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# Engineering characterization of intermediate geomaterials – A review

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**Abstract.** Intermediate Geomaterials (IGMs) are natural formation materials that exhibit the engineering behavior (strength and compressibility) between soils and rocks. The engineering behavior of such material is highly unpredictable as the IGMs are stiffer than soils and weaker/softer than rocks. Further, the characterization of such material needs exposure to both soil and rock mechanics. In most conventional designs of geotechnical structures, the engineering properties of the IGMs are either aligned with soils or rocks, and this assumption may end up either in an over-conservative design or under-conservative design. Hence, many researchers have attempted to evaluate its actual engineering properties through laboratory tests. However, the test results are partially reliable due to the poor core recovery of IGMs and the possible sample disturbance. Subsequently, in-situ tests have been used in recent years to evaluate the engineering properties of IGMs. However, the respective in-situ test finds its limitations while exploring IGMs with different geological formations at deeper depths with the constraints of sampling. Standard Penetration Test (SPT) is the strength-based index test that is often used to explore IGMs. Moreover, it was also observed that the coefficient of variation of the design parameters (which represents the uncertainties in the design parameters) of IGMs is relatively high, and also the studies on the probabilistic characterization of IGMs are limited compared with soils and rocks. With this perspective, the present article reviews the laboratory and in-situ tests used to characterize the IGMs and explores the shear strength variation based on their geological origin.

**Keywords:** heavily overconsolidated clay; indurated soil; probabilistic characterization; residual soil; soft rock

## 1. Introduction

A detailed characterization of geomaterials is essential for the safe and economical design of any geotechnical engineering structure. Clayton *et al.* (2014) evaluated different causes for the failure of engineering structures and reported that almost 20% of failure occurs due to improper characterization of geomaterials. The most-reported natural geomaterials in geotechnical practice are soil (fine-grained and coarse-grained soils) and rock. The coarse-grained soils are characterized based on the state variables (void ratio, confining pressure, stress history (relatively less significant compared with fine-grained soils)) and compositional variables (gradation, mean grain size, grain shape, grain mineralogy, and other constituents). Similarly, for fine-grained soils, the state variables also account for water content and compositional variables (including clay mineralogy, clay size fraction, specific surface area, cation exchange capacity, and other constituents). Rock is a solid mass of geological material. It could be characterized using Rock Mass Rating (RMR), Rock Quality Designation (RQD), Q-System, unconfined compressive strength, recovery ratio, presence of discontinuity, no of joints, degree of weathering, and other tests (Goodman 1989, Gannon *et al.* 1999). Carraro and Salgado (2004) listed other soil classification groups that are not commonly used in design practice and are named non-textbook soils. O'Neil and Reese (1999) and Brown *et al.* (2010) categorized the natural geomaterials into four categories for the design of drilled shafts: a) cohesive soil, b) cohesionless soil, c) rock, and d) intermediate geomaterials.

The term Intermediate Geomaterial (IGM) was first used in the design of drilled shaft foundations by Thompson and Brown (1994). O'Neil *et al.* (1996) developed a definition for Intermediate Geomaterials (IGMs) for designing deep foundations. They characterized the IGMs using unconfined compressive strength and SPT-N index value, presented in the following sections. According to O'Neil *et al.* (1996), intermediate geomaterial is a continuum material with engineering behavior between soils and rocks. It covers residual soils, completely weathered rock, glacial till, and very dense granular material. The subsequent works of O'Neil and Reese (1999) also adopted a similar definition of IGMs for the works of drilled shaft foundations. IGMs are also named soft rocks, formation materials, hard soils, indurated soils, and weak rocks (Mokwa and Brooks 2008, 2009). Therefore, the studies on the characterization of soft/weak rocks are also addressed for the present review.

It is common to encounter IGMs at both shallow and deeper depths. Lee *et al.* (2016) reported that almost 80% of the land area in Korea is covered with weathered rock (weak rock or decomposed rock), adding that 52% of the cast-in-situ piles in the country are founded on weathered

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rocks. Mokwa and Brooks (2008) also reported the encounter of cohesive and cohesionless IGMs in Montana, USA. Several researchers attempted to characterize and study the behavior of IGMs, for the design of deep foundations resting on them (O'Neil *et al.* 1996, O'Neil and Reese 1999, Mokwa and Brooks 2008, Gupta 2012, Stark *et al.* 2013, Long 2016, Lee *et al.* 2016, Adhikari *et al.* 2020, Zhao *et al.* 2020). Since the ultimate capacity of the foundation system depends on the actual shear parameters of IGMs, its identification and characterization are essential for safe and economical design. Moreover, the material characterization should be addressed, irrespective of whether the deterministic values are obtained using direct measurements through laboratory/field tests or indirect measurements using bootstrapping (Loehr *et al.* 2017).

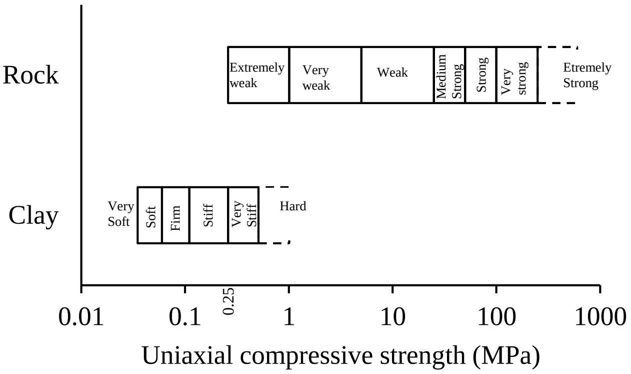

Fig. 1 The boundary between clay and rock based on uniaxial compressive strength (ISRM 1978)

## 2. The boundary between soils and rocks

Since the strength of IGMs exists between soils (cohesive and cohesionless) and rocks, defining the lower and upper bound shear strength limits of soils and rocks is essential. Peck *et al.* (1974) stated that no sharp definition exists to differentiate between soils and rocks because an intact rock may lose its strength during the weathering action. Similarly, indurated soils shall exhibit higher strength comparable with rock mass. Later, Morgenstern and Eigenbrod (1974) studied the behavior of muddy sediments and reported that a strength reduction by more than 60% upon inundation might behave as soil rather than rock. Further, they hypothesized that the mudstone with a UCS value less than 0.6 MPa may exhibit soil-like behavior, and this distinction was also supported by Grainger (1984). Wilson (1976) characterized the mudrock as a weak rock and reported that the engineering behavior of mudrock is like heavily overconsolidated clay.

Further studies by Cripps and Taylor (1981) on mudrocks stated that the distinction between soil-like and rock-like behavior could be inferred based on cementation and bonding knowledge. Dobereiner and De Freitas (1986) attempted to differentiate sand and weak sandstone through microstructural investigations. They used packing density (PD) and grain contact (GC) as quantitative tools to identify such differences. It was reported that the grain contact was 27% to 58% for rock, and packing density ranged between 67% and 82%. The grain contact for sand was observed as 4.5% at a packing density of 62%.

ISRM (1978) defined a boundary between saturated clays and rocks using unconfined compressive strength, presented in Fig. 1. As seen in Fig. 1; there is an overlap in the distinction because the UCS of clayey soil lies between 0.025 MPa and 0.5 MPa, whereas it varies between 0.25 MPa and 250 MPa for rocks. A similar observation was reported in IS 11315 -Part 5 (1987). Brown *et al.* (2010) also bounded the upper limit UCS value of cohesive soil to 0.5 MPa. Terzaghi *et al.* (1996) reported that many natural aggregates behave as neither soils nor rocks, hypothesizing that the geomaterial with compressive strength of 0.4 MPa and above might behave like a rock rather than the typical clayey soils. BS 5930 (2015) bounded the cohesion value of clayey soil to 0.6 MPa, and natural geomaterials with a cohesion value of more than 0.6 MPa require knowledge of rock mechanics for characterization.

## 3. Formation of intermediate geomaterials

The physical (unloading, thermal expansion and contraction, crystal growth, colloidal bulking, and organic activity) and chemical (hydrolysis, oxidation, carbonation, chelation, and cation exchange) weathering of rocks lead to the formation of cohesionless and cohesive soils, respectively (Mitchell and Soga 2005). However, the Intermediate Geomaterials (IGMs) are expected to form during the deterioration of rocks due to weathering action or strengthening process of the soil deposits, which is a function of the geological origin.

Dobereiner and De Freitas (1986) schematically presented the weak rock formations and emphasized that the characterization of weak rock is a function of soil and rock mechanics. They also stated that a weak sandstone might result from deposits of sandy sediments, where the mutation process of soil to complete rock formation has yet to be completed. Similarly, hard rock may deteriorate and lose its strength upon weathering action. In such a case, the weak sandstone's behavior could be categorized as soil-like behavior of sand or rock-like behavior of sandstones. In general, for a complete transformation from soil sediments to sedimentary rock, it must undergo lithification (cementation, recrystallization & compaction) processes, and incompletion of such processes may result in weak/soft rock formations (Dobereiner and De Freitas 1986, Akai 1993, Mokwa and Brooks 2008). The processes of lithification are also called diagenesis by geologists (Waltham 2002). Further, the incompletion of diagenesis processes may lead to loose texture formation, cavities, and poor cementation (Mokwa and Brooks 2008).

## 4. Characterization of intermediate geomaterial

Section 2.0 discussed the strength boundary limit for cohesive soils and rocks, and a similar approach is used for

Table 1 Characterization of IGMs based on Unconfined compressive strength

| **S.No** | **Type of IGMs** | **Unconfined Compressive Strength (MPa)** | **Reference** |
|---|---|---|---|
| 1. | Mudrocks (Non -durable) | 0.6 – 3.6 | Grainger (1984) |
| 2. | Weak Sand Stone | 0.5 - 20 | Dobereiner and De Freitas (1986) |
| 3. | All | 1 - 10 | Akai (1993) |
| 4. | Arenaceous | 1 - 25 | De Freitas (1993) |
| 5. | Cohesive IGMs | 0.5 - 5 | O’Neil *et al* (1996);<br>O’Neil and Reese (1999);<br>Brown *et al.* (2010);<br>IRC 78 (2014) |
| 6. | All | 0.6 – 12.5 | Gannon *et al.* (1999) |
| 7. | Cohesive IGMs | 0.5 - 25 | Mokwa and Brooks (2008) |

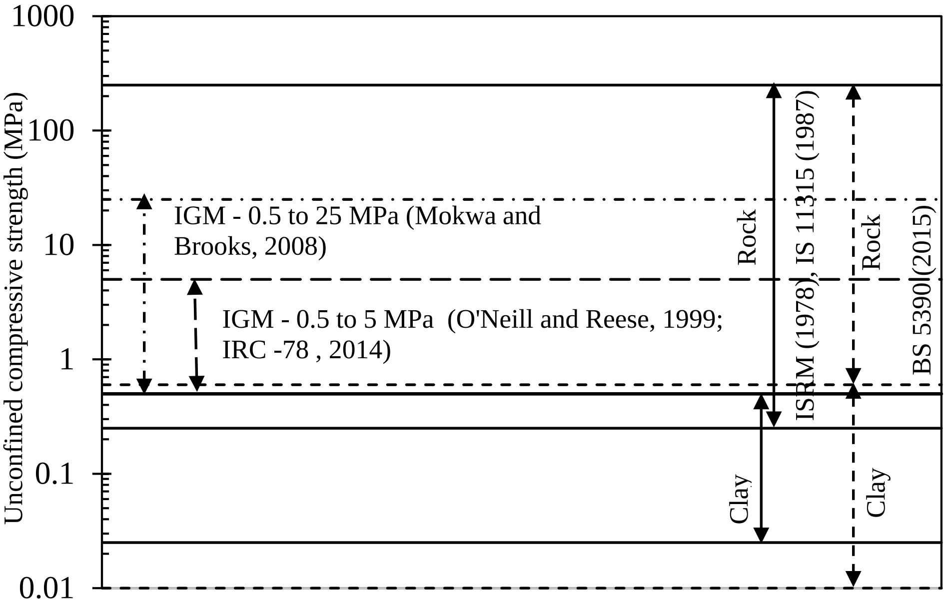

Fig. 2 Variation of unconfined compressive strength of cohesive soil, IGMs, and Rock

bounding the strength limit of IGMs. Samtani and Nowatzki (2006) reported that the distinction of IGMs from soils and rocks is framed based on the shear strength of the geomaterials. Based on the geological origin, IGMs can be divided into two types: a) Cohesive Intermediate Geomaterials and b) Cohesionless Intermediate Geomaterials. Natural geomaterials such as shale, claystone, siltstone, mudstone, limestone, argillaceous rocks (clay-based sedimentary rocks), and arenaceous rocks (sand-based sedimentary rocks) fall under the category of cohesive IGMs (O’Neil and Resse 1999, Mokwa and Brooks 2008, Rodgers *et al.* 2019). Argillaceous-based sediments tend to possess cohesive strength, whereas arenaceous-based deposits do not possess any cohesion until they are cemented or consolidated to extremely dense conditions (Blyth and De Freitas 1984). The problematic shale can be further divided into cemented and non-cemented shales. The non-cemented shale loses its strength upon being subjected to fewer cycles of wetting and drying tests (USACE 2003). Similarly, the mudstone experiences additional volume change upon being inundated with water, followed by strength reduction (Chai *et al.* 2014). Moreover, the rocks reported as IGMs are also problematic due to their susceptibility to degradation upon exposure to atmosphere/ weathering and strength reduction (Loehr *et al.* 2017).

Cohesionless IGMs cover very dense granular material where the shear resistance derives predominantly from the frictional resistance (Mokwa and Brooks 2008). Table 1 presents the unconfined compressive strength variation of IGMs reported by several researchers and standard guidelines. From Table 1, it could be inferred that the UCS values of IGMs vary from 0.5 MPa to 25 MPa. However, as discussed above, the term IGMs is more often reported by O'Neil *et al.* (1996), O'Neil and Reese (1999), Mokwa and Brooks (2008), IRC -78 (2014), and Adhikari *et al.* (2020). But Table 1 also includes the range of UCS values of soft/weak rocks, which comes under the classification of IGMs. Even though the studies on IGMs have been reported for more than three decades, there was no standard

Table 2 Variation of mean COV for mean UCS, PLI, and $q_t$-Brazilian (Phoon and Kulhawy 2016)

| S.No | Mean UCS (MPa) | Mean COV | Mean PLI (MPa) | Mean COV | Mean $q_t$-Brazilian (MPa) | Mean COV |
|---|---|---|---|---|---|---|
| 1 | <50 | 14 | <2 | 15 | <4 | 17 |
| 2 | 50-100 | 12 | 2-4 | 26 | 4-8 | 17 |
| 3 | 100-150 | 17 | 4-6 | 23 | 8-12 | 18 |
| 4 | 150-200 | 17 | 6-8 | 30 | 12-16 | 19 |
| 5 | 200-250 | 13 | 8-10 | 17 | 16-20 | 26 |
| 6 | >250 | 7 | >10 | 11 | - | - |

agreement among the researchers, as the lower range of UCS value lies between 0.5 and 5 MPa, and the upper range lies between 0.5 and 25 MPa. Therefore, there is yet to be a scientific consensus on the strength boundary limit of IGMs. For comparison, the variation of unconfined compressive strength of cohesive soils, intermediate geomaterials, and rocks is schematically presented in Fig. 2.

A recent study by Adhikari *et al*. (2020) classified the natural geomaterials as soils, soil-based IGMs, rock-based IGMs, and rocks for the design of driven piles. Classification of geomaterials based on $(N_1)_{60}$ (where $(N_1)_{60}$ stands for observed N value corrected for hammer efficiency and overburden pressure) was suggested. Geomaterials with $(N_1)_{60}$ >50 were classified as soil-based IGMs (Table 1). Further, a chart was proposed to differentiate rock from rock-based IGMs, based on pile toe resistance, RMR, and UCS. Barrett and Prendergast (2020) recently developed an empirical correlation to estimate the pile capacity resting on weak rocks. They define the sedimentary-based weak rock with a UCS value of 5.0 MPa and less.

### *4.1 Laboratory tests for IGMs*

The following laboratory tests such as moisture content, grain size, mineral composition, triaxial consolidated undrained/ drained, unconfined compression test (UCS), and point load index test (PLI), could be used to characterize the intermediate geomaterials (Wilson 1976, Irvine *et al*. 2015). Prakoso and Kulhawy (2011) conducted UCS, PLI, and Brazilian tensile strength tests and studied the inherent variability of strength of intact rock specimens. They concluded that the effect of moisture content and specimen diameter has limited influence on the strength of intact rocks, such that the mean COV (ratio of standard deviation to mean) is only 5-8%. Table 2 summarizes the mean COV for UCS, PLI, and $q_t$-Brazilian of intact rock. However, the above-discussed variation may not be applicable to IGMs because moisture content, fractures, cracks, etc., influence the strength properties.

Broch (1974) reported a 30 to 100% strength reduction in montmorillonite shale upon inundation. Later, Stroud (1974) attempted to predict the undrained shear strength of insensitive clay and soft rocks. Quick undrained triaxial tests were conducted on the soil specimens. However, the results were more scattered, particularly while analyzing the very stiff geomaterials which reflect the sample disturbance. The obtained results were validated using plate load tests. Wilson (1976) conducted laboratory tests such as point load index, cube compressive strength, and unconfined compressive strength on weak mudstone. In addition, the undrained shear strength of mudstone is correlated as 1/2 of UCC, 3/8 of cube strength, and 12 times the point load index. For the direct measurements, the COV was 42% for PLI and UCS tests and 25% for cube compressive strength tests.

Johnston (1995) compared the UCS and point load index test and showed that the axial splitting mechanism underestimates the shear strength compared with the shearing mechanism. However, the UCS may also underestimate the shear strength due to discontinuities, mainly while testing large-diameter specimens (Hoek and Brown 1997). In addition, Kanji (2014) also reported that the PLI test might produce erroneous results in soft or deformable rocks. Haeri *et al*. (2017) addressed the effect of specimen shape on PLI value by performing laboratory tests and numerical simulations. Therefore, the accuracy of the laboratory test result is highly influenced by the method of testing, sample disturbance, the geometry of the specimen, degree of saturation, drainage conditions, presence of discontinuities, end effects, and shearing rate (Stroud 1974, Johnston and Navello 1994).

O'Neil and Reese (1999) suggested RQD as an index guide to optimize the specimen's minimum diameter for laboratory testing. A minimum of 50 mm diameter specimen is preferable for RQD of 70% and above, and a 75 mm diameter specimen is preferable for geomaterial with RQD of less than 50%. Further, IRC 78 (2014) reported that if (RQD+CR)/2 is less than 30%, it may not yield samples for laboratory testing.

#### *4.1.1 Sampling of IGMs*

Mokwa and Brooks (2008) reported that the core recovery of the IGMs Varies from 0% to 100%. Hence, it is essential to have exposure to select suitable in situ tests for procuring samples for laboratory testing. Clarke and Smith (1992) classified the weak rocks based on their suitability for preparing the intact rock samples, which is summarized in Table 3. Category A represents the rock type that may provide intact specimens with a higher recovery ratio. Soft or weathered rocks tend to fall into the B and C category. Generally, geomaterials in category B demand in-situ testing as it provides very poor disturbed samples that may not be representative samples to perform the test in the

Table 3 Categories of weak rocks for testing the shear strength (after, Clarke and Smith 1992)

| Category | Sample Quality | In-situ Testing | | | |
|---|---|---|---|---|---|
| | | SPT | PL | Prebored PMT | Self-bored PMT |
| A | VG | VG | VG | VG | VG |
| B | G/M | VG | G | G | VG |
| C | P | G | M/P | M/P | VG |

Note: VG- Very Good; G- Good; M-Moderate; P-Poor; SPT- Standard Penetration Test; PL- Plate Load Test; PMT- Pressuremeter

laboratory environment. In addition, category C rocks pose difficulties during the drilling process and the extrusion of samples. Kanji (2014) also observed that the site conditions might not favor sampling when the observed SPT N-value exceeds 50. Generally, it is suggested to depend on a triple tube core barrel to obtain weak rock samples with a higher core recovery ratio (ISO 22475-1 2021). Asem (2020) recovered weak argillaceous rock samples using a double core barrel (NX size) to determine the unconfined compressive strength. Furthermore, the laboratory test results could be considered an approximate estimate until the test is performed on high-quality undisturbed IGM samples. Also, the complication arises when the laboratory data has to be extrapolated to field conditions. Therefore, researchers/ practitioners prefer to characterize the IGMs using in-situ tests.

### 4.1.2 Probabilistic characterization of IGMs

Among the civil engineering materials, the uncertainties associated with the geotechnical design parameters are relatively high when compared with quality-controlled engineering materials such as steel (COV < 1%), high-quality concrete (1% < COV < 3%), and timber (COV < 10%) (Melchers and Beck 2018). For instance, Benson (1993) reported a COV between 27 and 767% for hydraulic conductivity of the compacted geomaterials used for landfill liner applications. Generally, geotechnical design parameter uncertainties can be classified as aleatory and epistemic (Phoon and Kulhawy 1999a, Baecher and Christian 2003). Hence, it is appropriate to describe the uncertainties in the collected data and site variabilities using statistical parameters such as mean squared error, standard deviation, mean, median, variance, and coefficient of variation (COV) (Loehr *et al.* 2017). Furthermore, the statistical way of analyzing the data provides transparency and accounts for the modifications made by different practitioners. Phoon and Kulhaway (1999a, 1999b) presented a detailed discussion on the typical variation of COV for geotechnical design parameters. In general, among the soils, the design parameters of cohesionless soils possess lesser COV than that of cohesive soils (Phoon *et al.* 2016).

Prakoso (2002) studied the typical variation of COV of the strength of intact rocks and weathered rocks and showed that the COV increases for weathered rocks (slight to moderately weathered). Furthermore, Phoon *et al.* (2016) highlighted that the statistical characterization of rock is limited compared to soils. A recent study by Pandit *et al.* (2019) also addressed the limited availability of engineering properties of rock data and adopted bootstrapping to avail more random sampling from a single limited data set. However, the samples representing the population should be unbiased to perform bootstrapping.

Herrera and Jones (2016) studied the variability of the subsurface profile in the vicinity of Little River Waterway, Florida. The subsurface profile comprises cohesive soil (top layer), IGM (intermediate layer), and cohesive soil (bottom layer). The COV of the site variability was 179%, which shows that the site variability is relatively high. However, the typical COV for bridge foundation design could be less than 100%. Further, to optimize the deviation in the design parameters, Herrera and Jones (2016) limited the deviation to $\overline{X} \pm 1.0\ S$ ($\overline{X}$, S – sample mean and standard deviation, also known as point estimators), and this implies that there is only a 68% of probability that the expected mean value will lie within these upper and lower bounds. Even though the sample follows the normal distribution, and point estimators are used to infer the point estimate of the population mean (μ) and standard deviation (σ), the proposed confidence interval is not generalized and is site-specific. A recent study by Asem (2020) developed empirical correlations for argillaceous rock, but the uncertainties pertained to material properties were not discussed. Hence, there is a need for probabilistic characterization of IGMs to address the uncertainties, and it should not be site specific.

### 4.2 In situ evaluation and characterization of IGMs

As discussed in Section 4.1.1, due to the sampling constraints, researchers attempted to identify and evaluate the engineering properties of IGMs using in situ tests. Standard Penetration Test (SPT), plate load test, pre-bored pressuremeter, and self-boring pressuremeter are commonly used for supporting in-situ tests for the characterization of IGM (Stroud 1974, Coal and Strout 1976, Clarke and Smith 1992, Johnston 1995, Mokwa and Brooks 2008, Stark *et al.* 2013). The following section discusses the respective in-situ tests in detail.

### 4.2.1 Standard penetration test

SPT is the widely used strength-based field index test (developed in 1927) by practicing geotechnical professionals due to its availability, economic feasibility, and robustness. The testing procedure is well established, and numerous correlations have been developed that could be used to predict shear parameters of both cohesive and non-cohesionless soils. Hence, researchers have attempted to identify the IGMs based on the N-Value. Fig. 3 presents the procuring of disturbed samples of soft clay and weathered rock using a split spoon sampler.

Cole and Stroud (1976) conducted the SPT test and reported that an N-value of more than 60 could be used to identify the presence of weak rock formations (weathered rock). IS -2911-Part 1 (Sec-2) (1997) also adopted a similar approach for predicting the shear strength of weathered materials to design bored cast-in-situ piles. Later, Dobereiner and de Freitas (1986) showed that the SPT-N value of 50 may signify weak sandstone formations. The characterization continued with Mayne and Harris (1993),

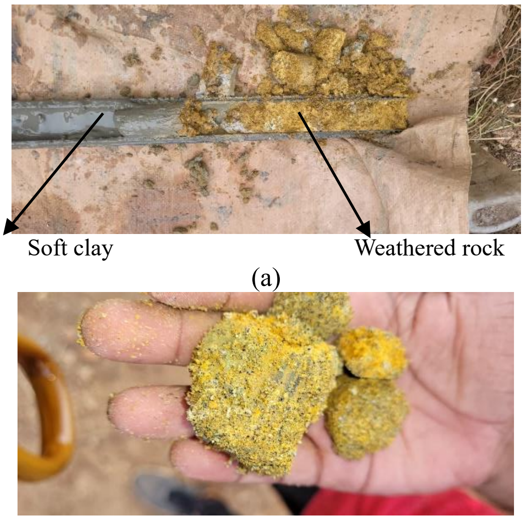

Fig. 3 (a) Split spoon sampler is partially penetrated into soft clay and IGM/ Weathered rock/ soft rock and (b) closer view of disintegrated pieces of weathered rock

where they faced difficulties in establishing a boundary between residual soil and partially weathered rock. In conclusion, they stated that $N > 50$ might be considered as a transitional material from soils to weathered rock. In addition, Clayton (1995) showed that $0 < N_{60} < 80$ and $80 < N_{60} < 200$ may indicate the presence of very weak and weak rock formations. O'Neil and Reese (1999) suggested that $50 < N_{60} < 100$ could be used as an index guide to identifying the Cohesionless intermediate geomaterial. Further, IRC 78-(2014) also recommended similar guidelines for the shear strength of Intermediate geomaterial to design deep foundations using the SPT N index value. For cohesive IGMs, the shear strength varies between 0.4 and 3.3 MPa, for the wide range of N-values from 60 to 300. Furthermore, using the SPT N value, the consistency of IGMs was categorized as very weak (60-100), weak (100 - 200), and moderately weak (200 to 300). Table 4 summarizes the characterization of IGMs using the SPT N-value adopted by various researchers and standard guidelines.

The SPT test is also called a one-parameter test because the obtained N value could be correlated to the shear parameters of either cohesive (c) or non-cohesive soils (ø). However, the SPT test loses significance while exploring c-ø soils.

### *Discussion on the use of N, $N_{60}$, and $(N_1)_{60}$*

As seen in the above discussion, different researchers frequently used the notations N, $N_{60}$, and $(N_1)_{60}$ to identify the IGMs/ weak rock/ soft rock; hence, confusion may arise about whether to use N or $N_{60}$ or $(N_1)_{60}$. Since the corrected SPT- N value is influenced by several factors, such as hammer efficiency, overburden stress, borehole diameter, rod length, and sampler type, it needs generalization. Further, Phoon and Kulhawy (1999a) studied the inherent variability of the field SPT -N value and reported COV between 25 and 50%. As a part of generalization, the N-value which represents the observed number of blows for 0.3 m penetration and corrected for 60% energy efficiency, is reported as $N_{60}$. IS 2911-Part 1 (1997), IS 1893 (2016), and IRC -78 (2014) used the notation N (not $N_{60}$) in their guidelines which is based on the standard penetration test protocol detailed in IS 2131 (1981). As IS 2131 (1981) suggested, the hammer efficiency should be at least 60%; hence, the observed N value is equivalent to $N_{60}$. Earlier researchers/ practitioners used N-value to identify the hard stratum for the sake of practicing professionals such that it aids in terminating the depth of exploration. Moreover, they also highlighted that the hammer efficiency was expected to vary from 50 to 55%, and hence, the difference between observed N and $N_{60}$ values would be marginal. In addition, Gannon *et al*. (1999) suggested that for identifying weak rock formations, using N or $N_{60}$ may not misguide if the corrected $N_{60}$ value does not deviate from the observed N value by more than 15%. Recently Adhikari *et al*. (2020) depended on $(N_1)_{60}$ to distinguish soils from soil-based IGMs. They suggested that $(N_1)_{60} > 50$ could be used as a criterion to differentiate soils from soil-based IGMs. However, the proposed boundary limit by Adhikari *et al*. (2020) for soil is slightly on the upper side, and this might be due to the inclusion of a wide range of soils into the characterization, such as ML, SC, SM, SP, SW, GC, GM, GP, and GW. But, Tokimatsu and Seed (1987) showed that $(N_1)_{60} > 35$ represents the apparent density for sandy soil in very dense conditions. In general, sandy soils with $N_{60} > 50$ represent a very dense stratum (Terzaghi *et al*. 1996, Day 2002, Coduto 2011, Briaud 2013). Hence, a general scientific consensus has to be reached in this regard.

### *Correlations for prediction of shear strength of cohesive intermediate geomaterial*

Several researchers attempted to develop an empirical correlation to predict the undrained shear strength of geomaterials through the SPT N-value. The general form of the equation relating SPT blow count (N) and shear strength (kPa) is presented as "$S_u = f_1 \times N$". Peck *et al*. (1974) established a correlation to predict the shear strength of soft to stiff cohesive material (Eq. 1). The $N_{60}$ values range from 2 to 30; the $f_1$ factor lies between 6.2 kPa and 6.7 kPa. Stroud (1974) conducted SPT to assess the undrained shear strength of insensitive clay and soft rocks (Eq. (2)). The quick undrained triaxial test was performed on 102 mm diameter soil specimens. The $f_1$ factor reported by Stroud (1974) lies between 4 kPa (high plastic clay) and 6 kPa (low plastic clay). The $f_1$ factor was slightly lesser than the value reported by Peck *et al*. (1974). Later, Abu Hejleh *et al*. (2003) attempted to use the SPT – N value to estimate the unconfined compressive strength of claystone (Eq. (3)). However, the proposed empirical equation is only suitable for the back prediction of the unconfined compressive strength of weak rocks of less than 860 kPa.

To summarize, it should be noted that the proposed correlations are site-specific and also differ for the type of geomaterial. Phoon and Kulhawy (2008) studied the uncertainties of the empirical correlations used for back-predicting the shear strength of cohesive and cohesionless soil. They reported COV of 50% to 70%, where the correlations were developed using the SPT N value.

Table 4 Identification and characterization of IGMs based on SPT-N values

| **S.No** | **Type of IGMs** | **SPT -N value (for 0.3 m penetration)** | **Reference** |
|---|---|---|---|
| 1. | Weak Sand Stone | $N \geq 50$ | Dobereiner and de Freitas (1986) |
| 2. | All | N> 50 | Mayne and Harris (1993) |
| 3. | All | $0 < N_{60} < 80$ -Very weak<br>$80 < N_{60} < 200$ - weak | Clayton (1995) |
| 4. | Cohesionless IGMs | $50 < N_{60} < 100$ | O'Neil and Reese (1999) |
| 5. | Cohesionless IGMs | N> 50 | Mokwa and Brooks (2008), |
| 6. | All | $N_{60} > 60$ | IS 2911 – (Part 1/ Sec2) -2010 |
| 7 | Cohesionless IGMs | $N_{60} > 50$ | AASHTO (2010), IRC 78 (2014) |
| 9. | Soil-based IGMs | $(N_1)_{60} > 50$ | Adhikari *et al.* (2020) |

Table 5 Empirical correlations for prediction of shear strength of soft clay and cohesive IGMs

| **Eq. No.** | **Empirical Correlation** | **Reference** | **Type of Geomaterials** |
|---|---|---|---|
| 1. | $S_u = 6.2 - 6.7 \times N_{60}$ | Peck *et al.* (1974) | Soft clay to stiff clay |
| 2. | $S_u = 4.0 - 6.0 \times N_{60}$ | Stroud (1974) | Insensitive clay to soft rock |
| 3. | $S_u = 5.75 \times N_{60}$ | Abu Hejleh *et al.* (2003) | Claystone |
| 4. | $S_u = 1.87 \times \grave{N}$ | Stark *et al.* (2013) | Soft Shale and Weak Rocks |
| 5. | $q_u = \dfrac{2.3 \times \grave{N}_{85}}{4 \times \sigma'}$ | Asem (2020) | Weak argillaceous rocks |

Note: $S_u$ (kPa), $q_u (MPa)$

### 4.2.2 Modified standard penetration test

Stark *et al.* (2013) developed a new Modified Standard Penetration Test (MSPT) method to explore intermediate geomaterials. The MSPT intends to overcome the shortcomings of the SPT test while conducting the test on weak rocks. Extrapolation of N values (design aspect) due to penetration of sampler lesser than 300 mm and damage of soil sampler could be considered some of the limitations of SPT test on weak/weathered rock. The testing equipment used for the MSPT is like that of SPT; however, the observation procedure slightly differs from the conventional.

Stark *et al.* (2013) proposed the term Penetration Rate ($\grave{N}$) and the same was used for the qualitative estimation of undrained shear strength of weak shale. Penetration rate measures the thickness of penetration (by measuring the length of the rod) for every ten blows which is recorded to a maximum of 100 blows. Fig. 4 presents the graphical method to determine the penetration rate from the MSPT test. As seen in Fig. 4, the inverse of the slope defines the penetration rate (1/ (penetration /blow count)). Table 4 includes the proposed correlation (Eq. (4)) to determine the shear strength based on the penetration rate. The shear strength of the tested shale typically varies from 100 kPa to 3590 kPa. Further, the detailed procedure for performing the MSPT could be referred to Stark *et al.* (2013). Asem (2020) developed correlations with the aid of penetration rate to predict the strength properties of argillaceous rocks (Eq. (5)). He claimed that the proposed empirical model accounts for the influence of water content, in-situ effective stress, and hammer efficiency (85%). However, the prediction is more significant, only when effective stress lies between 0 and 0.8 MPa. Certain building design guidelines like IS 1893 Part 1 (2016) and IRC 114 (2018) state that the hammer efficiency may vary from 45 to 85%. Hence the developed model (Equation No. 5) may need to be revised in predicting the shear strength for hammer efficiencies other than 85%.

A more recent study by Sun *et al.* (2022) addressed one of the limitations of SPT, the extrapolation of N values when the sampler penetration is lesser than 300 mm in weathered strata. They compared the extrapolated N values and measured N values of greater than 50, and showed that the measured N values are higher than the extrapolated N values because of non-linear variation between the blow count and penetration. The results are validated with field studies and additional regression analysis.

### 4.2.3 Pressuremeter tests

The pressuremeter test (PMT) is a widely used test to estimate the in-situ response of the geomaterial at shallow and deeper depths. ASTM D4719 (2020) suggests equal pressure increment (stress-controlled) and equal volume increment (strain-controlled) procedures, and the test results could be inferred to obtain the stress-strain response of the soils tested. The inferred Pressuremeter modulus (Menard modulus) and limit pressure values from the tests could be correlated to getting the geotechnical design parameters. Similar to bounding the shear strength limits for soils, the limit pressure values from the PMT could also be used, and it was estimated as 1.6 MPa for hard clay and 2.5 MPa for very dense sand (ASTM D4719). Clark and Smith (1992) attempted to use a self-boring pressuremeter to evaluate the shear strength of weak rocks, and the test was performed to a deeper depth. The typical limit pressure was found to be closer to 10 MPa. Further studies by Pierre and Michel (2014) utilized limit pressure to define the soft rocks, and it typically ranges from 2 MPa to 10 MPa. In addition, Cruz *et al.* (2014) reported the yield pressure (where the deformation is expected to be plastic in this phase) of very dense residual soil (N >60), and it was found to vary between 1.4 and 3.6 MPa. Hence, it could be emphasized

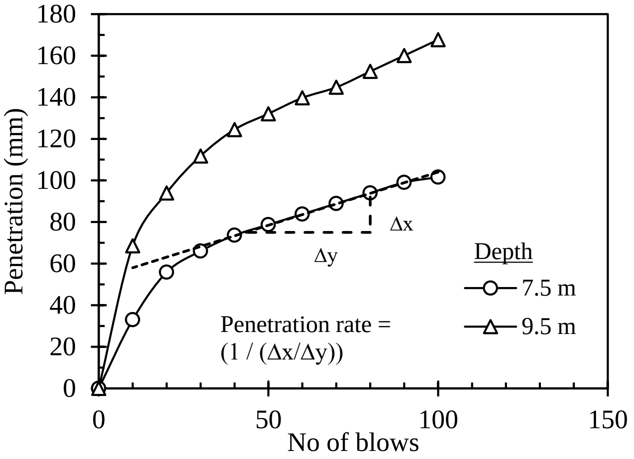

Fig. 4 MSPT blow count vs. depth of penetration (after Stark *et al*. 2013)

that geomaterial with a limit pressure of higher than 2.5 MPa could be treated as either IGMs or weak rock.

### *4.2.4 Plate load test*

The Plate Load Test (PLT) is a practical in-situ test for determining the shear strength of soils and rocks, particularly for jointed/ fractured geomaterials. Only limited works of literature on identifying IGMs using plate load tests exist. However, this test must maintain its significance while exploring IGMs at deeper depths and when the groundwater table is shallow. Such field constraints increase the complication in performing the test and, thereby the project's cost. However, as discussed in Section 4.1, Stroud (1974) conducted the plate load test on very stiff/ hard clay to validate the test results obtained from the triaxial test. Since very stiff and hard clays exhibit behaviors that are similar to IGMs, it can be inferred that the PLT is an efficient test to characterize IGMs. However, further research on the use of PLT for IGMs will ensure the accuracy and suitability of PLT in this regard.

Further, while conducting plate load tests on rock, the effect of Excavation Disturbed Zone (EDZ) should be considered in obtaining such geomaterials' strength and stiffness properties. The EDZ is defined as the zone in which the properties of the adjacent rock strata may change while excavating the rock strata, due to the redistribution of stress, formation of fissures/ cracks, and desaturation (Sato *et al*., 2000). The width of EDZ may vary from 0.3 to 0.5 m, depending on the excavation method (mechanical method or blasting). Therefore, the in-situ plate load test could effectively explore IGMs at shallow depths if the uncertainties of EDZ are addressed statistically. Furthermore, additional studies could be extended to develop an empirical model to back-predict the design parameters of IGMs.

## 5. Scope for future work

As summarized above, the definitions and correlations for IGMs mostly pertain to deep foundations. Unlike cohesive and cohesionless soils, there is no general classification chart for IGMs. Hence, a detailed probabilistic study on IGMs could be addressed for materials that possess high uncertainties. Addressing the suitable underlined probability distribution (normal, lognormal, etc.) for the IGMs could be used to fix interval estimation based on the confidence level (90%/ 95%/ 99%) and level of significance. Similar to works reported by Paikowsky *et al.* (2004) for site variability, the strength variation of IGMs could be studied and categorized based on COV as low, medium, and high. Fixing the upper and lower bound of COV for IGMs according to their geological origin (cohesive and cohesionless IGMs) may provide more insight while designing the structures. Further, as the existing correlations are primarily based on the SPT test and use the extrapolated N-Value, it is difficult to account for the variability in the prediction model. Hence as an alternative, more correlations could be established using penetration rate ($\dot{N}$) (obtained from MSPT), where the uncertainty due to extrapolation could be eliminated. Moreover, there are limited studies that address the consistency of penetration rate index value. Furthermore, the transformation uncertainty associated with the penetration rate is not yet explored.

## 6. Conclusions

This article explores the laboratory and in-situ test methods used to characterize intermediate geomaterials. From the above discussion, it was observed that most researchers and practitioners attempted to differentiate soil from rock through uniaxial compressive strength (shear strength). The same criteria were also used for IGMs. However, the shear strength boundary limit criteria for IGMs differ among practicing professionals. The uncertainty might be due to difficulty in capturing the mechanical behavior of IGMs that exists between soils and rocks. Furthermore, the significant deviation occurs due to the sampling constraints in the IGMs, which lead to poor estimation of their engineering properties. The review also finds that only limited studies are reported on the probabilistic characterization of IGMs, which is an important concern in the limit state design of geotechnical structures. The attempts are continuing to evaluate the strength of IGMs. Among the in-situ tests, SPT (strength-based index test) is widely used to characterize and predict the shear strength of such geomaterials using correlations.

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