

## 2. NATURE AND EXTENT OF THE ENVIRONMENT AFFECTED BY ACTIVITY

The following section provides a description of the baseline or status quo environment as well as the social-economic parameters that characterise the region and the study area, and is derived from various specialist studies as well as data sources including aerial photographs, topo-cadastral maps and national and provincial databases.

### 2.1 Biophysical aspects affected

#### 2.1.1 Geology

The site is underlain by transported silty and gravelly soils over residual soils developed over mudrock bedrock belonging to the Vryheid Formation from the Ecca group which forms part of the Karoo Supergroup. According to the Kruger report, dolomite and chert bedrock occurs at a deep below the surface of the site (Dolomite bedrock at a depth of 40m). The dolomite and chert bedrock belongs to the Malmani Subgroup, which forms part of the Transvaal Supergroup.

To determine the suitability of the proposed site for the construction of the pond and wetland wastewater treatment works, a site investigation was conducted by JOHANN VD MERWE (Pty) Ltd. (Consulting Applied Earth and Environmental Scientists). The full geotechnical report by Mr. Johann van der Merwe, entitled Factual Report on Geotechnical Investigation carried out for the proposed: Delmas Abattoir Waste Water Treatment Works on: Portion 21 of the farm Geluk 234-IR, Delmas District, Mpumalanga Province, can be viewed under Appendix D. The following information has been extracted from the report.

A site investigation was conducted on the proposed site in February 2010. The investigation was commissioned in order to determine the foundation conditions for the construction of the WWTW.

Twelve pits (Figure 20) were excavated using a backactor, supplied by Willem Groenewald from Delmas. These pits were entered, inspected and profiled by Mr. van der Merwe in accordance with the methods advocated by Jennings *et al.* (1973). Disturbed and undisturbed soil samples were taken and sent for analysis in Messrs SNALAB's Laboratory from Waltloo (JOHANN VD MERWE, 2011). The soil profiles are given as Figure 8 to Figure 19 at the end of this section.

The following geotechnical considerations were investigated:



### 2.1.2 Shear Strength Characteristics

An undisturbed and a remodelled sample (DA/1, DA/9) from the site were subjected to saturated consolidated drained direct shear tests. These tests simulate long-term conditions and a number of shear strength parameters were measured under normal stresses of 50, 100 and 200 kPa. The results of the tests are given below as Table 3.

Table 3: Shear Strength Test Results (JOHANN VD MERWE, 2011).

| Hole nr. | Depth (m) | Dry density (kg/m <sup>3</sup> ) | Normal stress (kPa) | Cohesion (Kn/m <sup>2</sup> ) | Angle of friction (degrees) |
|----------|-----------|----------------------------------|---------------------|-------------------------------|-----------------------------|
| DA/1     | 1.0       | 1 221                            | 50, 100, 200        | 0.7                           | 24.7                        |
| DA/9     | 2.2       | 1 637                            | 50, 100, 200        | 1.1                           | 28.1                        |

The results of the tests indicate moderate friction angles and very low cohesion values for the remoulded and in situ clayey sand and gravelly silt that blanket the site.

### 2.1.3 Groundwater and soil chemistry

No water seepages were encountered during the investigation which was carried out toward the end of the wet season. However, season perched water conditions may possibly occur at the interface of the dense, ferruginised sandy silt and the overlying very loose gravelly and sandy materials.

Foundation materials are expected to be potentially chemically aggressive towards buried ferrous pipes with electrical conductivity values of 0.008 – 0.077S/m and pH values between 5.64 – 6.97. Non-ferrous metal or plastic pipes would therefore be more suitable for underground use on the site.

### 2.1.4 Soil permeability

Undisturbed (DA/1 and DA/9) and remodelled (DA/1 and DA/2) samples from the site were subjected to falling head permeability tests. The test results are given in the table below.

Table 4: Permeability tests (JOHANN VD MERWE, 2011).

| Hole nr. | Depth (m) | Material type | Initial moisture content (%) | Dry density (kg/m <sup>3</sup> ) | Coefficient of permeability (cm/s) |
|----------|-----------|---------------|------------------------------|----------------------------------|------------------------------------|
| DA/1     | 0.0-1.4*  | 8.9           |                              | 1 995                            | 1.9E-08                            |
| DA/1     | 1.0       | 11.4          | 1 337                        |                                  | 9.03E-04                           |
| DA/2     | 1.4-2.8*  | 12.0          |                              | 1 875                            | 7.9E-09                            |
| DA/9     | 2.2       | 13.0          | 1 637                        |                                  | 6.7E-09                            |

\*Remoulded sample



The test results show that the coefficient of permeability for both the remoulded and lower lying (>1.8m deep) in situ samples is very low with a relative permeability of “very impervious” in its recompacted state. The in situ undisturbed samples of the soil horizons occurring above 1.8m have a high coefficient of permeability with a relative permeability of “pervious” in its undisturbed state (JOHANN VD MERWE, 2011).

As a result of the high coefficient of permeability found in the undisturbed samples of the soil horizons occurring above 1.8m, the WWTW will be completely lined with HDPE to prevent any seepage during treatment.

### 2.1.5 Excavation Characteristics

From the tests done it can be said that no problems should be encountered during excavations of the transported and residual soils down to a depth of 3.0 metres below the surface when using conventional earth-moving machines. During the duration of the dry season sidewalls of deep excavations are expected to remain stable, however instabilities may occur during the rainy season.

### 2.1.6 Earthworks

Upper site soils were tested to determine their compaction characteristics. The test results are given below in the table below.

Table 5: Compaction tests (JOHANN VD MERWE, 2011).

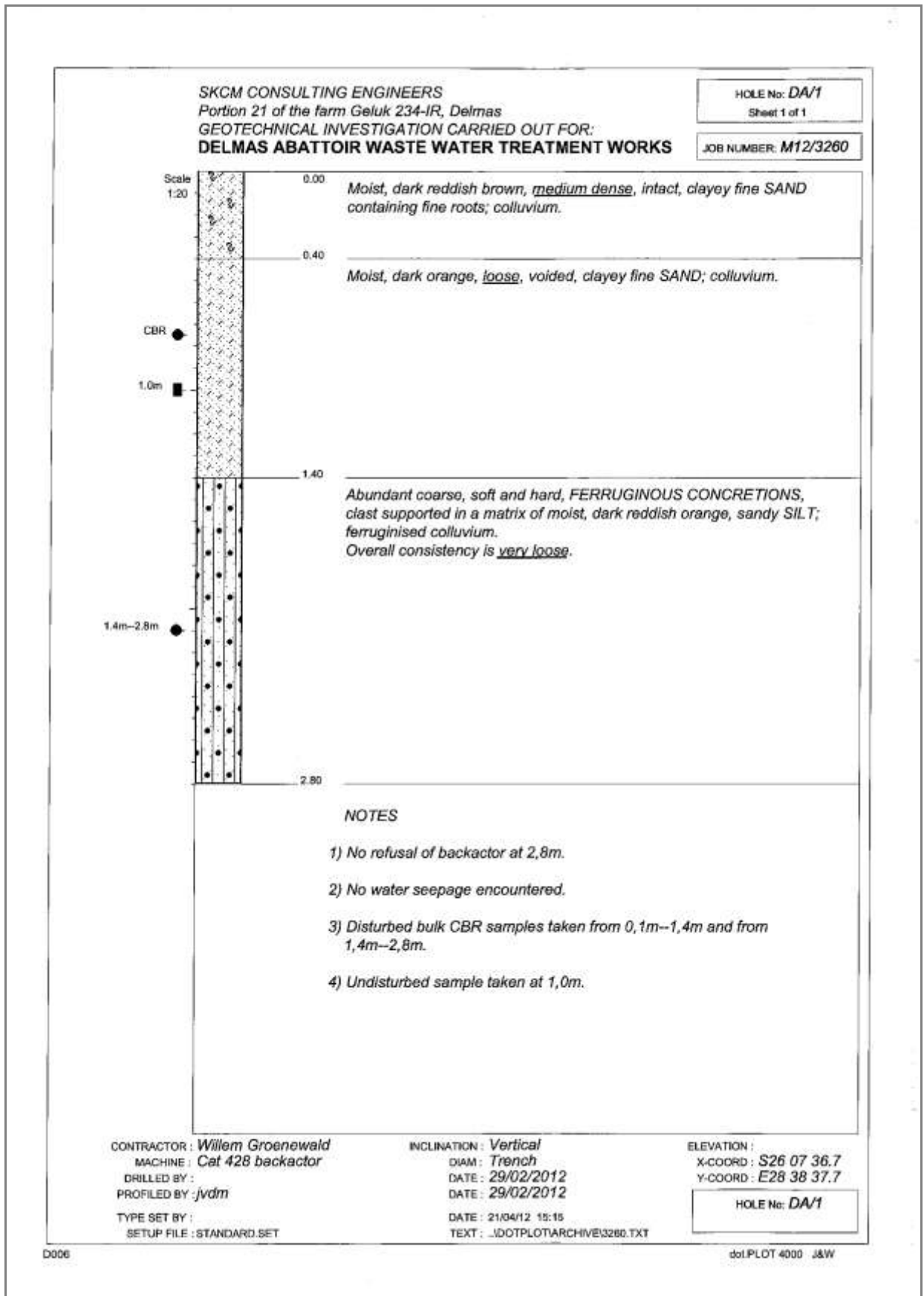
| Hole nr. | Depth (m) | Soil type     | PI | GM   | CBR  | TRH14 | SWELL |
|----------|-----------|---------------|----|------|------|-------|-------|
| DA/1     | 0.0-1.4   | Clayey SAND   | 8  | 0.59 | 12.1 | G9    | 0.75  |
| DA/1     | 1.4-2.8   | Gravelly SILT | 9  | 0.74 | 8.2  | G9    | -0.03 |

PI = Plasticity Index; GM = Grading Modulus; CBR = California Bearing Ration at 95% Mod AASHTO compaction

Based on the compaction tests, it is evident that the dark red and dark orange, clayey sand and gravelly silt have a fairly low compacted strength and a moderate to low predicted swell after compaction, respectively. Chemical stabilization or mechanical modification of these soils may improve the compacted strength thereof (JOHANN VD MERWE, 2011).

Please see Appendix D for more detail regarding the geotechnical study and in particular the tests performed.





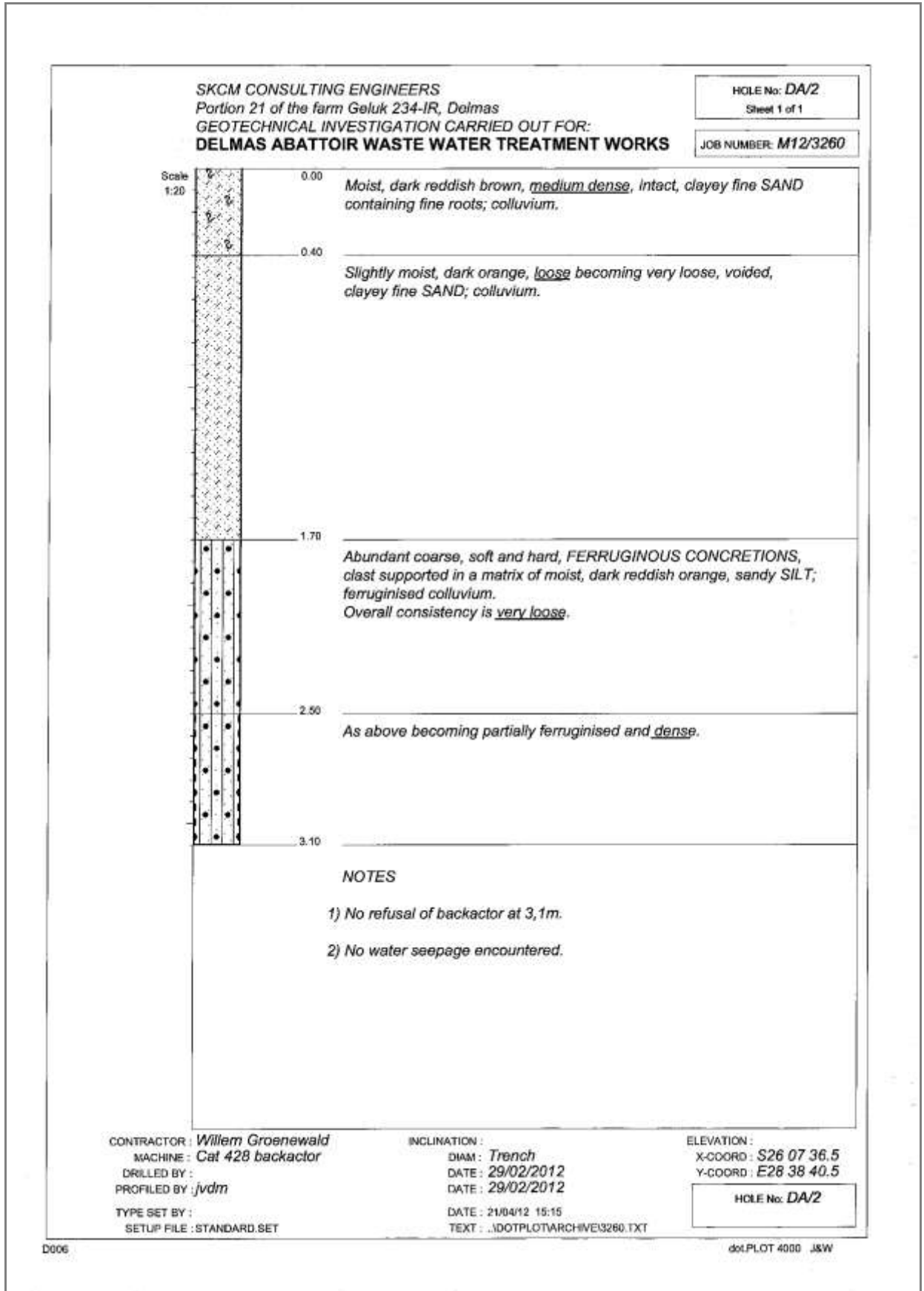


Figure 9: Soil profile for hole DA/2 (v.d. Merwe, J., 2010).

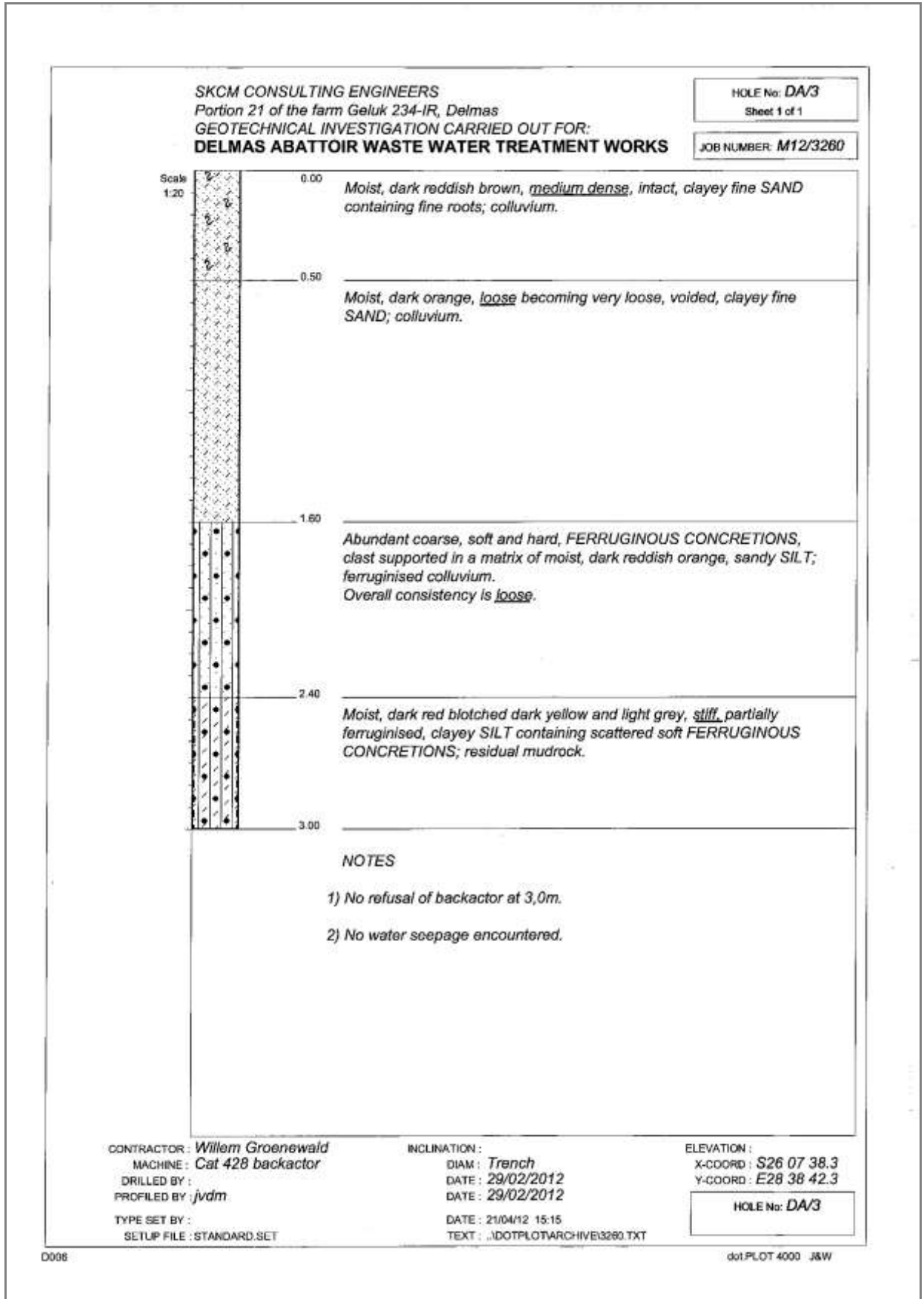


Figure 10: Soil profile for hole DA/3 (v.d. Merwe, J., 2010)

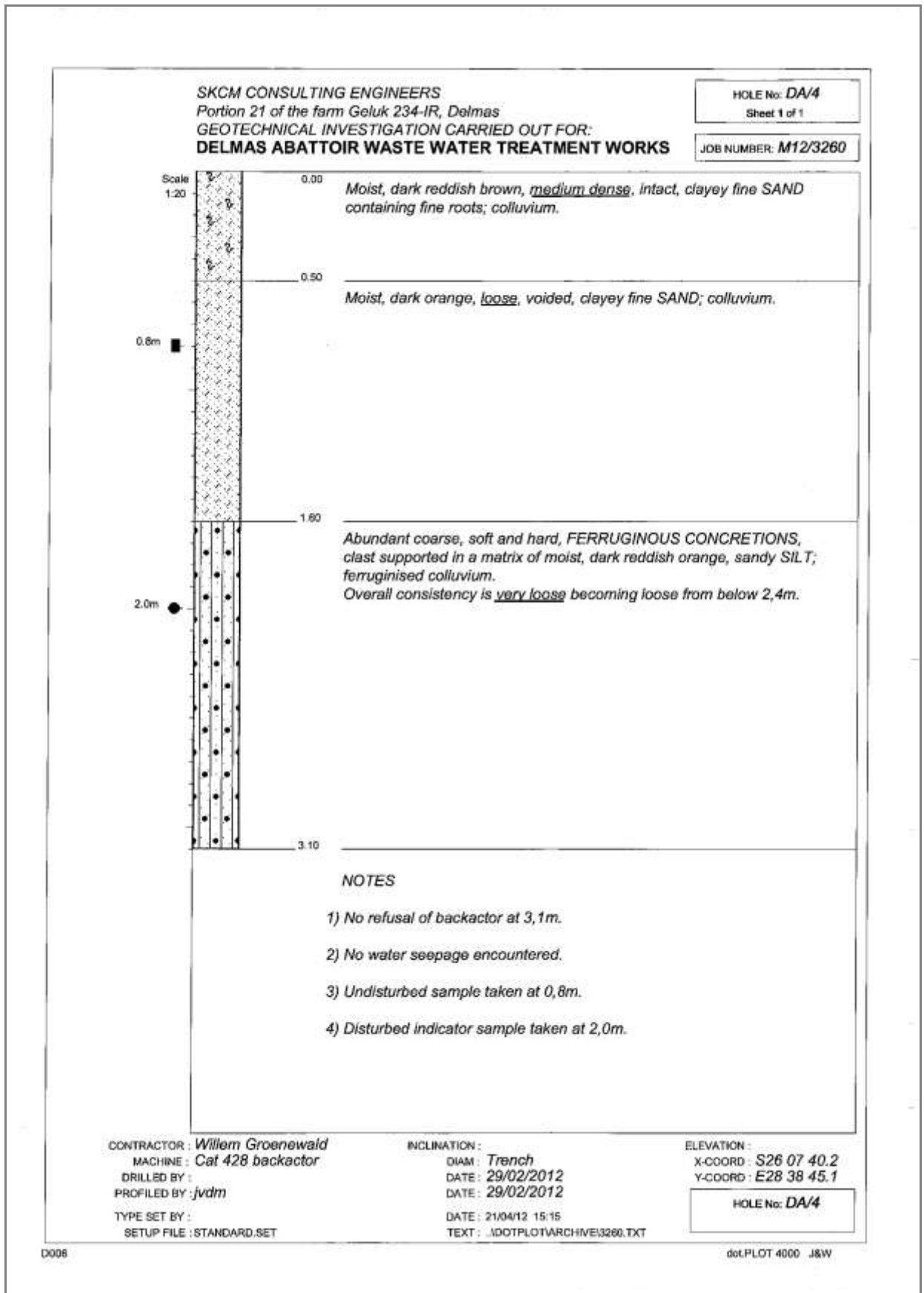


Figure 11: Soil profile for hole DA/4 (v.d. Merwe, J., 2010)

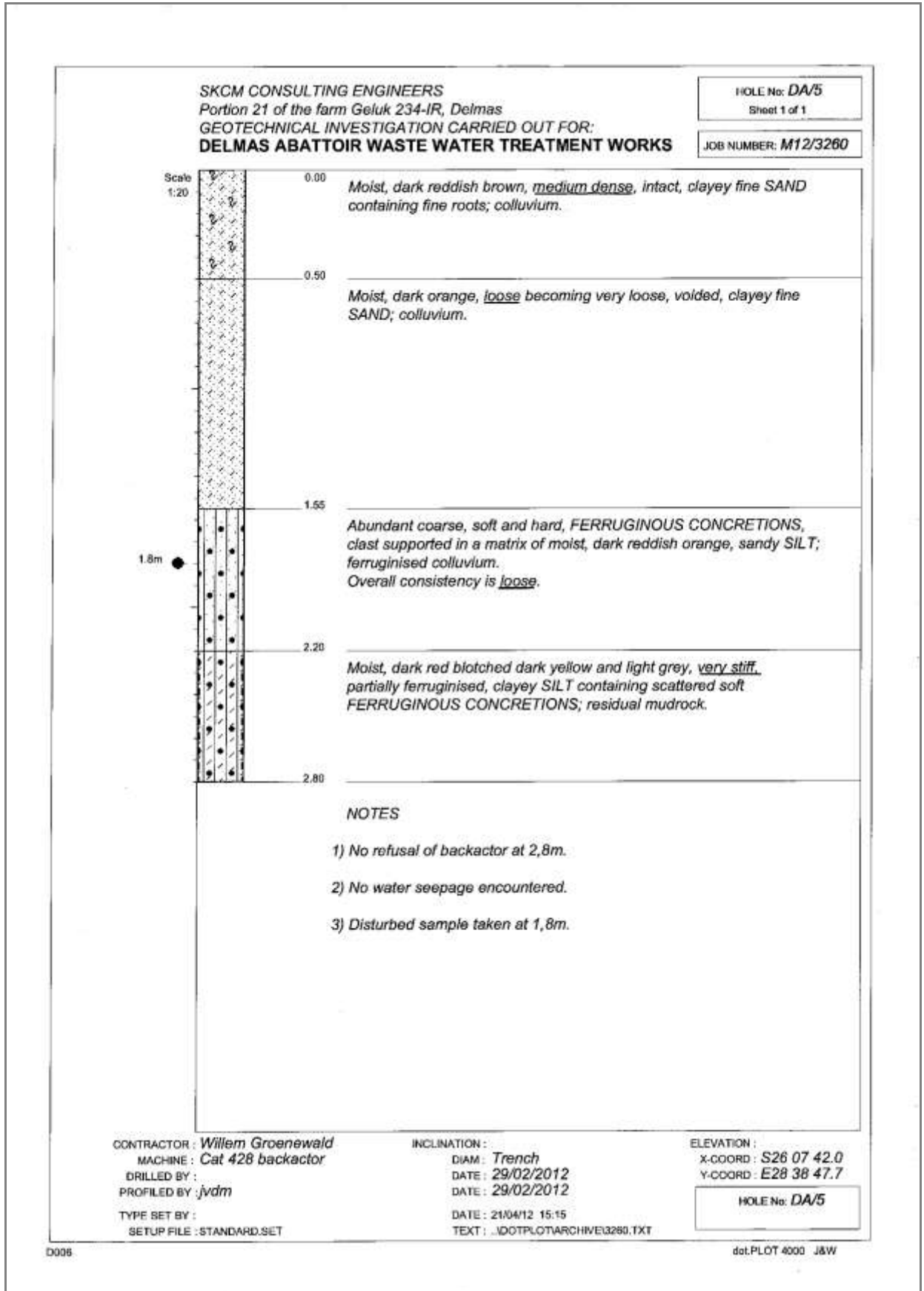


Figure 12: Soil profile for hole DA/5 (v.d. Merwe, J., 2010).



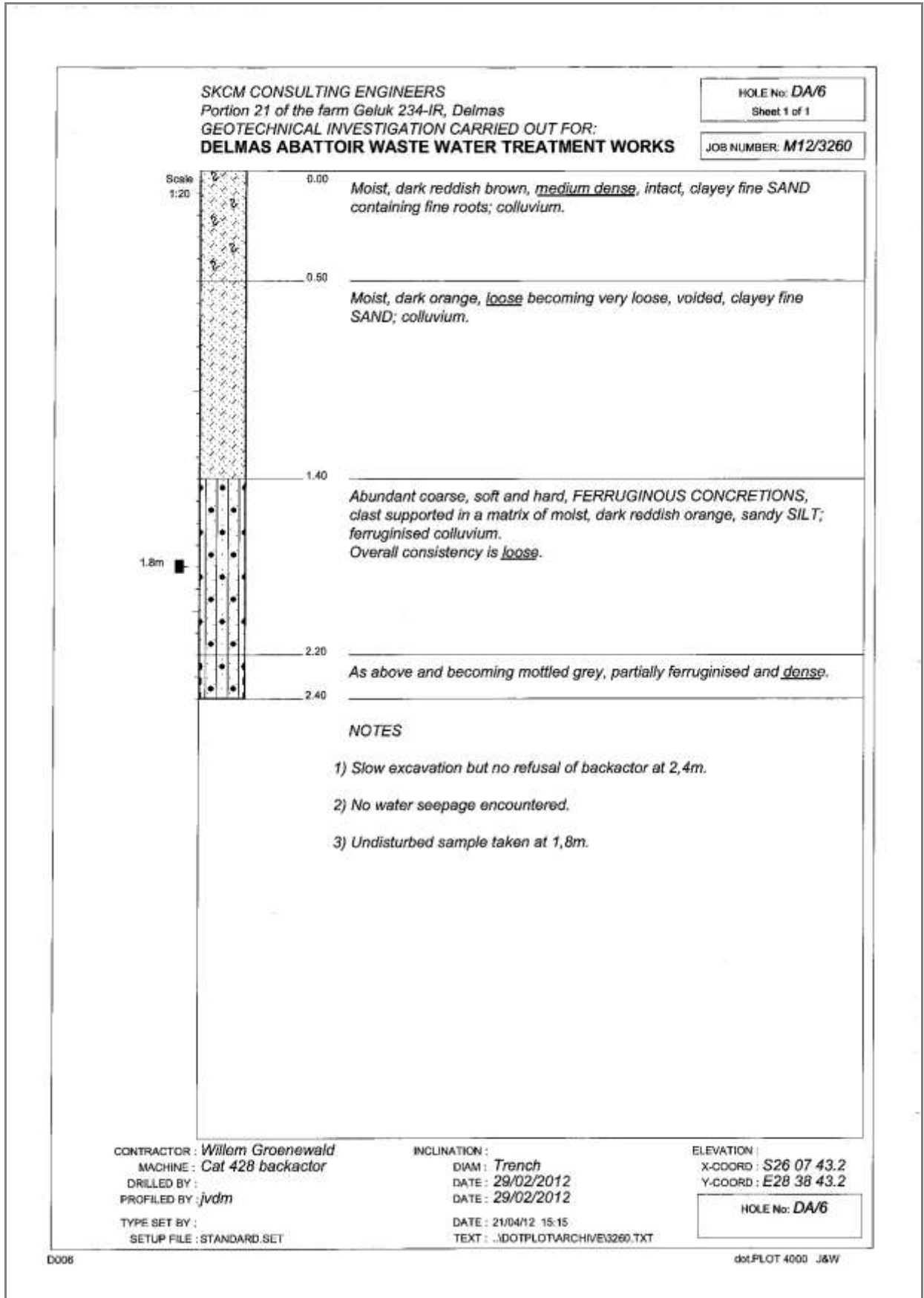


Figure 13: Soil profile for hole DA/6 (v.d. Merwe, J., 2010)



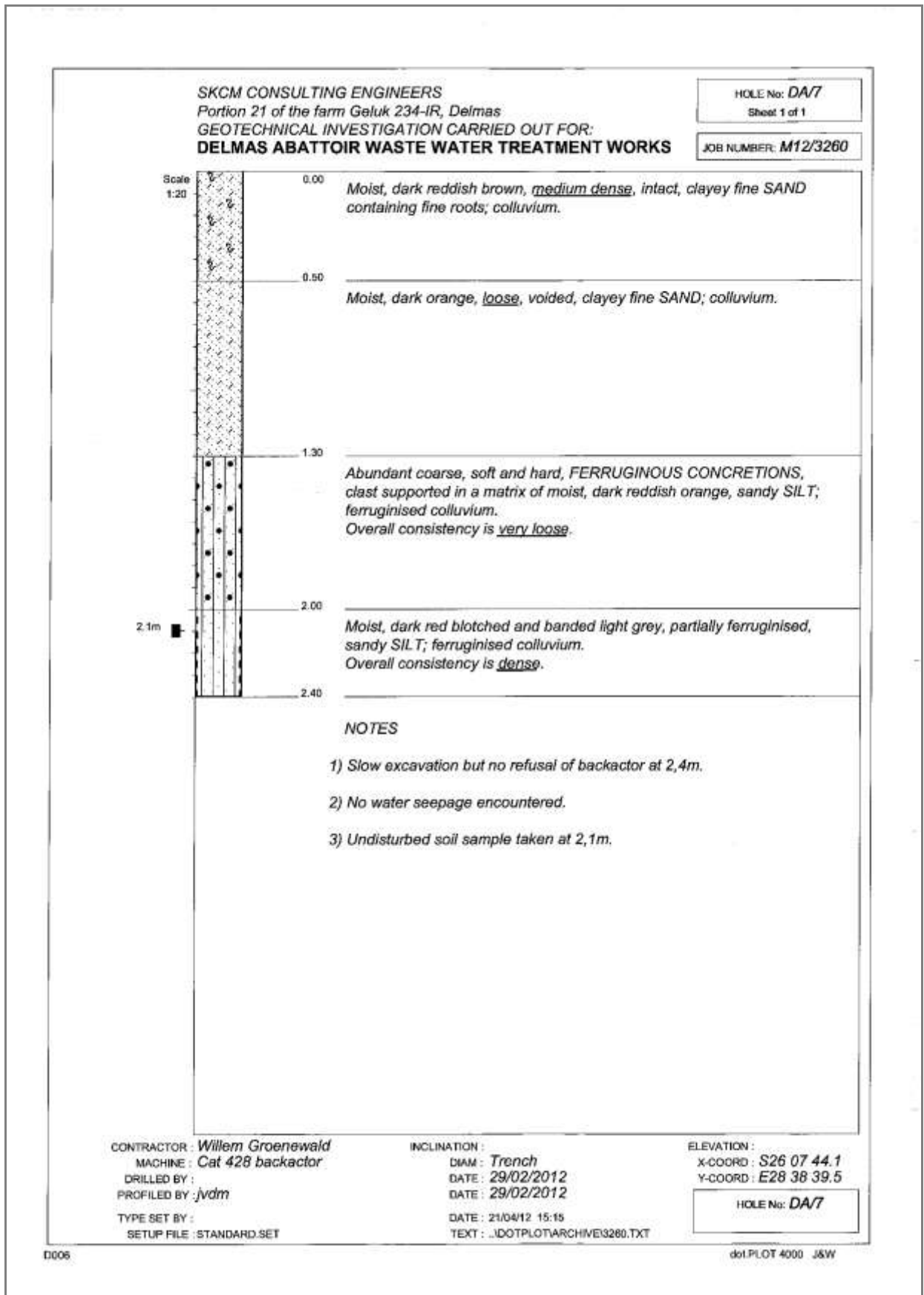


Figure 14: Soil profile for hole DA/7 (v.d. Merwe, J., 2010).



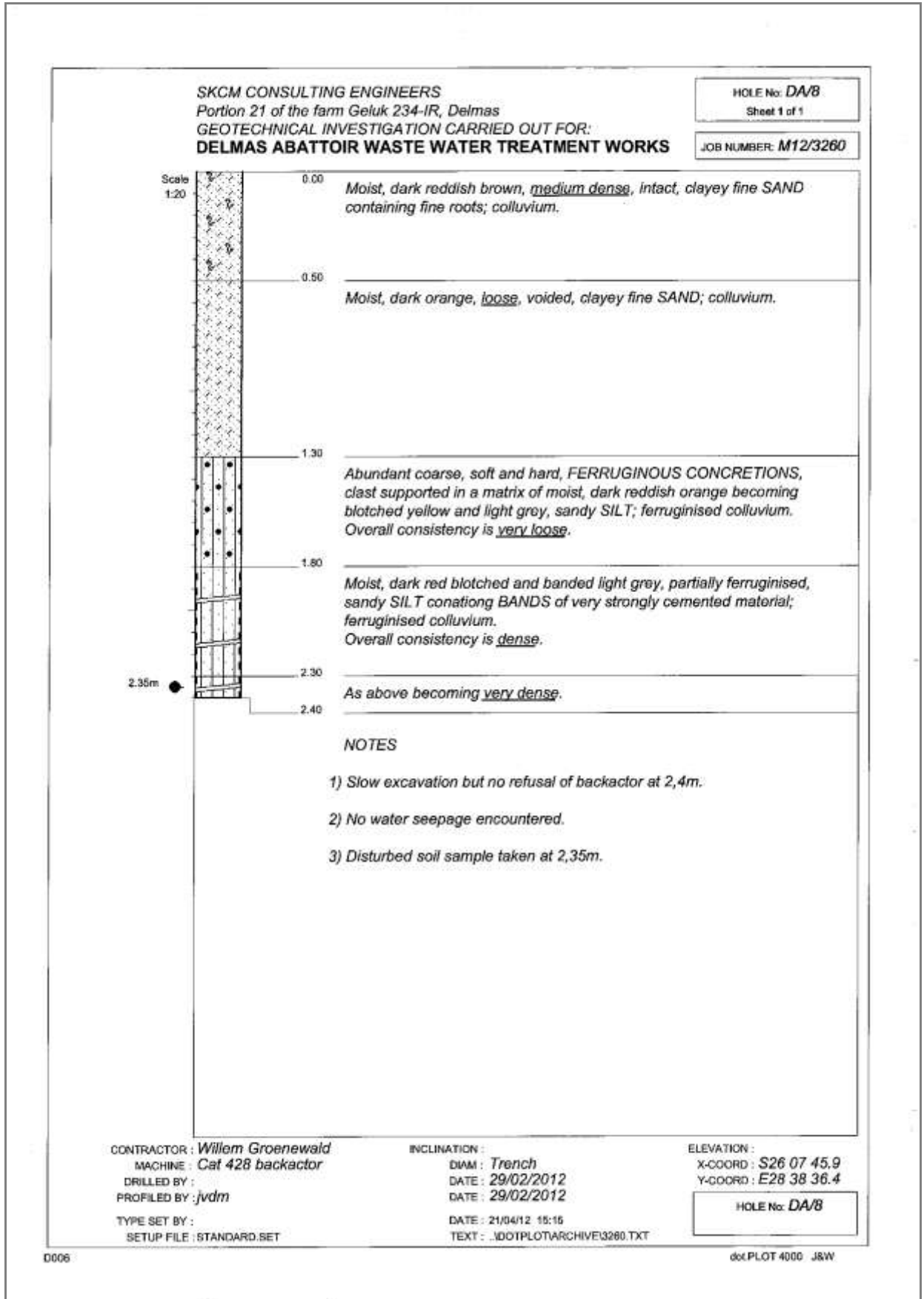


Figure 15: Soil profile for hole DA/8 (v.d. Merwe, J., 2010)

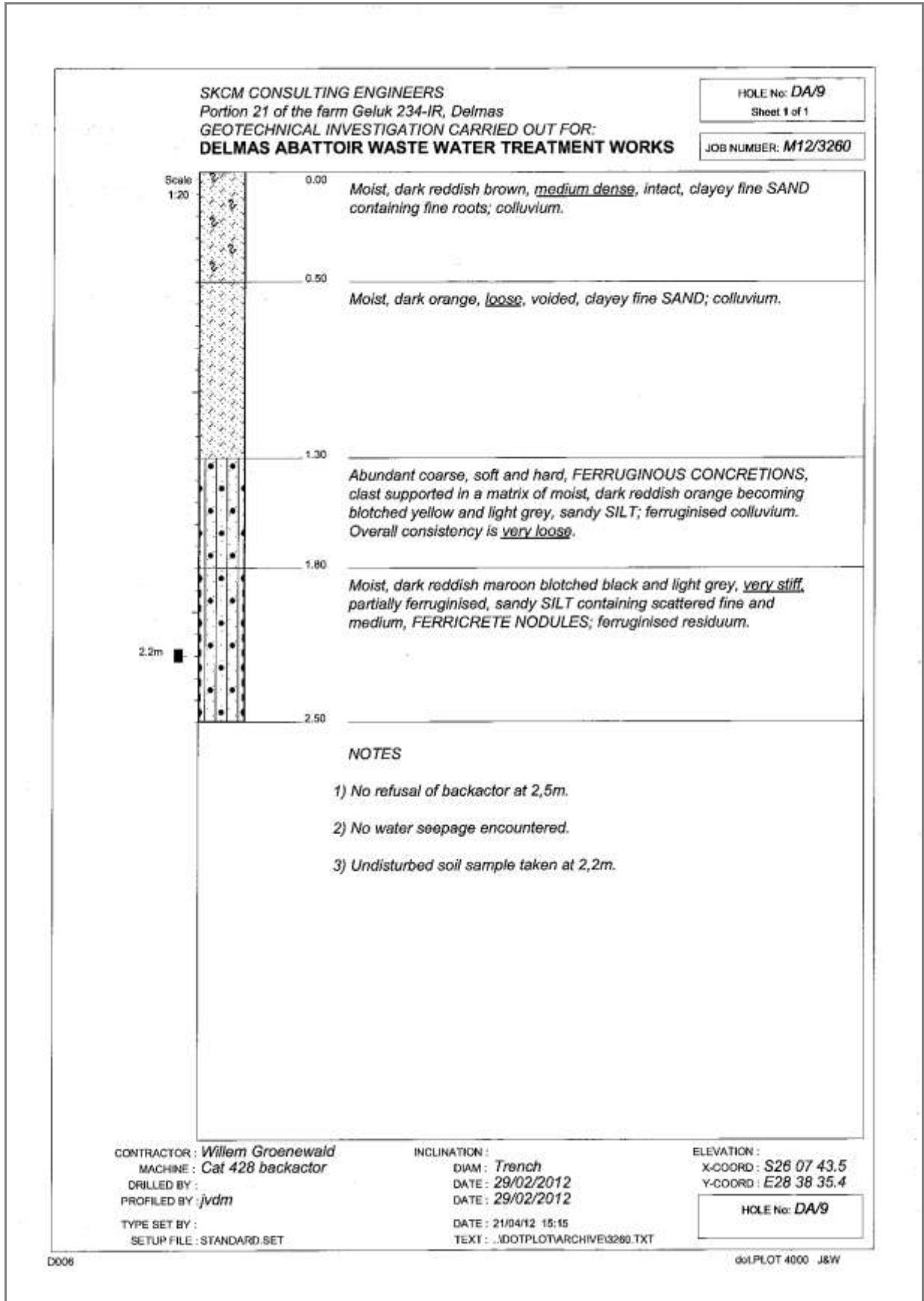


Figure 16: Soil profile for hole DA/9 (v.d. Merwe, J., 2010)



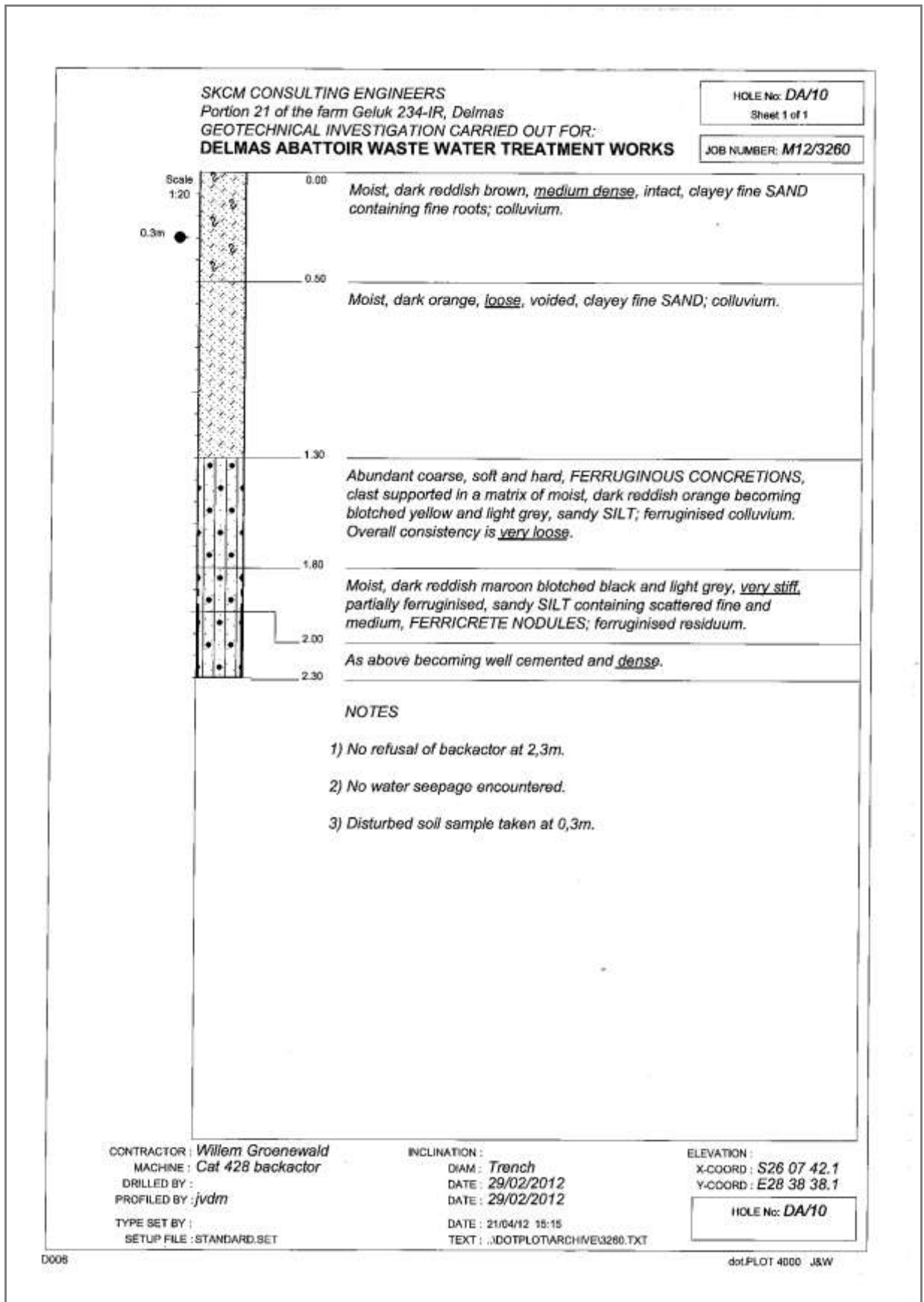


Figure 17: Soil profile for hole DA/10 (v.d. Merwe, J., 2010)

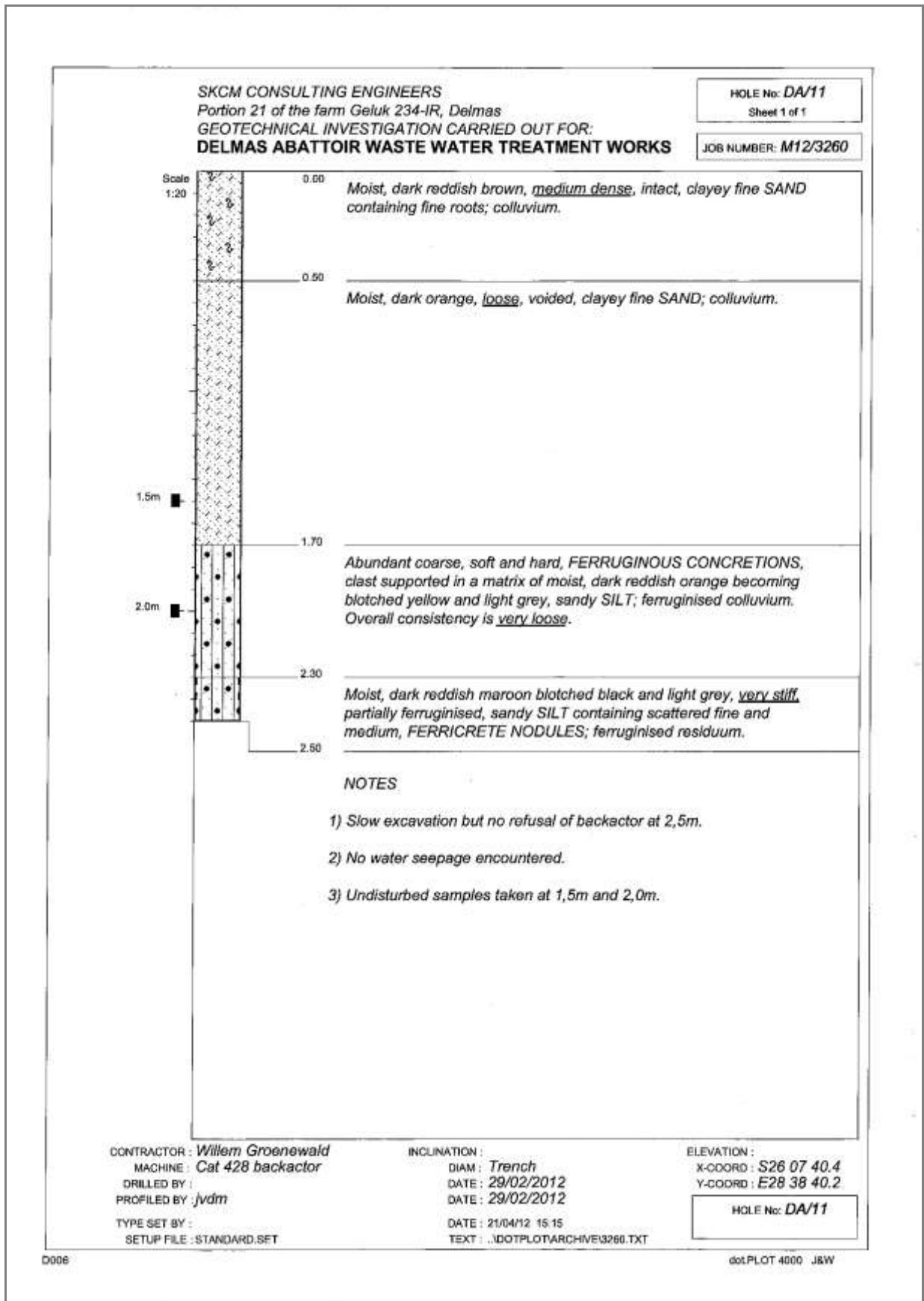


Figure 18: Soil profile for hole DA/11 (v.d. Merwe, J., 2010).

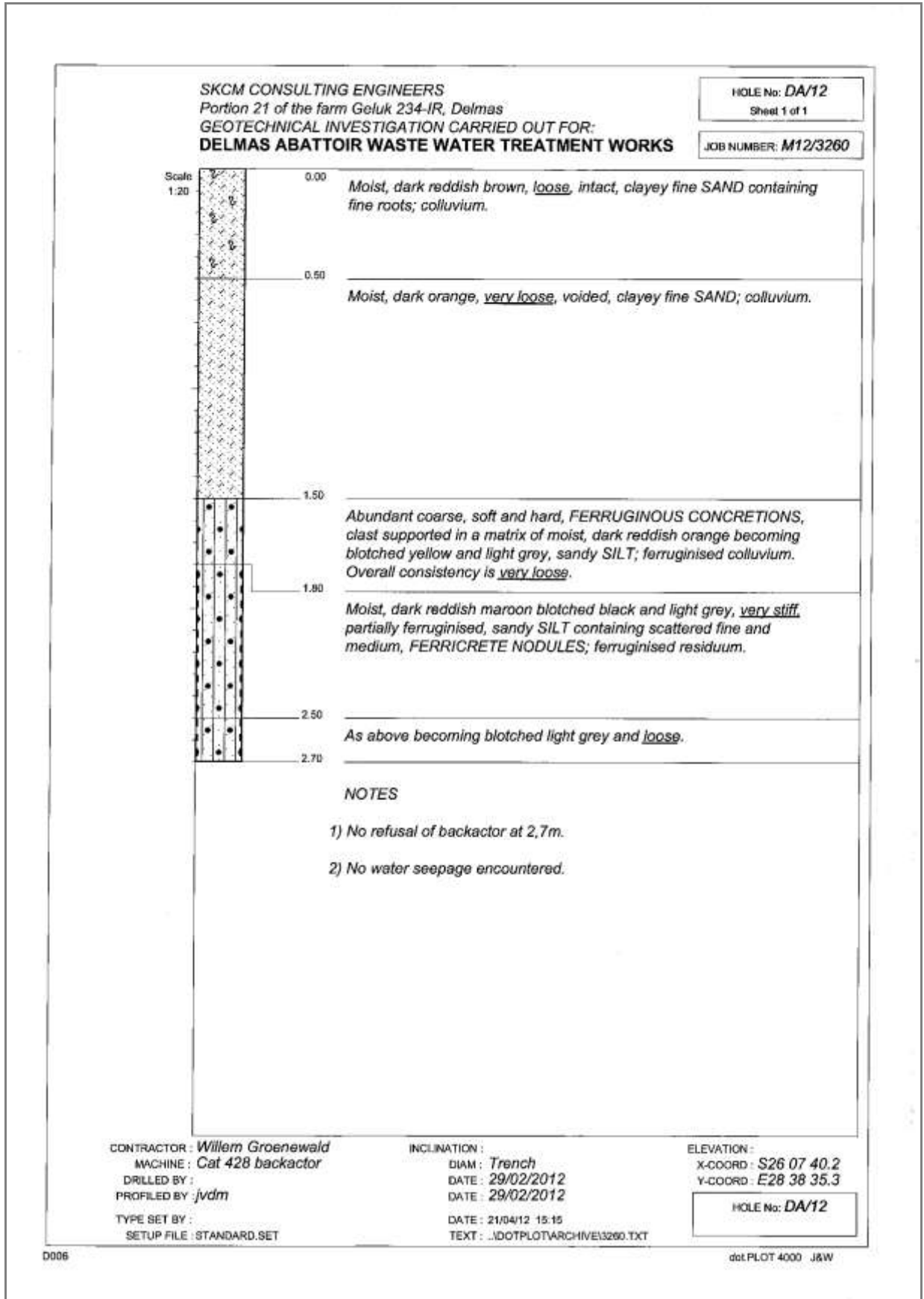


Figure 19: Soil profile for hole DA/12 (v.d. Merwe, J., 2010).

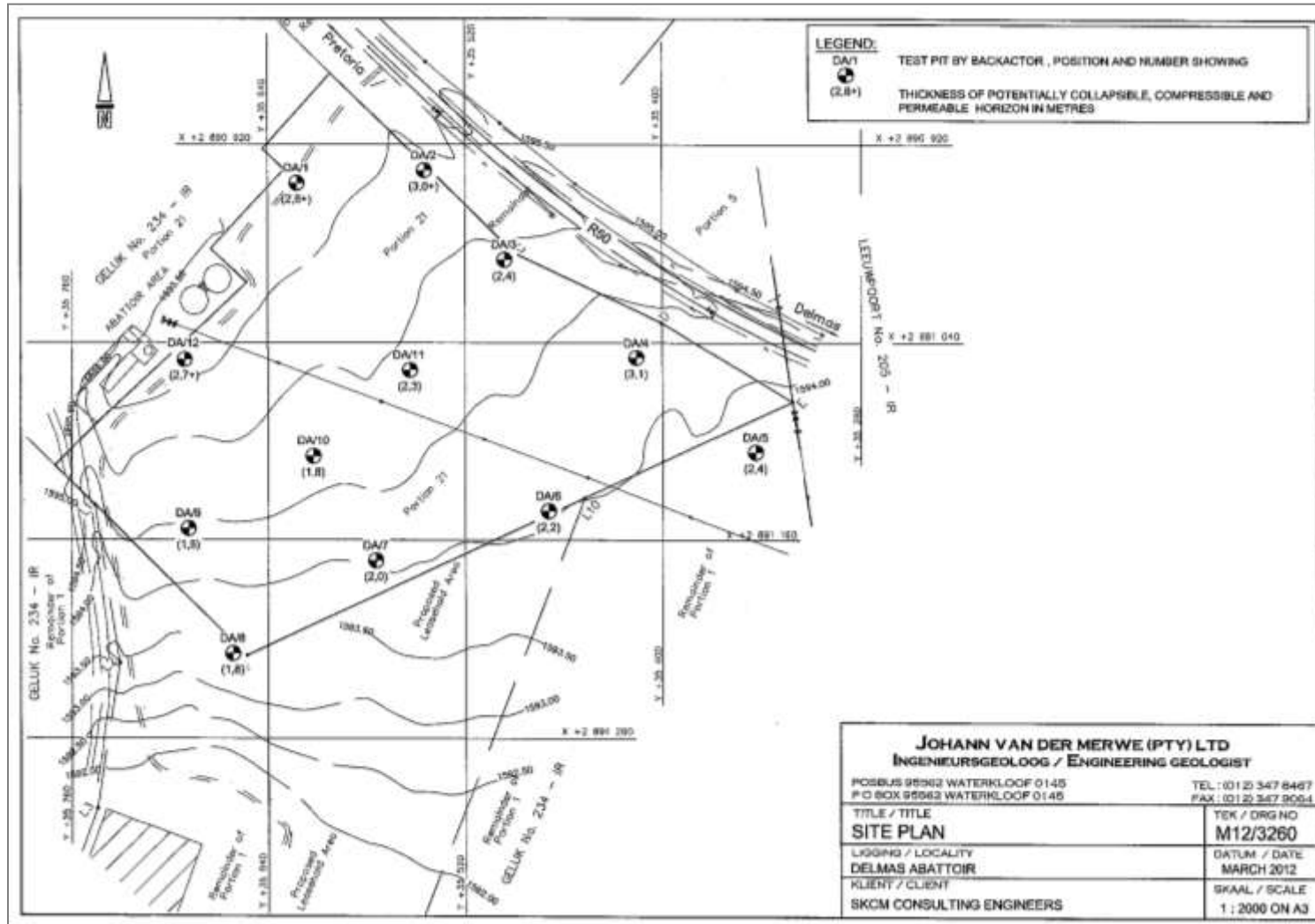


Figure 20: Location of test pits (Johann v.d. Merwe, 2010).





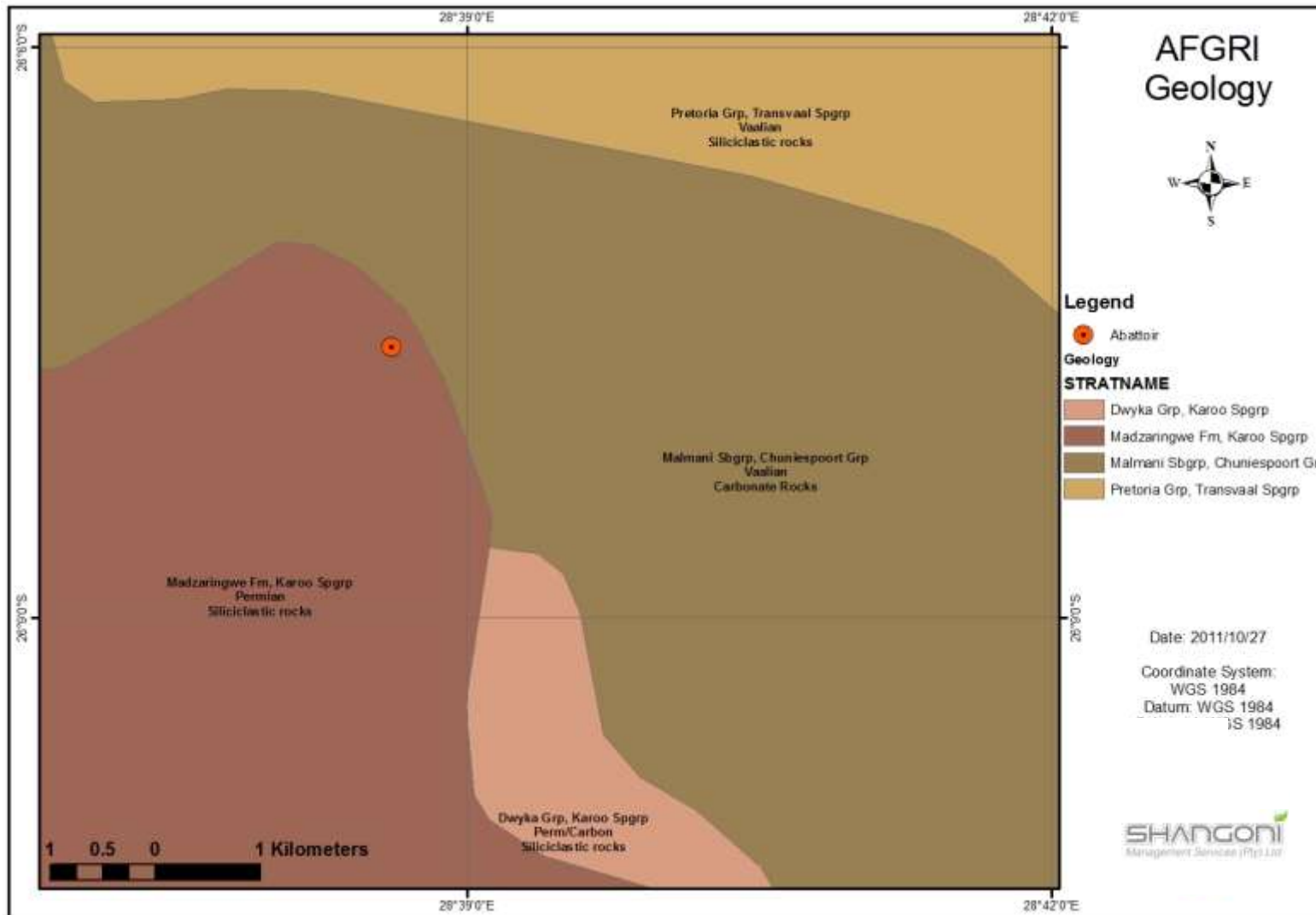


Figure 21: Geology of the site.

## 2.2 Regional climate

The climate of the site is typical of Highveld conditions with relatively warm to hot summers and fairly high rainfall, and moderate to cool winters with little or no rain. Valleys and wetlands are much cooler at night and more prone to frost than higher lying areas. The area experiences thunderstorms during the summer months, which usually occur in the late afternoons.

### 2.2.1 Rainfall

The site occurs in a summer rainfall area receiving a mean average annual rainfall of 426.3 mm. The average precipitation (taken over a period of 5 years from 2005 to 2009) is 127 mm in January and 6mm in July. The relative humidity varies between 12 % as minimum and 93 % as maximum.

The variability of rainfall as well as high intensity events will affect the construction phase of the project. It could hinder construction activities with potential soaking of cement mixtures or foundation concrete during the early phases of the construction process.

Construction should preferably be planned for the winter months to avoid construction delays that might have a negative socio-economic impact on the development.

The potential impact of the rainfall should be low if mitigated properly.

Figure 22 for the Average Monthly Rainfall of Delmas as provided by [www.weathersa.com](http://www.weathersa.com).

See Figure 23 for the Average Yearly Rainfall of Delmas as provided by [www.weathersa.com](http://www.weathersa.com).

See Figure 24 for the Mean Annual Precipitation for Mpumalanga.

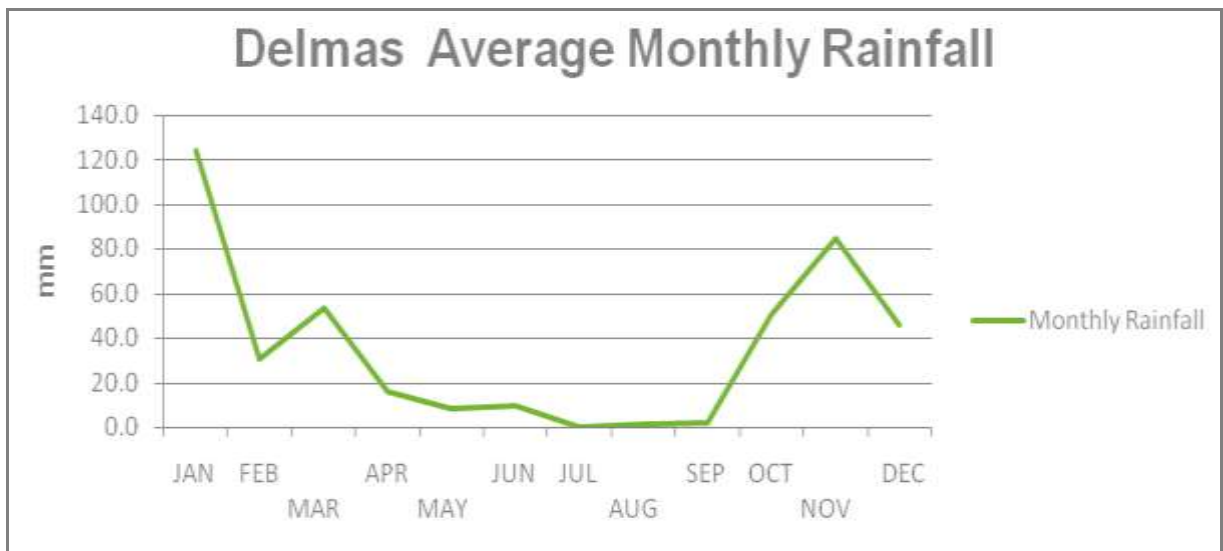


Figure 22: Delmas Average Monthly Rainfall.



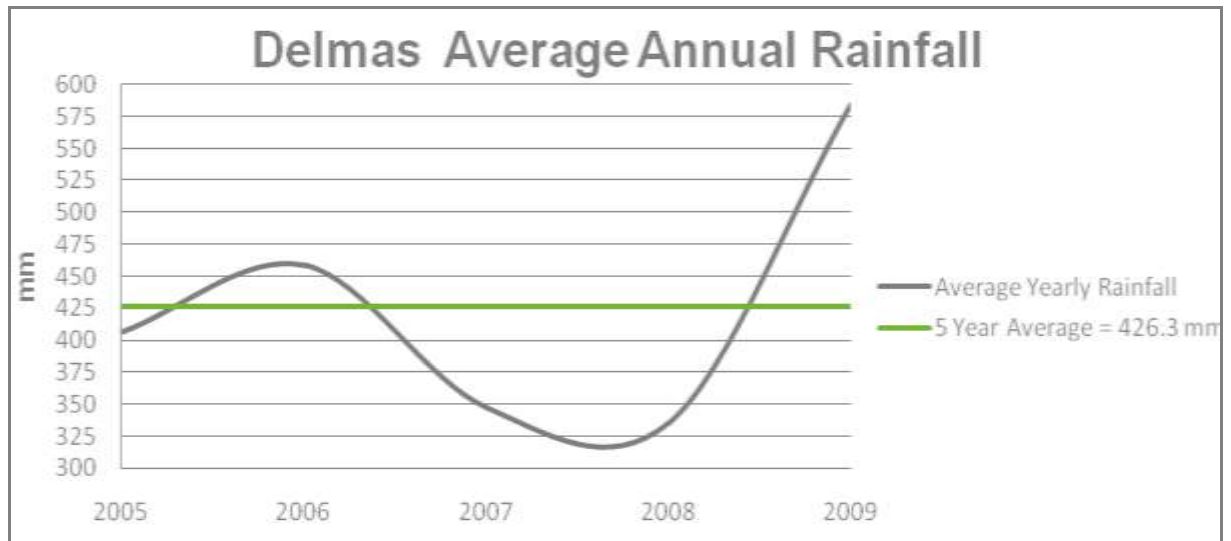


Figure 23: Delmas Average Annual Rainfall

