Multi-method in situ geophysical testing in a high porosity chalk mass

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ABSTRACT

Chalk is a silt-sized soft biomicrite rock often encountered as a low to medium density, high porosity, structured material within a fractured mass. In recent years, there has been increased interest in the behaviour of chalk and the development of new design procedures for pile foundation installation design, motivated by several large-scale onshore and offshore infrastructure projects. Recent modelling has demonstrated the importance of accurately characterising the operational stiffness of the chalk mass. While several methods exist to measure the chalk's stiffness in situ, they are often subject to significant scatter, with no guidance available to the end user on interpretation or on which method should be used as a baseline. A new programme of multi-method in situ geophysical testing in chalk at a well-characterised onshore test site in Southern England is described that forms part of a wider research project. The chalk deposit is shown to be relatively uniform with depth which provides a unique opportunity to apply multiple methods and interpretations without the influence of significant layering. The experimental programme is described and the interpretation and selected results of downhole geophysical tests at depths up to 40m are presented. The chalk's remarkably high shear stiffnesses are shown to be highly repeatable and consistent when rigorous test execution and analysis is applied.

Keywords: chalk; geophysical testing; small-strain stiffness.

1. Background

Several geotechnical design procedures require information on the small-strain shear stiffness, G_0. In situ shear wave velocities, V_s, measured during invasive geophysical tests are often used to calculate G_0:

$$G_0 = \rho V_s^2 \qquad (1)$$

where ρ is the density of the material. Accurate measurement of V_s is key since it is squared in Eq. (1). Seismic cone penetration tests (SCPT), PS logging (PSL), Downhole (DH) and crosshole (CH) geophysical tests are often employed to determine V_s in situ. The basic principle of the test involves measuring the travel time, Δt of shear waves over a ray path distance, ΔL. Recently, several Authors have highlighted the difficulties in accurately obtaining such measurements and the uncertainties inherent in data sets (see e.g. Gibbs et al., 2018, Parasie et al., 2022, Stolte and Cox, 2019). Small-strain shear stiffness can also be measured in the field using pressuremeter tests (PMT). In situ tests are often preferred over laboratory techniques, such as bender elements (BE) or the resonant column (RC), where retrieving samples representative of in situ conditions is challenging.

There are very few studies in the literature that consider chalk, a soft silt-sized rock found widespread across Northern Europe and within the footprint of several onshore and offshore infrastructure projects. Some of the only results are reported by Matthews et al. (2000) who found that (i) the ratios of laboratory to in situ shear stiffness were typically much greater than one and (ii) the trends in G_0 depended on both the degree of fracturing and intact dry density.

Recent research has included intensive characterisation of a low-medium density chalk deposit at an established research site to support the interpretation of a large programme of foundation load testing see e.g. Liu et al., 2022, Vinck et al., 2022 and Buckley et al., 2022 and Jardine et al., 2023. While the laboratory strength and stiffness trends reported were highly consistent, the in situ stiffness results showed significant scatter, particularly above the water table. Laboratory bender tests confirmed the high ratios between laboratory and in situ stiffness observed by Matthews et al. (2000).

The scattered and apparently method-dependent small strain stiffness trends complicated the detailed modelling of load test experiments at the site e.g. in order to match the laterally-loaded field test results reported by McAdam et al. (2021), Pedone et al. (2023) were forced to use an operational shear stiffness ≈1/3 of the average values from scattered in situ seismic tests.

A new extensive programme of characterisation has been carried out, at the same low-medium density chalk site that includes: (i) drilling and sampling and in situ testing to depths of up to 44 below current ground level (mbgl) (ii) SCPT, PSL, DH and CH geophysics and (ii) bored PMT. The testing programme aimed to establish the influence of the testing technique and execution on the shear stiffness profile while also considering data quality and interpretation methods. Material specific factors for the chalk are also considered. The tests are part of a wider programme of work that aims to reduce the uncertainty associated with key soil and rock parameters for use in design (Rieman et al., 2024, Shinde et al., 2024). An overview of the chalk testing programme is presented here, along with typical results from the downhole testing campaign.

2. Background

The present study utilises a new area of the disused chalk quarry near the village of St Nicholas-at-Wade in Kent, England, described previously by Buckley et al. (2018) and Vinck et al. (2022). Fig. 1 shows the study area and the previous pile testing areas utilised by the ALPACA/+projects.

Four rotary-cored boreholes were installed to between 25 and 44mbgl of ≈6.7mAOD (metres above ordnance datum) along with CPTs and SCPTs installed using a truck mounted rig. At the end of borehole drilling, 90mm PVC casing was grouted into place, in preparation for geophysical testing, with the grout density chosen to mimic that of the surrounding chalk (ASTM:D7400, 2019). Pauses in drilling operations were necessary to carry out the PMT and PSL. Cement bond log (CBL) testing (Winn et al., 1962) was key for assessing the quality of the grout/chalk interface and supporting the interpretation.

The water table was shown previously to lie ≈0.9±0.25mAOD from tensiometer tests. The new BHs conducted for the present study showed high total core and solid core recoveries and facilitated detailed logging of the stratigraphy (see Buckley et al., 2024).

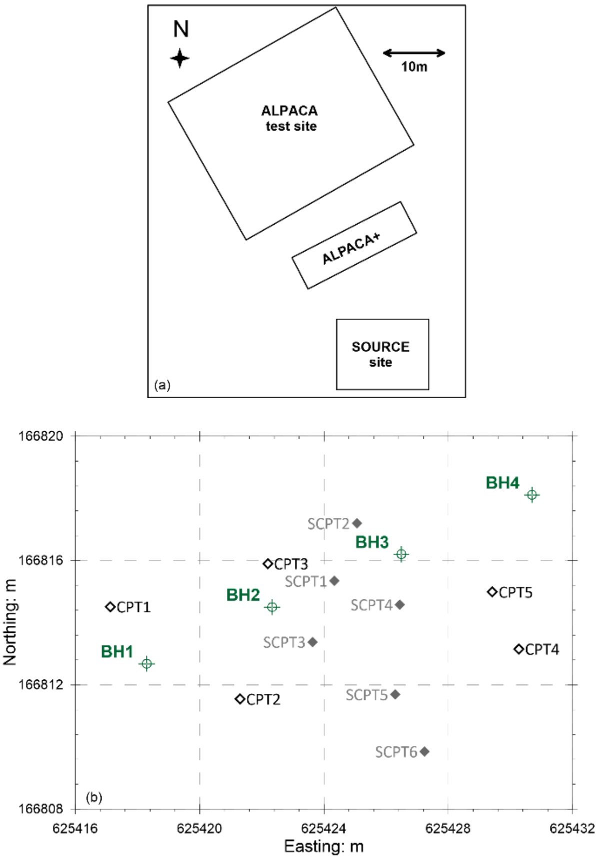

Figure 1. Study area location plan (a) wider site including pile testing areas (b) current study area

The Seaford chalk formation, encountered from the base of the inspection pits in the boreholes, consisted of well-structured, clean, very weak-to-weak low-medium density white chalk with fractures slightly open and spaced at 150-200 mm (CIRIA grade B3/B2 (Lord et al., 2002)). The fractures were typically sub-horizontal and sub-vertical and moderately speckled/stained. Few small to medium nodular flints were encountered with more frequent and larger flints found in BH2 and between 2 and -2mAOD in BH4. The chalk fractures became mostly closed and widely spaced with depth with the grade improving to A3/A2 from ≈-4mAOD. From ≈-21.5mAOD the fracture spacing reduced to 300-1000mm (Grade A2/A1) up to the end of borehole at ≈-37mAOD.

Significant flint bands were encountered at the study area which led to early refusal of the CPTs in all but CPT3; see Fig. 2. Deeper penetrations may have been possible with different cone configurations and additional reaction force. The corrected cone resistances, q_t follow the same trends as seen previously at the site, lying between 5 and 35MPa with spikes seen in thin, discontinuous, flint bands. The sleeve friction, f_s, ranged from 50 to 100kPa while the excess pore water pressure measured at the u_2 position showed values up to 7MPa is measured as the chalk de-structured during cone penetration. Pore pressures up to 10MPa at the u_1 position were previously reported by Buckley et al., 2021. A local layer of low resistance was observed at depth see e.g. CPT3 between -3 and -7mAOD. The low resistance layer in CPT3 lies just above a zone of significant core loss seen in the nearby BH2, with CBL testing also indicating the presence of a possible void in this region.

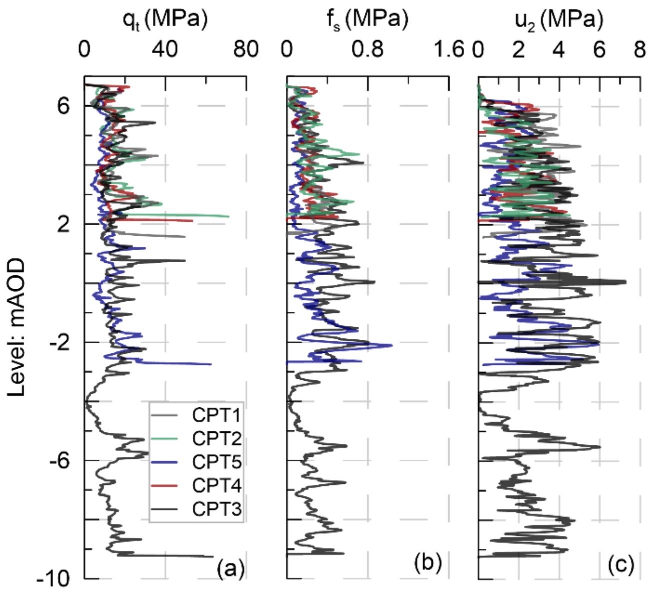

Figure 2. Study area location plan (a) wider site including pile testing areas (b) current study area

3. Testing programme

Each of the four boreholes included downhole geophysical logging and PS Logging, with SCPTs conducted at adjacent locations. The SCPT seismic module was connected to the CPT rods and incorporated two uni-axial horizontal geophones spaced at 0.5m. DH tests in the boreholes involved a pair of vertically-installed receivers (multi-axial BGK5 (Geotomographie)), spaced at 2m, clamped to the borehole wall. Each receiver included one vertical (V) and 4 horizontal (H1, H2, H3, H4 in clockwise order) sensors, separated by 45°. In the case of the DH and SCPTs, shear waves were generated on the ground surface by striking a hammer on a shear beam weighted by a vehicle (see Fig. 3 (a) or (b)). The position of the shear beam in relation to the axis of the rods/borehole was located precisely, while checks on the penetration were made during testing using an independent reference point placed above ground level.

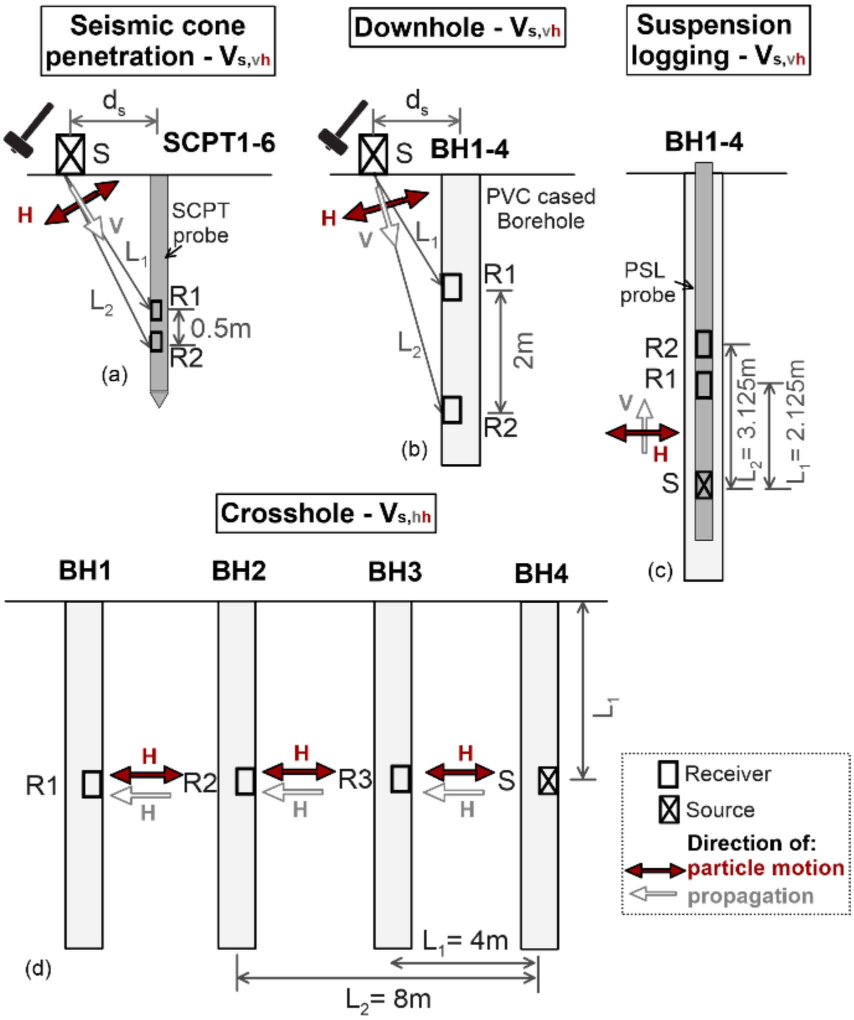

Figure 3. Geophysical methods considered (a) seismic cone penetration testing (b) downhole borehole geophysics (c) PS logging (d) crosshole borehole geophysics.

PS logging involved a single probe consisting of a reversible-polarity horizontal solenoid and strike cylinder arrangement aligned to two bi-axial receivers (Hen-Jones et al., 2024) and separated by filter tubes (Fig. 3 (c)). The source motion creates an impulsive pressure wave in the borehole fluid that is converted, at the borehole wall, to compression and shear waves propagating in the surrounding material. These waves in turn cause pressure waves to be generated in the fluid surrounding the receivers spaced 1m apart. The tests were conducted at 0.5m test intervals in cased boreholes. A repeat test prior to installation of the casing in BH4 was used to check the influence of the casing on the results.

The DH tests involved testing in increments of 1m to maximum depths of ≈24mbgl (BH1-3) and ≈42mbgl (BH4) while the PS Logging reached ≈20m (BH1-3) and ≈40m (BH4) in 0.5m test increments. The SCPTs, which involved test increments of 0.25 to 0.5m, reached early refusal in all cases. Deeper tests were attempted in SCPT4, by pushing a "dummy" cone to depth and placing the seismic module in the pre-made hole. Multiple shots were acquired and the data stacked in all cases to reduce the signal-to-noise ratio.

Crosshole tests generated shear waves at different depths within BH4 which were measured by receivers placed at the same elevation in BHs 1-3 (Fig. 3(d)). The boreholes were spaced ≈4m apart. Horizontally-polarised waves were generated every 1m using an impulse generator within a BIS-SH-DS source (Geotomographie) clamped to the borehole wall. The same BGK5 system was used as a receiver in BH3. BH1 and BH2 included BGK7 (Geotomographie) multi-axial sensors comprising one vertical and six horizontal components. The source and receivers were advanced in increments of 1m.

Self-boring pressuremeters are preferable in chalk due to the de-structuration that can be caused by pushed-in types (Whittle et al., 2017). A standard SBP probe was installed using the reaction provided by the rotary core rig. Where flints could not be avoided reaming pressuremeters and high pressure dilatometer tests were carried out in pockets pre-bored using a core barrel.

Additional laboratory testing has supplemented the existing extensive characterisation, which did not include information below ≈9.5mAOD. Index and unconfined compressive strength tests were used to augment the existing profiles and check for changes in stratigraphy in the deeper chalk layers. An intensive programme of resonant column and cyclic triaxial testing on intact samples is underway at the University of Glasgow.

4. Interpretation of seismic data

Pre-processing of the 1160 traces analysed from DH, CH, PSL and SCPT included:

- application of a zero phase-shift low-pass Butterworth filter with a cut-off frequency, f_c, selected to remove unwanted noise and additional artifacts; typical frequency spectra are shown on Fig. 4.
- application of a Hamming window function, in selected cases, to minimise distortion due to different propagating modes.
- in the case of the SCPTs, which were sampled at only 5kHz, up-sampling of the signals to a 100 times higher sampling rate (Karl et al., 2006)

For the dual receiver arrangements employed, V_s is calculated by dividing the difference in wave travel path between the two receivers, ΔL by Δt:

$$V_s = \frac{L_2 - L_1}{t_2 - t_1} = \frac{\Delta L}{\Delta t} \quad (1)$$

The true-interval travel times, Δt between receivers at vertical distances, d_1 (R1) and d_2 (R2) from ground level, where $d_2 > d_1$ (Fig. 3) were determined using the cross-correlation (CC) technique (Baziw, 1993, Campanella and Stewart, 1991) in >80% of cases. CC is typically considered the most reliable method since it uses information from the whole signal, can be easily automated and is relatively free of human bias. The remaining data were assessed to be of lower quality and underwent a careful manual processing approach.

The relative travel times between dominant peaks (P-P) on both signals was used for a portion of the data. Where reverse polarity (RP) signals were available, the relative travel times between characteristic cross-over points on the signal (e.g. before dominant peak) were used. This applied primarily to the CH data, where RP allowed the most consistent picking in ≈60% of cases. A straight ray assumption between source and receiver was adopted since the source offsets were small and the profile was assessed to be relatively homogeneous.

An assessment of data quality, for signals such as those shown on Fig. 5 can help analysts to make relative judgements in analyses. The method for quantifying uncertainty suggested by Zheng et al. (2024) in these proceedings could not be applied here, as the un-stacked data was not saved by the data acquisition system. Instead, the data quality classification proposed by Baziw and Verbeek (2017) that ranks traces based on their individual quality and makes recommendations for processing, was applied. The classification proved invaluable in assessing the relative quality of the large number of signals obtained. The results of the complete testing programme are reported by Buckley et al. (2024). The following Section provides an overview of typical results; for brevity only selected typical results from DH, and PSL are described.

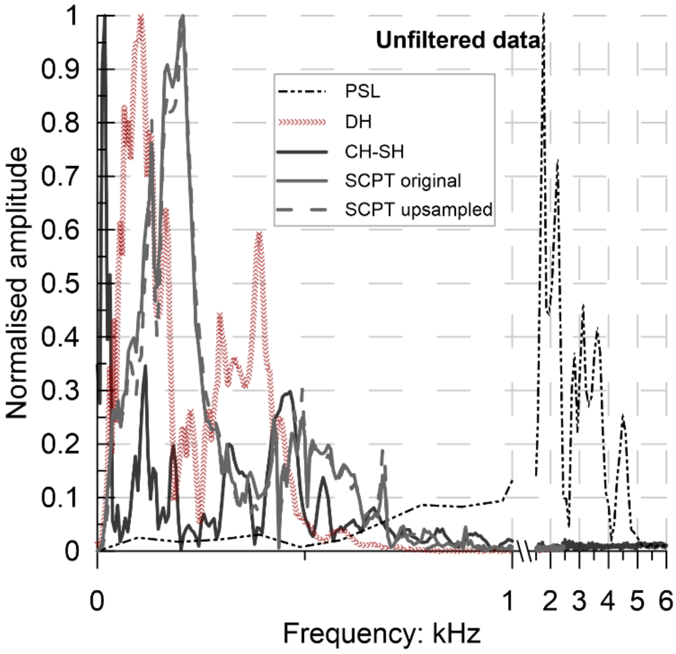

Figure 4. Frequency spectra of typical signals recorded at the chalk site

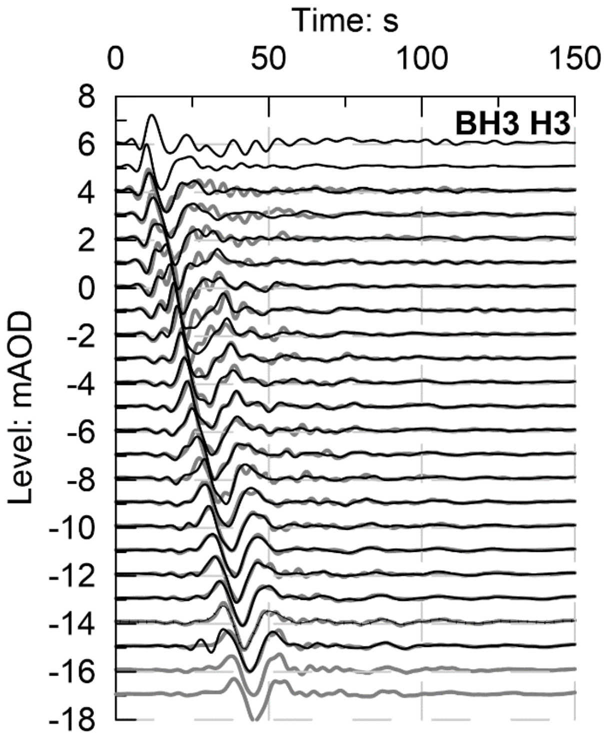

Figure 5. Example downhole test data from one receiver at BH3

5. Typical downhole testing results

Example profiles of vertically travelling and horizontally polarised shear wave velocites, $V_{s,vh}$, interpreted from DH testing in BH1 to 4 are presented in Fig. 6. Also shown is the fracture index (average number of fractures per 1m) trend from all four boreholes. Below the water table and across the study area, the DH $V_{s,vh}$ values are remarkably consistent with average values typically of 880m/s and σ values of <5% of the mean.

Above the water table, where the chalk mass is more heavily fractured and the fractures are air-filled, the results exhibit more scatter. The average $V_{s,vh}$ was 583±86 m/s, and the trends were consistent between different sensors and source offsets. It is worth noting that this is consistent with the G_0≈500MPa utilised by Pedone et al. (2023) to match the results of laterally loaded piles installed above the water table. The mean $V_{s,vh}$ at -35.8mAOD in BH4 was 934m/s at an in situ vertical stress of ≈480kPa representing a weak trend for $V_{s,vh}$ to increase with depth over vertical effective stress.

The geometry of the PSL probe and location of the water table led to the first PSL measurements occurring at ≈0.5mAOD. Fig 7 plots a comparison of the $V_{s,vh}$ in the uncased and cased BH4 showing very similar trends and minimal influence of the casing on the measured values. While this approach worked well in the relatively high stiffness chalk, it is unlikely to be suitable in formations with larger stiffness contrasts between the formation and the PVC. In general, the PS logging results followed the trends in mean DH $V_{s,vh}$, with values of 892±88m/s and also showe little variation with depth or burial stress.

6. Summary

There are very few examples of in situ stiffness measurements in chalk available in the literature. Existing information at an established test site indicates trends that are both method and in situ stress/fracture dependent. A new extensive in situ geophysical and pressuremeter testing programme has been carried out and selected results are presented here. Preliminary results presented suggest (i) a strong influence of fracture pattern on in situ stiffness, particularly above the water table where the fractures are open and air-filled (ii) a limited influence of burial depth or in situ stress on stiffness trends. Overall the results appear more consistent than the data collected previously, and will provide benchmark data for input into empirical design approaches and the calibration of numerical models.

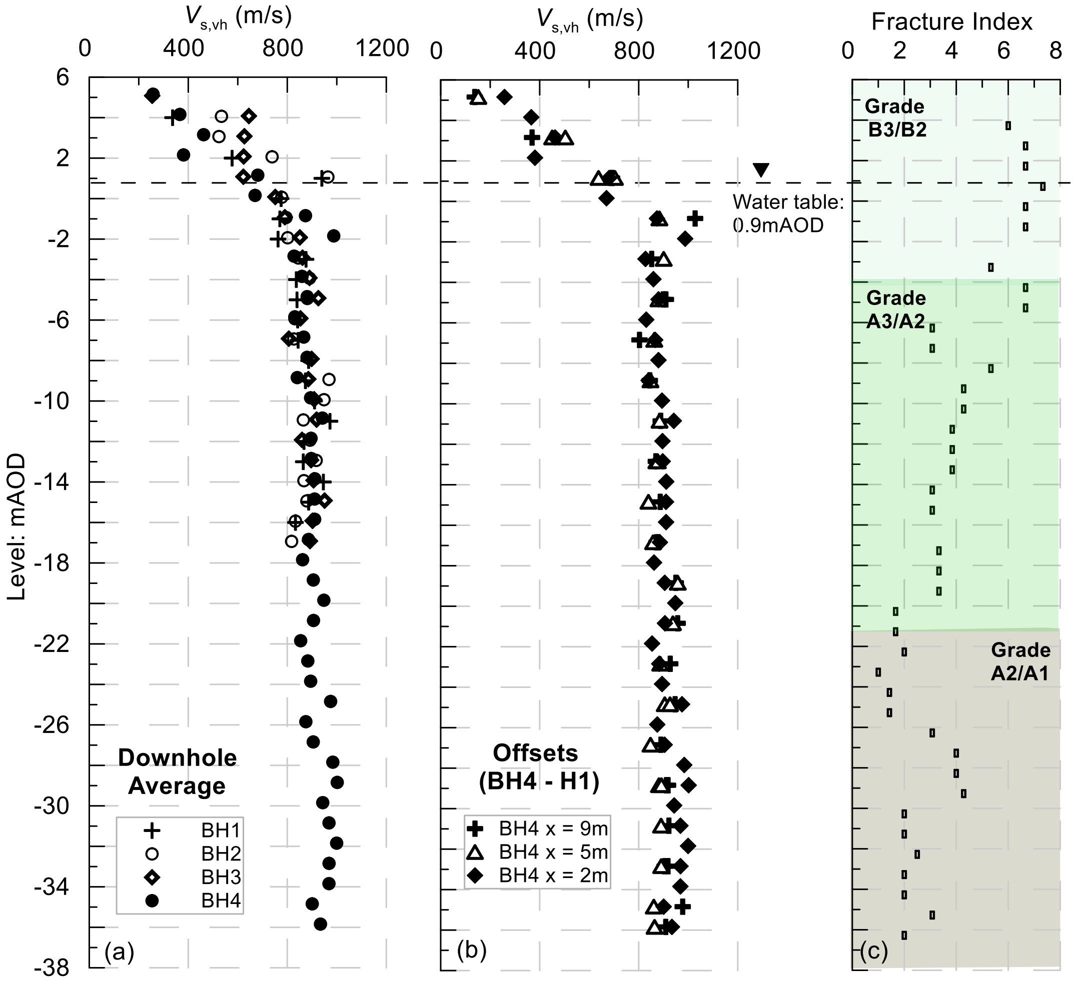

Figure 6. Downhole testing (a) Average results in BH1 to BH4 (b) varying offsets in BH4 and (c) fracture index

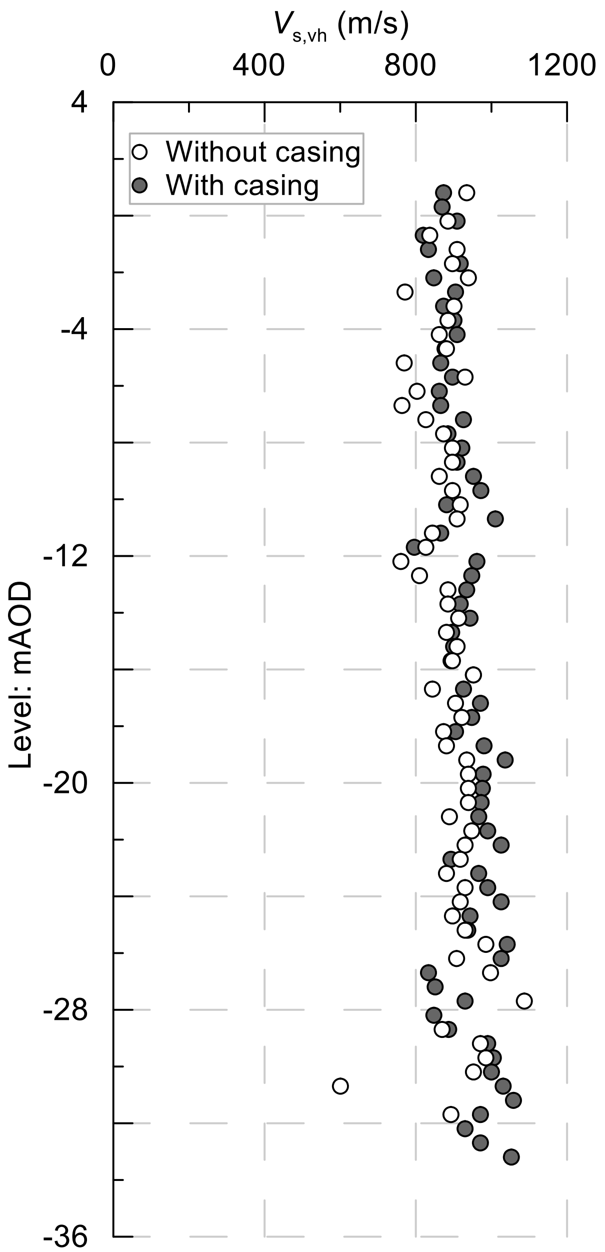

Figure 7. PS logging in cased and uncased borehole BH4

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