GEOTECHNICAL / GEOHYDROLOGICAL INVESTIGATION FOR THE NEW FIRESTATION ON FLAMINGO AIRPORT AT KRALENDIJK, BONAIRE

Geotron report nr. GC 1202

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1. INTRODUCTION

By order of Ministerie van BZK, Rijksgebouwendienst, Geotron Caribbean BV., onderzoeksbureau voor grond en grondwater, performed a geotechnical soil investigation for the new fire station on Flamingo Airport at Kralendijk, Bonaire.

In the framework of a geotechnical and geohydrological investigation we performed twelve borings to a maximum depth of 8m –ground surface [grade] and six Dynamic Penetration Tests to a maximum depth of 1.70m –ground surface [grade] on this particular location.

Borings and Dynamic Penetration Tests [DPT] are performed at the proposed location of the new fire station on Flamingo Airport, Kralendijk.

The scope of work here by is the following;

- Set out exploratory locations
- 6 DPT tests till a max. Blow count of 50 blows / 10 cm(predrilled).
- 6 boreholes till an estimated depth of 8 mbgl.
- 6 cavitie detection boreholes till an estimated depth of 5 mbgl.
- Sampling of the encountered rock.
- Measuring the stabilized water table[s] during drilling and the installation of monitoring wells in three borehole's to measure the groundwater with a dipper during the pumptest period.
- Execute one only pump test to determine the water permeability in the encountered substrata.
- Laboratory testing on selected samples [ia. UCS and Pointloads].
- Preparation of a full geotechnical report, incl. all appendixes concerning field and laboratory datasheets.

The lay out of the soil investigation is in accordance with the Dutch Standard: Geotechnics TGB-1990- Basic requirements and loads, standard NEN 6740 [In principle the centre to centre distance between each investigation point will be some 25 m].

The purpose of this report is to define the soil / rock conditions as encountered in the soil investigation, to analyze and evaluate the field test data obtained, and to submit recommendations regarding feasible foundation design and construction.

The following companies were involved in this project:

- Principal: Ministerie van BZK, Rijksgebouwendienst
- Geotechnical investigation & consultancy: Geotron Caribbean BV.

2. SOIL INVESTIGATION

2.1. General

The soil investigation complies with the actual standards [NEN, ASTM, British Standard and DIN].

The **Dynamic Penetration Test** [DPT] is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of a soil. This is a dynamic load test that uses a dynamic sounding technique to calculate the loading capacity and compaction properties of the soil. The test is used in situ, inter alia to control the compaction of backfills and embankments.

Between the Standard Penetration Test [SPT] and the Dynamic Penetration Test [DPT] in sandy material exists an empirical equivalence. Penetration resistance values from the two tests are often taken as equivalent for design purposes. When examine SPT and DPT resistance values from different geological depositional and weathering environments, it is apparent that energy losses are greater in the DPT test than in the SPT, leading to higher resistance values in the former. The SPT is carried out within a borehole, whereas the DPT is continuously driven into the soil. The dynamic force applied to the DPT rods causes soil to fill the small air annulus around the rods, exerting a frictional resistance. The different geological settings of a test site will reveal that, although different factors cause the friction, the equivalence varied in a similar manner.

The probe, equipped with a cone tip is driven into the ground by a 10-50 kilogram weight dropped freely from a height of 50 centimetres. The number of blows for penetrations per 10-20 centimetre layer is registered and converted into an average penetration per stroke. The data gathered is used to calculate the Californian bearing ratio for each depth of soil – internationally recognized measurement for the load-bearing capacity of soils used for roads, pavements and airstrips. To evaluate the density and consistency empirical relations are used, developed in America and Germany. In table 1 a global relation is given between the DPT/N_{10}-value and the consistency or density of the soil.

In this case the dynamic penetration tests [DPT's] were performed as continuous "Dynamic Probing Heavy" DPH15. For these DPT's a falling weight of 50 kg is used at a falling height of 0,50 m. The area of the cone used is 1500 mm^2. The N-values have been measured during the testing phase and are an indication for the quality of the subsoil layers. **It is noted that Geotron Caribbean BV. presents the results as N_{10}-values.**

Table 1. Global relation between DPH15 / N_{10}-value and consistency / density of the soil.

DPH15 N_{10}-value in clay (cohesive sediments)	Consistency	DPH15 N_{10}-value in sand (non-cohesive sediments)	Relative density
< 1	very soft	< 2	very loose
1 - 2	soft	2 - 5	loose
2 - 4	medium soft	5 - 15	medium dense
4 – 7.5	stiff	15 - 25	dense
7.5 - 15	very stiff	> 25	very dense
> 15	hard		

The borings are performed as rotary core drilling, whereby the coring device was an integral part of the drill rod string, and the latter serves as a casing. Core samples with a diameter of 62 mm were obtained by removing the inner barrel assembly from the core barrel portion of the drill rod. The HQ triple tube wireline coredrilling technique was used.

2.2. Fieldwork

The soil investigation was performed in June 2012 and consists of:

- Dynamic Penetration Tests S01 to S06 to a maximum depth of 1.70 m – ground surface [grade];
- Borings BH01 to BH06 to a maximum depth of 8 m – ground surface [grade];
- Cavity detection borings S01 to S06 till a maximum depth of 5 m-groundsurface [grade];
- Survey of the DPT- and Boring-grade levels. As reference level a fixed point VP with reference level 0.000 m± NSP, was used;
- Pump test in boring BH01 with measuring water table in dedicated monitoring wells;
- Visual inspection of the terrain surface, on sinkholes, fissures, water wells etc..

The locations of the boreholes and soundings are given on Annex 1. For the results of the field work see Annexes 2-4.

2.3Laboratory investigation

Core-samples for visual classification and point load testing were obtained during the execution of the borings.
On specific core samples of white voided coral we determine the Unconfined Compressive Strength with determination of E. Modulus. The lab testing was performed by Laboratory

Engineering Geology, University of Technology Delft in the Netherlands. The results of the UCS with E. Modulus are summarized in table 5 and admitted in Annexes 7.

3. RESULTS OF SOIL INVESTIGATION

On the basis of the soil investigation the soil conditions can be summarized as follows, see table 2:

Table 2. Soil conditions.

From m – grade	To m - grade	Soil description
0.00	8.00 [max. investigated depth]	**coral limestone, occasionally and rare clay**

From the performed survey it follows that the ground water table lies between 1.08 and 1.90 m - grade. These observations are instantaneous and are indicative only because layering and local conditions can have a disturbing influence. It can be expected that due to percolating groundwater, containing dissolved CO_2 [carbon dioxide], the coral limestone chemical solutes, forming cavities and sinkholes. In one only borehole [partly filled] cavities are detected, between 2.10 m – and 8.00 m – grade [BH02]; see table 3.

The results of Borehole logs are presented in Annex 3 and are summarized in table 3.

Table 3. Rock / soil recovery ratios L_r and RQD.

Boring number	Depth m - grade	Recovery L_r (%)	RQD (%)	Remarks
BH01	0.0 – 1.4	100	20	Cemented limestone, very dense
	1.4 – 2.4	100	33.3	Ditto
	2.4 – 2.5	95	25	Cemented limestone, very porous, medium dense
	2.5 – 3.9	100	33.3	Cemented limestone, weak porous, very dense
	3.9 – 8.0	95 100 100 100	25 50 50 40	Cemented limestone, weak porous, medium dense

Boring number	Depth m - grade	Recovery L_r (%)	RQD (%)	Remarks
BH02	0.0 – 0.9	100	25	Cemented limestone, weak porous, very dense
	0.9 – 2.1	85	40	Cemented limestone, medium porous, loose
	2.1 – 2.7	85	33.3	Loose parts of limestone, porous, **small cavities**, loose
	2.7 – 4.8	100	33.3	Cemented limestone, weak porous, very dense, some cavities
	4.8 – 6.3	85 75	33.3 25	Loose parts of coral, very porous, **many small cavities**, medium dense, loose
	6.3 – 8.0	100 100	33.3 23.3	Cemented limestone, medium porous, dense, **some cavities**
BH03	0.0 – 1.3	95	75	Light cemented limestone, weak porous, medium dense
	1.3 – 2.7	100	40	Light cemented limestone, medium porous, medium dense
	2.7 – 3.0	80	50	Loose parts of limestone, porous, loose
	3.0 – 4.1	100	33.3	Cemented limestone, weak porous, very dense
	4.1 – 5.4	100	33.3	Ditto
	5.4 – 8.0	85 95 95	50 33.3 33.3	Light cemented limestone, weak porous, medium dense
BH04	0.0 – 1.1	100	30	Cemented limestone, very dense
	1.1 – 1.9	100	20	Cemented limestone, weak porous, medium dense
	1.9 – 2.0			Cemented limestone, medium porous, medium dense
	2.0 – 3.0	75	50	Cemented limestone, very dense
	3.0 – 3.3			Loose parts of limestone, medium porous, medium dense
	3.3 – 3.5	80	33.3	**Clay**, pieces of coral, soft
	3.5 – 5.7	75	50	Cemented limestone, weak porous, very dense
	5.7 – 8.0	95 100 100	16.66 50 23.33	Cemented limestone, medium porous, very dense
BH05	0.0 – 0.3	65	37.3	Cemented limestone, very dense
	0.3 – 1.1			Loose parts of limestone, medium porous,

Boring number	Depth m - grade	Recovery L_r (%)	RQD (%)	Remarks
				medium dense
	1.1 – 2.2	100	40	Cemented limestone, medium porous, very dense
	2.2 – 2.8	100	33.3	Ditto
	2.8 – 3.1			Cemented limestone, medium dense
	3.1 – 4.6	100	33.3	Cemented limestone, medium porous, medium dense
	4.6 - 8.0	100 95 100 100	50 33.3 33.3 35	Ditto
BH06	0.0 – 1.25	97	37.5	Cemented limestone, very dense
	1.25 – 2.3	100	80	Ditto
	2.3 – 3.5	98	50	Cemented limestone, medium dense
	3.5 – 5.6	100 98	50 50	Cemented limestone, medium porous, medium dense
	5.6 – 8.0	95 100 100	50 33.3 35	Ditto
S01	0.0 – 1.3	100	30	Cemented limestone
	1.3 – 1.5			Ditto
S02	0.0 – 1.0	100	25	Ditto
	1.0 – 1.5			Ditto
S03	0.0 – 0.5	100	30	Ditto
	0.5 – 0.7			Ditto
S04	0.0 – 1.4	100	35	Ditto
	1.4 – 1.5			Ditto
S05	0.0 – 0.9	100	25	Ditto
	0.9 – 1.5			Ditto
S06	0.0 – 0.5			Ditto
	0.5 – 1,45	100	25	Ditto
	1,45 - 1,5			Ditto

L_r = recovery = [Length of sample] / [Length of core advance].

RQD = Rock Quality Designation = [Σ Length of intact pieces of core > 100 mm] / **[**Length of core advance].

RQD Rock mass quality
<25%very poor
25-50%poor

50-75%fair
75-90%good
90-100%excellent

Like mentioned before; in one only borehole [ia. BH02] we did find cavities in different sizes, but all detected as very small to small. The approximately sizes are noted as followed; very small 1-5 cm, small 5-10 cm, medium 10-30 cm and large 30-100 cm. After this it become caves / sinkholes.

From the RQD index the rock mass can be classified as very poor to fair rock. In general the quality [RQD-factor] of the cap rock formation [Klip] can be classified as poor [classification according Deere et al., 1967]. Some spots have a very poor formation quality with a relative low RQD-value.

Part of the present research aims to gain insight into the water permeability of the substrate. This in context of sustainable building and designing with rainwater. The permeability is the ability of soil to let through liquid or gas. The passage factor [k] is a measure of the permeability to water and can be defined as a proportionality factor in Darcy's law for water flow in the ground.

In order to determine the water permeability of the Limestone, one can use a variety of methods, such as;

- ex-situ, off-site, laboratory tests [for instance constant or falling head test, depending on the substrata];
- in-situ, on-site, field testing [pump test on / in boreholes, piezometers, Slug-test or inverse borehole method, Guelph permeameter or Cone permeameter].

By means of a pump test in borehole BH01, on the south east part of the premises, the research took place. The results of the water permeability test [pump test; elaboration k-value by Hooghoudt] are fully included in annex 4 and shown in the table below.

Table 4. Results permeability tests, June 2012.

Borehole number	**Groundwaterlevel (m –grade)**	**Pumpsection (m –grade)**	**k-value (m/d)**	**k-value (m/s)**
BH01	1.90	1.90 - 8.00	450.8778	$5.20 \cdot 10^{-3}$

During the test no decrease from the water levels could be registered in adjacent boreholes.

The value observed in the field, correspond to the values of K as stated in literature for **coarse sand with fine gravel; 1000-100 m / day.** The passage ratio can especially in fluvial deposits differ in horizontal and vertical direction [anisotropy]. This should in certain cases be taken into account. The let-through element is not always constant in time but may change, for example by compaction, swelling and shrinkage as well as by utilization of the adsorption complex.

If the results of the permeability tests are related to the classification according to the following table and the technical culture guide, we can conclude that the substrate, in the saturated zone [1.90 to 8.00m –grade] on the investigated location / position at the new fire station on Flamingo Airport has a **very good water permeability**. The drainage characteristics are also good [high permeability classification and good drainage characteristics].

Table 5. Permeability and drainage characteristics of main soil types [coefficient of permeability in m/s]

<table>
<tr><td colspan="2">k = □0</td><td>-1</td><td>-2</td><td>-3</td><td>-4</td><td>-5</td><td>-6</td><td>-7</td><td>-8</td><td>-9</td><td>-10</td><td>-11</td><td>-12</td></tr>
<tr><td colspan="2">Drainage Characteristics</td><td colspan="6">GOOD</td><td colspan="2">POOR</td><td colspan="4">PRACTICALLY IMPERVIOUS</td></tr>
<tr><td colspan="2">Permeability Classification</td><td colspan="3">HIGH</td><td colspan="2">MEDIUM</td><td colspan="2">LOW</td><td colspan="2">VERY LOW</td><td colspan="3">PRACTICALLY IMPERMEABLE</td></tr>
<tr><td colspan="2" rowspan="2">General Soil Type</td><td colspan="2" rowspan="2">GRAVELS</td><td colspan="2">CLEAN SANDS</td><td colspan="4">FISSURED AND WEATHERED CLAYS</td><td colspan="4" rowspan="2">INTACT CLAYS</td></tr>
<tr><td colspan="6">VERY FINE OR SILTY SANDS</td></tr>
<tr><td rowspan="2">Test Methods</td><td>Direct</td><td colspan="2">Large CH cell</td><td colspan="3">Standard CH cell</td><td colspan="4">FH cell</td><td colspan="2">FH in Oedomtr.</td><td></td></tr>
<tr><td>Indirect</td><td></td><td colspan="4">Computation from PSD</td><td colspan="4"></td><td colspan="3">From consolidation data</td></tr>
</table>

CH = Constant Head, **FH** = Falling Head, **PSD** = Particle Size Distribution Analysis

The new fire station will have a roof of approximately 2,300 square meters, due to the tilted ground surface [towards the sea] and the run off by rain gutters this surface will be lost for water storage in the soil. One could collect the rain water for fire fighting use [!?]. By leading the water to a clarifier the fresh water could be reserved. The excess water can / should be infiltrated in the subsoil.

Normally one can say that infiltration of rainwater is interesting if:

- the permeability is greater than about 0.4 meters / day;
- the bottom of the infiltration facility is prevailing at least 1 meter above the water table.

The first and most important parameter is amply satisfied. In this case the effective infiltration should take place in the saturated soil, by for instance a borehole until 10m –grade with a diameter of 200 mm.

Core-samples for visual classification and point load testing were obtained during the execution of the borings.

On specific core samples of white voided coral we determine the Unconfined Compressive Strength with determination of E. Modulus. The lab testing was performed by Laboratory Engineering Geology, University of Technology Delft in the Netherlands. The results of the UCS with E. Modulus are summarized in table 7 and admitted in Annexes 7.
The results of the pointload tests are summerised in table 6.

Table 6. Results of the Point Load Tests on limestone cores.

Boring	Depth	Corediameter [mm]	Intrusion [mm]	Point load strenght [MPa]
BH01	4.50-4.70	63.0	10.0	2,32
	5.30-5.50	63.0	30.0	2,75
	7.50-7.75	62.8	11.5	1,49
BH02	2.70-2.90	63.0	7.5	4,2
	3.30-3.50	63.1	17.0	6,55
	4.00-4.25	63.1	23.1	3,89
	7.60-7.80	63.0	22.1	3,16
BH03	0.50-0.60	62.3	14.0	1,23
	2.20-2.35	62.8	13.5	2,38
	4.10-4.30	63.1	9.3	2,74
	4.30-4.50	63.2	9.6	2,54
	6.60-6.80	62.7	4.0	0,64
	7.50-7.65	63.1	15.0	2,16
BH04	1.40-1.55	62.8	14.0	2,4
	3.40-3.55	62.8	14.0	3,19
	4.10-4.25	63.5	15.0	4,23
	6.30-6.50	62.7	16.2	2,56
	7.10-7.30	63.2	10.1	3,60
BH05	1.00-1.20	62.7	0.0	0,59
	3.00-3.85	63.0	10.0	1,74
	4.35-4.50	62.7	9.5	1,45
	6.75-6.95	63.2	16.1	3,2
	7.80-8.00	63.1	12.0	2,42
BH06	2.80-3.50	62.8	12.0	2,94
	4.30-4.45	63.0	19.0	2,59
	5.50-5.70	63.2	11.0	2,69
	6.85-7.00	62.5	23.0	3,72
	7.75-7.95	62.9	4.0	1,28

Table 7. Test results of the UCS with E. Modulus

Sample Number	Depth m. - grade	Unconfined Compressive Strength MN / m2	E. Modulus Tangent 50% [GPA]
Bon BH01	0.30 – 0.45	11.83	9.01
Bon BH02	1.30 – 1.50	12.62	11.82
Bon BH03	3.40 – 3.60	31.47	28.2
Bon BH04	0.35 – 0.50	15.27	10.38
Bon BH05	2.60 – 2.75	7.80	15.69
Bon BH06	1.85 – 2.00	3.00	4.15

With UCS results in the range 3.00 and 31.47 MPa the quality of the cap rock formation [Klip] can be classified as Weak [1.25 – 5.00 MPa] over Moderately weak [5.00 -12.50 MPa] to Moderate strong [12.50 – 50 MPa; classification according to Geological Society of London, 1977]. ***For illustration purposes only, the end bearing capacity of piles driven into soft rock, indication of ultimate capacity is 6 x UCS***.

As the several calculation methods are based on CPT-value, the table mentioned below reproduce the relation between DPT-values and the CPT-values.

Table 8. Relation between DPT-values and CPT-values.

DPT	S01 DPH/CPT*	S02 DPH/CPT*	S03 DPH/CPT*	S04 DPH/CPT*	S05 DPH/CPT*	S06 DPH/CPT*
Depth (m -grade)						
1.00	predrilled	predrilled	predrilled	predrilled	predrilled	predrilled
1.50	61 – 33.65	45 – 24.82	56 – 30.89	54 – 29.79	48 – 26.48	65 – 35.86
2.00	75 – 40.05	75 – 40.05	75 – 40.05	81 – 43.25	75 – 40.05	70 – 37.38
2.50						

- *DPT -H as N-value / CPT in MPa [dynamic cone resistance]*
- *Dutch drive formula; q_d = [(0,833xN10)/1,36 + 0,1d]*

From the Dynamic Penetration Tests [ia. very dense] and the point load tests it follows that the coral limestone cap [Klip] itself consists of a medium strong rock with very weak zones and pockets.

The boring and DPT-grade-levels were surveyed. As reference level a fixed point VP with reference level 0.000 m± NSP, was used. The results of the survey are summarized in table 9.

Table 9. Topographic survey of the borings and DPT -grade levels.

BH	Survey level (m in relation to reference level VP)	Water level (m in relation to reference level VP)
S01	1.85	
S02	1.95	0.50
S03	2.21	1.13
S04	1.95	
S05	1.86	0.78
S06	2.10	
BH01	2.13	0.23
BH02	2.01	0.37
BH03	1.97	0.17
BH04	1.77	0.32
BH05	1.37	0.29
BH06	2.31	0.80

The results of the topographic survey indicate that the ground surface of the investigation points dips roughly from east to west / north west. Groundwater flows roughly from north / north west to south .

4. SITE GEOLOGY

The heart of the island consists of strongly folded and faulted rocks of volcanic origin, silica rich sediments and turbidites, all formed during the Cretaceous era some 120 million years BP [Beets, 1972a,1972b]. Overlying this are later fossil reef and reef-generated calcareous deposits. It are these calcareous formations which make up the coastline in the form of coral-rubble beaches [coral shingle and calcareous sand] or iron shore, except in the north where low limestone cliffs are found [Zonneveld et al., 1972].

Most of the older rock formations of Bonaire developed underwater, in times when Bonaire was not yet an island [90-100 million years ago]. After the emergence of these formations [60-70 million years ago] two important events started to happen:

1 The rocks became exposed to erosion, the debris was carried off by rain water and so hills, valleys and plains developed;
2 Around the island, coral started to grow, waves attacked the shores and the wind accumulated [coral] sand, as a consequence of which, dunes developed. In some places the dunes moved far inland.

On Bonaire, one can see two vastly different types of hard rock that tell us about the origin and subsequent history of the island. We observe dark rocks, either unstratified or thinly laminated, in the last case the strata are tilted, and dip 35 to 40 degrees down to the Northeast. Very different are yellowish or white limestone deposits in sub horizontal position. The dark rocks date back from the Cretaceous period, that ended 65 millions years ago. The light-coloured limestone's are much younger, their formation started only about 5 million years ago.

Most rocks have not been seen silently deposited as settling dust on the dark ocean floor. The tops of the Brandaris [240m.], Ceru'i Mangel [149 m.] and other hills consist of dark rock with white feldspar crystals. These rocks show a peculiar way of prismatic columnar jointing; there is no stratification at all. Such type of stones constitute the bulk of Bonaire's Cretaceous backbone. They certainly are ancient volcanic; chilled lavas, petrified tuffs. The rounded pillows in which lava flows are that are seen on Bonaire betray that there must have been a series of submarine volcanoes.

Younger strata are not tilted, we find them at most places along the coast and a close look tells us that these creamy-white limestone's originated as reefs. We find branching coral and [chalky white] coralline algae that prove this contention. As nearly all organisms found petrified in these reefs are still existing, their age can not be great. Radio-carbon gives an age of 110.000 years. In those times the sea stood about 15 meters higher than it does now. The reef has been preserved as a terrace that fringes the island's coastlines. At some places even two or three terraces are present, since the sea has known very different levels, depending whether the polar ice caps retained more of less rain pour.

Most of the rivulets [temporary water streams] do not empty directly into the sea. Some empty into island bays or Salinas, salt lakes, the bottoms of which have sometimes partly dried up [eg. Saliña Matijs and Boca Bartol]. These bays are drowned valleys. When, in former times, the sea level was lower than at present, the rivers could erode their valleys to a lower level than the present sea level. When the water rose, the lower part of the valleys became submerged. Still later the mouth was blocked by walls of coral shingle put there by the surf. The well known Goto and Slagbaai owe their existence to such a process.

Substantial changes in sea level have left up to four stranded terraces above the present mean sea level, and one below, most of which can be distinguished by "solution notches" [undercutting caused by bio erosion and / or wave action] in the elevated seaward facing limestone cliffs.

As far as we know the coastal area near Morotin consists of the Lower Terrace limestone formation, overlying the Washikemba formation, the latter mainly formed from porphyrites and tuffs.

The limestone is highly permeable, and karstic phenomena have developed in many places. At some locations, the limestone rests upon the basement of the crystalline rock, and sometimes a deep bed of alternating sand and clay layers underlies the limestone. Small aquifers may occur in the sands and clays of the limestone regions that are often saturated with brackish or seawater, especially at sites near the coast [De Buisonjé, 1974].

The volcanic and crystalline rocks are nearly impermeable [Abtmeier, 1978]. They may show permeability along fracture lines and in the weathered zone.

Rather than a regional system flowing under a regional water-table regime, there is isolated flow through a number of fracture systems where the position of the water table is governed by the geometric and hydraulic characteristics of each system [Grontmij & Sogreah, 1968].

Temperature of the groundwater is generally between 30^0C and 31^0C [The groundwater quality of Aruba, Bonaire and Curaçao: a hydrogeochemical study, M.H.G. van Sambeek, H.G.M. Eggenkamp & M.J.M.Vissers, 2000].

5. STRUCTURAL DATA

On a dedicated site on the Flamingo Airport at Kralendijk the building of a new fire station will take place. This project consists mainly of the realization of a one-story-building with exception of the office part which will be an two-story-building. The masonry inside and out will consist of concrete stone.

Few data on the loading conditions are available:

- Held permissible ground pressure; max. 120 kN / m2;
- Foundation will take place on beams and slabs with erosion edge.

Further details are yet unknown.

In addition to the purely gravity loads, the structures will be required to carry loads from wind [lateral forces plus overturning moments] as well as seismic forces from earthquakes.

For the architectural engineer hereby applies that new buildings should / will be hurricane resistant. The requirement for the main return period in this case will be 50 years. In accordance with the recommendation from TNO, this means one has to calculate with double thrust as indicated in the TGB 1990 [table 10, region 1 no buildings]. In this context we also refer to the TNO report 96-CON-R0781-3 'Hurricane Hazard at the Netherlands Antilles and wind loads on buildings'.

6. SEISMICITY

Seismic activity in the Caribbean is strongly concentrated along the Caribbean plate boundaries. In the south, east and north the South American- and North American plate boundaries, and in the west the Cocos plate boundary. The Caribbean plate is overriding the North and South American plates, while on the west, the Cocos plate is subducting beneath the Caribbean plate. The southern boundary of the Caribbean plate has complicated multiple transform faults and a relatively low seismic hazard. Bonaire , Latitude 12^0 N, is situated 220 km north of the main southern transform fault [Latitude approximately 10^0 N].

From the ANSS–earthquake –catalogue it follows that in the vicinity of Bonaire the level of seismicity in the past was lesser than M < 5.9. Several quakes were recorded in the vicinity of Bonaire in the period of 1966 - 2007. Table 10 summarizes some characteristic earthquakes in the region [from 10^0 50´ N / 67^0 W to 13^0 30'N / 70^0 W = 330 km x 275 km], starting from 1966. These finding correspond with the North Andean Region data from CERESIS, 1996 [Centro Regional de Sismología para América Latina].

Table 10. Characteristic earthquakes in the Bonaire region [source ANSS, 2007].

Date	Latitude	Longitude	Location	Distance (km)*	Magnitude (Richter scale)
1989/04/30	10.9600 N	68.325 W	Mainland Venezuela, south of Bonaire	129 km	M 5.9
1977/7/24	10 49 25.38 N	68 47 29.66 W	Mainland Venezuela, south south west of Bonaire	155 km	M 5.4
1989/5/4	11 02 45.79 N	68 16 39.88 W	Golfo Triste, south of Bonaire	120 km	M 5.4
1975/4/5	13 00 30.28 N	68 00 24.88 W	Caribbean sea, north north east	101km	M 5.3
1997/4/15	10 42 16.65 N	69 29 25.12 W	Mainland Venezuela, south west of Bonaire	206km	M 5.2
1970/5/19	10 53 29.87 N	68 55 53.46 W	Mainland Venezuela, south west of Bonaire	155 km	M 5.1
1966/9/9	10 52 27.48 N	69 23 01.97 W	Mainland Venezuela, south west of Bonaire	184 km	M 5.0
1986/07/19	10.9170 N	69.3940 W	Mainland Venezuela, south west of Bonaire	182 km	M 4.9
1989/04/30	11.2970	68.0710 W	Golfo Triste, south of Bonaire	95 km	M 4.9
1968/05/24	10.9440 N	69.3690 W	Mainland Venezuela, south west of Bonaire	177 km	M 4.8
2003/11/16	12.9240 N	68.000 W	Caribbean sea, north north east	92 km	M 4.8
1983/12/20	12 10 45.41 N	69 48 16.16 W	Caribbean Sea, west of Bonaire	166 km	M 4.8
1970/06/08	10.860 N	69.0520 W	Mainland Venezuela, south west of Bonaire	164 km	M 4.7
1979/03/22	11 01 13.08 N	69 10 00.74 W	Mainland Venezuela,	157 km	M 4.6

Date	Latitude	Longitude	Location	Distance (km)*	Magnitude (Richter scale)
			south west of Bonaire		
1989/04/30	11.7580 N	68.4160 W	Caribbean Sea, south of Bonaire	44 km	M 4.6
1989/04/30	11.2380 N	68.0530 W	Golfo Triste, south of Bonaire	102 km	M 4.6
1992/04/24	10.9500 N	69.5800 W	Mainland Venezuela, south west of Bonaire	193 km	M 4.6
1988/06/20	10.9690 N	69.3050 W	Mainland Venezuela, south west of Bonaire	170 km	M 4.6
1986/09/12	10.860 N	69.3910 W	Mainland Venezuela, south west of Bonaire	186 km	M 4.6
1980/11/17	10.8600 N	69.5130 W	Mainland Venezuela, south west of Bonaire	195 km	M 4.6
2007/02/03	11.0810 N	69.3260 W	Mainland Venezuela, south west of Bonaire	163 km	M 4.5
1966/06/20	11.2000 N	69.3000 W	Mainland Venezuela, south west of Bonaire	152 km	M 4.4
2006/07/15	12.3300 N	69.9200 W	Caribbean Sea, west of Bonaire	180 km	M 4.4
1991/12/21	12 09 36.42 N	69 30 58.52 W	Caribbean Sea, west of Bonaire	135 km	M 4.3
1997/03/04	10.9690 N	69.6110 W	Mainland Venezuela, south west of Bonaire	194 km	M 4.3
2005/10/19	11.3700 N	69.6100 W	Mainland Venezuela, south west of Bonaire	168 km	M 4.3
1984/11/13	10.9750 N	69.3970 W	Mainland Venezuela, south west of Bonaire	177 km	M 4.2
1997/3/7	11 03 12.92 N	69 27 18.28 W	Mainland Venezuela,	176 km	M 4.2

Date	Latitude	Longitude	Location	Distance (km)*	Magnitude (Richter scale)
			south west of Bonaire		
1968/09/02	10.9570 N	69.3570 W	Mainland Venezuela, south west of Bonaire	175 km	M 4.1
1966/09/12	11.2000 N	69.2000 W	Mainland Venezuela, south west of Bonaire	144 km	M 4.0
1967/10/23	11.2000 N	68.9000 W	Mainland Venezuela, south west of Bonaire	124 km	M 4.0
2006/10/28	11.4200 N	69.6000 W	Mainland Venezuela, south west of Bonaire	164 km	M 4.0
1967/03/11	11.0840 N	69.6610 W	Mainland Venezuela, south west of Bonaire	190 km	M 3.9
2004/11/20	11.7860 N	69.3580 W	Caribbean sea , south west west of Bonaire	124 km	M 3.9
1987/04/22	11.4990 N	69.6000 W	Mainland Venezuela, south west west of Bonaire	160 km	M 3.7

2003/12/15	11.8050 N	68.1800 W	Caribbean sea , south south east of Bonaire	38 km	M 3.7
1987/04/22	11.4990 N	69.6000 W	Caribbean sea , south west of Bonaire	160 km	M 3.7
2003/12/15	11.8050 N	68.1800 W	Caribbean sea , south south east of Bonaire	38 km	M 3.7
1993/03/02	10.9970 N	69.5570 W	Mainland Venezuela, south west of Bonaire	187 km	M 3.3
1993/06/17	11.1430 N	68.3060 W	Golfo Triste, south of Bonaire	110 km	M 3.3

* distance to Flamingo Intl. Airport. [12 07 58.02 N & 68 16 30.22 W]

CERESIS studied the earthquake hazard for South America and considered the adjoining mainland of Venezuela to have a moderate hazard for earthquakes. The Peak Ground Acceleration [PGA] for the most northern part of the Gulf of Venezuela, the peninsula of Paraguana, is assessed on 0.8 to 1.6 m/sec^2, having 10% probability of exceeding in 50 years. This acceleration range corresponds with a return period of 475 years. [Source: MIDAS, Seismic Hazard for the North Andean region & USGS]. See also figure 1.

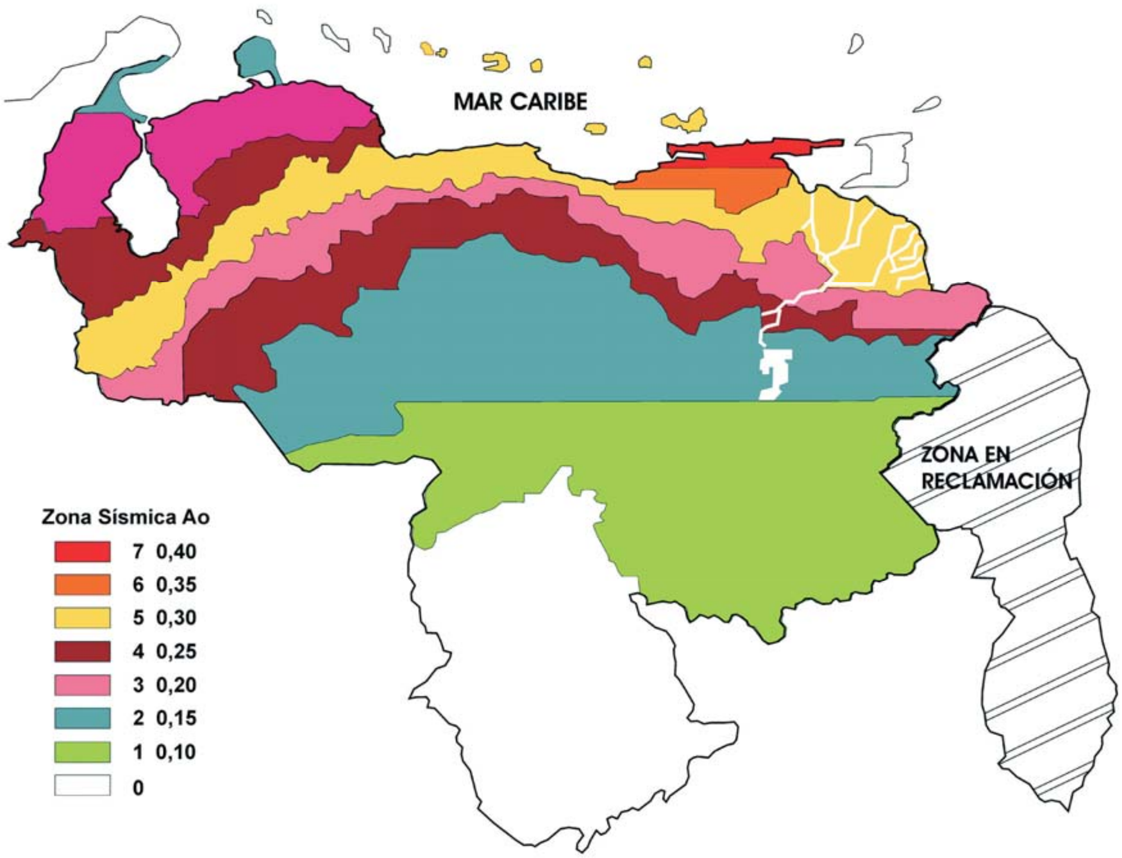


Figure 1: Seismic zone Venezuela [1998; reviewed 2001].

Unfortunately local specific PGA-data for the island of Bonaire are not available. However, given the short distance to the mainland of Venezuela and the fact that there are none significant geological changes between the mainland of Venezuela and Bonaire, there are in our opinion no reasons to change the Venezuelan PGA-levels. The Venezuelan Seismic Code of 1987, COVENIN 1756-87, gives a peak acceleration of 0.15 g [Seismic zone 2] for the peninsula of Paraguana. Because the distances between the principal faults on the main land and Paraguana / Bonaire are approximately the same, it is recommended to use in the development of the seismic response spectrum an a_g factor of 0.15g.

The soil underneath the foundation level can be classified as profile S1 of the Venezuelan Seismic Code; Rock; hard and or dense soil with a depth of less than 50 meters above the rock base.

Table 11: Relation between the Magnitude and the PGA.

Richter scale	2.0-2.9	3.0-3.9	4.0-5.9	6.0-6.9	7.0 and higher
PGA [g's]	0.17-0.27	0.27-0.35	0.35-0.59	0.59-0.65	0.65-1.00
Ground effects	Hardly felt, but detectable by seismometer	Vibrations akin to the passing of a big truck	Slight damage; windows and chimneys break	Major damage in old or substandard buildings	Many buildings shift off their foundations

PGA peak ground acceleration [in G-forces] measures how hard the ground shake during a tremor

[source; U.S. Geological Survey].

To meet the International Building Code, a structure must be able to withstand two-thirds of the force of an earthquake with a 2 percent chance of occurring in 50 years. It's up to local officials to adopt these provisions and ensure that builders follow them.

For the purposes of the evaluation of seismic site class, we employed the definitions of Ground Types described in table 16-J-Soil Profile Types of the Uniform Building Code , dated 1997. The Code defines six categories of ground types, designated S_A through S_F.

Considering the nature of materials encountered in the soil investigation, the applicable Ground type / Soil Profile Type for the site, underneath the recommended footing depth, is S_C. This profile is described in the Code [UBC97] as follows: "Very Dense Soil and Soft Rock" with an average SPT value of >50 blows / foot and / or average undrained shear strength of >100 kPa over the top 30 m. *In one only borehole [ia. BH06] we encountered a Ground type / Soil Profile Type, S_E. And this profile is described in the Code [UBC97] as follows: "Soft Soil Profile" with an average SPT value of <15 blows / foot and / or average undrained shear strength of <50 kPa over the top 30 m.*

Due to the results of the geotechnical investigation we recommend a shallow foundation.

7. FOUNDATION DESIGN

7.1General

Strength and stiffness parameters static

Below, two approaches are used in order to derive the soil parameters for the designated soil zone, eg.:

- An interpretation of the dynamic probing DPH, using known correlations for sandy soils;
- Oedometer testing in the laboratory on undisturbed core samples.

DPH-soundings:

Power [1982] presented a classification of chalk grades based on a comprehensive study with CPT-testing in various types of chalk. His classification is based on q_c [cone resistance] and R_f [friction ratio]. Because the measured R_f values are typically for sandy soils [0.75% to 2 %], it is here assumed that the chalk will behave as a cohesion less material. Further he derives a correlation between SPT N-values and q_c.

The following approach was chosen:

The DPH15-N_{10} values were "translated" to SPT-N_{30} values, using the DIN 4094 method for single topped sand. After that, the Power correlation [1982] is used to derive the CPT cone resistance q_c, where after the oedometer stiffness E_{oed} and friction property □' could be calculated in accordance with the DIN method for single topped sands [SE-classification] using the CPT values. The used formula and calculation results are given below:

$N_{30} = 1.65 * N_{10} - 2$ Sand, single graded; DIN 4094

Above groundwater: $N_{30} = 1.4 * N_{10}$

$\frac{q_c}{N_{30}} = 0.5$ Chalk; Power, 1982

$v = 167 * \log(q_c) + 113$ Sand, single graded; DIN 4094

$w = 0.5$ Sand, single graded; DIN 4094

$E_{oed} = v * p_a * \left(\frac{200}{p_a}\right)^w$ Sand, single graded; DIN 4094

$\varphi ' = 13.5 * \log(q_c) + 23$ Sand, single graded; DIN 4094

In view of the encountered subsoil conditions, the following foundation solution is recommended:

- A shallow foundation in the most upper part of the overburden [Sand and Limestone], with when necessary superficial **ground improvement**, with for instance Kalichi / granite.

In the following paragraphs recommendations for the foundation design are given. They are based upon the standards:

DIN 4019 I, II — Analysis of settlements for shallow foundations;
DIN 4017 I, II — Analysis of bearing capacity for shallow foundations.

7.1.1 Large-scale ground improvement / egalisation

In this case a high quality ground improvement [ia. the excavated / increased / leveled soil], with for instance Kalichi / granite, to a maximum depth / height of 1.70 m +NSP is recommended. In the proposed building site the whole ground / overburden will be leveled, excavated and / or increased, if dry, with the encountered limestone or comparable hard rocky and granular material and compress it. If the excavation / hard layer is located below the groundwater level, lean concrete or flowable fill or similar will be used until the groundwater level and than advance with limestone and compress the latter. The use of diabase near the groundwater level is not recommended due the possible disintegration under the influence of water.

On the basis of the soil investigation the excavation / increasing / leveling depth can be summarized as follows, see table 12:

Table 12. Construction level shallow foundation.

Boring / DPT	Elevation (m +NSP)	Level (m +NSP)	Construction level foundation (m +NSP)	Excavation / increasing / levelling depth (m +NSP)
BH01	2.13	2.000	1.70	1.70
BH02	2.01	2.000	1.70	1.70
BH03	1.97	2.000	1.70	1.70
BH04	1.77	2.000	1.70	1.70
BH05	1.37	2.000	1.70	1.70

Boring / DPT	Elevation (m +NSP)	Level (m +NSP)	Construction level foundation (m +NSP)	Excavation / increasing / levelling depth (m +NSP)
BH06	2.31	2.000	1.70	1.70
S01	1.85	2.000	1.70	1.70
S02	1.95	2.000	1.70	1.70
S03	2.21	2.000	1.70	1.70
S04	1.95	2.000	1.70	1.70
S05	1.86	2.000	1.70	1.70
S06	2.10	2.000	1.70	1.70

7.1.2 Bearing capacity for a shallow foundation

The footings can be placed in the improved overburden [ia. the excavated / increased / leveled soil] on a shallow depth of 0,30 m minimum. A thorough inspection of the excavation levels is necessary. Soft spots / areas with low bearing capacity should be excavated deeper and filled for instance with lean concrete. Cavities and sinkholes, found at the bottom of the excavation should be filled with grout prior to foundation construction.

The [ultimate] bearing capacity for a shallow foundation have been determined, based upon the following assumptions:

Surcharge : 0.3 m soil or 0.2 m concrete as a minimum.
Pad footing : $1 * 1\ m^2$ to $4 * 4\ m^2$.
Strip footing : width 0.5 m to 4.0 m.
length 6 m
Groundwater >1 m - grade
Vertical loading : H / V = 0
Inclined loading : H / V = 0.33
Inclined loading : H / V = 0.5
Centric loaded footing.
Horizontal ground level.

The soil properties are assessed in accordance with general engineering practice, and represent low [5%] representative parameters.

Table 13. Soil schematization in behalf of a shallow foundation.

Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

The acceptable values of the bearing capacity and the assessed settlement characteristics are given on Annexes 5 and 6.

The following conditions were calculated:

- Pad footing with a [length] = b (width), see Annex 5;
- Strip footing with a [length] = 6 m, see Annex 6.

The load inclinations vary between 0 (H / V), 0.33 (H / V) and 0.5 (H / V). *See Annexes 5.1 to 5.3, respectively Annexes 6.1 to 6.3.*

The following partial safety factors were applied:

- Material factor γ_{GR} = 1.4
- Load factors γ_G = 1.35 [permanent action]
  γ_{QQ} = 1.50 [changeable action]

Ratio of changeable [Q] and total loads [G+Q] = 0.50

The results of the calculations with an explanation of he used symbols are given on Annexes 5 and 6.

When other values for the load factors are considered to be more appropriate, a linear interpolation of the bearing capacity design values is allowed.

In the following, a worked example is given for a vertical and centric loaded, strip footing *[see Annex 6.1]:*

Strip footing 2.5 x 6.0 m^2.

Surcharge 0.3 m soil [minimum embedment].

- Maximal acceptable bearing pressure 260 kPa [kN/m^2]; having an overall factor of safety of approximately 1.4 x (1.35 + 1.5) / 2 = 2.0, a settlement of 12 mm, and a modulus of subgrade reaction equal 21.2 MN/ m^3.
- Bearing pressure 150 kPa; overall factor of safety >> 2.0, settlement 7 mm, modulus of subgrade reaction 21.2 MN/ m^3.

In case of load-eccentricity the following approach can be used:

b_{eff} = effective foundation width, (= b – 2 e – 2x)

b = foundation width

e = eccentricity of the vertical loading component, distance to the centre of the foundation

x = M / V

M = external moment

V = vertical component of load

$\sigma_{allowable}$ = value of the bearing capacity (foundation pressure), working on the effective foundation width.

To determine the resistance to sliding, a friction value [tang((ϕ_d)] of 0.60 [calculation value with $\gamma_{m;\phi'}$ = 1.15] is appropriate.

The settlement calculations were performed for the serviceability limit state, using a material factor equal 1.0 [oedometer or constrained modulus] and actual loading pressures. On Annexes 5 and 6 the coefficients of subgrade reaction k_s , depending on the foundation print [a = footing length, b = footing width], are given.

The settlement behavior of the subsoil will be nearly elastic and the amount of consolidation- and secondary-compression will be negligible. Therefore a substantial amount of the assessed settlement will occur during construction.

The footings on the outer borders of the building need a surcharge of at least 0.4 m soil.

Upon request specific bearing capacity and [rotational] settlement calculations can be performed.

Annex 1:
Location of
Dynamic Penetration Tests and boreholes (in 1 page)

WIJZ.	DATUM	OMSCHRIJVING	GET	GEC	STATUS
A	MEI-2012	SITUATIE / HOOGTEMETING	EB	RG	FINAL

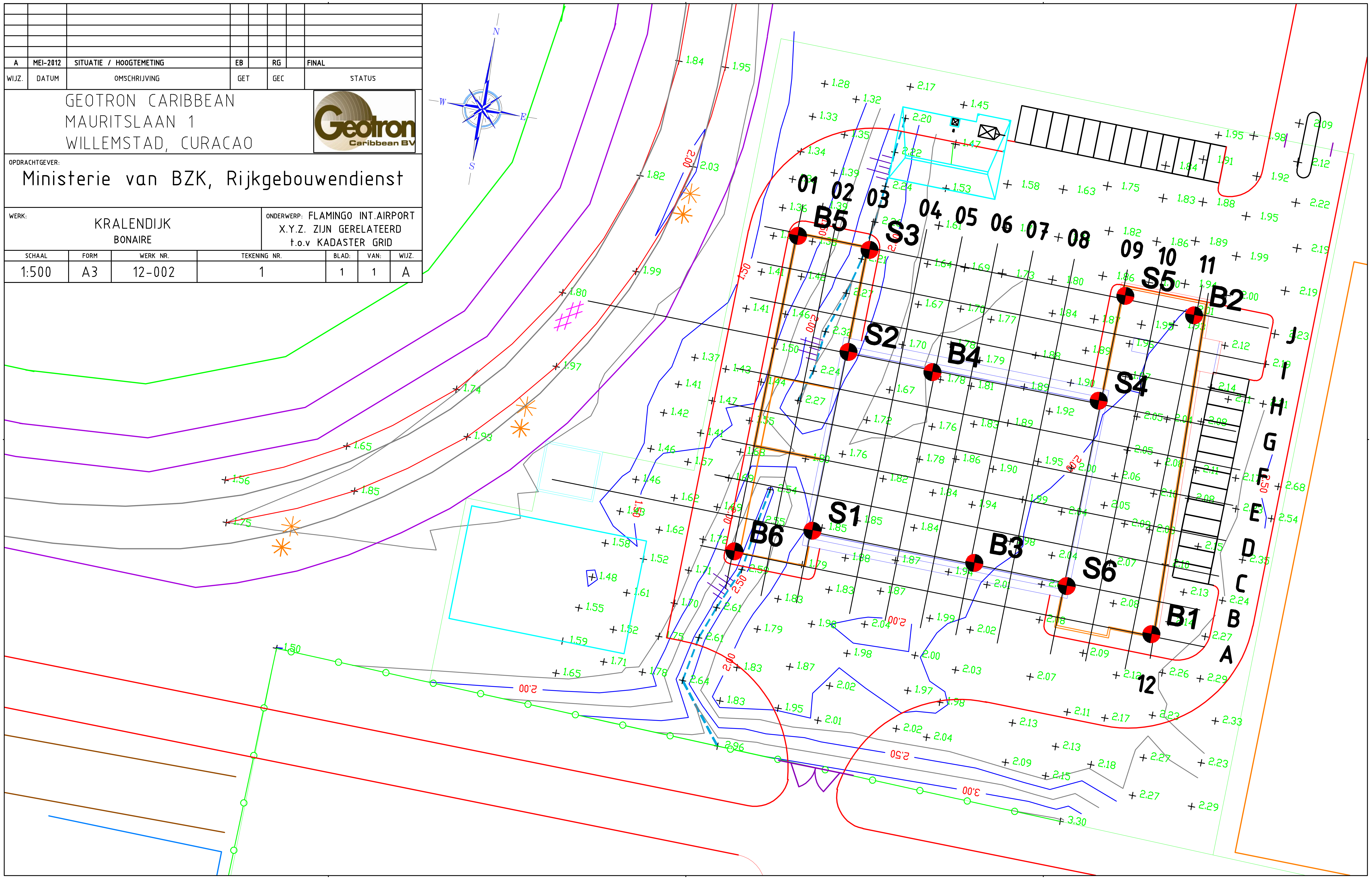
GEOTRON CARIBBEAN
MAURITSLAAN 1
WILLEMSTAD, CURACAO

OPDRACHTGEVER:
Ministerie van BZK, Rijkgebouwendienst

WERK: KRALENDIJK
BONAIRE

ONDERWERP: FLAMINGO INT.AIRPORT
X.Y.Z. ZIJN GERELATEERD
t.o.v KADASTER GRID

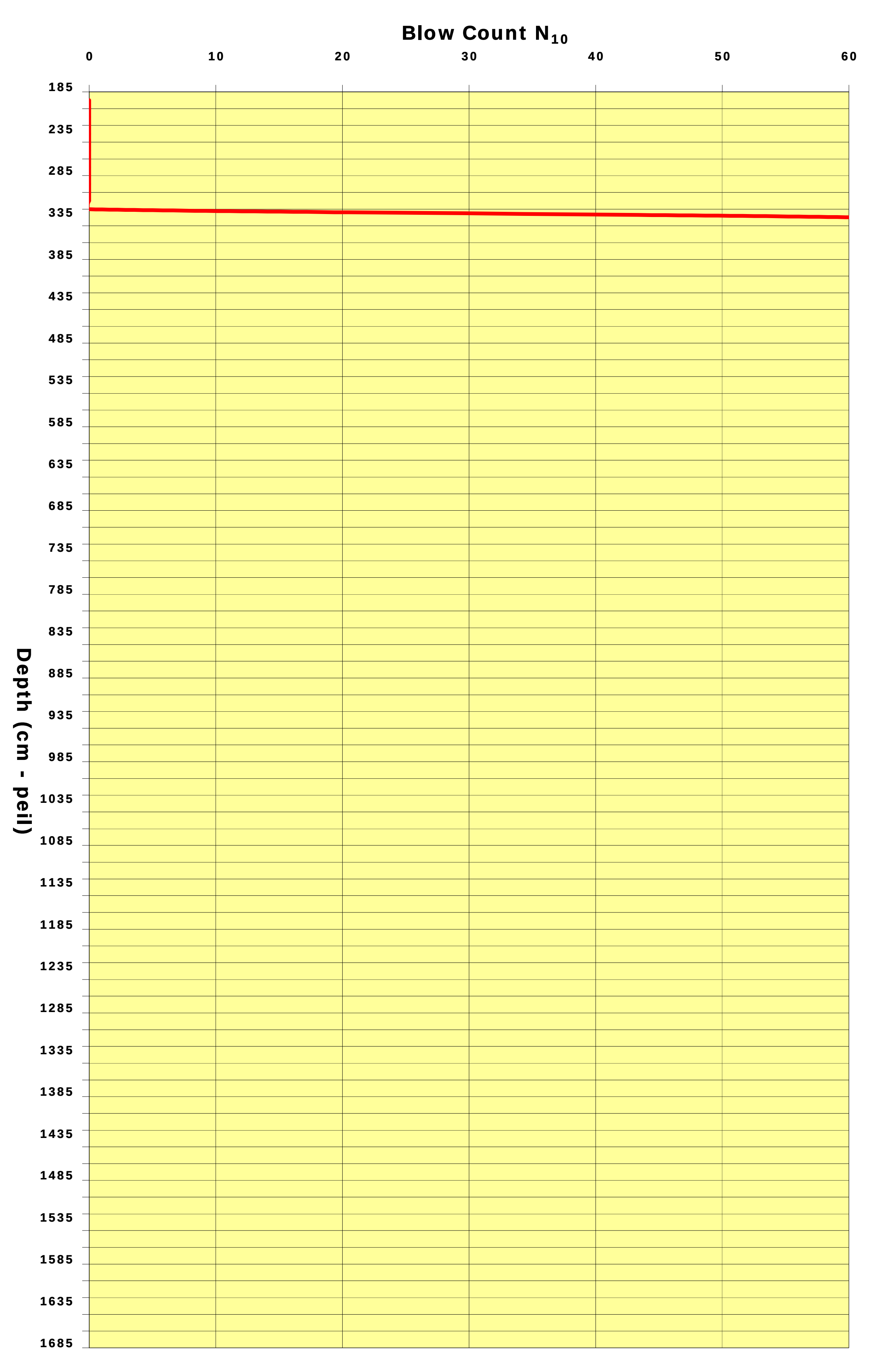
SCHAAL	FORM	WERK NR.	TEKENING NR.	BLAD:	VAN:	WIJZ.
1:500	A3	12-002	1	1	1	A



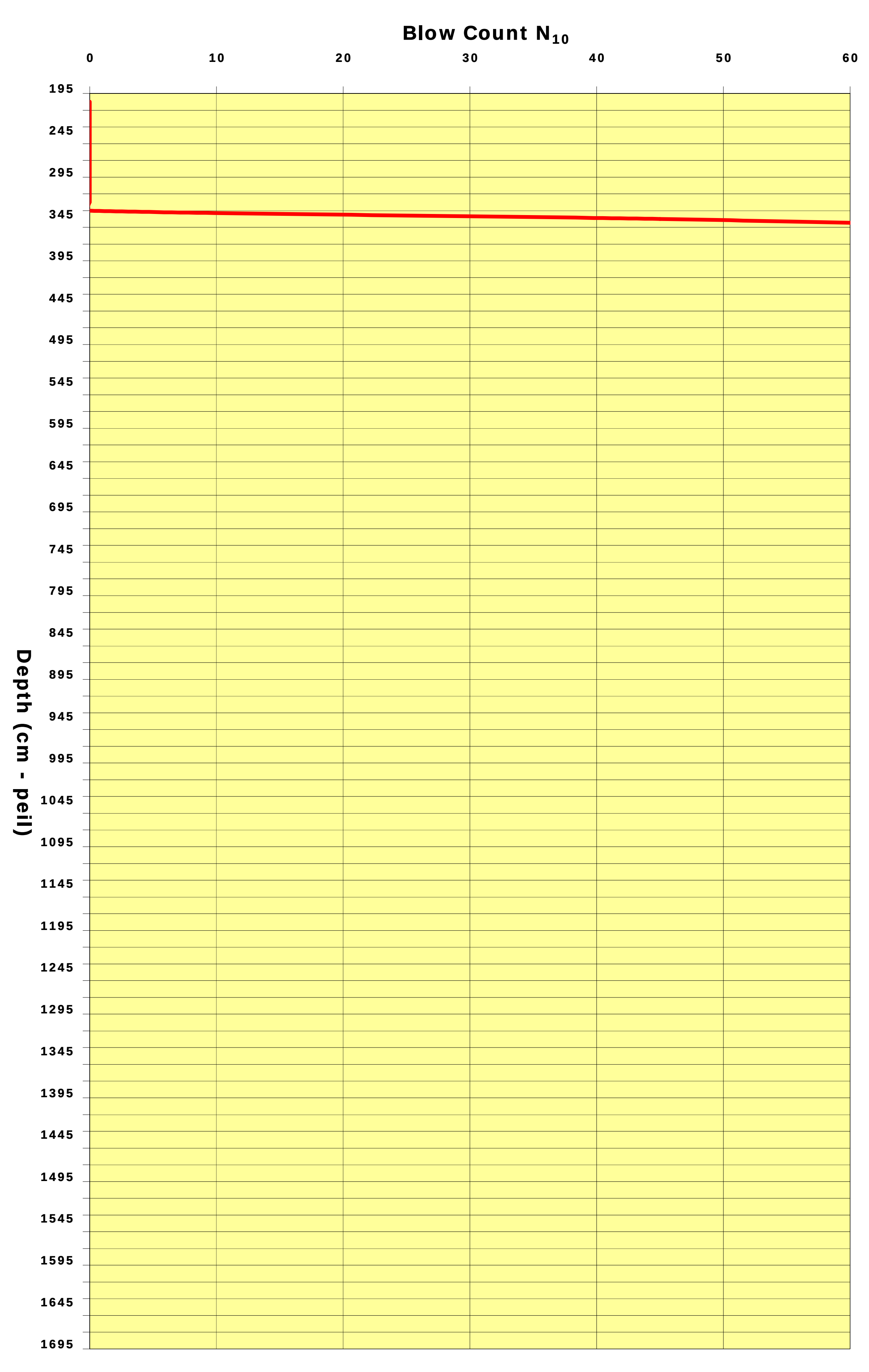
Annex 2:

Dynamic Penetration Tests (in 6 pages)

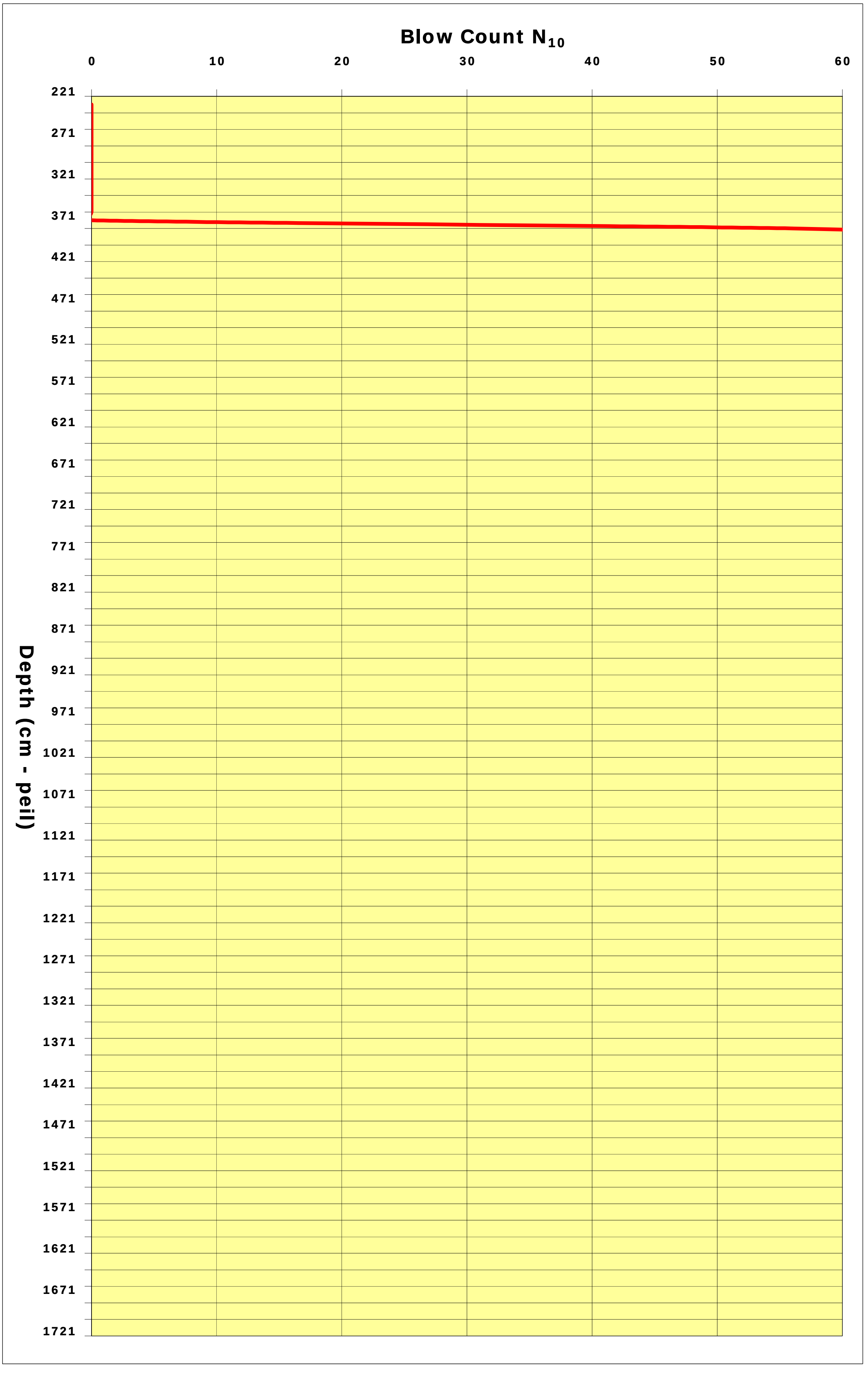
Projectnumber	**GC 1202**		**Location**		
Site	**Bonaire**		**X**	**0**	lat
Number DPT	**1**		**Y**	**0**	lon
Weight	**50**	kg	**Z**	**185**	cm-NSP



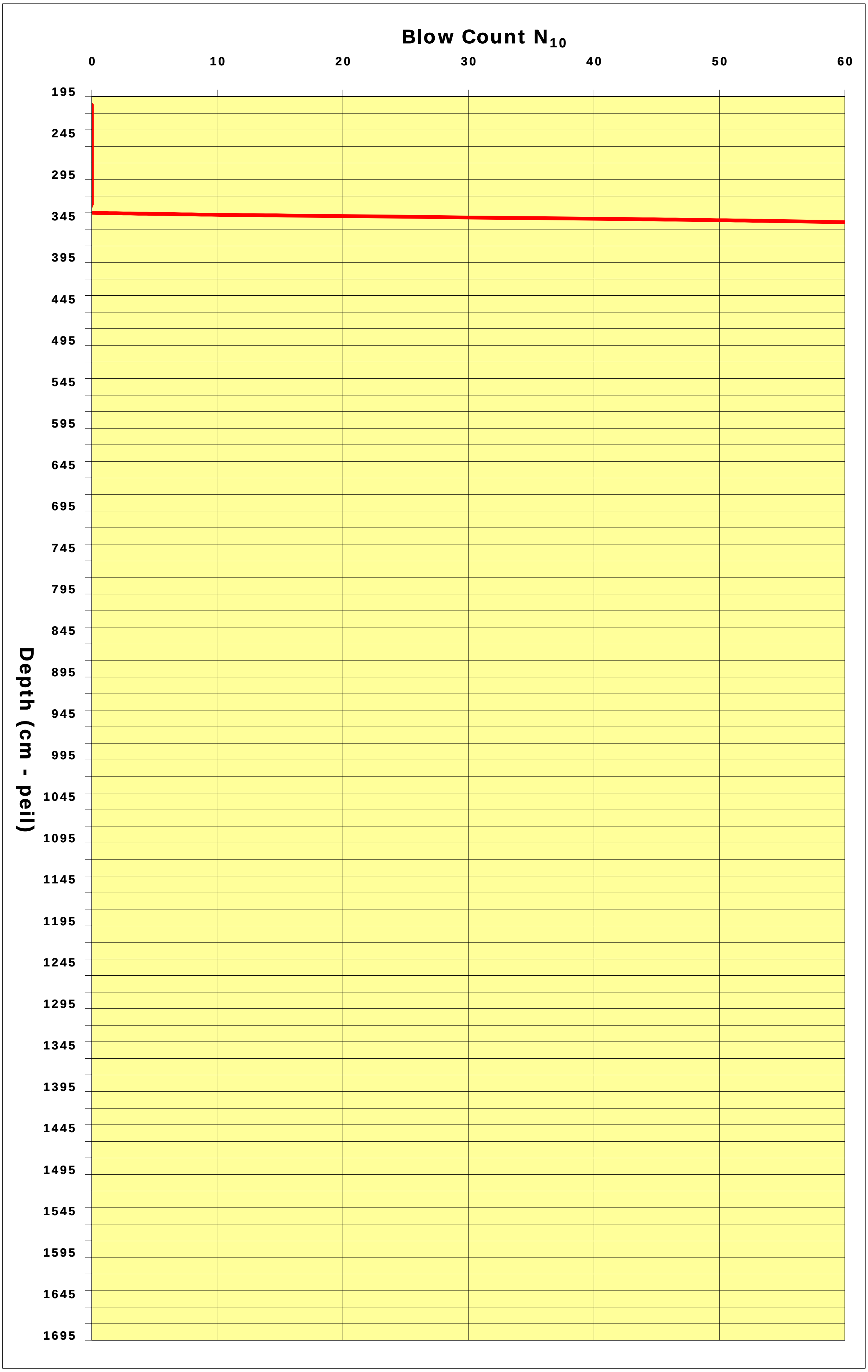
Projectnumber	**GC1202**	**Location**		
Site	**Bonaire**	**X**	**0**	lat
Number DPT	**2**	**Y**	**0**	lon
Weight	**50** kg	**Z**	**195**	cm-NSP



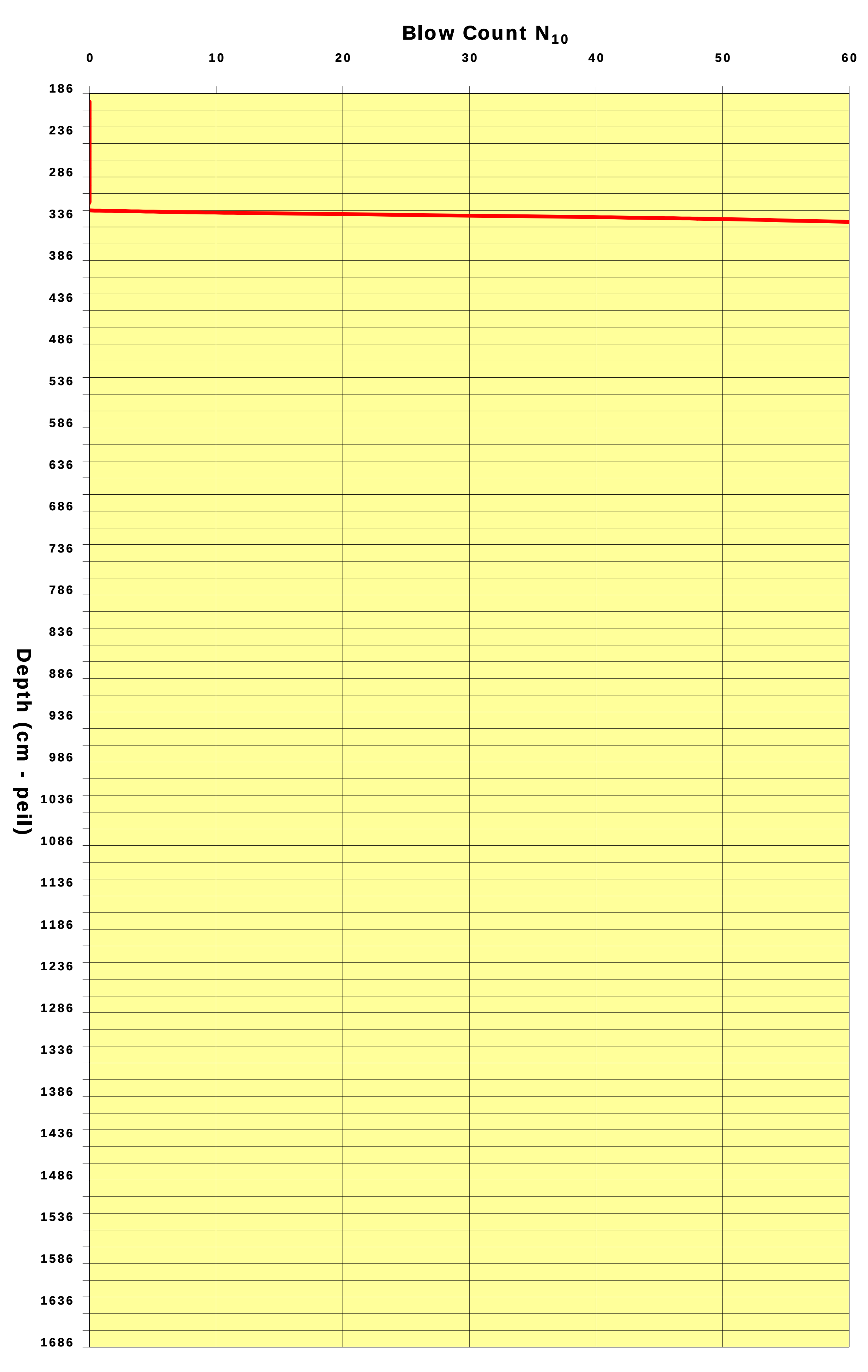
Projectnumber	GC1202	**Location**		
Site	Bonaire	**X**	0	lat
Number DPT	3	**Y**	0	lon
Weight	50 kg	**Z**	221	cm-NSP



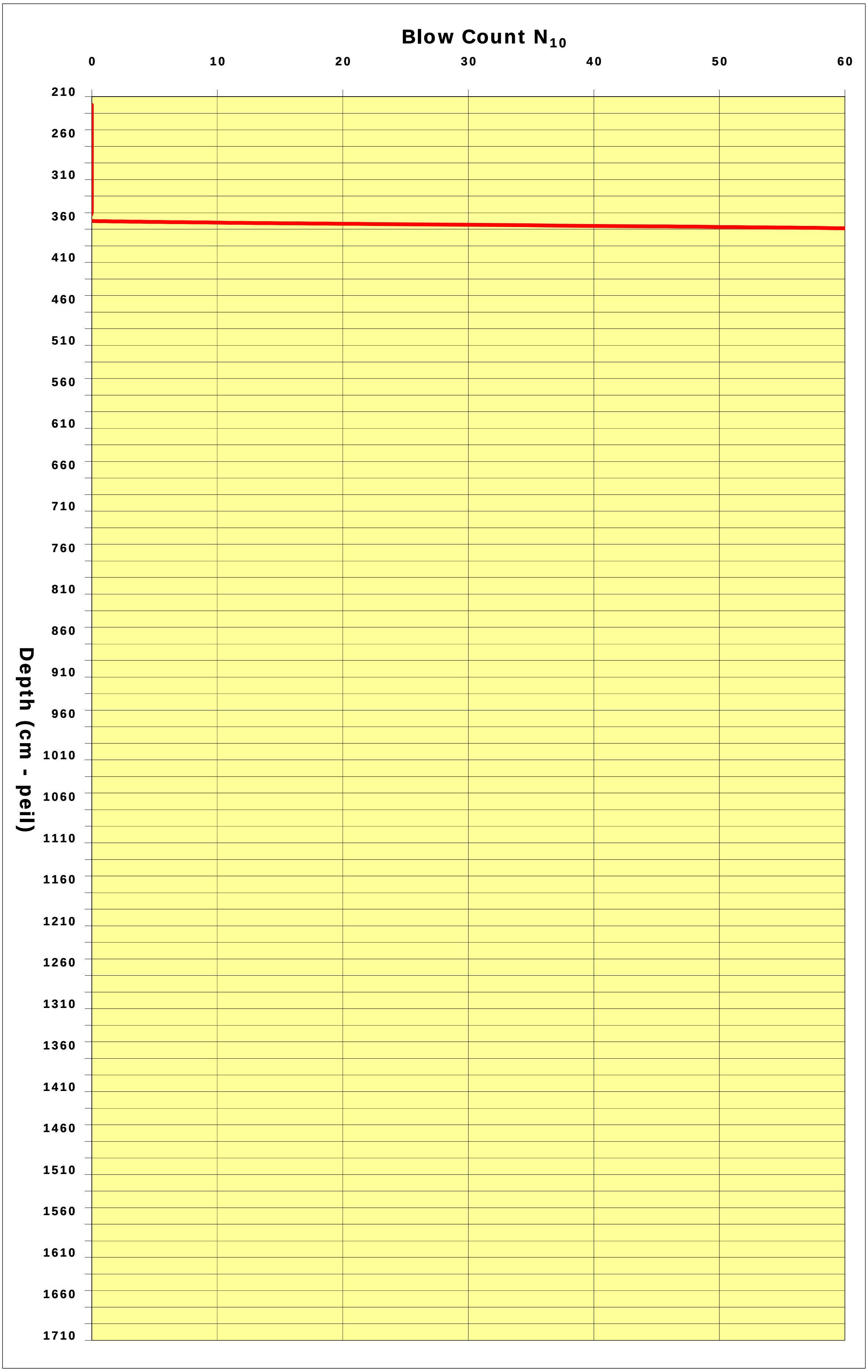
Projectnumber	GC1202		Location		
Site	Bonaire		X	0	lat
Number DPT	4		Y	0	lon
Weight	50	kg	Z	195	cm-NSP



Projectnumber	GC1202		Location		
Site	Bonaire		X	0	lat
Number DPT	5		Y	0	lon
Weight	50	kg	Z	186	cm-NSP



Projectnumber	GC1202		Location		
Site	Bonaire		X	0	lat
Number DPT	6		Y	0	lon
Weight	50	kg	Z	210	cm-NSP



Annex 3:

Borehole logs (in 12 pages)

Project No: GC 1202

BH Number: BH 01

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	2,13					
	0		Cemented brownish beige fine grained LIMESTONE, non porous and no caveties		100	83	55		
	1			0,73					
	2		Cemented brownish beige fine grained LIMESTONE, weakly porous no caveties and some shell remnants Groundwater: 1,90 mbgl		100	86	43		
				-0,27					
	3		Cemented brownish beige fine grained LIMESTONE, very porous no caveties and some shell remnants		95	93	58		
	4		Cemented beige fine grained LIMESTONE, weakly porous no caveties and some shell remnants	-1,77	100	94	67		
	5		Cemented beige medium grained CORAL, weakly porous no caveties		95	90	46		
	6				100	88	83		
	7				100	88	79		
					100	94	71		
	8			-5,87					
			End of Borehole						
	9								
	10								

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0 m

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0 m

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: BH 02

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Description	Elevation
Surface	2,01
Cemented brownish beige medium grained LIMESTONE, weakly porous no caveties	1,11
Cemented brownish beige medium grained LIMESTONE, medium porous no caveties Groundwater: 1,64 mbgl	-0,09
Brownish beige coarse grained LIMESTONE, porous and some small caveties	-0,69
Cemented brownish beige LIMESTONE, weakly porous and some caveties	-2,79
Brownish beige coarse grained CORAL, very porous and abundant caveties	-4,29
Slightly cemented beige CORAL, medium porous some caveties and some shell remnants	-5,99
End of Borehole	

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

TCR%	SCR%	RQD%
100	95	61
85	96	54
85	66	39
100	97	83
85	70	70
75	20	0
100	94	74
100	96	81

Columns: Core-run | Depth | Legend | Description | Elevation | TCR% | SCR% | RQD% | Depth fracture | Angle fracture

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: BH 03

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Description	Elevation
Surface	1,97
Slightly cemented brownish beige fine grained LIMESTONE, weakly porous no caveties	0,67
Slightly cemented brownish beige fine grained LIMESTONE, medium porous no caveties and some shell remnants Groundwater: 1,80 mbgl	-0,73
Brownish beige fine grained LIMESTONE, porous no caveties and some shell remnants	-1,03
Cemented beige fine grained LIMESTONE, weakly porous no caveties	-2,13
Cemented brownish beige fine grained LIMESTONE, weakly porous no caveties and some shell remnants	-3,43
Slightly cemented beige medium grained CORAL, weakly porous no caveties	-6,03
End of Borehole	

TCR%	SCR%	RQD%
95	65	21
100	73	63
80	42	27
100	69	69
100	81	62
85	61	16
95	81	73
95	91	31

Columns: Core-run, Depth (0–10), Legend, Description, Elevation, TCR%, SCR%, RQD%, Depth fracture, Angle fracture

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: BH 04

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
	0		Surface	1,77					
	0–1.1		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	0,67	100	90	78		
	1.1–1.9		Cemented brownish beige fine grained LIMESTONE, weakly porous no caveties and some shell remnants Groundwater: 1,45 mbgl	-0,13	100	26	20		
			Cemented brownish beige fine grained LIMESTONE, medium porous no caveties and some shell remnants	-1,23	75	48	27		
			Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	-1,53					
			Brownish beige LIMESTONE, medium porous		80	31	30		
			Soft grey CLAY with some coral remnants						
			Cemented brownish beige fine grained LIMESTONE, weakly porous no caveties	-3,93	75	79	57		
	5.7–8		Cemented beige fine grained CORAL, medium porous no caveties	-6,23	95	85	60		
					100	83	73		
					100	84	57		
	8		End of Borehole						

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: BH 05

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Depth	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
0	Surface	1,37					
0,00–0,30	Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	1,07	65	39	31		
0,30–1,10	Beige medium grained LIMESTONE, medium porous no caveties Groundwater: 1,08 mbgl	0,27					
1,10–2,20	Cemented brownish beige fine grained LIMESTONE, medium porous no caveties	-0,83	100	68	63		
2,20–2,80	Cemented beige medium grained LIMESTONE, medium porous no caveties	-1,43	100	73	73		
2,80–3,10	Cemented brownish beige medium grained LIMESTONE, nonporous no caveties and some coral remnants	-1,73	100	69	77		
3,10–4,60	Cemented brownish beige medium grained CORAL, medium porous no caveties	-3,23	100	65	65		
4,60–8,00	Beige coarse grained CORAL, medium porous no caveties	-6,63	95	65	46		
			100	73	69		
			100	91	77		
8,00	End of Borehole						

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: BH 06

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	2,31					
	0–1,25		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	1,06	97	61	38		
	1,25–2,3		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties and some shell remnants Groundwater: 1,51 mbgl	0,01	100	89	67		
	2,3–3,5		Cemented beige medium grained LIMESTONE, nonporous no caveties and some shell remnants	-1,19	98	64	65		
	3,5–5,6		Cemented beige medium grained CORAL, medium porous no caveties and some shell remnants	-3,29	100	84	71		
					98	72	58		
	5,6–8,0		Cemented beige coarse grained CORAL, medium porous no caveties	-5,69	95	68	92		
					100	80	77		
					100	88	65		
	8,0		End of Borehole						

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 8,0

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 8,0

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S01

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	1,85					
	0		Well cemented brownish beige fine grained LIMESTONE, nonporous no caveties	0,60	95	58	40		
	1		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties and some shell remnants	0,35					
	2		Medium hard LIMESTONE, drilled destructive. No cavities detected	-3,15					
	3								
	4								
	5		End of Borehole						
	6								
	7								
	8								
	9								
	10								

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S02

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	1,95					
	0		Well cemented brownish beige fine grained LIMESTONE, nonporous no caveties	0,85	100	85	75		
	1		Cemented brownish beige fine grained LIMESTONE, weakly porous no caveties and some shell remnants Groundwater: 1,45 mbgl	0,45					
	2		Weak to medium strong LIMESTONE drilled destructive. No cavities detected	-3,05					
	3								
	4								
	5		End of Borehole						
	6								
	7								
	8								
	9								
	10								

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S03

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	2,21					
	0		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	1,91					
	1		Well cemented beige medium grained LIMESTONE, medium porous no caveties Groundwater: 1,08 mbgl	1,11	75	50	40		
			Cemented brownish beige fine grained LIMESTONE, medium porous no caveties	0,71					
	2 3 4		Medium hard LIMESTONE, drilled destructive. No cavities detected	-2,79					
	5 6 7 8 9 10		End of Borehole						

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S04

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	1,95					
	0		Cemented brownish beige medium grained LIMESTONE, weakly porous no caveties	1,05	100	90	58		
	1		Cemented brownish beige medium grained LIMESTONE, medium porous no caveties	0,45					
	2		Medium hard LIMESTONE, drilled destructive. No cavities detected						
	3								
	4			-3,05					
	5		End of Borehole						
	6								
	7								
	8								
	9								
	10								

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S05

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Geotron Caribbean BV

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	1,86					
	0		Cemented brownish beige fine grained LIMESTONE, nonporous no caveties	1,56	100	96	64		
	1		Cemented beige medium grained LIMESTONE, medium porous no caveties Groundwater: 1,08 mbgl	0,76					
			Cemented brownish beige fine grained LIMESTONE, medium porous no caveties	0,36					
	2								
	3		Medium hard LIMESTONE drilled destructive. No caveties detected						
	4								
	5		End of Borehole	-3,14					
	6								
	7								
	8								
	9								
	10								

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

Seafloorlevel:

Sheet: 1 of 1

Project No: GC 1202

BH Number: S06

Project: Brandweerkazerne Bonaire

Client: RGD Bonaire

Operator: M.Lamore

Site Location: Flamingo airport

Logger: M.Lamore

Core-run	Depth	Legend	Description	Elevation	TCR%	SCR%	RQD%	Depth fracture	Angle fracture
			Surface	2,10					
	0 – 1,3		Slightly cemented brownish beige fine grained LIMESTONE, weakly porous no caveties	0,80	100	75	40		
	1,3 – 5		Slightly cemented brownish beige fine grained LIMESTONE, medium porous no caveties and some shell remnants						
			Medium hard LIMESTONE, drilled destructive. No cavities detected	-2,90					
	5		End of Borehole						

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Drill Equipment: Wizard 1

Coordinate X:

Penetration (m): 5

Drill Date: Juni 2012

Coordinate Y:

Recovery (m): 1,5

Remarks:

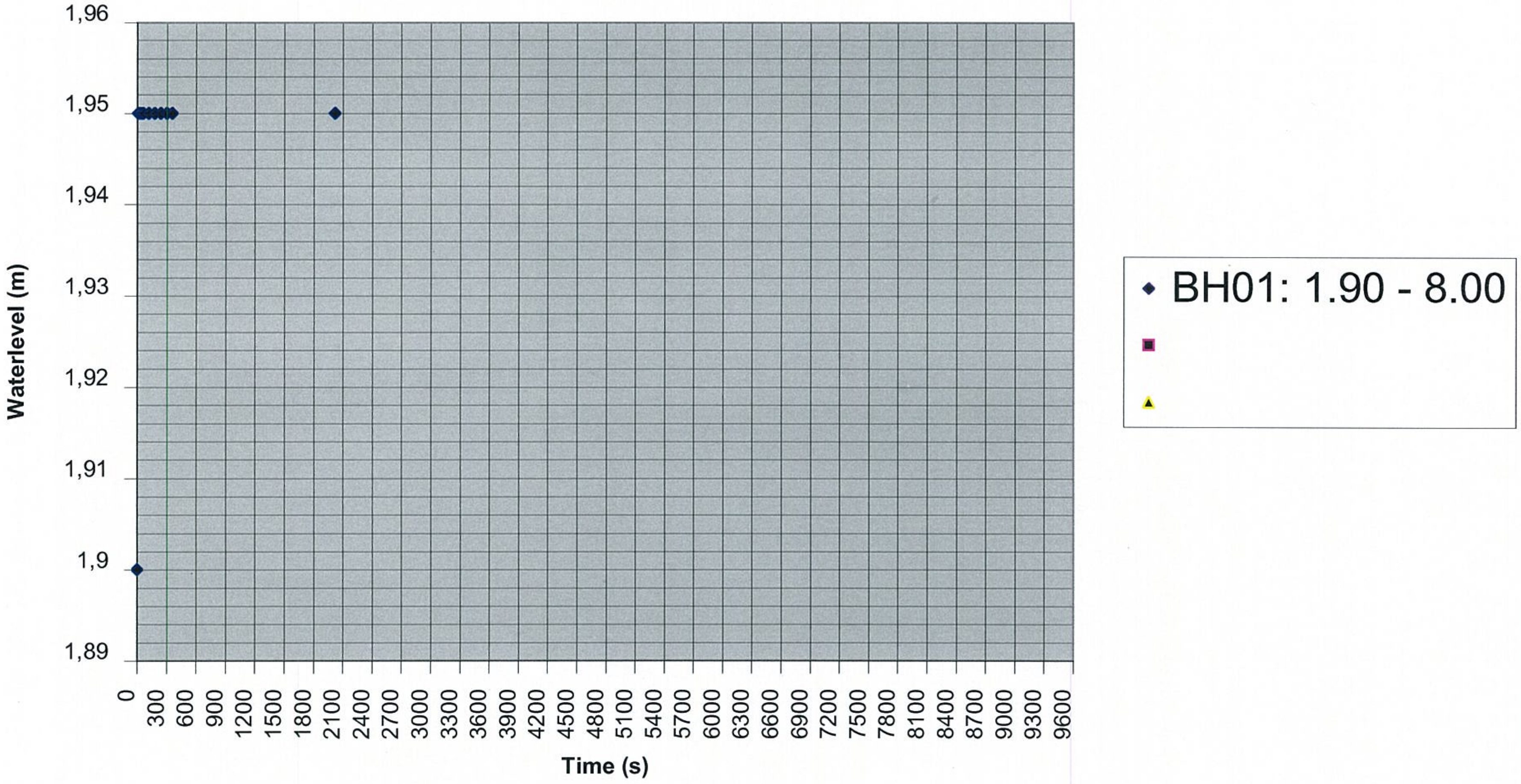
Seafloorlevel:

Sheet: 1 of 1

Annex 4:

Results of the pump test (in 3 pages)

Geotron Caribbean permeability tests GC1202, Flamingo Airport, Bonaire



BH01; diameter 100 mm, top casing = surface

Project:	permeability substrata flamingo airport, bonaire
Projectcode:	GC1202
Datum:	190712
Onderwerp:	Uitwerking Hooghoudt-puntproef volgens Zangar en RID

Zangar: k=Q/(C*L*r)

Waarin: k: radiale verzadigde waterdoorlatendheid in m/dag
Q: constant debiet in m3/dag
r: straal boorgat in m
L: (H**2-h**2)/2*H
waarin: H: diepte onderkant filter onder grondwaterspiegel voor afpompen
h: diepte onderkant filter onder grondwater tijdens afpompen
C: geometriefactor, afhankelijk van h en r
Lineaire benadering: C=10**(0,69+0,72*log(h/r))

RID: k=Q/(A*pi*r*fi)
fi: H-h (zie boven)
A: geometriefactor, afhankelijk van l en r
A= 2*X**2/(X*ln(X+(X**2+1)**-2)-(X**2+1)**-2+1)
waarin: X=l/r (l= filterlengte in m)

Invoergegevens

Peilbuis nummer	Onderk. filter (m-mv)	Lengte filter (m)	Straal filter (m)	Stijgh. voor afp. (m-mv)	Stijgh. bij afp. (m-mv)	Hoeveelh. afgep. (l)	Tijd (s)
BH01	8	7,1	0,05	1,9	1,95	10	4

Berekening volgens RID

Peilbuis nummer	Q (m3/d)	X=l/r	A	fi=H-h (m)	k (m/d)
BH01	216	142	60,99651	0,05	450,8778

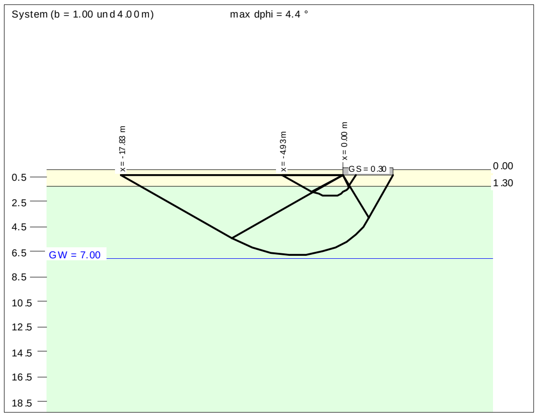
Annex 5:

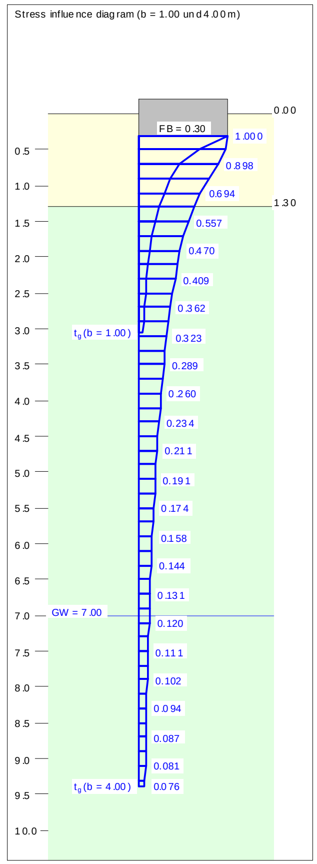
Bearing capacity and settlement characteristics for a shallow pad footing (in 4 pages).

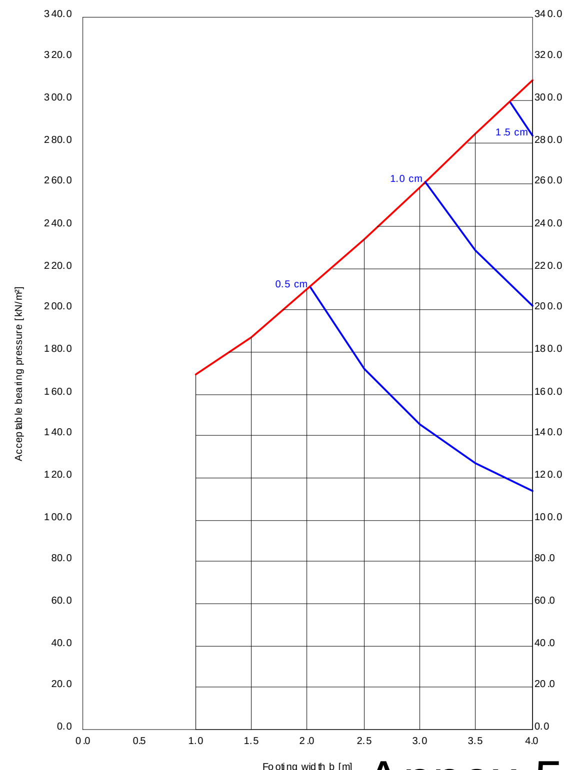
Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Pad footing (a/b = 1.00)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %

Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: PadAnnex5-1.gdg
Acceptable bearing pressure
Settlements







a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_{Ü}$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
1.00	1.00	169.4	169.4	0.17	32.3	0.00	15.73	4.80	3.04	2.02	100.1
1.50	1.50	187.3	421.5	0.31	31.6	0.00	15.54	4.80	4.02	2.82	60.6
2.00	2.00	209.6	838.6	0.49	31.2	0.00	15.43	4.80	4.99	3.61	42.8
2.50	2.50	233.6	1460.2	0.71	31.0	0.00	15.35	4.80	5.94	4.40	32.9
3.00	3.00	258.4	2326.0	0.97	30.8	0.00	15.30	4.80	6.88	5.20	26.6
3.50	3.50	283.7	3475.7	1.29	30.7	0.00	15.26	4.80	8.10	5.99	22.0
4.00	4.00	309.3	4948.9	1.66	30.6	0.00	15.23	4.80	9.39	6.78	18.6

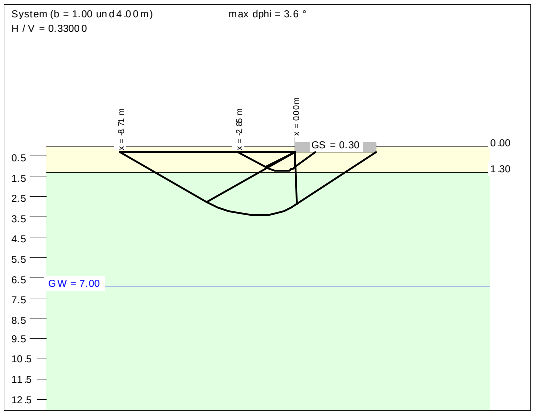
zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable(Q)/total loads(G+Q) [-] = 0.50

Annex 5.1

Soil	Depth [m]	γ [kN/m³]	γ ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

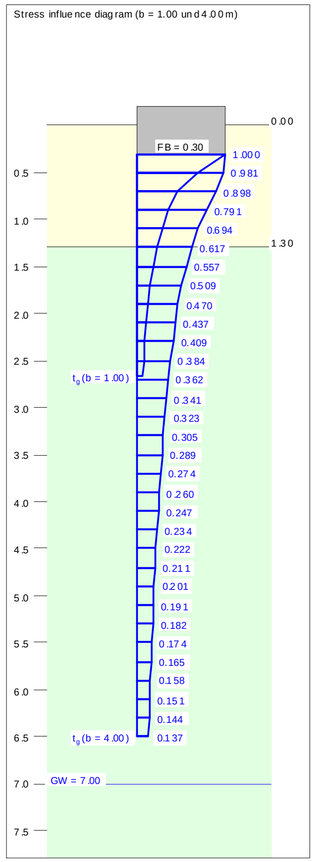
Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Pad footing (a/b = 1.00)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %

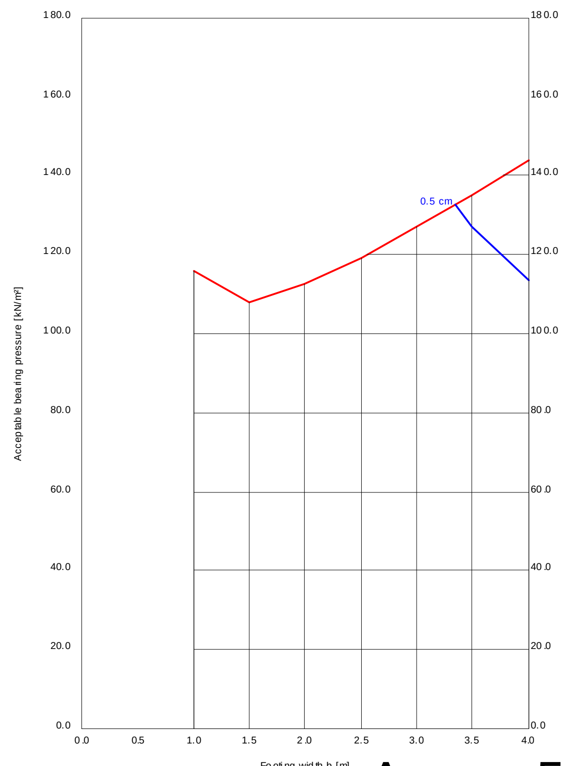
H/V = 0.3300
Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: PadAnnex5-2.gdg
Acceptable bearing pressure
Settlements



a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_{Ü}$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
1.00	1.00	115.8	115.8	0.11	35.0	0.00	16.00	4.80	2.67	1.24	106.4
1.50	1.50	107.9	242.7	0.16	33.3	0.00	15.90	4.80	3.30	1.59	66.4
2.00	2.00	112.4	449.6	0.24	32.6	0.00	15.77	4.80	3.96	1.96	47.4
2.50	2.50	119.2	744.7	0.32	32.1	0.00	15.67	4.80	4.61	2.33	36.7
3.00	3.00	126.9	1142.3	0.43	31.8	0.00	15.59	4.80	5.24	2.69	29.9
3.50	3.50	135.2	1656.8	0.54	31.6	0.00	15.53	4.80	5.88	3.05	25.1
4.00	4.00	143.9	2302.7	0.67	31.4	0.00	15.48	4.80	6.50	3.41	21.6

zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable(Q)/total loads(G+Q) [-] = 0.50



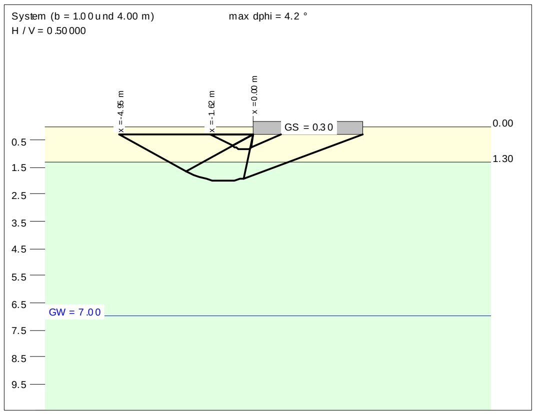


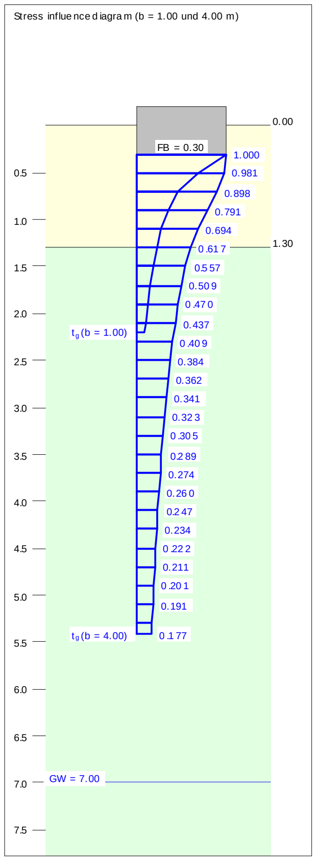
Annex 5.2

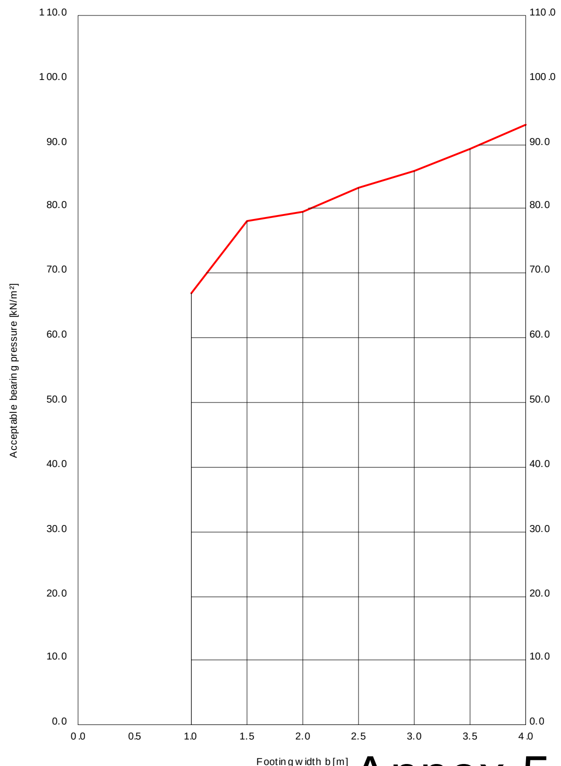
Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Pad footing (a/b = 1.00)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %

H/V = 0.5000
Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: PadAnnex5-3.gdg
Acceptable bearing pressure
Settlements







a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_Ü$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
1.00	1.00	67.0	67.0	0.06	35.0	0.00	16.00	4.80	2.20	0.83	119.2
1.50	1.50	78.2	176.0	0.11	35.0	0.00	16.00	4.80	2.92	1.10	71.0
2.00	2.00	79.6	318.4	0.16	34.2	0.00	15.99	4.80	3.46	1.37	51.0
2.50	2.50	83.2	520.0	0.21	33.7	0.00	15.95	4.80	3.99	1.50	39.7
3.00	3.00	86.0	773.7	0.26	33.3	0.00	15.89	4.80	4.47	1.68	32.5
3.50	3.50	89.4	1094.8	0.32	32.9	0.00	15.83	4.80	4.95	1.85	27.5
4.00	4.00	93.1	1489.9	0.39	32.7	0.00	15.78	4.80	5.41	2.02	23.8

zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable (Q)/total loads(G+Q) [-] = 0.50

Annex 5.3

Explanation of used symbols:

a = footing length
b = footing width
☐ = slope angle grade (ground level)

Allowable ☐ = maximum acceptable bearing pressure (under footing)
☐$_{0f;k}$ = characteristic value of the failure bearing pressure (under footing)
k_s = modulus of subgrade reaction

☐$_{GR}$ = partial safety factor for bearing capacity
☐$_{G}$ = partial safety factor for permanent action
☐$_{Q}$ = partial safety factor for changeable action
G = permanent load
Q = changeable load

☐ = natural volume weight (dry)
☐' = effective volume weight (saturated)
☐ = angle of internal friction
c = cohesion
E_s = E_{oed} = constrained (compression) modulus

cal ☐ = weighted value of ☐
cal c = weighted value of c
☐$_{2}$ = weighted value of volume weight

☐$_{Ü}$ = soil pressure due to surcharge load (on footing base depth)
t_g = stress influence depth for the settlement calculation
Base LS = depth of Logarithmic Spiral

Annex 6:

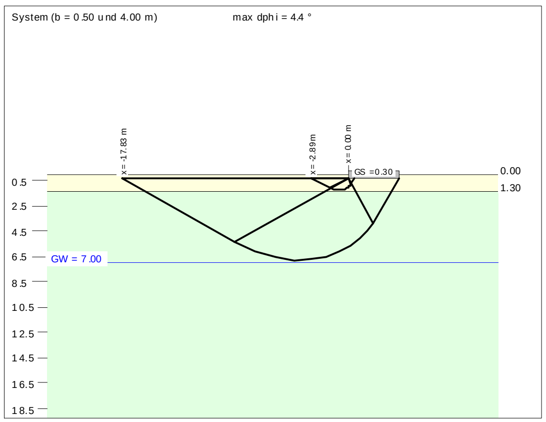
Bearing capacity and settlement characteristics for a shallow strip footing (in 4 pages).

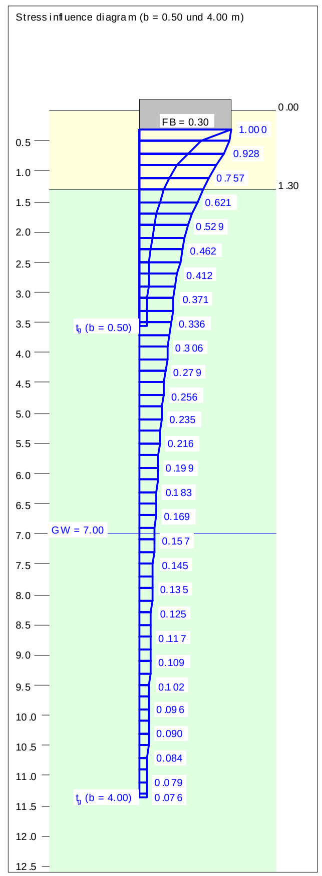
Geotron Caribbean BV

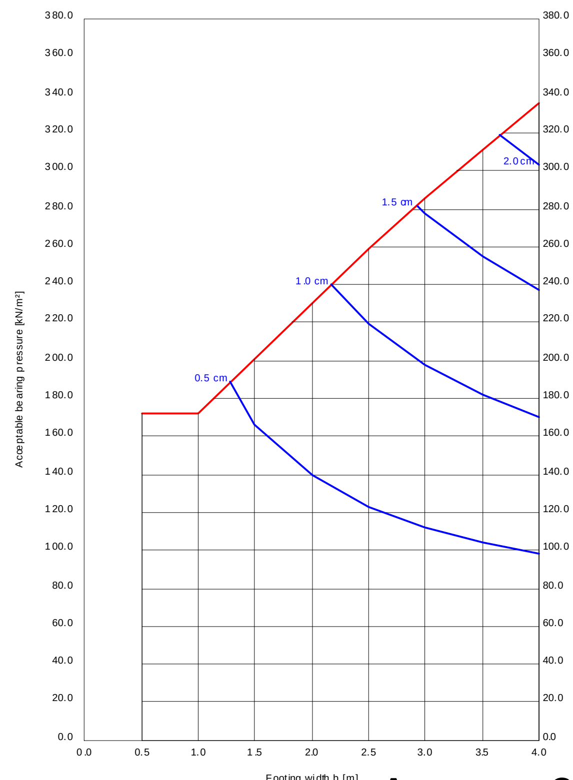
Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Strip footing (a = 6.00 m)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %

Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: StripAnnex6-1.gdg
Acceptable bearing pressure
Settlements







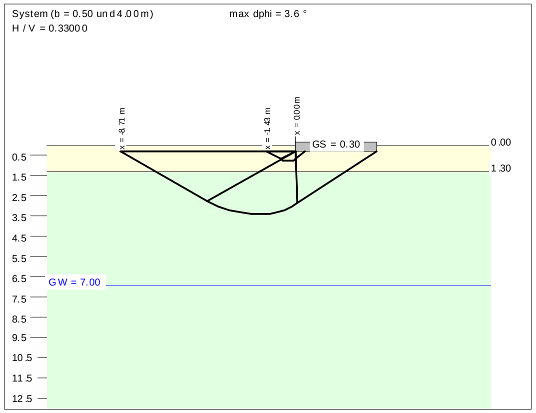
a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN/m]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_Ü$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
6.00	0.50	172.4	86.2	0.21	35.0	0.00	16.00	4.80	3.55	1.25	82.8
6.00	1.00	172.4	172.4	0.38	32.3	0.00	15.73	4.80	4.70	2.02	45.4
6.00	1.50	200.6	300.9	0.62	31.6	0.00	15.54	4.80	5.86	2.82	32.1
6.00	2.00	229.9	459.7	0.90	31.2	0.00	15.43	4.80	6.90	3.61	25.5
6.00	2.50	258.4	646.0	1.22	31.0	0.00	15.35	4.80	8.13	4.40	21.2
6.00	3.00	285.6	856.8	1.55	30.8	0.00	15.30	4.80	9.29	5.20	18.4
6.00	3.50	311.2	1089.3	1.90	30.7	0.00	15.26	4.80	10.36	5.99	16.4
6.00	4.00	335.2	1340.7	2.25	30.6	0.00	15.23	4.80	11.35	6.78	14.9

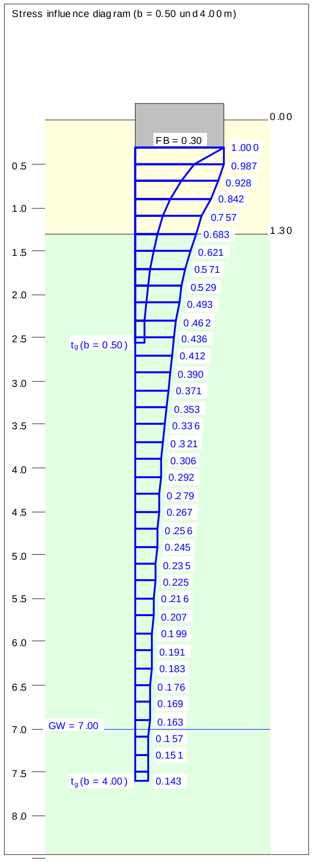
zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable (Q)/total loads(G+Q) [-] = 0.50

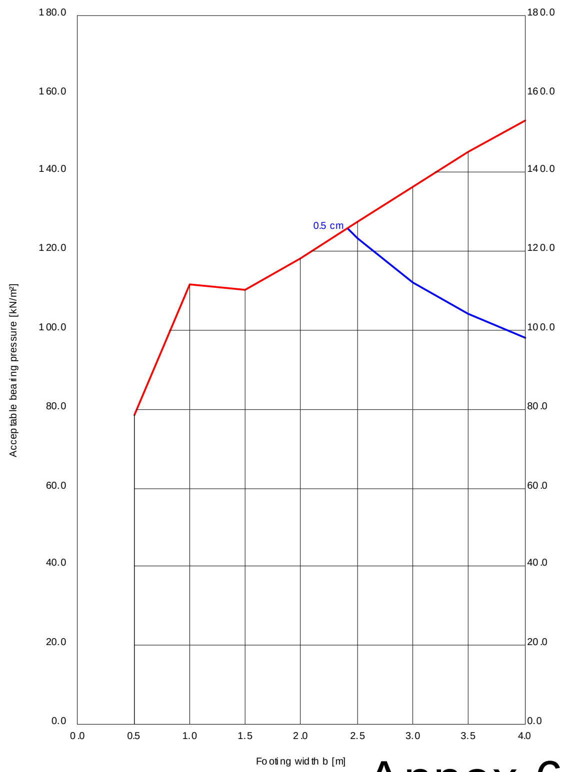
Annex 6.1

Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Strip footing (a = 6.00 m)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %
H/V = 0.3300
Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: StripAnnex6-2.gdg
Acceptable bearing pressure
Settlements







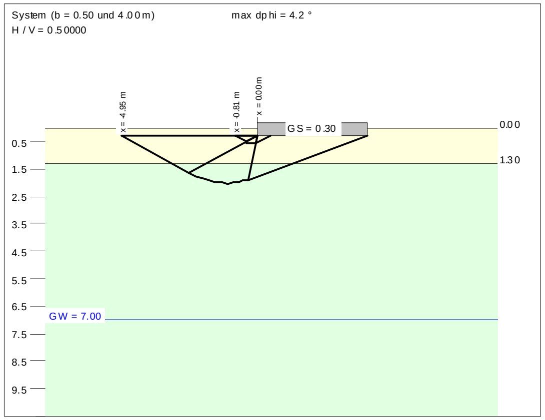
a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN/m]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_{Ü}$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
6.00	0.50	78.5	39.3	0.08	35.0	0.00	16.00	4.80	2.56	0.77	99.1
6.00	1.00	111.4	111.4	0.22	35.0	0.00	16.00	4.80	3.91	1.24	49.6
6.00	1.50	110.1	165.1	0.30	33.3	0.00	15.90	4.80	4.56	1.59	36.1
6.00	2.00	118.3	236.7	0.41	32.6	0.00	15.77	4.80	5.24	1.96	28.9
6.00	2.50	127.4	318.5	0.52	32.1	0.00	15.67	4.80	5.87	2.33	24.4
6.00	3.00	136.4	409.3	0.64	31.8	0.00	15.59	4.80	6.43	2.69	21.5
6.00	3.50	145.1	507.9	0.75	31.6	0.00	15.53	4.80	6.95	3.05	19.3
6.00	4.00	153.3	613.3	0.88	31.4	0.00	15.48	4.80	7.60	3.41	17.5

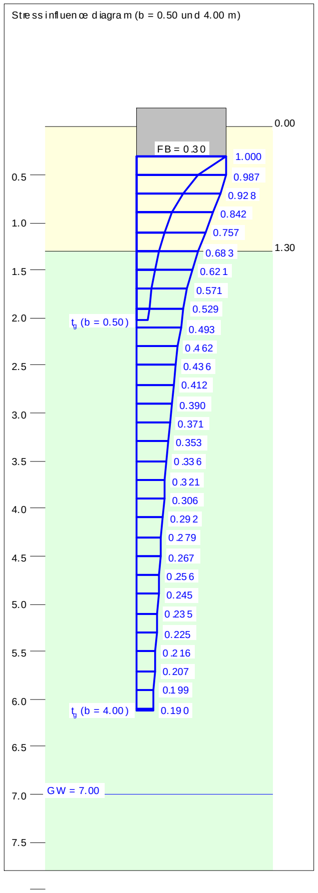
zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable(Q)/total loads(G+Q) [-] = 0.50

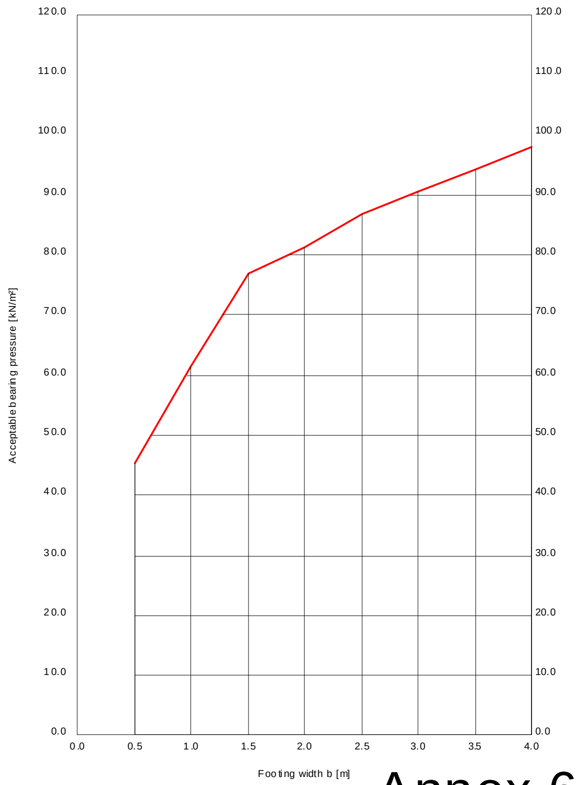
Annex 6.2

Soil	Depth [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Designation
	1.30	16.0	6.0	35.0	0.0	100.0	0.00	Coral limestone (Klip)
	>1.30	15.0	5.0	30.0	0.0	40.0	0.00	Sand, calc., compact

Initial calculation data:
WEB Recips III Aruba
Bearing cap. equation after DIN 4017 (neu)
Partial safety factor concept
Strip footing (a = 6.00 m)
γ (Gr) = 1.40
γ (G) = 1.35
γ (Q) = 1.50
Proportion of changeable loads = 50.0 %
H/V = 0.5000
Footing base depth = 0.30 m
Groundwater = 7.00 m
Limiting depth with p = 20.0 %
Limiting depth determined with stress variable
File: StripAnnex6-3.gdg
Acceptable bearing pressure
Settlements







a [m]	b [m]	Allow. σ [kN/m²]	Allow. R [kN/m]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_Ü$ [kN/m²]	t_g [m]	Base LS [m]	k_s [MN/m³]
6.00	0.50	45.3	22.7	0.04	35.0	0.00	16.00	4.80	2.03	0.57	117.3
6.00	1.00	61.5	61.5	0.11	35.0	0.00	16.00	4.80	3.03	0.83	57.7
6.00	1.50	76.9	115.3	0.20	35.0	0.00	16.00	4.80	3.90	1.10	39.4
6.00	2.00	81.2	162.4	0.26	34.2	0.00	15.99	4.80	4.45	1.37	31.6
6.00	2.50	86.7	216.8	0.32	33.7	0.00	15.95	4.80	4.96	1.50	26.8
6.00	3.00	90.5	271.4	0.38	33.3	0.00	15.89	4.80	5.38	1.68	23.6
6.00	3.50	94.3	330.0	0.44	32.9	0.00	15.83	4.80	5.77	1.85	21.3
6.00	4.00	97.9	391.6	0.50	32.7	0.00	15.78	4.80	6.12	2.02	19.6

zul $\sigma = \sigma_{0f,k} / (\gamma_{Gr} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.40 \cdot 1.43) = \sigma_{0f,k} / 1.99$
Ratio of changeable(Q)/total loads(G+Q) [-] = 0.50

Annex 6.3

Explanation of used symbols:

a = footing length
b = footing width
□ = slope angle grade (ground level)

Allowable □ = maximum acceptable bearing pressure (under footing)
$\square_{0f;k}$ = characteristic value of the failure bearing pressure (under footing)
k_s = modulus of subgrade reaction

$\square_{GR}$ = partial safety factor for bearing capacity
$\square_{G}$ = partial safety factor for permanent action
$\square_{Q}$ = partial safety factor for changeable action
G = permanent load
Q = changeable load

□ = natural volume weight (dry)
□' = effective volume weight (saturated)
□ = angle of internal friction
c = cohesion
E_s = E_{oed} = constrained (compression) modulus

cal □ = weighted value of □
cal c = weighted value of c
$\square_2$ = weighted value of volume weight

$\square_{Ü}$ = soil pressure due to surcharge load (on footing base depth)
t_g = stress influence depth for the settlement calculation
Base LS = depth of Logarithmic Spiral

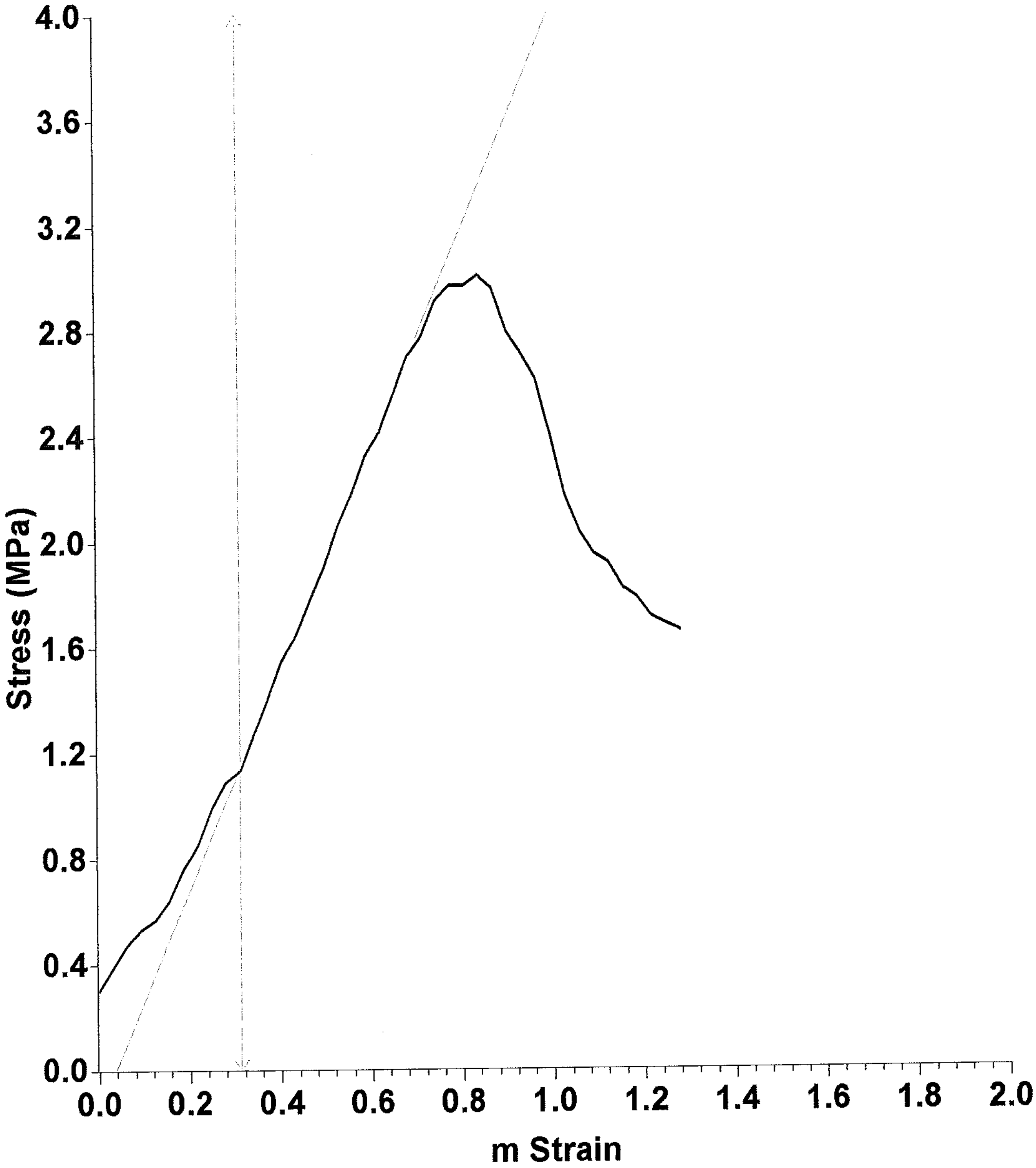
Annex 7:

Lab testing results (in 6 pages)

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b6 1.85-2.00
Sample diameter (mm) =63.2
Sample length (mm) =129

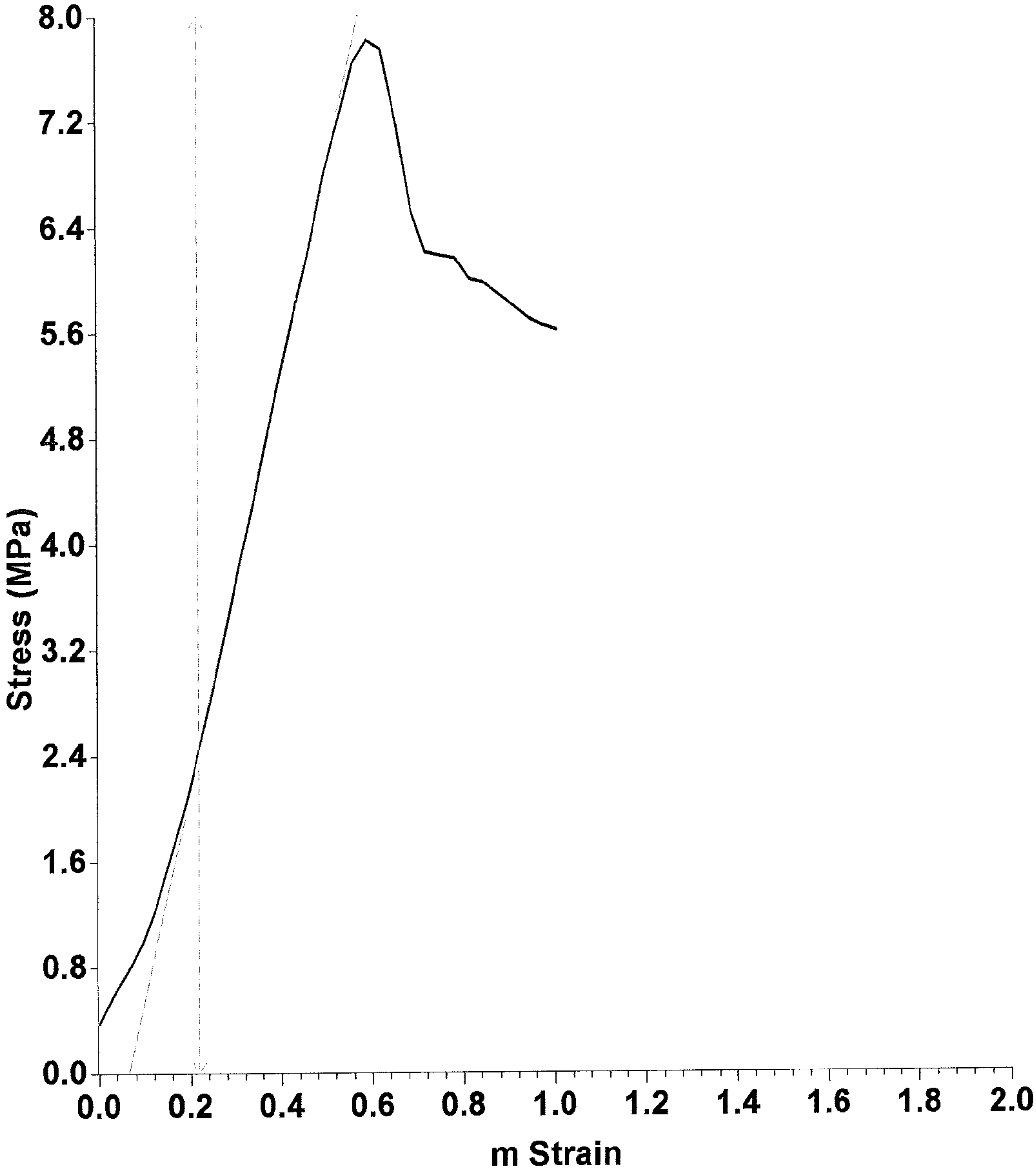


Maximum Stress(MPa)=3
E. Modulus tangent 50 % (GPa) =4.15
File name=c:\metingen\Data\bon b6 1.85-2.00 1156.csv
Laboratory Engineering Geology, University of Technology Delft

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b5 2.60-2.75
Sample diameter (mm) =63.1
Sample length (mm) =127.1

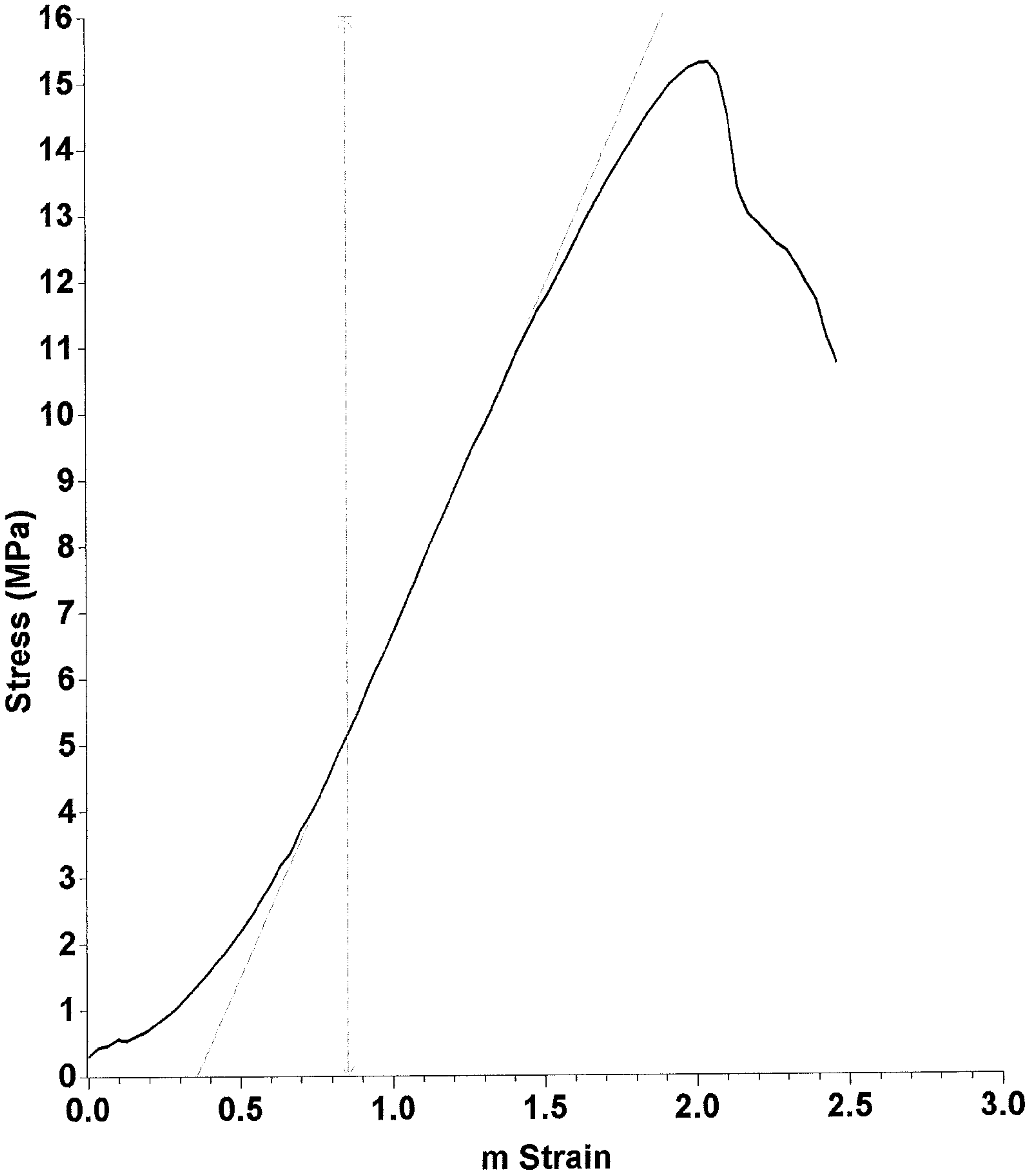


Maximum Stress(MPa)=7.8
E. Modulus tangent 50 % (GPa) =15.69
File name=c:\metingen\Data\bon b5 2.60-2.75 1155.csv
Laboratory Engineering Geology, University of Technology Delft

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b4 0.35-0.50
Sample diameter (mm) =62.6
Sample length (mm) =127.2

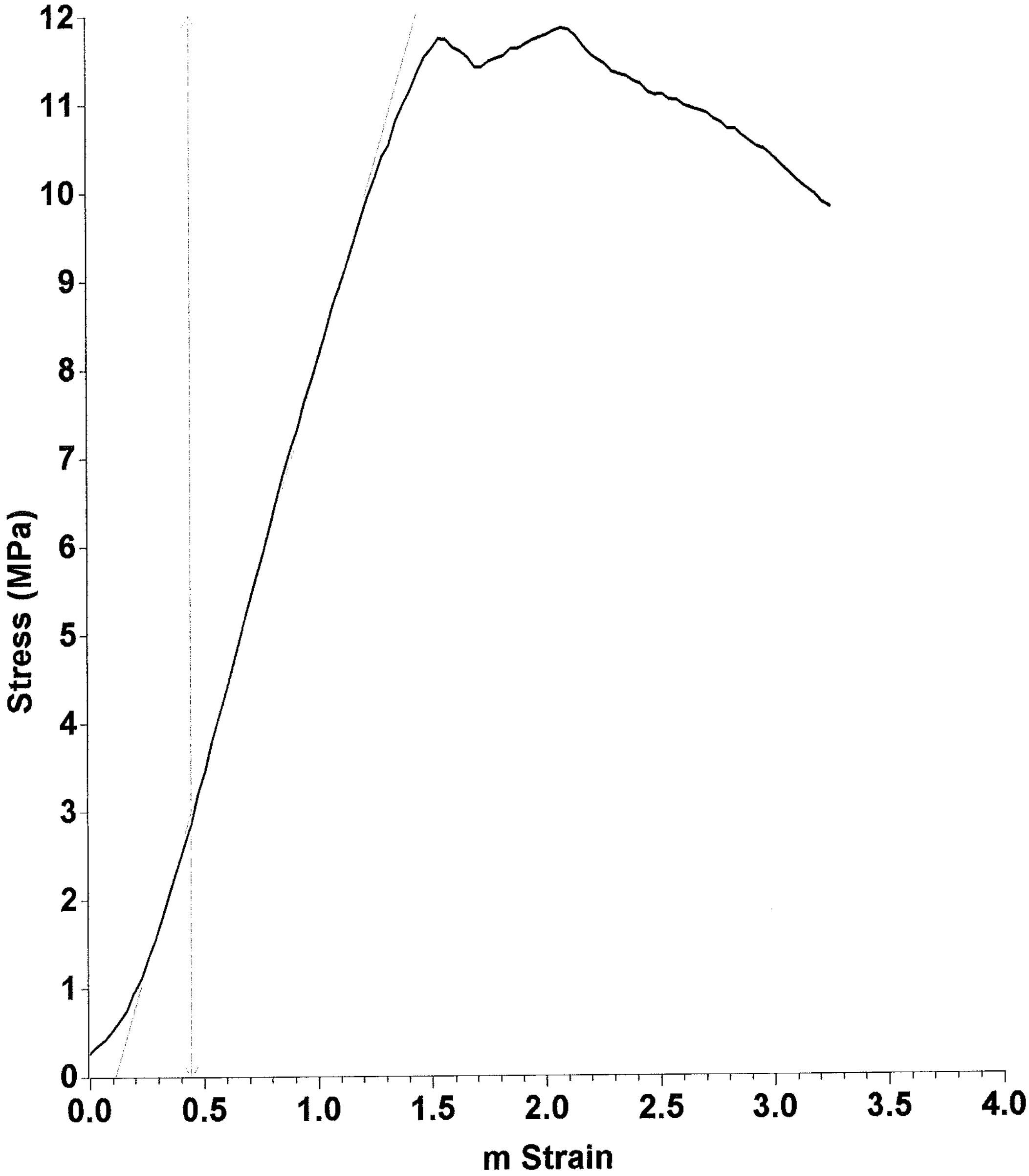


Maximum Stress(MPa)=15.27
E. Modulus tangent 50 % (GPa) =10.38
File name=c:\metingen\Data\bon b4 0.35-0.50 1154.csv
Laboratory Engineering Geology, University of Technology Delft

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b1 0.30-0.45
Sample diameter (mm) =62.3
Sample length (mm) =127.4

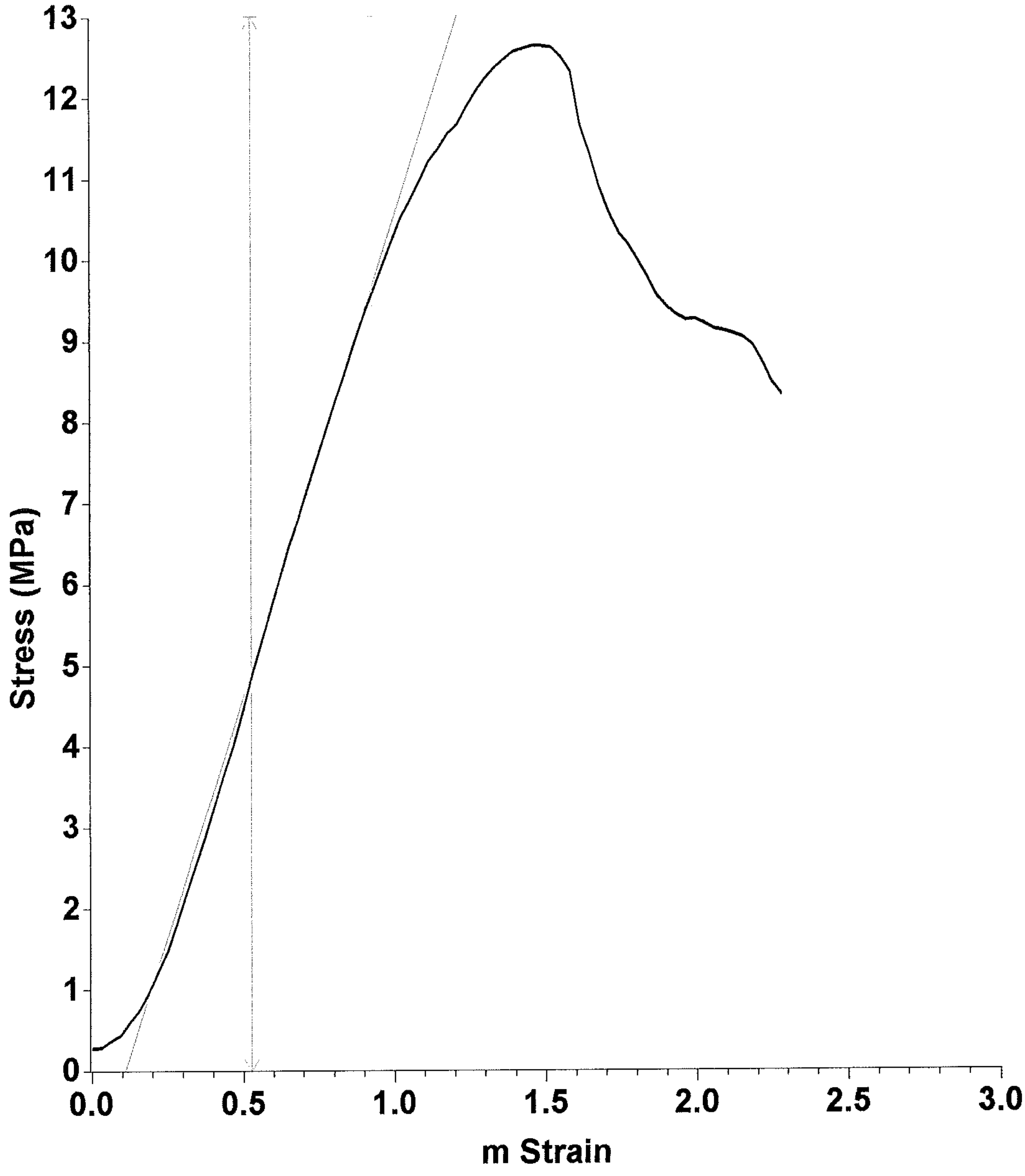


Maximum Stress(MPa)=11.83
E. Modulus tangent 50 % (GPa) =9.01
File name=c:\metingen\Data\bon b1 0.30-0.45 1153.csv
Laboratory Engineering Geology, University of Technology Delft

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b2 1.30-1.50
Sample diameter (mm) =63.1
Sample length (mm) =128.7

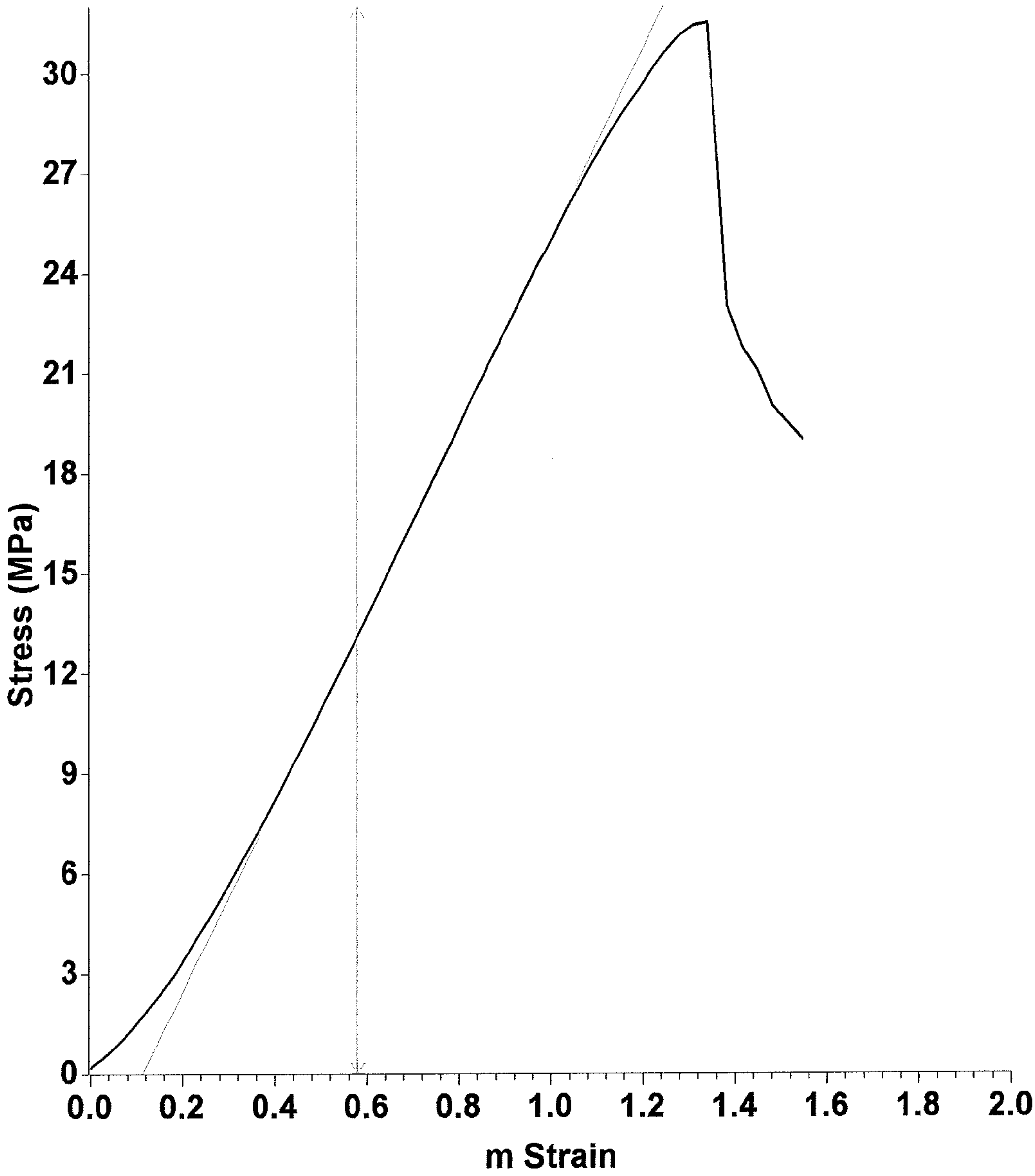


Maximum Stress(MPa)=12.62
E. Modulus tangent 50 % (GPa) =11.82
File name=c:\metingen\Data\bon b2 1.30-1.50 1152.csv
Laboratory Engineering Geology, University of Technology Delft

Unconfined Compressive Strength test

Jul. 04, 2012

Sample name =bon b3 3.40-3.60
Sample diameter (mm) =63.24
Sample length (mm) =128.5



Maximum Stress(MPa)=31.47
E. Modulus tangent 50 % (GPa) =28.2
File name=c:\metingen\Data\bon b3 3.40-3.60 1151.csv
Laboratory Engineering Geology, University of Technology Delft