# FINAL INVESTIGATION REPORT BY SUBSTRUCTURE ANALYSIS AND SURROUNDINGS COMMITTEE

**12 MARCH 2008**

# PERAKUAN PENYERAHAN DOKUMEN LAPORAN

**OLEH**

**BADAN PENYIASATAN BEBAS (INDEPENDENT BOARD OF INQUIRY) KES KEJADIAN BANGUNAN RUNTUH (KOMPLEKS PELANCONGAN) DI PULAU BANDING, GERIK, PERAK DARUL RIDZUAN.**

Dengan ini saya memperakukan bahawa telah menyerahkan laporan seperti diatas kepada Pengerusi Badan Penyiasatan Bebas (Indepenent Board Of Inquiry) Kes Kejadian Bangunan Runtuh (Kompleks Pelancongan) Di Pulau Banding, Gerik, Perak Darul Ridzuan pada 12-03-2008.

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## REFERENCE

Gue SS & Chen CS (1998) A Comparision of Dynamic & Static Load Test on Reinforced Concrete Driven Piles Proceedings of the 13th Southeast Asian Geotechnical Conference 16th – 20th November 1998 – Taipei, Taiwan

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# FINAL INVESTIGATION REPORT
# BY
# SUBSTRUCTURE ANALYSIS AND SURROUNDINGS COMMITTEE

## 1.0 Introduction

The Committee on "Substructure Analysis and Surroundings" aims to complement the other two technical subcommittees in providing an independent professional investigation and to assist the independent board of inquiry to carry out its entrusted duty in accordance with the four stated terms of reference as established.

The investigation shall include findings based on site evidence and analysis to be tabled to the main committee.

## 2.0 Extent and Sequence of Failure

Based on the photograph captured on 24th October 2007 by JKR Perak (ref: Preliminary Report by Forensic & Structure Unit JKR Perak), slope failure at the edge and under the building was observed as shown in Plate 1A. Cracks were also observed at a few locations on the beams and walls of the building (Plate 1B).

On the 23rd Sept 2007 during the second site visit it was observed a few piles beneath the columns of the building were also exposed and deflected. The columns at this zone had also deflected towards the lake side as shown (Plate 1C).

The slope extended down into the lake where water level fluctuates from 240.03m to 244.33m based on water level records Tasek Temenggor by TNB Gerik without toe protection as shown in Table 1.1.

On 10$^{th}$ November 2007, part of the building collapsed followed by a total collapse of the whole building on 13$^{th}$ November 2007.

The scar of the failed slope is shown in Fig.1.1.

## 3.0 Design Records

The consultant confirmed (reference : interview on 4$^{th}$ February 2008) site reconnaissance study was not carried out. The consultant explained that the conceptual design was intended to be eco-friendly whereby the building was supposed to be built on natural ground without disturbing the existing site and soil condition.

There was only one(1) borehole being carried out during design stage and no undisturbed samples were ordered for the tests to obtain the necessary soil parameters for slope analysis.

No slope stability analysis and pile foundation design was carried out and these were confirmed by the design consultant during the interview.

## 4.0 Ground Surface Profile

The building is located on the eastern side slope of Pulau Banding comprising of both cut and fill slope.

On the west of this building, there is a cut slope (C1) outside the building collapse area comprising of 8 berms with about 1:1 slope (45%), as indicated in Figure 1.1. A slope failure (R1) was observed on the day of site visit (26th July 2007) and had blocked the inner roadside drain of the cut slope. This failure was also captured in the survey plan by Jurukur Abadi produced on 7th Jan 2004 (as shown in Figure 1.2). This figure also shows the existence of depression in southeast direction at failure location before site possession on 30th Ogos 2004.

The working platform (formation level) was designed to a profile by cutting the earth at the high area and filling the original ground at the lower level. Generally both the cut and fill slopes are designed to a gradient of 1v:1.5h to 1v:1h, as shown in Figures 1.3 and 1.4. The As-built Drawings also indicate gradients up to 1v:1h in both the cut and fill slopes, as shown in Figures 1.5 and 1.6.

The slope extended down into the lake where water level fluctuates to about 4 meters without toe protection in the area of fluctuation, and this was confirmed by the design consultant during the interview on the 4th. February 2008. A prudent design should have to provide toe protection such as rip-rap, etc.

## 5.0 Rainfall Records

Rainfall data before the event were obtained from Jabatan Pengairan dan Saliran (JPS) and Jabatan Meteorologi Malaysia (JMM). The JMM rainfall stations are located at Ayer Dala and at Kemar which are 15 km and 35 km away from the collapsed building respectively. JPS has 3 rainfall stations nearby, the nearest is located at Pulau Banding Station approximately 1.5 km from the building site, the others located 25 km and 29 km away from the site respectively. The rainfall station located 1.5 km away from the site is used for reference in view of its close proximity to the site.

Analysis of the rainfall pattern was carried out for the 30 and 7-day period prior to 13th November 2007. The rainfall of 355mm corresponding with 22 rainy days was recorded within the 30-day period prior to the total collapse of the building. Figure 1.7 shows the 30-day rainfall pattern. A total of 138mm rainfall was recorded during the 7-day period prior to the event. The total rainfall of 355mm at Pulau Banding Station 30 days prior to the collapse is normal and the return period is less than 1 in 5 years.

## 6.0 Geology and Geomorphology

The site is located in a geologically region known as Kruh Formation. It is made up of meta-tuff that was formed during the Silurian and Devonian period. The mineral composition of fresh rocks consists mainly of silica with some mica and quartz lenses in the between the foliation measuring from several milliliters to several centimeters. The residual soils is of gravelly SILT (Plate 1D) and the soil investigation confirms the observation. However there is no geological fault found within the building collapse area.

## 7.0 As- built Drainage Facilities

The as-built drainage system as shown in Figure 1.5 was such that, the surface runoff from the upslope would be intercepted by concrete drain and discharged away from the building site. The water from down pipes of the buildings would be collected by concrete drain of 450mm and 600mm width and discharged into the lake by cascaded drain.

JKR Perak reported that the condition of some of drainage facilities before the building collapse occurred appeared to be heavily silted and have settled following the slope instability movements (Plate 1C). The discharge points of the rainwater down pipes had also been silted up or damaged locally and drainage facilities were completely damaged particularly at the failure location at the down slope (Plate 1E & 1F).

## 8.0 Subsurface Investigation

### 8.1 Soil Investigation

The proposed soil investigation program consisted of:

- Four (4) numbers exploratory boreholes,
- Thirty two (32) numbers JKR probe tests,
- Standard Penetration Tests (SPT) at every 1.0m interval,
- Disturbed and undisturbed samplings,
- Four number standpipes (one in each borehole), and
- A series of laboratory tests.

The soil investigation was carried out by Kumpulan IKRAM Sdn Bhd between 28$^{th}$ December 2007 till 13$^{th}$ January 2008. The soil investigation results are presented in the factual report prepared by Kumpulan IKRAM Sdn Bhd.

The locations of boreholes, standpipes and JKR probe tests are shown in Figure 1.8

## 8.2 Sub-Surface Profile from Boreholes

Four number exploratory boreholes, referred to as BH1, BH3, BH5 and BH6 were drilled. Figure 1.9 shows the simplified engineering borehole logs derived from the soil classifications based on the grain size distribution (sieve analysis) tests and Atterberg limits tests on the disturbed and undisturbed samples obtained from the boreholes. The soil classifications were performed according to the British Soil Classification System in BS 5930:1999 (Code of Practice for Soil Investigations).

In general, the subsoils can be classified into three layers based on soil type and Standard Penetration Test (SPT-N) value, as follows:

(a) Subsoil 1 generally consists of soft to very stiff gravelly SILTS of SPT-N values between 2 and 21.

(b) Subsoil 2 generally consists of medium dense to dense silty GRAVEL of SPT-N values between 18 and 35.

(c) Subsoil 3 generally consists of very dense silty GRAVEL of SPT-N values greater than 50.

Meta-tuff bedrock was encountered in all the four boreholes. The depths of meta-tuff bedrock vary from 4.0m to 15.0m, as summarized in Table 1.2.

**Table 1.2: Depth of Meta-tuff Bedrock**

| Borehole | Reduced Level of Borehole (RL m) | Depth to Meta-tuff Bedrock (m) | Reduced Level of Meta-tuff Bedrock (RL m) |
|---|---|---|---|
| BH3 | 271.582 | 11.70 | 259.882 |
| BH6 | 268.480 | 15.00 | 253.480 |
| BH5 | 261.096 | 5.50 | 255.596 |
| BH1 | 249.735 | 4.00 | 245.735 |

In general, the top 1.5m of meta-tuff bedrock is graded as Grade III (moderately weathered), and the subsequent 3.0m is graded as Grade II (slightly weathered).

### 8.3 Groundwater

The groundwater table at the site was monitored from four standpipes since 1st January 2008 to 20th February 2008, as shown in Figure 1.10 The depths of groundwater measured in standpipes vary from 0 to 11.50m.

## 9.0 Soil Properties

### 9.1 Bulk Density

Table 1.3 presents the bulk density values obtained from the laboratory tests on the soil samples. The bulk density values range between 1.524 and 1.926Mg/m$^3$ with an average value of 1.816Mg/m$^3$, or equivalent to an average bulk unit weight of 17.8kN/m$^3$.

**Table 1.3: Bulk Density of Soil**

| Borehole | Sample | Depth (m) | Soil Type | SPT-N Values above and below Undisturbed Sample | Bulk Density (Mg/m$^3$) | Remark |
|---|---|---|---|---|---|---|
| BH3 | MZ1 | 1.00 –2.00 | Very clayey GRAVEL | Nil, 3 | 1.847 | From UCT |
| BH5 | MZ1 | 1.00 – 2.00 | Sandy SILT | Nil, Nil | 1.839 | From UCT |
| | | | | | 1.868 | From CIU |
| | | | | | 1.845 | From CIU |
| | | | | | 1.821 | From CIU |
| | | | | | 1.838 | From DSB |
| BH6 | MZ3 | 6.00 – 7.00 | Slightly gravelly SILT | 6, 18 | 1.526 | From UCT |
| | | | | | 1.524 | From DSB |
| | MZ4 | 8.00 – 9.00 | Sandy SILT | 18, 50 | 1.833 | From CIU |
| | | | | | 1.821 | From CIU |
| | | | | | 1.798 | From CIU |
| | | | | | 1.848 | From DSB |
| | MZ5 | 12.00 – 13.00 | Gravelly SILT | 12, 27 | 1.926 | From CIU |
| | | | | | 1.903 | From CIU |
| | | | | | 1.891 | From CIU |
| | | | | | 1.923 | From DSB |
| Average | | | | 1.816Mg/m$^3$ or 17.8kN/m$^3$ | | |

Note: UCT denotes unconfined compression test, CIU denotes isotropically consolidated undrained triaxial tests, and DSB denotes direct shear box test.

## 9.2 Effective Shear Strength from Triaxial Tests

Only the results from isotropically consolidated undrained (CIU) triaxial tests were considered in determining the shear strength parameters. Results from shear box tests were not included because the preparation of the square shaped sample of 60 mm by 60mm ( which have diagonal of 85 mm) from the Mazier samples of 75mm diameter are doubtful and so are the results obtained.

The three numbers of CIU samples obtained from the exploratory boreholes are summarized in Table 1.4.

**Table 1.4: Results of CIU Triaxial Tests**

| Borehole | Sample | Depth (m) | Soil Type | SPT-N Values above and below Undisturbed Sample | Effective Cohesion, C' (kPa) | Effective Angle of Friction, $\phi'$ (°) |
|---|---|---|---|---|---|---|
| BH5 | MZ1 | 1.00 – 2.00 | Sandy SILT | Nil, Nil | 6.0 | 28.0 |
| BH6 | MZ4 | 8.00 – 9.00 | Sandy SILT | 18, 50 | 15.0 | 26.0 |
| | MZ5 | 12.00 – 13.00 | Gravelly SILT | 12, 27 | 8.0 | 28.0 |

The average value of effective shear strength can be obtained by plotting the stress invariant $s' = (\sigma'_1+\sigma'_3)/2$ versus stress invariant $t'= (\sigma'_1-\sigma'_3)/2$ for all the three samples, as shown in Figure 1.11. The average effective cohesion (C') is 9.7kPa and the average effective angles of friction ($\phi'$) is 27.0°.

## 10.0 Analysis of Results

### 10.1 Slope Stability

#### 10.1.1 Cross-sections and Locations

The following two cross-sections are used for slope stability analyses:

(a) Cross-section CH. 120 from the As-built Survey Drawing dated 27 April 2006 prepared by Jurukur Rakyat. The As-built Survey Drawing was reproduced as Figure 1.5 and the cross-section was reproduced as Figure 1.6.

(b) Cross-section F-F from the latest Survey Plan (after the collapse of the building) dated 15 January 2008 prepared by Abdullah Taha & Rakan-Rakan. The Survey Plan was reproduced as Figure 1.8 in Section 8.0 of this report, and the cross-section was reproduced as Figure 1.12.

### 10.1.2 Methodology and Assumptions

Analysis of slope stability were carried out using the limit equilibrium method.

For each cross-section, stability analyses were performed for the following two sets of effective shear strengths of soil:

(a) **Case 1:** Average effective shear strength obtained from the CIU triaxial tests from mazier samples obtained from the site after the failure, as shown in Table 1.5.

**Table 1.5: Average effective shear strength from CIU triaxial tests**

| Soil Type | Effective Cohesion, C' (kPa) | Effective Angle of Friction, ϕ' (°) |
|---|---|---|
| **Subsoil 1** (Soft to very stiff gravelly SILTS of SPT-N values between 2 and 21) | 9.7 | 27.0 |
| **Subsoil 2** (Medium dense to dense silty GRAVEL of SPT-N values between 18 and 35) | 1.0 | 38.0 |

(b) **Case 2:** Soil strengths obtained from a previous project of the investigator in the same island, as shown in Table 1.6.

**Table 1.6: Soil strength from a previous project of the investigator in the same island**

| Soil Type | Effective Cohesion, C' (kPa) | Effective Angle of Friction, ϕ' (°) |
|---|---|---|
| Subsoil 1 | 3.0 | 32.0 |
| Subsoil 2 | 1.0 | 38.0 |

The bulk unit weight of subsoil adopted in the analyses was assumed to be $17.8kN/m^3$ (see Section 9.1 of this report).

The lowest groundwater levels (see Section 8.3 of this report) were used in the analyses. It should be noted that groundwater can rise higher than the adopted, particularly during the monsoon seasons. When the groundwater level rises, it gives negative impact to the stability of slope.

The water level in the lake was assumed to be at RL 244.33m , which is also the flood level recorded on 1st September 2007. Again, the lake water level can rise higher than the adopted, particularly during the monsoon seasons. When the groundwater level rises, it gives negative impact to the stability of slope.

No surcharges and no seepage forces due to transient flow of water were considered in the analyses.

The minimum factor of safety that is acceptable is 1.4 as recommended in Geotechnical Manual for Slopes published by the Geotechnical Engineering Office of Hong Kong.

### 10.1.3 Stability Analysis – Global and Localised Failures

The computed factors of safety (FOS) for all slope stability analyses performed are presented in Table 1.7.

**Table 1.7: Results of Slope Stability Analyses**

| Figure No. | Description | Factor of Safety | Comments |
|---|---|---|---|
| 1.13(a) | ▪ Cross-section CH. 120, Case 1<br>▪ Subsoil 1: C' = 9.7kPa, $\phi'$ = 27.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check for upper slope | 1.34 | ▪ Potential deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Unacceptable** |
| 1.13(b) | ▪ Cross-section CH. 120, Case 1<br>▪ Subsoil 1: C' = 9.7kPa, $\phi'$ = 27.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check 1 for lower slope | 0.74 | ▪ Shallow slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |
| 1.13 (c) | ▪ Cross-section CH. 120, Case 1<br>▪ Subsoil 1: C' = 9.7kPa, $\phi'$ = 27.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check 2 for lower slope | 0.99 | ▪ Relatively deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |
| 1.13(d) | ▪ Cross-section CH. 120, Case2<br>▪ Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check for upper slope | 1.15 | ▪ Potential relatively deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Unacceptable** |
| 1.13(e) | ▪ Cross-section CH. 120, Case2<br>▪ Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check 1 for lower slope | 0.72 | ▪ Shallow slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |
| 1.13(f) | ▪ Cross-section CH. 120, Case2<br>▪ Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0°<br>▪ Stability check 2 for lower slope | 0.99 | ▪ Deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |
| 1.13(g) | ▪ Cross-section F-F, Case 1<br>▪ Subsoil 1: C' = 9.7kPa, $\phi'$ = 27.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0° | 1.16 | ▪ Potential deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Unacceptable** |
| 1.13(h) | ▪ Cross-section F-F, Case 2<br>▪ Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0° | 1.12 | ▪ Potential deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Unacceptable** |
| 1.13(i) | ▪ <u>If groundwater level rises for 1.0m</u><br>▪ Cross-section F-F, Case 1<br>▪ Subsoil 1: C' = 9.7kPa, $\phi'$ = 27.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0° | 0.97 | ▪ Deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |
| 1.13(j) | ▪ <u>If groundwater level rises for 1.0m</u><br>▪ Cross-section F-F, Case 2<br>▪ Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°<br>▪ Subsoil 2: C' = 3.0kPa, $\phi'$ = 36.0° | 0.91 | ▪ Relatively deep slip failure<br>▪ Inadequate FOS (**<1.4**)<br>▪ **Failure Expected** (**<1.0**) |

In the case of Cross-section CH. 120, the computed factors of safety are less than unity, hence failures are expected. As for Section F-F, the computed factors of safety are about 1.1, which are unacceptable because the required is 1.4. If the groundwater table (as well as the lake water level) in Section F-F rises for 1.0m during the monsoon season, the factors of safety reduces from about 1.1 to about 0.9, and failure would be expected. The lake water level during the SI carried out by IKRAM on 29th December 2007 was 247.234 meters.

Results of the slope stability analyses also show that the two sets of soil strength parameters (Case 1 and Case 2) do not make significant difference on the computed factor of safety.

## 10.2 Pile Foundation

### 10.2.1 Building Foundations on Slope

The adopted foundation system for the building founded on slope consisted of 250mm x 250mm reinforced concrete driven piles. The total pile point was 209 but only 192 number pile driving records were made available for this investigation.

The pile lengths are summarized and plotted in Figure 1.14. Statistical analyses on pile length show that

(a) Minimum pile length = 2.7m

(b) Maximum pile length = 22.2m

(c) Average length = 8.7m

In average, the pile lengths were shorter than 9.0m and the minimum pile length was merely 2.7m. Out of the 192 piles, a total of 105 piles (about 54%) have lengths of not more than 9m. Short piles are comparatively weak in lateral resistance because of the low confining stresses resulted from the subsoil.

### 10.2.2 Pile Bearing Capacity Calculations

It is desired to assess the available pile bearing capacity particularly for the short driven piles. The following information was made available:

(a) The adopted working load for 250mmx250mm Grade 45 reinforced concrete driven piles was about 75 Ton or 750kN.

(b) The proposed hammer weight used for pile driving was 2.5 Ton and the hammer drop height was 750mm.

(c) The pile set adopted for pile driving termination was 20mm per 10 blows.

In general, pile termination will be achieved when driving concrete piles in hard or very dense soils of SPT-N about 50, depending on the overburden soils and length of the pile. For smaller piles, the SPT-N value at termination can be much lower. It is expected in the pile capacity calculation that the bearing soil stratum has SPT-N values of 30 - 50.

Table 1.8 summarizes the expected ultimate bearing capacities for different pile lengths. The factors of safety over the adopted working load of 750kN range from 1.22 to 3.44, compared to the commonly adopted factor of safety of 2 or higher. Note that BS 8004:1986 (Code of Practice for Foundations) recommends a factor of safety of between 2.0 and 3.0 for a single pile. Details of pile capacity calculations are given in Table 1.9.

**Table 1.8: Estimated Ultimate Bearing Capacity of Pile**

| Pile Length (m) | SPT-N = 30 at Pile Termination | | SPT-N = 50 at Pile Termination | |
|---|---|---|---|---|
| | Ultimate Bearing Capacity (kN) | Factor of Safety | Ultimate Bearing Capacity (kN) | Factor of Safety |
| 2.7 (minimum) | 912 | 1.22 **(<2.0, Inadequate)** | 1,412 | 1.88 **(<2.0, Inadequate)** |
| 8.7 (average) | 1,272 | 1.70 **(<2.0, Inadequate)** | 1,772 | 2.36 **(≥2.0, Acceptable)** |
| 22.2 (maximum) | 2,082 | 2.78 **(≥2.0, Acceptable)** | 2,582 | 3.44 **(≥2.0, Acceptable)** |

Pile capacity calculations show that the expected ultimate bearing capacity to be about 1,290kN for pile length up to 9m if the SPT-N value at pile termination is 30. If the SPT-N value at pile termination is 50, then the expected ultimate bearing capacity is 1,250kN to 1,790kN. The factors of safety over the adopted working load of 750kN therefore may range from about 1.0 to 2.4 for pile length up to 9m, compared to the commonly adopted factor of safety of 2 or higher. Note that BS 8004:1986 (Code of Practice for Foundations) recommends a factor of safety of between 2.0 and 3.0 for a single pile. Details of pile capacity calculations are given in Table 1.9.

### 10.2.3 Mobilized Pile Capacity Measured from Dynamic Load Tests (PDA)

During the interview on 4th February 2008, the Consultant informed that no static load test was carried out because of the difficult site conditions. Instead, a total of 12 numbers (about 6% of total piles) dynamic load test (PDA) were performed to determine the pile mobilized capacity. The PDA test results showed that all the 12 number pile tested were able to mobilize capacities of at least twice the working load, i.e. with factors of safety of at least 2. The predicted pile top settlements under the working load and twice the working load were also acceptable according to the Specification for Piling Works in the Contract.

It is prudent to accept the results of the dynamic load test if they are calibrated with the conventional static load tests. In this case, no conventional static load was done. Gue & Chen (1998) have indicated that the results of the dynamic load test could predict its capacity by more than 60% of the conventional static load test.

## 11.0 Conclusion

*(a) Soil Investigation*

Only one borehole and nine Mackintosh probes were carried out and this is grossly inadequate. There were also no undisturbed samples collected for tests to obtain soil shear strength parameters needed for slope analyses. This is unacceptable and inappropriate.

*(b) Designs*

No slope stability and pile foundation analyses were carried out. This is imprudent.

*(c) Slope stability*

The Sub-Committee analyses indicate that the factors of safety are in the range of 0.72 to 1.34, which indicate failures are expected for Factor Of Safety = 1 in some of the slopes and creeps are expected in the slopes having inadequate Factor Of Safety. Both of these will induce additional bending moments and shear forces in the structure of the building.

*(d) Piles*

Factors of safety of the piles vary from 1.22 to 2.78 compared to the required factor of safety of 2 or higher. Piles with lower factors of safety especially near to unity are expected to settle more. This will induce additional bending moments and shear forces in the structure of the building.

(e) *Mechanism of Failure*

Localized slip failures are expected for those slopes having factor of safety less than 1, and creep will occur on those slopes having factor of safety less than 1.4, the required factor of safety. Any of these conditions will cause lateral and vertical movements of the slopes and the pile foundations. When these happen, additional bending moments and shear forces would be induced in the structure of the building, and the structure will fail when the bending and shear capacities are exceeded.

# PLATES

Plate 1A : Slip at the edge and below the building

Plate 1B : Hairline cracks in localized beams and walls

**Plate 1C : Columns deflected towards the lake side and drainage facilities heavily silted**

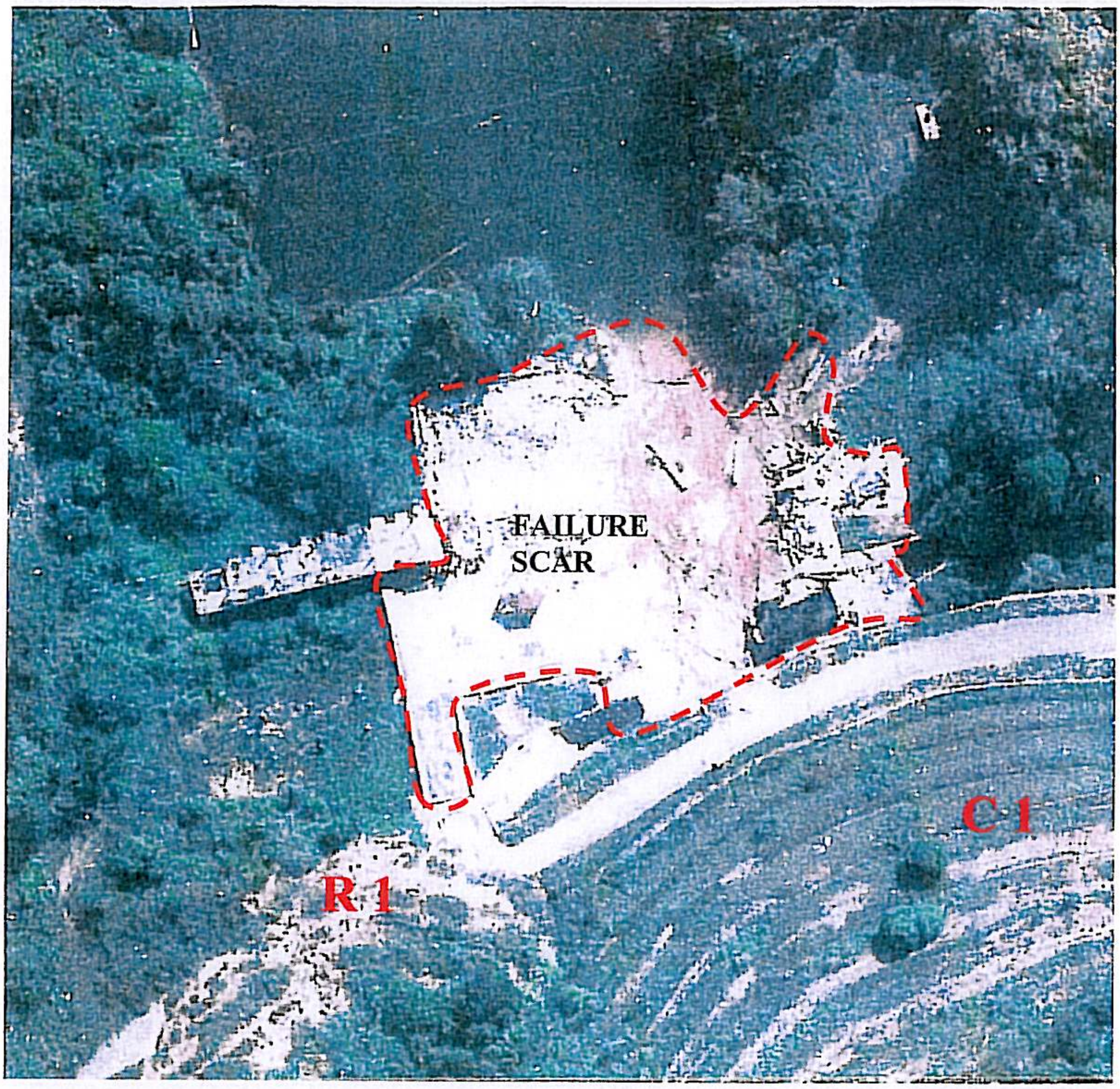
**Plate 1D: Residual soil of gravelly silt and sand at the**

Plate1E

Plate 1F

**Plate 1E & 1F :**
**Discharge points of rainwater and drainage facilities completely damage at failure location**

# FIGURES




SKALA FOTO UDARA 1 : 10 000
SKALA PELOT 1 : 5 000

U

**Figure 1.1 : Close up view of Pulau Banding resort after the collapse (captured on 15th January 2008)**

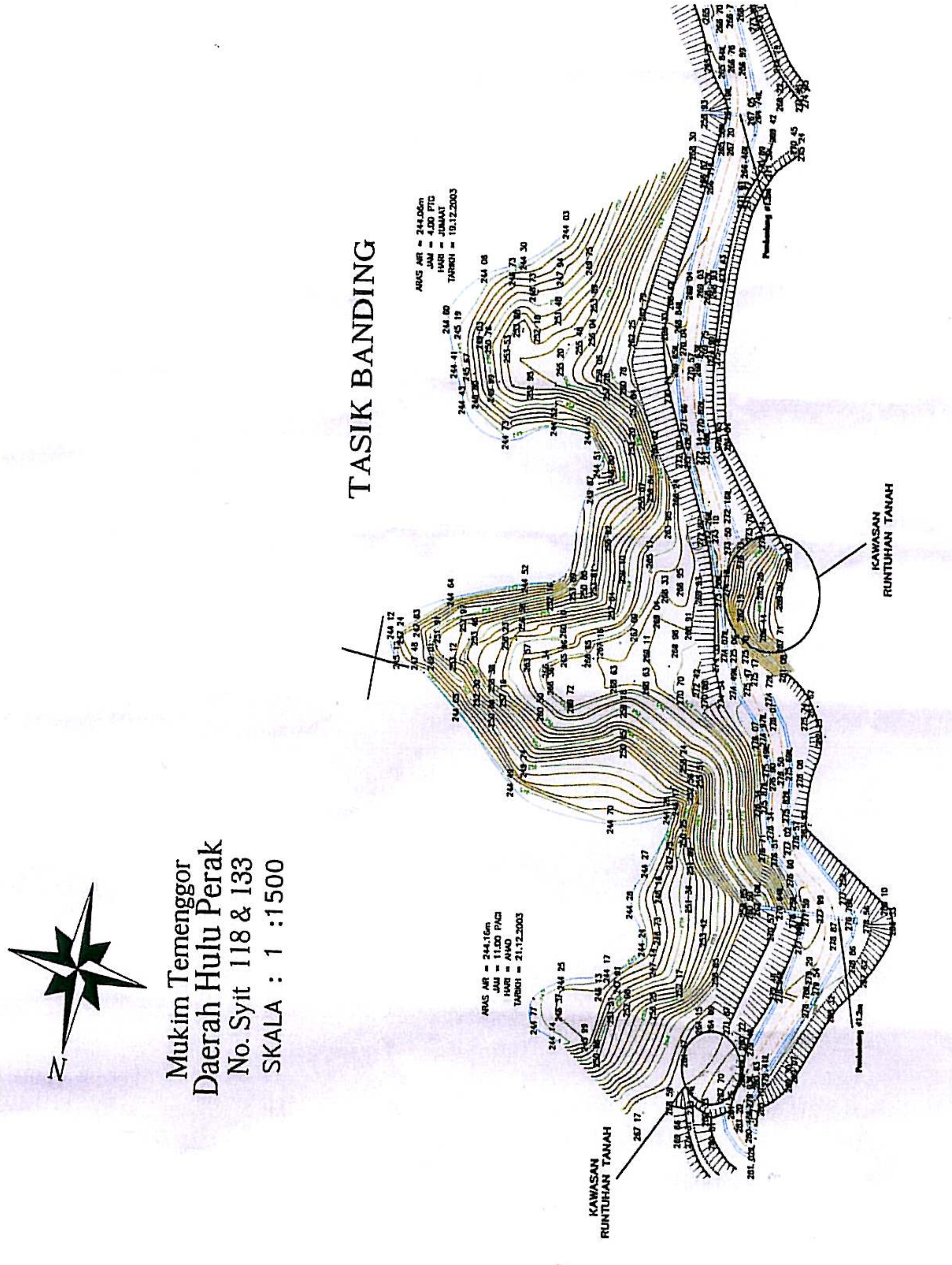



**Figure 1.2 : Failure scar captured in the Survey Plan before building construction.**

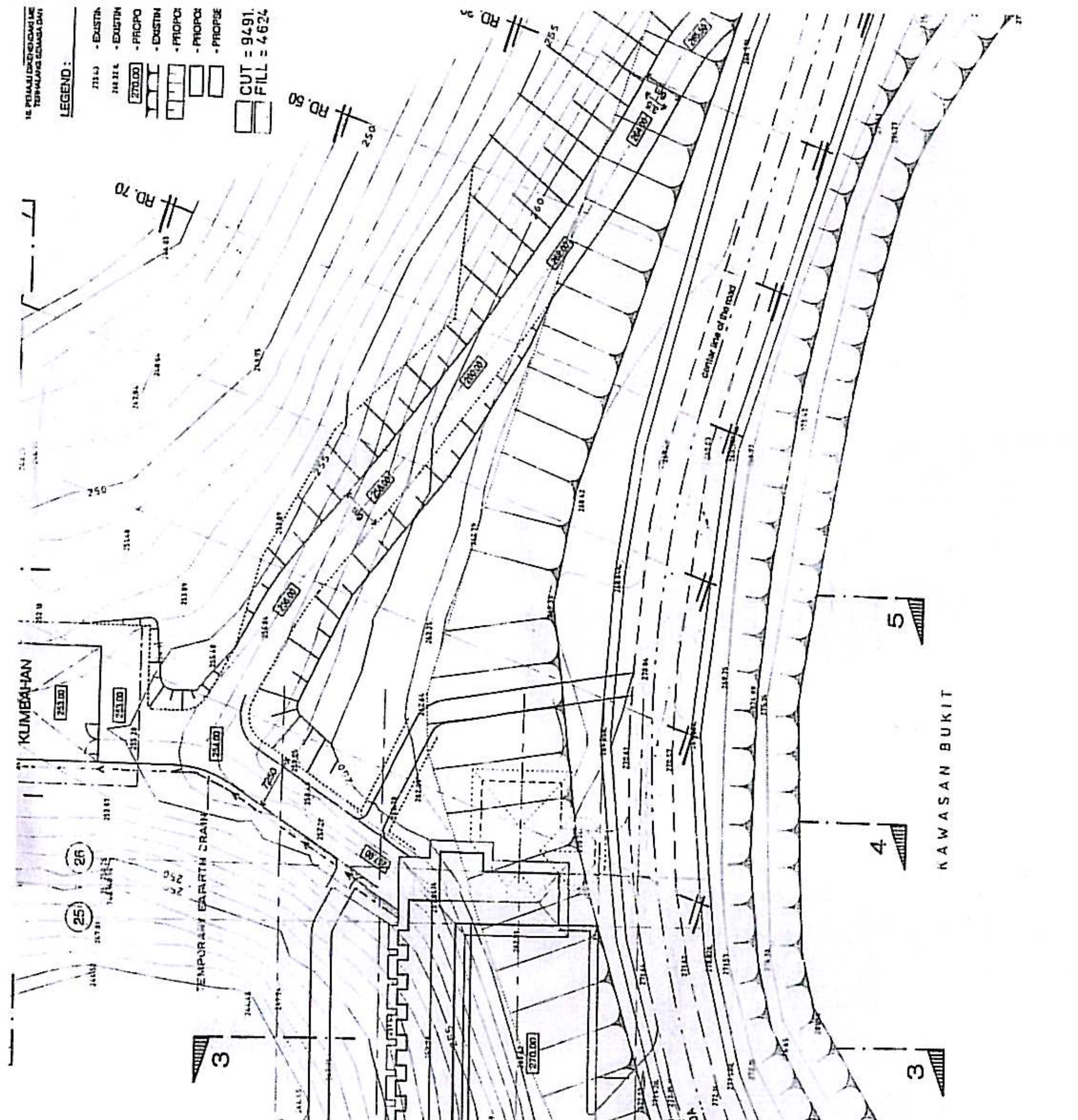
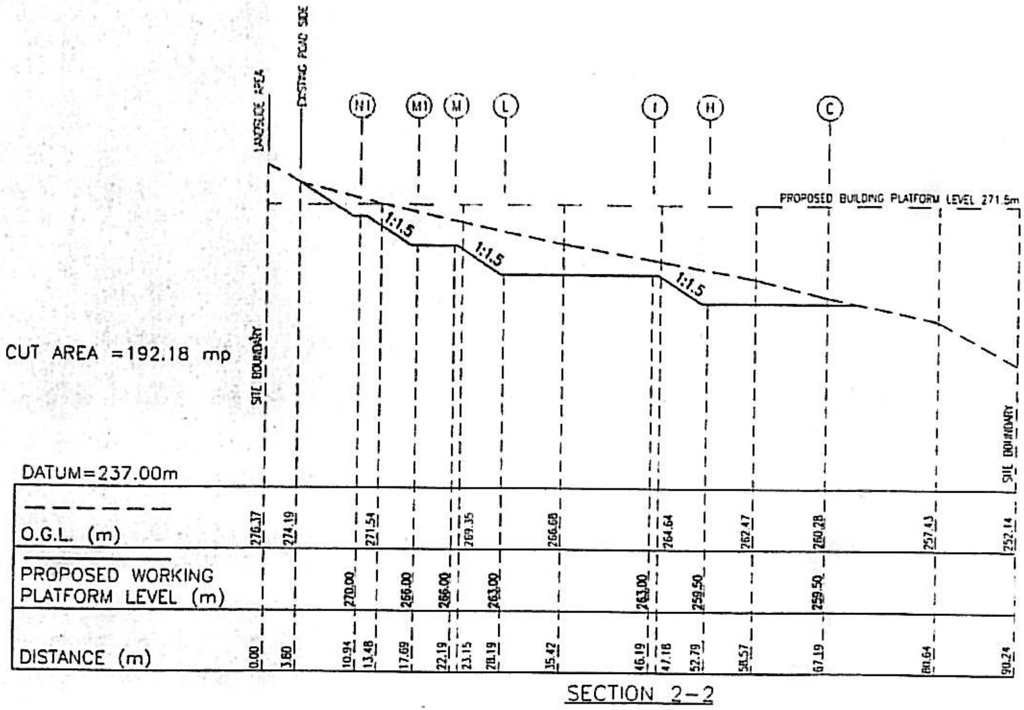



**Figure 1.3 : Pelan susunatur kerja tanah**
**( Dwg. No. CT18/68/03/EW-1)**



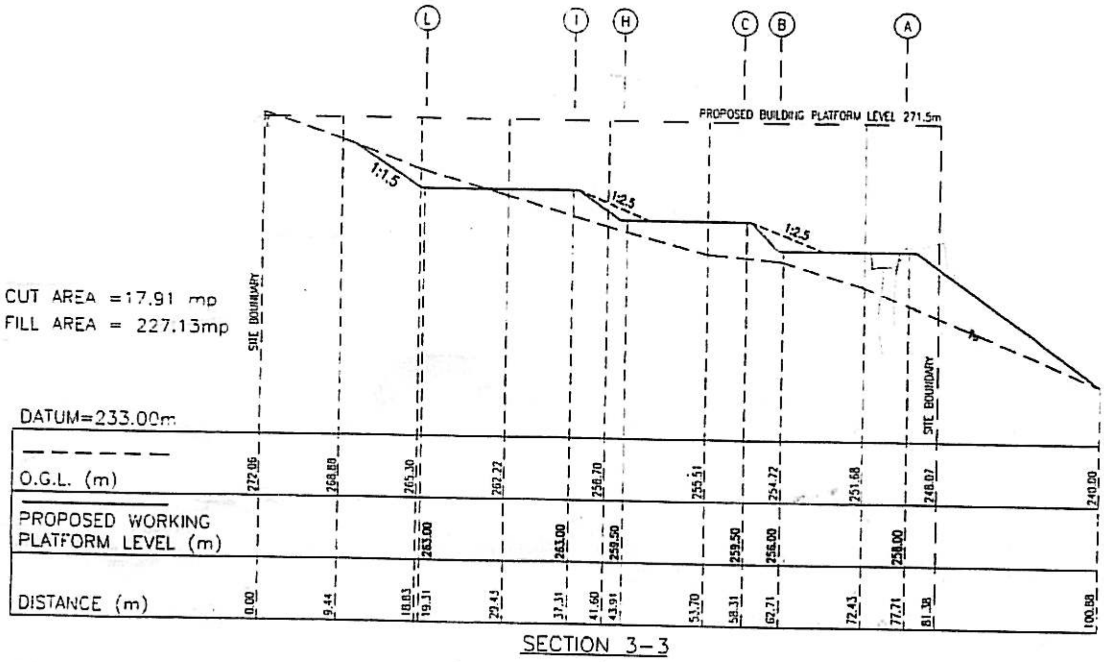
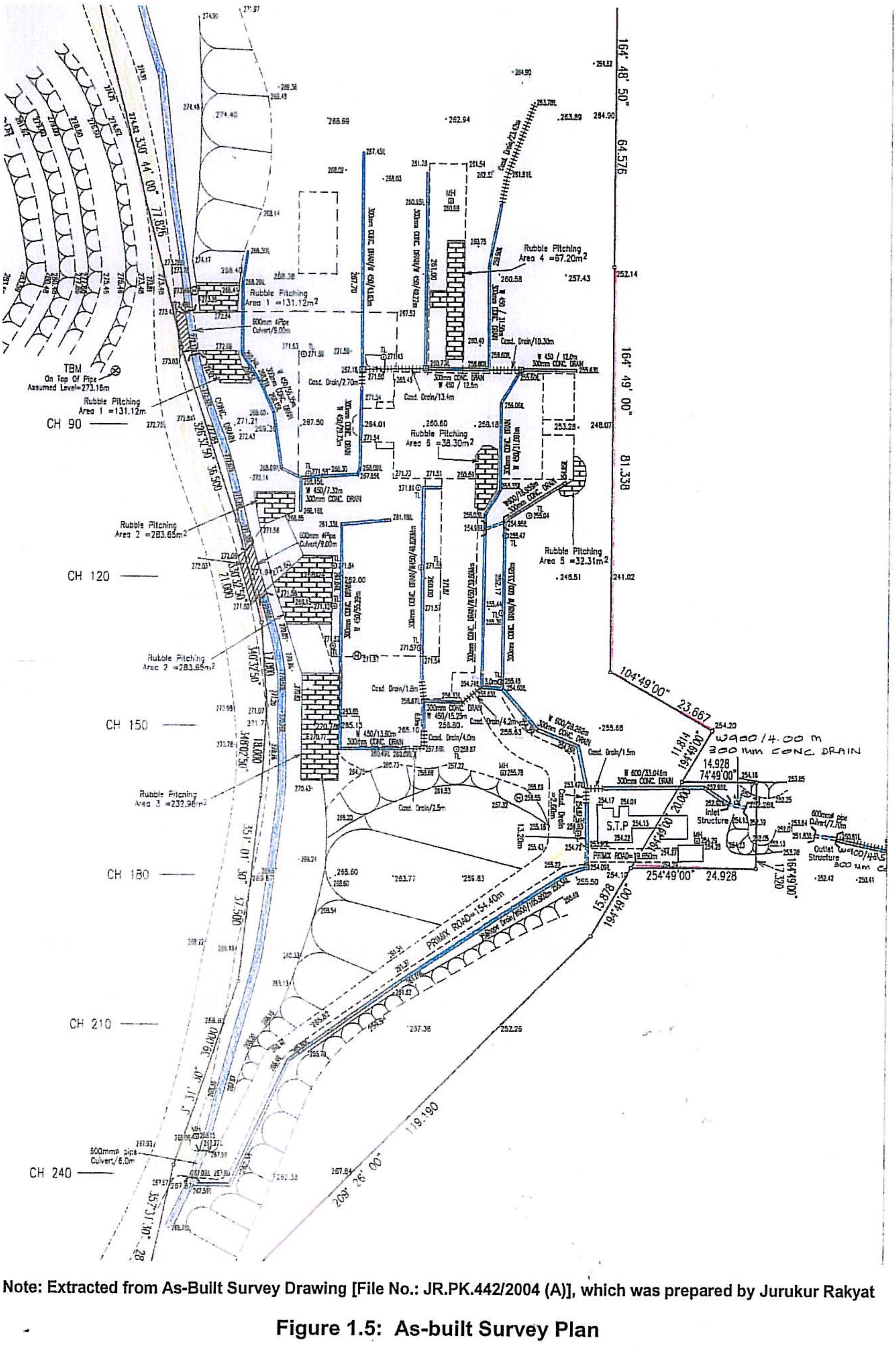
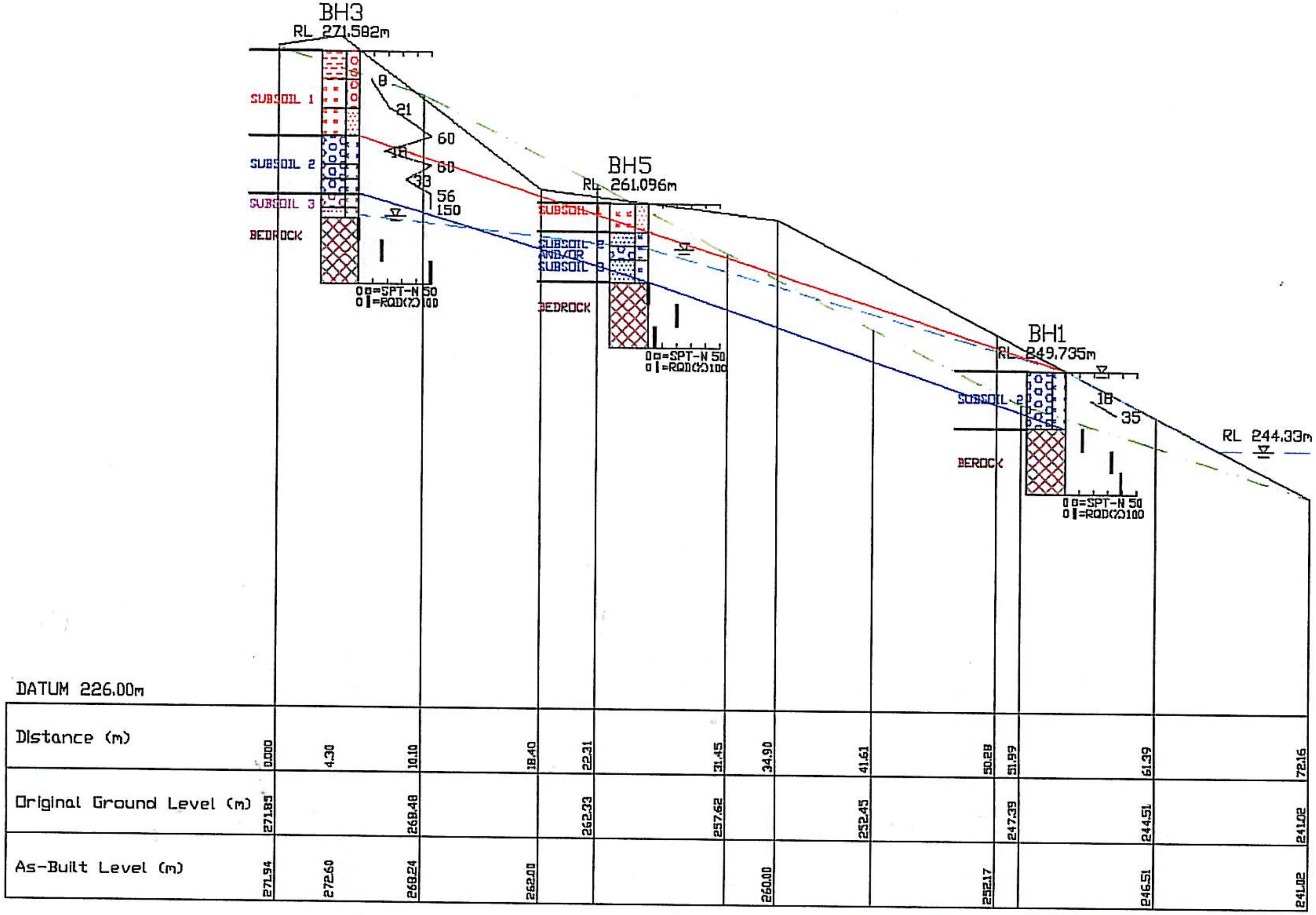




**Figure 1.4 : Design cross section of section 2-2 & 3-3 (FAIL NO. : JR.PK.442/2004(A) - 22-09-2004)**




Note: Extracted from As-Built Survey Drawing [File No.: JR.PK.442/2004 (A)], which was prepared by Jurukur Rakyat

**Figure 1.5: As-built Survey Plan**




Note: Reproduced from As-Built Survey Drawing [File No.: JR.PK.442/2004 (A)], which was prepared by Jurukur Rakyat

**Figure 1.6: Cross-section CH. 120**

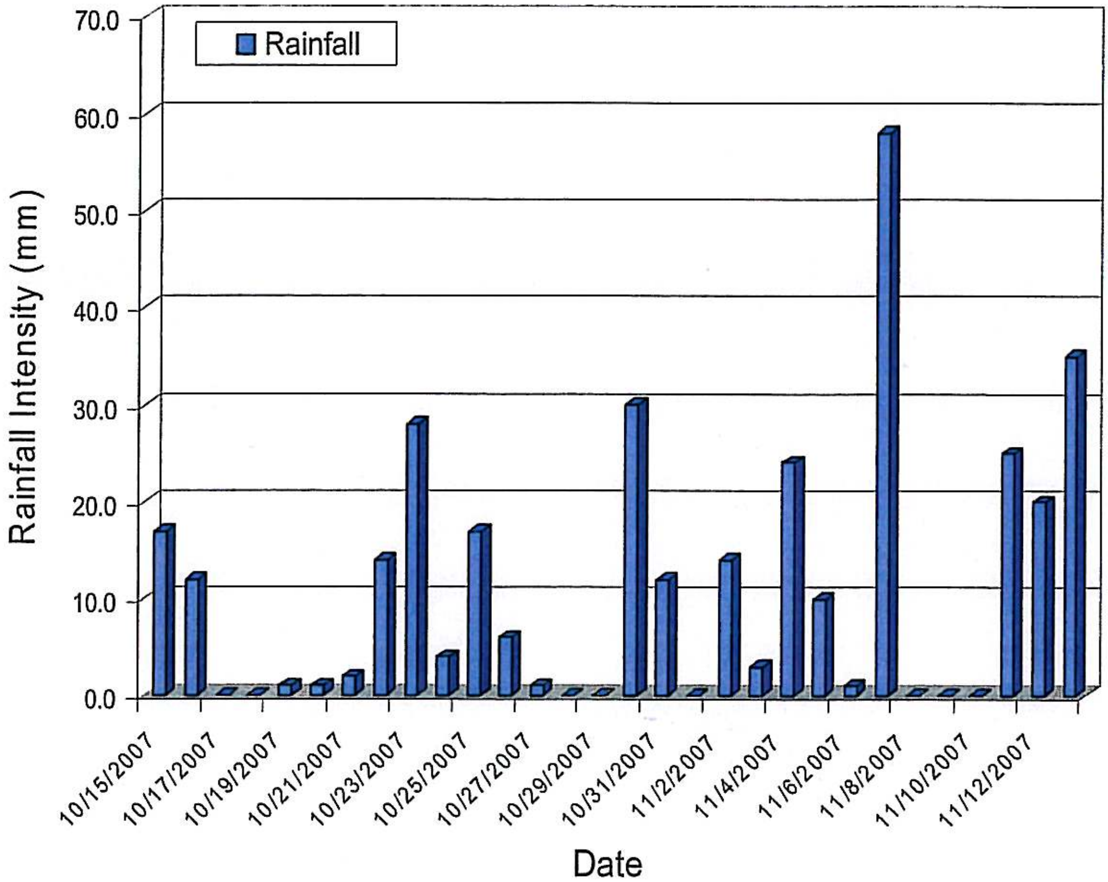
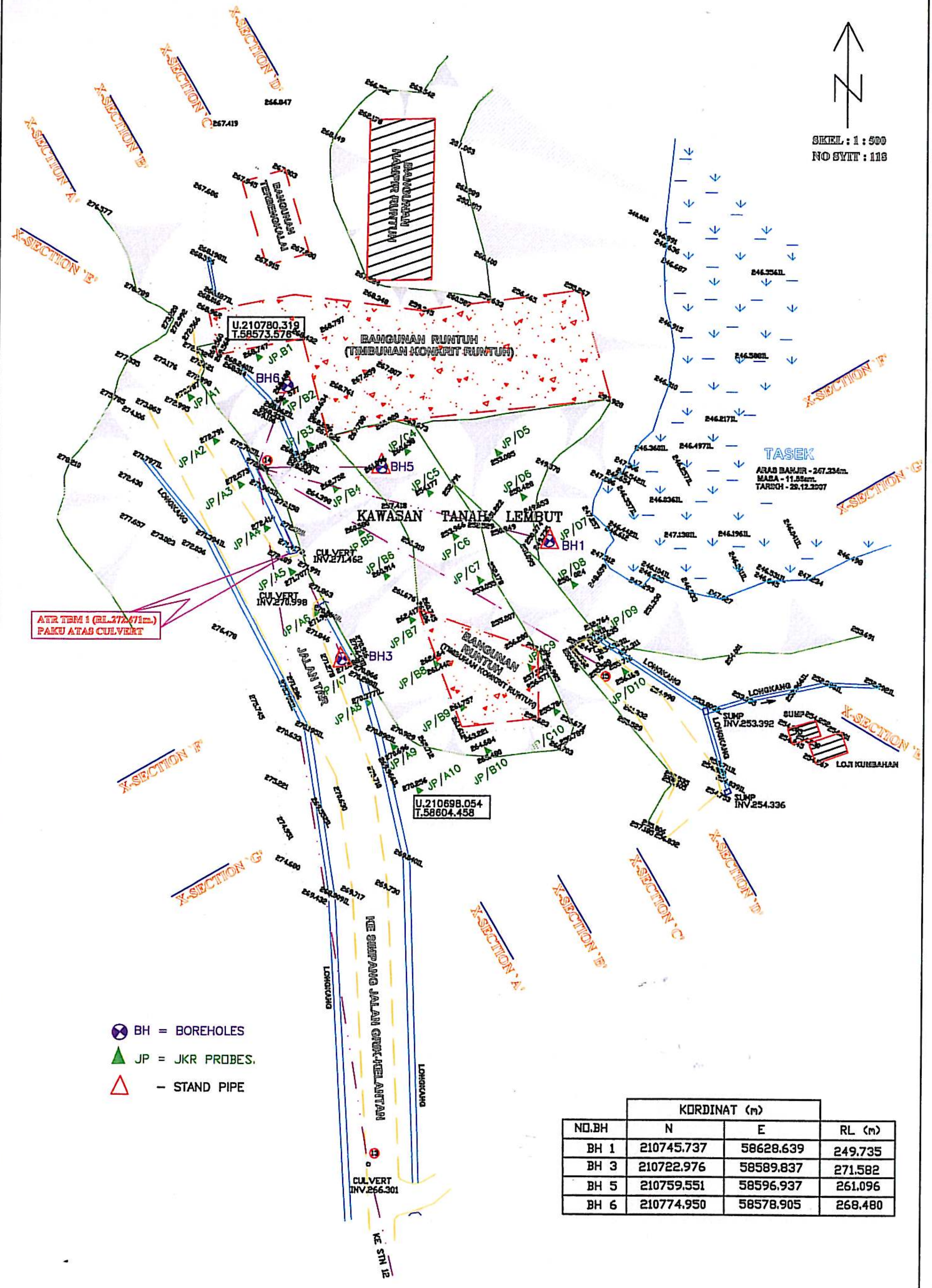



**Figure 1.7 : Daily rainfall record at Banding Station 30 days prior to the building failure**




|  | KORDINAT (m) |  |  |
|---|---|---|---|
| NO.BH | N | E | RL (m) |
| BH 1 | 210745.737 | 58628.639 | 249.735 |
| BH 3 | 210722.976 | 58589.837 | 271.582 |
| BH 5 | 210759.551 | 58596.937 | 261.096 |
| BH 6 | 210774.950 | 58578.905 | 268.480 |

Figure 1.8 : Location of Soil Investigation

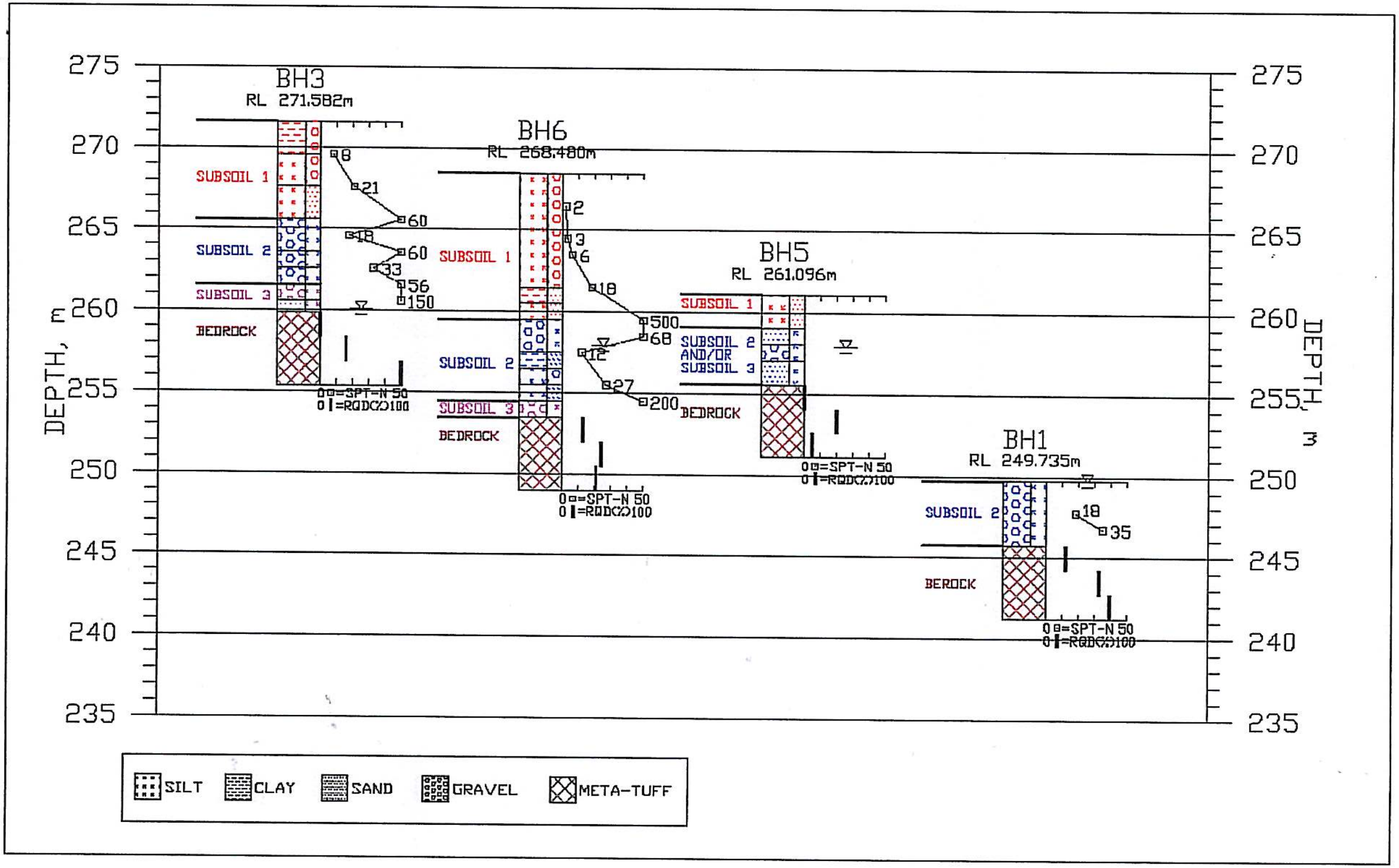



**Figure 1.9: Simplified Engineering Borehole Log**

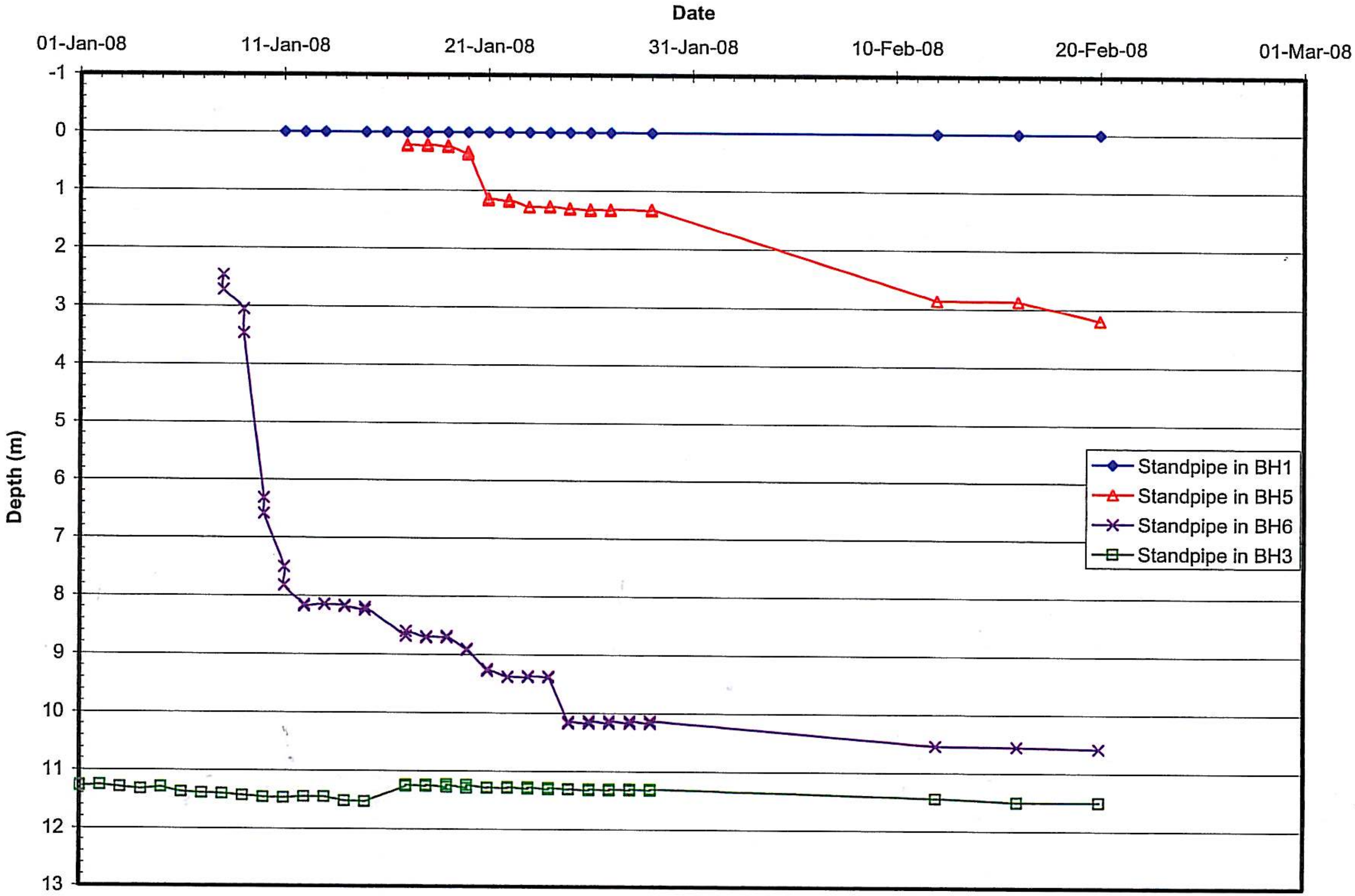



**Figure 1.10: Groundwater Monitoring in Standpipes**

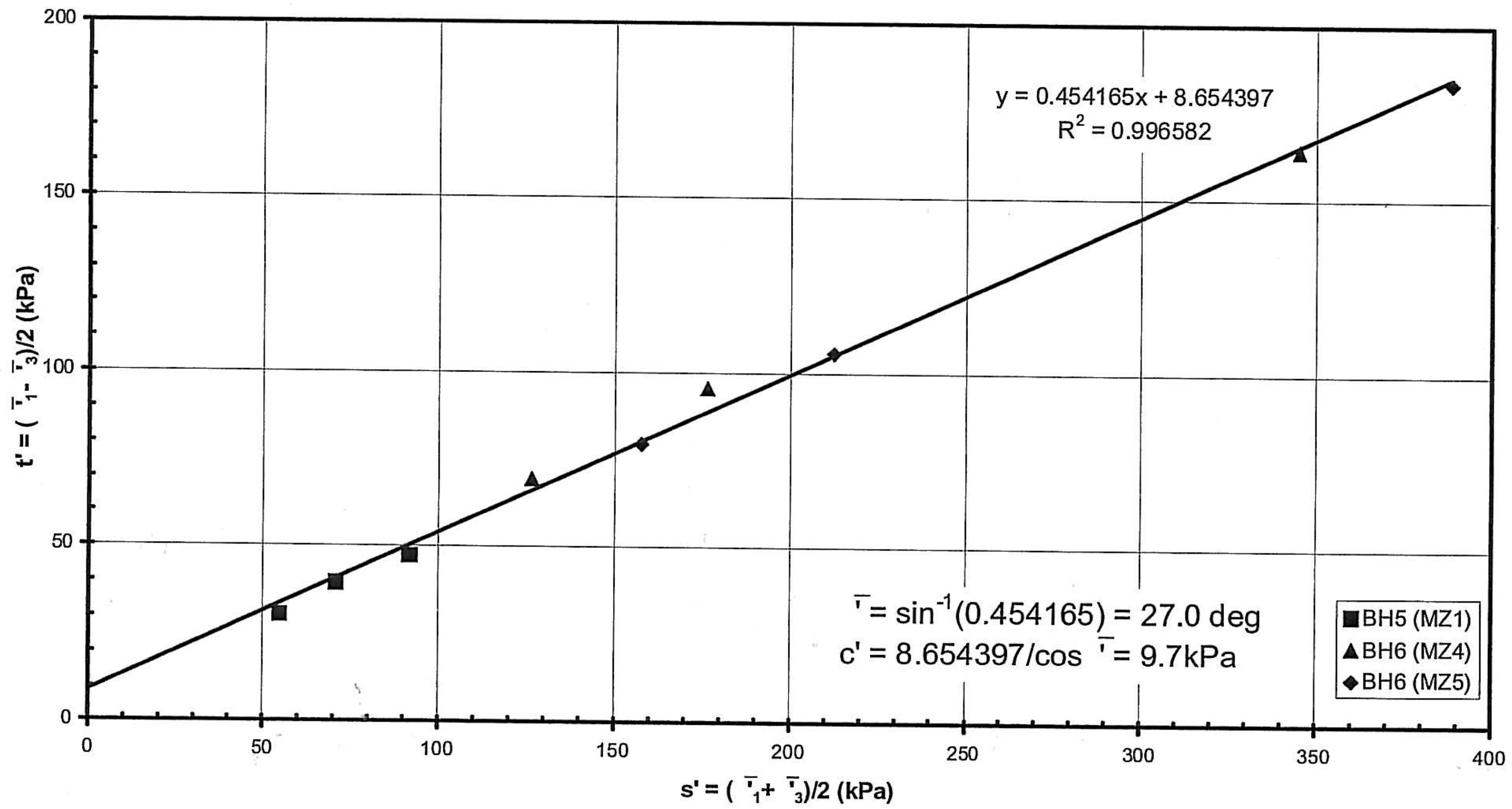
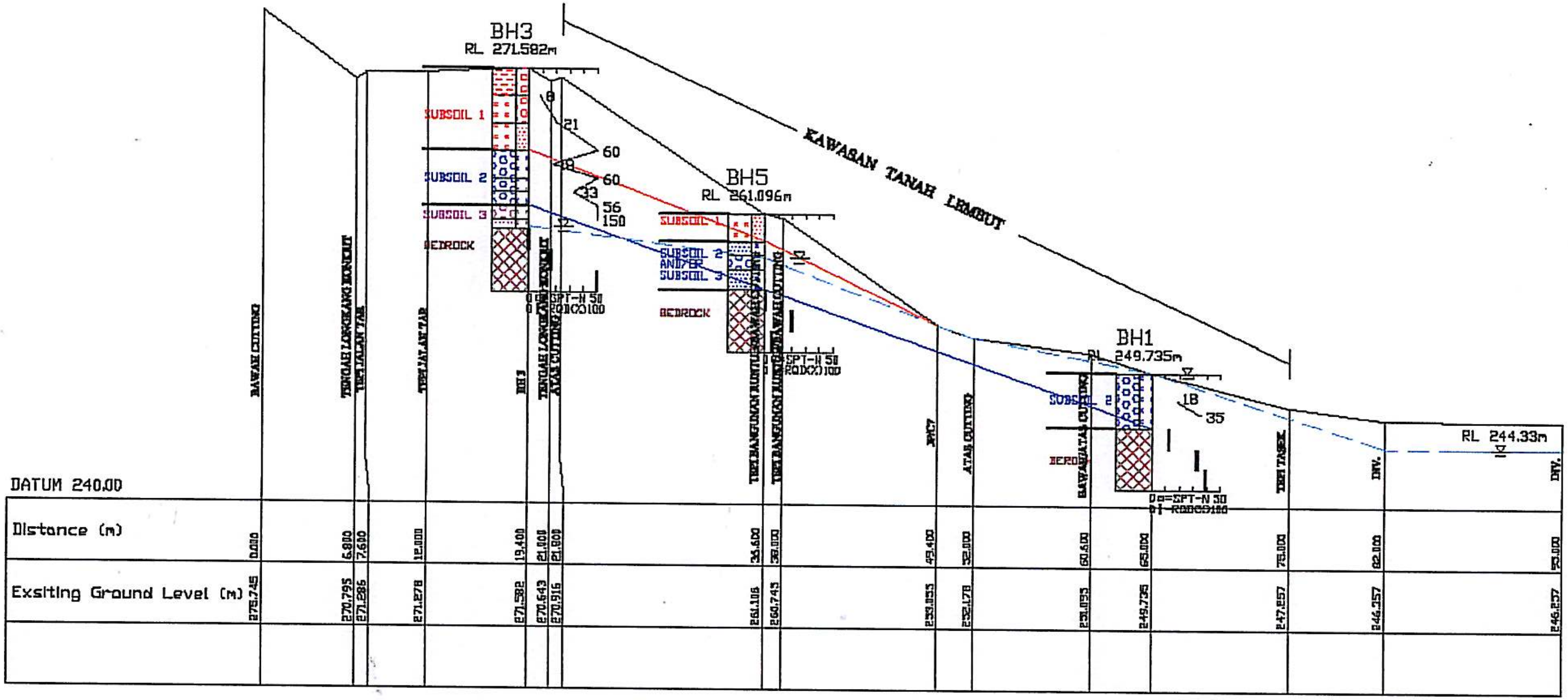



**Figure 1.11: CIU Triaxial Test: s' – t' Plot**




Note: Reproduced from Survey Plan No. ATRPK/1E/1/2008/A1, which was prepared by Abdullah Taha & Rakan Rakan

**Figure 1.12 : Cross-section F-F**

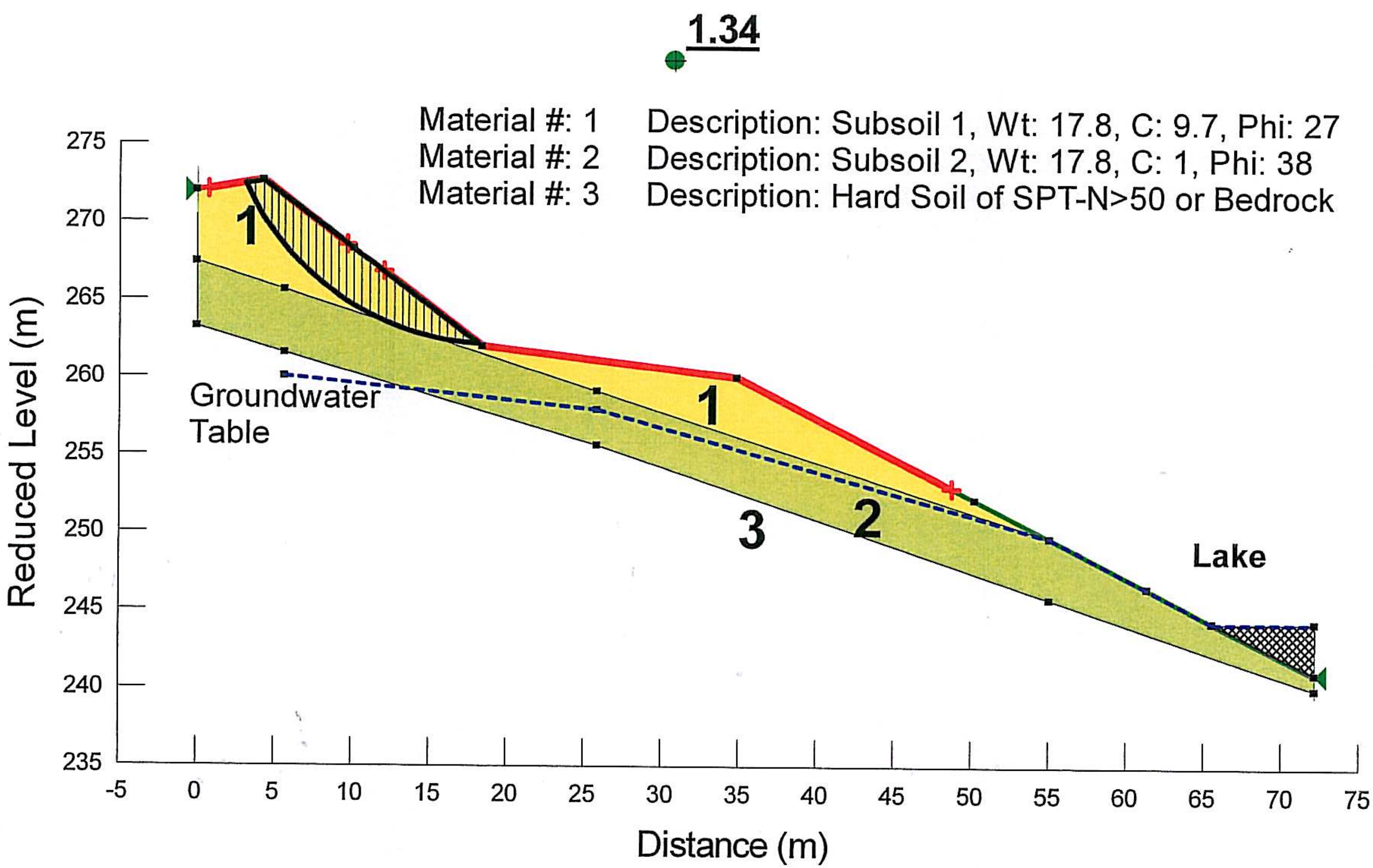



**Figure 1.13(a): Stability Analysis of Cross-section CH. 120**

**Case 1 - Subsoil 1: C' = 9.7kPa, $\phi$' = 32.4°; Subsoil 2: C' = 1.0kPa, $\phi$' = 38.0°;**
**Stability Check for Upper Slope: Factor of Safety = 1.34 (<1.40, Unacceptable)**

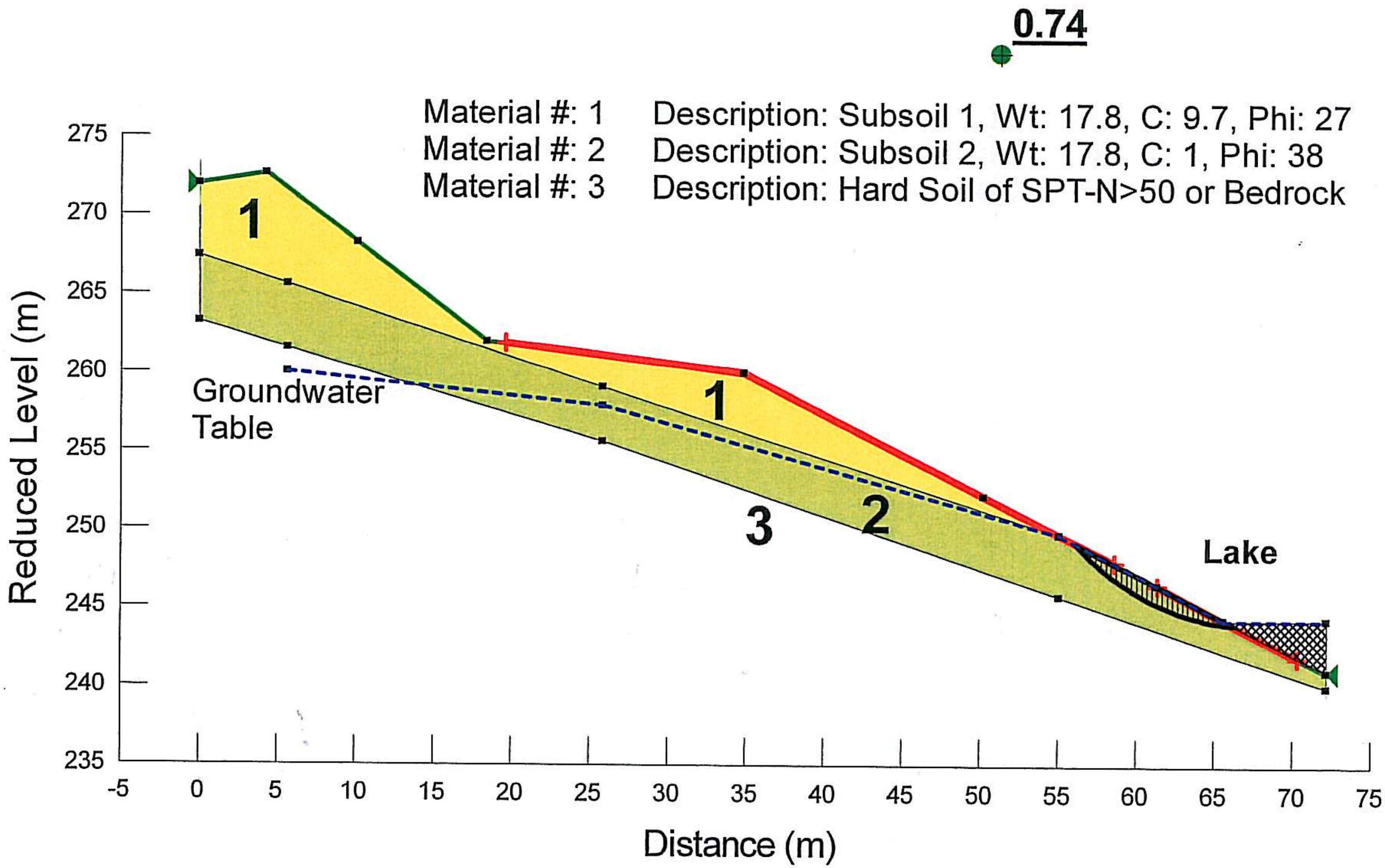



**Figure 1.13(b): Stability Analysis of Cross-section CH. 120**

**Case 1 - Subsoil 1: C' = 9.7kPa, $\phi'$ = 32.4°; Subsoil 2: C' = 1.0kPa, $\phi'$ = 38.0°;**

**Stability Check 1 for Lower Slope: Factor of Safety = 0.74 (<1.00, Failure expected)**

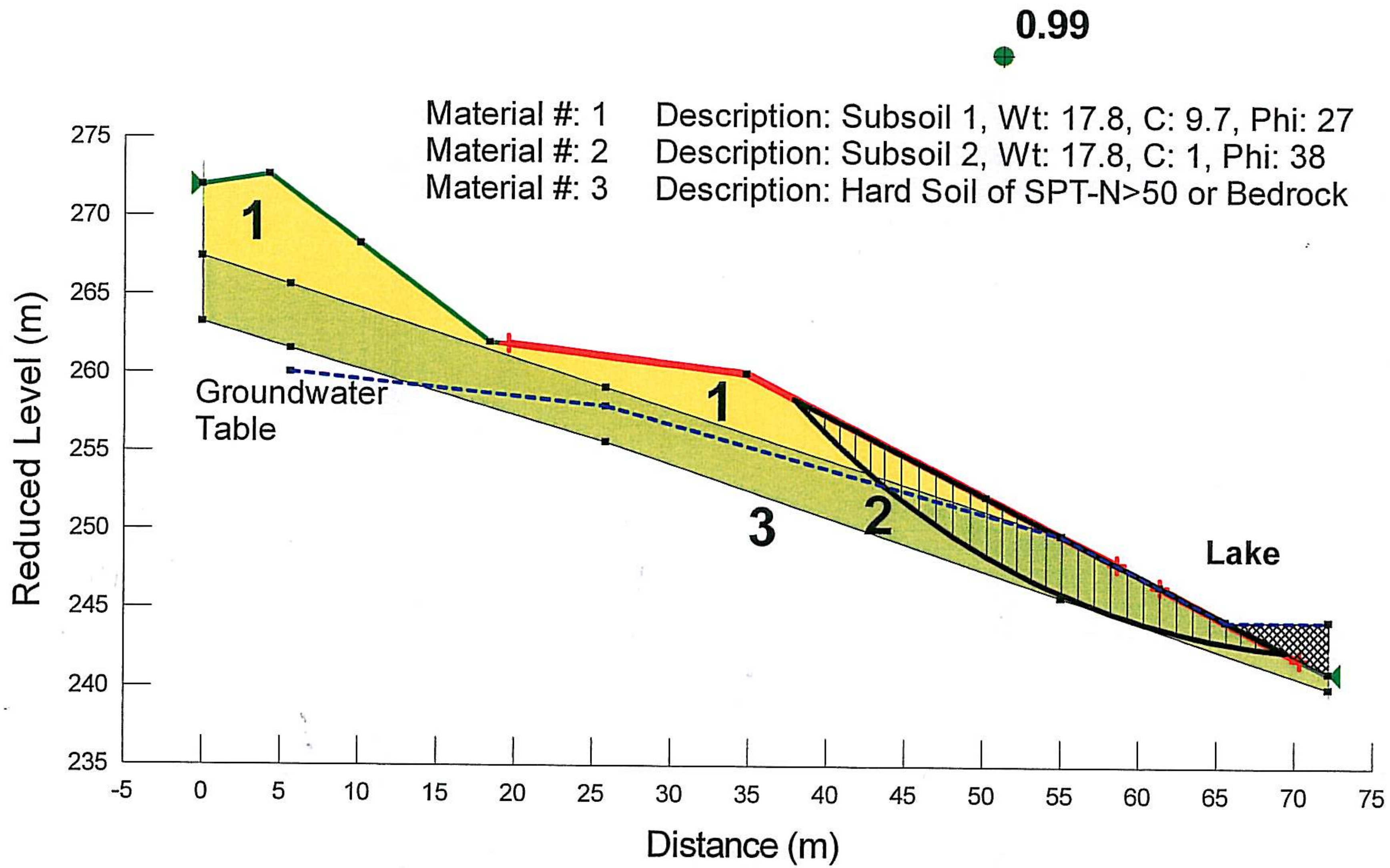



**Figure 1.13(c): Stability Analysis of Cross-section CH. 120**

**Case 1 - Subsoil 1: C' = 9.7kPa, $\phi'$ = 32.4°; Subsoil 2: C' = 1.0kPa, $\phi'$ = 38.0°;**
**Stability Check 2 for Lower Slope: Factor of Safety = 0.99 (<1.00, Failure expected)**

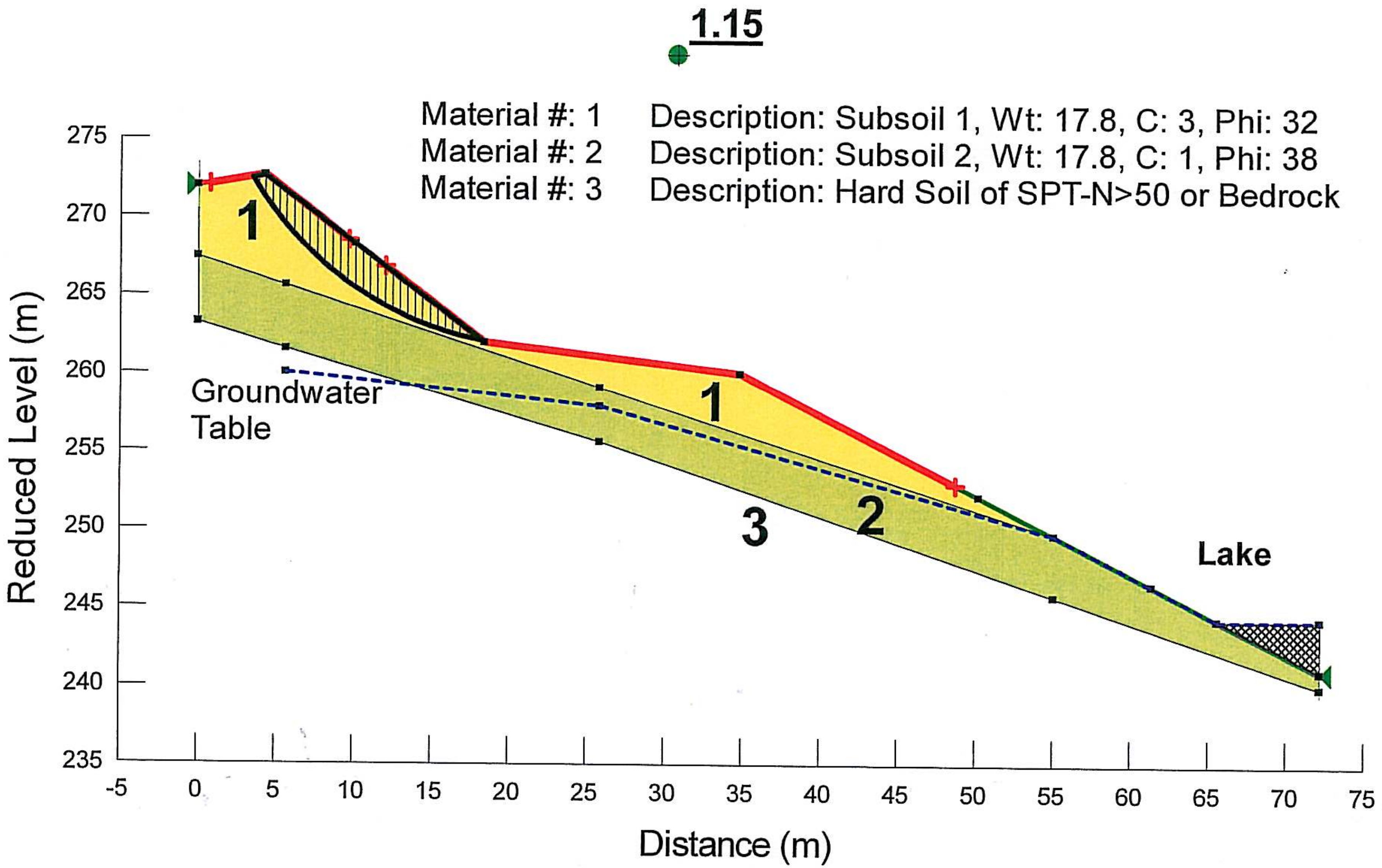



**Figure 1.13(d). Stability Analysis of Cross-section CH. 120**

**Case 2 - Subsoil 1: C' = 3.0kPa, ϕ' = 32.0°; Subsoil 2: C' = 1.0kPa, ϕ' = 38.0°**

**Stability Check for Upper Slope: Factor of Safety = 1.15 (<1.40, Unacceptable)**

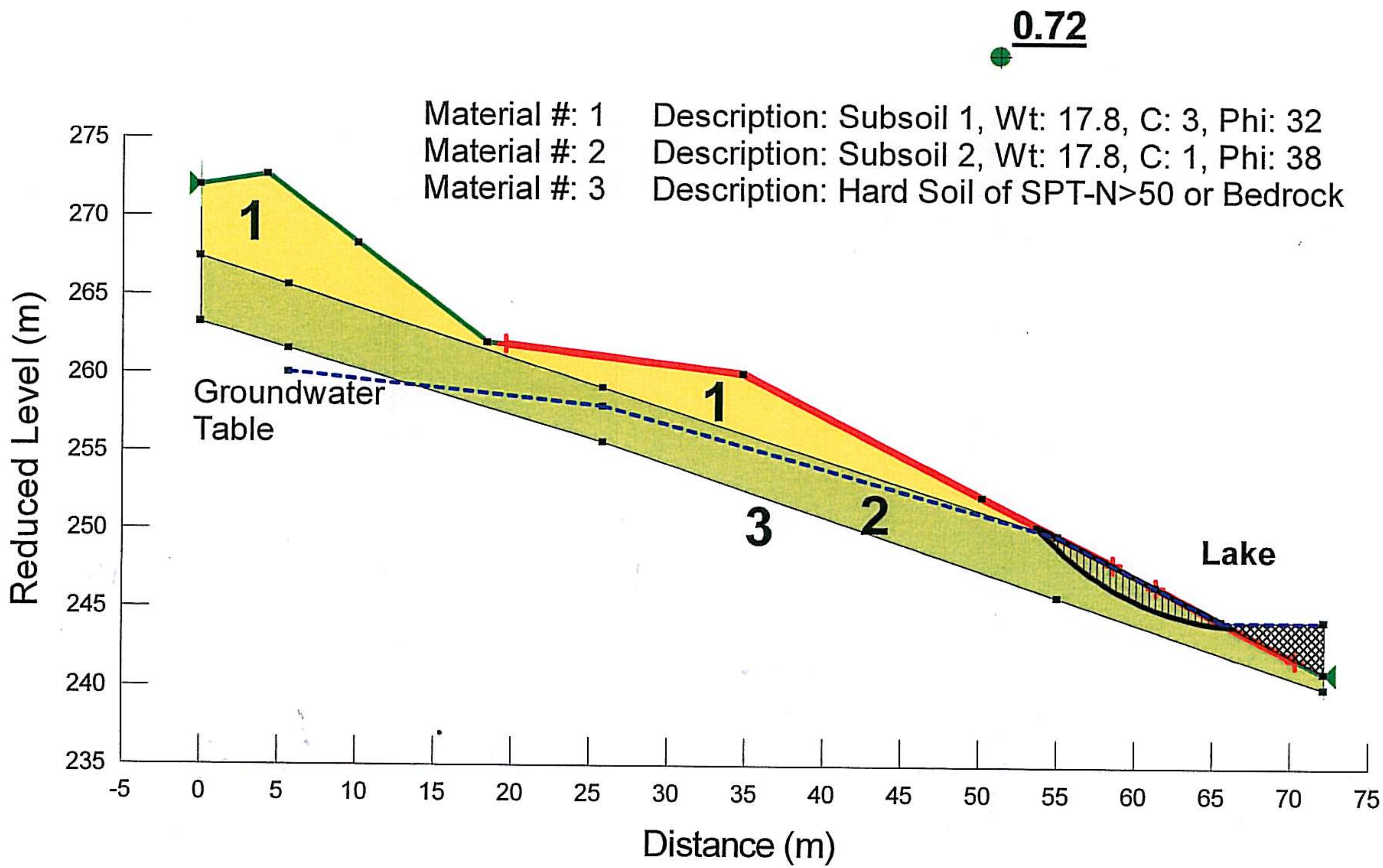



**Figure 1.13(e). Stability Analysis of Cross-section CH. 120**

**Case 2 - Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°; Subsoil 2: C' = 1.0kPa, $\phi'$ = 38.0°**

**Stability Check 1 for Lower Slope: Factor of Safety = 0.72 (<1.00, Failure expected)**

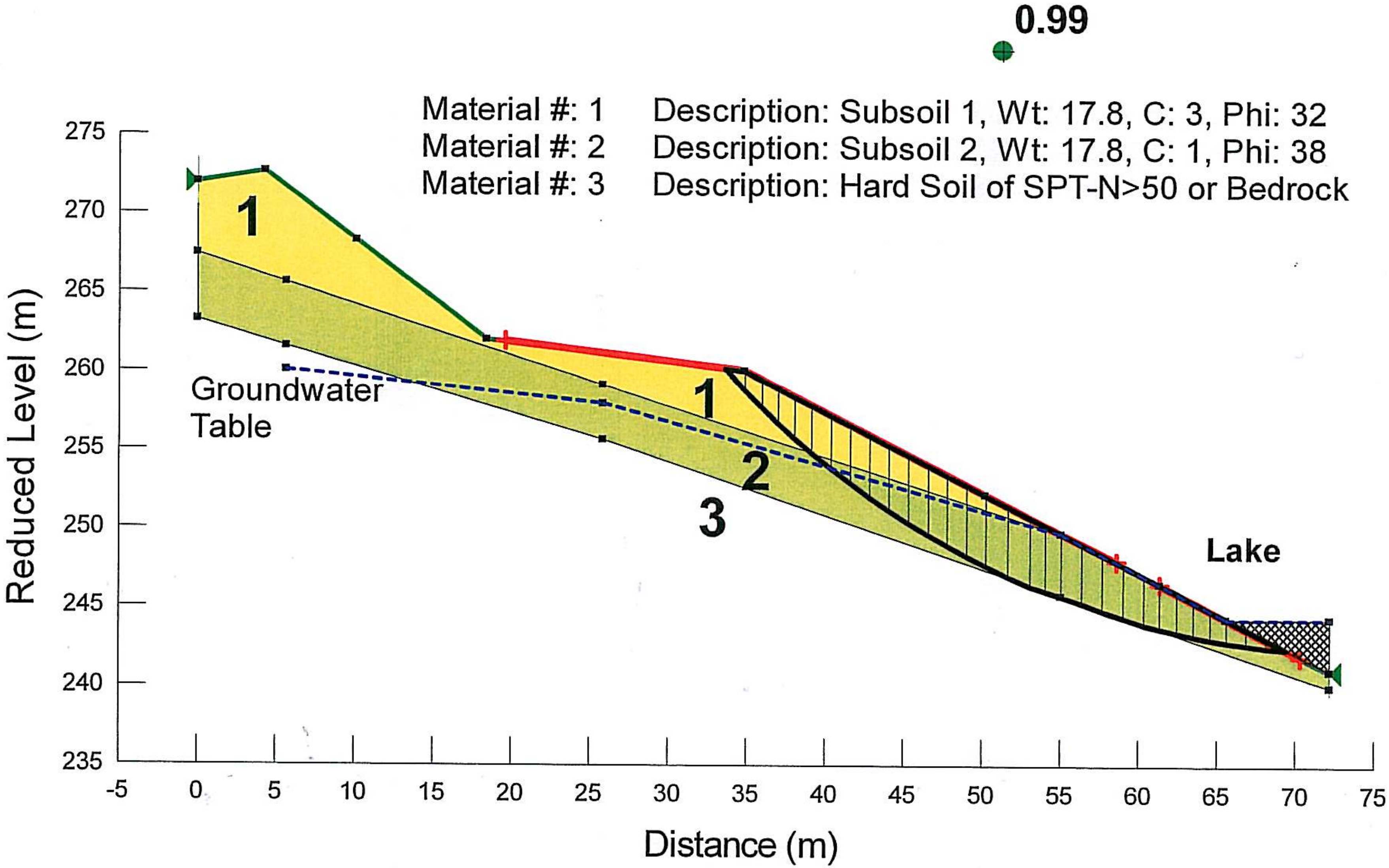



**Figure 1.13(f): Stability Analysis of Cross-section CH. 120**

**Case 2 - Subsoil 1: C' = 3.0kPa, ϕ' = 32.0°; Subsoil 2: C' = 1.0kPa, ϕ' = 38.0°**

**Stability Check 2 for Lower Slope: Factor of Safety = 0.99 (<1.00, Failure expected)**

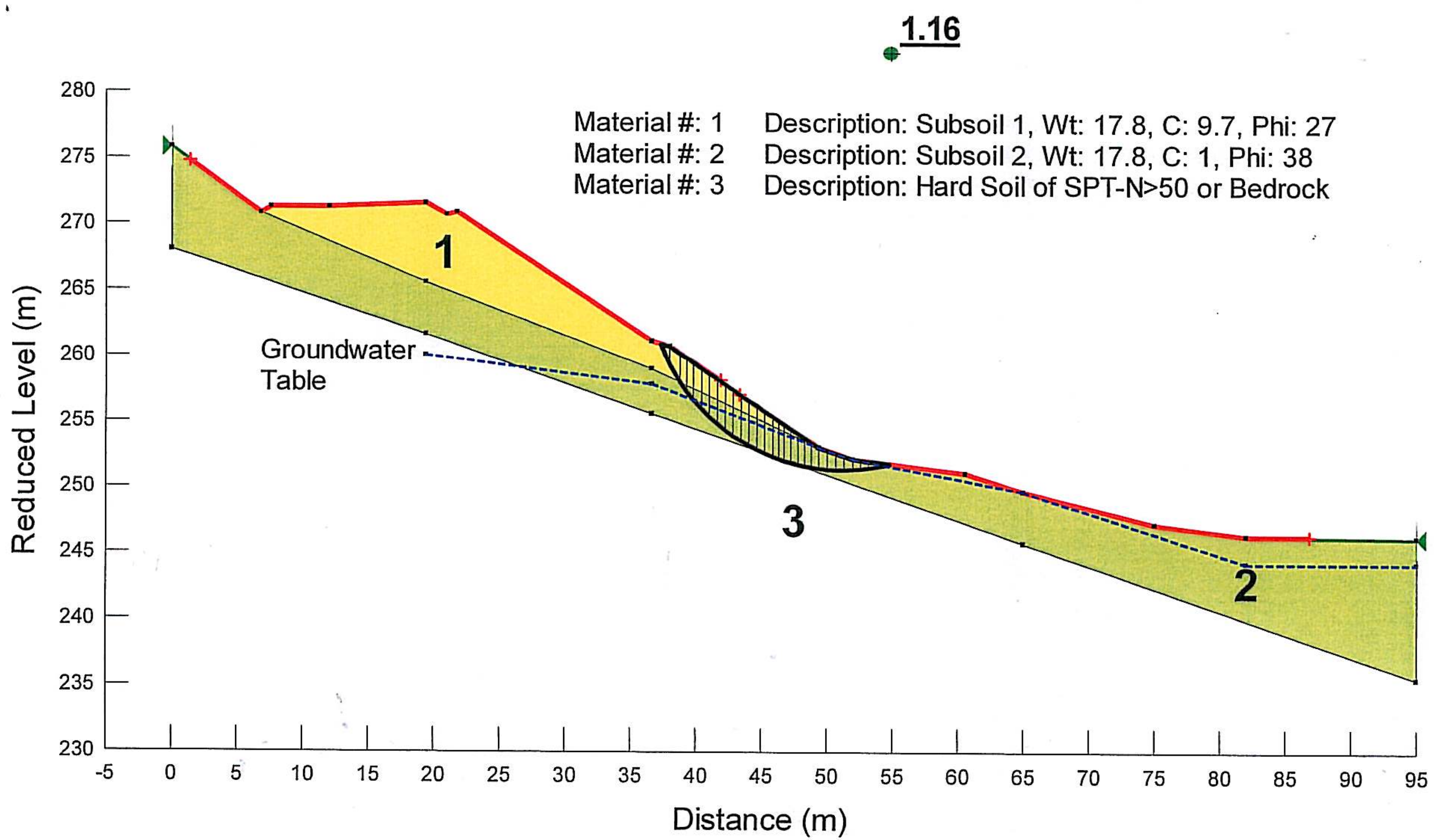



**Figure 1.13(g): Stability Analysis of Cross-section F-F**

**Case 1 - Subsoil 1: C' = 9.7kPa, ϕ' = 32.4°; Subsoil 2: C' = 1.0kPa, ϕ' = 38.0°**

**Factor of Safety = 1.16 (<1.40, Unacceptable)**

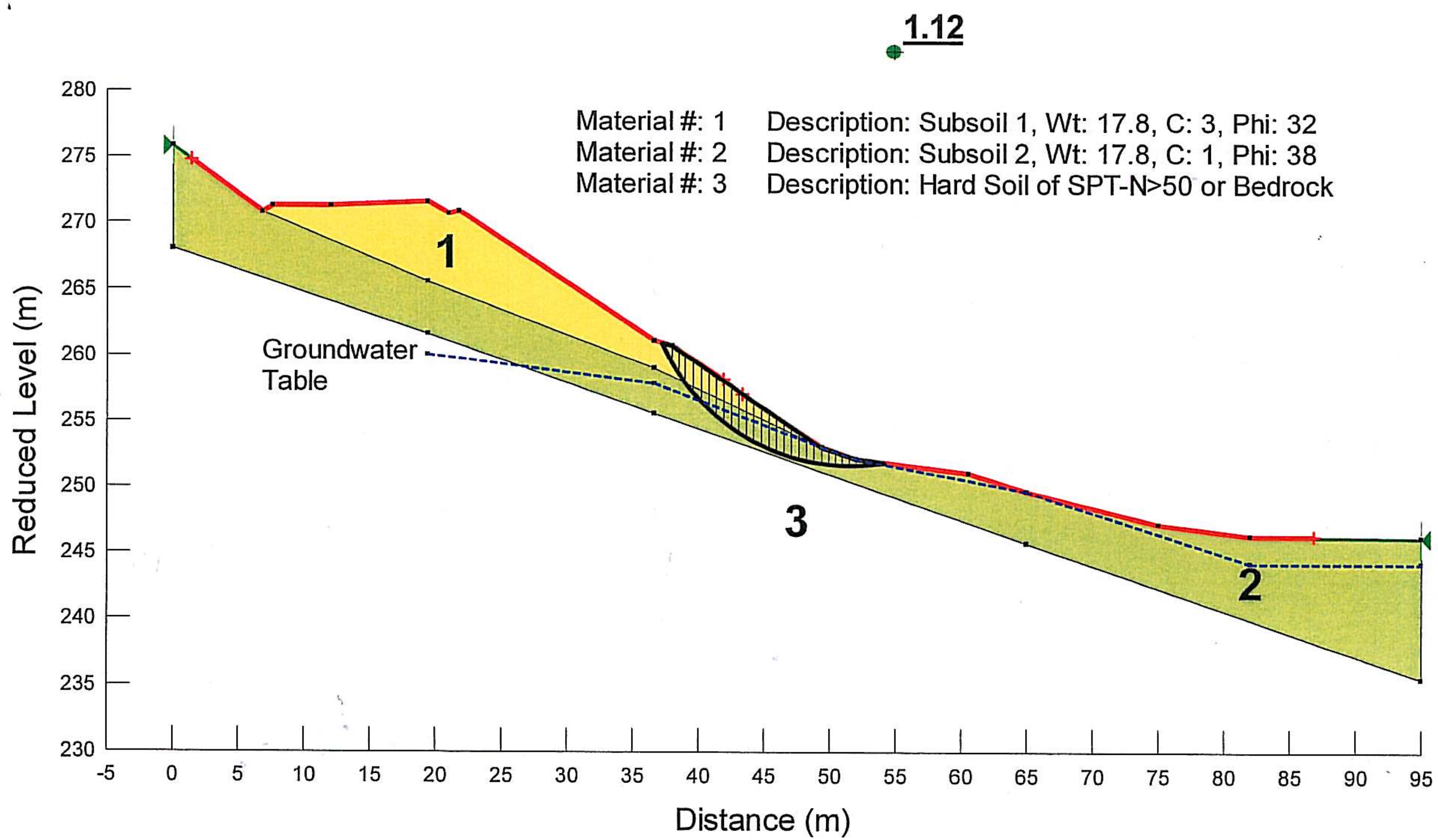



**Figure 1.13(h): Stability Analysis of Cross-section F-F**

**Case 2 - Subsoil 1: C' = 3.0kPa, $\phi'$ = 32.0°; Subsoil 2: C' = 1.0kPa, $\phi'$ = 38.0°**

**Factor of Safety = 1.12 (<1.40, Unacceptable)**

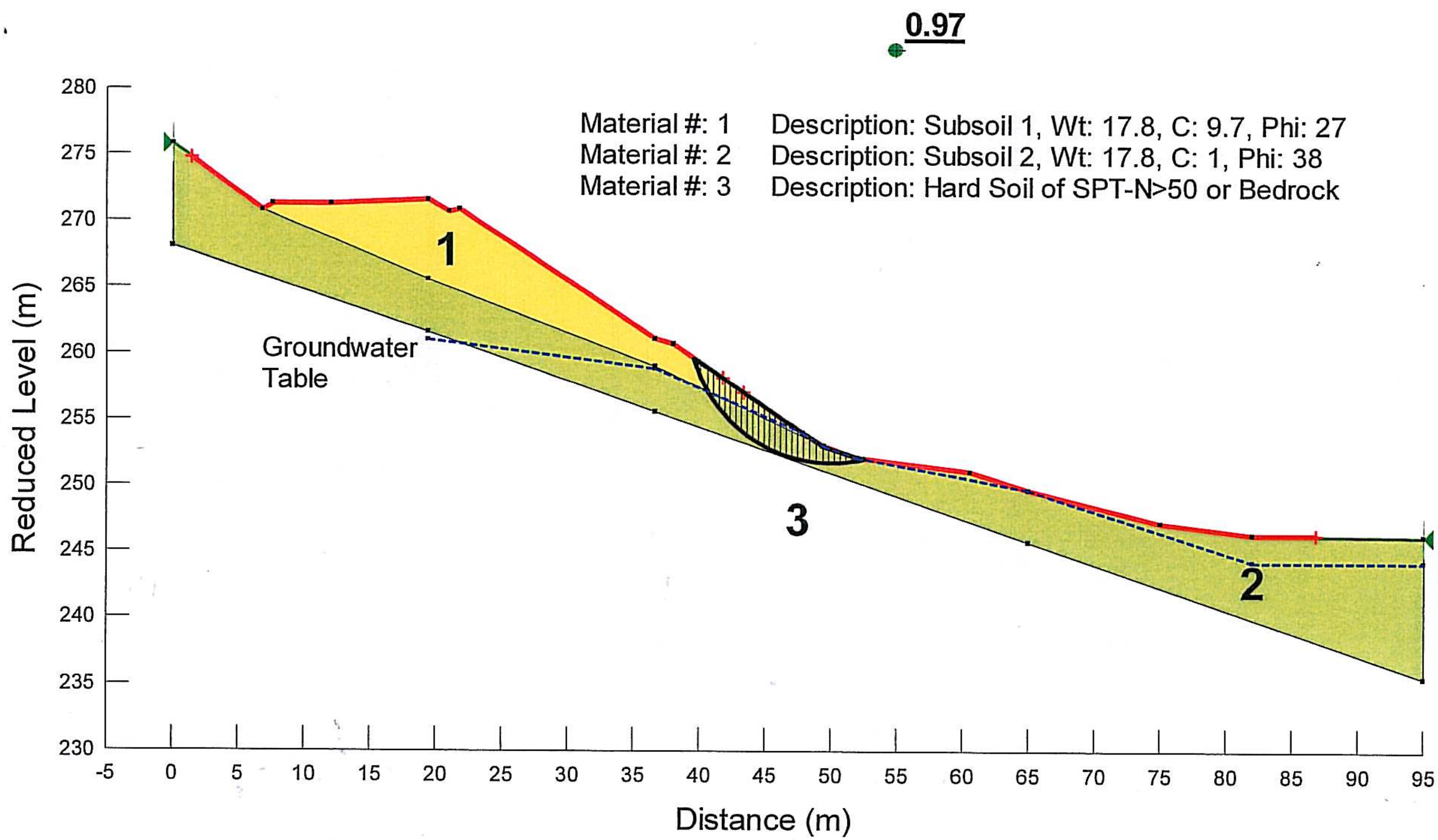



**Figure 1.13(i): Stability Analysis of Cross-section F-F if Groundwater Level Rises for 1m**

**Case 1 - Subsoil 1: C' = 9.7kPa, ϕ' = 27.0°; Subsoil 2: C' = 1.0kPa, ϕ' = 38.0°**

**Factor of Safety = 0.97 (<1.00, Failure expected)**

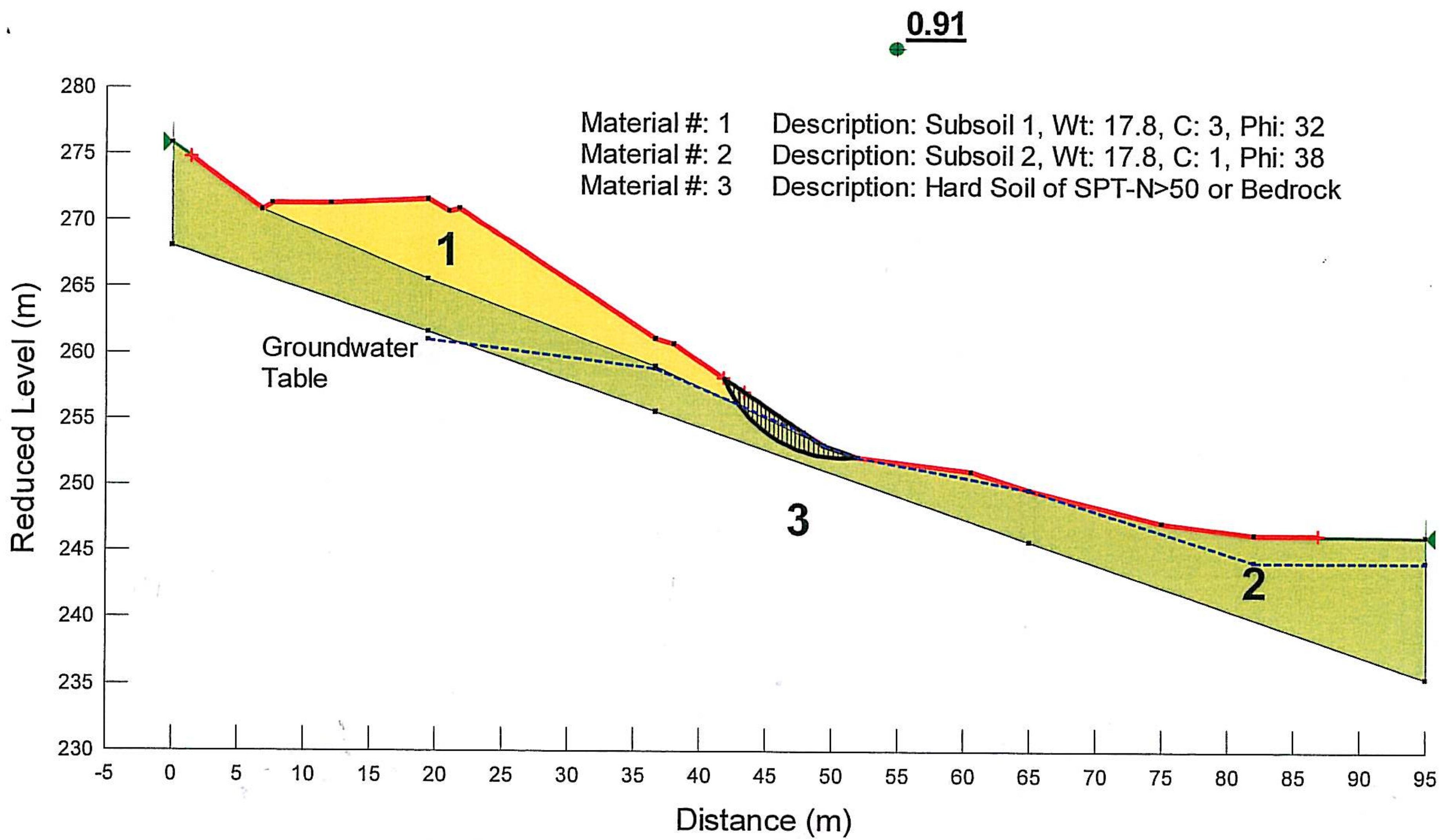



**Figure 1.13(j): Stability Analysis of Cross-section F-F if Groundwater Level Rises for 1m**

**Case 2 - Subsoil 1: C' = 9.7kPa, $\phi$' = 27.0°; Subsoil 2: C' = 1.0kPa, $\phi$' = 38.0°**

**Factor of Safety = 0.91 (<1.00, Failure expected)**

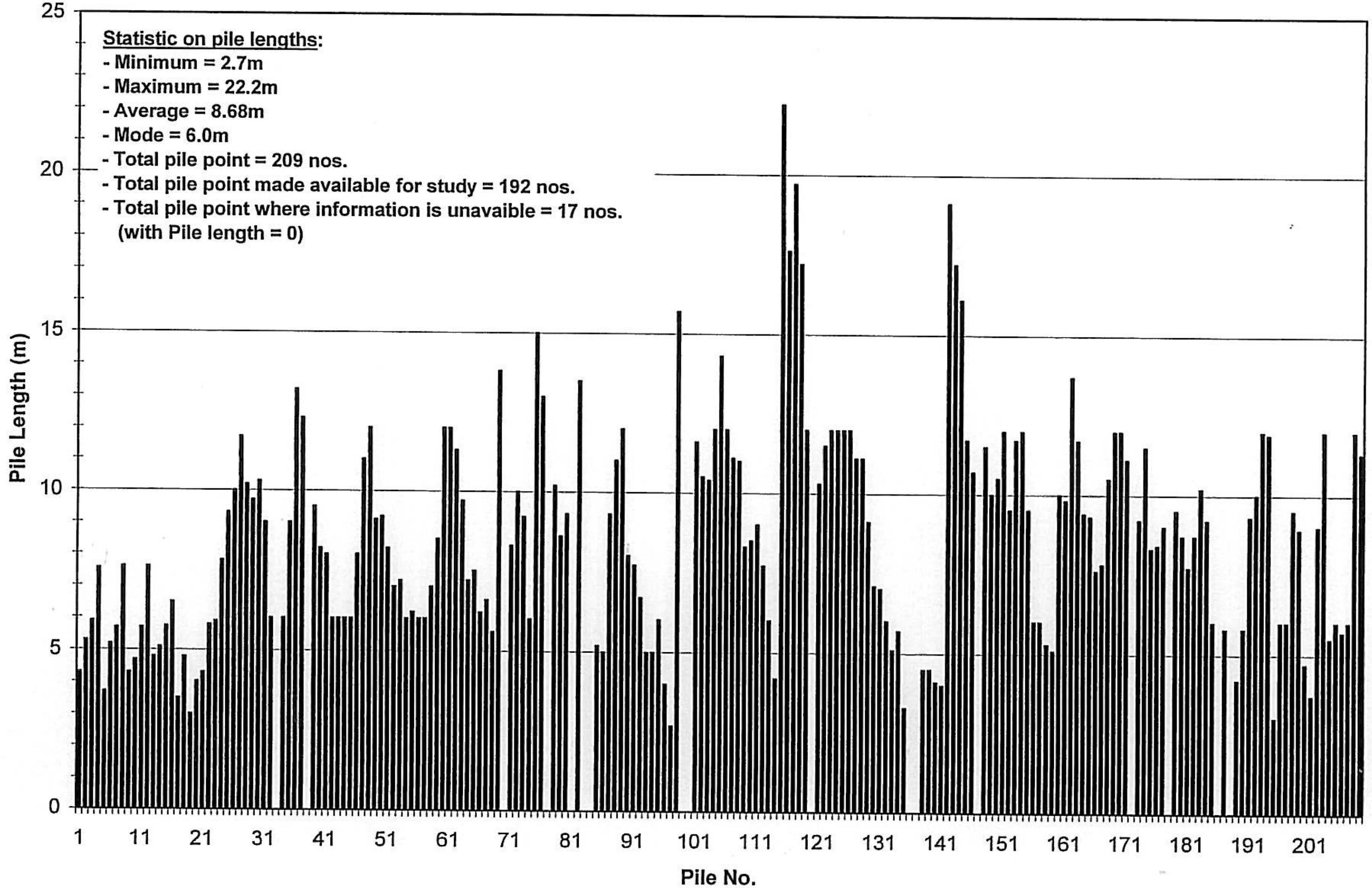



Figure 1.14: Summary of Pile Lengths

# APPENDICES

**PARAS AIR TASEK TEMENGGOR**

| September-07 | | |
|---|---|---|
| 1/9/2007 | | 244.33 |
| 2/9/2007 | | 244.29 |
| 3/9/2007 | | 244.28 |
| 4/9/2007 | | 244.19 |
| 5/9/2007 | | 244.12 |
| 6/9/2007 | | 244.02 |
| 7/9/2007 | | 243.94 |
| 8/9/2007 | | 243.96 |
| 9/9/2007 | | 243.89 |
| 10/9/2007 | | 243.88 |
| 11/9/2007 | | 243.76 |
| 12/9/2007 | | 243.67 |
| 13/9/2007 | | 243.58 |
| 14/9/2007 | | 243.51 |
| 15/9/2007 | | 243.44 |
| 16/9/2007 | | 243.34 |
| 17/9/2007 | | 243.29 |
| 18/9/2007 | | 243.19 |
| 19/9/2007 | | 243.08 |
| 20/9/2007 | | 242.96 |
| 21/9/2007 | | 242.84 |
| 22/9/2007 | | 242.74 |
| 23/9/2007 | | 242.64 |
| 24/9/2007 | | 242.54 |
| 25/9/2007 | | 242.44 |
| 26/9/2007 | | 242.35 |
| 27/9/2007 | | 242.23 |
| 28/9/2007 | | 242.11 |
| 29/9/2007 | | 241.99 |
| 30/9/2007 | | 241.85 |

| OKTOBER 2007 | | |
|---|---|---|
| 1/10/2007 | | 241.81 |
| 2/10/2007 | | 241.72 |
| 3/10/2007 | | 241.62 |
| 4/10/2007 | | 241.49 |
| 5/10/2007 | | 241.36 |
| 6/10/2007 | | 241.42 |
| 7/10/2007 | | 241.48 |
| 8/10/2007 | | 241.37 |
| 9/10/2007 | | 241.14 |
| 10/10/2007 | | 241.02 |
| 11/10/2007 | | 240.95 |
| 12/10/2007 | | 240.85 |
| 13/10/2007 | | 240.76 |
| 14/10/2007 | | 240.77 |
| 15/10/2007 | | 240.80 |
| 16/10/2007 | | 240.77 |
| 17/10/2007 | | 240.74 |
| 18/10/2007 | | 240.67 |
| 19/10/2007 | | 240.61 |
| 20/10/2007 | | 240.57 |
| 21/10/2007 | | 240.50 |
| 22/10/2007 | | 240.45 |
| 23/10/2007 | | 240.42 |
| 24/10/2007 | | 240.38 |
| 25/10/2007 | | 240.31 |
| 26/10/2007 | | 240.25 |
| 27/10/2007 | | 240.18 |
| 28/10/2007 | | 240.13 |
| 29/10/2007 | | 240.07 |
| 30/10/2007 | | 240.11 |
| 31/10/2007 | | 240.07 |

| November-07 | | |
|---|---|---|
| 1/11/2007 | | 240.03 |
| 2/11/2007 | | 240.03 |
| 3/11/2007 | | 240.06 |
| 4/112007 | | 240.11 |
| 5/11/2007 | | 240.12 |
| 6/11/2007 | | 240.10 |
| 7/11/2007 | | 240.17 |
| 8/11/2007 | | 240.16 |
| 9/11/2007 | | 240.14 |
| 10/11/2007 | | 240.12 |
| 11/11/2007 | | 240.10 |
| 12/11/2007 | | 240.11 |
| 13/11/2007 | | 240.15 |
| 14/11/2007 | | 240.20 |
| 15/11/2007 | | 240.25 |
| 16/11/2007 | | 240.28 |
| 17/11/2007 | | 240.31 |
| 18/11/2007 | | 240.33 |
| 19/11/2007 | | 240.39 |
| 20/11/2007 | | 240.45 |
| 21/11/2007 | | 240.55 |
| 22/11/2007 | | 240.60 |
| 23/11/2007 | | 240.64 |
| 24/11/2007 | | 240.66 |
| 25/11/2007 | | 240.68 |
| 26/11/2007 | | 240.70 |
| 27/11/2007 | | 240.70 |
| 28/11/2007 | | 240.71 |
| 29/11/2007 | | 240.68 |
| 30/11/2007 | | 240.67 |

Table 1.1: Paras air Tasek Temenggor bagi Bulan September, Oktober dan November pada jam 8.00pg

**( Bahan Rujukan dari TNB Gerik)**

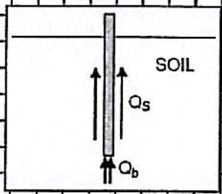
# TABLE 1.9

# CALCULATIONS OF PILE CAPACITY

| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 2.7m, Pile terminates at SPT-N=30) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

| REF. | CALCULATIONS | OUTPUT |
|---|---|---|

**Evaluation of Pile Capacity**




Type = R. C. Driven Pile

BH =

**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m

Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$

Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5

Factor of safety (End Bearing),$F_2$ = 3.0

| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 2.7 | 2.7 | 20 | 3 | 162.00 | | 0.00 | 162.00 | 108.00 |
| 2.7 | onwards | | 30 | | 162.00 | 400 | 750.00 | **912.00** | 358.00 |

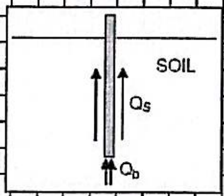
Structural capacity under working load > Geotechnical allowable capacity

750.00 kN > 358.00 kN

| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 2.7m, Pile terminates at SPT-N=50) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

REF. | CALCULATIONS | OUTPUT

**Evaluation of Pile Capacity**




Type = R. C. Driven Pile

BH =

**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m

Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$

Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5

Factor of safety (End Bearing),$F_2$ = 3.0

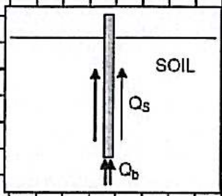
| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 2.7 | 2.7 | 20 | 3 | 162.00 | | 0.00 | 162.00 | 108.00 |
| 2.7 | onwards | | 50 | | 162.00 | 400 | 1250.00 | **1412.00** | 524.67 |

| Structural capacity under working load | > | Geotechnical allowable capacity |
|---|---|---|
| 750.00 kN | | 524.67 kN |

| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 8.7m, Pile terminates at SPT-N=30) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

| REF. | CALCULATIONS | OUTPUT |
|---|---|---|

**Evaluation of Pile Capacity**




**Type** = R. C. Driven Pile
**BH** =

**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m
Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$
Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5
Factor of safety (End Bearing),$F_2$ = 3.0

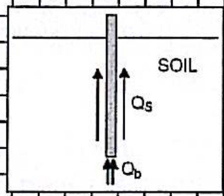
| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 8.7 | 8.7 | 20 | 3 | 522.00 | | 0.00 | 522.00 | 348.00 |
| 8.7 | onwards | | 30 | | 522.00 | 400 | 750.00 | 1272.00 | 598.00 |

Structural capacity under working load > Geotechnical allowable capacity
750.00 kN          598.00 kN

| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 8.7m, Pile terminates at SPT-N=50) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

| REF. | CALCULATIONS | OUTPUT |
|---|---|---|

**Evaluation of Pile Capacity**




Type = R. C. Driven Pile

BH =

**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m

Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$

Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5

Factor of safety (End Bearing),$F_2$ = 3.0

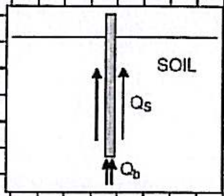
| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 8.7 | 8.7 | 20 | 3 | 522.00 | | 0.00 | 522.00 | 348.00 |
| 8.7 | onwards | | 50 | | 522.00 | 400 | 1250.00 | **1772.00** | 764.67 |

| Structural capacity under working load | < | Geotechnical allowable capacity |
|---|---|---|
| 750.00 kN | | 764.67 kN |

| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 22.2m, Pile terminates at SPT-N=30) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

| REF. | CALCULATIONS | OUTPUT |
|---|---|---|

**Evaluation of Pile Capacity**




Type = R. C. Driven Pile

BH =

**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m

Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$

Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5

Factor of safety (End Bearing),$F_2$ = 3.0

| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 22.2 | 22.2 | 20 | 3 | 1332.00 | | 0.00 | 1332.00 | 888.00 |
| 22.2 | onwards | | 30 | | 1332.00 | 400 | 750.00 | **2082.00** | 1138.00 |

| Structural capacity under working load | < | Geotechnical allowable capacity |
|---|---|---|
| 750.00 kN | | 1138.00 kN |

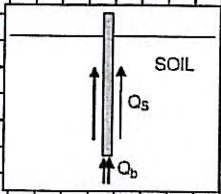
| CALCULATION SHEET | Project Title: | | | Job No. |
|---|---|---|---|---|
| | Serial No. | | | Sheet No. |
| Calculation Title: Evaluation of Pile Capacity using Meyerhof Equation (Pile Length = 22.2m, Pile terminates at SPT-N=50) | Design: CSH | Check: LCS | Review: GSS | Date: 12-Mar-08 |

| REF. | CALCULATIONS | OUTPUT |
|---|---|---|

**Evaluation of Pile Capacity**

Type = R. C. Driven Pile

BH =




**a) Structural Capacity of Pile (Working condition)**

Size = 250x250 m

Shape = S (C = CIRCLE , S = SQUARE)

Cross sectional area,Ab = 0.0625 $m^2$

Perimeter of the pile, As = 1.0000 m

Structural capacity of pile under working condition = 750 kN

**b) Geotechnical Capacity of Pile** $Q_{all} = (B_1NA_s)/F_1 + (B_2NA_b)/F_2$

Factor of safety (Skin Friction),$F_1$ = 1.5

Factor of safety (End Bearing),$F_2$ = 3.0

| DEPTH (m) | | $D_b$ (m) | SPT'N | $B_1$ | Total Skin Friction (kN) | $B_2$ | Total End Bearing (kN) | Total $Q_{ult}$ (kN) | Allowable $Q_{al}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 22.2 | 22.2 | 20 | 3 | 1332.00 | | 0.00 | 1332.00 | 888.00 |
| 22.2 | onwards | | 50 | | 1332.00 | 400 | 1250.00 | 2582.00 | 1304.67 |

| Structural capacity under working load | < | Geotechnical allowable capacity |
|---|---|---|
| 750.00 kN | | 1304.67 kN |