchor could be varied from 1 to 10d, which may lead to tensile failure prior to pull-out.

Maruyama et al. (1994) also designed a test set-up to examine the pull-out behavior of 16-mm-diameter anchor

Table 3 (continued)

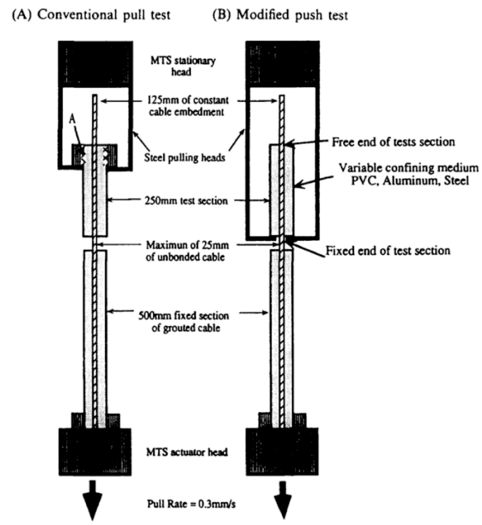
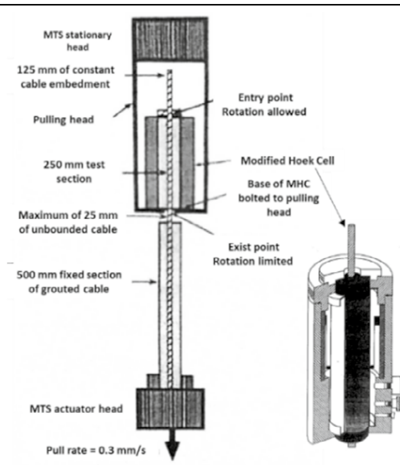
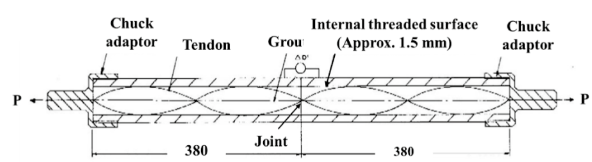
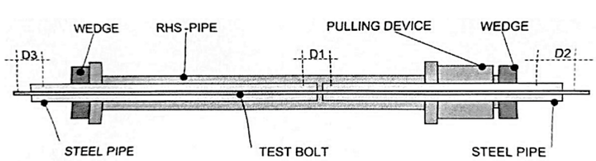
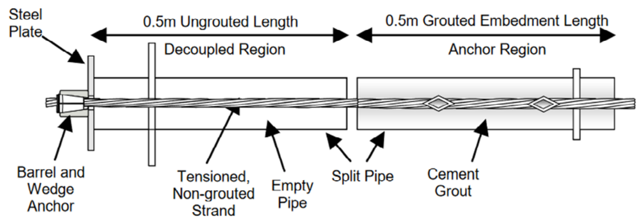
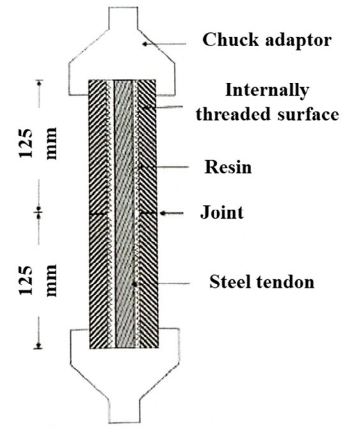
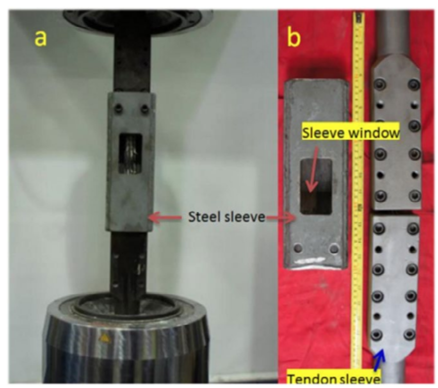
Hyett et al. (1992)	**Modification of Hutchins Pull Test** • Modified version of the rig proposed by Hutchins et al. (1990) • The apparatus involved two cylindrical steel sections with different embedment lengths. • Three different types of material, including steel, Aluminium, and PVC pipes, were used to examine the influence of radial wall stiffness of the confining medium. • The problem with stress concentration at the pulling head was addressed by resting the support test section. • Since two different embedment lengths were considered for the two sections, the pull-out could occur in the shorter section. • The pulling head covered the grout surface at the pull-out zone, affecting the pull-out initial stiffness. • The loading rate was quite high (0.3 mm/s), which did not represent the static loading condition. • Only cable bolts were tested.	
McSporran (1993)	**Modification of Hyett Pull Test** • Modified version of Hyett et al. (1992) pull test apparatus. • The equipment consisted of two cylindrical steel sections with different embedment lengths. Therefore, the pull-out could occur in the shorter section. • The modified Hoek cell was used to add constant radial confining pressure. • The size of the modified Hoek cell limited the embedment length to 250 mm and led to a 25 mm unbounded cable length between two embedded sections, which could lead to tensile failure of the cable before pull-out. • Only cable bolts were tested.	
SCT (1996)	**The Gun Barrel Pull Test** • This apparatus consisted of two cylindrical steel sections with the same embedment length (300 mm). • The equal embedment length led to an equal likelihood of failure on both sides. • It removed some of the uncertainties involved with pull-out testing, as free-end elongation corrections and compressive face loads were prevented. • Both cable and rock bolts were tested. • The problem of rigid steel confinement remained.	
Satola and Hakala (2001)	**Double Embedment Pullout Test Setup** • The test arrangement involved two cylindrical steel sections with different embedment lengths. Thus, the pull-out could occur in the shorter section. • It allowed pull testing in a 2000-mm long embedment length. • Both cable and rock bolts were tested. • The loading capacity of the arrangement was not high enough (350 kN). • The ultimate pull-out displacement was limited to 300 mm. • The problem of rigid steel confinement remained.	

Table 3 (continued)

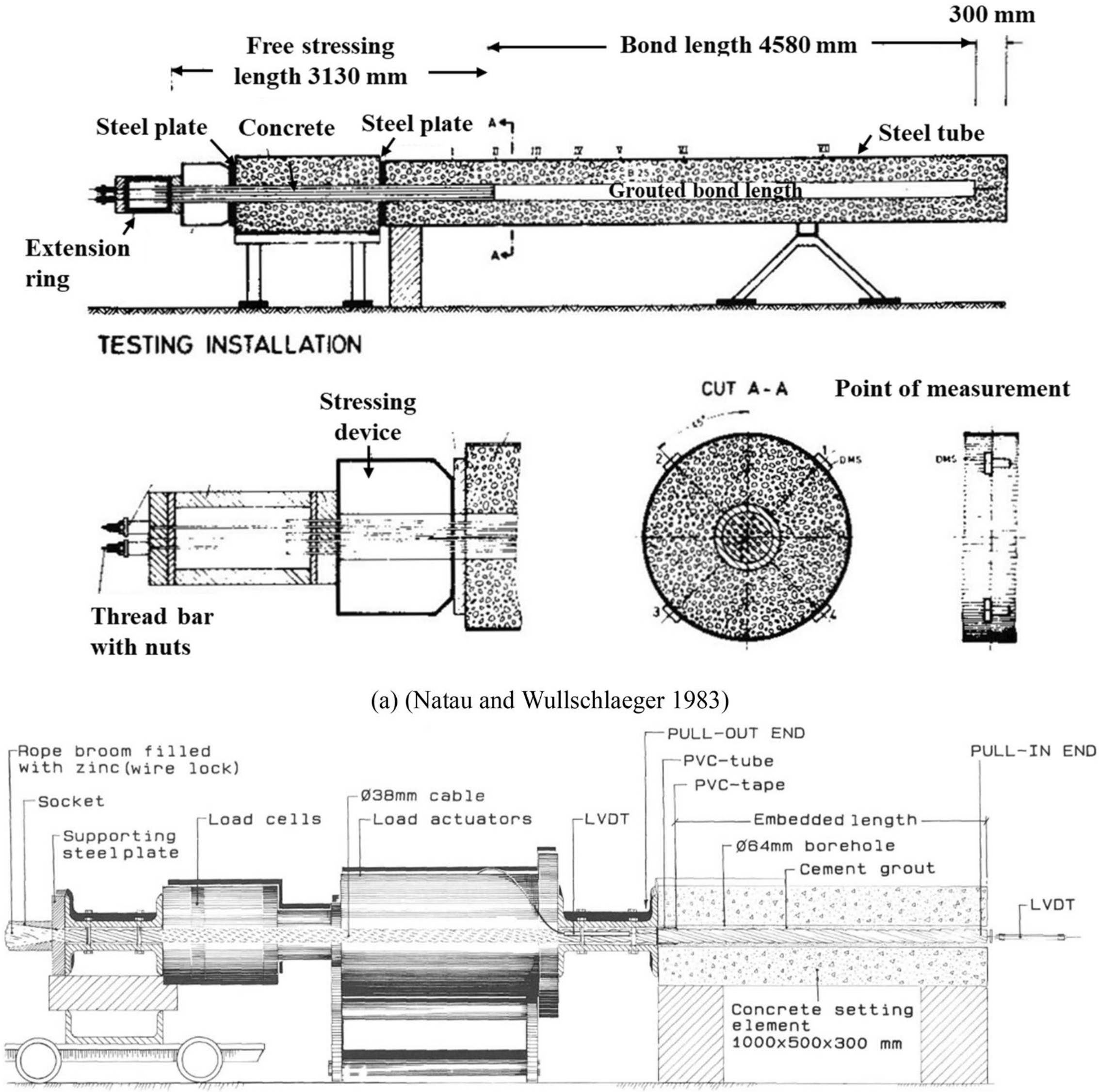
Table 3. Continued.

Developer	Features and Comments	Apparatus Structure
Hassell et al. (2006)	**Split Pipe Test** • The split pipe testing system involved two galvanised 15-mm thick steel pipes, each 500 mm long. • The top 500 mm was not grouted and was decoupled to allow for tensioning of the cable. • The thick still pipe might overestimated the results. • Only cable bolts were tested.	
BS 7861-2 (2009)	**British Standard Pull Test** • The first standard procedure for pull testing of cable bolts. • It consisted of two 125-mm long steel sections with no gap in between. • To induce failure at the cable/grout interface, steel tubes were internally threaded. • A pin between the two sections was used to prevent cable rotation during the test. • Due to rigid steel confinement, there was inconsistency between the laboratory and field results.	
Aziz et al. (2016)	**Short Encapsulation Double Embedment Steel Split Set** • It is a Short Encapsulation Double Embedment Steel Split Set (SEDESSS) apparatus. • Although the length of each section was equal, the tendon was encapsulated in different lengths on these sides (230 mm and 170 mm) to guarantee the failure on the shorter bond length. • To prevent rotation during the tests, an anti-rotation system was bolted to the steel sleeves. • Steel sleeve confinement overestimated the rigidity of the host medium. • Both cable and rock bolts could be used.	

bolts (Fig. 8d). Their test design, concerning the loading machine capacity (20 t) and the host medium size, was not strong enough to test high-strength anchors, and the influence of confining pressure was neglected. Benmokrane et al. (1995a) carried out the pull-out tests by fully encapsulating the steel rebar in the 1400-mm-long concrete cylinders with a diameter of 600 mm (Fig. 8e). However, since vibrating-wire strain gauges were micro-welded on the surface of bolts, a large hole diameter (127 mm) was required to accommodate the instrumented blot, which was much larger than the practical hole size (e.g., 76 mm).

In another research, Benmokrane et al. (1995b) developed short and long encapsulation pull-out test rigs (Fig. 8f). The embedment length varied between 4 and 20 times the diameter of the tendon, installed in a 200-mm-diameter concrete cylinder without lateral confinement. In their study, the loading ram capacity was limited to 270 kN (around 27 t) which was only suitable for low-strength tendons. Furthermore, the loading rate of 5 kN/s could not represent a static pull-out test as it is relatively quick.

The test arrangement was built by Ito et al. (2001) which was able to test both rock and cable bolts. They encapsulated 35-cm length of the tendons inside the concrete samples of dimensions $50 \times 50 \times 100$ cm^3 (Fig. 9a). The rate of loading was 0.05 kN/min, which was slow enough to perform the static pull-out test. Nevertheless, the compressive strength and elastic modulus of the concrete blocks were around 82 MPa and 35 GPa, only simulating the hard rock condition. Aoki et al. (2002) introduced a short encapsulation pull set-up. In their experimental design, 35 cm of the bulbed



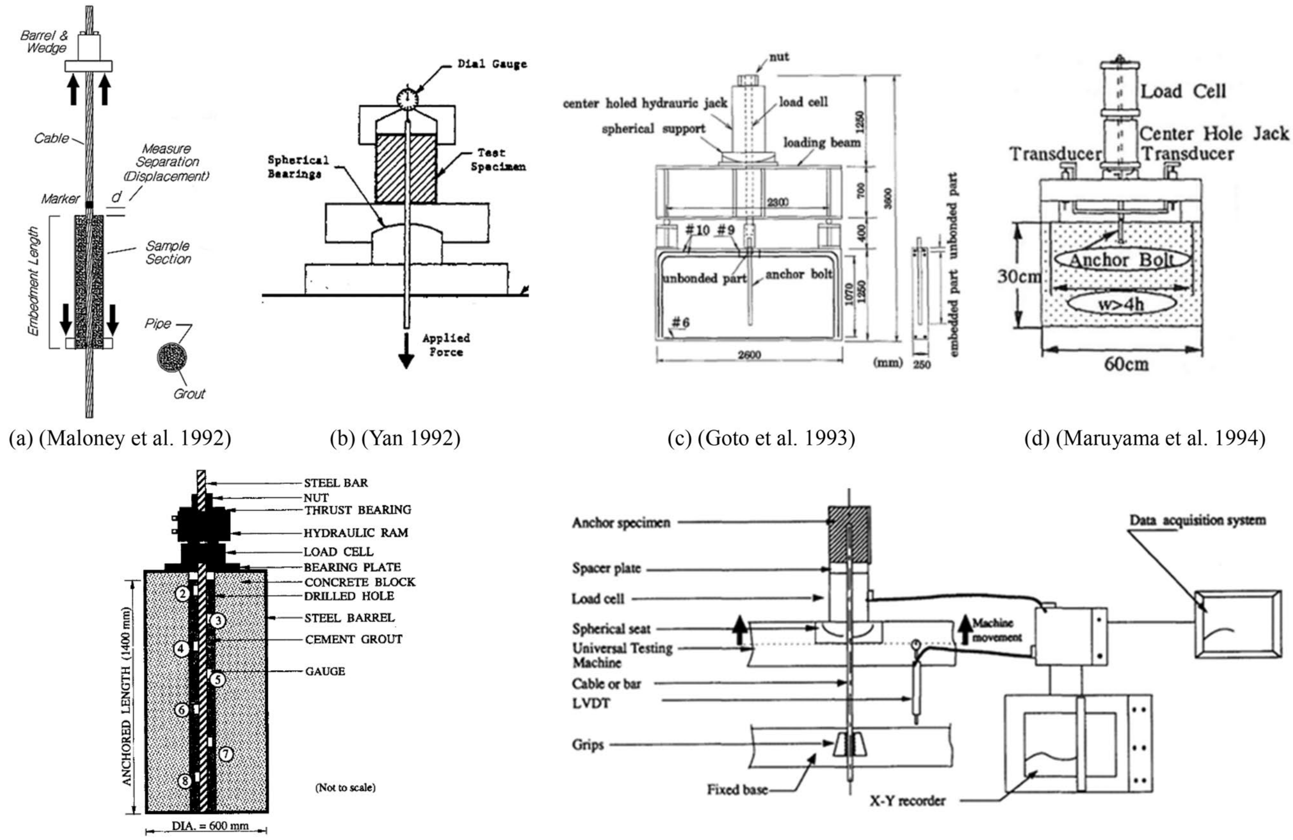
(a) (Natau and Wullschlaeger 1983)

(b) (Stillborg 1984)

Fig. 7 The structures of the developed SEPT rigs in the 1980s

cable was encapsulated inside the steel tube (Fig. 9b). However, their tests were not successful due to the steel confinement failure. In a new test device, Aoki et al. (2003) carried out some pull-out tests on bulbed cables embedded in the concrete cubes (Fig. 9c), but they used a loading rate of 10 kN/min, which was not able to represent the static pull-out test. In the same year, Wecker (2003) developed the bed separation pull test machine. Although, in this rig (Fig. 9d), the load was transferred through the biaxial cell to a rare bearing plate bolted to the bottom of the samples to simulate bed separation, there was no confinement or load at the top surface of the samples.

In 2004, the Speed Index test machine (Fig. 9e) is proposed by ASTM to examine the effectiveness of chemical grouting (ASTM F432-19 2004). This test machine was only used for cables and the rigid steel confinement overestimated the host medium strength. To provide a constant radial pressure on the test sample and also measure the dilation during the pull tests, Moosavi et al. (2005) used the MHC in their test set-up (Fig. 9f). As indicated, the MHC was instrumented internally by cantilever strain gauge arms, and 150 mm of the rebar was encapsulated in the test sample. Despite all advantages, the main drawbacks of this arrangement were that the embedment length was quite short, and the diameter of the sample was only 61 mm, which is the same size as the boreholes generally used to install the bolts in the rock mass. In the same year, Oh and Kim (2007) developed a testing method that was resembled to the rig



(a) (Maloney et al. 1992) (b) (Yan 1992) (c) (Goto et al. 1993) (d) (Maruyama et al. 1994)

(e) (Benmokrane et al. 1995a) (f) (Benmokrane et al. 1995b)

Fig. 8 Structures of the developed SEPT rigs in the 1990s

developed by Farah and Aref (1986), but they used rectangular concrete specimens instead of cylindrical samples. As presented in Fig. 9g, to avoid the entire concrete sample being pulled out during the tests, a specially assembled steel frame was employed, consisting of four steel rods and two steel plates with a hole in the bottom plate. The sample was situated on the bottom plate and the bolt was pulled out through the bottom hole at the loading rate of 0.3 mm/min, which was adequately slow to represent the static pull test. However, in this rig, there was no external confinement, which might be unrealistic.

In 2009, an apparatus was presented by the British Standard Institution (BS 7861-2 2009). As shown in Fig. 9h, the biaxial cell was used for confining the sandstone rock sample up to 10 MPa. The new set-up was able to test both rock bolts and cables. Meanwhile, the loading rate of 1 mm/min was the advantage of their apparatus, which represented the static test condition. However, the restriction for cable rotation was overlooked in this standard. In addition, the confining applied by the biaxial pressure cell was fixed at 10 MPa which is not necessarily a true value and does not replicate the ground conditions. The confining pressure is a dynamic value in the cable bolt life; however, it was constant in this study. Ivanović and Neilson (2009) developed a testing equipment similar to the design of Oh and Kim (2007). They anchored 22-mm-diameter rock bolts in 200-mm-diameter concrete cylinders confined by thin-wall PVC pipes. The length of the embedment varied between 200 and 400 mm. Besides, to avoid the entire concrete cylinder being pulled out during the tests, a specially assembled steel frame was employed, consisting of two steel plates placed at the top and bottom of the sample with a hole in the top plate. This hole was designed to allow space for the bolt and the Linear Variable Differential Transformer (LVDT) (Fig. 9i). However, it was deemed that the thin-wall plastic confinement could not be representative of real field conditions.

Li et al. (2016) used a test arrangement (Fig. 10a) for testing rock bolts encapsulated in a 110-MPa strength cubic concrete sample with a dimension of $950 \times 950 \times 950$ mm^3. In their design, they employed a barrel and wedge instead of nut and thread to prevent premature failure of the rock bolt due to threading. Therefore, the bolt head is not weakened because of threading. They tested different lengths of embedment varying from 100 to 300 mm. Meanwhile, the free length of the bolt was 600 mm. In those tests in which long embedment might lead to shank failure, the tests were

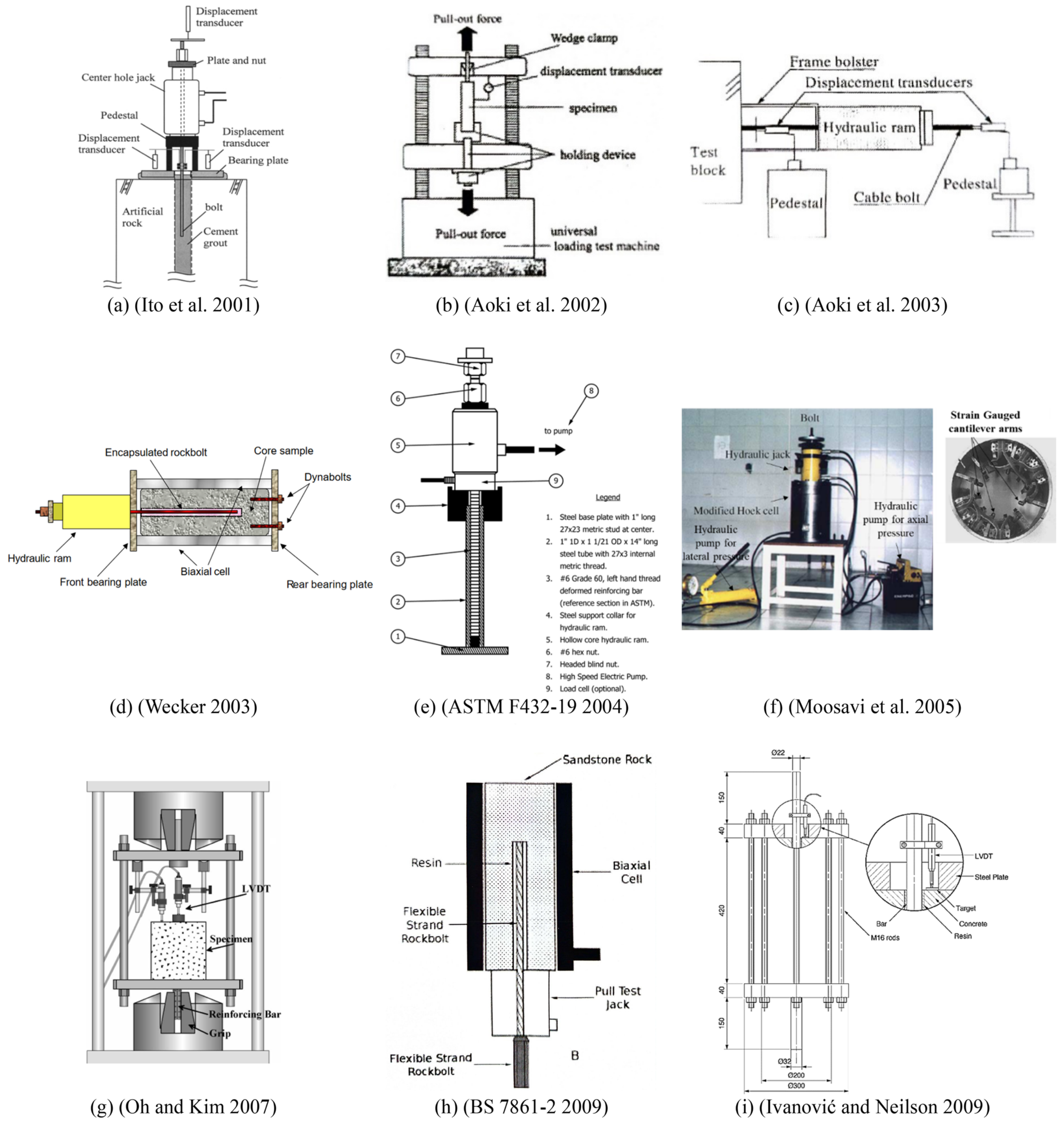


Fig. 9 The structures of the developed apparatuses for pull-out testing in the 2000s

terminated intentionally once the load reached the tensile strength of the bolt.

Similar to Li et al. (2016), Tistel et al. (2017) employed large 90-MPa strength concrete cubes ($1 \times 1 \times 1$ m^3) as the host medium for pull testing of rock bolts. They used a frame to install the bolts in 400-mm-long holes for maintaining their accurate vertical position during curing. Four different embedment lengths, including 70 mm, 200 mm, 250 mm, and 270 mm, were applied in this study. As can be seen from Fig. 10b, a 200-mm-long plastic pipe was used to isolate the encapsulated length of 70 mm from the ungrouted length. In this test arrangement, the static load rate was adjusted in each test from 0.12 to 0.47 kN/s based on the embedment length of the bolt, indicating the static test condition.

In another research, Wang et al. (2018) anchored 25-mm-diameter rock bolts in 75-mm-diameter and 100-mm-long steel pipes to examine the mechanical performance of the bolt under high temperatures (Fig. 10c). Three types of 5-mm-thick steel pipes, including single-threaded, double-threaded, and non-threaded, were used to examine the influence of internal wall roughness. However, the constant normal confinement provided by the steel pipe may overestimate the ultimate pull-out force. Yu et al. (2019) proposed Pull-out Testing Machine (PMT) and applied 1500-mm-long embedment length in their test design (Fig. 10d). The host medium was simulated by 150-mm-diameter cylindrical concrete samples with a length of 1500 mm. The tests were conducted by a 300-kN capacity hollow hydraulic jack. Although the test arrangement was able to apply confining pressure of 20 MPa, in this study, no lateral confinement was applied, resulting in concrete radial cracks during the tests.

Chong et al. (2021) utilized a large-scale rig (Fig. 11a), allowing them to carry out pull-out tests at different lengths of embedment from 200 to 1100 mm. Bolts were anchored in cylindrical cement mortar samples cast in 260-mm-diameter polyethylene pipes and tested using a 60-t capacity loading ram. In this research, boreholes were rifled and only rock bolts were tested. However, it was shown that the host medium and the medium confinement were not strong enough to prevent radial cracks during the tests. In the experimental set-up of Høien et al. (2021), an internally threaded steel tube with a diameter of 46.5 mm was used as the host medium to provide constant confinement or constant radial stiffness (Fig. 11b). The free length of the rock bolt was 600 mm. Moreover, the embedded length of the bolt varied from 300 to 600 mm. In this experimental set-up, a spacer was employed between the steel pipe and the jack to let the bolt form a cone where it exited the grout, allowing it to simulate the joint widening scenario. Nonetheless, the steel tube overestimated the stiffness of the rock mass.

Han et al. (2022) developed a test apparatus for testing cables as indicated in Fig. 11c. Cables were encapsulated in a 150-mm-diameter cylinder with a length of 120 mm and only cement mortar was used as the host medium, which may not represent the actual field condition. In addition, even though the rate of displacement was low enough (0.06 mm/s) to represent the static test condition, the capacity of the loading system was limited to 300 kN (about 30 t), which is only appropriate for testing of lower strength cables. Furthermore, the cable rotation was not restricted in this test design. In another test machine (Fig. 11d) developed by Shi et al. (2022), 250-mm length of the rock bolts was encapsulated in the concrete cubes ($300 \times 400 \times 400$ mm^3). In this research, a constant displacement rate of 0.005 mm/s (0.3 mm/min) was applied which was low enough to mimic the static test state. The issue with this device was that the effect of lateral confinement was ignored.

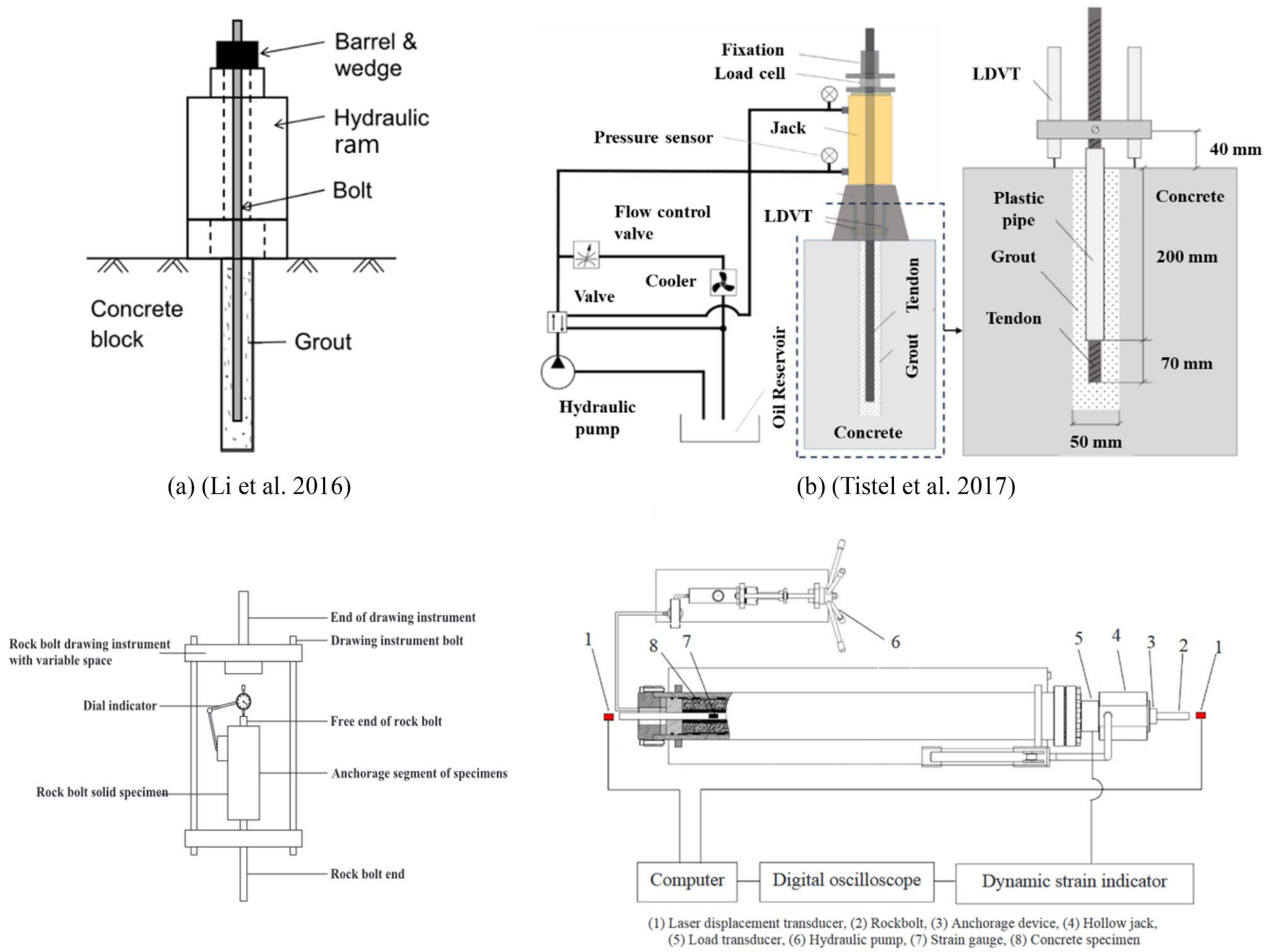
Klar et al. (2023) employed a test configuration shown in Fig. 11e. To investigate the influence of medium confinement, various types of 200-mm-long testing pipes, including steel, copper, and PVC, were used. The diameter of the pipes varied from 100 to 114 mm. In addition, a greased PVC tube was put around the bolt at the base of the casting platform to decrease the edge effect when pulling the bolt. The tests were performed under a constant displacement rate of 0.3 mm/s. However, instead of using concrete or rock samples, the grout was used as the host medium.

Li et al. (2023) used the MHC to provide constant radial pressure during the tests, allowing them to examine the influence of different confining pressures (Fig. 11f). In this experimental set-up, 140-mm lengths of the rock bolts were anchored in a 70-mm-diameter cylindrical cavity that existed inside the MHC. Nevertheless, the host medium was not representative of the real field condition. Later on, Wang et al. (2023) used a testing device (Fig. 11g), in which a 120-mm length of the rock bolts was embeded in the grout cubes ($150 \times 150 \times 150$ mm^3) without any external confinements, leading to radial cracks during the tests. In this research, to simulate static test conditions, the loading rate was set at about 0.1 kN/s.

2.2.3 Laboratory Short Encapsulation Pull Test (LSEPT)

The main issue with the SEPT of cables is the ungrouted section of the cable. This free section (unconstrained) led to cable rotation during the tests due to its low torsional stiffness. Thus, the bond strength is lower than the real field condition (Hutchinson and Diederichs 1996). In fact, rotation is an unrealistic phenomenon as cables cannot unscrew in real field conditions (Thenevin et al. 2017). To overcome this problem, the LSEPT was originally developed by Clifford et al. (2001). The LSEPT system consists of two parts including anchor and embedment sections. In this system, the free length of the tendon is encapsulated in a long tube called an anchor tube, which is used to transfer the pull-out load to the tendon and acts as an anti-rotation element by firmly griping the tendon during the test. Moreover, unlike the conventional DEPT setups, in which tendons were fully encapsulated in two stiff steel sections, in LSEPT, the end of the tendon is encapsulated in a cylindrical sample (embedment length) (Rastegarmanesh et al. 2022). Therefore, this method does not restrict lateral dilation, influencing the magnitude and distribution of stress within the rock mass (Holden and Hagan 2014). As shown in Fig. 12, in the field condition, the length of the tendon is far away from the excavation boundary, representing the anchor section. Further, the embedment section is in the vicinity of the excavation margin, simulating the rock mass sliding from the tendon occurring in the field condition (Chen et al. 2018).

The features and limitations of the previously developed LSEPT arrangement are introduced in Table 4. Based on the literature, Clifford et al. (2001) employed the MHC for



(a) (Li et al. 2016) (b) (Tistel et al. 2017) (c) (Wang et al. 2018) (d) (Yu et al. 2019)

Fig. 10 The structures of the developed rigs for pull-out testing in the 2010s

providing confinement. Blanco-Martín (2012) and Thenevin et al. (2017) used the biaxial cell to provide either constant radial pressure or constant radial stiffness for medium confinement. In their study, the pressure vessel and bladder were adopted to apply confinement pressure and constant stiffness up to 25 MPa. In other research, the biaxial cell and the MHC were replaced with the split steel tube, providing constant radial stiffness, to simplify the tests (Thomas 2012; Chen et al. 2016, 2018; Anzanpour et al. 2021; Rastegarmanesh et al. 2022).

It should be noted when a split steel tube was used for confinement by different authors, it was reported that the samples were cracked radially due to the hairline gap that existed between the two cylindrical steel clamps bolted tightly together. Meanwhile, Holden and Hagan (2014) did not apply any lateral confinement in their test configuration, leading to sample cracking. All in all, when the test samples are cracked, the outcomes might not be reliable. To overcome this problem, Anzanpour et al. (2021) introduced the Reverse Pull-out Test Machine (RPTM) and employed samples which were directly cast in the steel tube to provide uniform radial confinement. Moreover, it was found that some slippage might occur in the anchor tubes. Thus, Chen et al. (2016) applied a longer anchor tube (608 mm) threaded internally in their test machine to remove any possible slippage in this section. Also, in another research, Chen et al. (2017) employed this modified design and tested samples with various diameters ranging from 150 to 508 mm. They concluded that, in confined samples, the maximum pull-out leveled off when the diameter was larger than 300 mm.

Another issue that was undertaken in the previous studies is reducing the embedment length during the test. In LSEPT, as the cable is pulled out of the sample, the embedment length gets shorter, meaning that the load during the post-peak behavior is compromised. For this, in 2018, Chen et al. (2018) added a 90-mm-long ungrouted section to the end

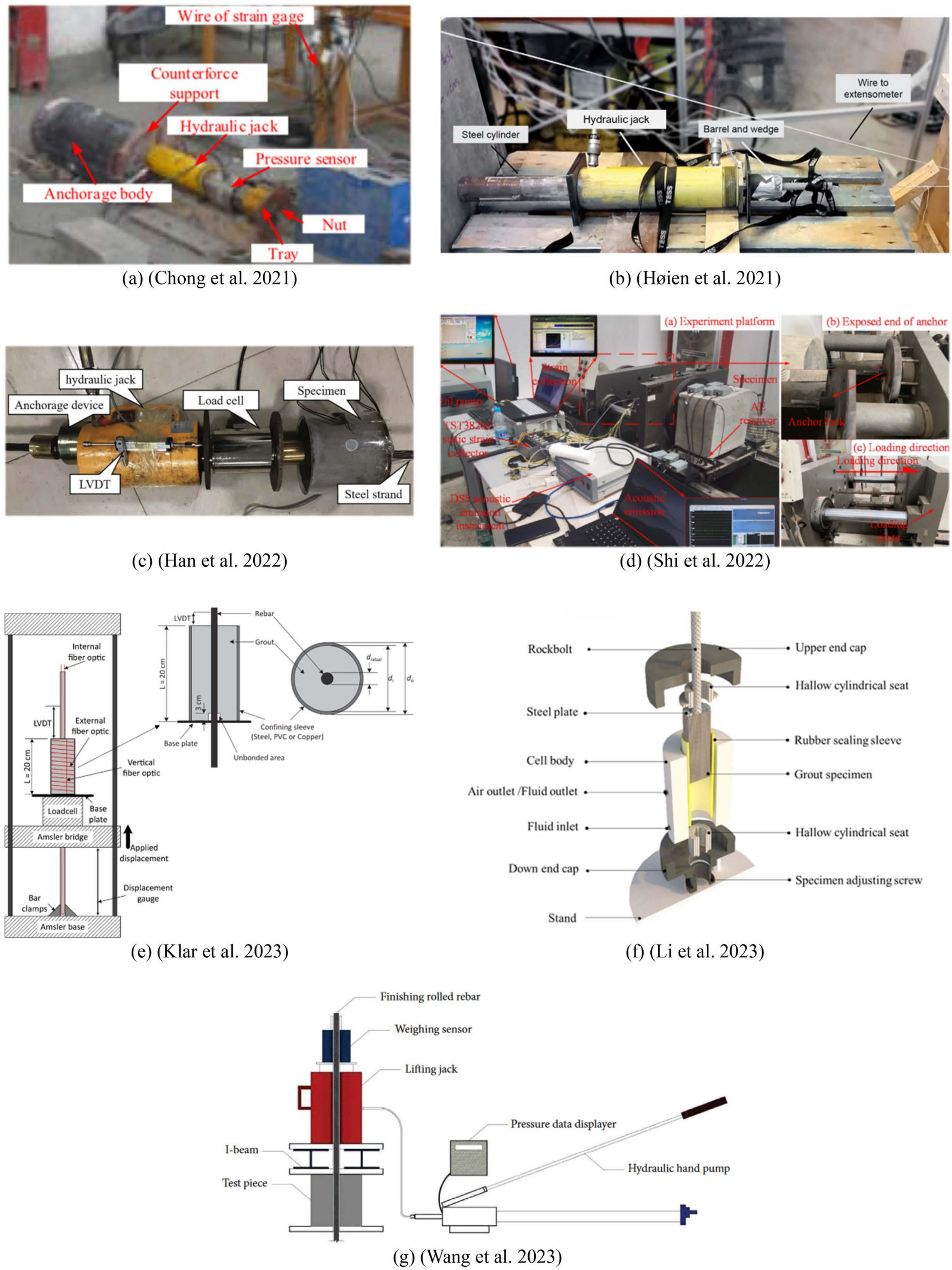


Fig. 11 The structures of the developed rigs for pull-out testing in the 2020s

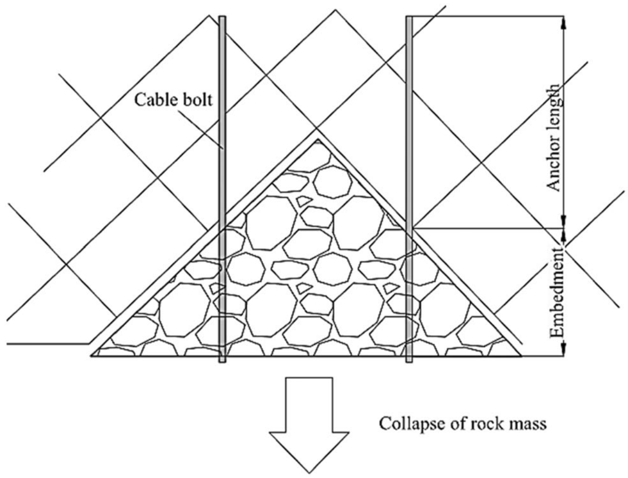


Fig. 12 A normal failure form of the tendon in the field condition (Chen et al. 2018)

of the borehole. Hence, the engaged length remained constant during the test. The new arrangement was also used in another research (Li et al. 2018). Later on, Anzanpour et al. (2021) and Rastegarmanesh et al. (2022) increased this ungrouted length to 150 mm in their test setups. Furthermore, it should be noted that the long-grouted anchor tubes are load-bearing elements; hence, they must be capable of resisting the axial capacity of the tendon. In addition, purchasing and machining of the hollow tubes are costly as anchor tubes are required to be replaced after each test. To address this issue, Rastegarmanesh et al. (2022) employed barrel and wedge in their experimental set-up to transfer the axial load to the cable instead of an anchor tube. It was concluded that the barrel and wedge are more affordable and their replacement is much easier. Hence, they showed that the anchor tubes can be noticeably smaller and only act as the anti-rotation part.

3 Shear Loading of Tendon Support System

As shown in Fig. 13, to evaluate the shear performance of tendons, since 1970, researchers commenced laboratory tests using the Single Shear Test (SST), in which the tendon is sheared through one shear plane. Around 47% of the available shear rigs were based on the single shear method. The application of SST has gradually increased since then and it is still being used as a method for assessing tendon shear behavior. In the following two decades, a new method, in which a combined loading mode has been used, and new methods have also been developed. When reinforced rock blocks in an underground opening displace downwards, it stimulates both joint dilation and shear (Bawden et al. 1994). Therefore, the combined methods try to perform pull-and-shear loading of tendons. However, combined axial and shear tests are complicated and need specialized experimental facilities (Hutchinson and Diederichs 1996). As can be seen from Fig. 13, about 23% of the proposed test set-ups have been built based on the combined method. It has been found that, although the application of combined methods reduced in the 2000s and 2010s, it has increased again in the 2020s, which significantly relates to the availability of advanced laboratory equipment.

Further, in addition to SST, another common method for shear testing is known as the Double Shear Test (DST). This method involves shear testing at two points on the tendon (Aziz et al. 2003). It is shown that DST rigs proposed since the late 1990s, forming a quarter of the developed rigs. It is worth mentioning that, if proper apparatus dimensions and boundary conditions are applied in the experiments, both SST and DST can produce reasonable shear performance (Li et al. 2017). It should be noted although the DST rig is symmetrical, the DST results do not simply double the SST results. Because true symmetry does not exist in practice (Haile et al. 1995). Moreover, as shown in Fig. 14, similar to the axial loading of tendons, Australia has been a leader in the field of developing apparatuses for shear loading.

The basic features of the shear rigs introduced throughout the last decades are listed chronologically in Table 5. The results show that about 26% of the shear test set-ups allow shear testing of tendons at different angles of inclination to the shear surface, whereas 74% of them are only able to test tendons oriented perpendicular to the joint plane (Fig. 15a). It can be seen from Fig. 15b that the majority of the test arrangements employed concrete samples as the host medium (64%), and the second most used material was rock samples. Other types of host mediums (e.g., steel tube) were also used, although they might either underestimate or overestimate the rock mass strength. Aziz et al. (2003) reported that the strength of the host medium influenced the shear load level; hence, it is important to pay attention to the type of host medium in such tests. In addition, Li et al. (2017) found that a stiff squeeze contact between the tendon and steel tubes could lead to stress concentration and consequently premature failure of the tendon. It indicates that the diametrical difference between the tendon and the confining medium is not sufficient (mainly due to limited thickness of the encapsulating grout) when small-diameter steel tubes are used.

Another aspect that can be mentioned is the effect of frictional resistance on the shear plane. According to Aziz et al. (2015), in shear testing, around 30% of the shear force is used to overcome the roughness between joint surfaces. Thus, friction may overestimate the shear strength of the tendons. Furthermore, joint friction could lead to shearing

Table 4 Developed LSEPT apparatuses

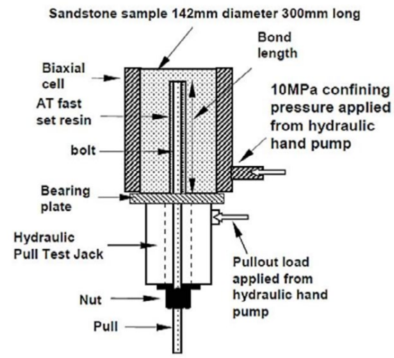
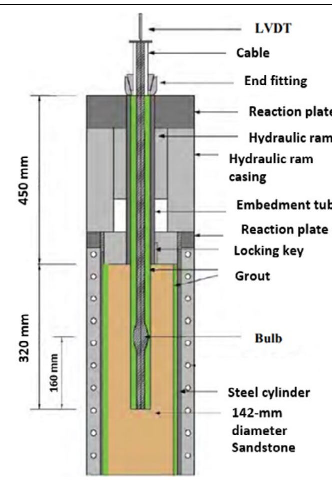
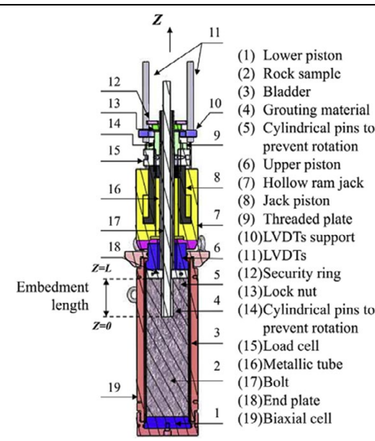
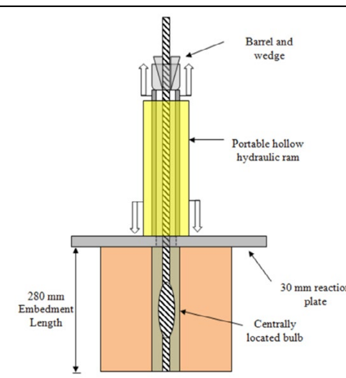
Developer	Features and Comments	Apparatus Structure
Clifford et al. (2001)	**Biaxial Loaing Pull Test** • Combined the idea of DEPT and the MHC to develop the LSEPT method. • 142-mm diameter sandstone samples were used as the host medium. • MHC allowed to apply different lateral pressures up to 10 MPa. • A 500-mm long steel tube was used as the anchor tube to eliminate cable rotation during the tests. • Embedment length and anchor length were 320 mm and 500 mm, respectively. • Procurement and machining of the hollow tubes is costly as anchor tubes required to be restored after each test • Only cable bolts were tested.	
Thomas (2012)	**Modification of Clifford Pull Setup** • It is modified version of the test facility developed by Clifford et al. (2001). • Barrel & wedge were applied to fix the cable. • Steel tube was used as the anchor tube to eliminate cable rotation during the tests. • 142-mm diameter sandstone samples were used as the host medium. • For simplification, the MHC was replaced by a split steel tube for confining the sample. • To apply hole rifling, six various drill bit sizes were used. • 450 mm of the cable was encapsulated in the anchor tube placed in the ram casing and 320 mm of embedment length in the rock sample. • The load was applied at the rate of 10 kN/s, which could not be representative of the static test condition. • The lateral confinement was not strong enough and radial cracks were observed during tests. • Procurement and machining of the hollow tubes is costly as anchor tubes required to be restored after each test • Only cable bolts were tested.	
Blanco-Martín (2012)	**Improved Confinement Pull Test** • The medium confinement was done by a biaxial cell up to 25 MPa. • The boundary condition can be either constant radial stiffness or constant confining pressure using a biaxial cell. • The embedment length hardly exceeded 250 mm. • The rate of displacement was less than 1 mm/s, which represents a static test condition. • Sandstone cylindrical samples were used as the host medium. • The end plate placed on top of the sample covered the grout column, adding extra resistance against the pull-out load. • Only rock bolts were tested.	
Holden and Hagan (2014)	**Unconfined Single Embedment Setup** • Employed a 600 kN capacity RCH606 hollow hydraulic ram for loading. • Steel tube was used as the anchor tube to eliminate cable rotation during the tests. • The embedment length was 280 mm. • The boreholes were rifled. • The plate placed on top of the concrete covered the grout column, adding extra resistance against the pull-out load. • There was no lateral confinement and concrete cracks during the tests, meaning the results were not reliable. • Procurement and machining of the hollow tubes is costly as anchor tubes are required to be restored after each test. • Only cable bolts were tested.	

Table 4 (continued)

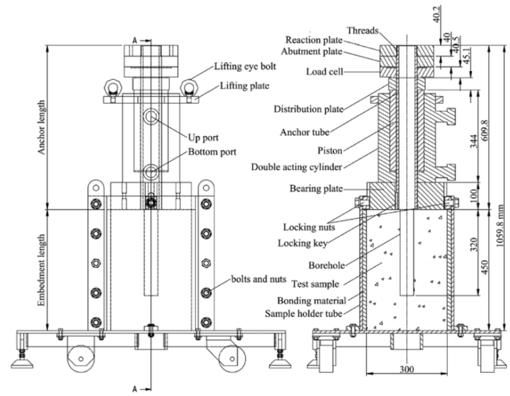
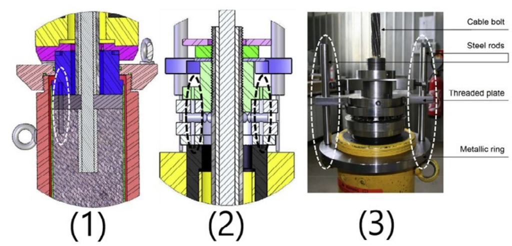
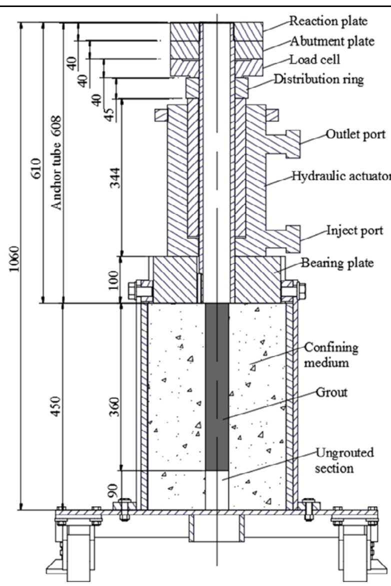
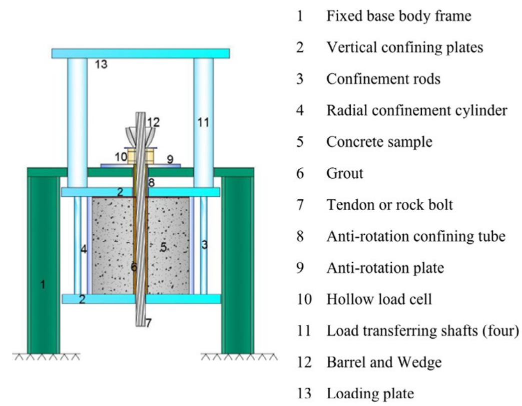
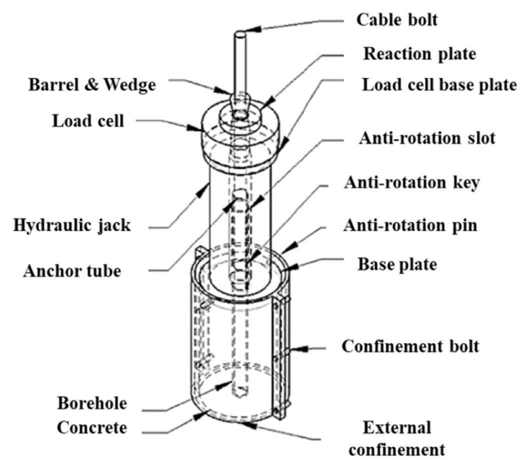
Chen et al. (2016)	**LSEPT Rig** • It is a modified version of the test facility developed by Holden and Hagan (2014). • The embedment length increased to 320 mm. • The boreholes were rifled. • Concrete samples with a length of 450 mm and a diameter of 300 mm were employed as the host medium. • A split steel tube was used to provide passive confinement to the concrete sample. • The cable was gripped by the anchor tube during the tests to prevent rotation. • A 608-mm long anchor tube, threaded internally, was used to remove any possible slippage in this section. • The 950-kN capacity ram was used for loading. • The displacement rate was 0.3 mm/s, which represents the static pull test condition. • The hairline gap between the steel clamps led to radial cracks during the tests, especially when bulbed cables were tested. • Procurement and machining of the hollow tubes is costly as anchor tubes required to be restored after each test • Only cable bolts were tested.	
Thenevin et al. (2017)	**Modification of Improved Confinement Pull Test** • It is a modified version of the test facility developed by Blanco-Martín (2012). • Anti-rotation parts were modified to be able to test cables. • The rock and the fixed part of the jack were linked together by three cylindrical steel pins that were placed in three holes in the rock sample, the end plate, and the upper piston (1). • The rock and the fixed part of the jack were interdependent; so, no relative rotation was allowed between them (2). • Two pins were inserted between the jack piston and the threaded plate so that any possible rotation between that piston and the anchor tube was blocked (3). • To prevent relative movement between the fixed part of the jack and its piston, a metallic ring was screwed around the fixed body of the jack and blocked in rotation using three cone-point screws (3). • A barrel & wedge system was mounted on top of the anchor tube to prevent the tube from sliding off the cable.	
Chen et al. (2018)	**Modified LSEPT Rig** • It is a modified version of the test facility developed by Chen et al. (2016). • Length of embedment increased to 360 mm. • A 90-mm long ungrouted section was added to the end of the borehole, so the engaged length stays constant during the test. • The boreholes were rifled. • Concrete samples with a length of 450 mm and a diameter of 300 mm were employed as the host medium. • A split steel tube was used to provide passive confinement to the concrete sample. • The cable was gripped by the anchor tube during the tests to prevent rotation. • The displacement rate was 0.3 mm/s, which represents the static pull test condition. • The hairline gap between the steel clamps led to radial cracks during the tests. • Procurement and machining of the hollow tubes is costly as anchor tubes required to be restored after each test • Only cable bolts were tested.	

Table 4 (continued)

	Reverse Pull-out Test Machine (RPTM)	
Anzanpour et al. (2021)	• RPTM is based on push-to-pull loading of the tendon. • Both static and dynamic loading can be carried out. • The loading system was able to apply loads at a rate of 0.5 mm/min, representing the static test condition. • Cylindrical concrete samples with 300 mm of diameter and 450 mm of height can be used as the host medium. • The embedment length was 300 mm. • A 160-mm long anti-rotation anchor tube was used along an anti-rotation plate to restrain cable unwinding. • The 150 mm long of cable was encapsulated in a PVC tube to keep this section of the cable ungrouted. • The external split steel tube provides radial confinement around the concrete sample. • Due to the hairline gaps existing between the steel clamps; radial cracks may occur during tests. • Concrete samples can be cast directly in steel tubes to provide a uniform confinement around the sample. • Both cable bolts and rock bolts can be tested.	
	Modified LSEPT	
Rastegarmanesh et al. (2022)	• It is a modified version of the test facility developed by Chen et al. (2018). • A more robust loading ram (100-t capacity) was used to test stronger cable bolts. • Barrel & wedge were applied to transfer the axial load to the cable. • Concrete samples with a length of 450 mm and a diameter of 300 mm were used as the host medium. • The boreholes were rifled. • The first 150 mm of the cable was wrapped with a non-porous foam to keep this section of the cable ungrouted. • The embedment length was 300 mm. • 12.7-mm thick split steel tube with a diameter of 330 mm and length of 500 mm was used to provide constant normal stiffens confinement. • The hairline gap between the steel clamps led to radial cracks during the tests. • The loading rate was approximately 6 mm/min. • Only cable bolts were tested.	

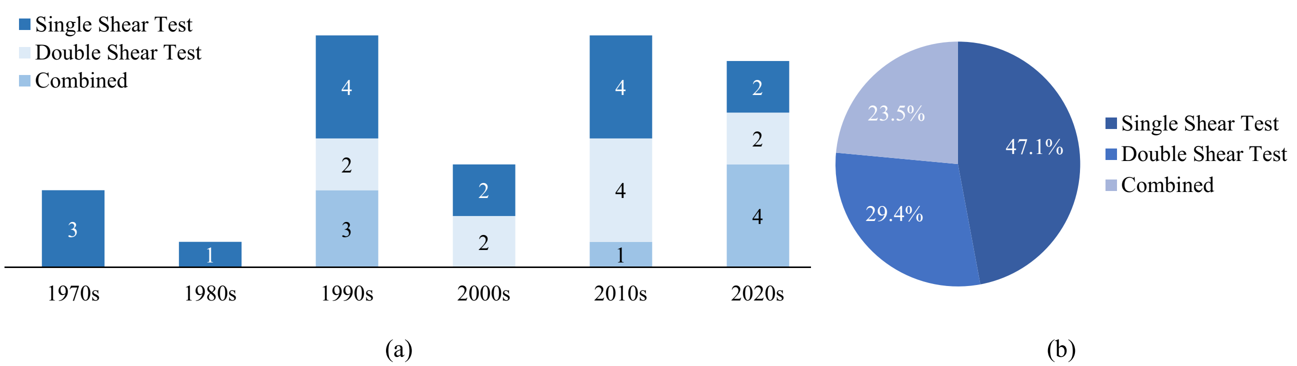


Fig. 13 **a** Classification of testing methods based on the type and year and **b** percentages of the different shear testing technologies

plane rotation according to the roughness distribution and the normal load applied to the joint, affecting the shear test results (Grasselli 2005). In this regard, as it is shown in Fig. 15c, 16% of the presented shear rigs were frictionless, created normally by adding a gap between the shear planes. Meanwhile, in 26% of cases, this effect was minimized by making the joint surfaces as smooth as possible or using Teflon sheets between the blocks. However, in more than one-half of the test designs, the role of the friction was not removed.

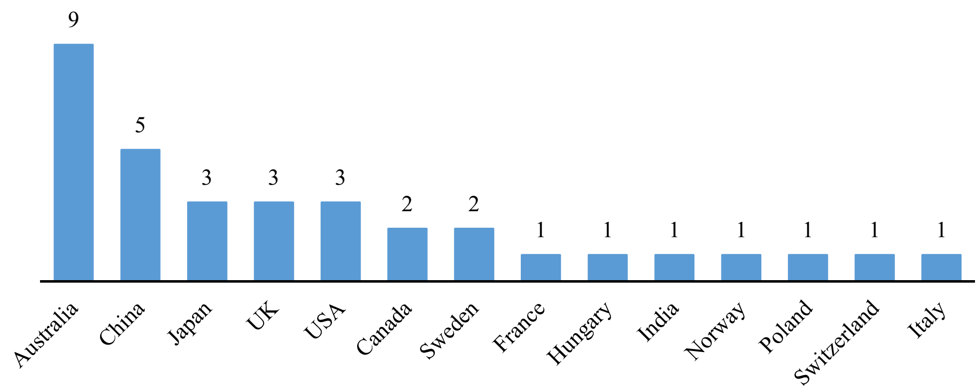


Fig. 14 The frequency of developed shear rigs in various countries

Table 5 Features of the developed test arrangements for static shear testing of tendons

Developer	Type	Tendon orientation to the joint surface	Host medium		Medium confinement	Friction	Tested tendon
			Type	Shape			
Dulacska (1972)	SST	Inclined	Concrete	Rectangular	–	Effective	Rock bolt
Bjurstrom (1974)	SST	Inclined	Rock	Rectangular	–	Effective	Rock bolt
Haas (1976)	SST	Inclined	Concrete	Rectangular	–	Effective	Cable bolt
Ludvig (1984)	SST	Inclined	Concrete/Rock	Rectangular	Steel box	Effective	Cable/Rock bolt
Spang and Egger (1990)	SST	Inclined	Concrete/Rock	Rectangular	Steel box	Effective	Rock bolt
Thompson and Windsor (1993)	C/T-S	Normal	Steel tube	Cylindrical	Steel tube	Effective	Cable bolt
Bawden et al. (1994)	T-S	Normal	Steel tube	Cylindrical	Steel tube	Effective	Cable bolt
Maruyama et al. (1994)	SST	Normal	Concrete	Rectangular	–	–	Rock bolt
Maruyama et al. (1994)	T-S	Normal	Concrete	Rectangular	–	–	Rock bolt
Haile et al. (1995)	DST	Normal	Concrete	Rectangular	–	Effective	Rock bolt
Ferrero (1995)	SST	Normal	Concrete/Rock	Rectangular	–	Minimized	Rock bolt
Goris et al. (1996)	SST	Normal	Concrete	Rectangular	Steel box	Minimized	Cable bolt
Ferrero et al. (1997)	DST	Normal	Concrete	Rectangular	–	Effective	Rock bolt
Aziz et al. (2003)	DST	Normal	Concrete	Rectangular	Steel box	Effective	Cable/rock bolt
Grasselli (2005)	DST	Inclined	Concrete	Rectangular	–	Minimized	Rock bolt
Mahony and Hagan (2006)	SST	Normal	Concrete	Rectangular	Steel plate	Effective	Rock bolt
BS 7861-2 (2009)	SST	Normal	Steel tube	Cylindrical	Steel tube	Effective	Cable/rock bolt
Craig and Aziz (2010)	DST	Normal	Concrete	Rectangular	Steel plate	Effective	Cable/rock bolt
MacKenzie and King (2014)	SST	Normal	Concrete	Cylindrical	Split steel tube	Minimized	Cable bolt
Chen (2014)	T-S	Normal	Concrete	Rectangular	–	Eliminated	Rock bolt
Srivastava and Singh (2015)	SST	Normal	Concrete	Rectangular	Steel box	Effective	Rock bolt
Maiolino and Pellet (2015)	SST	Inclined	Concrete	Rectangular	–	Minimized	Rock bolt
Mirzaghorbanali et al. (2017)	DST	Normal	Concrete	Rectangular	Steel plate	Eliminated	Cable/rock bolt
Liu et al. (2019)	DST	Normal	Concrete	Rectangular	Steel box	Effective	Rock bolt
Aziz et al. (2019)	DST	Normal	Concrete	Cylindrical	Split steel tube	Eliminated	Cable/rock bolt
Wu et al. (2019)	SST	Normal	Plaster	Rectangular	Steel box	Effective	Rock bolt
Cui et al. (2020)	SST	Inclined	Plaster	Rectangular	Steel box	Effective	Rock bolt
Pinazzi et al. (2020)	T-S	Normal	Steel tube	Cylindrical	Steel tube	Eliminated	Rock bolt
Pytlik (2020)	SST	Normal	Steel tube	Cylindrical	Steel tube	–	Rock bolt
Shan et al. (2022)	DST	Normal	Concrete	Rectangular	Steel box	Minimized	Cable bolt
Aziz et al. (2022)	DST	Inclined	Concrete	Rectangular	–	Effective	Cable bolt
Sun et al. (2022)	T-S	Normal	Rock	Rectangular	Steel plate	Minimized	Rock bolt
Knox and Hadjigeorgiou (2023)	T-S	Normal	Concrete	Rectangular	–	Eliminated	Rock bolt
Srisangeerthanan et al. (2023)	T-S	Normal	Concrete	Rectangular	–	Minimized	Cable bolt

3.1 Single Shear Testing (SST) Technologies

In Table 6, those SST rigs developed for shear testing of tendons only installed perpendicular to the shear plane are listed. As can be seen, the differences between the introduced test machines arise from many reasons. One of these reasons is the versatility or adaptability of test rigs. For example, researchers, such as Maruyama et al. (1994), Srivastava and Singh (2015), and Wu et al. (2019), proposed SST rigs which were only suitable for testing small anchor bolts.

On the other hand, the British Standard Single Shear Test (BSSST) apparatus proposed by BS 7861-2 (2009) was able to examine the shear performance of both cable bolts and rock bolts. Later, the Megabolt Integrated Single Shear Rig (MISSR) was initially reported by MacKenzie and King (2014), and later reported by Aziz et al. (2018b) in ACARP Project C24012. The MISSR allowed shear testing of larger capacity tendons up to 3600-mm long. Another reason is the roughness of the shear plane. In this regard, Mahony and Hagan (2006), Srivastava and Singh (2015), and Wu et al. (2019) did not eliminate the effect of joint roughness in their test configurations. While other researchers adopted various strategies to deal with joint friction. For example, Ferrero (1995) and Goris et al. (1996) used smooth shear planes to somehow reduce the influence of the friction. Ferrero (1995) also removed the normal force on the shear plane to minimize the effect of friction as much as possible. On the other side, although cable pre-tensioning was allowed in the MISSR, for tightening the shear planes, a Teflon sheet was employed between the concrete cylinder surfaces to avoid joint roughness.

Furthermore, it should be noted that the BSSST apparatus and the test device proposed by Pytlik (2020) were guillotine-style tools, leading to pure shear failure of the tendon, in which the steel-tendon contact could lead to immature failure of the tendon due to the stiffer contact between the steel and tendon. The pure shearing of a tendon is possible if the cable strand confinement is strong enough (Aziz et al. 2018a). Nonetheless, based on Aziz et al. (2014), when the tendon is sheared in a rock mass, in reality, failure would occur as a combination of tensile and shear failures. Because the in situ host medium strength is not quite high enough to cause pure shear failure. To overcome this drawback, in the MISSR, tendons were encapsulated in the 250-mm-diameter concrete cylinders, resembling the host medium condition more.

3.2 Double Shear Test (DST) Technologies

As mentioned, the DST methodology is an alternative method for shear testing of tendons. As it is shown in Fig. 16a, first generation of shear testing of tendons at two unique shear surfaces dates back to the early 1990s when Haile et al. (1995) produced a report on a small-sized DST device. However, no concrete test results were reported in this research. Next, Ferrero et al. (1997) proposed a laboratory equipment, consisting of three concrete blocks with dimensions of $60 \times 60 \times 100$ cm^2 (Fig. 16b). These blocks were reinforced using a lateral confinement system, comprising steel slabs and bolts. Besides, the rig was able to test single bar or double bars set at different distances. However, the main issue was that the influence of joint friction between the blocks was not removed, overestimating the bolt shear strength. Moreover, there was no external confinement in this set-up, leading to sample cracking during the tests. Later on, Aziz et al. (2003) introduced a small-scale double shear box called MK-I (Fig. 17a). It consisted of two outer cubes of 150-mm-side and 150-mm-side × 300-mm-long rectangular unit. The host medium was concrete with an axial central rifled hole to facilitate the axial installation of tendons grouted either by cementitious or resin grouts. The concrete blocks were externally wrapped or enclosed with a 20-mm-thick steel plate to confine concrete samples and prevent radial cracking of the concrete. The rate of shear displacement was set as 1 mm/mine, indicating static test condition. However, the application of MK-I was limited

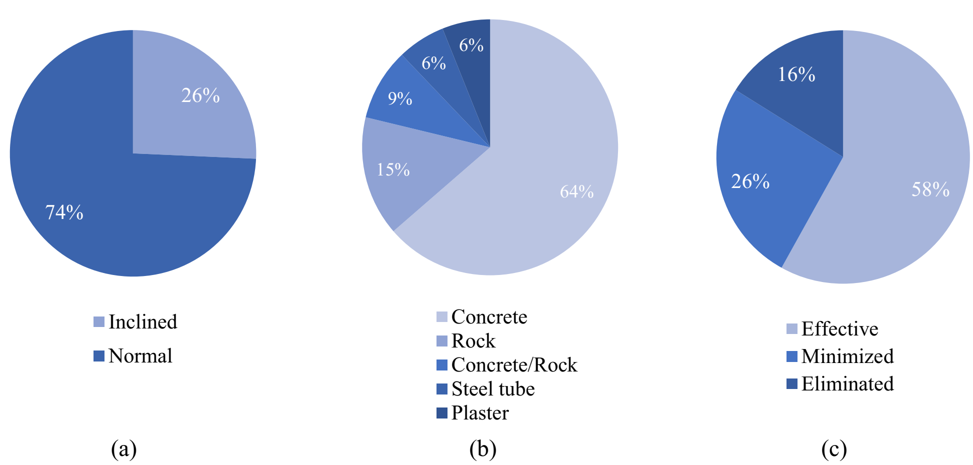
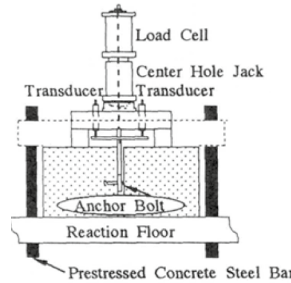
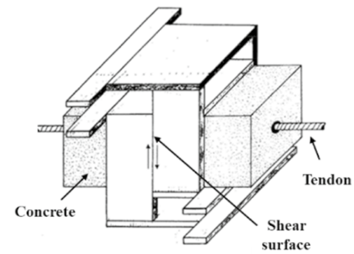


Fig. 15 Classification of shear rigs based on the **a** angle of inclination, **b** host medium type, and **c** method of addressing friction

Table 6 Single shear testing rig (SSTR)

<table>
<tr><th>Developer</th><th>Features and Comments</th><th>Apparatus Structure</th></tr>
<tr><td>Maruyama et al. (1994)</td><td>Anchor Bolt Shear Test Setup
• To apply shear force correctly, another concrete block was set to maintain the reaction point.
• Two concrete specimens were fixed firmly to the device floor.
• Concrete specimens broke due to the lack of medium confinement in some tests.
• The load was applied by a 20-t capacity centre hole oil jack, which was only suitable to test low-strength bolts.
• The test set-up is not strong enough to test high-strength bolts. So, it is only suitable for small-size anchor bolts.</td><td>
</td></tr>
<tr><td>Ferrero (1995)</td><td>Single Shear Box
• There was no gripper to restrict the bolt ends.
• To avoid vagueness due to the joint surface roughness, the surfaces were macroscopically smooth.
• The normal force on the shear plane was omitted. Therefore, the effect of joint friction was somehow minimized.
• Only steel bars were tested.</td><td>
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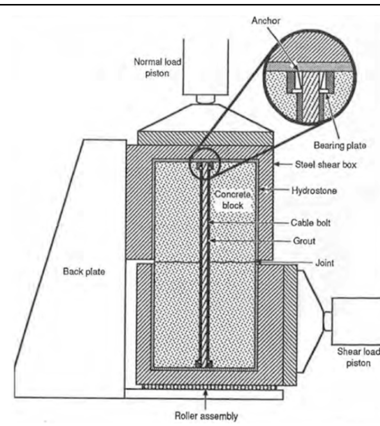
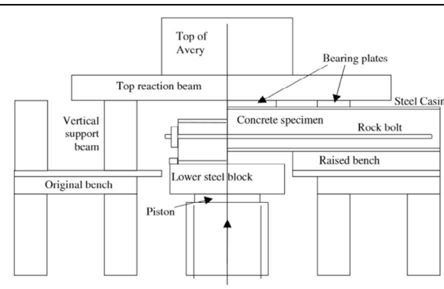
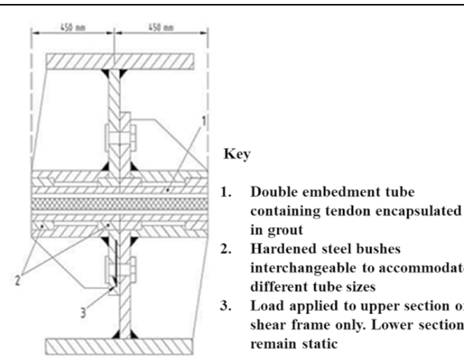
<tr><td>Goris et al. (1996)</td><td>Direct-Shear Rig
• It was able to apply 1334 kN load on both the shear and normal axes.
• There was full contact between the cable and the two concrete blocks through grouting.
• The inside dimension of the steel shear boxes was 30.48 cm^3.
• 0.025-m^3 concrete blocks were applied as the host medium.
• The 0.63 cm gap between the concrete block and the steel boxes was filled by the Hydrostone grout.
• The influence of friction was evaluated by shear testing on both rough and smooth joint surfaces.
• Cable pretensioning was not allowed in this test arrangement.
• Due to the limited space for the steel shear boxes, the ultimate shear movement was around 4.06 cm.
• There was an absence of any tightening mechanism.</td><td>
</td></tr>
<tr><td>Mahony and Hagan (2006)</td><td>Single Shear Test Facility
• Two concrete blocks were used as the host medium.
• The lengths of the concrete blocks were 250 mm and 1000 mm with cross-sectional dimensions of 280 mm × 280 mm.
• The samples were confined by using a 10-mm thick steel casing.
• Friction was effective, which could impact the test outcomes.
• The process of the test set-up was complex.
• Only rock bolts were tested.</td><td>
</td></tr>
<tr><td>BS 7861-2 (2009)</td><td>British Standard Single Shear Test (BSSST) Apparatus
• The bolt was grouted into two separated steel pipes.
• No end constraint structure and tendon was in the passive form. Thus, pre-tensioning could not be applied, underestimating the shear strength.
• The steel pipe lengths were limited.
• There was a steel-tendon contact.
• The equipment used was a guillotine-style tool, leading to pure shear failure, which may not represent the field condition.
• The steel confinement could not replicate the field condition.
• Both cables and rock bolts can be used.</td><td>
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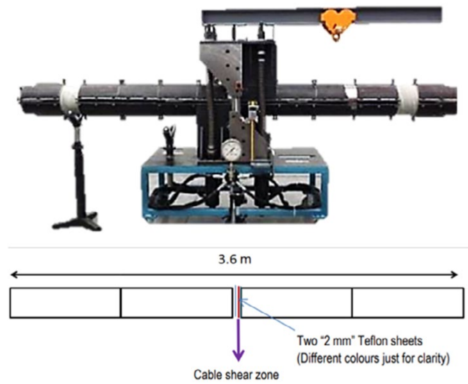
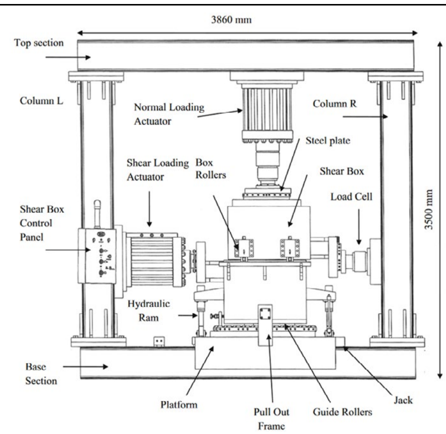
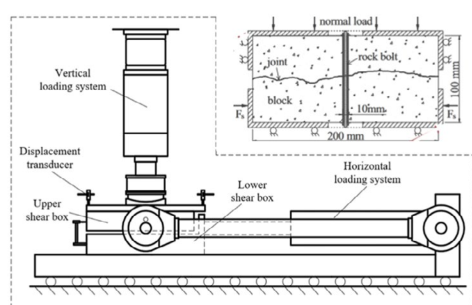
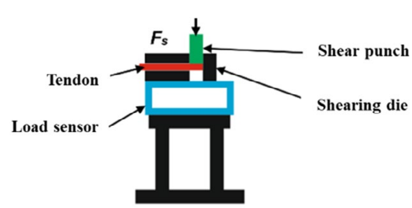
Table 6 (continued)

Developer	Features and Comments	Apparatus Structure
MacKenzie and King (2014) & Aziz et al. (2018b)	**Megabolt Integrated Single Shear Rig (MISSR)** • MISSR is a modified version of BS 7861-2 (2009). • The 250-mm diameter concrete cylinders with a length of 900 mm was employed to form the host medium. • In the first version, each shearing cylinder consisted of two 900-mm concrete cylinders glued together as reported by MacKenzie and King (2014) • In the second version, Aziz et al. (2018b) doubled the concrete length size to 3.6 m (1.8m per side) • It allows studying the rock-grout interface by encapsulating the cable in the concrete samples. • Steel clamps were used to provide external confinement around the concrete samples. • The joint friction was avoided via Teflon plates. • There was no steel tube-tendon contact. • Boreholes were rifled to simulate field conditions. • Cable pretensioning was allowed. • Debonding partially occurred during the testing of 900-mm long concrete per side but not in 1800-m long per side. • Total sample length was reached at 3600 mm, making it difficult to setup. • Sample preparation was quite intense and time-consuming, affecting further research to replicate the test condition. • Only cables were tested.	
Srivastava and Singh (2015)	**Servo-Controlled Large Direct Shear Test Machine** • 150×150×150-mm^3 concrete cubes were employed to form the host medium with dimensions of 750×750×900 mm^3. • The rock bolts were installed normal to the shear direction in the simulated boreholes, and in three different configurations. • The concrete blocks were piled inside the shear box and the simulated joint faces were aligned along the shearing plane. • The normal load was applied on a steel plate (750×750×100 mm), placed at the top of the specimen. • The shear displacement rate was 1.25 mm/min, which represents static test condition. • Since the dilation was extensively high, and there was a possibility of tilting the upper half of the shear box, damaging the rig, the tests were stopped just after the peak. • The effect of friction was not eliminated. • Boreholes were not rifled. • Only low-strength bars with 6 mm of diameter were used.	
Wu et al. (2019)	**Direct-Shear Test Machine** • Rock-like material made of plaster was used to simulate the jointed host medium. • The size of all the samples employed in the tests was 200×100×90 mm^3. • Shear loading was applied cyclic (five cycles). • The rate of shear displacement was 1 mm/min, representing the static test mode. • Shear boxes were used as the confinement. • The effect of friction was not eliminated. • Due to the size of the rig, only small samples were used. • As the size of the specimens was limited, only low-strength bolts could be tested. • The strength of the bolts was reduced to prevent affecting test results. Thus, the small diameter rebars (6 mm) made from iron were tested instead of steel rebars.	
Pytlik (2020)	**Bolt Rod Shearing Device** • It was developed based on a Polish standard (PN-G-15092:1999). • The 250 mm-long bolt rod sample was placed in a hole. • The cylinder-shaped punch in the die had a very small clearance, allowing the bending stresses in the sheared bolt to be minimised. • The ultimate rate of loading was up to 5 kN/s, which was very quick and did not represent static test mode. • It is a guillotine-style test facility and led to pure shear, which may not represent the real field condition. • Only rock bolts were tested.	

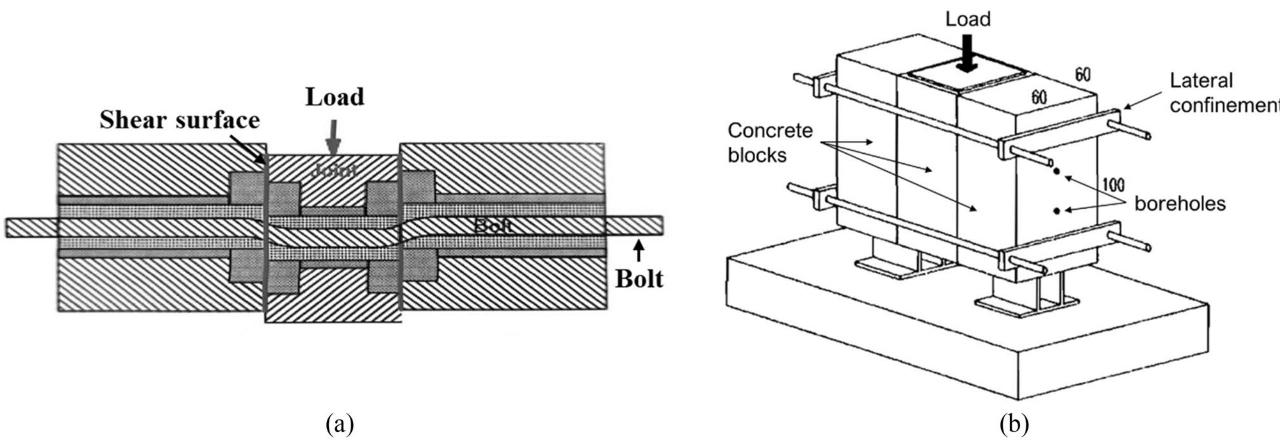


Fig. 16 The DST set-up, adapted from **a** Haile et al. (1995) and **b** Ferrero et al. (1997)

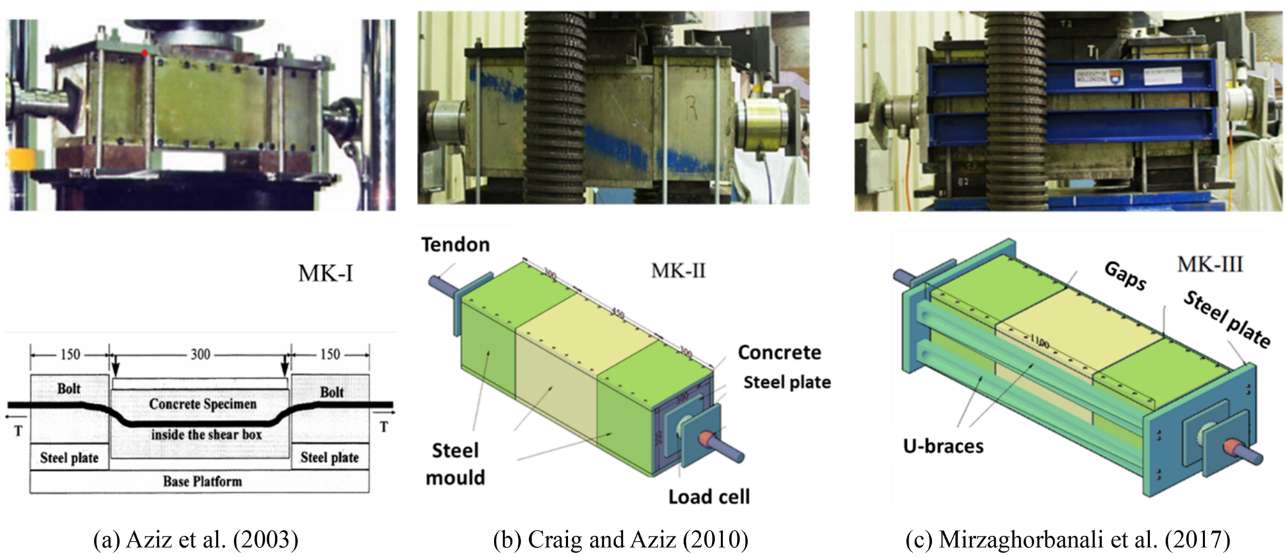


Fig. 17 Perpendicular DST experimental set-ups proposed in recent years

by its size. Currently, MK-I is utilized to assess rock bolts with a smaller diameter. To overcome the MK-I weak points, a 1050-mm-long large-scale rig, called MK-II, was subsequently reported by Craig and Aziz (2010), allowing for the testing of larger capacity tendons (Fig. 17b). It comprised two 300-mm-side outer blocks and a 450-mm-long central block with a $300 \times 300\text{-mm}^2$ cross-sectional area. Similar to MK-I, the concrete blocks are confined with 20-mm-thick steel plates. Although MK-II provides shear testing of large-capacity tendons, the problem of joint friction still exists.

To cope with MK-II issues, Mirzaghorbanali et al. (2017) presented a frictionless double shear experimental device called MK-III (Fig. 17c). To hold the side blocks in place the shear box was fitted with a Lateral Truss System (LTS), consisting of four 90-mm × 60-mm close channels fixed onto two 500-mm × 400-mm steel plates of 30-mm thickness outer side plates. As illustrated, there were gaps to prevent the friction between concrete blocks. As a result, the applied load is only used to shear the bolt not to overcome frictional force on joint surfaces. Despite this benefit, it was found that the steel plates could not provide a uniform confinement around the blocks, and both lateral and radial crackings of the specimens were inevitable during the experiments. However, because of the use of barrel and wedge on both sides of cables, the influence of debonding on shear performance could not be monitored and examined.

Further, a new cylindrical frictionless shear rig, also known as "MK-IV", was developed by Aziz et al. (2019) to prevent lateral and radial cracking of concrete samples due to the ineffective confinement system (Fig. 18a). As shown, the 300-mm-diameter steel circular clamps permit the application of external confinement to the cylindrical

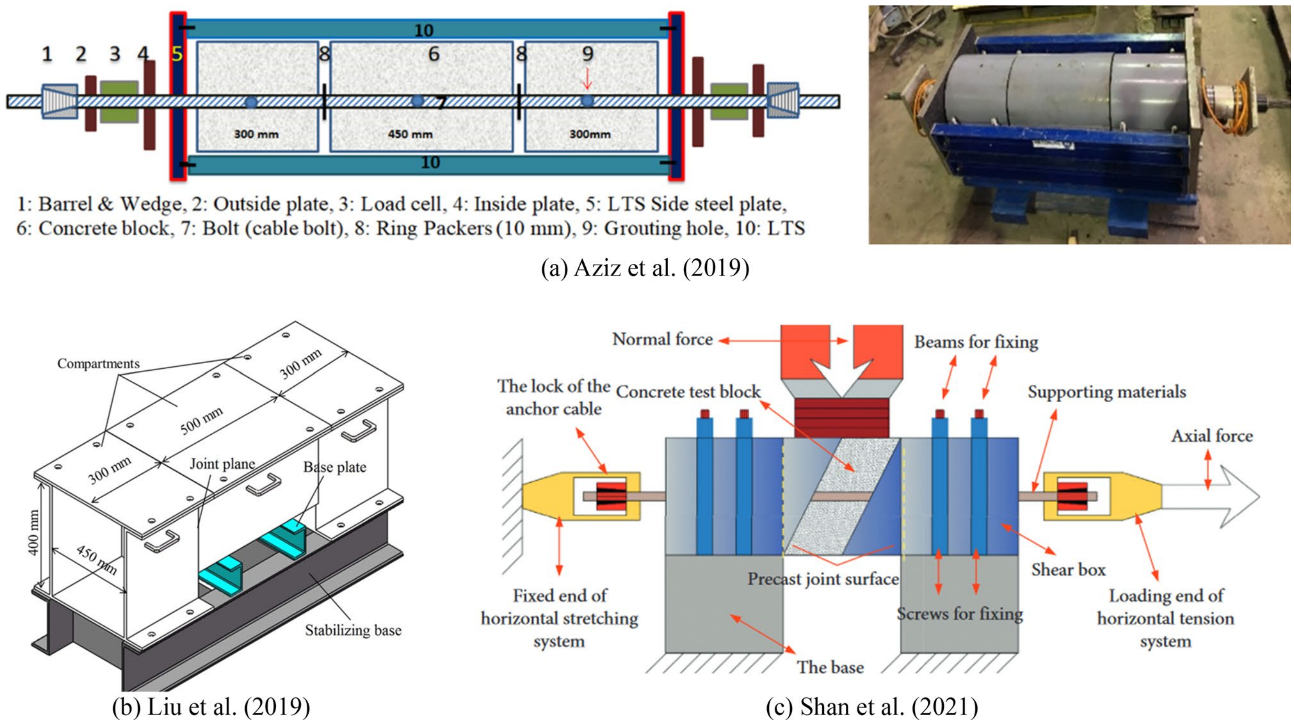
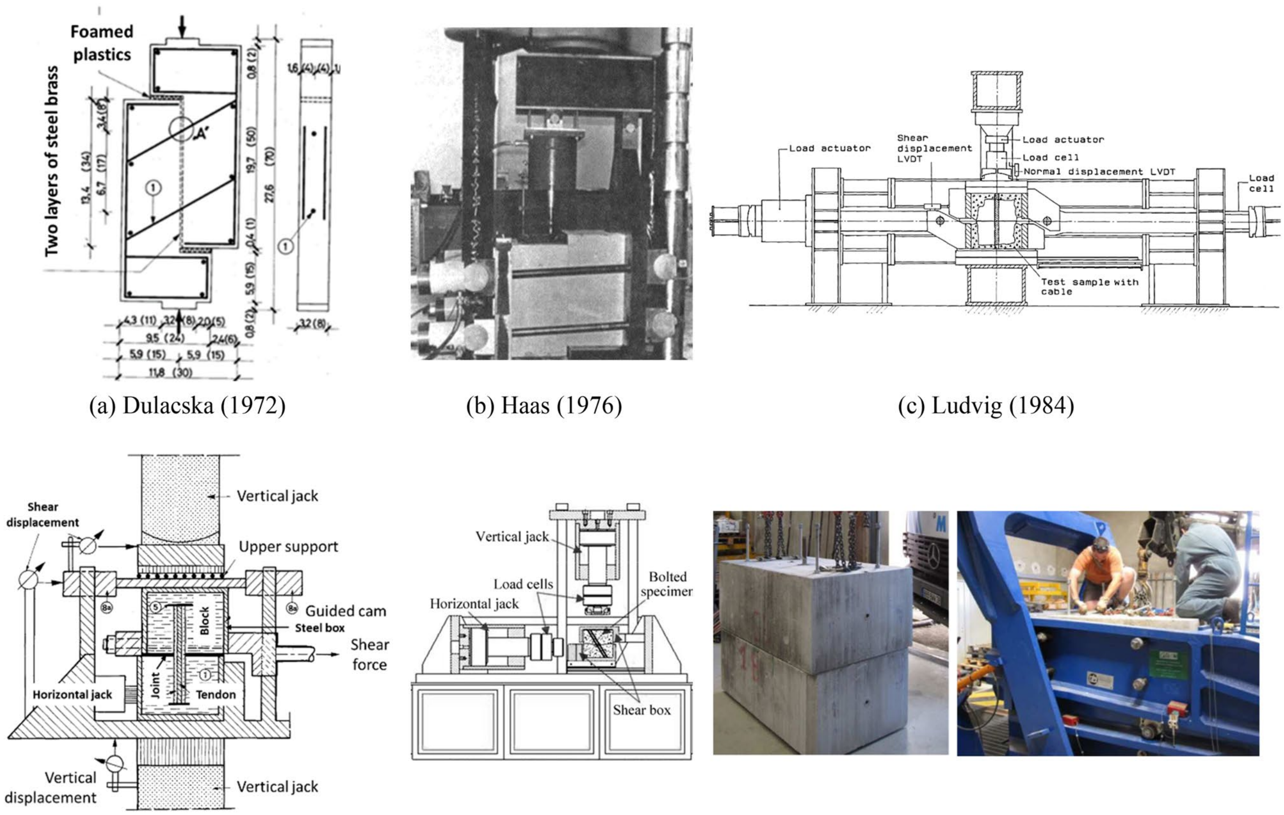


Fig. 18 Perpendicular DST experimental set-ups proposed in the recent years

concrete medium. In practice, it was found that the confinement in MK-IV was more effective compared to previously proposed shear boxes thanks to the circular cross-section of the concrete sample, leading to a uniform confining pressure. However, in testing larger capacity cables, the blocks were cracked because of the hairline gap existing between the steel clamps. Thus, in further tests, the concrete was internally confined using a 150-mm steel tube with 5 mm of thickness, which greatly increased its integrity during the tests. However, the effect of debonding still could not be assessed. Liu et al. (2019) proposed a large-scale double shear box similar to MK-II (Fig. 18b). The host medium was concrete blocks (side blocks of $450 \times 300 \times 400$ mm^3 and middle block of $450 \times 500 \times 400$ mm^3). Steel compartments were used to confine the samples externally, and the loading rate was 2 mm/min, which was slow enough. In this study, only rock bolts were tested and the friction between the block's surfaces was not prevented. In addition, Shan et al. (2021) used a double shear box for testing cables which was similar to the MK-II set-up (Fig. 18c). They maintained the same dimension for the side and central blocks ($300 \times 300 \times 300$ mm^3). In this design, a pre-tension load was applied by the tensile system, and the shear loading rate was 2 mm/min. However, the holes were not rifled, and the rectangular confinement provided by the steel boxes could not prevent axial and radial cracking of the concrete blocks during the tests. Besides, to minimize the effect of joint roughness, there was no normal force on the shear plane, meaning that the cable was not pre-tensioned, but was stretched using the horizontal stretching system.

3.3 Angle Shear Test (AST) Technologies

The shear rigs mentioned in the previous sections were only allowed to assess tendon shear performance at 0°, where the tendon is laid perpendicular to the direction of shear load. To be able to perform angle shear testing of tendons, some apparatuses were developed to examine the shear behavior of tendons at various angles of inclination. In this regard, the first attempts date back to the 1970s when Dulacska (1972) examined the shear performance of the steel bars using a single shear device (Fig. 19a), allowing to test the bolts under various orientations. In their study, four different angles of 10°, 20°, 30°, and 40° to horizontal direction were applied. Despite this advantage, the application of this test design was limited to smaller concrete specimens, lower shear loads, and lower strength bolts. Because the entire structure of the device was small. Another issue is that the influence of friction along the shear surface was not overlooked, which may lead to erroneous conclusions about the bolt shear strength. Moreover, it was not possible to apply



(a) Dulacska (1972) (b) Haas (1976) (c) Ludvig (1984)

(d) Spang and Egger (1990) (e) Cui et al. (2020) (f) Maiolino and Pellet (2015)

Fig. 19 Developed apparatuses allowing angle SST

pre-tensioning. Another pioneer in this domain was Bjurstrom (1974) who used granite rock samples to simulate a jointed rock mass reinforced by a rock bolt. In this research, the tests were conducted at various angles of inclination, and it was reported that the bolt failed in tension when the angle was less than 35°. However, the influence of the joint face roughness was not neglected. A few years later, Haas (1976) introduced a larger-scale SST configuration (Fig. 19b). They tested rock bolts at an inclined angle of 45° and 135° to the horizontal direction. In this research, 0.61-cm rock cubes (Shale and Limestone) were split to form a rock joint and considered as the host medium. They tried to eliminate the effect of friction along the simulated crack by making it initially smooth. The normal force on the shear plane started from 0.17 up to 1.72 MPa, which was not as high as in situ loads in the fields. Therefore, it was found that in low normal loads, the left-hand block was split due to the bolt deformation. The effect of medium confinement was omitted.

In the 1980s, Ludvig (1984) conducted SST using a large-scale experimental set-up, allowing to test of both cables and rock bolts (Fig. 19c) via 3000-kN capacity loading system. In this study, granite rock samples were utilized to resemble the host medium. To prepare the test sample, two halves of rock blocks with irregular shapes were cast into concrete. To create an artificial joint, two halves of blocks were held together before casting. Meanwhile, the contact between the upper and lower surrounding concrete was avoided to free the joint from concrete. In this apparatus, the bolts were installed at different inclinations to the joint surface (horizontal direction), including 90°, 60°, and 45°. Furthermore, the blocks were externally confined by using two 50-mm-thick steel boxes with dimensions of $450 \times 600 \times 300$ mm^3. However, in this device, the effect of the joint friction was not avoided, and pre-tensioning was also not allowed. Later on, Spang and Egger (1990) presented another SST machine (Fig. 19d) and tested rock bolts at 0°, 15°, 30°, and 45° angle of inclination to vertical direction. They employed both rock samples (sandstone and granite) and concrete blocks as the host medium. The upper and lower blocks of $150 \times 150 \times 130$ mm^3 and $220 \times 200 \times 150$ mm^3 formed a simulated shear plane. In addition, the effect of medium confinement was counted by casting samples directly into two steel boxes. To avoid the rotation of the upper and lower boxes, a guided cam and a horizontal jack were utilized. The main issue is that the size of the tested rock bolts was limited by the scale of the testing machine. Also, the influence of the joint friction was not removed.

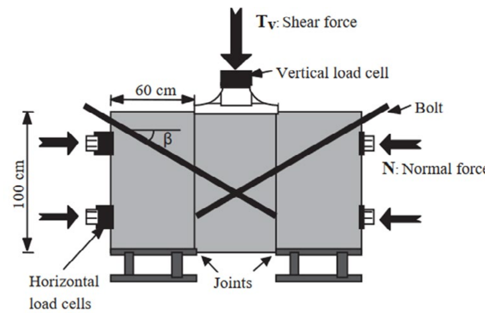
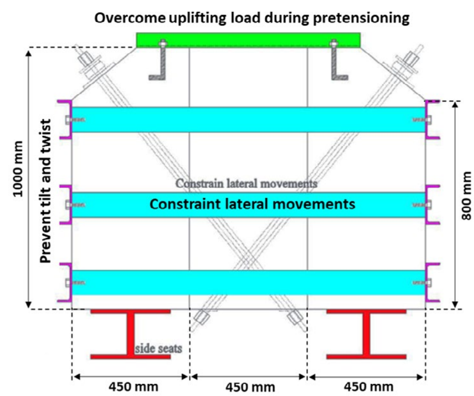
Maiolino and Pellet (2015), using a quite large-scale direct SST apparatus, carried out angle shear tests of steel rebars (Fig. 19f). To simulate field conditions, two concrete blocks with dimensions of $150 \times 100 \times 62.5$ cm^3 were used as the host medium. The upper and lower blocks formed a joint surface, which was made smooth to minimize the effect of the joint friction. The considered angles between the bolt and shear plane were 45°, 60°, and 90°. Additionally, the normal and shear forces were applied by 4-MN capacity hydraulic jacks, respectively. The first issue with this shear rig is that the rotation of the upper block was not restricted. Moreover, the external confinement of the concrete blocks was overlooked. A few years later, Cui et al. (2020) used an SST machine (Fig. 19e) for testing rock bolts at different angles of inclination, including 30°, 45°, 60°, 75°, and 90°. The experimental arrangement allowed testing to be carried out under both constant normal load and constant normal stiffness. In this research, the blocks made of soft Plaster were used to simulate the host medium with a shear surface. The matched joint specimen was placed in two split shear boxes, each of a dimension $100 \times 100 \times 47.5$ mm^3. The shear displacement was set as 0.005 mm/s, representing the static load condition. The issue around this test arrangement is that the maximum loading capacity of the system was limited to 300 kN (around 30 t), which is only suitable for low-strength tendons. In addition, the Uniaxial Compressive Strength (UCS) of the host medium was about 14 MPa, only representing the soft rocks condition. In addition to the above-mentioned test devices, during the last years, two DST apparatuses have also been presented for angle shear testing which are compared in Table 7. However, it is worth mentioning that the major issue with these rigs is casting the large-scale concrete blocks, which can be notably time-demanding and intensive.

3.4 Tension/Compression-Shear Test Technologies

In the early 1990s, a test facility was developed by Thompson and Windsor (1993) that enabled the researchers to carry out tests under a combined mode, as shown in Fig. 20a. In this testing configuration, the free dilation of the shearing plane was prohibited, and also the direction of movement was permanently parallel to the shear surface (separation plane), which is similar to the sliding of the rock block due to the gravity illustrated in Fig. 1. Moreover, by changing the orientation of the cable bolt encapsulated into two steel tubes, the test mode could be varied, including pure shear (90°), compression–shear (45°), and tension–shear (135°). However, in pure shear mode, there was a possibility of steel–cable contact, leading to immature cable failure. Next, Bawden et al. (1994) proposed a test design which allowed to test tendons in the combined axial and shear loading mode (Fig. 20b). This method tried to simulate a cable installed perpendicular to a layered hanging wall. Therefore, in this set-up, the cable bolt was always oriented perpendicular to the shear surface. The joint was artificially created using two 100-mm-thick steel plates machined in the form of 120°-degree arcs with matching canters. The cable was encapsulated into two 75-mm-diameter steel tubes. One of these tubes simulates a reinforced rock block moving downwards into the excavation. Thus, it allowed both joint dilation and shear at numerous installation angles. Meanwhile, the angle could be varied by rotating the steel plates. However, this test machine could not achieve the ultimate capacity of the cable bolts in most of the tests. Maruyama et al. (1994) also used a test machine for tension–shear testing (Fig. 20c). They encapsulated 16-mm anchor bolts in the concrete blocks and pulled them out at various angles to simulate tension–shear loading mode. However, due to the test machine scale and the limited capacity of the loading system, only low-strength bolts were tested.

Around 2 decades later, an NTNU/SINTEF bolt test rig was introduced by Chen (2014) for conducting pull-and-shear loading tests. In this new rig (Fig. 20d), the axial and shear loads were applied by two separate hydraulic cylinder systems. Using these cylinders, the orientation of the rock bolt to the shear plane could be varied from 0° (pure tension) to 90° (pure shear). Two concrete blocks with dimensions of $950 \times 950 \times 950$ mm^3, placed in a frame, were used as the host medium. In this study, rock bolts were installed in the holes pneumatically drilled. To guide the concrete blocks, roller bearings were installed between the blocks, which also eliminated the friction between them. Thus, it was allowed to perform tests with different joint gap openings. Furthermore, the frame and the roller bearings prevented the concrete rotation during the tests. In this test set-up, the sample preparation process was time-consuming and intensive. Pinazzi et al. (2020) employed the idea of the British standard SST apparatus and developed a new laboratory test rig to examine the performance of rock bolts under combined load (Fig. 20e). This design allowed them to evaluate the influence of the axial movement on shear load capacity and vice versa. Moreover, the rock bolt was inserted into the steel rig, and the removable steel pipes inside the rig were adjusted to the spacing required for the tests. Thus, a gap was considered to remove the frictional resistance of the shear plane. In addition, to remove the bolt movement during the tests, the bolt was tightened at both ends using nuts and plates. In this study, the effect of combined loads was only assessed using ungrouted rock bolts, while the role of this loading mode on the rock bolt, grout, and the host medium should be studied. Furthermore, the tests executed on samples whose dimensions were restricted to laboratory size, could not represent the field condition.

Table 7 The previously proposed angle double shear rigs

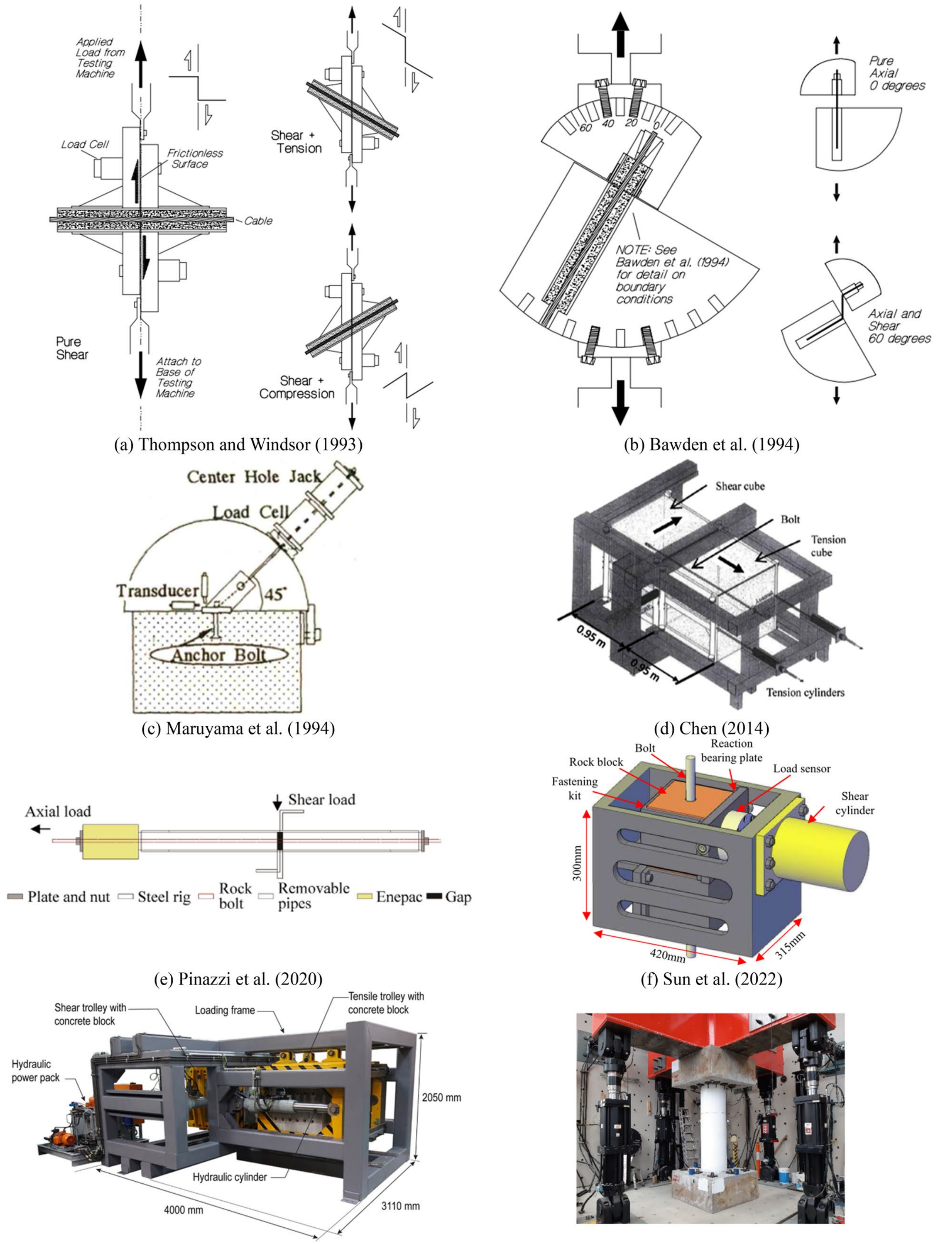
Grasselli (2005)	**Large-scale Angle Shear Rig** • DST at inclined angles of 0°, 15°, 30°, and 45°. • Three concrete blocks with dimensions of 1000×600×600 mm^3 were used as the host medium to form two joints. • The joints were macroscopically smooth to prevent further vagueness associated with joint roughness. • Symmetric set-up essentially prevented the rotation of concrete blocks. • The preparation of the blocks was challenging due to their large size. • There was no host medium confinement. • Due to the potential of concrete failure, only low-strength rock bolts could be tested.	
Aziz et al. (2022)	**Angle Double Shear Box** • DST at inclined angles of 30° and 45°. • Three large-scale concrete samples were used as the host medium. • The LTS, including structural steels, were used to constrain lateral movements, tilts, twists. • The boreholes were rifled to simulate field conditions. • The displacement rate was 1 mm/min, indicating static test condition. • No external confinement was employed. • Pretensioning was difficult to evaluate 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. • Casting concrete blocks was challenging. • The effect of friction along the simulated joints was not overlooked. • Only the shear behaviour of cables was examined.	

Another pull-shear rig was presented by Sun et al. (2022), as shown in Fig. 20f. This rig mainly consisted of tension and shearing cylinders, which were arranged vertically and horizontally, respectively. The upper and lower granite rock blocks with dimensions of $150 \times 150 \times 150$ mm^3 were used as the host medium of the 18-mm-diameter rock bolt. The movement of the lower block was limited during the shear process. It tried to minimize the influence of the joint roughness by smearing the block surfaces with butter. However, the friction was still effective. The application of the test set-up is limited to the rock bolts due to their small size. Knox and Hadjigeorgiou (2023) introduced a modified version of the NTNU/SINTEF bolt test rig called the Epiroc Combination Shear and Tensile (CST) Rockbolt Pull Tester shown in Fig. 20g. In comparison with the SINTEF configuration, the new rig employed a modified hydraulic control system which can accommodate a longer sample length and had a greater shear capacity. In this regard, shear and tensile displacements increased to 300 mm and 500 mm, respectively. Meanwhile, the rig was only used to test rock bolts. Next, a large-scale apparatus (Fig. 20h), called the multi-axis substructure testing (MAST) facility, was developed by Srisangeerthanan et al. (2023) for testing cable bolts under combined loading conditions. This rig consisted of two embedment sections made of concrete cylinders, including a top section (forming the unstable rock strata) and a bottom section (the stable rock strata that will impose movement). To minimize frictional resistance along the shear plane, a Teflon sheet was used at the joint interface. The top and bottom embedment were cast at a diameter of 600 mm, and in lengths of 1000 mm and 1600 mm, respectively. Nevertheless, the embedment length appeared inadequate to avoid the radial blow-out type cracking of the concrete medium, indicating the importance of the host medium strength.

4 Discussion

From the literature review of both axial and shear testing technologies reported in this paper, it is obvious that each method used has inevitably some varied effects on the test outcome and results. Recognizing the fact that tendons alone constitute only one element of the support system integrity and, that there are several factors and parameters, that have a significant influence on tendon installation and performance and deserve further elaboration.

Type of the host medium: The host medium has a great influence on the pull-out and shear load levels. Almost 20% or one-fifth of the proposed pull-out rigs and 6% of the shear



(a) Thompson and Windsor (1993) (b) Bawden et al. (1994)

(c) Maruyama et al. (1994) (d) Chen (2014)

(e) Pinazzi et al. (2020) (f) Sun et al. (2022)

(g) Knox and Hadjigeorgiou (2023) (h) Srisangeerthanan et al. (2023)

Fig. 20 The facilities for combined tests rigs use small-diameter steel pipes as the host medium. Although steel pipes provide a stiff medium, they might overestimate the strength of the host medium, leading to higher pull or shear load values as compared to those under real conditions. To overcome this issue, in most cases, composite mediums of varied strength, like concrete samples, should be employed as a representative of the real rock mass condition. It is worth mentioning that real rock samples can and have been used for this aim, even though the preparation of such samples with the required size for testing purposes may pose difficulties.

Medium confinement: It was found that the integrity of the host medium is highly controlled by the external confinement, simulating the in situ condition. As the results showed, almost 32% of the pull-out rigs with their host mediums were not confined, and experienced axial and lateral cracking during the loading stage, adversely affecting test outcomes. Once the tested specimen is cracked, the resistance against the pull-out and shear loads is diminished. In contrast, some researchers used two different strategies for reinforcing the host medium, including casting the concrete medium in steel tubes or the use of CRP (via the MHC or biaxial cell) and CRS (via rigid steel tubes, split steel pipes, or steel plates). It was reported that the application of these kinds of confinement considerably enhanced the host medium integrity.

The shape of the host medium: In addition to the type of host medium and the external confinement, it was found that the cross-sectional shape of the medium is also important, affecting its integrity during the test. Results indicate that for both axial and shear testing of tendons, those test arrangements employing cylindrical samples lead to notably less cracking after the test compared with rectangular ones. It is much easier to apply uniform lateral confinement on cylindrical samples than rectangles, especially when specimens are directly cast inside the steel tubes. However, even in the cylindrical samples, where split steel pipes (semi-circular steel clamps) were utilized for confinement purposes, the hairline gaps existing between the steel halves caused radial cracks in the tested samples. Therefore, it is clear that the uniformity of the lateral confinement is very essential for preventing cracks.

Frictional resistance at joint plane: It was shown that, in shear testing, around one-third of the applied shear load was consumed to overcome joint roughness resistance. Thus, it might overestimate the obtained shear strength of the tendon. To avoid shear plane friction, some researchers, for example, Mirzaghorbanali et al. (2017) created a gap between the sheared planes to eliminate its effect in their test design. Alternatively, in some studies, attempts were made to minimize the effect of roughness using Teflon sheets or by making the shear surfaces as smooth as possible, though it was still effective.

Test scale: It is another important issue in developing a test device. The scale of the testing facility should replicate the field conditions. In some of the developed rigs (whether in axial or in shear testing), their applications were limited to low-strength tendons due to their small-scale structure. In contrast, it was found that there are large-scale apparatuses in which their sample preparation was quite intense and time-demanding, affecting further research to replicate the test condition. Therefore, to attain reasonable results, it is crucial to consider the scale of the samples in a way which resembles the field condition properly. For example, Stillborg (1984) employed a larger-scale rig to replicate the field condition more.

Loading (displacement) rate: To prepare static test conditions, it is vital to consider an appropriate loading rate or displacement rate by controlling the hydraulic flow rate in the laboratory-based studies. Indeed, in comparison with the static loading in an underground space, the minimum rate of loading in the laboratory is in general much greater. However, it can be classified as a quasi-static loading mode if no kinetic load is generated during the loading process. According to the results, unfortunately, many of the reviewed papers did not refer to the applied loading rate in their studies. On the other hand, in some studies, an acceptable rate of loading (e.g., 1–3 mm/s) was set to simulate the static loading condition, which is also significantly greater than ground movement under normal ground movement conditions. One must also consider the test environment based on ground Seismicity. Meanwhile, many of the researchers used a high loading rate like 10 kN/s which could not represent the static test mode. This high rate of loading depicts dynamic conditions.

Shear testing: In general, shear testing is relatively difficult to undertake in the laboratory. It has yet to be ventured into field study, because of the extreme challenge faced in securing more resources and costs involved to undertake this sort of test with varied results depending on the methodology of shear testing adopted. Also, the method is relatively recent with the general lack of trained expertise on the topic and other than what is available from a limited laboratory study.

Cable rotation: When it comes to pull-out testing of cables, the rotation phenomenon should be considered. It was found that the main shortcoming of the SEPT method was the free length of the cable, leading to rotation. Cable rotation cannot happen in the real field condition, and if it happens in the laboratory, the obtained bond strength is lower than the field condition. In the DEPT rigs, some researchers utilized the anti-rotation parts bolted to the two embedment sections, for instance, Aziz et al. (2016). Besides, another method that is widely used to address this issue is using an anchor tube in the LSEPT (Clifford et al. 2001). However, it was found that these strategies might not be able to restrict cable unscrewing solely. Therefore, further research is required in this area.

5 Conclusions

This paper aims to provide a good understanding of the currently developed technologies for axial and shear testing of tendons in static mode. Based on the reviewed literature, both pull-and-shear tests bear importance for the correct evaluation of the strength and applicability of any tendon type and use for their application for effective ground reinforcement and stabilization.

In the axial tensile loading test, the pull-out method is a preferred method of testing tendons for realistically replicating the field conditions and the purpose of its application for ground reinforcement. It is also an easier method. The selection of any testing method of pull testing should reflect on the tested tendon type. Pull testing of cables should be carried out using the DEPT method to prevent unwinding. The method of tendon push testing is not a valid method of tendon strength characterization as it defies the purpose of bolting philosophy.

Various factors and parameters, including host medium strength, medium shape, and size, medium confinement (internal and external), borehole thickness, encapsulating grout type, rate of pull, and tested tendon type, have a bearing on the test characterization and results.

Consistency of the test results, from the laboratory or the field, will depend on the consistency of the test method used. One paramount importance is reliance on various parameters and factors associated with bolt installation and subsequent loading. These factors include host medium characteristics, medium shape and confinement, testing method, and tendon installation angle.

The above-listed methods are equally relevant to shear testing methods; furthermore, the host medium shape, size, joint surface roughness, and confinement are of particular relevance in enabling shear testing of tendons to be undertaken to yield an acceptable result for effective design leading to the construction of permanent underground structures and tunneling. In particular, shear testing in cylindrically shaped and confined host medium was found to contribute to the minimizing of host medium crack formation, axially and laterally for consistent test results.

Using the Megabolt integrated single shear rig reported by Aziz et al. (2018b) yielded consistent shear test results. The method also prevents debonding with an increased length of encapsulation.

The availability and knowledge accumulation of different techniques gathered in this paper should provide the opportunity of advancing research going forward to different levels and including in situ studies in underground operation, and for enhancing the strength characteristics of the reinforced ground with tendon technology. It is worth mentioning that as the depth of mining and tunneling increases, the ground stabilization may face further complexities and challenges, including stress fracturing, high in situ stress, squeezing and creeping rocks, dilation over time, and rock burst (dynamic loading). To employ rock reinforcement systems more effectively and to provide a safer working environment, such important complexities need to be considered. For instance, tendons are also susceptible to sustaining dynamic loads, which can jeopardize the stability and safety of the underground space. The performance of tendons under dynamic loading is different with static and quasi-static loading modes. In this regard, as it was reported by Hadjigeorgiou and Potvin (2011) and Mottahedi et al. (2024), various testing methods (e.g., drop weight and momentum transfer) have been developed for simulating seismic events like rockburst conditions to assess the dynamic capacity of tendons. Hence, the strength evaluation topic could also be extended into dynamic testing to suit seismically active ground conditions.

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Declarations

Conflict of interest The authors declare that they have no conflict of interest; the research does not involve human and/or animal participation; and all participants have given their informed consent.

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