

GEOTECHNICAL INVESTIGATION REPORT for the Proposed Factory of Pepsico India Holdings Pvt Ltd., at Village Nalbari, Assam

EXECUTIVE SUMMARY

Pepsico India Holdings Private Limited, Punjab - 148026 are in the process of constructing a Factory Complex at Village Nalbari, Assam. Space Geo Tech, Bangalore - 560010 are the Consultants for the proposed project.

The proposed structures are Factory Buildings.

The Geotechnical investigation programme has been undertaken at the site, as per the scope of investigation, as stipulated by the client. The scope of work consisted of conducting boreholes in soil strata down to 20m depth at ten locations and down to 30m depth at one location, conducting cross hole seismic test down to 20m depth at one location, opening trial pits down to a maximum depth of 1.5m below the existing ground level at five locations for collection of CBR samples and conducting laboratory california bearing ratio test on five soil sample collected from the site inclusive of conducting modified proctor density test, conducting standard proctor density test on six soil samples collected from the site, conducting field permeability test in one borehole at five different depths, conducting laboratory permeability test on eleven soil samples.

The borehole investigations indicate that the subsoil strata predominantly consists of silty soils down to about 10m depth below the existing ground level and also beyond a depth of about 15m down to the maximum depth investigated i.e. 30m below the existing ground level. However subsoil strata between 10m and 15m depths predominantly consists of sandy soils. Additionally occasional thin layers of sandy soils have been encountered beyond 15m depth down to about 25m depth below the existing ground level.



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The N-values (N-values 2 - 40) indicate that the subsoil is loose to medium dense down to the maximum depth investigated.

Ground water table had been encountered in all the boreholes at a depth of about 2m below the existing ground level, during the period of field investigations i.e February 2023.

The assessment of the susceptibility to liquefaction of the subsoil at site indicate, the presence of sandy soils layer sandwich between silty soils is susceptible to liquefaction between 10m and 15m depths. Hence, subsoil down to about 15m depth below the existing ground level has been considered to be susceptible to liquefaction.

In view of the large depth of subsoil susceptible to liquefaction, to ensure the feasibility of both shallow as well as deep foundations, appropriate ground improvement will have to be carried out.

Considering the subsoil conditions at the present site and the large depth of subsoil required to be improved to mitigate the susceptibility to liquefaction of the subsoil, the most appropriate method of ground improvement is the Vibro Compaction method as this is a relatively faster and more economical method.

Accordingly, ground improvement by Vibro Compaction has been recommended to be carried out throughout the area of the site to achieve the required densification of the sandy soils layer between 10m and 15m depths below the existing ground level at a spacing of the points/nodes of 1.5m to 1.8m and thus eliminate the susceptibility to liquefaction of the subsoil. The foundations for the proposed structures can then be placed over such improved ground.

Considering the type of structures involved and the presumed subsoil characteristics post ground improvement, **Straight Bored Pile foundations** have been recommended.

The following safe pile capacities have been recommended for various diameters for 20m and 25m effective lengths of piles.



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Pile Dia (mm)	Pile Effective Length (m)	Pile Vertical Load Capacity (tons)	Uplift Capacity of pile (tons)	Safe Lateral Capacity of pile (tons)	
				Free Head	Fixed Head
600	20	70	45	0.7	1.7
	25	90	60		
800	20	115	70	1	2.7
	25	140	95		

However, For Minor Structures at ground level, Isolated/ Strip Footings have also been recommended at a depth between 1.5m and 2m below the existing ground level with net allowable bearing pressures varying between 2 - 5 t/m² for various widths of foundations for an allowable settlement of 50mm.

Precautions :

For Straight Bored Piles, after reaching the required depth in the pile bores, 15cm thick layer of gravel should be placed and compacted at the bottom so that the slush formed at the bottom is diminished.

For Isolated / Strip Footings, foundation surface must be compacted heavily. If any loose pockets are observed, the same shall be filled with brickbats/gravel and compacted. Foundations can subsequently be placed over such a prepared surface.



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1.0 INTRODUCTION

- 1.1 Pepsico India Holdings Private Limited, Concentrate Division, District Sangrur, Punjab - 148026 are in the process of constructing a Factory Complex at Village Nalbari, Assam. Space Geo Tech, "Sri Guru Krupa", #985, 18th A Main, 66th Cross, 5th Block, Rajajinagar, Bangalore - 560010 are the Consultants for the proposed project.
- 1.2 To design the substructures for the proposed structures, adequate information regarding the subsoil conditions is required. For this purpose, detailed geotechnical investigations had been undertaken.
- 1.3 This report contains the details of the geotechnical investigations conducted along with the results and analysis of the investigations and the recommendations thereof.
- 1.4 Geotechnical investigations for this purpose have been carried out as per the verbal authorization of the authorised signatory of Space Geo Tech. This authorization has been given in response to our offer no NCD/Q/SGT/151/2022 dated 7th January 2023.

2.0 PROJECT DETAILS

2.1 Site Location

- 2.1.1 The site for the proposed project is located near Village Banbhag Solmari, Nalbari, Assam and is situated at a distance of about 6 km from Nalbari Railway Station towards Baram, Nalbari, Assam.

2.2 Site Layout and Topography

- 2.2.1 A schematic site plan showing the dimensions and other details of the site is enclosed in this report (fig.1a).



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2.2.2 Maximum level differences of about 2m had been observed at the site, during the period of field investigation. However, the general level of the site had been observed to be about 1m below the nearby road level.

2.2.3 Three numbers of small ponds had been observed to be existing at the site, during the period of field investigations.

2.2.4 Vegetation in the form of grass and occasional trees had been observed at the site, during the period of field investigations.

2.2.5 The colour of the surface earth had been observed to be Yellowish brown.

2.3 Seismic Zone

The present site is located in the Seismic Zone V which is the zone of highest seismic intensity, as per the seismic zoning map of India given in BIS code IS:1893 (Part1)-2016.

2.4 The Structure/s

2.4.1 The proposed structures are Factory Buildings.

2.4.2 The structures are understood to be framed ones and the construction to be of Reinforced Cement Concrete.

3.0 OBJECT OF INVESTIGATIONS

3.1 For designing the foundation system of the proposed structures, the following data are required :

- a) Type of foundation
- b) Depth below the ground level at which the foundation system is to be laid
- c) Allowable bearing pressure at the foundation level

3.2 To determine the above factors, the following information would be required:

- a) The subsoil profile indicating thickness of the various soil strata, to a depth within



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the influence zone below the foundations

- b) Engineering properties of the soil strata at various levels
- c) Physical characteristics of the soil strata
- d) Variation of strength of soil strata with depth

3.3 For evaluating the above parameters, field investigations and laboratory investigations on the soil samples collected during the field investigations, have been carried out.

3.4 The results from these investigations have been analysed to provide the recommendations for the design of foundations.

4.0 SCOPE OF INVESTIGATIONS

4.1 The scope of investigations as stipulated by the client consists of :

- a) Conducting boreholes in soil strata down to 20m depth at ten locations and down to 30m depth at one location.
- b) Conducting Cross hole seismic test at one location.
- c) Opening Trial Pits (TP) down to a maximum depth of 1.5m below the existing ground level, at five locations. Also collecting soil samples for CBR tests from all the trial pits.
- d) Conducting field density at two locations
- e) Conducting field permeability test in one boreholes at five different depths.
- f) Conducting laboratory permeability test on eleven soil samples collected from the site.
- g) Conducting laboratory California Bearing Ratio (CBR) test on five soil sample collected within the site, inclusive of conducting modified proctor density test for determination of optimum moisture contents and maximum dry densities.



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- h) Conducting laboratory Standard Proctor test on six soil samples collected from the site.
- i) Conducting chemical analysis on soil/ water samples.
- j) Conducting relevant laboratory tests on soil samples recovered.
- k) Preparation and submission of a technical report in three copies containing the details of the tests carried out, their analysis and recommendations regarding the foundation system to be adopted.

4.1.2 The following operations were to be undertaken while progressing the boreholes:

- a) Conducting standard penetration tests at 1.5/3m intervals.
- b) Recovering undisturbed soil samples from various levels of the subsoil strata.
- c) Recording ground water table levels, if met with.

5.0 FIELD INVESTIGATIONS

5.1 Preliminary Details

5.1.1 Field investigations had been carried out between 5th February 2023 and 2nd March 2023.

5.1.2 A schematic site plan showing the tests locations is given in fig. 1a.

5.1.3 Photographs showing view of the site and tests under progress are also enclosed in this report (fig. 1b to 1d).

5.2 Boreholes

5.2.1 Boreholes were progressed by shell and auger method. Casing pipe were used to stabilize the sides of the boreholes below the water table level.

5.2.2 All the boreholes (BH1 to BH11) had been progressed down to the stipulated depth of 20m below the existing ground level except borehole BH3 which had been progressed down to 30m depth below the existing ground level.



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- 5.2.3 The diameter of the boreholes was 150/100mm.
- 5.2.4 Standard penetration tests were conducted at 1.5/3m intervals. Disturbed soil samples recovered from split spoon samplers were retained for identification purposes.
- 5.2.5 Undisturbed soil samples were recovered by thin walled Shelby tubes conforming to IS : 2132. These tubes had an area ratio of less than 10%.
- 5.2.6 The diameter of undisturbed soil samples was 50mm and the length was 45cm.
- 5.2.7 The ends of sample tubes were sealed by wax to prevent loss / ingress of moisture. Disturbed soil samples were enclosed in polythene bags.
- 5.2.8 The samples thus recovered were transported to the laboratory for testing purposes
- 5.2.9 Ground water table had been encountered in all the boreholes at a depth varying between 1.5m and 2.7m below the existing ground level, during the period of field investigations i.e February 2023.

5.3 Cross Hole Seismic Test

5.3.1 Preliminary Details

- 5.3.1.1 The cross hole seismic test is carried out to determine the dynamic shear modulus used in the design of the machine foundations.
- 5.3.1.2 Cross Hole Seismic Test had been conducted at the location as shown in the schematic site plan, fig no. 1a.
- 5.3.1.3 For carrying out the cross hole seismic test, 3 boreholes located in-line at a spacing of 3m had been progressed down to a maximum depth of 20m below the existing ground level and the cross hole test had been conducted at regular depth intervals of 1.5m.



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5.3.2 Principle of the Test

- 5.3.2.1 The cross hole seismic test is carried out to determine the shear and compression wave velocities and thence the dynamic shear moduli and the dynamic elastic moduli in the soil/ rock strata.
- 5.3.2.2 The test involves the installation of three component geophones (i.e. one vertical and two horizontal mounted in orthogonal directions), in two prebored boreholes at a desired depth and generation of a vibration impulse in the soil by use of suitable seismic sources at the same desired depth as above in a third borehole inline with and to one side of the two boreholes in which the geophones had been installed.
- 5.3.2.3 The vibration impulse generated is recorded by the geophones in the two predrilled boreholes. The analysis of the vibrations recorded by the geophones gives the time taken for the vibration impulse to travel from the first set of geophones installed in the first borehole to the second set of geophones installed in the second borehole.
- 5.3.2.4 The shear and compression wave velocities in the soil strata are then determined as the ratio of the known distance between the boreholes and the time taken for the different types of waves to travel between the boreholes.
- 5.3.2.5 From the so determined wave velocities, the dynamic shear modulus and the dynamic elastic modulus are determined.
- 5.3.2.6 The test is repeated at different depth intervals down to the desired maximum depth at which the test is to be conducted.



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5.3.2.7 The results of the cross hole seismic test are analysed to provide the profile of the dynamic elastic modulus of the soil/rock strata with depth.

5.3.3 Test Setup

5.3.3.1 Three boreholes had to be conducted in line at the required spacing for carrying out the test. Two of the three boreholes conducted for the cross hole test in which the geophones had been installed, had been predrilled down to required depth below the existing ground level. Whereas the third borehole in which the seismic source had been positioned, had been progressed at intervals down to the maximum depth at which the cross hole test had to be conducted.

5.3.3.2 PVC Casing pipes had been used in the boreholes to support the sides of the boreholes and thus prevent collapse of the boreholes. The gap between the casing pipe and the walls of the boreholes had been suitably filled with soil slurry to ensure complete contact between the casing and the surrounding soil and thus obtain satisfactory recording of the vibrations.

5.3.3.3 The three component geophones (i.e. one vertical and two horizontal components mounted orthogonally) had been installed in each of the two boreholes at the desired depth.

5.3.3.4 The geophones had been connected to a high speed data acquisition card installed on a laptop computer for recording the signals from the geophones directly onto the laptop computer.

5.3.3.5 The seismic source used in the test had been a hammer blow given by a 65kg hammer falling through a height of about 1m onto a split spoon sampler resting at the desired depth in the third borehole.



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5.3.3.6 The test had been conducted at depth intervals of 1.5m starting from 1.5m depth below the existing ground level down to maximum 30m depth below the existing ground level.

5.4 Trial Pits

5.4.1 Five Trial Pits (i.e. of size 1.5m × 1.5m) viz TP1 to TP5 had been excavated down to 1.5m depth below the existing ground level at different locations for collection of disturbed and undisturbed soil samples at depths of 0.5m, 1m and 1.5m below the existing ground level. Additionally, bulk disturbed samples were also collected at the above depths for conducting laboratory CBR tests.

5.5 Field Permeability Tests

5.5.1 Field permeability tests had been conducted in boreholes BH11 at various depths of 1m, 6m, 11m, 16m and 20m below the existing ground level.

5.5.2 The permeability tests had been conducted by falling head method using the open ended casing method.

5.5.3 All the permeability tests had been conducted below the water table level.

5.5.4 In this method, a borehole had been conducted down to the level at which test had been performed. The casing had been simultaneously sunk down to the full depth of the borehole as the boring of the borehole was in progress. After the required level had been reached, the borehole had been properly cleaned.

5.5.5 After the borehole had been cleaned, the borehole had been filled with water upto a fixed level and the fall in level against time had been recorded. The test is repeated three times to check for consistency in the observations of fall in water level against time as per the subsoil conditions.



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6.0 *LABORATORY INVESTIGATIONS*

6.1 Tests on soil samples from boreholes

6.1.1 The soil samples brought to the laboratory were subjected to various tests to determine the following properties :

- a) Type of soil and its gradation
- b) Consistency limits
- c) Natural Bulk Density & Water Content
- d) Strength parameters like cohesion, angle of shearing resistance

6.1.2 In order to determine the above properties, the following tests have been conducted :

- a) Sieve analysis on coarse grained soil fraction
- b) Hydrometer analysis on fine grained soil fraction
- c) Liquid and plastic limits tests
- d) Natural Density and Water Content determination
- e) Triaxial compression tests
- f) Free Swell Index tests

6.2 Chemical Analysis

6.2.1 The water samples and soil samples collected from the boreholes have been tested to determine the pH-value and the presence of salts harmful to reinforced cement concrete construction namely Chloride and Sulphate contents.

6.3 Standard Proctor Density Tests

6.3.1 Disturbed samples for standard proctor density test had been collected from the locations adjacent to respective boreholes BH4, BH5, BH6, BH8, BH9 and BH11 (as indicated in fig. 1a) from a depth of 0.5m below the existing ground level.



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- 6.3.2 The standard proctor density test is conducted to determine the maximum dry density and corresponding optimum moisture content.
- 6.3.3 Standard proctor density test have been conducted on the samples collected from the site from various locations.
- 6.3.4 The standard proctor density test is conducted on a representative portion of air dried soil sample finer than 20mm.
- 6.3.5 The test involves first mixing thoroughly the air dried soil sample with a fixed known amount of water and filling the mixed soil in a mould of 1000cc capacity with base plate attached, in three different layers of approximately equal weight with simultaneous compaction of each layer by giving 25 blows from a standard 2.5 kg hammer falling through a standard 31cm height above the soil with the blows being distributed uniformly over the surface of each layer.
- 6.3.6 The density and the water content of the soil in the mould is determined. This process is repeated by gradually increasing the water content of the soil by adding fixed quantities of water till the measured densities after compaction show a decrease with increase in water content.
- 6.3.7 The maximum dry density and optimum moisture content of the soil is then determined from a plot of the measured dry density vs the corresponding measured water content of the soil.

6.4 Modified Proctor Density Tests

- 6.4.1 The modified proctor density tests are conducted to determine the maximum dry densities and corresponding optimum moisture contents of the soil samples. The remoulded soil samples for the laboratory CBR tests have to be prepared at these maximum dry densities



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and optimum moisture contents.

6.4.2 The modified proctor density test is conducted on a representative portion of air dried soil sample finer than 20mm.

6.4.3 The test involves first mixing thoroughly the air dried soil sample with a fixed known amount of water and filling the mixed soil in a mould of 1000cm³ capacity with base plate attached, in five different layers of approximately equal weight with simultaneous compaction of each layer by giving 25 blows from a standard 4.9kg hammer falling through a standard 45cm height above the soil surface with the blows being distributed uniformly over the surface of each layer.

6.4.4 The density and the water content of the soil in the mould is determined. This process is repeated by gradually increasing the water content of the soil by adding fixed quantities of water till the measured densities after compaction show a decrease with increase in water content.

6.4.5 The maximum dry density and optimum moisture content of the soil is then determined from a plot of the measured dry density vs the corresponding measured water content of the soil.

6.5 Laboratory California Bearing Ratio Tests

6.5.1 The soil samples for the determination of California Bearing Ratio had been collected from the locations adjacent to respective boreholes BH1, BH2, BH3, BH7 and BH10 (as indicated in fig. 1a) from a depth of 0.5m below the existing ground level.

6.5.2 The soil samples for the laboratory CBR tests are prepared in a standard CBR mould by compacting the soil having the optimum moisture content, to the maximum dry density obtained from the modified proctor density tests. To prepare such a soil sample for the



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laboratory CBR test, oven dried soil sample passing through 20mm of about 5kg weight is taken filled into the CBR mould in five layers by giving 55 blows for each layer from a standard 4.9kg hammer falling through a standard height of 45cm. After preparation of the soil sample, the CBR mould containing the soil sample is kept submerged in water under a surcharge weight of 5kg for a period of 96 hours to facilitate soaking of the soil sample.

6.5.3 After 96 hours of soaking, the CBR mould containing the soil sample is taken out of the water and placed on a load frame wherein a plunger of 50mm dia is made to penetrate into the soil sample in the CBR mould at a displacement rate of 1.25mm/min. The load required to penetrate the plunger into the soil, is recorded corresponding to different penetration depths of the plunger.

6.5.4 A curve of loading intensity vs penetration is plotted. From the analysis of this curve the CBR values are determined as given below :

$$CBR \text{ value} = \frac{P_t}{P_s} \times 100$$

where P_t = corrected loading intensity corresponding to the chosen penetration from the load penetration curve for the test

P_s = standard loading intensity for the same depth of penetration

6.6 Laboratory Permeability Test

6.6.1 The coefficient of permeability 'k' has been determined in the laboratory, for selected samples by falling head method using the Oedometer.



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7.0 RESULTS & ANALYSIS

7.1 Presentation of Results

7.1.1 The results of the boreholes investigations have been presented in the form of soil profile tables.

7.1.2 The soil profile tables indicate the following:

- a) Standard penetration test values at various depths
- b) Soil description identifying the type of soil
- c) Grain size analysis indicating composition of subsoil
- d) Atterberg limits
- e) Natural density and water content
- f) Triaxial compression test results

7.2 Soil Profile

7.2.1 A perusal of the data presented in the soil profile tables indicates that the subsoil mainly consists of the following three strata in the eleven boreholes:

- a) Stratum - I : Sandy clayey silt with occasional gravel/ clayey sandy silt
- b) Stratum - II : Silty sand with occasional clay and gravel
- c) Stratum - III : Sand with traces of silt and occasional gravel/ sand with silt

7.2.2 The thickness of the three strata in the eleven boreholes are as follows :

BH. No.	Strata (depth in m : from : to)		
	Stratum - I	Stratum - II	Stratum - III
1	0 - 7.5 14.8 - 20	7.5 - 10.2	10.2 - 14.8



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BH. No.	Strata (depth in m : from : to)		
	Stratum - I	Stratum - II	Stratum - III
2	0 - 2.8 3.5 - 6.9 8.4 - 9.9 15.5 - 17.8	2.8 - 3.5 6.9 - 8.4 17.8 - 20	9.9 - 15.5
3	0 - 9 15.5 - 23.2 25.5 - 30	23.2 - 25.5	9 - 15.5
4	0 - 8.4 14.4 - 20	8.4 - 9.9	9.9 - 14.4
5	0 - 9 15 - 17.2	17.2 - 19	9 - 15 19 - 20
6	0 - 11 15.9 - 20	N.E.	11 - 15.9
7	0 - 8.2	8.2 - 9 15.7 - 19	9 - 15.7 19 - 20
8	0 - 9.9 14.8 - 17.8	9.9 - 14.8	17.8 - 20
9	1.5 - 10.5 15.7 - 20	0 - 1.5 10.5 - 12	12 - 15.7
10	0 - 9.9 15.9 - 17.4	17.4 - 20	9.9 - 15.9
11	0 - 9.7 15.5 - 20	N.E	9.7 - 15.5

N.E. : not encountered

7.2.3 The above results show that :

- a) Stratum - I consisting predominantly of silty soils with varying percentages of clay and sand, is predominant down about 10m depth below the existing ground level. Additionally, this stratum has also been encountered beyond 14.4m depth down to the maximum depth investigated except for occasional thin layers of Stratum - II



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and Stratum - III inbetween down to a maximum depth of about 25.5m depth.

- b) Stratum - II consisting predominantly of sandy soils with significant percentages of silt and occasional clay, has been encountered in occasional thin layers down to about 3.5m depth below the existing, between 7.5m and 12m depths and also between 15.7m and 25m depths, in random boreholes.
- c) Stratum - III consisting predominantly of sandy soils, has been encountered between 9m and 16m depths below the existing ground level in all the boreholes. Additionally, this stratum has also been encountered between 17.8m and 20m depths in occasional boreholes.

7.3 Soil Composition

7.3.1 The grain size distributions of the soil samples collected from various depths in the eleven boreholes have been presented in the form of grain size analysis curves in figs. 9a to 9u.

7.3.2 The variations in the grain size distributions in each of the three strata in the eleven boreholes are as follows:

- a) Stratum - I : Sandy clayey silt with occasional gravel/ clayey sandy silt

BH. No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	0 - 8	2 - 22	44 - 79	10 - 44
2	0	10 - 53	37 - 73	8 - 32
3	0	2 - 33	46 - 62	8 - 42
4	0	5 - 20	53 - 75	17 - 42
5	0	5 - 40	53 - 64	6 - 36
6	0	4 - 29	46 - 75	8 - 40
7	0	4 - 36	47 - 60	14 - 50
8	0	4 - 25	57 - 71	8 - 32



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BH. No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
9	0	3 - 39	49 - 76	10 - 38
10	0	3 - 37	51 - 79	11 - 46
11	0 - 2	5 - 39	39 - 76	9 - 40

b) Stratum - II : Silty sand with occasional clay and gravel

BH. No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	0	68 - 77	23 - 32	0
2	0	53 - 67	32 - 37	10
3	0	73	27	0
4	0	71	29	0
5	3	73	24	0
6		N.E.		
7	0	44 - 70	29 - 39	0 - 17
8		N.E.		
9	0	56 - 59	35 - 40	4 - 6
10	0	48	38	14
11		N.E.		

N.E. : not encountered

c) Stratum - III : Sand with traces of silt and occasional gravel/ sand with silt

BH. No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	0	93 - 97	3 - 7	0
2	0	91 - 97	3 - 9	0
3	0	93 - 97	3 - 7	0
4	0	95 - 98	2 - 5	0
5	0 - 2	91 - 98	2 - 7	0
6	0	89 - 96	4 - 11	0



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BH. No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
7	0 - 2	94 - 97	3 - 4	0
8	0	85 - 97	3 - 15	0
9	0	93 - 98	2 - 7	0
10	0	95 - 98	2 - 5	0
11	0 - 2	95 - 98	2 - 4	0

7.3.3 The above results indicate that :

- Stratum - I consists of about 37% to 79% of silt, 2% to 53% of sand with 6% to 46% of clay and occasional gravel.
- Stratum - II consists of about 44% to 77% of sand and 23% to 40% of silt with occasional clay and gravel.
- Stratum - III consists of an average of about 90% of sand and 10% of silt with occasional gravel.

7.4 Natural Density and Water Content

7.4.1 The natural bulk densities, water contents and dry densities of soil samples collected at various depths in eleven boreholes vary as follows :

BH. No.	Bulk Density (g/cm ³)	Water Content (%)	Dry Density (g/cm ³)
1	1.85 - 1.98	17.7 - 28.7	1.52 - 1.60
2	1.77 - 2.03	14.8 - 29.9	1.50 - 1.75
3	1.84 - 2.00	15.0 - 26.8	1.54 - 1.65
4	1.88 - 2.02	15.9 - 26.4	1.56 - 1.71
5	1.94 - 2.02	17.5 - 26.7	1.54 - 1.71
6	1.74 - 2.07	16.4 - 28.3	1.46 - 1.76
7	1.92 - 2.05	12.9 - 25.6	1.52 - 1.73



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BH. No.	Bulk Density (g/cm³)	Water Content (%)	Dry Density (g/cm³)
8	1.70 - 2.03	10.9 - 25.3	1.53 - 1.70
9	1.92 - 2.03	21.2 - 26.3	1.56 - 1.65
10	1.92 - 2.10	11.9 - 27.2	1.53 - 1.78
11	1.83 - 2.07	18.7 - 26.6	1.52 - 1.74

7.4.2 The variations in the dry densities of the soil have also been presented in the form of plots of dry density vs depth for the eleven boreholes conducted, in figs. 3a to 3c.

7.4.3 The above results indicate that the subsoil is in a loose to medium dense state down to the maximum depth investigated.

7.5 Atterberg Limits

7.5.1 The Atterberg limits indicate that the subsoil is predominantly low plastic down to about 10m depth and also beyond 14.4m depth down to the maximum depth investigated below the existing ground level except for occasional thin layers of non-plastic soils in between. Additionally, medium plastic soils have also been encountered at occasional depths in some boreholes. Notably, the subsoil between 10m and 15m depths is predominantly non - plastic.

7.6 Standard Penetration Test Values (N-values)

7.6.1 The observed Standard Penetration Test values (N-values) vary between 2 and 40 as indicated in the soil profile tables and as also shown in the figs. 4a to 4c wherein the observed N-values have been plotted with respect to depths.

7.6.2 The above results indicate that the subsoil is in a loose to medium dense state down to the maximum investigated depth.



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7.7 Triaxial Test Results

7.7.1 The results of triaxial tests are indicated in the respective soil profile tables. These results have been considered in providing the recommendations.

7.8 Compiled Soil Profile

7.8.1 An overview of the results and their analysis has been presented in the form of a compiled soil profile (fig. 2).

7.8.2 The above figure shows the various strata encountered and their thicknesses in each of the boreholes and also gives the soil composition and the observed N - values at various depths.

7.9 Free Swell Index Tests

7.9.1 As significant percentages of clay have been observed in thin layers in some boreholes at occasional depths, free swell index tests have been conducted on selected soil samples at these depths of different boreholes to assess the swelling characteristics of the subsoil.

7.9.2 The results of the free swell index tests are given below :

Borehole No	Depth (m)	FSI (%)
1	3.9	Nil
1	5.4	Nil
1	17.4	Nil
2	4.5	Nil
3	3.9	Nil
3	15.9	3.7
4	4.5	Nil
4	7.5	Nil
4	15	Nil



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Borehole No	Depth (m)	FSI (%)
5	3.9	Nil
6	4.5	Nil
6	7.5	Nil
7	3.9	Nil
7	6.9	Nil
8	4.5	Nil
9	3.9	Nil
9	9.9	Nil
10	3	Nil
10	4.5	Nil
11	2.4	Nil
11	5.4	Nil

7.9.3 The above results indicate that the free swelling index of the subsoil is Nil.

7.10 Field Density

7.10.1 The results of field density, dry density and water content for two undisturbed soil samples collected from 0.3m depth below the existing ground level are given below :

Test no.	In-situ Density (g/cm ³)	Water Content Content(%)	Dry Density gm/cm ³
1	2	19.1	1.68
2	2.02	15.6	1.73

7.11 Proctor Density Tests

7.11.1 Modified Proctor density tests have been conducted on the samples collected for the CBR tests from the specified locations, for the purpose of preparation of CBR samples.

7.11.2 However, Standard proctor density tests have also been conducted on the soil samples collected from the specified locations.



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7.11.3 The results of the Modified Proctor density and Standard Proctor density tests have been presented in the form of plots of dry density vs water content in figs. 7a to 7k.

7.11.4 The maximum dry densities and corresponding optimum moisture contents (OMC) determined from the above given tests for the various samples are as follows :

a) Modified Proctor Density

Modified Proctor Density		
Test No.	Maximum Dry Density (g/cm³)	Optimum Moisture Content (%)
1	1.87	13.4
2	1.91	11.6
3	1.85	13.2
7	1.84	11
10	1.95	10.6

b) Standard Proctor Density

Standard Proctor Density		
Test No.	Maximum Dry Density (g/cm³)	Optimum Moisture Content (%)
4	1.77	14.4
5	1.72	12.4
6	1.7	14.4
8	1.69	14.8
9	1.6	17.6
11	1.57	17.4



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7.11.5 The results of Modified Proctor density indicate that the top soil at the site has maximum dry density or Modified Proctor density varying between 1.84 g/cm³ and 1.95g/cm³ for optimum moisture content varying between 11% and 13.4 %.

7.11.6 The results of Standard Proctor density indicate that the top soil at the site has maximum dry density or Standard Proctor density varying between 1.57 g/cm³ and 1.77g/cm³ for optimum moisture content varying between 12.4% and 17.6 %.

7.12 Laboratory CBR Tests

7.12.1 The results of the laboratory CBR tests conducted on soil samples prepared at the Modified Proctor densities and soaked for 96 hours, have been presented in the form of plots of load intensity vs penetration in figs. 8a to 8e.

7.12.2 The laboratory CBR values corresponding to 2.5mm and 5mm penetration respectively, obtained after due corrections to the load intensity vs penetration plot, are as follows :

CBR No.	CBR 2.5mm	CBR 5mm
1	12.1	13.3
2	6.3	6.9
3	20.7	20
7	12.1	17.1
10	19.3	28.1

7.12.3 The above results indicate that the laboratory CBR values corresponding to 2.5mm penetration are generally lower than the laboratory CBR values at 5mm penetration except for occasional tests wherein the laboratory CBR values corresponding to 5mm penetration are lower than the laboratory CBR values at 2.5mm penetration. Hence, lower of the two values may be adopted for design purposes.



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7.13 Chemical Analysis

7.13.1 The results of the chemical analysis conducted on water and soil samples collected from the twenty boreholes, for determining the presence of any harmful salts which can have adverse effects on concrete construction, are as follows :

a) Water Samples

Borehole no.	pH value	Chloride Content (ppm)	Sulphate Content (ppm)
BH1	6.5	32	41
BH2	6.5	28	38
BH3	6.5	29	39
BH4	7	34	45
BH5	6.5	18	62
BH6	7	32	58
BH7	7	26	48
BH8	6.5	22	56
BH9	6.5	35	45
BH10	6.5	30	41
BH 11	7	37	51

b) Soil Samples

Borehole no./ Depth (m)	pH value	Chloride Content (ppm)	Sulphate Content (ppm)
BH1/2.4m	6	22	28
BH2/4.5m	6	16	42
BH3/3.9m	6	21	36
BH4/7.5m	6	22	51
BH5/9.9m	6.5	24	40
BH6/6m	6	18	28



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Borehole no./ Depth (m)	pH value	Chloride Content (ppm)	Sulphate Content (ppm)
BH7/8.4m	6.5	16	25
BH8/12m	6.5	26	26
BH9/11.4m	7	32	17
BH10/9m	6	18	25
BH11/14.4m	6	20	19

IS LIMITS

<i>pH value</i>	<i>Not less than 6</i>
<i>Chloride content (ppm)</i>	<i>Maximum 500 ppm</i>
<i>Sulphate content (ppm)</i>	<i>Maximum 400 ppm</i>

7.13.2 The above results of chemical analysis are within the IS limits indicating that the water and soil encountered in the boreholes shall not have any aggressive effect on normal concrete construction work.

7.14 Field Permeability Tests

7.14.1 The permeability by *falling head method* is obtained from the following relation.

$$k = \frac{\pi \cdot r}{5.5(t_2 - t_1)} \times \log_e \frac{H_1}{H_2}$$

k = coefficient of permeability(cm/s)

H₁ = Piezometric head at time t₁

H₂ = Piezometric head at time t₂

r = Inside radius of casing (cm) = 10cm

7.14.2 The coefficient of permeability 'k' determined at in boreholes BH11 at various depths by falling head method has been given hereunder.



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Sl. No.	Depth (m)	Coefficient of Permeability, k (cm/sec)
1	1m	Impermeable
2	6m	8×10^{-7}
3	11m	4.8×10^{-6}
4	16m	Impermeable
5	20m	Impermeable

7.14.3 The above values of the coefficient of permeability are in the range typically observed for silty soils with significant percentages of clay.

7.15 Laboratory Permeability Tests

7.15.1 The coefficient of permeability 'k' determined for various insitu soil samples collected from depths in various boreholes by falling head method in the laboratory has been given hereunder.

Sl. No.	Borehole No./ Depth (m)	Coefficient of Permeability, k (cm/sec)
1	BH1/1.5m	1×10^{-3}
2	BH2/2.4m	1.1×10^{-4}
3	BH3/3m	5.4×10^{-5}
4	BH4/5.4m	1.7×10^{-4}
5	BH5/4.5m	1.1×10^{-6}
6	BH6/6.9m	2.5×10^{-4}
7	BH7/6m	Impermeable
8	BH8/8.4m	9.7×10^{-7}
9	BH9/9m	Impermeable
10	BH10/9.9m	4.7×10^{-4}
11	BH11/16.5m	Impermeable



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7.15.2 The above values of the coefficient of permeability are in the range typically observed for silty sand, silty soils with significant percentages of clayey.

7.16 Cross Hole Seismic Test

7.16.1 The results of the cross hole seismic test have been presented in the form of plots of wave velocities vs depth and dynamic elastic moduli vs depth in figs. 5a & 5b for the respective tests.

7.16.2 The results of the cross hole seismic test have also been presented in the form of a table giving the wave velocities and dynamic elastic moduli at different depths in Annexure - C1 of this report.

7.16.3 The shear wave velocity ' V_s ' and the compression wave velocity ' V_p ' have been determined as the ratio of the known distance between the boreholes and the respective travel times for the different waves as determined from the analysis of the recorded signals.

7.16.4 The dynamic shear modulus ' G_d ' is determined using the following formula :

$$G_d = \rho \cdot V_s^2$$

where ρ = density of the soil/rock = γ/g

γ = unit weight of the soil/rock in g/cm^3

g = acceleration due to gravity = 981 cm/s^2

7.16.5 The dynamic Poisson's ratio ' μ_d ' is determined as the mean of the values determined at each depth at which the test has been conducted, using the following formula :

$$\mu_d = \frac{\left(\left(\frac{V_p}{V_s} \right)^2 - 2 \right)}{2 \cdot \left(\left(\frac{V_p}{V_s} \right)^2 - 1 \right)}$$



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7.16.6 The dynamic elastic modulus 'E_d' is then determined using the following formula :

$$E_d = G_d \cdot 2 \cdot (1 + \mu_d)$$

7.16.7 The bulk elastic modulus 'K_d' is then determined using the following formula :

$$K_d = \frac{E_d}{3 \cdot (1 - \mu_d)}$$

7.17 Trial Pits

7.17.1 Trial pits had been excavated down to 1.5m depth below the existing ground level. The subsoil strata had been observed to be consisting predominantly of yellowish brown clayey sandy silt/ sandy clayey silt down to the depth excavated.

7.17.2 Photographs showing the cross sections of the trial pits have been given figs. 6a to 6d.

7.17.3 The grain size distributions and insitu properties of the of the soil samples collected from 0.5m, 1m and 1.5m depths in the Trial Pits are given below :

TP No.	Depth (m)	Grain Size Distribution				Insitu Properties		
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Insitu Density (g/cm ³)	Water Content (%)	Dry Density (g/cm ³)
TP1	0.5	0	9	63	28	1.87	16.3	1.61
	1	0	8	65	27	1.85	17.2	1.58
	1.5	0	18	70	12	1.83	19.5	1.53
TP2	0.5	0	13	65	22	1.93	20.5	1.6
	1	0	24	59	17	1.88	21.5	1.54
	1.5	0	9	73	18	1.94	24.8	1.55
TP3	0.5	0	25	56	19	1.76	13.2	1.55
	1	2	34	51	13	1.95	22.2	1.59
	1.5	2	7	73	18	1.98	23.9	1.6
TP4	0.5	0	31	52	17	1.97	16.3	1.69



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TP No.	Depth (m)	Grain Size Distribution				Insitu Properties		
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Insitu Density (g/cm ³)	Water Content (%)	Dry Density (g/cm ³)
	1	0	36	56	8	1.86	17.7	1.57
	1.5	0	17	73	10	1.96	21.8	1.56
	0.5	0	31	54	15	1.89	15.6	1.63
TP5	1	0	51	44	5	1.88	18.4	1.58
	1.5	0	41	56	3	1.84	18.2	1.55

8.0 ANALYSIS OF LIQUEFACTION SUSCEPTIBILITY

- 8.1.1 Analysis of the susceptibility to liquefaction of the subsoil at the site has been carried out in the form of a comparison of the shear stress induced/developed in event of an earthquake with the in-situ shear resistance of the subsoil at various depths in the different boreholes.
- 8.1.2 This method has been found to be very reliable and is used as one of the main criteria in almost all countries. The shear stress induced/developed at different depths during an earthquake depends upon the overburden pressure and the soil characteristics while the shear resistance is calculated based on the N-values and other soil properties for a maximum repeated strain of 2%.
- 8.1.3 The shear stress induced/developed and the shear resistance offered by the soil are evaluated in terms of the Cyclic Stress Ratio (CSR) and the Cyclic Resistance Ratio (CRR) respectively. The CSR is the ratio of the shear stress developed to the effective overburden pressure at various depths while the CRR is the ratio of the shear resistance of the soil to the effective overburden pressure at various depths. The analysis of variation of the CSR and the CRR with depth based on the results of each of the boreholes conducted, have been given in figs C1 to C11 of this report.



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- 8.1.4 The soil will liquefy in the depth zones where the CSR exceeds the CRR i.e. where the shear stress developed exceeds the shear resistance or in other words, where the factor of safety defined as the ratio of the CRR to CSR, is less than 1.
- 8.1.5 Subsoil strata predominantly consists of silty soils down to about 10m depth and also beyond 15m depth down to the maximum depth investigated except occasional thin layers of sandy soils inbetween. However the above analysis of liquefaction indicate that the subsoil strata predominantly consisting of sandy soils between 10m and 15m depths, is susceptible to liquefaction.
- 8.1.6 As the sandy soils layer is sandwiched (i.e. bound above and below) by silty soils which generally have very low permeability, the drainage of water from within the layers of sandy soils cannot occur during the short duration of an earthquake. Hence, while the sandy soil in this layer may liquefy, the possible significant ground settlements can not be ruled out.

8.2 Comments on Liquefaction Analysis

- 8.2.1 Considering the above detailed analyses and also that the site is located in Seismic Zone - V of seismic zoning map of India, the subsoil strata between 10m and 15m depths can be considered to be susceptible to liquefaction. Hence, both shallow foundations as well as deep foundations will not be feasible unless appropriate ground improvement is carried out to mitigate the susceptibility to liquefaction of the subsoil at the site.
- 8.2.2 At the present site, the subsoil consists of sandy soils sandwiched between silty soils. Under such soil conditions, and considering the depth of soil required to be compacted and the large area of the site which needs to be improved, the most appropriate method of ground improvement is the Vibro compaction method as this is a relatively faster and more economical method.



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9.0 DESIGN CRITERIA

9.1 Design Parameters

9.1.1 The parameters required for the design of the foundation system for the proposed structures are:

- a) Type of foundation to be adopted
- b) Depth at which the foundations have to be laid
- c) Allowable bearing pressure on the soil/rock at the foundation level

9.1.2 On the basis of the analysis of the results of the investigations, the required design parameters have been arrived at as given in the following sections.

9.2 Type of Foundation

9.2.1 The type of foundation depends upon the following :

- a) Subsoil/rock conditions
- b) Type of structure
- c) Configuration of loading points
- d) Loading intensity on each column at the foundation level

9.2.2 The structures are Factory Buildings. Considering the above, medium heavy loads can be anticipated on the foundations.

9.2.3 The results of the investigations have shown that the subsoil is susceptible to liquefaction between 10m and 15m depths below the existing ground level. Considering the subsoil strata consisting of sandy soils in loose to medium dense state under submerged condition, relatively down deeper depths and for significantly larger area, an appropriate ground improvement method has to adopt to improve the ground conditions down to 15m depth below existing ground level.



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9.2.4 As discussed in the earlier sections, under the prevailing subsoil, ground improvement by Vibro compaction method has to be carried out for the densification of the subsoil strata and also to eliminate the susceptibility to liquefaction. The foundations for the proposed structures can be placed over such improved ground.

9.2.5 In view of the above, the proposed structures can be supported **on *Straight Bored Pile Foundations***.

9.2.6 However, minor structures can be supported over ***Isolated / Strip Footings*** at ground level.

9.3 Depth of Pile Foundation

9.3.1 The minimum depth of pile foundations depends upon the following factors :

- a) Top filled-up strata / loose soil, if any
- b) Adequate depth of soil strata below the founding level of the pile foundations of requisite strength to mobilize the safe end bearing capacity
- c) Adequate depth of soil strata above the founding level of the pile foundations of requisite strength to mobilize the safe friction capacity.

9.3.2 Taking the above into consideration, the pile foundations for the proposed structures shall be made to rest at a depth greater than 22m below the existing ground level. Accordingly, considering the cut-off level of the piles as about 2m below the existing ground level, effective length of piles of **20m and 25m**, the founding levels of the piles will be about **22m and 27m**, below the existing ground level.

9.3.3 The soil encountered at the founding levels of the pile foundations will either be ***yellowish grey to grey sandy clayey silt or silty sandy soils***.



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9.4 Pile Capacities

9.4.1 Load carrying capacities of bored pile foundation have been analysed and given for piles having various diameters and effective lengths.

9.4.2 The parameters considered for the analysis of the pile capacities are given below :

S. No.	Parameter	Value/s
1	Effective length (m)	20 and 25
2	Cohesion (kg/cm ²)	0.15 & 0.05
3	Angle of shearing resistance	15° , 18° and 26°
4	Submerged density (g/cm ³)	1
5	SPT Value at pile tip	15
6	Factor of safety	2.5

9.4.3 Considering the above parameters, the following safe pile capacities have been recommended for various diameters and effective lengths of piles.

Pile Dia (mm)	Pile Effective Length (m)	Pile Vertical Load Capacity (tons))	Uplift Capacity of pile (tons)	Safe Lateral Capacity of pile (tons)	
				Free Head	Fixed Head
600	20	70	45	0.7	1.7
	25	90	60		
800	20	115	70	1	2.7
	25	140	95		

9.4.4 The typical calculations of safe pile vertical load capacity and safe uplift pile capacity have been given in Appendix - A. Whereas the typical calculations of the safe lateral capacities of the pile has been given Appendix - B of this report.

9.4.5 The group action of piles depends upon the number of piles in the group and their arrangement. Therefore, the efficiency factor for each pile group can be considered and checked as per the relevant BIS codes.



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9.5 Depth of Isolated / Strip Foundations

9.5.1 The minimum depth of shallow foundations depends upon the following factors :

- a) Top filled-up strata / loose soil, if any
- b) Adequate depth of soil above founding level, to ensure mobilization of full safe bearing capacity
- c) Adequate depth of soil strata below founding level of requisite strength to mobilize the safe bearing capacity

9.5.2 Considering the above, the foundations for minor structures can be placed at a depth between ***1.5m and 2m below the existing ground level.***

9.5.3 The soil available at the founding level will be ***yellowish brown to grey clayey sandy silt / sandy clayey silty soils.***

9.6 Allowable Bearing Pressure

9.6.1 Allowable bearing pressure depends upon the allowable settlement. An allowable settlement of 50mm has been considered to evaluate the allowable bearing pressure for **Isolated/Strip Footings.**

9.6.2 Allowable bearing pressure has been evaluated by :

- a) Shear failure criterion using average soil data
- b) Settlement criterion taking SPT values

9.6.3 On the basis of the above analysis, the following recommendations regarding the net allowable bearing pressure are being made :



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Net allowable bearing pressure for various widths of Isolated/Strip Footings for an allowable settlement of 50mm

Borehole No.	Net Allowable Bearing Pressure (t/m²) for widths of footings		
	1.5	3	>4.5
<i>BH1, BH9 & BH11</i>	2	3	3
<i>All boreholes except BH1, BH9 & BH11</i>	2	4	5

9.7 Ground Improvement

9.7.1 The required depth of ground improvement is basically the depth to which the subsoil is considered to be susceptible to liquefaction. At the present site, the required depth of ground improvement is 15m below the existing ground level. As the subsoil down to deeper depths consists of sandy soils under submerged condition (*i.e. between 10m and 15m depths below the existing ground level*), Vibro Compaction method is relatively more reliable method of ground improvement under such ground conditions.

9.7.2 The Vibro compaction method of ground improvement involves use of a vibroflotation device which is inserted down to the required depth into the ground using air or water jetting and thereafter, slowly withdrawn with simultaneous and continuous vibrations of the vibroflotation device. This procedure is carried out at close spacings throughout the area of the site to be improved. The vibrations induced by the vibroflotation device densifies the sandy soil. This method is to be used with care in high ground water table conditions.

9.7.3 The design parameters for the Vibro compaction method of ground improvement are :



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- a) Required depth of ground improvement
- b) Spacing of points/nodes at which Vibro compaction is to be carried out.

9.7.4 The spacing of points/nodes at which vibro compaction are to be done by Vibroflotation is determined based on the required improvement in the relative density of the sandy soil assessed in terms of the N-values (i.e. SPT values). To ensure mitigation of the susceptibility to liquefaction of the sandy soil, the target N-value of the improved soil should be well above 15 and closer to 25 at depths greater than 10m below ground level. As the results of the investigations have shown that on an average, the N-values between 10m and 15m is on an average is between 10 and 15, the required increase in N-value is about 15 (i.e. from an N-value of 10 to an N-value of 25).

9.7.5 The spacing of the points/nodes are then determined based on the work performance record of the Vibroflotation equipment used and is typically in the range of 1.5m to 1.8m.

9.7.6 As is clearly evident from the above discussion, in view of the observed subsoil conditions and the fact that the design of the ground improvement by Vibro Compaction is primarily based on the work performance record, initial trials in a limited area with subsequent confirmation of the degree of improvement by follow-up soil investigations, are essential to confirm whether satisfactory level of ground improvement has been achieved.

10.0 RECOMMENDATIONS

10.1 Type of Foundations

- a) *Straight Bored Pile Foundations*
- b) *Isolated / Strip Footings for Minor Structures at ground level*



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10.2 Depth of Foundations

a) For Straight Bored Pile Foundations

Considering the cut-of level of the piles as 2m below the existing ground level, for effective lengths of pile of 20m and 25m, the piles shall rest at a depth of 22m, and 27m respectively, below the existing ground level.

b) For Isolated / Strip Footings for Minor Structures

Between 1.5m and 2m depth below the existing ground level

10.3 Safe Pile capacities

Pile Dia (mm)	Pile Effective Length (m)	Pile Vertical Load Capacity (tons)	Uplift Capacity of pile (tons)	Safe Lateral Capacity of pile (tons)	
				Free Head	Fixed Head
600	20	70	45	0.7	1.7
	25	90	60		
800	20	115	70	1	2.7
	25	140	95		

10.4 Allowable Bearing pressure (for Minor Structures), for an allowable settlement of 50mm

Borehole No.	Net Allowable Bearing Pressure (t/m ²) for widths of footings		
	1.5	3	>4.5
BH1, BH9 & BH1	2	3	3
All boreholes except BH1, BH9 & BH11	2	4	5



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10.5 Ground Improvement

Ground Improvement by Vibro Compaction method to be carried out for the densification of the subsoil strata between 10m and 15m depth below the existing ground level at a spacing of the points/nodes of 1.5m to 1.8m, to eliminate the susceptibility to liquefaction of the subsoil.

10.6 Appendices

10.6.1 An appendix sheet showing the typical analysis of pile capacity and pile uplift capacity calculations have been given in Appendix - A of this report.

10.6.2 An appendix sheet showing the typical analysis of pile lateral capacity calculations has been given in Appendix - B of this report.

10.6.3 An appendix sheet showing the typical analysis of the allowable bearing pressure for Isolated/ Strip footings has been given in Appendix - C.

10.6.4 An appendix sheet showing the typical procedure of liquefaction analysis is given in Appendix - D of this report

10.6.5 An annexure sheet showing the results of cross hole seismic test is given in Annexure - E of this report

10.6.6 A list of IS Codes referred for providing the recommendations and that which might be required to implement the same is also enclosed in this report in Appendix - F.

10.7 Special Notes

10.7.1 For straight bored pile foundations, after reaching the required depth in the pile bore, 15cm thick layer of gravel should be placed and compacted at the bottom so that the slush formed at the bottom is diminished.



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10.7.2 For shallow foundations, the founding surface must be compacted heavily. If any loose pockets are observed, the same shall be filled with brickbats/gravel and compacted. Foundations can subsequently be placed over such a prepared surface.

10.8 NOTE

The recommendations given in this report have been arrived at on the basis of design parameters which have been judiciously adopted by giving due consideration to the results of field and laboratory investigations as well as NAGADI's experience of over four decades in working in various types of soil and rock conditions all over India.

10.9 LIMITATIONS

This soil investigations have been carried out at locations in the site chosen by the clients so as to represent the entire site. The recommendations provided in this report are hence valid only for these test locations. However, if there is any change in subsoil conditions and properties at places between or beyond chosen test locations, fresh investigations will have to be carried out at such locations.

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CALCULATIONS FOR PILE CAPACITY

A.1 Methodology

A.1.1 A typical calculation for the pile capacity has been given hereunder for a pile of diameter 'D' = 600mm and for a length of pile 'L' = 20m.

A.1.2 The pile is designed primarily as an end bearing cum friction pile.

A.1.3 The end bearing capacity has been determined based on observed N-values while the friction capacity has been determined as per the relevant provisions of the BIS code IS:2911(part1/sec4).

A.1.4 For determining the end bearing capacity, the N- value at the level of the pile tip has been taken as 15.

A.1.5 For determining the friction capacity, three layers of strata have been consider and average cohesion and average angle of shearing resistance ϕ have been taken as follows:

Depth below EGL (m)	Length of the Pile in i^{th} Layer (m)	Adhesion Factor (α) in i^{th} Layer	Average Cohesion (c) of the Soil along the Pile Shaft (kg/cm ²)	Angle of Internal friction ϕ (°)
2 (COL)*	-	-	-	-
10	8	1	0.15	15
15	5	0.75	0.05	26
30	12	1	0.15	18

*COL= cutoff level

A.2 DETERMINATION OF PILE CAPACITY

A.2.1 Pile end bearing capacity based on N-values



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A.2.1.1 The ultimate end bearing capacity of the Bored pile foundation is determined as given below:

$$Q_{bu} = 10 \cdot N \cdot A_p$$

where N = N-value at the level of the pile tip

$$A_p = \frac{\pi \cdot D^2}{4} = \text{cross - sectional area of pile toe} = 2827 \text{ cm}^2$$

D = diameter of pile shaft = 600 mm

A.2.1.2 Therefore, the ultimate end bearing capacity of the bored pile foundation is :

$$Q_{bu} = 42.4 \text{ tons}$$

A.2.2 Pile friction capacity based on IS:2911 (Part 1/Sec 4)

A.2.2.1 The ultimate friction capacity of the pile is :

$$Q_{fu} = \pi \cdot D \cdot L \cdot \alpha \cdot c + \sum_1^n K \cdot P_{Di} \cdot \tan \delta \cdot A_{si}$$

where c_i = cohesion of the soil along the pile shaft = 0.15 & 0.5 kg/cm²

α_i = adhesion factor = 1 & 0.75

K = coefficient of earth pressure = 1

P_{Di} = effective overburden pressure at for the i^{th} layer = $\gamma \cdot d_i$

γ = submerged unit weight of soil below water table = 1 g/cm³

d_i = depth of the i^{th} layer from ground level

A_{si} = surface area of the pile stem in the i^{th} layer

δ = angle of wall friction between pile and soil

= ϕ_i = angle of shearing resistance of the soil = 15° & 26°

A.2.2.2 For calculation purposes, the submerged unit weight of the soil has been taken as the effective unit weight of the soil. The submerged unit weight considered is 1 g/cm³.



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A.2.2.3 As per the IS provisions, the effective overburden pressure P_{Di} is limited to a value equivalent to a depth of overburden equal to 15 times the diameter of the pile.

A.2.2.4 The skin friction capacity ' Q_{fu} ' is determined as :

$$Q_{fu} = \pi \cdot d \cdot L \cdot \alpha \cdot c + K \cdot \tan \delta \cdot \left[\frac{1}{2} \cdot (\gamma \cdot 15 \cdot D) \cdot (15 \cdot D) \cdot (\pi \cdot D) + (\gamma \cdot 15 \cdot D) \cdot (L - 15 \cdot D) \cdot (\pi \cdot D) \right]$$

A.2.1.5 The ultimate friction capacity of the pile in the i th layer :

Depth below EGL (m)	Length of the Pile in i^{th} Layer (m)	Ultimate Friction Capacity (Q_{fui}) in i^{th} Layer (tons)
2 (COL)*	-	-
10	8	38.78
15	5	44.45
30	12	100.1

*COL= cutoff level

$$\sum_{i=1}^n Q_{fui} = 183.3 \text{ tons}$$

A.2.2.6 The skin friction capacity ' Q_{fu} ' of the Bored pile foundation is :

$$Q_{fu} = 183 \text{ tons}$$

A.3 SAFE PILE CAPACITY

A.3.1 The safe pile capacity is determined by applying appropriate factors of safety to the ultimate values determined above. A factor of safety of 2.5 has been adopted for determination of the safe pile capacity.

A.3.2 Therefore, the safe pile capacity of Bored pile foundation is :

$$Q_s = \frac{Q_{bu} + Q_{fu}}{2.5}$$



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Hence,

$$Q_s = 90.2 \text{ tons say } 90 \text{ tons}$$

A.4 SAFE UPLIFT CAPACITY

The safe uplift capacity of the pile is determined by applying appropriate factors of safety to the ultimate friction capacity of the pile as determined above. A factor of safety of 3 has been adopted for determination of the safe uplift pile capacity.

Hence,

$$\text{Safe.Uplift.Capacity} = \frac{Q_{fu}}{3}$$

$$Q_{su} = 61 \text{ tons say } 60 \text{ tons}$$



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TYPICAL CALCULATION OF LATERAL PILE LOAD CAPACITY*(As per IS : 2911 (Part1/Sec2) - 2010)***B.1 Methodology**

B.1.1 A typical calculation for the lateral pile load capacity has been given hereunder for a pile of diameter 'D' = 600mm.

B.1.2 For determining the lateral pile load capacity, the depth of fixity of the piles or in other words, the equivalent cantilever length of the pile, has to be first determined. The depth of fixity is determined based on the modulus of horizontal subgrade reaction η_h value which in turn is considered based on the relative density/ soil consistency of the soil as observed in the boreholes conducted, as per table 3 of Annex - C of the BIS code IS:2911(part1/sec2).

B.1.3 Considering that the soil in the top one-half length of the pile is on an average in a loose to medium dense state, the modulus of horizontal subgrade reaction can be taken as $0.2 \times 10^3 \text{ kN/m}^2$ or 0.02 kg/cm^3 applicable for submerged conditions (as ground water table is at a shallow depth), as per table 1 of Annex - C of the BIS code IS:2911(part1/sec2).

B.2 Stiffness Factor & Depth of Fixity (L_e)

B.2.4 The stiffness factor T is calculated as per C - 2.3.1 of Annex - C of the BIS code IS:2911(part1/sec2) , as given below:

$$\text{Stiffness. factor} = T = 5 \sqrt{\frac{EI}{\eta_h}}$$



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Stiffness factor = $T = 4.87 \text{ m}$

B.2.2 Hence, the equivalent cantilever length or in other words the depth of fixity (L_e) of the pile works out to be based on stiffness factor (T) and the plot given in fig.4 in C - 3 of Annex - C of the BIS code IS:2911(part1/sec2)

$$L_e = 1.9 \times T = 1.9 \times 4.87 = 9.26\text{m} \text{ (for free head condition)}$$

$$L_e = 2.2 \times T = 2.2 \times 4.87 = 10.72\text{m} \text{ (for fixed head condition)}$$

B.2.3 The lateral pile load capacity is then determined as the horizontal force required for causing the allowable horizontal deflection of the pile head when considering the pile as a cantilever.

B.2.4 For calculation purposes, the concrete of pile has been considered to be of M30 grade.

B.2.5 For the determination of the lateral pile load capacity, the allowable horizontal deflection of the pile head has been considered as 5mm based on the provisions of clause 7.4 of BIS code IS:2911 (Part4) - 1985.

B.3 Determination of Lateral Pile Load Capacity

B.3.1 The horizontal deflection of the pile head, is determined by the following equations :

$$y = \frac{Q_L \cdot L_e^3}{3 \cdot E \cdot I} \text{ (for free head condition)}$$

$$y = \frac{Q_L \cdot L_e^3}{12 \cdot E \cdot I} \text{ (for fixed head condition)}$$

B.3.2 The flexural rigidity 'E·I' of the pile section has to be then determined by taking into account the flexural rigidity provided by the reinforcement. For this purpose, the moment of inertia 'I' is determined by taking into account the area of reinforcement along with the appropriate value of modular ratio 'm'.

B.3.3 The elastic modulus of concrete is determined as :



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$$E = 5000 \cdot \sqrt{f_{ck}}$$

Where f_{ck} = characteristic strength of concrete = 30N/mm²

Hence, $E = 27,386 \text{ N/mm}^2$ or $273,860 \text{ kg/cm}^2$

B.3.4 The moment of inertia of the pile section is determined as :

$$I = \frac{\pi \cdot d^4}{64}$$

Hence, $I = 6361725123 \text{ mm}^4 = 636173 \text{ cm}^4$

B.3.5 **Considering an allowable horizontal deflection of the pile head of 5mm, the lateral pile load capacity works out to be :**

$Q_L = 1 \text{ ton}$ (for free head condition)

$Q_L = 2.7 \text{ tons}$ (for fixed head condition)



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ANALYSIS FOR ALLOWABLE BEARING PRESSURE

(All the Boreholes Except BH1, BH9 & BH11)

Data

(i) Soil Properties :

$$c \text{ (kg/cm}^2\text{)} = 0.15 \quad \phi = 15^\circ \quad \gamma \text{ (g/cm}^3\text{)} = 1.75$$

(ii) Depth of Foundation, D (m) = 2

(iii) Allowable Settlement, s (mm) = 50

Shear Failure Criterion (Ref. IS : 6403)

$$N'_c = 9.68 \quad N'_q = 3.22 \quad N'_\gamma = 1.94 \quad R_w' = 0.5 \quad R_w = 0.5$$

$$q_s = \frac{1}{3} \cdot \left(c \cdot N'_c + 0.5 \cdot \gamma \cdot B \cdot N'_\gamma \cdot R_w + \gamma \cdot D \cdot N'_q \cdot R_w \right) - \gamma \cdot D \cdot R_w$$

$$B \text{ (m)} \quad 1.5 \quad 3 \quad >4.5$$

$$q_s \text{ (t/m}^2\text{)} \quad 5.4 \quad 5.8 \quad 6.3$$

Settlement Criterion (Ref. IS : 8009)

(i) From N Values

$$B \text{ (m)} \quad 1.5 \quad 3 \quad >4.5$$

$$H \text{ (m)} \quad 3 \quad 6 \quad 9$$

$$N_{av} \quad 5 \quad 7 \quad 8$$

$$q_a \text{ (t/m}^2\text{)} \quad 2.4 \quad 5 \quad 4.9$$

(ii) From Consolidation Tests $q_a = \frac{s}{\sum s_i} = \frac{s}{\sum m_{vi} \cdot \Delta p_i \cdot H_i} (\Delta p_i \text{ for } p = 1)$

$$\text{Layer} \quad 1 \quad 2 \quad 3$$

$$\text{Depth (m)}$$

$$m_v \text{ (t/m}^2\text{)}^{-1}$$

$$B \text{ (m)} \quad 1.5 \quad 3 \quad >4.5$$

$$H \text{ (m)} \quad 3 \quad 6 \quad 9$$

$$q_a \text{ (t/m}^2\text{)}$$

ADOPT	B (m)	1.5	3	>4.5
	q (t/m ²)	2	4	5



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TYPICAL PROCEDURE FOR LIQUEFACTION ANALYSIS**D.1 EVALUATION OF CYCLIC STRESS RATIO (CSR)**

$$CSR = (\tau_{av} / \sigma'_{vo}) = 0.65(a_{\max} / g) \cdot (\sigma_{vo} / \sigma'_{vo}) \cdot r_d \quad (i)$$

where, $r_d = (1.0 - 0.00765z)$, for, $(z \leq 9.15m)$

$$r_d = (1.174 - 0.0267z)$$
, for, $(9.15m < z \leq 23m)$

D.2 EVALUATION OF LIQUEFACTION RESISTANCE (CRR)

$$CRR_{7.5} = f(N_1)_{60} \quad (ii)$$

$$i.e. CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200}$$

D.2.1. Influence of Fine Content

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$$

where, $\alpha = 0$, for, $(FC \leq 5\%)$

$$\alpha = \exp[1.76 - (190 / FC^2)], \text{ for, } (5\% < FC < 35\%)$$

$$\alpha = 5, \text{ for, } (FC \geq 35\%)$$

$$\beta = 1, \text{ for, } (FC \leq 5\%)$$

$$\beta = [0.99 + (FC^{1.5} / 1,000)], \text{ for, } (5\% < FC < 35\%)$$

$$\beta = 1.2, \text{ for, } (FC \geq 35\%)$$



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D.2.1. Other Corrections

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$$

D.3 NOTATIONS

Notation	Description
CSR	Cyclic Stress Ratio
CRR	Cyclic Resistance Ratio
σ_{vo}	Total Vertical Overburden Stresses
σ'_{vo}	Effective vertical overburden stresses
τ_{av}	Average horizontal shear stress acting on soil layer during shaking generated by given earthquake
a_{max}	Peak horizontal acceleration at ground surface
g	Acceleration of gravity
r_d	Stress reduction coefficient to account for flexibility in soil profile
z	Depth below ground surface (m)
$CRR_{7.5}$	Cyclic resistance ratio for $M_w = 7.5$ earthquakes
$(N_1)_{60}$	Corrected standard penetration resistance
$(N_1)_{60cs}$	$(N_1)_{60}$ adjusted to equivalent clean-sand value
N_m	Measured standard penetration resistance
C_N	Correction factor for overburden pressure applied to SPT
C_E	Correction factor for hammer energy
C_B	Correction factor for borehole diameter
C_R	Correction factor for drilling rod length
C_S	Correction factor for split spoon sampler without liners
α, β	Coefficients, that are functions of fines content, used to correct $(N_1)_{60}$ to $(N_1)_{60cs}$



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(refer para no. 10.6.5)

Results of Cross Hole Seismic Test

Depth (m)	S-wave Velocity (m/s)	P-wave Velocity (m/s)	Dynamic Poisson's Ratio	Dynamic Shear Modulus (kg/cm ²)	Dynamic Elastic Modulus (kg/cm ²)
1.5	147	266	0.28	443	1133
3	147	261	0.27	439	1116
4.5	167	296	0.27	566	1438
6	191	346	0.28	743	1901
7.5	190	342	0.28	735	1882
9	172	308	0.27	606	1539
10.5	192	336	0.26	750	1890
12	151	264	0.26	462	1165
13.5	164	285	0.25	547	1367
15	265	473	0.27	1432	3637
16.5	306	538	0.26	1911	4815
18	248	452	0.28	1257	3219
19.5	250	463	0.29	1274	3287



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LIST OF IS CODES**Field Investigation**

1. IS : 1892 - 1979 : Code of practice for sub surface investigations for foundations
(First revision)
2. IS : 2131 - 1981 : Method of Standard Penetration Tests for soils (First revision)
3. IS : 2132 - 1986 : Code of practice for thin walled tube sampling of soils (Second revision)
4. IS: 4968 (P III) - 1976: Method for subsurface sounding for soils - Static
Cone Penetration test (First Revision; First Reprint
June 1983)

Laboratory Tests

1. IS : 2720 - 1983 (Part 1) : Methods of test for soils: Preparation of dry soil
samples for various tests (Second revision)
2. IS : 2720 - 1980 (Part 2) : Method of test for soils: Determination of water
content (Second revision) Amendment 1
3. IS : 2720 - 1980 (Part 3/sec 1) : Method of Test for Soils : Determination of
Specific Gravity : Fine Grained Soils. (First
Revision)
4. IS : 2720 - 1980 (Part 3/Sec 2): Method of test for soils : Determination of Specific
Gravity : Fine, Medium & Coarse grained soils. (First
revision).



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5. IS : 2720 - 1985 (Part 4) : Method of test for soils : Grain size analysis (Second revision)
6. IS : 2720 (Part 8) -1983: Determination of water content - dry density relation using heavy compaction. (First revision)
7. IS : 2720 - 1985 (Part 5) : Method of test for soils : Determination of liquid and plastic limit (Second revision)
8. IS 2720-1981 Part 12: Method of tests for soils : Determination of shear strength parameters using triaxial apparatus.
9. IS : 2720 -1987 (Part 26): Methods of tests for soils: Determination of pH value (Second revision).
10. IS :2720 - 1977 (Part 27): Methods of tests for soils: Determination of total soluble sulphate (First revision).
11. IS : 2720 (Part16) - 1979: Laboratory determination of CBR. (First revision)

Foundation Construction

1. IS : 2911 - 1979 (P 1/Sec 2): Code of practice for design and construction of pile foundations : Concrete piles - Bored cast insitu (First revision) Amendment 3
2. IS : 1904 - 1986 : Code of practice for design and construction of foundation in soils: General requirements (Third revision)
3. IS : 6403 - 1981 : Code of practice for determination of bearing capacity of shallow foundations : First revision (Amendment 1)



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4. IS : 8009 - 1976 (Part 1) : Code of practice for calculation of settlements of foundations: Shallow foundations subject to symmetrical static vertical loads (Amendment 2)



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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam												
		B.H. Location :		Water Table : 2.35m		Term. Depth : 20m		B.H. No.: 1						
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water	Type	c (kg/cm ²)	φ (°)	
18	9.9	Brownish grey silty sand	0	77	23	0	Non	Plastic	Sample	slip	CD	0.16	22	
	10.2	Change of the strata												
	10.5													
	23	11.4	Grey sand with traces of silt	0	93	7	0	Non	Plastic	Sample				slip
		12.0												
	24	12.9	Grey sand with traces of silt	0	96	4	0	Non	Plastic	Sample				slip
		13.5												
	16	14.4	Grey sand with traces of silt	0	97	3	0	Non	Plastic	Sample				slip
		14.8	Change of the strata											
		15.0												
	11	15.9	Brownish grey sandy clayey silt with gravel	8	20	46	26	38	24	1.98				27.6
		16.5												
9	17.4	Brownish grey sandy clayey silt	0	15	44	41	42	26	1.95	24.6				
	18.0													
8	20.0	Grey sandy clayey silt	0	5	54	41	43	26	1.95	27.9				
		Observed 'N' Values												



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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam											
		B.H. Location :		Water Table : 2.65m		Term. Depth : 30m		B.H. No.: 3					
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test		
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	c (kg/cm ²)	φ (°)
11	9.9	Grey sand with traces of silt	0	94	6	0	Non	Plastic					
	10.5								Sample	slip			
15	11.4	Grey sand with traces of silt	0	95	5	0	Non	Plastic					
	12.0								Sample	slip			
12	12.9	Grey sand with traces of silt	0	97	3	0	Non	Plastic					
	13.5								Sample	slip			
4	14.4	Grey sand with traces of silt	0	93	7	0	Non	Plastic					
	15.0								Sample	slip			
	15.5	Change of the strata											
3	15.9	Yellowish grey sandy clayey silt	0	12	46	42	34	21					
	16.5								1.92	20.0	CD	0.16	20
21	17.4	Yellowish grey sandy clayey silt	0	16	60	19	32	20					
	18.0								1.95	20.2			



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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam											
		B.H. Location :		Water Table : 2.2m		Term. Depth : 20m		B.H. No.: 4					
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test		
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm³)	Water Cont (%)	Type	c (kg/cm²)	φ (°)
13	9.9	Change of the strata							Sample	slip			
	10.5	Grey sand with traces of silt	0	95	5	0	Non	Plastic					
	11.4								Sample	slip			
22	12.0	Grey sand with traces of silt	0	98	2	0	Non	Plastic					
	12.9								Sample	slip			
22	13.5	Grey sand with traces of silt	0	98	2	0	Non	Plastic					
	14.4	Change of the strata							1.96	24.3			
8	15.0	Grey sandy clayey silt	0	14	54	32	31	21	2.02	25.2			
	15.9												
10	16.5	Grey sandy clayey silt	0	18	63	19	30	20	2.00	17.7	CD	0.12	
	17.4											21	
15	18.0	Yellowish grey sandy clayey silt	0	9	72	19	30	21					
39	20.0								1.96	17.8			
		Observed 'N' Values											



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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam											
		B.H. Location :		Water Table : 2.3m		Term. Depth : 20m		B.H. No.: 5					
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test		
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm³)	Water Cont (%)	Type	c (kg/cm²)	φ (°)
7	0.0	Yellowish brown clayey sandy silt	0	37	56	7	25	17	1.94	22.8	CD	0.10	22
	0.9												
	1.5												
8	2.4	Grey clayey sandy silt	0	40	54	6	26	17	1.97	17.5			
	3.0												
4	3.9	Grey sandy clayey silt	0	5	59	36	41	26	1.95	26.7			
	4.5												
8	5.4	Grey sandy clayey silt	0	20	56	24	33	22	2.00	20.6			
	6.0												
7	6.9	Yellowish grey sandy clayey silt	0	6	64	30	34	22	1.95	24.9			
	7.5												
14	8.4	Grey clayey sandy silt	0	35	53	12	27	19	1.98	21.5			
	9.0	Change of the strata											



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 NAGADI		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam										
		B.H. Location :				Water Table : 1.7m		Term. Depth : 20m		B.H. No.: 6		
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test	
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	ϕ (°) c (kg/cm ²)
6	0.0	Yellowish brown clayey sandy silt	0	15	75	10	27	18	1.84	19.9	CD	0.15
	0.9											
13	1.5	Yellowish brown clayey sandy silt	0	29	63	8	25	17	1.99	20.9		
	2.4								1.74	18.8		
4	3.0	Grey sandy clayey silt	0	4	68	28	35	22	1.96	16.4		
	3.9								1.95	24.7		
10	4.5	Grey sandy clayey silt	0	15	65	20	30	20	1.97	28.3		
	5.4											
10	6.0	Yellowish grey sandy clayey silt	0	5	55	40	42	26				
	6.9											
11	7.5	Yellowish grey sandy clayey silt	0	4	61	35	39	25				
	8.4											
	9.0	Yellowish grey sandy clayey silt	0	4	61	35	39	25				

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Sheet No. : 13

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 NAGADI		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam										
		B.H. Location :				Water Table : 2.4m		Term. Depth : 20m		B.H. No.: 8		
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test	
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	ϕ (°) c (kg/cm ²)
6	0.0	Grey clayey sandy silt	0	21	71	8	25	18	1.70	10.9	CD	0.11
	0.9								1.86	16.5		
11	1.5	Grey clayey sandy silt	0	25	65	10	27	18	1.90	17.8		
	2.4								2.02	22.5		
4	3.0	Grey clayey sandy silt	0	25	65	10	27	18	1.90	17.8		
	3.9								2.02	22.5		
5	4.5	Grey sandy silty clay	0	4	64	32	46	27	1.84	18.6		
	5.4								1.95	23.4		
8	6.0	Grey sandy clayey silt	0	19	57	24	34	22	1.84	18.6		
	6.9								1.95	23.4		
14	7.5	Grey sandy clayey silt	0	20	59	21	30	20	1.84	18.6		
	8.4								1.95	23.4		
	9.0	Yellowish grey sandy clayey silt	0	9	64	27	37	24				

SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam											
		B.H. Location :		Water Table : 2.4m		Term. Depth : 20m		B.H. No.: 8					
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test		
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	c (kg/cm ²)	φ (°)
14 <													

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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam											
		B.H. Location :		Water Table : 2.5m		Term. Depth : 20m		B.H. No.: 10					
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test		
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	c (kg/cm ²)	φ (°)
9	0.0	Yellowish brown clayey sandy silt	0	27	62	11	24	17	1.99	11.9	CD	0.14	23
	0.9								1.92	25.6			
3	1.5	Grey sandy clayey silt	0	9	66	25	32	22	1.92	25.6			
	2.4								2.00	27.2			
7	3.0	Grey sandy clayey silt	0	3	51	46	44	28	2.00	27.2			
	3.9								1.96	27.6			
8	4.5	Grey sandy clayey silt	0	5	79	16	30	20	1.96	27.6			
	5.4								1.99	23.0			
13	6.0	Grey sandy clayey silt	0	9	63	28	35	23	1.99	23.0			
	6.9								2.01	21.2			
17	7.5	Yellowish grey clayey sandy silt	0	37	53	10	25	18	2.01	21.2			
	8.4												
	9.0												



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SOIL PROFILE		Project : Proposed Factory of Pepsico India Holdings Pvt Ltd at Village Nalbari, Assam																				
		B.H. Location :		Water Table : 1.5m		Term. Depth : 20m		B.H. No.: 11														
N - Value	Depth (m)	Soil Description	Grain Size Analysis				Atterberg		In-situ		Triaxial Test											
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid (%)	Plastic (%)	Density (g/cm ³)	Water Cont (%)	Type	c (kg/cm ²)	φ (°)									
7 4 4 7 8 14	0.0	Yellowish brown clayey sandy silt	0	29	55	16	29	20	1.83	20.4	CD	0.11	23									
	0.9																					
	1.5																					
	2.4	Yellowish brown sandy clayey silt	0	7	76	17	30	21	1.90	23.9												
	3.0																					
	3.9																					
	4.5	Yellowish brown sandy clayey silt	0	9	56	35	37	24	1.96	25.6												
	5.4													Grey sandy clayey silt	0	5	63	32	38	26	1.99	26.1
	6.0																					
	6.9	Yellowish grey sandy clayey silt with gravel	2	10	61	27	33	22	1.97	26.6												
	7.5																					
	8.4													Grey clayey sandy silt	0	36	55	9	24	19	1.97	27.4
	9.0																					
	9.7	Change of the strata																				



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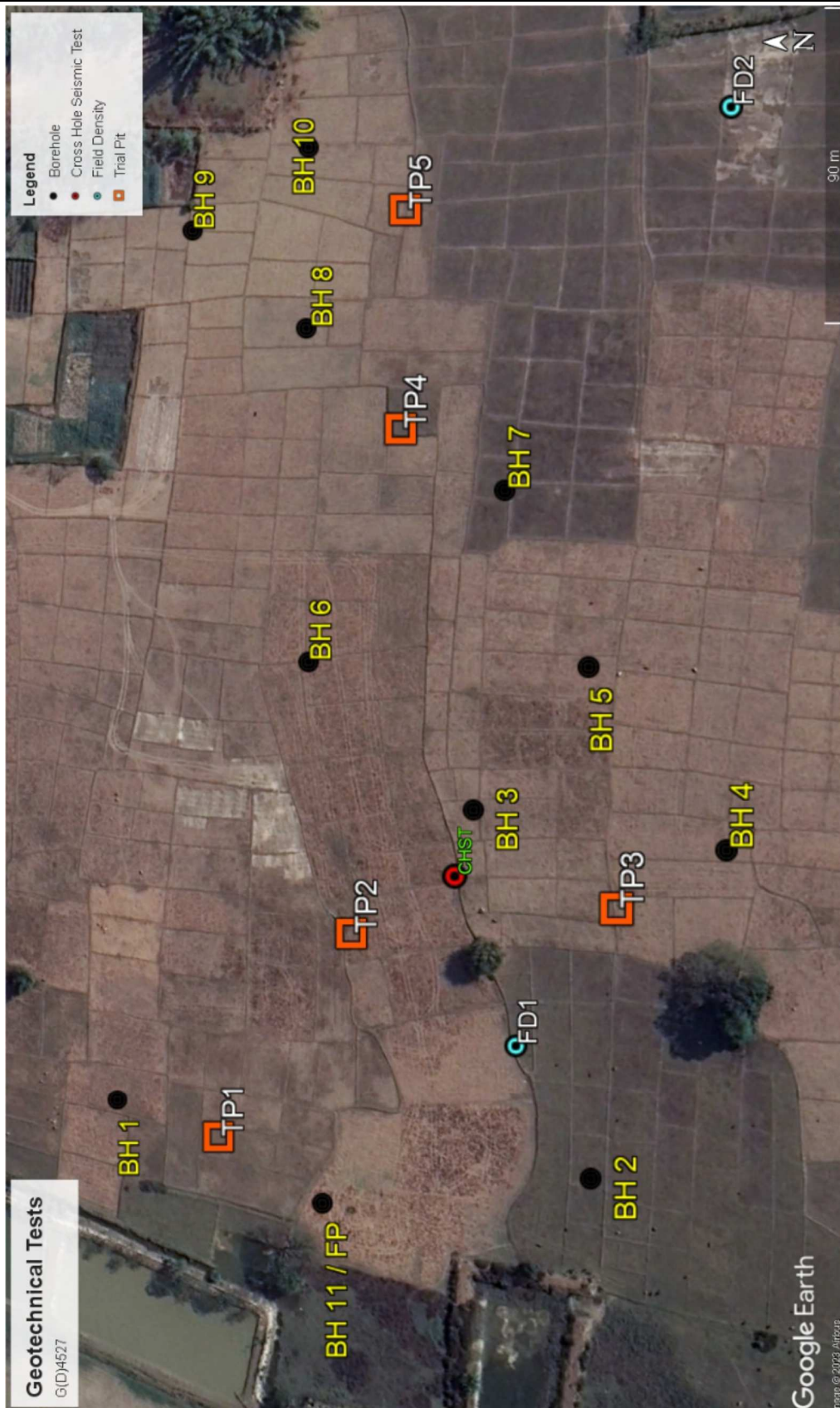
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NOTE : Bulk soil sample collected for CBR no. 1, 2, 3, 7 and 10 adjacent to respective boreholes
Bulk soil sample collected for Standard proctor test no. 4, 5, 6, 8, 9 and 10 adjacent to respective boreholes

SCHEMATIC SITE PLAN



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**PHOTOGRAPHS SHOWING
VIEW OF THE SITE & BOREHOLES UNDER PROGRESS**

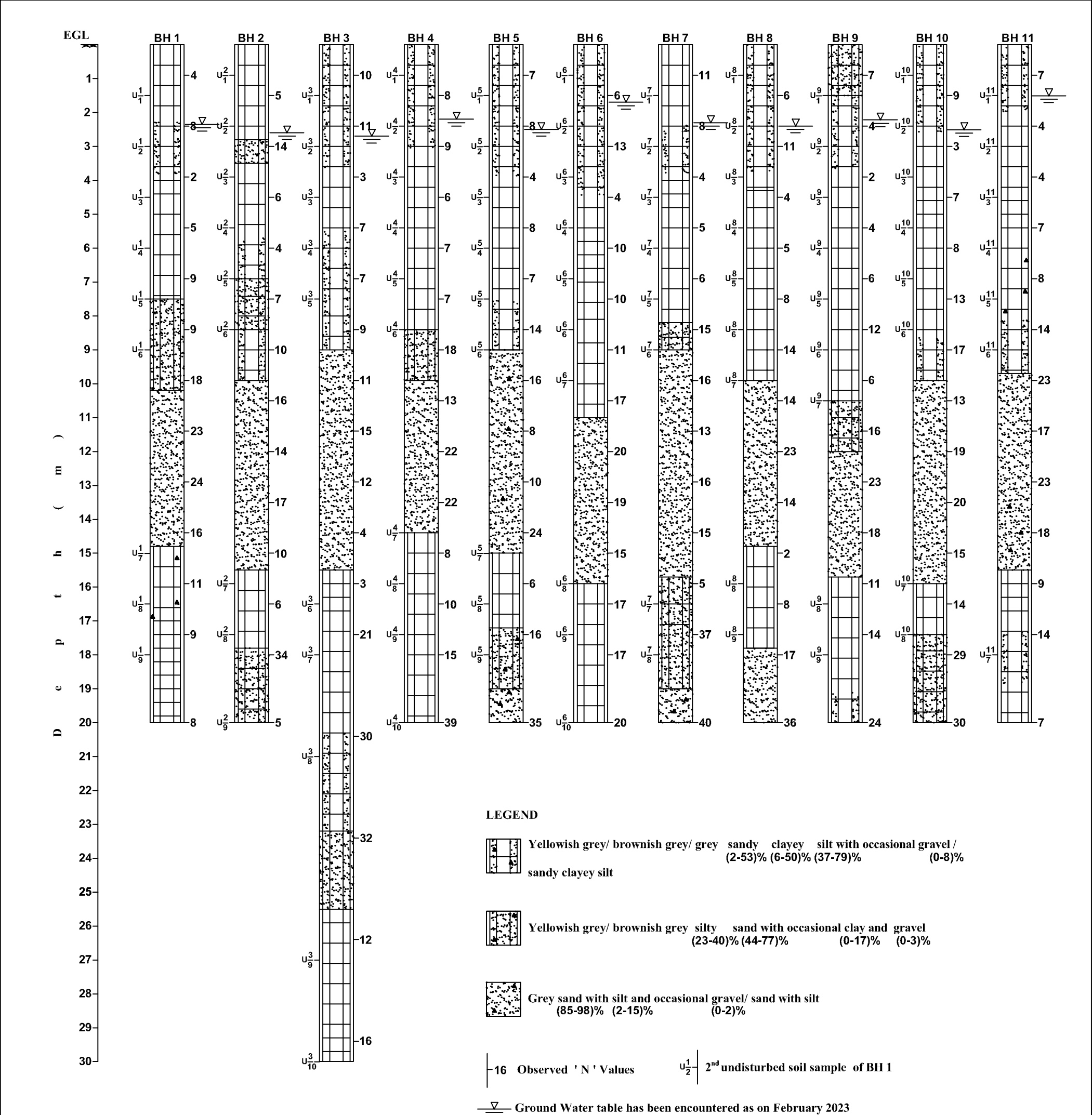


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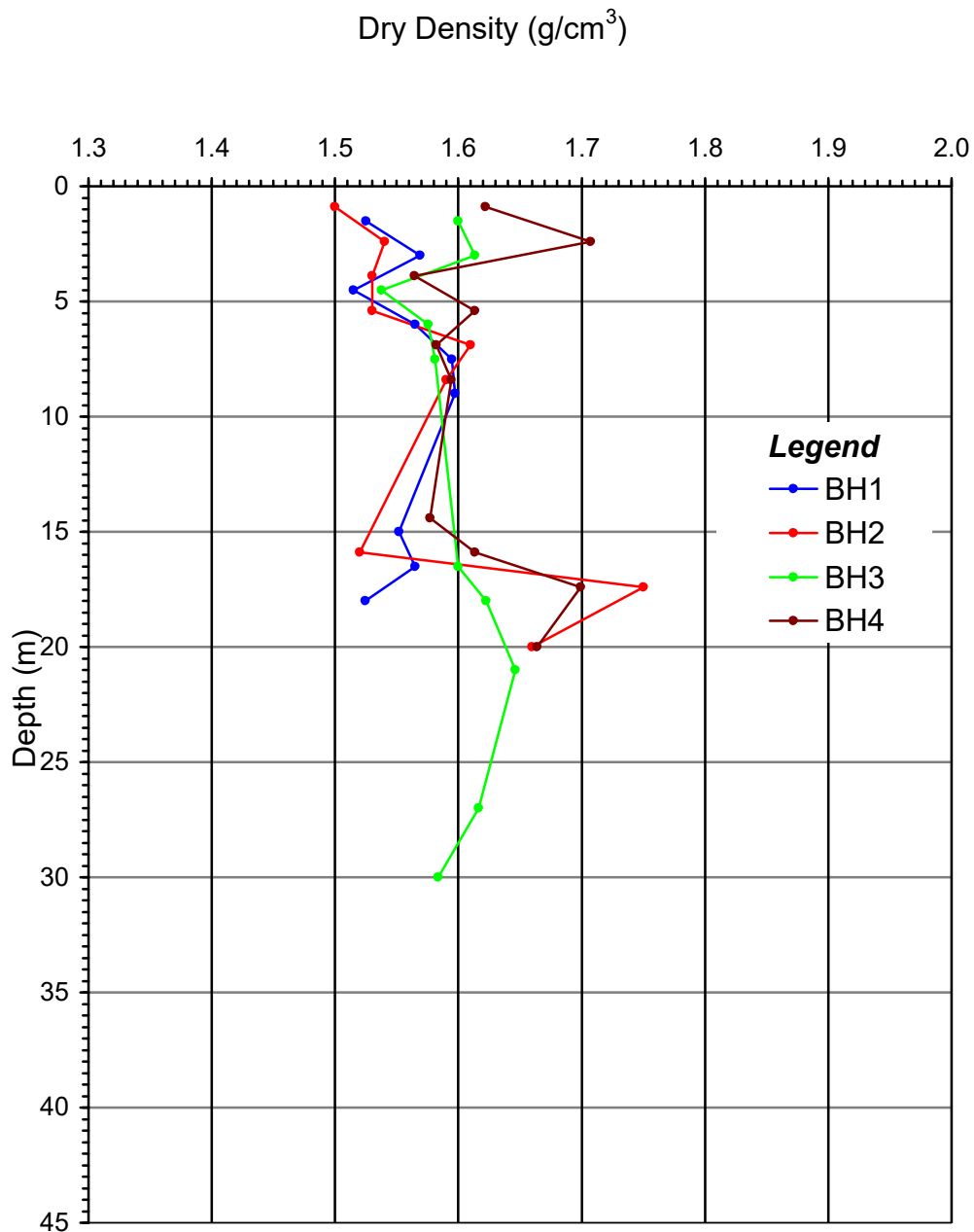
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COMPILED SOIL PROFILE



Dry Density vs Depth Curves
(Refer paragraph no. 7.4.2)

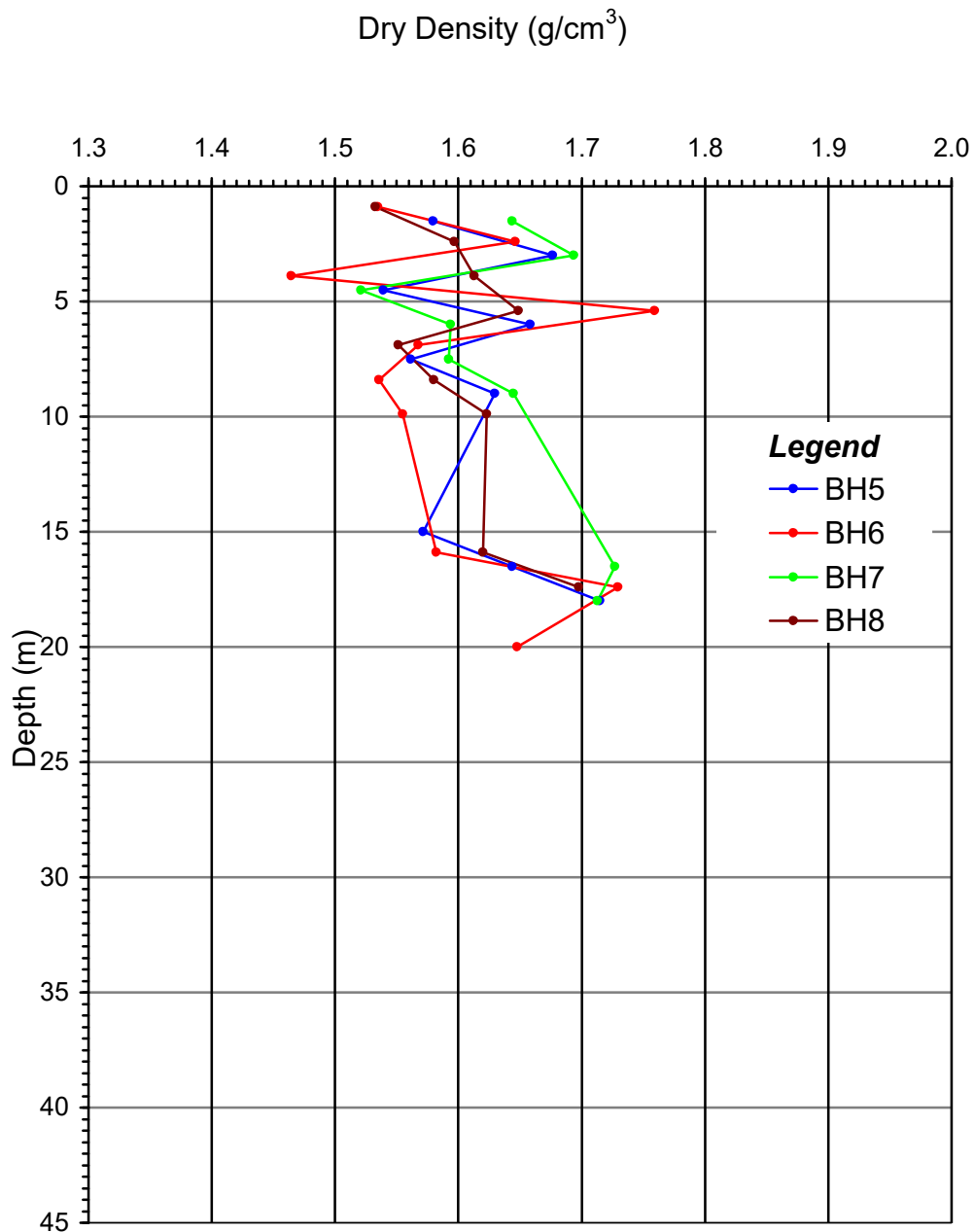


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Sheet No. : 3a



Dry Density vs Depth Curves
(Refer paragraph no. 7.4.2)

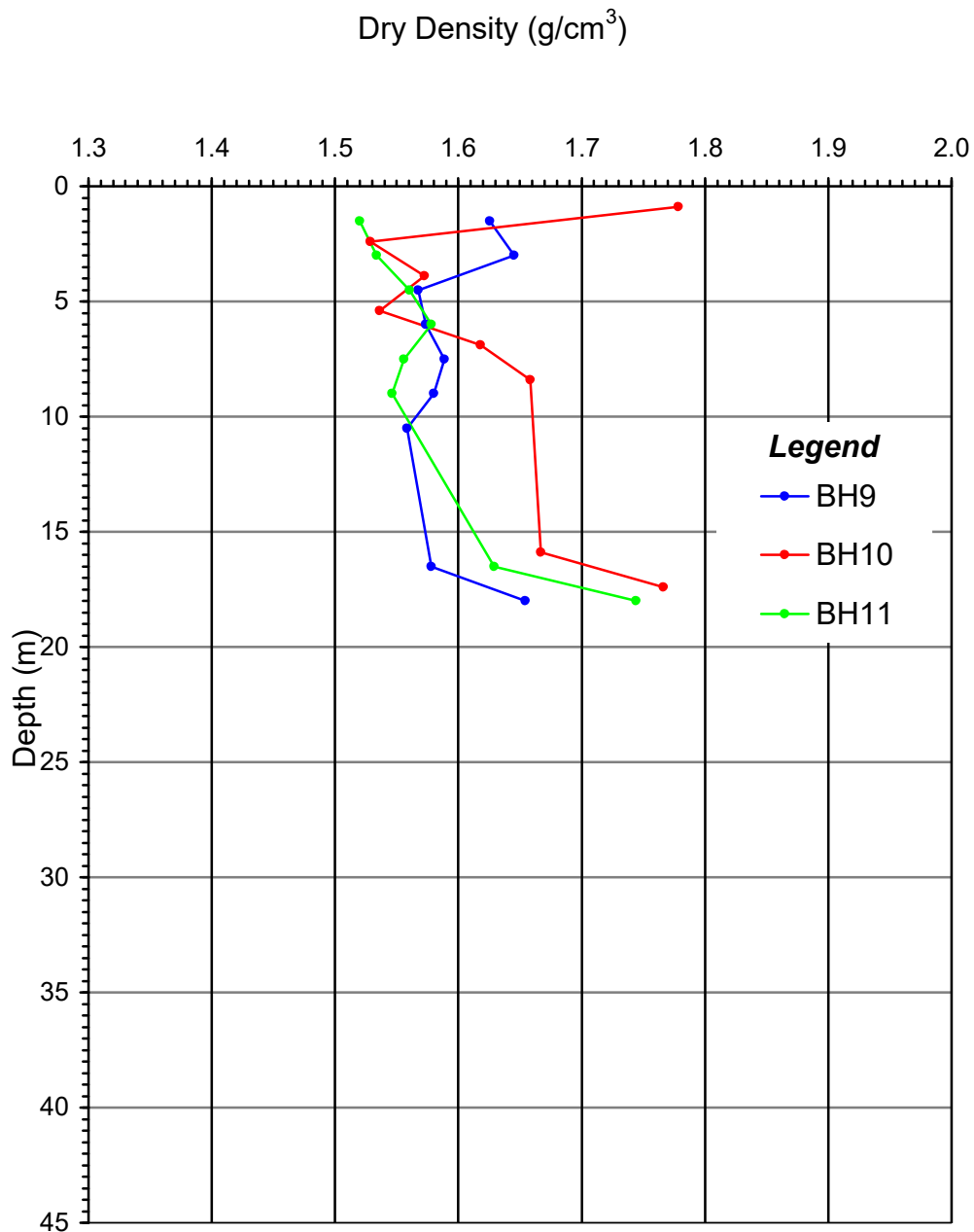


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Sheet No. : 3b



Dry Density vs Depth Curves

(Refer paragraph no. 7.4.2)



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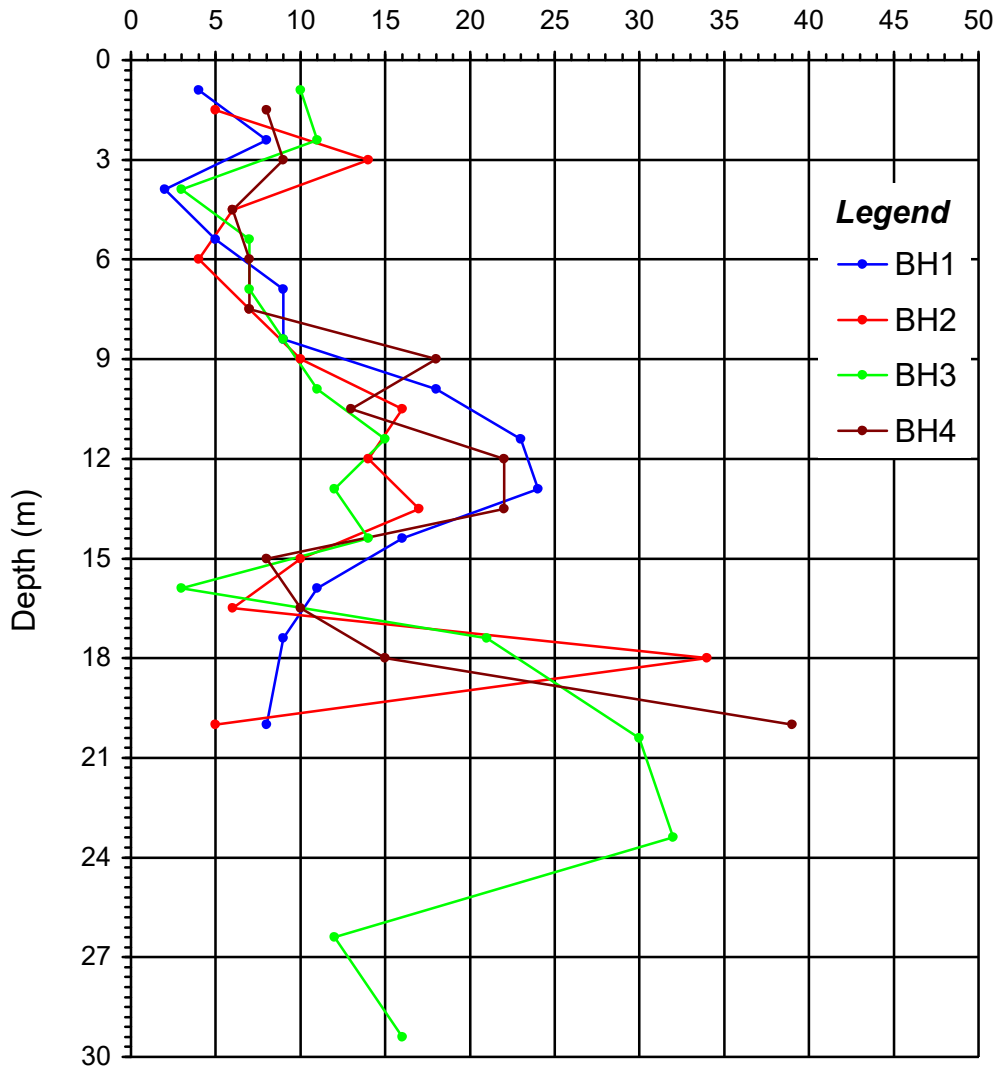
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N - Values (Observed)



N - Values vs Depth Curves

(Refer paragraph no. 7.6.1)



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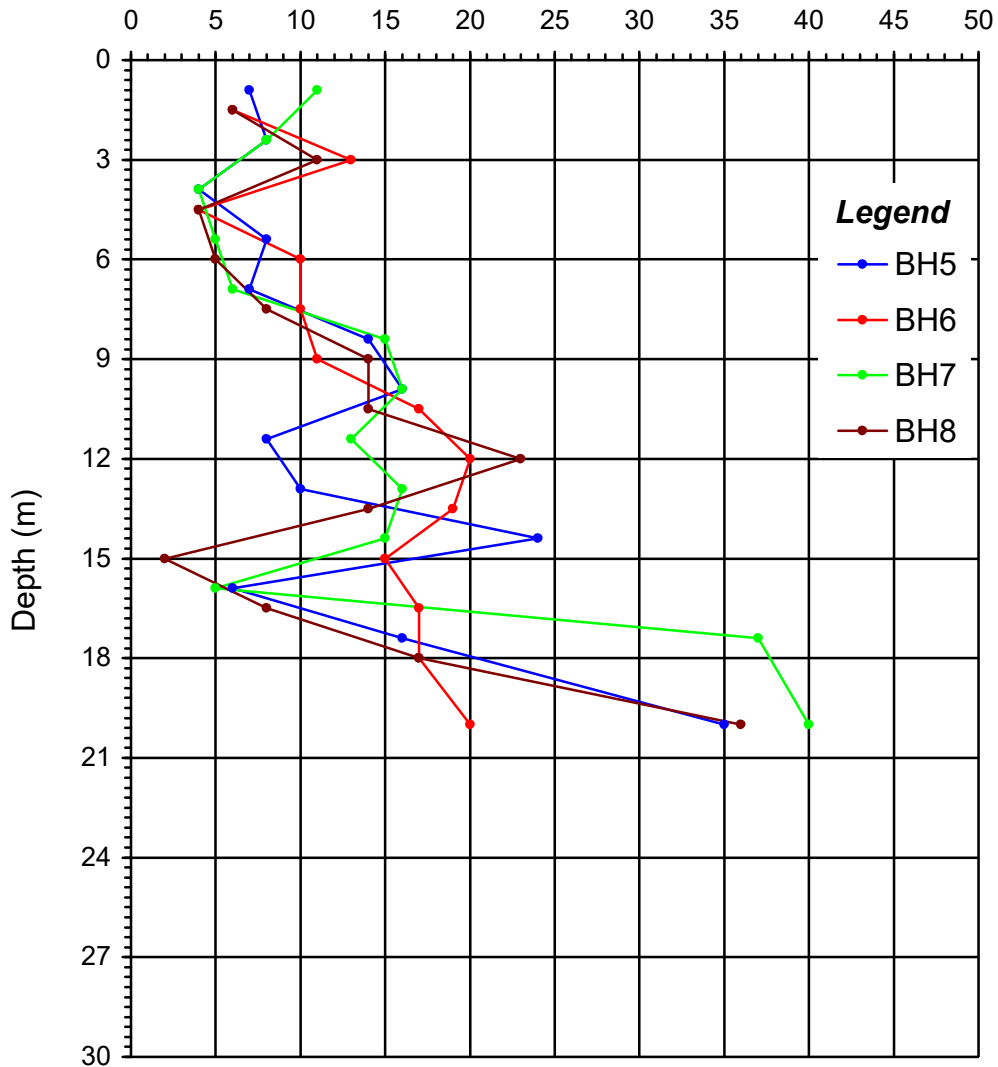
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Sheet No. : 4a

N - Values (Observed)



N - Values vs Depth Curves

(Refer paragraph no. 7.6.1)



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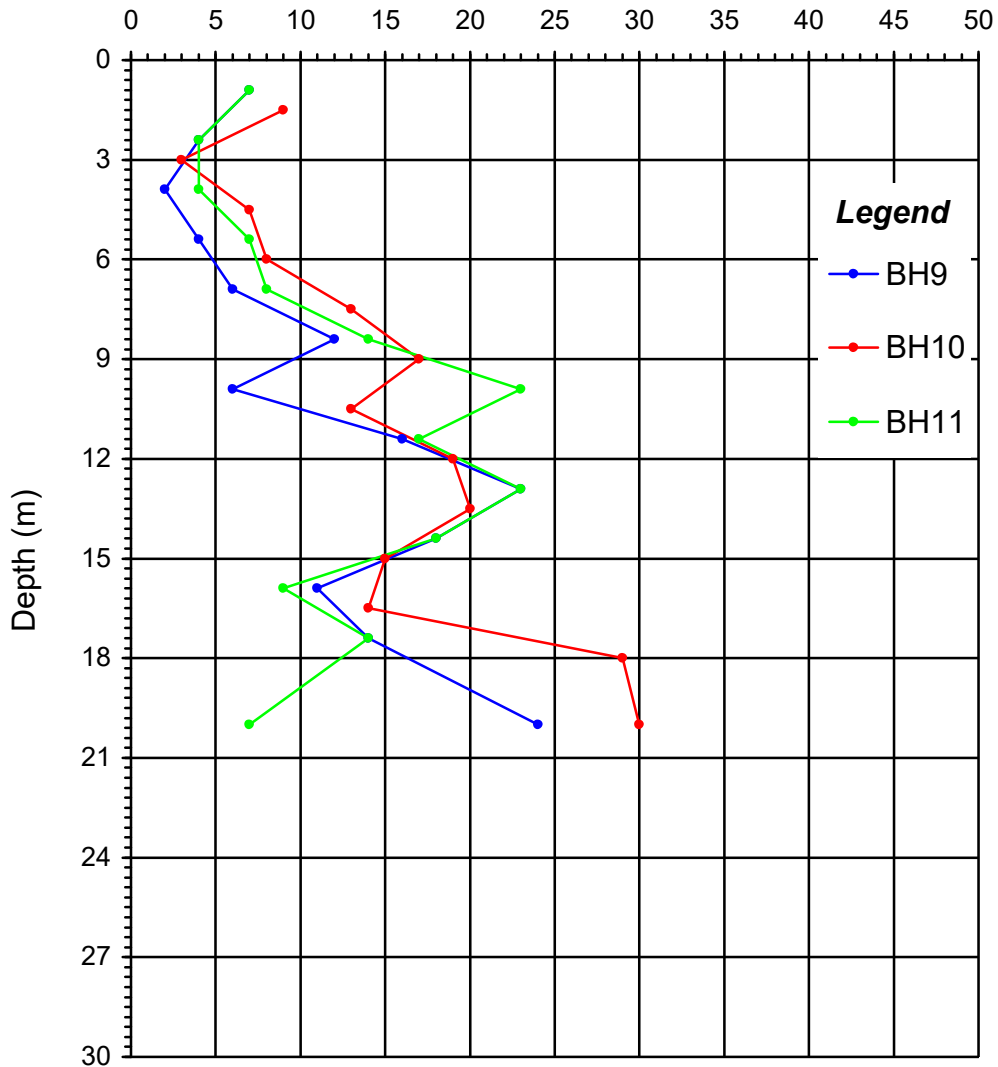
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Sheet No. : 4b

N - Values (Observed)



N - Values vs Depth Curves

(Refer paragraph no. 7.6.1)



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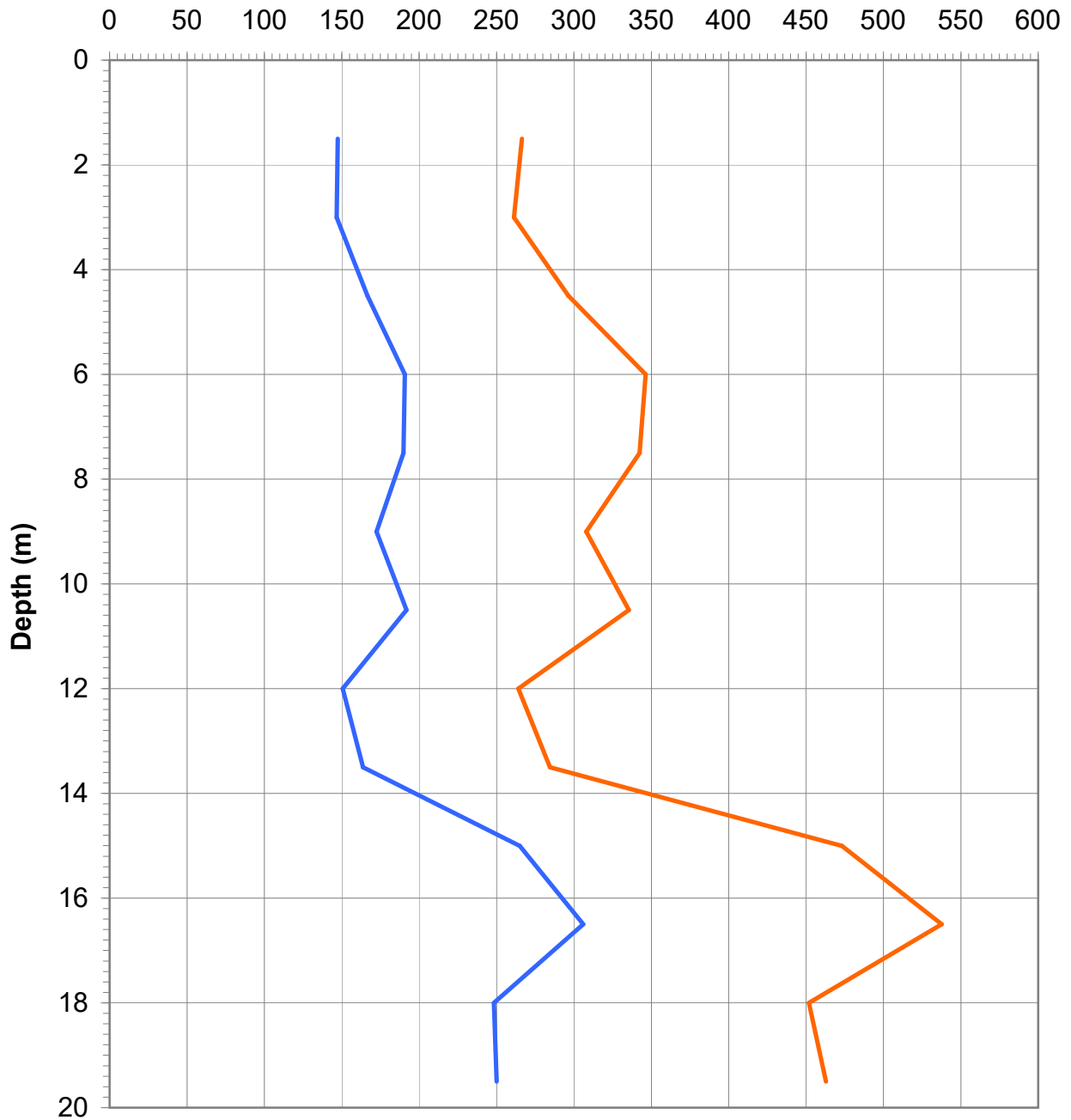
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— P-Wave or Compression Wave Velocity

— S-Wave or Shear Wave Velocity

Wave Velocity (m/s)



Cross Hole Seismic Test - 1



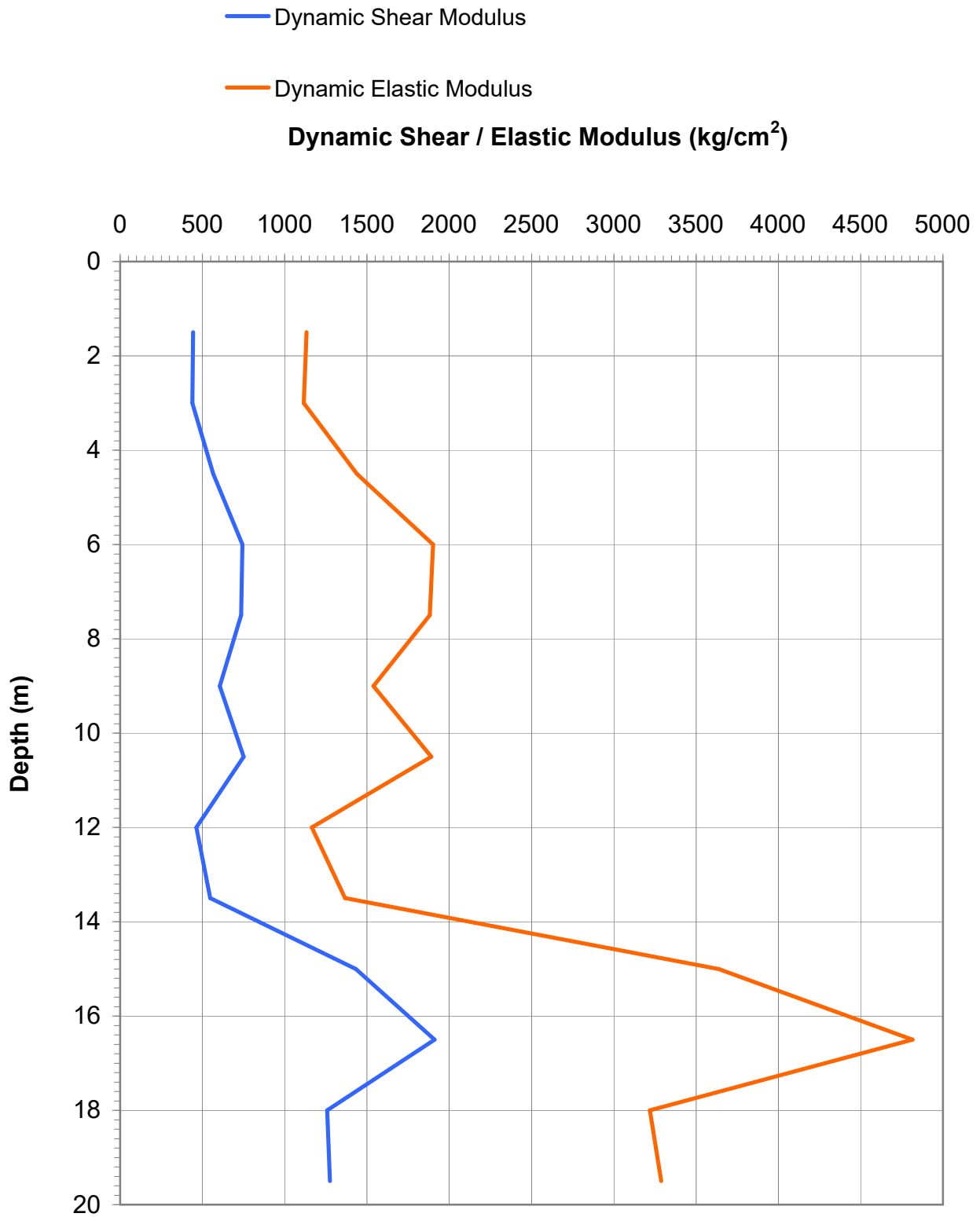
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Cross Hole Seismic Test - 1



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Sheet No. : 5b



Size of Trial Pit : 1.5m × 1.5m
Depth of Trial Pit : Excavation down to 1.5m depth below EGL
Subsoil Strata : Yellowish brown clayey sandy silt / sandy clayey silt

PHOTOGRAPH SHOWING VIEW OF TRIAL PIT (TP1)



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Sheet No. : 6a



Size of Trial Pit : 1.5m × 1.5m
Depth of Trial Pit : Excavation down to 1.5m depth below EGL
Subsoil Strata : Yellowish brown clayey sandy silt / sandy clayey silt

PHOTOGRAPH SHOWING VIEW OF TRIAL PIT (TP2)



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Job No. : G(D)4527

Sheet No. : 6b



Size of Trial Pit : 1.5m × 1.5m
Depth of Trial Pit : Excavation down to 1.5m depth below EGL
Subsoil Strata : Yellowish brown clayey sandy silt / sandy clayey silt

PHOTOGRAPHS SHOWING VIEW OF TRIAL PITS (TP3 & TP4)



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Sheet No. : 6c



Size of Trial Pit : 1.5m × 1.5m
Depth of Trial Pit : Excavation down to 1.5m depth below EGL
Subsoil Strata : Yellowish brown clayey sandy silt / sandy clayey silt

PHOTOGRAPHS SHOWING VIEW OF TRIAL PITS (TP5)

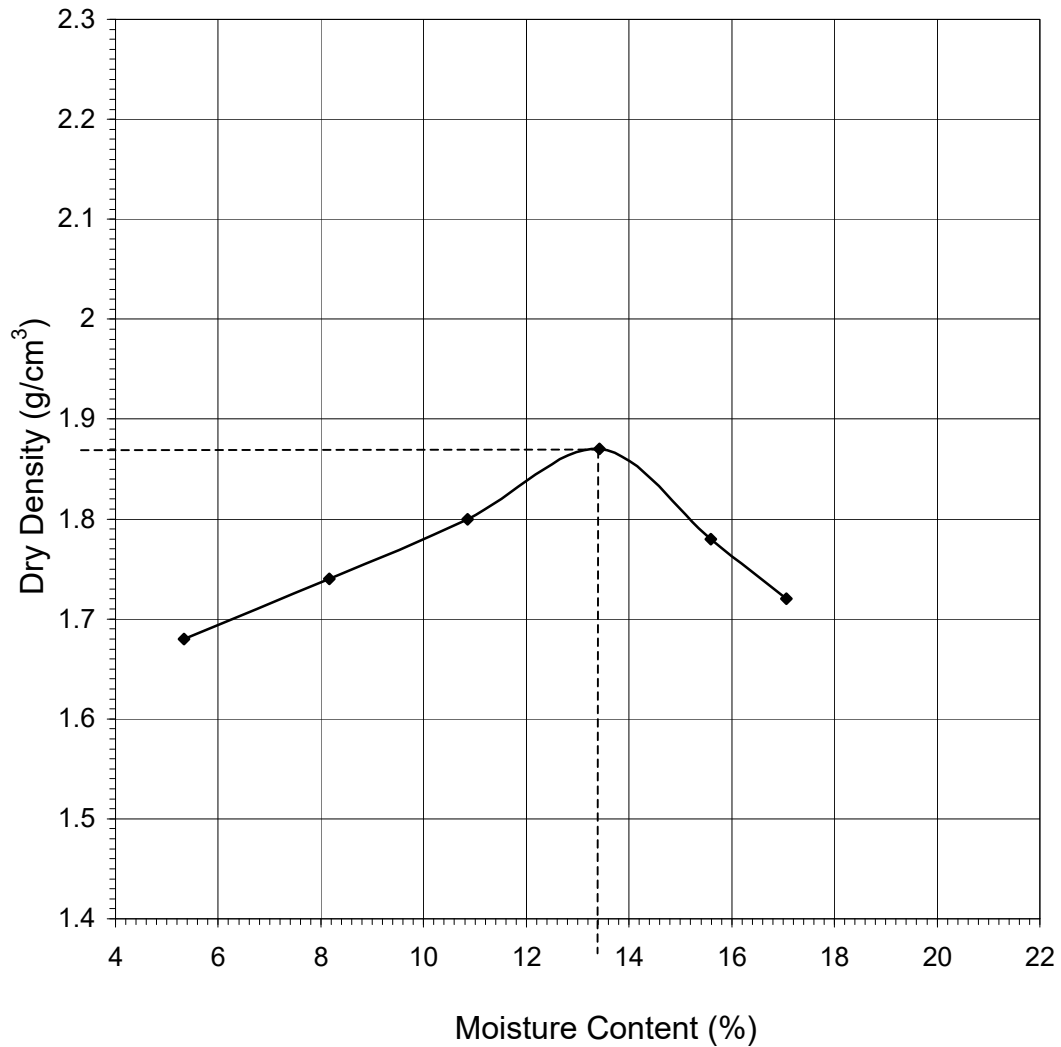


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Maximum Dry Density = 1.77 g/cm³
Optimum Moisture Content = 12.4 %

Modified Proctor Density Test - 1

(Refer paragraph no. 7.11.3)

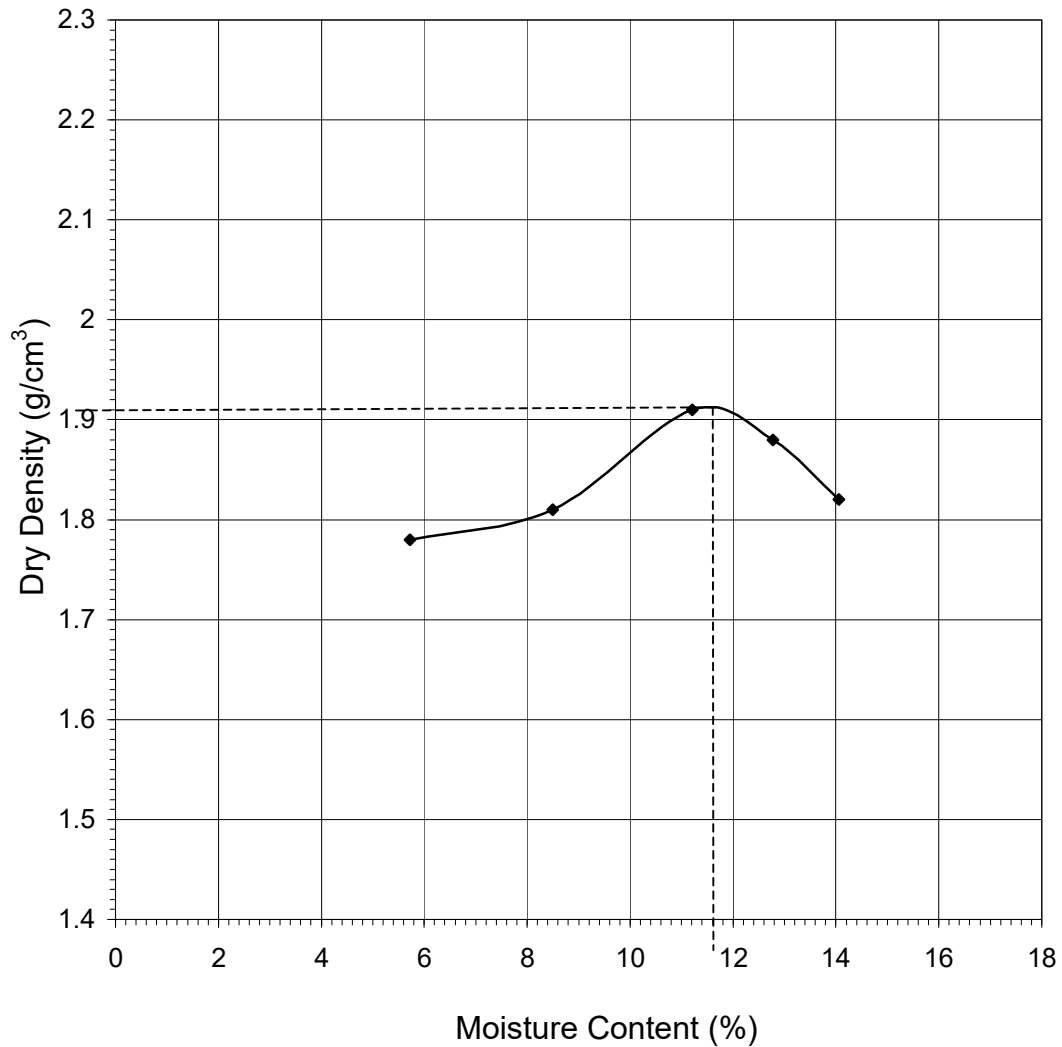


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Job No. : G(D)4527

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Maximum Dry Density = 1.65 g/cm³
Optimum Moisture Content = 15.8 %

Modified Proctor Density Test - 2

(Refer paragraph no. 7.11.3)

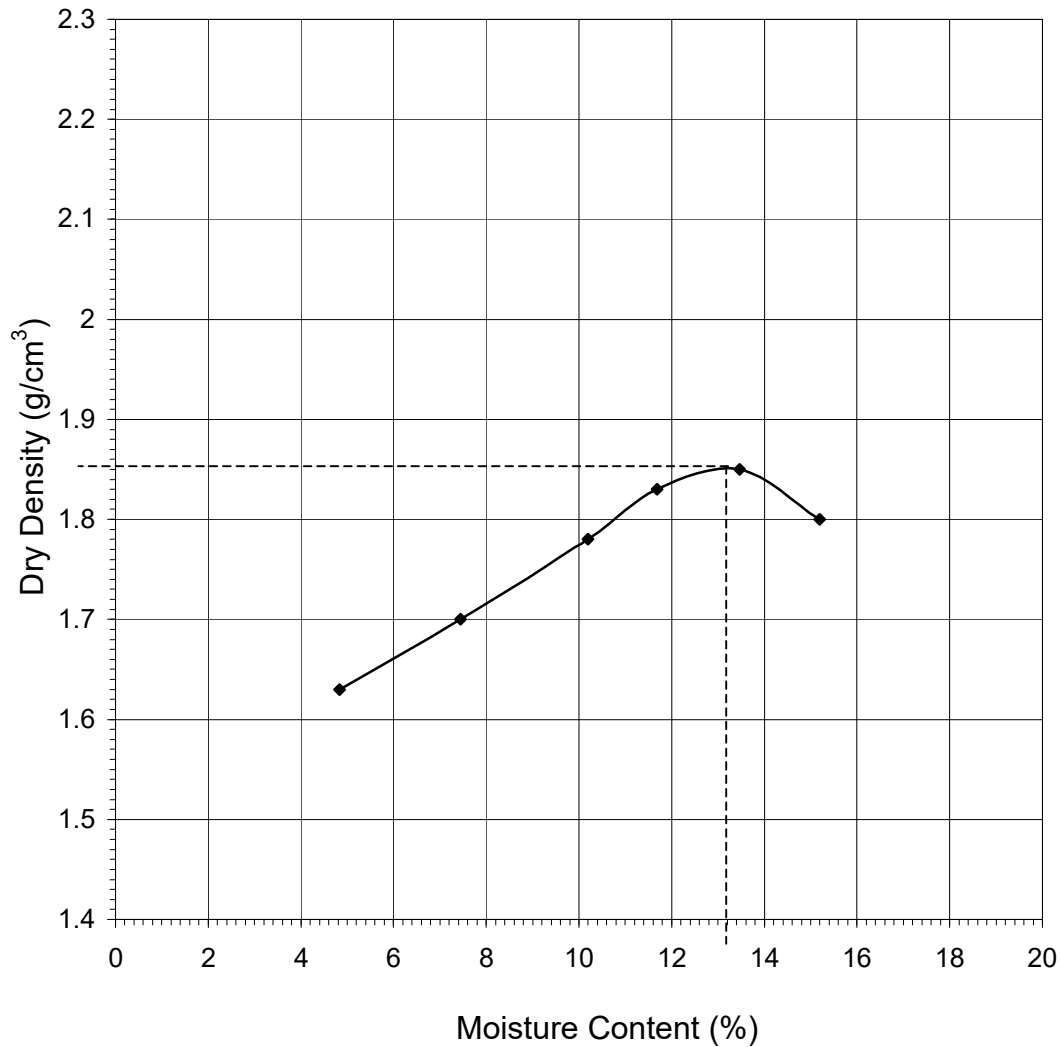


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Job No. : G(D)4527

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Maximum Dry Density = 1.75 g/cm³
 Optimum Moisture Content = 14.4 %

Modified Proctor Density Test - 3

(Refer paragraph no. 7.11.3)

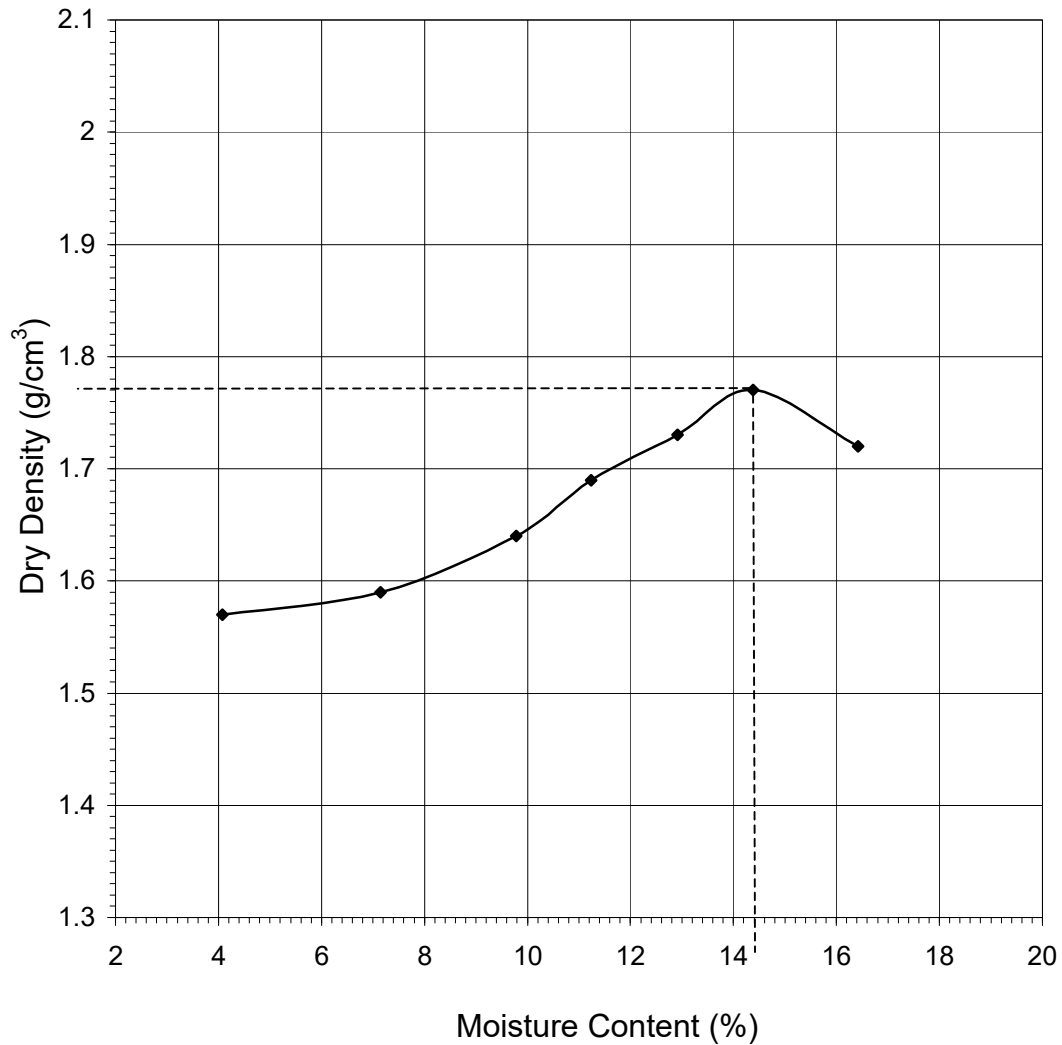


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Maximum Dry Density = 1.81 g/cm³
Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 4

(Refer paragraph no. 7.11.3)

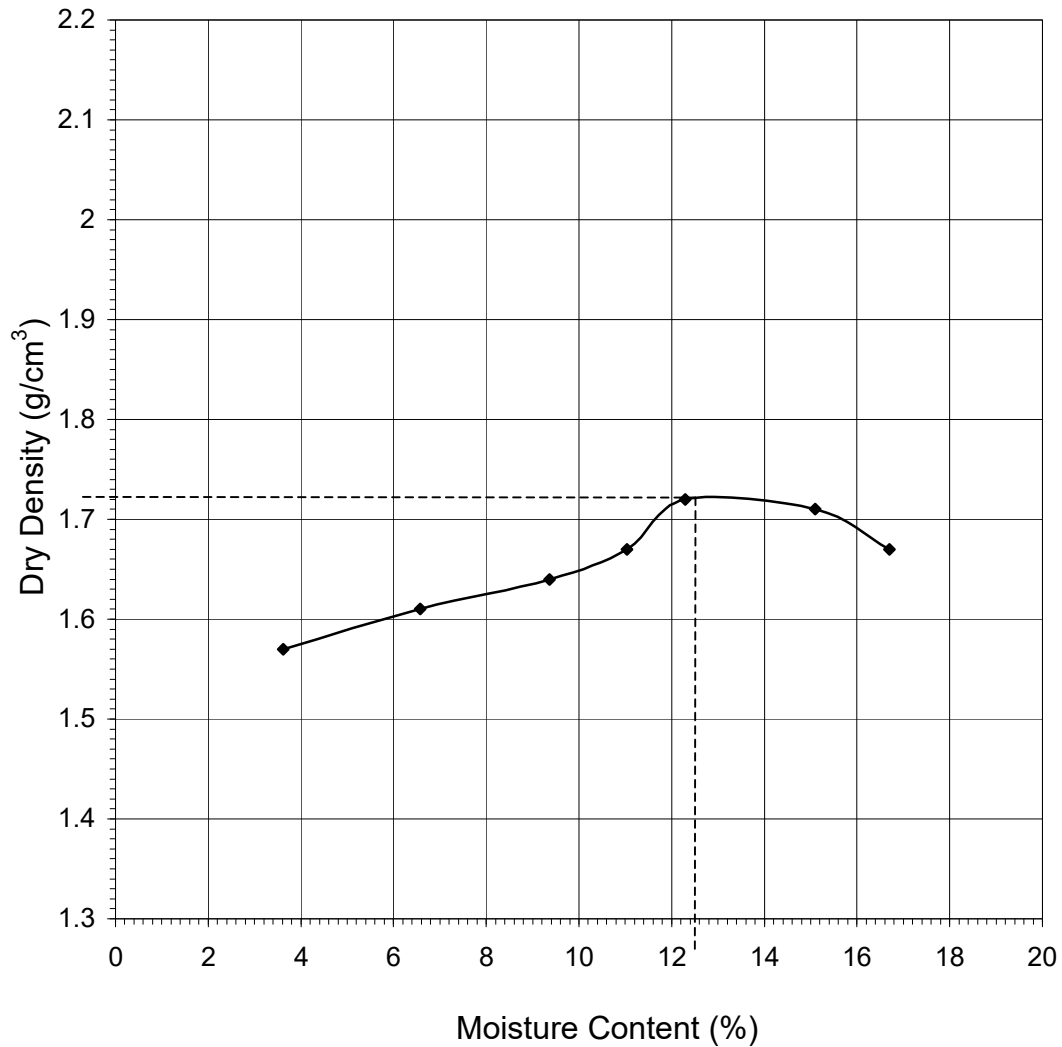


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Maximum Dry Density = 1.81 g/cm³
Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 5

(Refer paragraph no. 7.11.3)

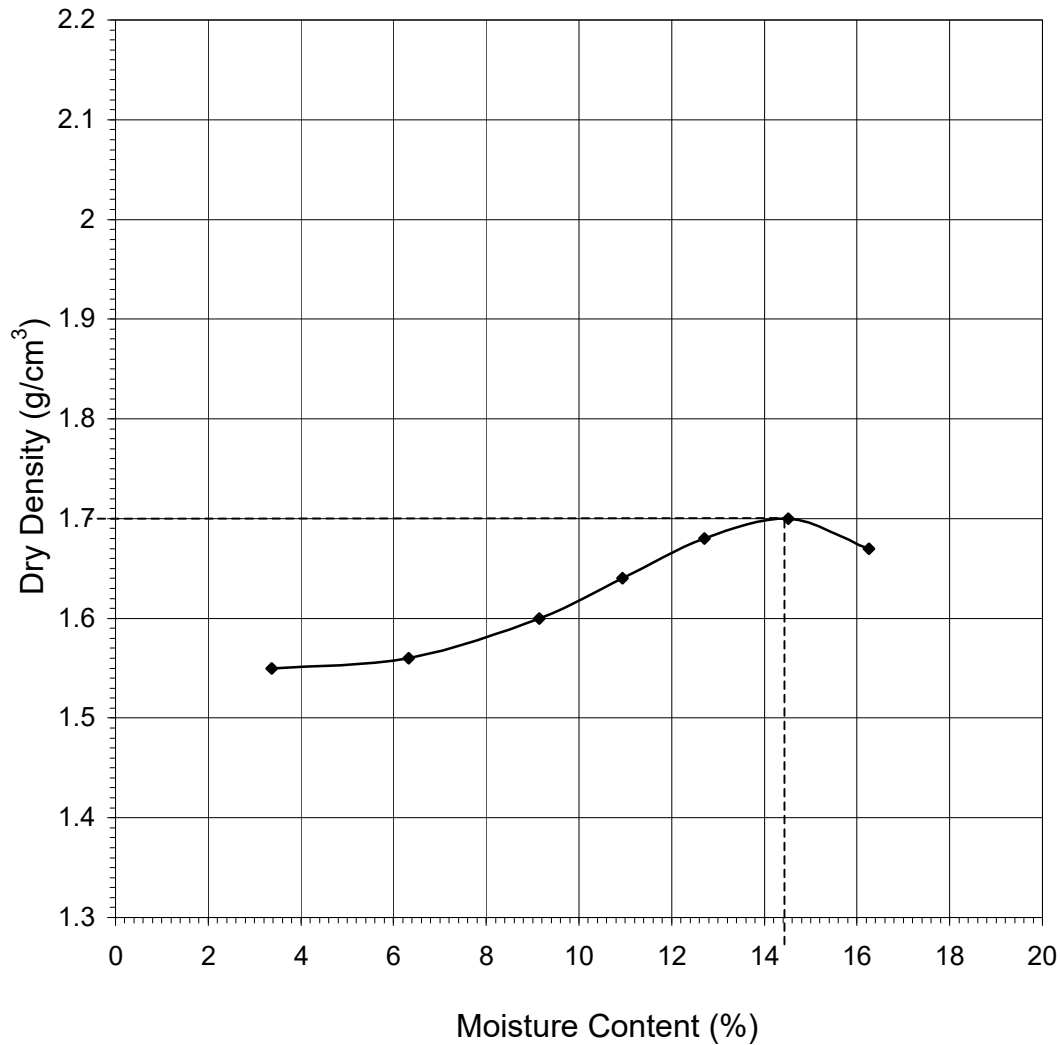


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Maximum Dry Density = 1.81 g/cm³
 Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 6

(Refer paragraph no. 7.11.3)

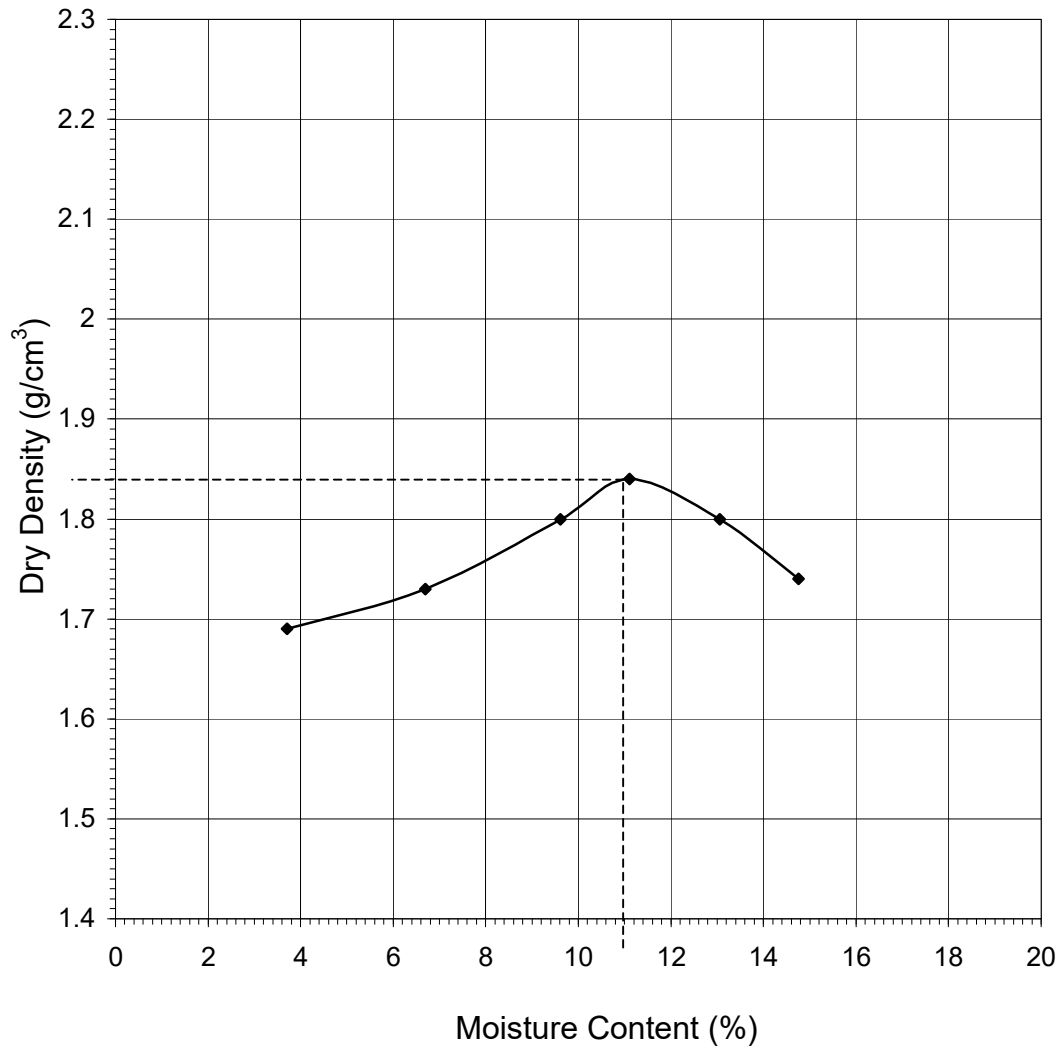


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Job No. : G(D)4527

Sheet No. : 7f



Maximum Dry Density = 1.76 g/cm³
 Optimum Moisture Content = 15.8 %

Modified Proctor Density Test - 7

(Refer paragraph no. 7.11.3)

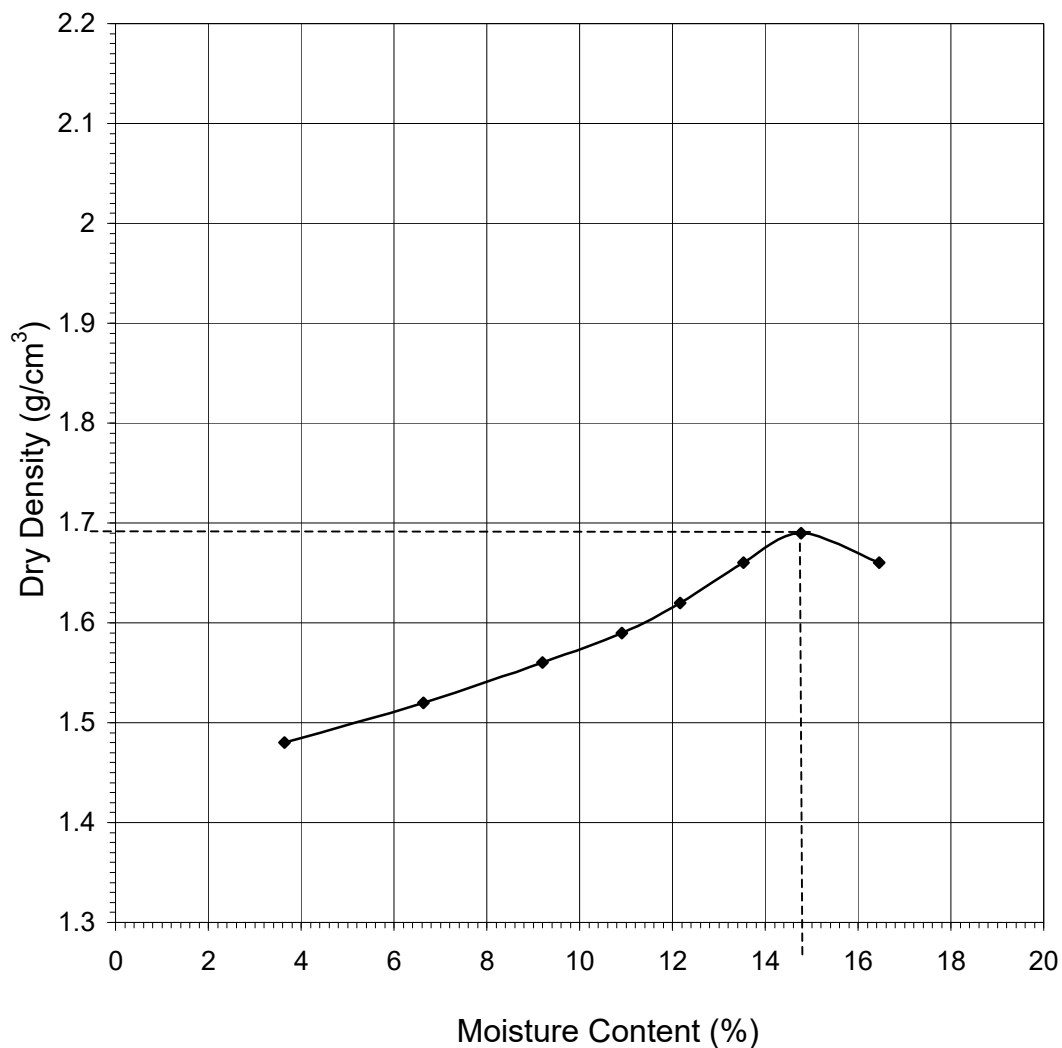


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Job No. : G(D)4527

Sheet No. : 7g



Maximum Dry Density = 1.81 g/cm³
 Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 8

(Refer paragraph no. 7.11.3)

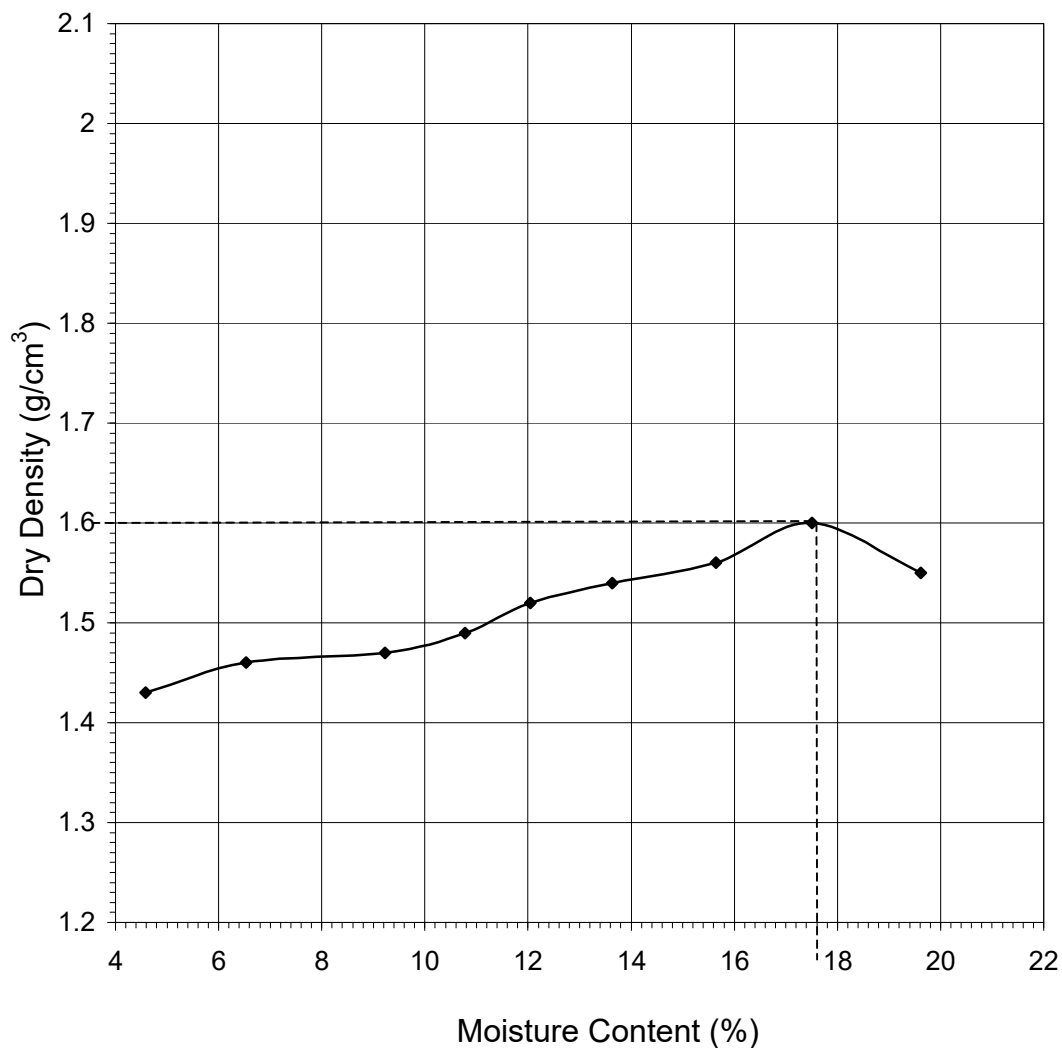


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Job No. : G(D)4527

Sheet No. : 7h



Maximum Dry Density = 1.81 g/cm³
 Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 9

(Refer paragraph no. 7.11.3)

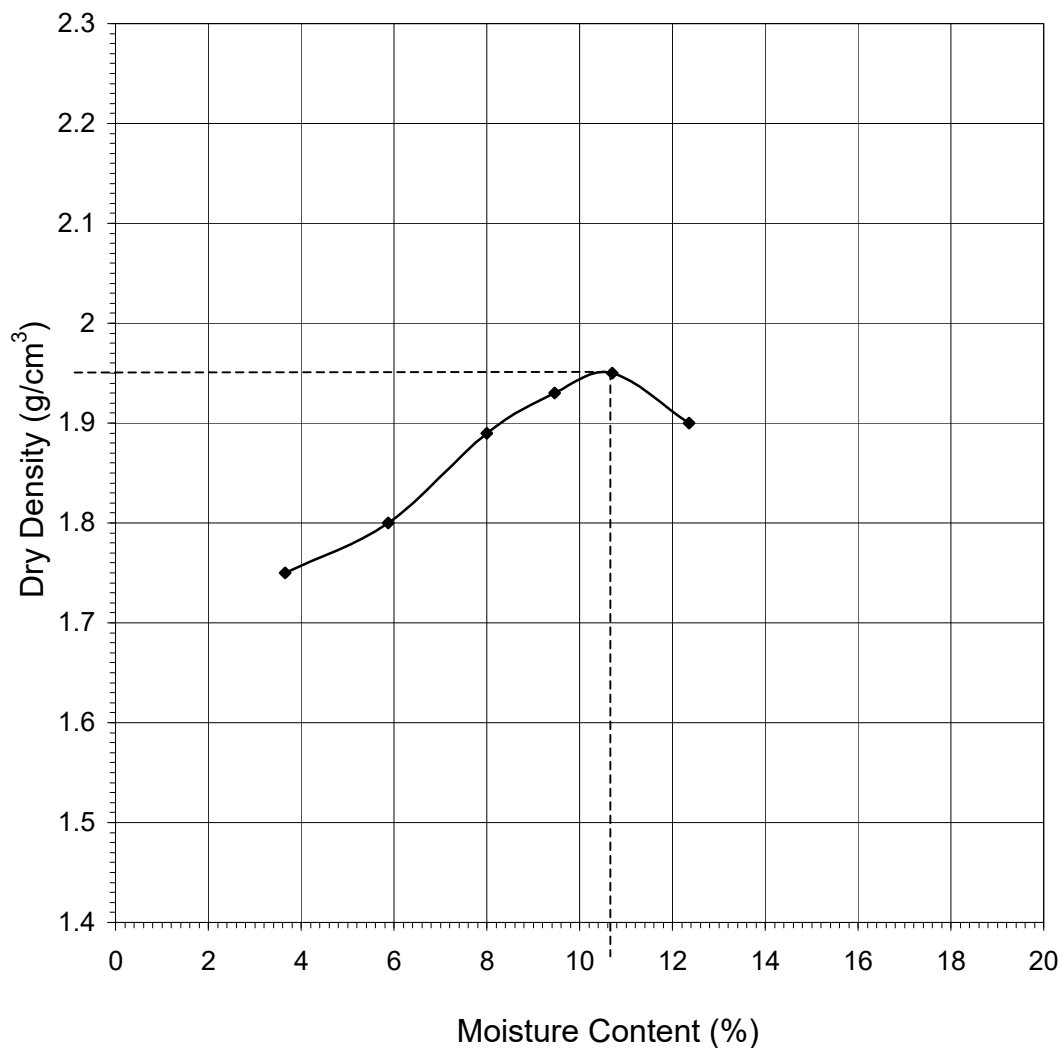


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Job No. : G(D)4527

Sheet No. : 7i



Maximum Dry Density = 1.77 g/cm³
 Optimum Moisture Content = 16.0 %

Modified Proctor Density Test - 10

(Refer paragraph no. 7.11.3)

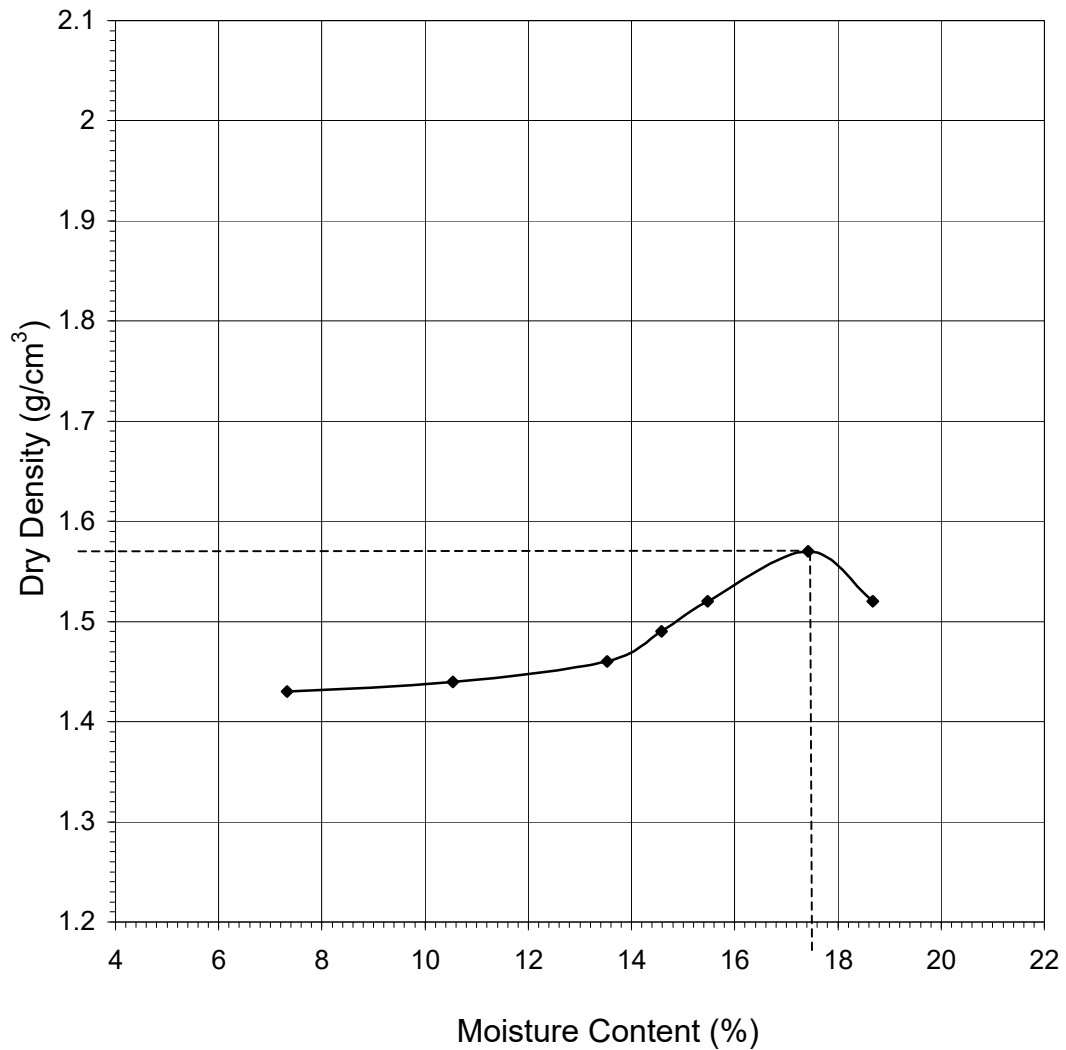


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Job No. : G(D)4527

Sheet No. : 7j



Maximum Dry Density = 1.81 g/cm³
Optimum Moisture Content = 13.6 %

Standard Proctor Density Test - 11

(Refer paragraph no. 7.11.3)

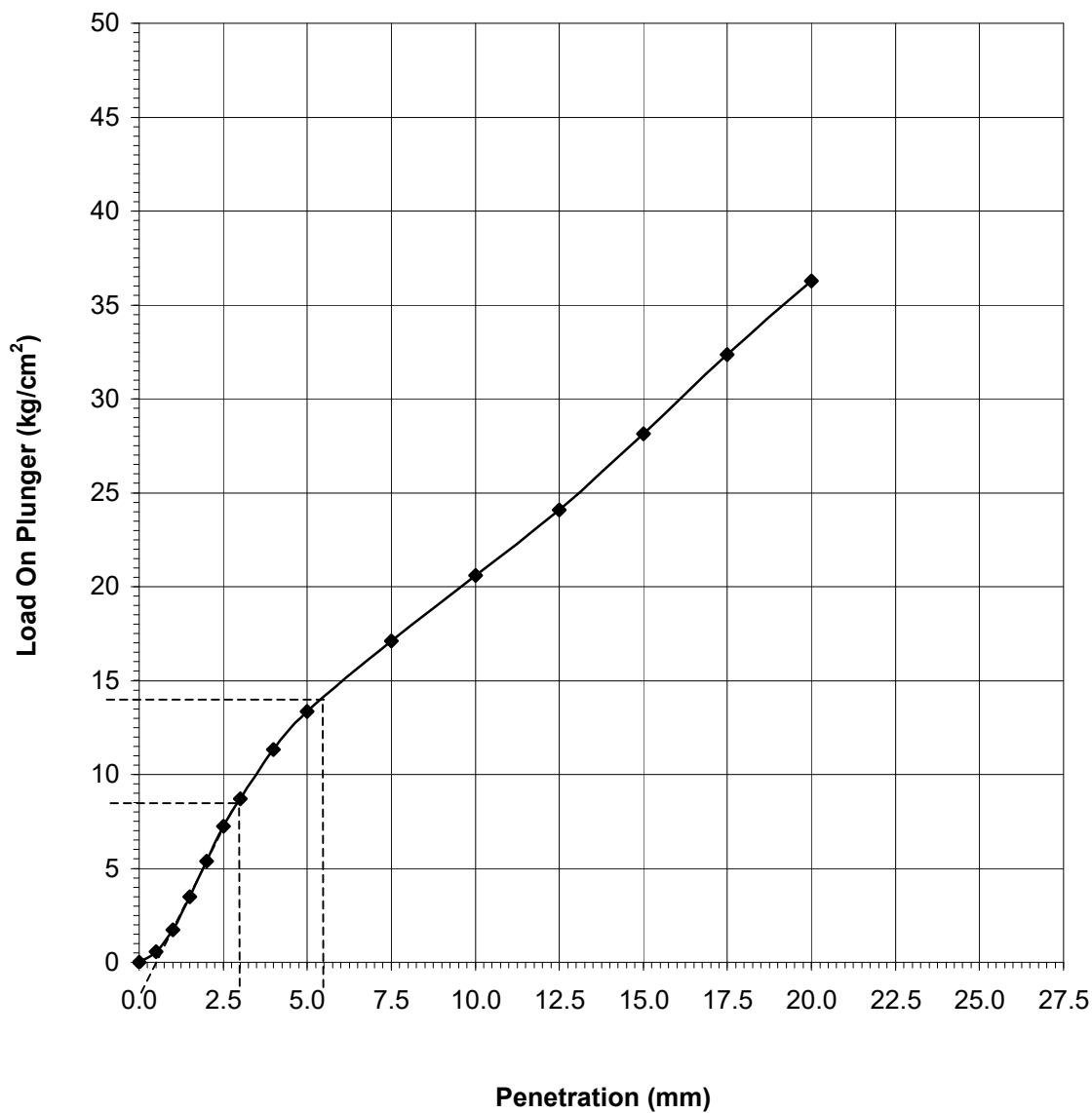


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Job No. : G(D)4527

Sheet No. : 7k



PENETRATION	2.5 mm	5.0 mm
CBR VALUE	12.1	13.3

Laboratory CBR Curve - 1
(Refer paragraph no. 7.12.1)

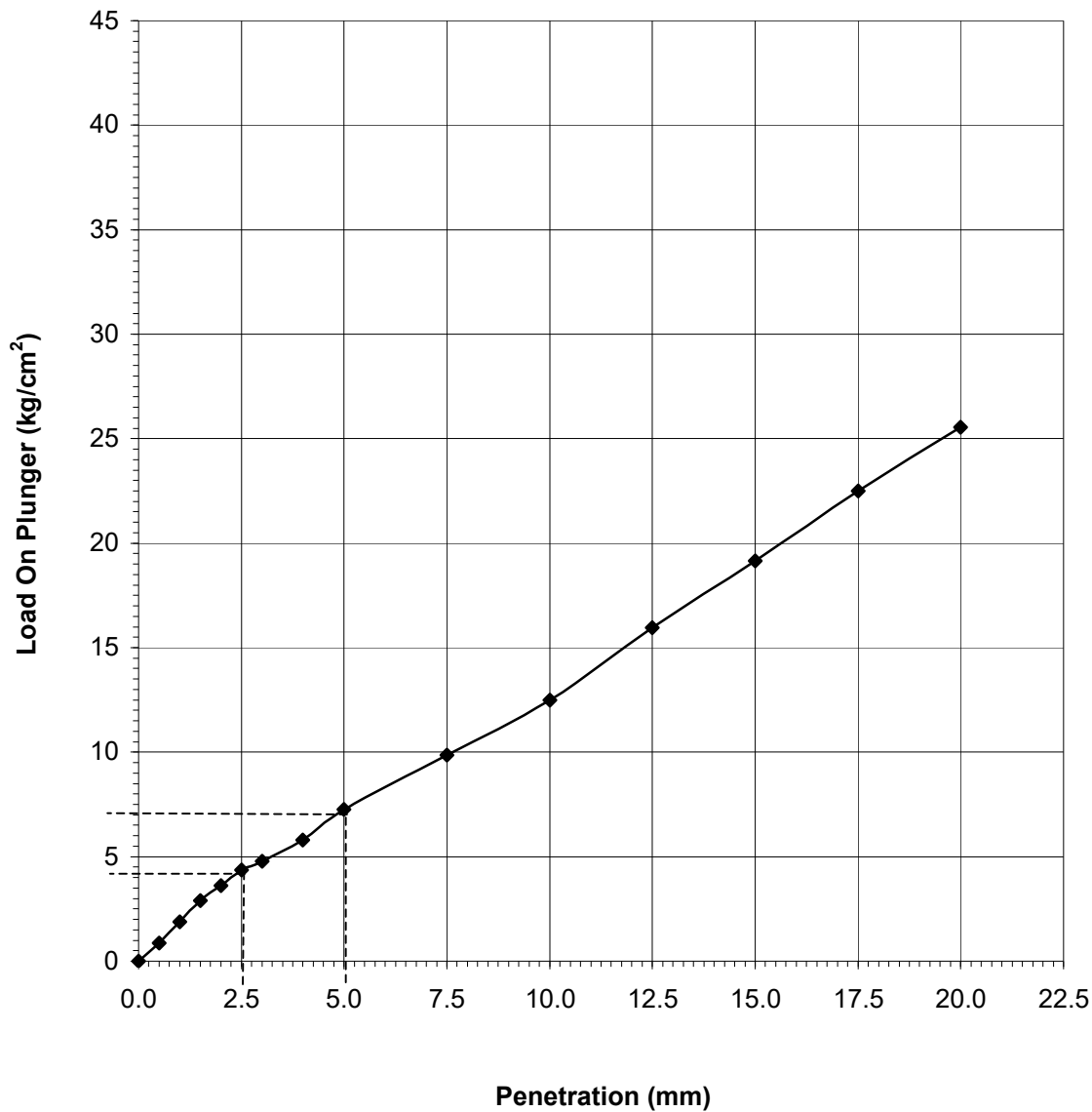


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Sheet No. : 8a



PENETRATION	2.5 mm	5.0 mm
CBR VALUE	6.3	6.9

Laboratory CBR Curve - 2
(Refer paragraph no. 7.12.1)

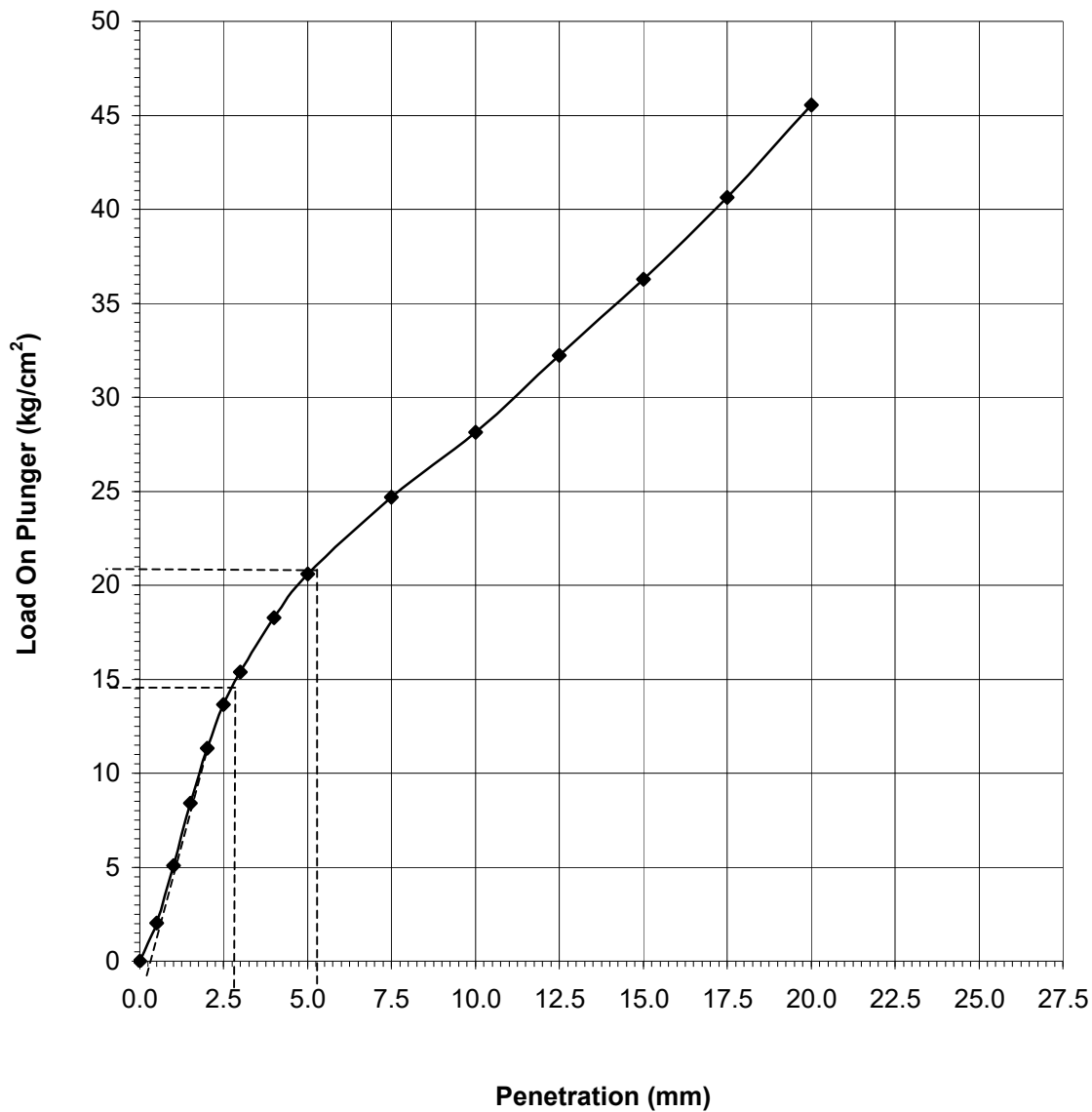


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Job No. G(D) 4527

Sheet No. : 8b



PENETRATION	2.5 mm	5.0 mm
CBR VALUE	20.7	20

Laboratory CBR Curve - 3
(Refer paragraph no. 7.12.1)

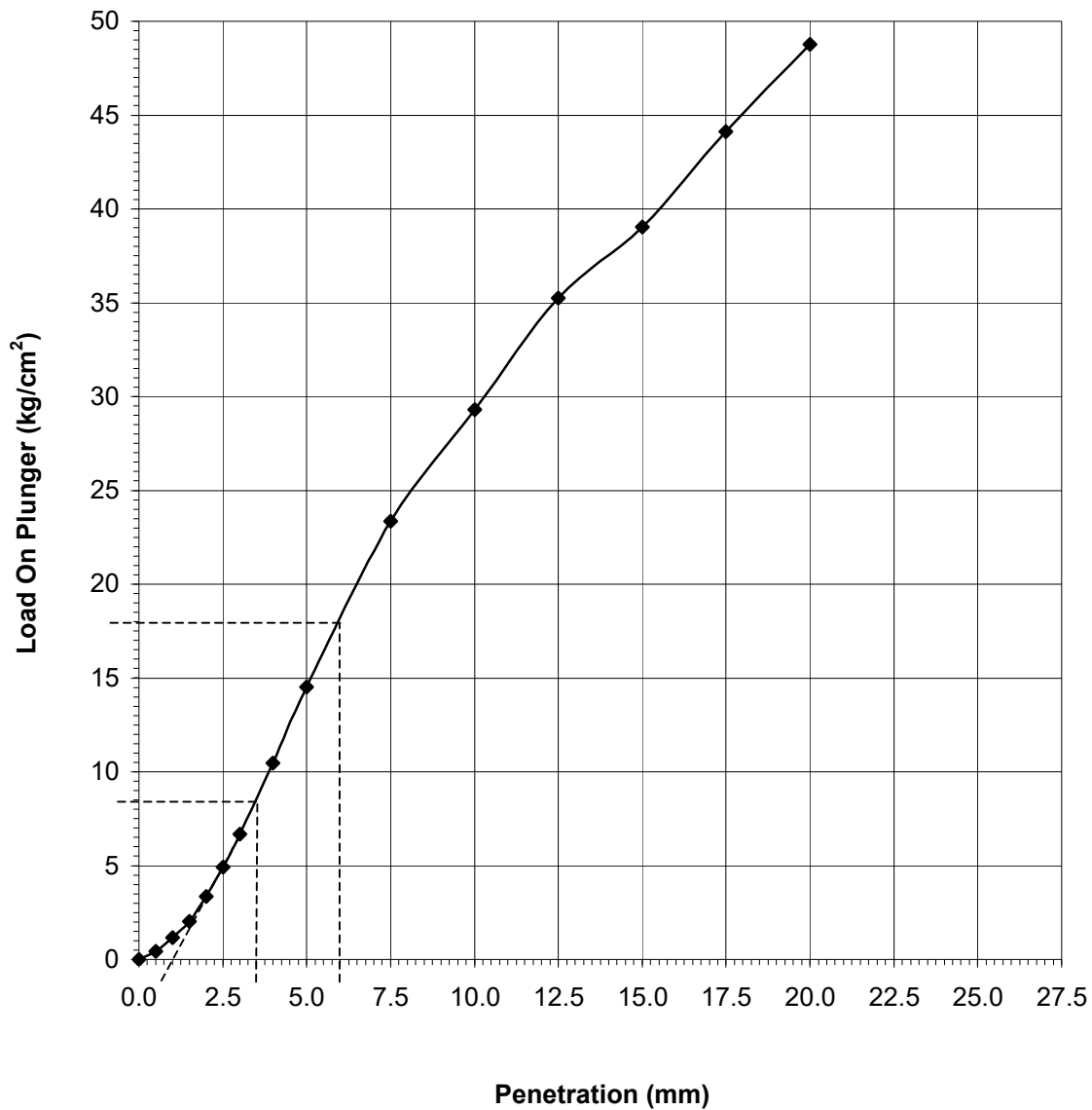


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Sheet No. : 8c



PENETRATION	2.5 mm	5.0 mm
CBR VALUE	12.1	17.1

Laboratory CBR Curve - 7

(Refer paragraph no. 7.12.1)



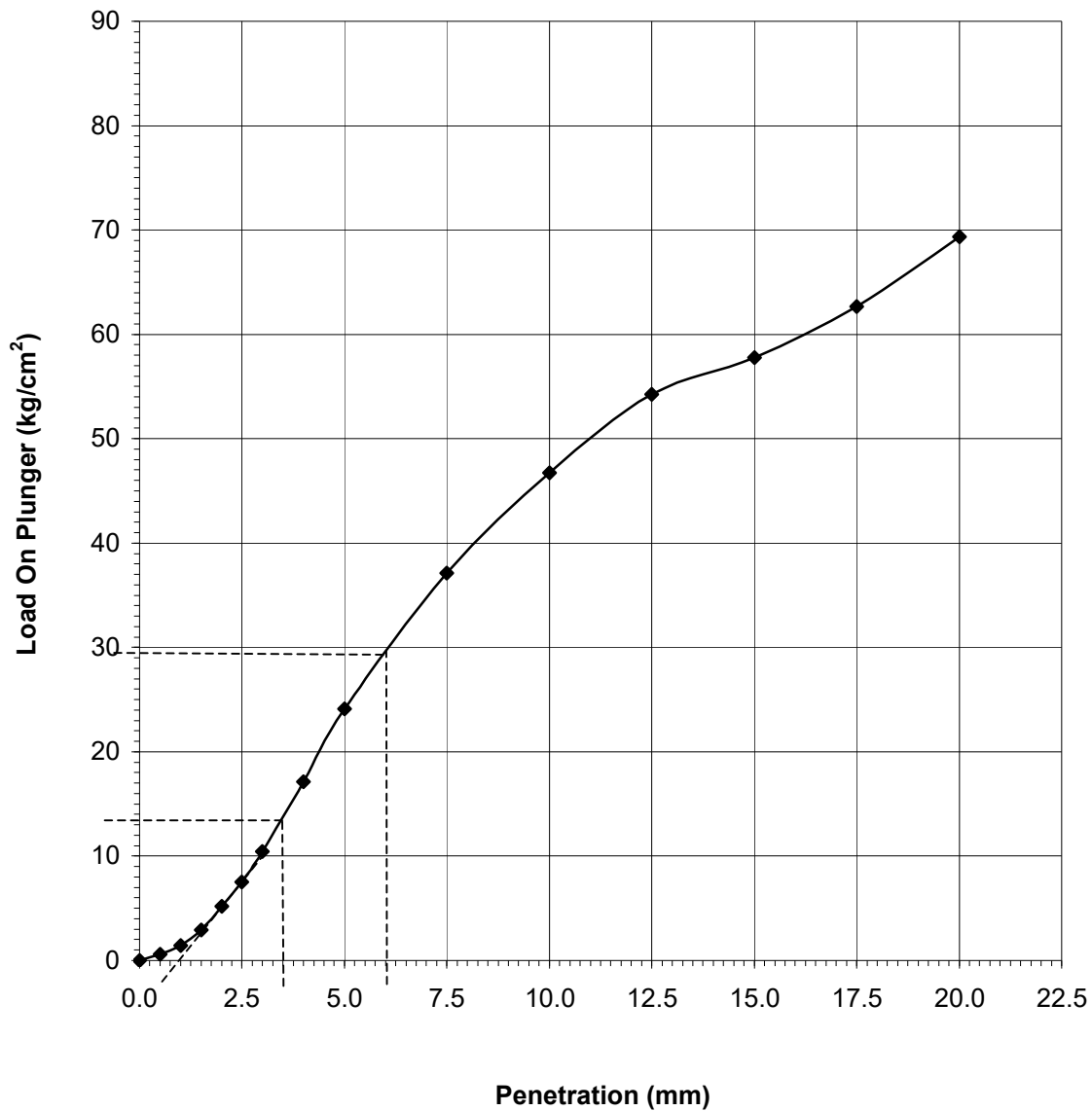
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PENETRATION	2.5 mm	5.0 mm
CBR VALUE	19.3	28.1

Laboratory CBR Curve - 10

(Refer paragraph no. 7.12.1)



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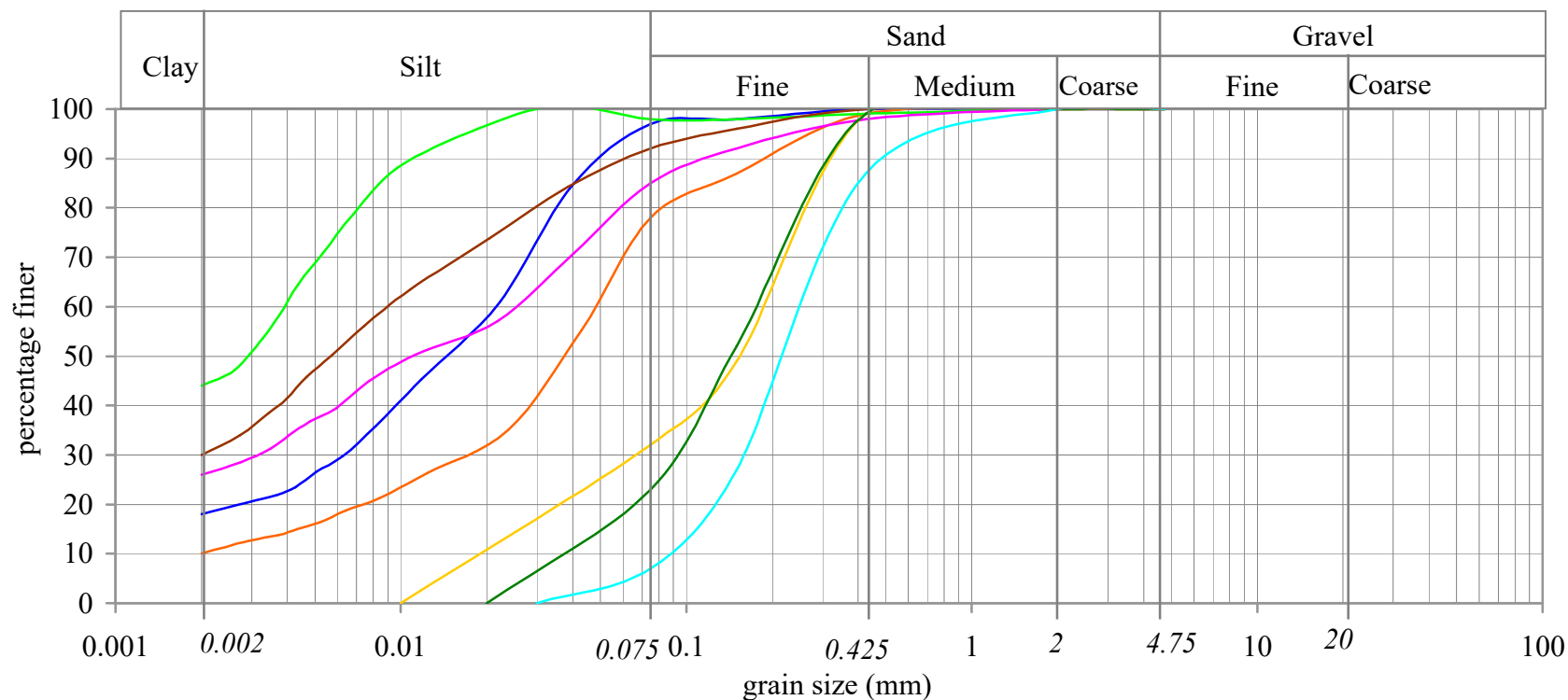


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	1	0.9	Sandy clayey silt	0	3	79	18	0.022	-	-
	1	2.4	Clayey sandy silt	0	22	68	10	0.049	0.002	24.5
	1	3.9	Sandy clayey silt	0	2	54	44	0.0039	-	-
	1	5.4	Sandy clayey silt	0	8	62	30	0.0081	-	-
	1	6.9	Sandy clayey silt	0	15	59	26	0.025	-	-
	1	8.4	Silty sand	0	68	32	0	0.19	0.019	10.0
	1	9.9	Silty sand	0	77	23	0	0.18	0.038	4.7
	1	11.4	Sand with traces of silt	0	93	7	0	0.25	0.09	2.8

Job No. : G(D)4527

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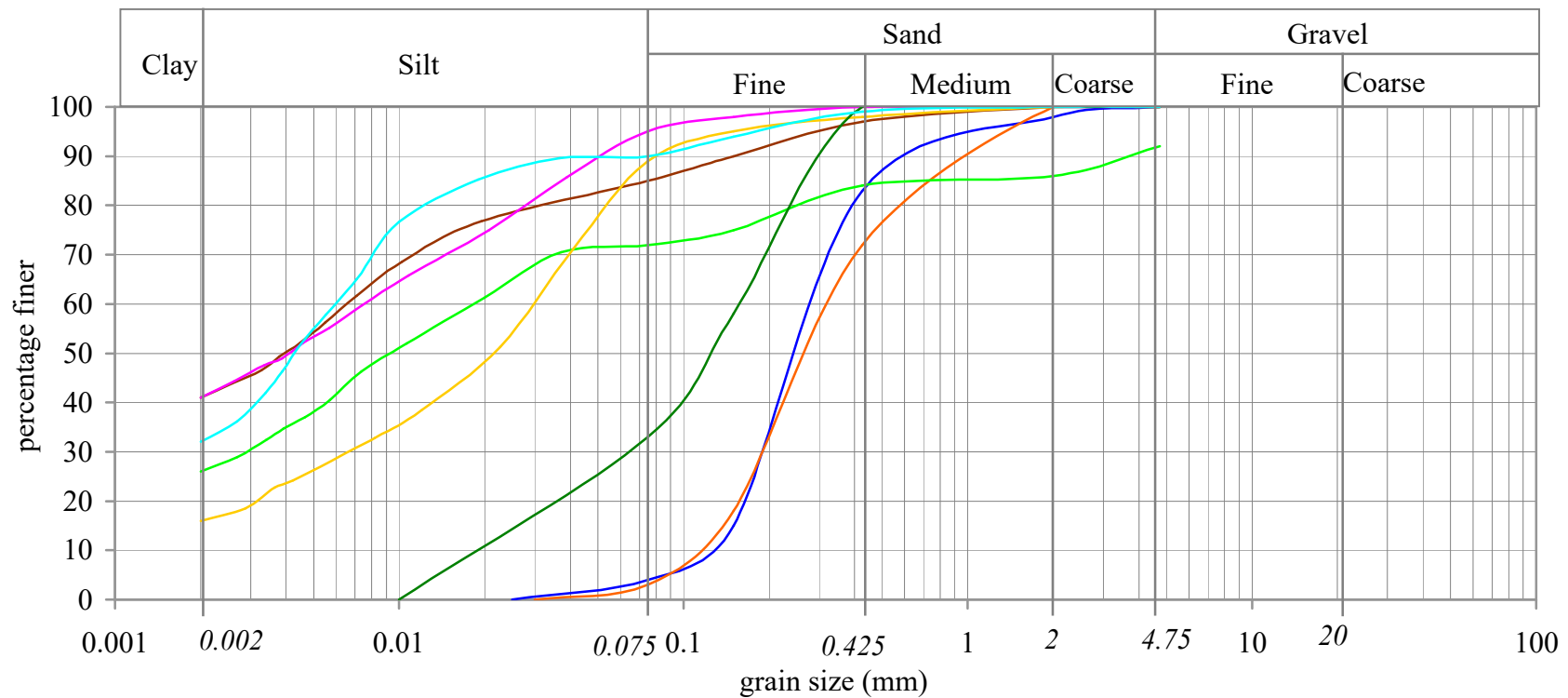
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Job No. : G(D)4527
 Sheet No. : 9b

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
—	1	12.9	Sand with traces of silt	0	96	4	0	0.26	0.13	2.0
—	1	14.4	Sand with traces of silt	0	97	3	0	0.31	0.12	2.6
—	1	15.9	Sandy clayey silt with gravel	8	20	46	26	0.019	-	-
—	1	17.4	Sandy clayey silt	0	15	44	41	0.0065	-	-
—	1	20	Sandy clayey silt	0	5	54	41	0.0075	-	-
—	2	1.5	Sandy clayey silt	0	11	73	16	0.03	-	-
—	2	3	Silty sand	0	67	33	0	0.15	0.019	7.9
—	2	4.5	Sandy clayey silt	0	10	58	32	0.0058	-	-

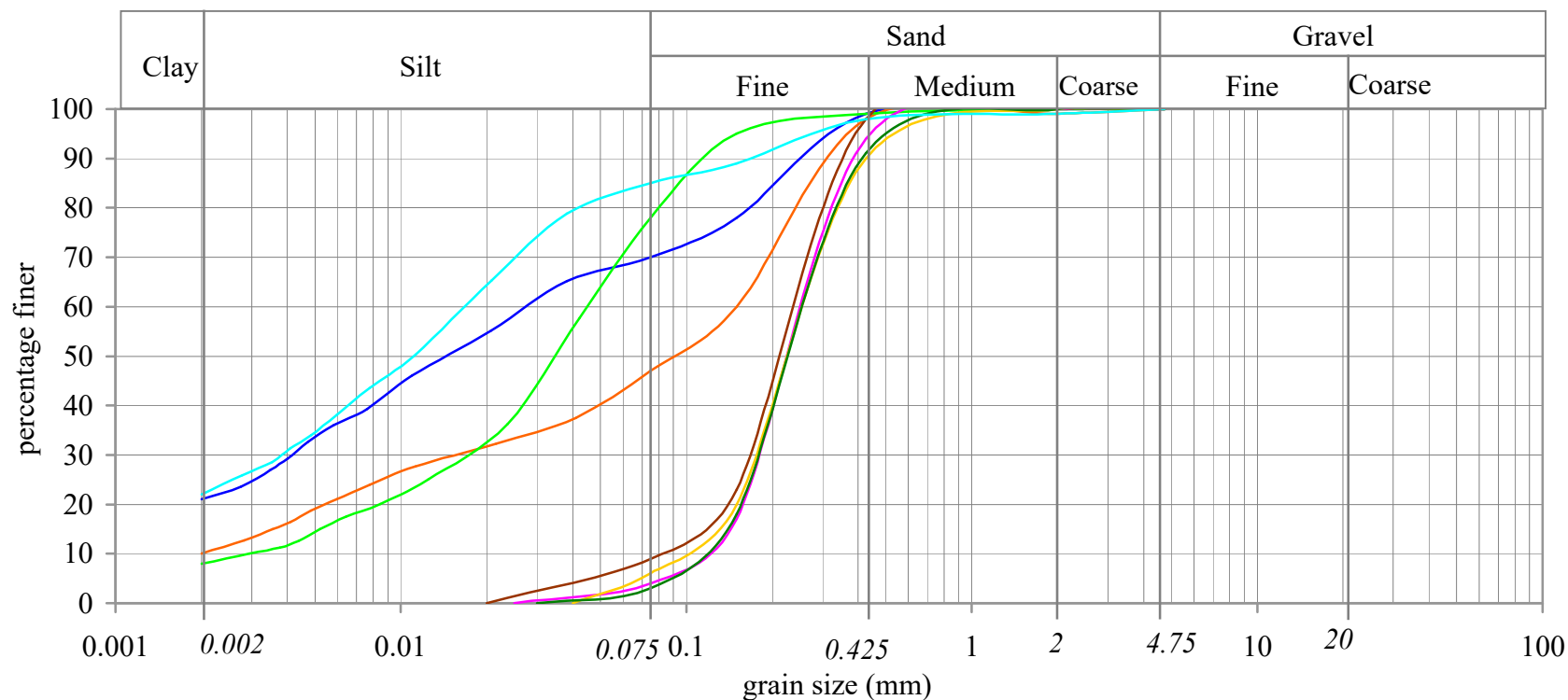


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	2	6	Clayey sandy silt	0	30	49	21	0.028	-	-
	2	7.5	Clayey silty sand	0	53	37	10	0.15	0.002	75.0
	2	9	Clayey sandy silt	0	22	70	8	0.045	-	-
	2	10.5	Sand with traces of silt	0	91	9	0	0.25	0.081	3.1
	2	12	Sand with traces of silt	0	96	4	0	0.25	0.13	1.9
	2	13.5	Sand with traces of silt	0	94	6	0	0.25	0.11	2.3
	2	15	Sand with traces of silt	0	97	3	0	0.25	0.13	1.9
	2	16.5	Sandy clayey silt	0	15	63	22	0.019	-	-

Job No. : G(D)4527

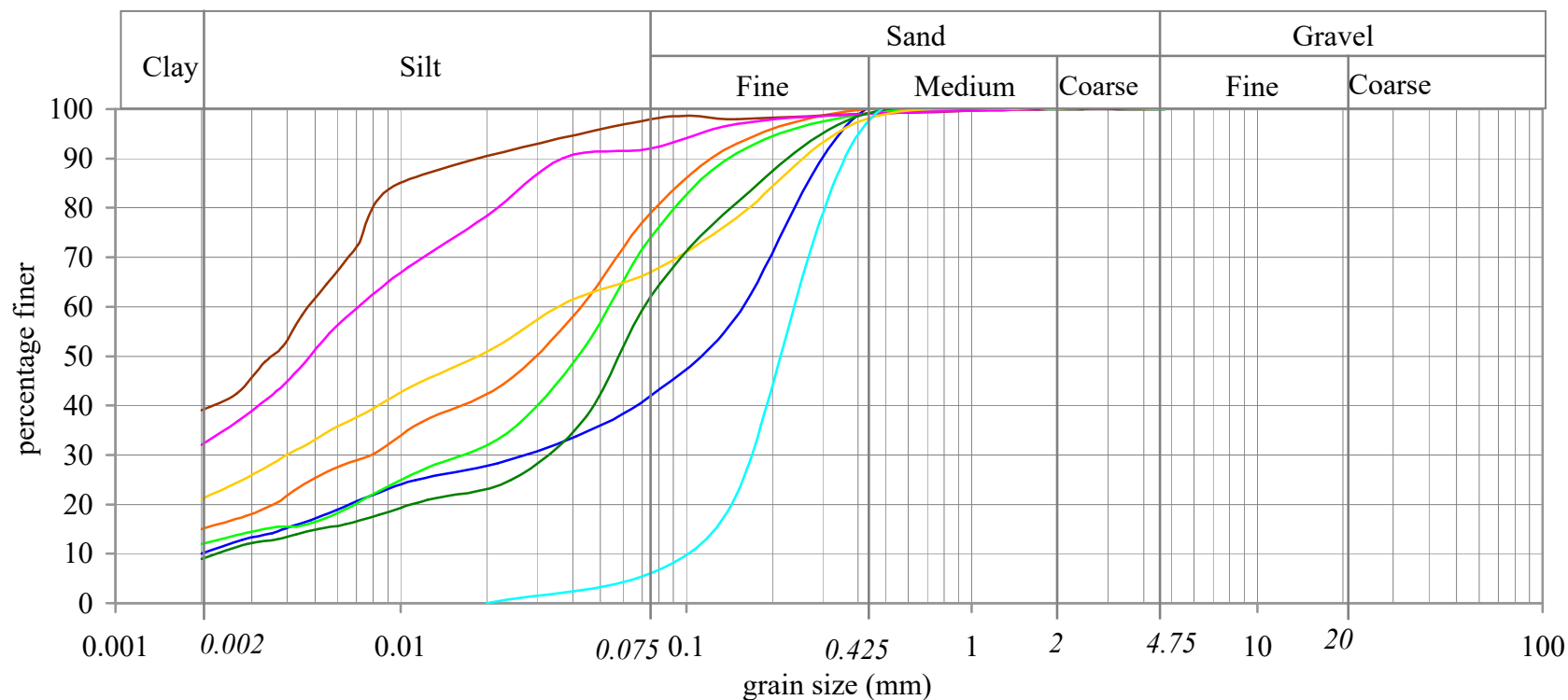
Sheet No. : 9c



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Job No. : G(D)4527
 Sheet No. : 9d

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
—	2	18	Clayey silty sand	0	58	32	10	0.17	0.002	85.0
—	3	0.9	Clayey sandy silt	0	21	64	15	0.042	-	-
—	3	2.4	Clayey sandy silt	0	26	62	12	0.052	-	-
—	3	3.9	Sandy clayey silt	0	2	59	39	0.0049	-	-
—	3	5.4	Sandy clayey silt	0	8	60	32	0.0071	-	-
—	3	6.9	Clayey sandy silt	0	33	46	21	0.035	-	-
—	3	8.4	Clayey sandy silt	0	38	53	9	0.071	0.0022	32.3
—	3	9.9	Sand with traces of silt	0	94	6	0	0.24	0.1	2.4

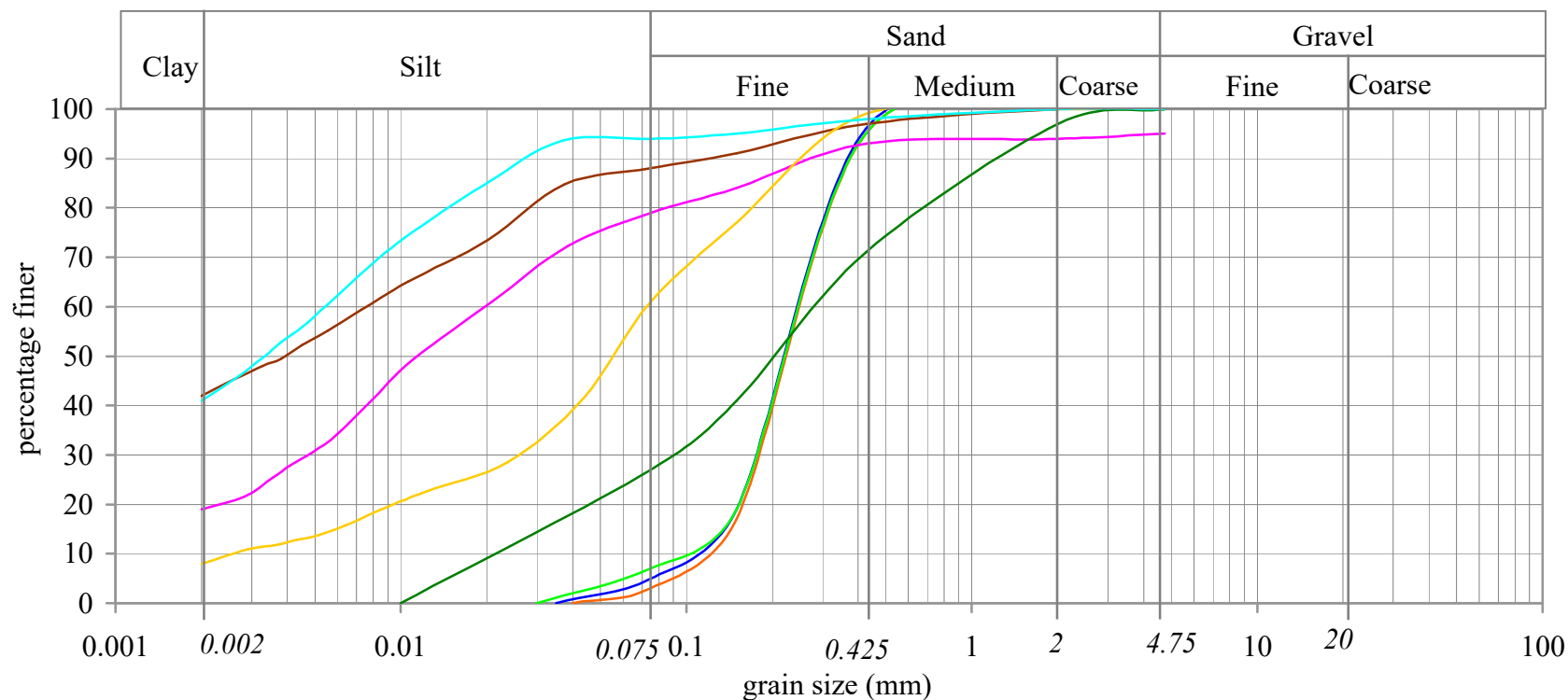


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	3	11.4	Sand with traces of silt	0	95	5	0	0.25	0.11	2.3
	3	12.9	Sand with traces of silt	0	97	3	0	0.25	0.13	1.9
	3	14.4	Sand with traces of silt	0	93	7	0	0.25	0.11	2.3
	3	15.9	Sandy clayey silt	0	12	46	42	0.0079	-	-
	3	17.4	Sandy clayey silt with gravel	5	16	60	19	0.021	-	-
	3	20.4	Clayey sandy silt	0	39	53	8	0.075	0.0025	-
	3	23.4	Silty sad	0	73	27	0	0.25	0.021	11.9
	3	26.4	Sandy clayey silt	0	6	53	41	0.25	-	-

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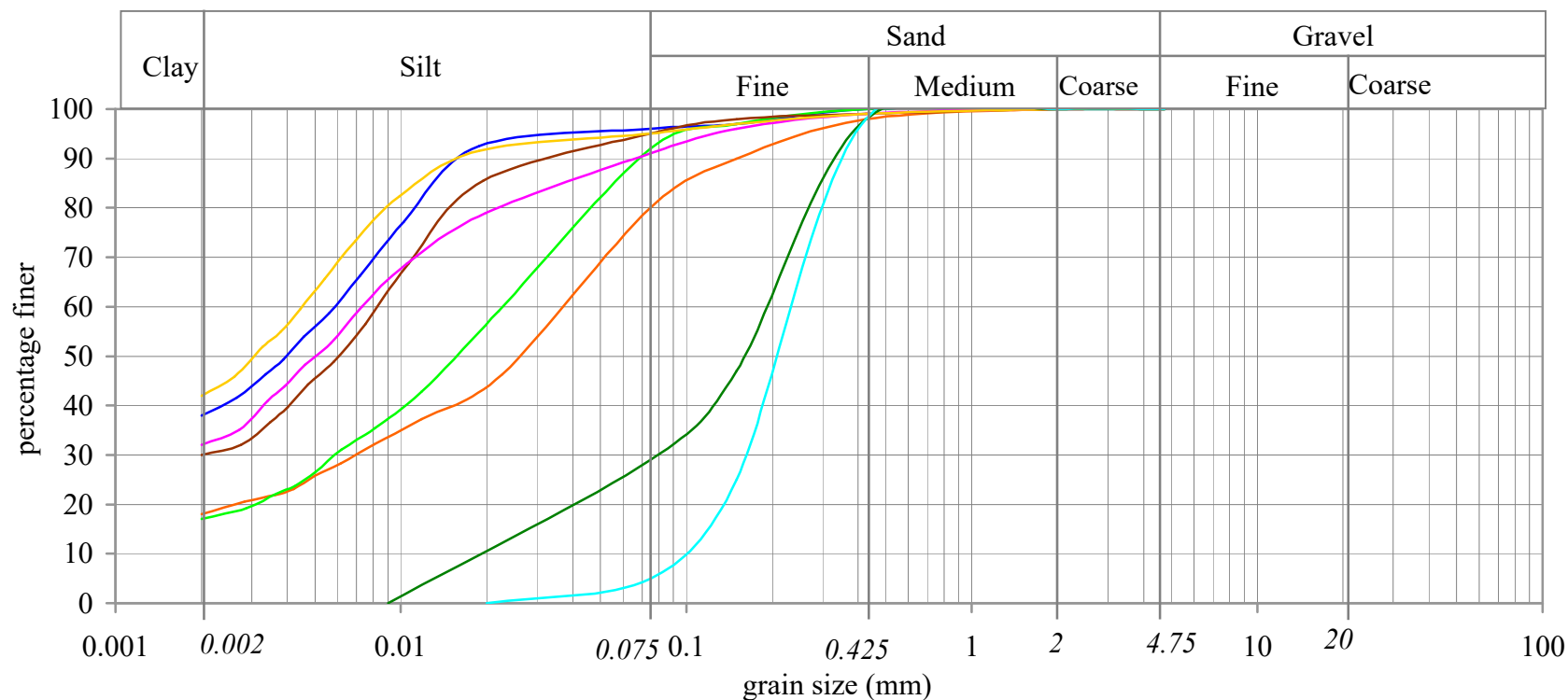


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	3	29.4	Sandy clayey silt	0	4	58	38	0.0059	-	-
	4	1.5	Clayey sandy silt	0	20	62	18	0.036	-	-
	4	3	Sandy clayey silt	0	8	75	17	0.023	-	-
	4	4.5	Sandy clayey silt	0	5	65	30	0.19	-	-
	4	6	Sandy clayey silt	0	9	59	32	0.0072	-	-
	4	7.5	Sandy clayey silt	0	5	53	42	0.0045	-	-
	4	9	Silty sand	0	71	29	0	0.19	0.019	10.0
	4	10.5	Sand with traces of silt	0	95	5	0	0.23	0.1	2.3

Job No. : G(D)4527

Sheet No. : 9f

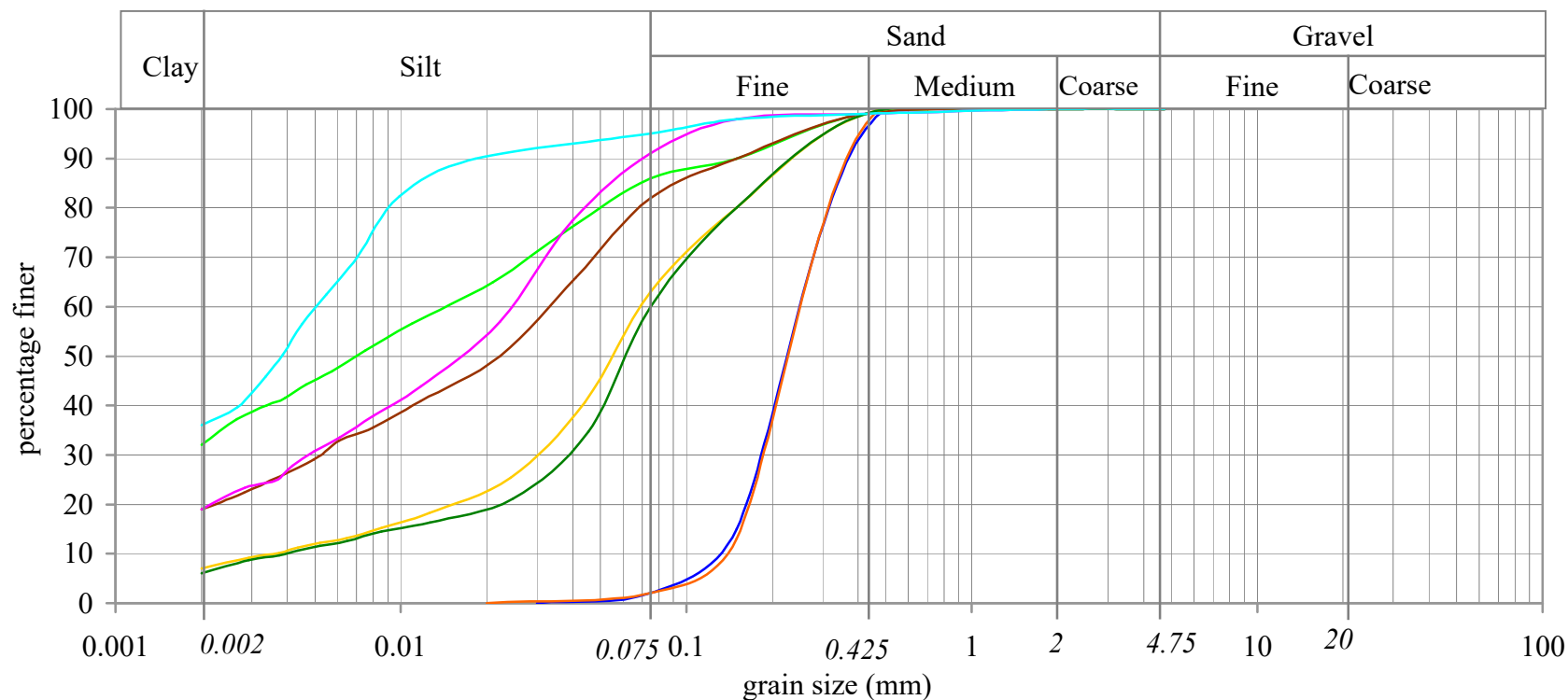


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	4	12	Sand with traces of silt	0	98	2	0	0.25	0.14	1.8
	4	13.5	Sand with traces of silt	0	98	2	0	0.25	0.15	1.7
	4	15	Sandy clayey silt	0	14	54	32	0.014	-	-
	4	16.5	Sandy clayey silt	0	18	63	19	0.032	-	-
	4	18	Sandy clayey silt	0	9	72	19	0.025	-	-
	5	0.9	Clayey sandy silt	0	37	56	7	0.069	0.0035	19.7
	5	2.4	Clayey sandy silt	0	40	54	6	0.078	0.0039	20.0
	5	3.9	Sand clayey silt	0	5	59	36	0.0051	-	-

Job No. : G(D)4527

Sheet No. : 9g



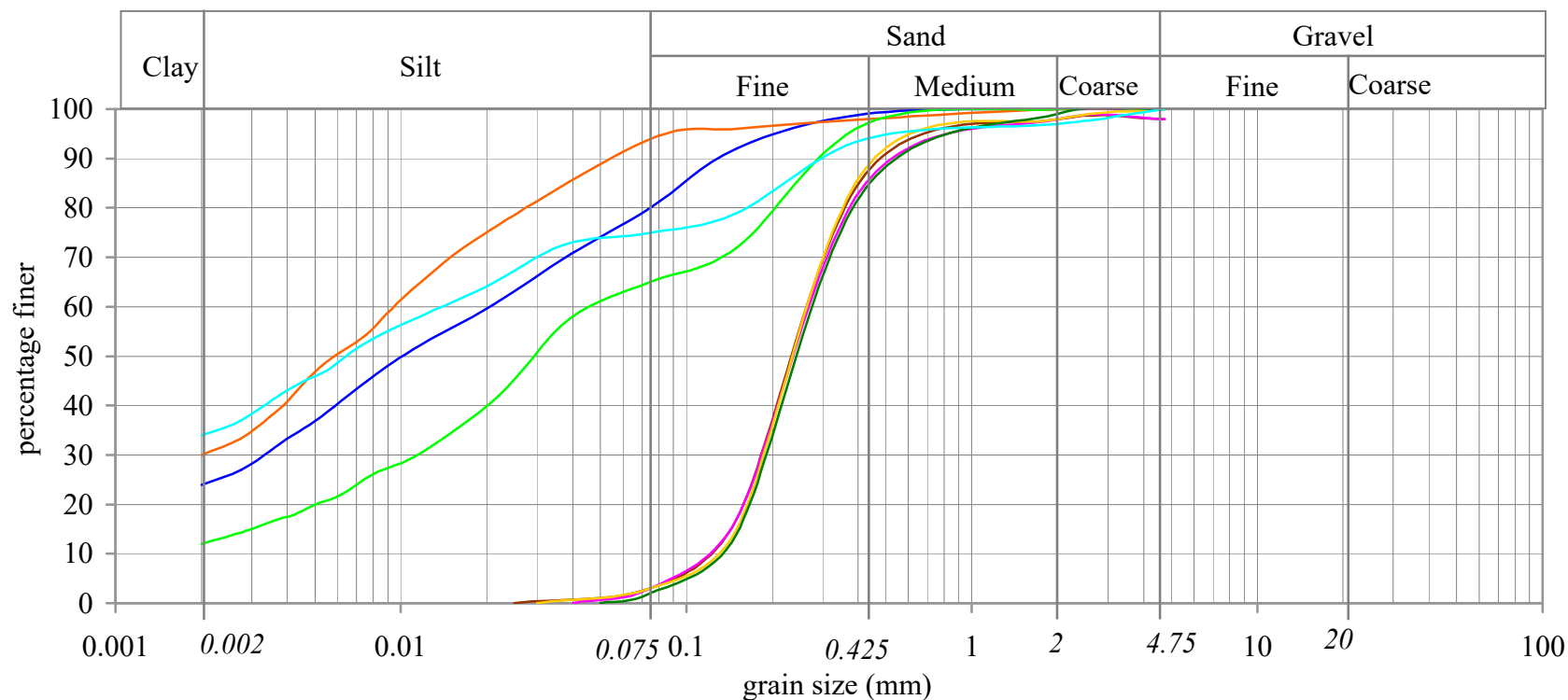
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Job No. : G(D)4527
 Sheet No. : 9h

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	5	5.4	Sandy clayey silt	0	20	56	24	0.021	-	-
	5	6.9	Sandy clayey silt	0	6	64	30	0.0095	-	-
	5	8.4	Clayey sandy silt	0	35	53	12	0.046	-	-
	5	9.9	Sand with traces of silt and gravel	2	95	3	0	0.27	0.12	-
	5	11.4	Sand with traces of silt and gravel	2	95	3	0	0.26	0.12	2.2
	5	12.9	Sand with traces of silt	0	97	3	0	0.26	0.14	1.9
	5	14.4	Sand with traces of silt	0	98	2	0	0.27	0.14	1.9
	5	15.9	Sandy clayey silt	0	25	41	34	0.015	-	-

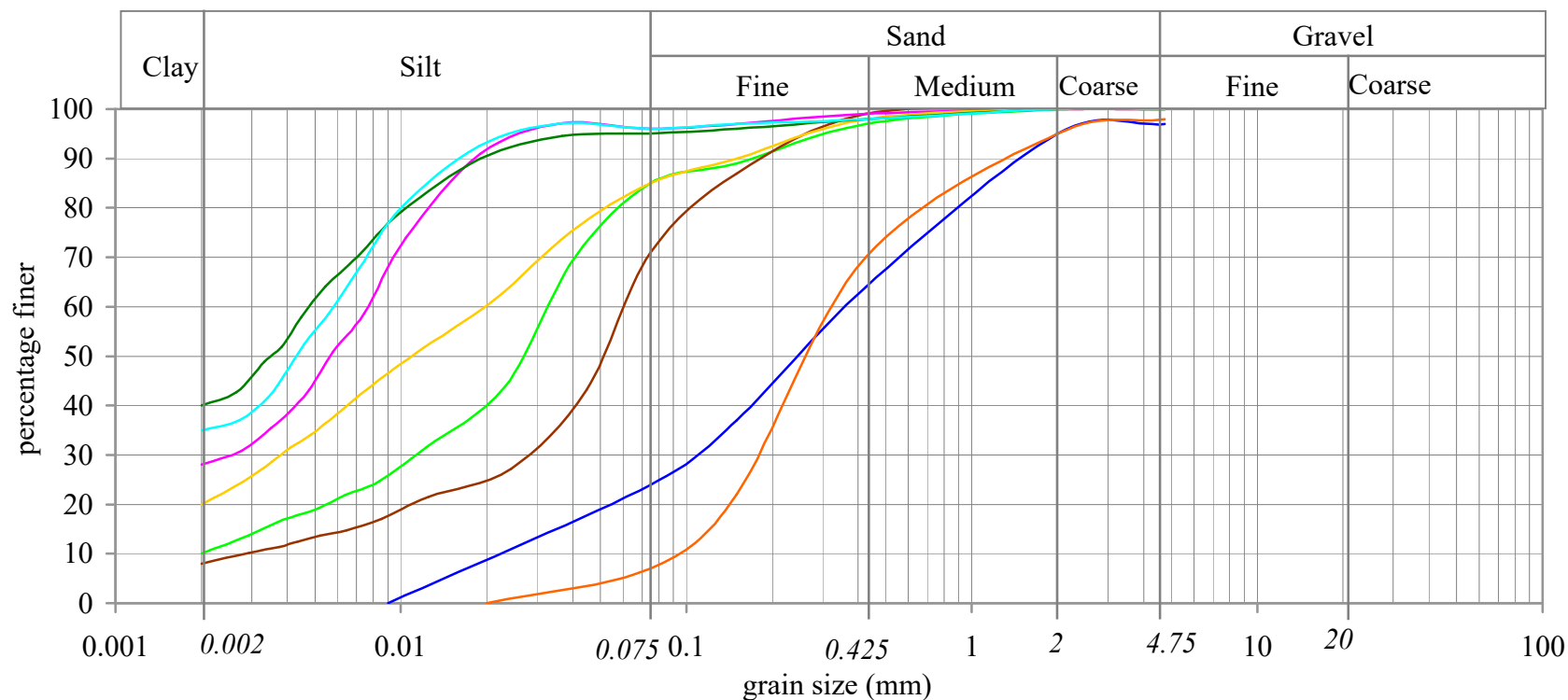


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	5	17.4	Silty sand with gravel	3	73	24	0	0.38	0.022	17.3
	5	20	Sand with traces of silt and gravel	2	91	7	0	0.22	0.095	-
	6	1.5	Clayey sandy silt	0	15	75	10	0.032	0.002	16.0
	6	3	Clayey sandy silt	0	29	63	8	0.061	0.0028	21.8
	6	4.5	Sandy clayey silt	0	4	68	28	0.0076	-	-
	6	6	Sandy clayey silt	0	15	65	20	0.021	-	-
	6	7.5	Sandy clayey silt	0	5	55	40	0.039	-	-
	6	9	Sandy clayey silt	0	4	61	35	0.048	-	-

Job No. : G(D)4527

Sheet No. : 91

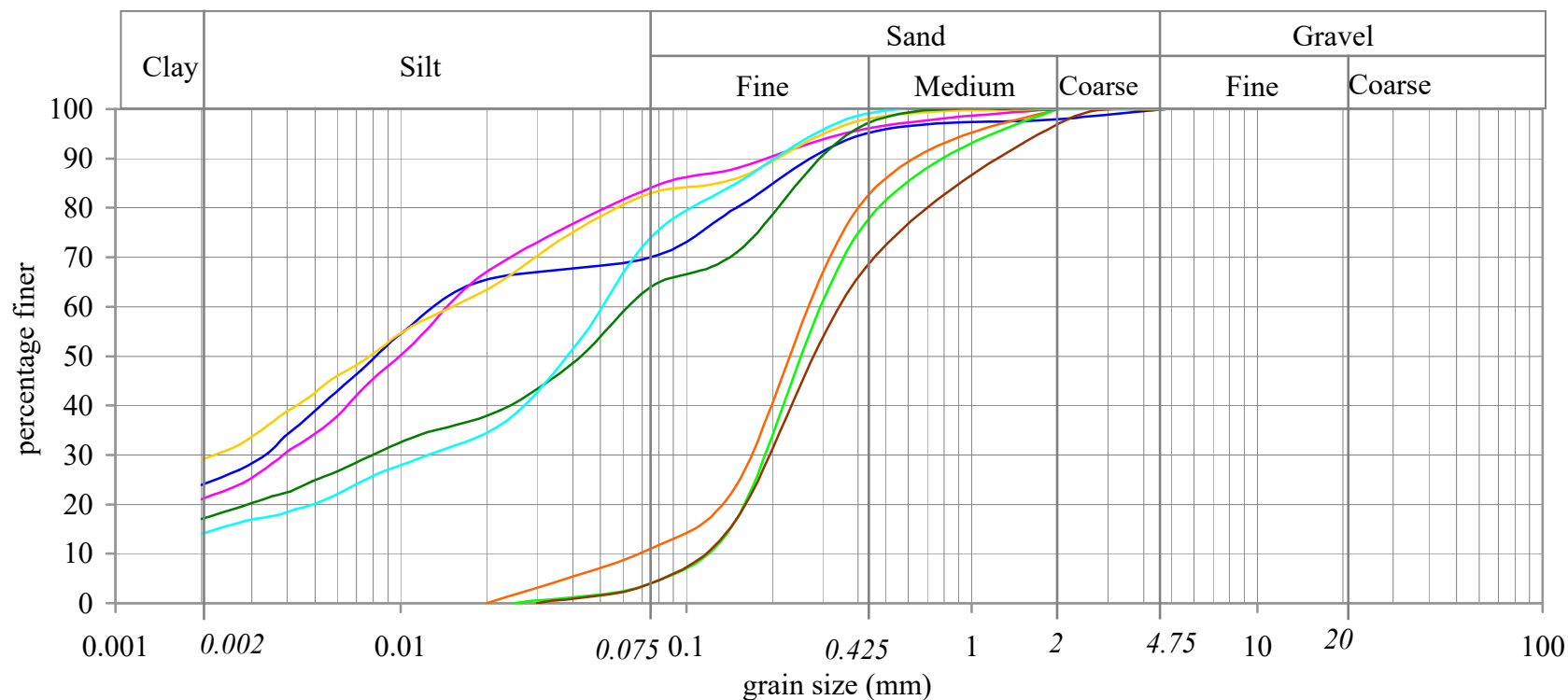


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	6	10.5	Clayey sandy silt	0	30	46	24	0.013	-	-
	6	12	Sand with traces of silt	0	89	11	0	0.26	0.065	4.0
	6	13.5	Sand with traces of silt	0	96	4	0	0.29	0.12	2.4
	6	15	Sand with traces of silt	0	96	4	0	0.35	0.12	2.9
	6	16.5	Sandy clayey silt	0	16	63	21	0.025	-	-
	6	18	Sandy clayey silt	0	17	54	29	0.016	-	-
	7	0.9	Clayey sandy silt	0	36	47	17	0.062	-	-
	7	2.4	Clayey sandy silt	0	26	60	14	0.051	-	-

Job No. : G(D)4527

Sheet No. : 9j

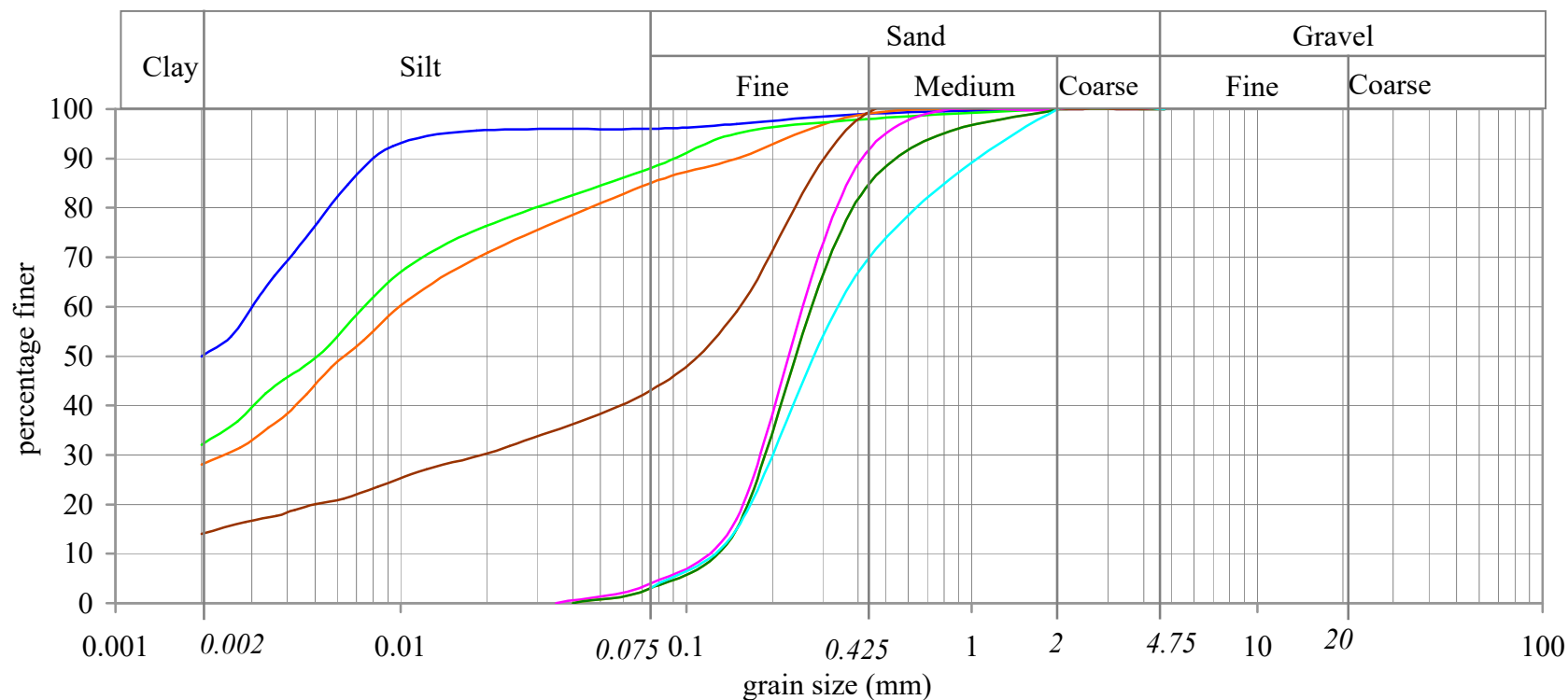


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	7	3.9	Sandy silty clay	0	4	46	50	0.0031	-	-
	7	5.4	Sandy clayey silt	0	15	57	28	0.01	-	-
	7	6.9	Sandy clayey silt	0	12	56	32	0.0065	-	-
	7	8.4	Clayey silty sand	0	57	29	14	0.15	-	-
	7	9.9	Sand with traces of silt	0	96	4	0	0.26	0.13	2.0
	7	11.4	Sand with traces of silt	0	97	3	0	0.27	0.13	2.1
	7	12.9	Sand with traces of silt	0	97	3	0	0.27	0.14	1.9
	7	14.4	Sand with traces of silt	0	97	3	0	0.24	0.14	1.7

Job No. : G(D)4527

Sheet No. : 9k

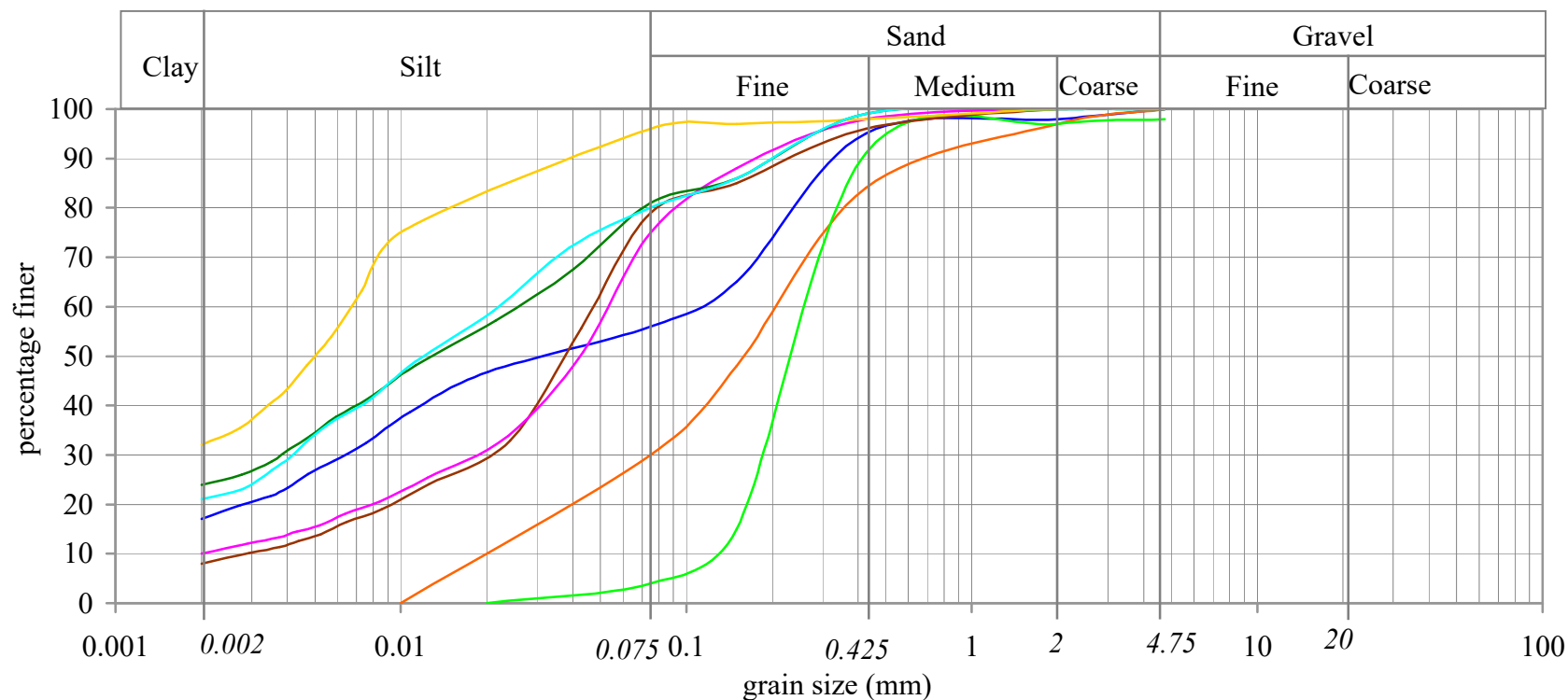


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	7	15.9	Clayey silty sand	0	44	39	17	0.12	-	-
	7	17.4	Silty sand	0	70	30	0	0.21	0.021	10.0
	7	20	Sand with traces of silt and gravel	2	94	4	0	0.26	0.11	2.4
	8	1.5	Clayey sandy silt	0	21	71	8	0.048	0.0028	17.1
	8	3	Clayey sandy silt	0	25	65	10	0.055	0.002	27.5
	8	4.5	Sandy clayey silt	0	4	64	32	0.0069	-	-
	8	6	Sandy clayey silt	0	19	57	24	0.025	-	-
	8	7.5	Sandy clayey silt	0	20	59	21	0.023	-	-

Job No. : G(D)4527

Sheet No. : 91

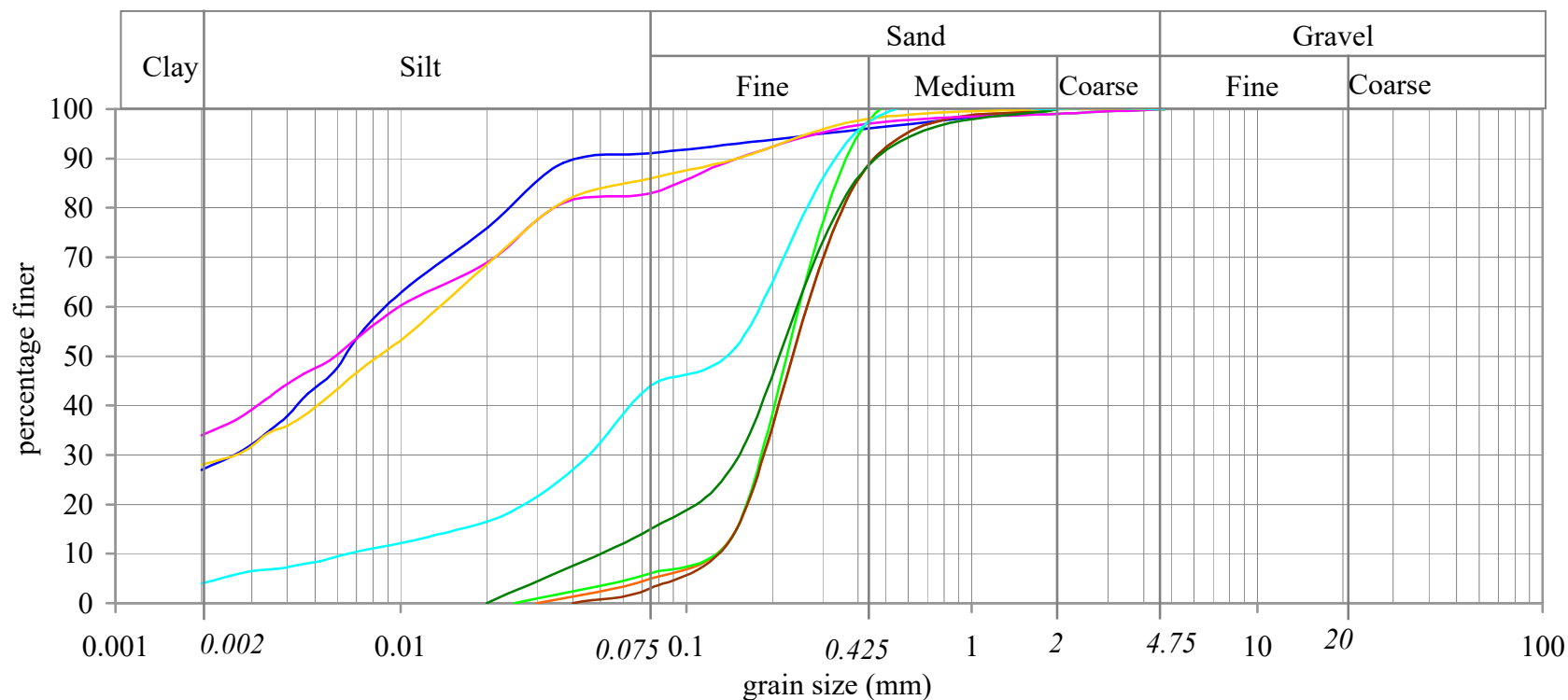


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	8	9	Sandy clayey silt	0	9	64	27	0.0089	-	-
	8	10.5	Sand with traces of silt	0	95	5	0	0.26	0.13	2.0
	8	12	Sand with traces of silt	0	94	6	0	0.25	0.13	1.9
	8	13.5	Sand with traces of silt	0	97	3	0	0.26	0.14	1.9
	8	15	Sandy clayey silt	0	17	49	34	0.01	-	-
	8	16.5	Sandy clayey silt	0	14	58	28	0.015	-	-
	8	18	Sand with silt	0	85	15	0	0.25	0.051	4.9
	9	0.9	Clayey silty sand	0	56	40	4	0.19	0.0061	31.1

Job No. : G(D)4527

Sheet No. : 9m



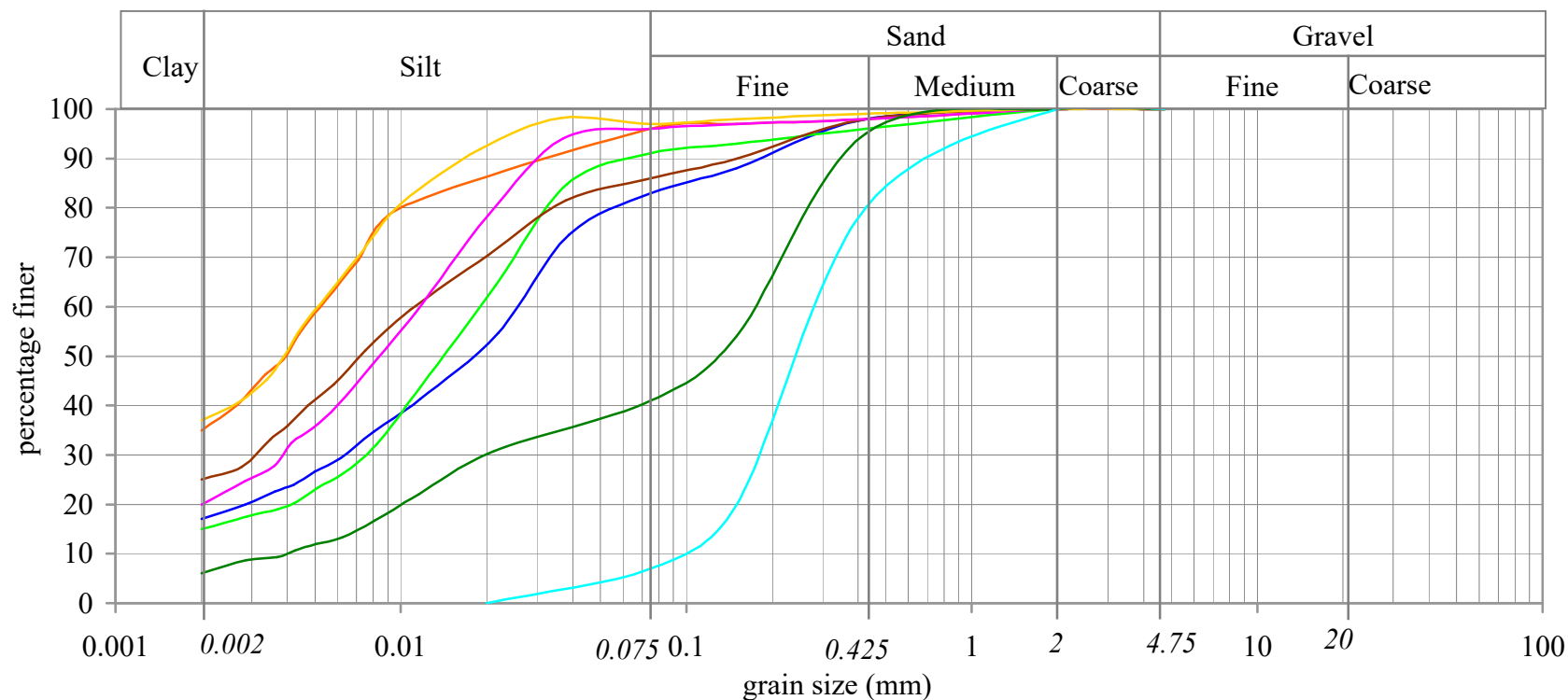
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Job No. : G(D)4527
 Sheet No. : 9n

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	9	2.4	Clayey sandy silt	0	17	66	17	0.026	-	-
	9	3.9	Sandy clayey silt	0	4	61	35	0.0041	-	-
	9	5.4	Sandy clayey silt	0	9	76	15	0.019	-	-
	9	6.9	Sandy clayey silt	0	14	61	25	0.012	-	-
	9	8.4	Sandy clayey silt	0	4	76	20	0.013	-	-
	9	9.9	Sandy clayey silt	0	3	60	37	0.0051	-	-
	9	11.4	Clayey silty sand	0	59	35	6	0.17	0.0038	44.7
	9	12.9	Sand with traces of silt	0	93	7	0	0.27	0.099	2.7

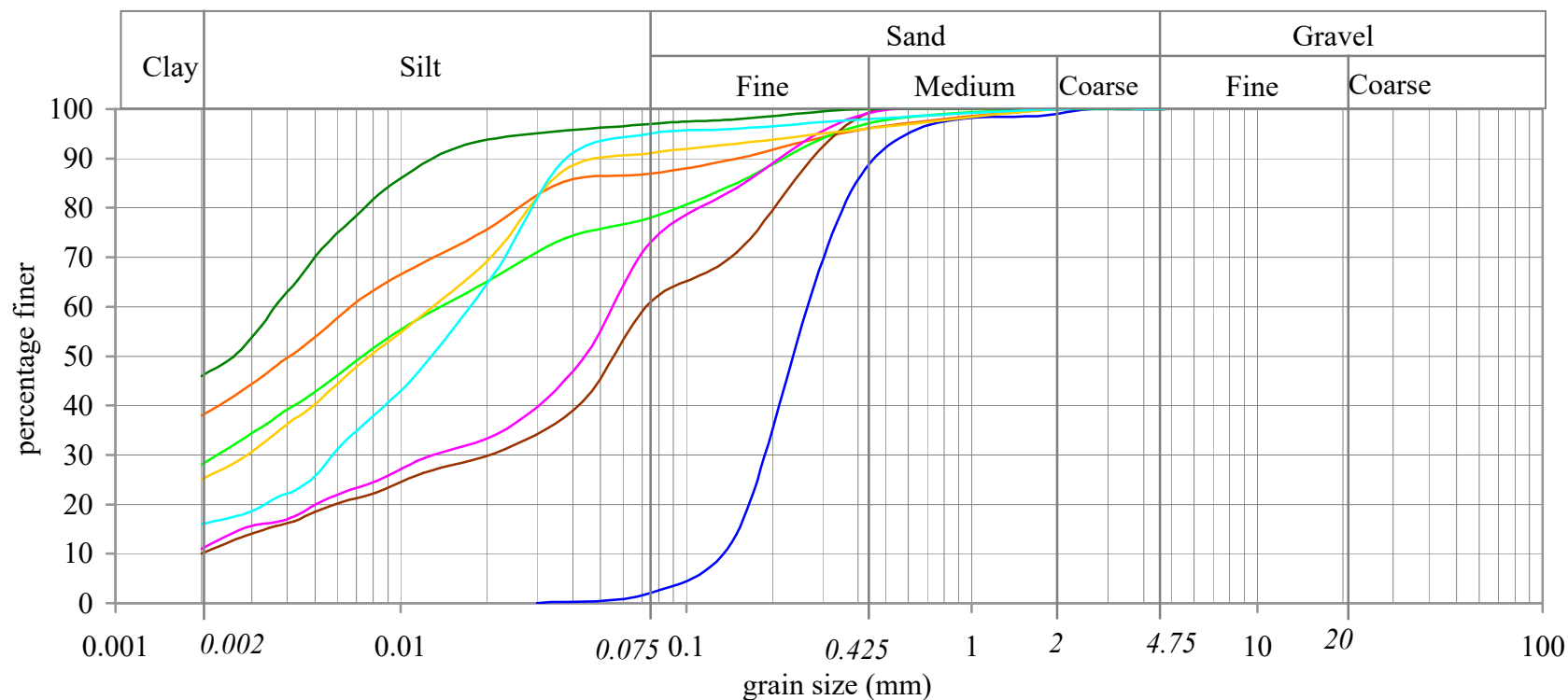


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	9	14.4	Sand with traces of silt	0	98	2	0	0.26		#DIV/0!
	9	15.9	Sandy clayey silt	0	13	49	38	0.0069	-	-
	9	17.4	Sandy clayey silt	0	22	50	28	0.015	-	-
	9	20	Clayey sandy silt	0	39	51	10	0.071	0.002	35.5
	10	1.5	Clayey sandy silt	0	27	62	11	0.055	-	-
	10	3	Sandy clayey silt	0	9	66	25	0.014	-	-
	10	4.5	Sandy clayey silt	0	3	51	46	0.0035	-	-
	10	6	Sandy clayey silt	0	5	79	16	0.017	-	-

Job No. : G(D)4527

Sheet No. : 90



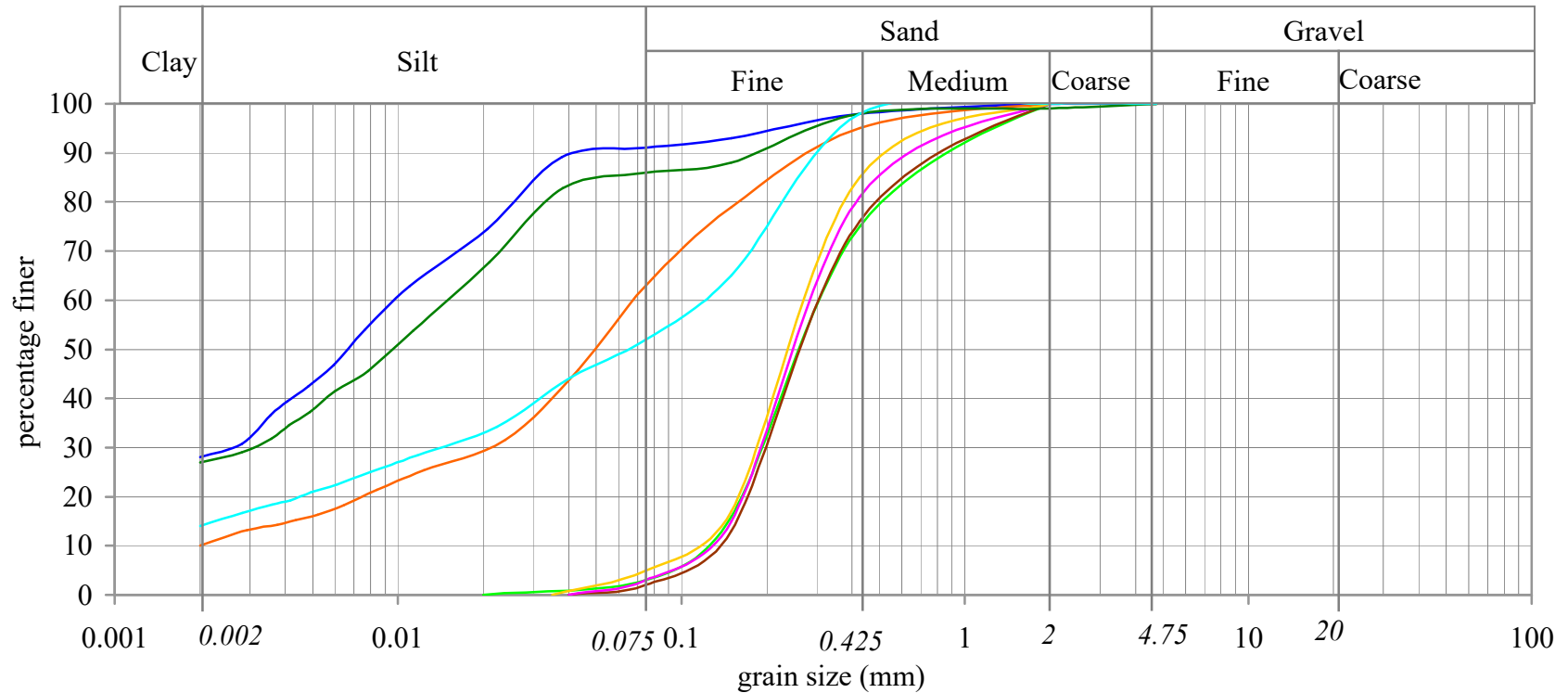
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Job No. : G(D)4527
 Sheet No. : 9p

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
—	10	7.5	Sandy clayey silt	0	9	63	28	0.0099	-	-
—	10	9	Clayey sandy silt	0	37	53	10	0.078	0.002	39.0
—	10	10.5	Sand with traces of silt	0	97	3	0	0.31	0.15	2.1
—	10	12	Sand with traces of silt	0	98	2	0	0.31	0.14	2.2
—	10	13.5	Sand with traces of silt	0	97	3	0	0.27	0.14	1.9
—	10	15	Sand with traces of silt	0	95	5	0	0.27	0.14	1.9
—	10	16.5	Sandy clayey silt	0	14	59	27	0.015	-	-
—	10	18	Clayey silty sand	0	48	38	14	0.14	-	-



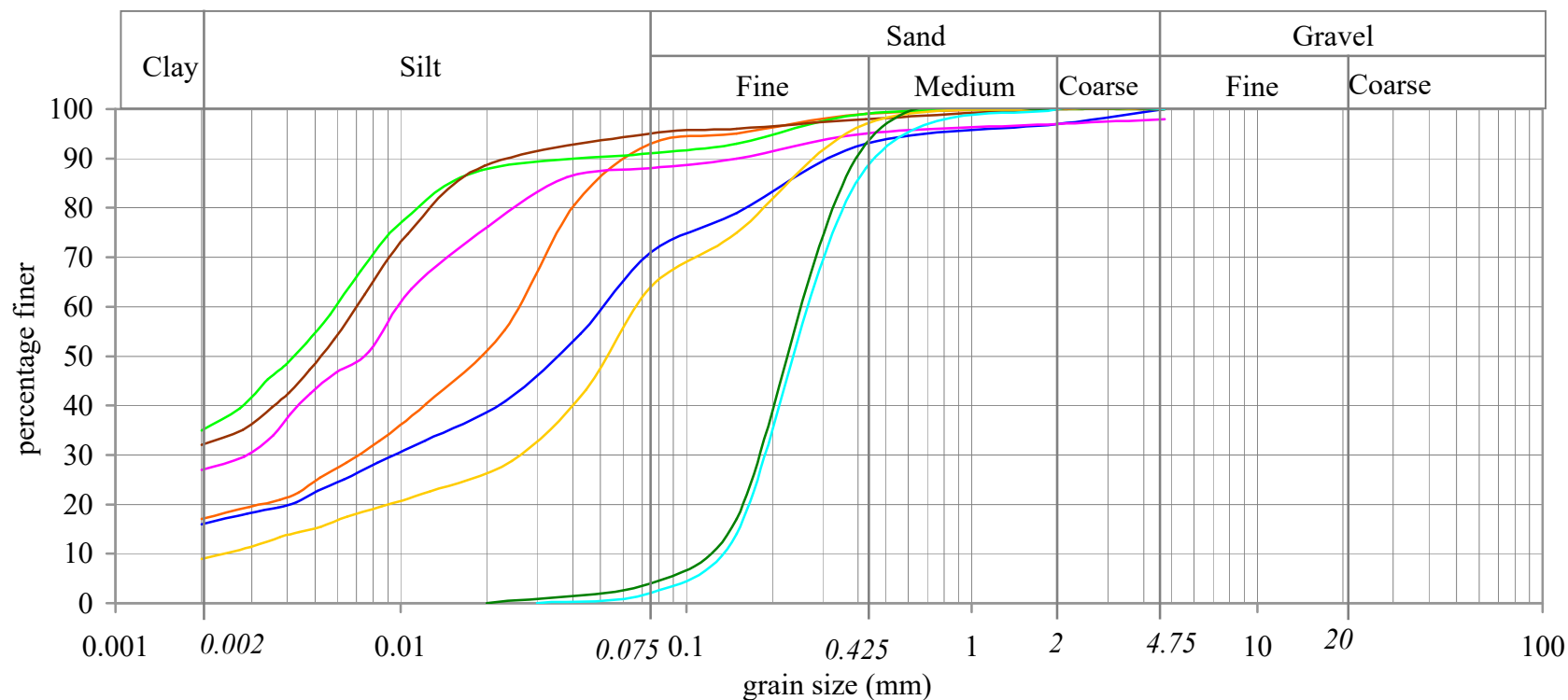
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Job No. : G(D)4527
 Sheet No. : 9q

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	Bore hole	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	11	0.9	Clayey sandy silt	0	29	55	16	0.051	-	-
	11	2.4	Sandy clayey silt	0	7	76	17	0.026	-	-
	11	3.9	Sandy clayey silt	0	9	56	35	0.0059	-	-
	11	5.4	Sandy clayey silt	0	5	63	32	0.0071	-	-
	11	6.9	Sandy clayey silt with gravel	2	10	61	27	0.0089	-	-
	11	8.4	Clayey sandy silt	0	36	55	9	0.068	0.0022	30.9
	11	9.9	Sand with traces of silt	0	96	4	0	0.26	0.12	2.2
	11	11.4	Sand with traces of silt	0	98	2	0	0.25	0.13	1.9



Job No.: G(D)4527

Sheet No.: 9r

Figure 1.1 is a semi-logarithmic plot showing the percentage finer (Y-axis, 0 to 100) versus grain size (mm) (X-axis, 0.001 to 100). The plot is divided into regions for Clay, Silt, Sand (Fine, Medium, Coarse), and Gravel (Fine, Coarse). Four curves are plotted, representing different soil samples. The green curve is the highest, followed by the magenta, brown, and blue curves. The blue curve shows a sharp increase in percentage finer between 0.075 mm and 0.425 mm, indicating a sand-sized soil.

[illegible]



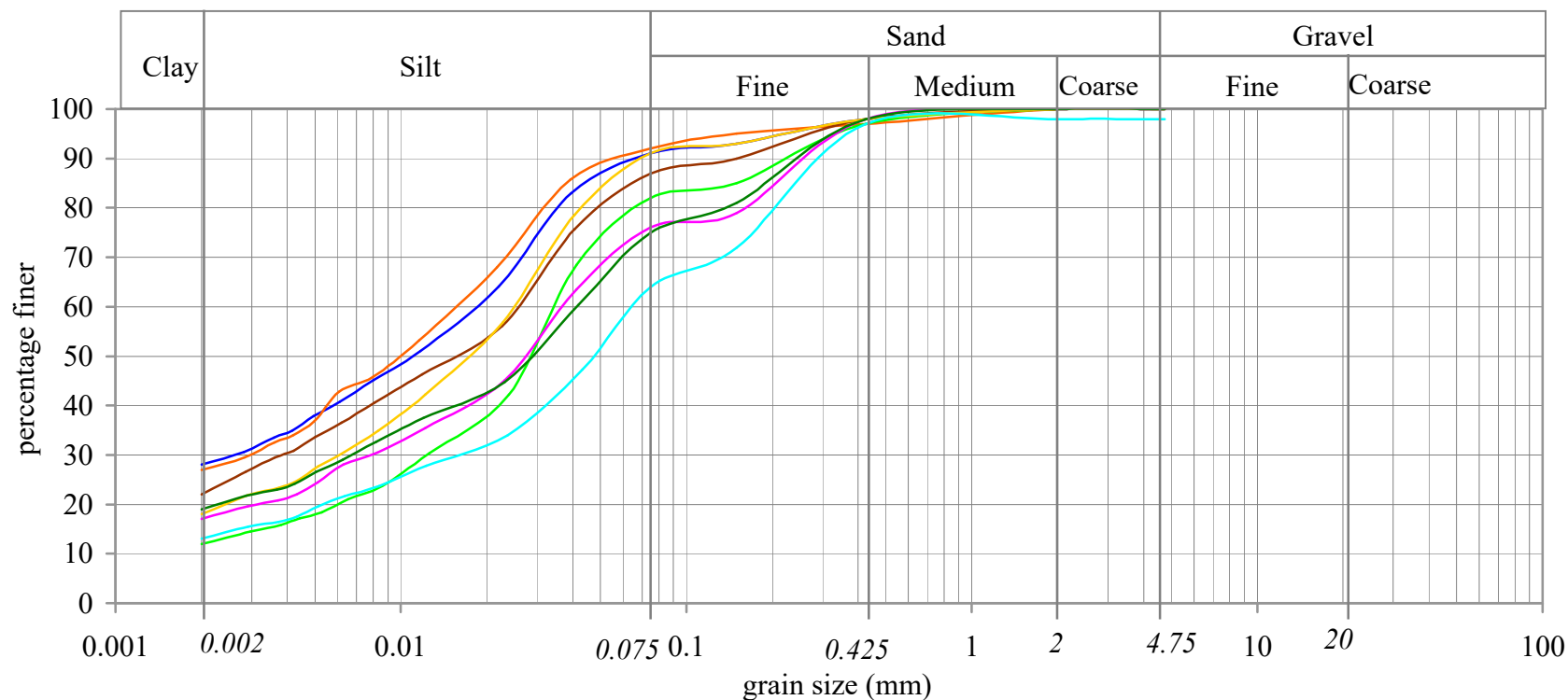
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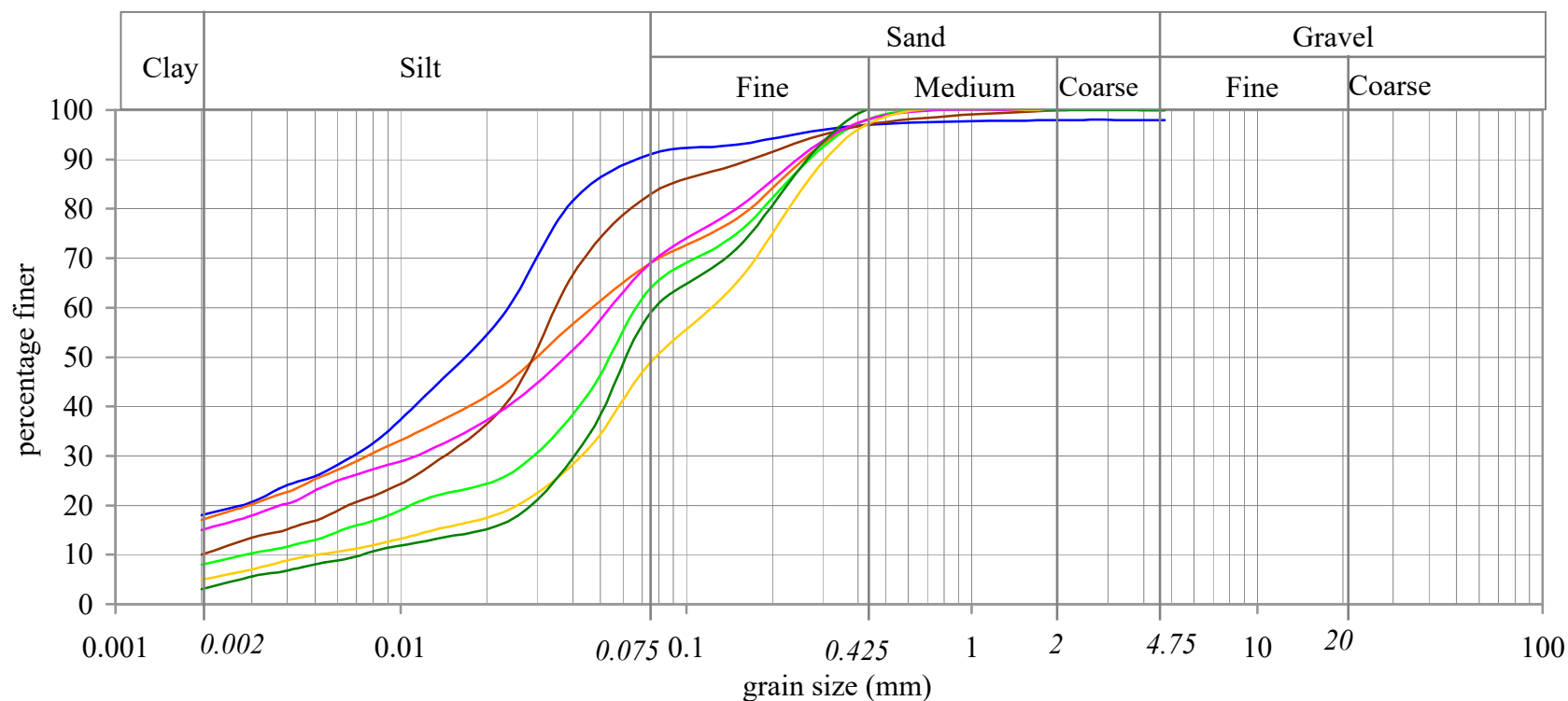
Job No. : G(D)4527
 Sheet No. : 95

Grain Size Analysis Curves (refer paragraph no. 7.3.1)



Line Style	TRIAL PIT	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
—	TP1	0.5	Sandy clayey silt	0	9	63	28	0.018	-	-
—	TP1	1	Sandy clayey silt	0	8	65	27	0.016	-	-
—	TP1	1.5	Clayey sandy silt	0	18	70	12	0.035	-	-
—	TP2	0.5	Sandy clayey silt	0	13	65	22	0.025	-	-
—	TP2	1	Clayey sandy silt	0	24	59	17	0.035	-	-
—	TP2	1.5	Sandy clayey silt	0	9	73	18	0.025	-	-
—	TP3	0.5	Clayey sandy silt	0	25	56	19	0.041	-	-
—	TP3	1	Clayey sandy silt with gravel	2	34	51	13	0.065	-	-

Grain Size Analysis Curves (*refer paragraph no. 7.3.1*)

[illegible]

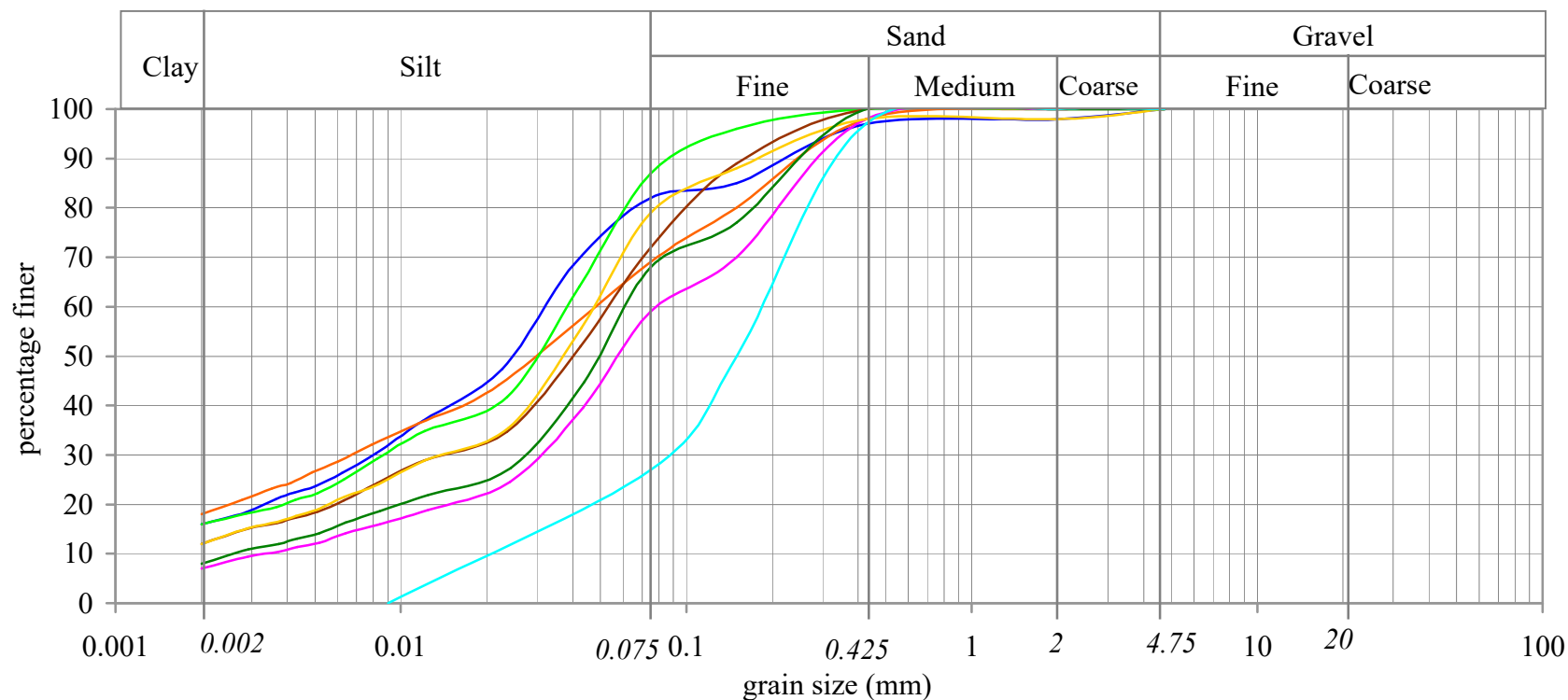


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Grain Size Analysis Curves (refer paragraph no. 7.3.1)

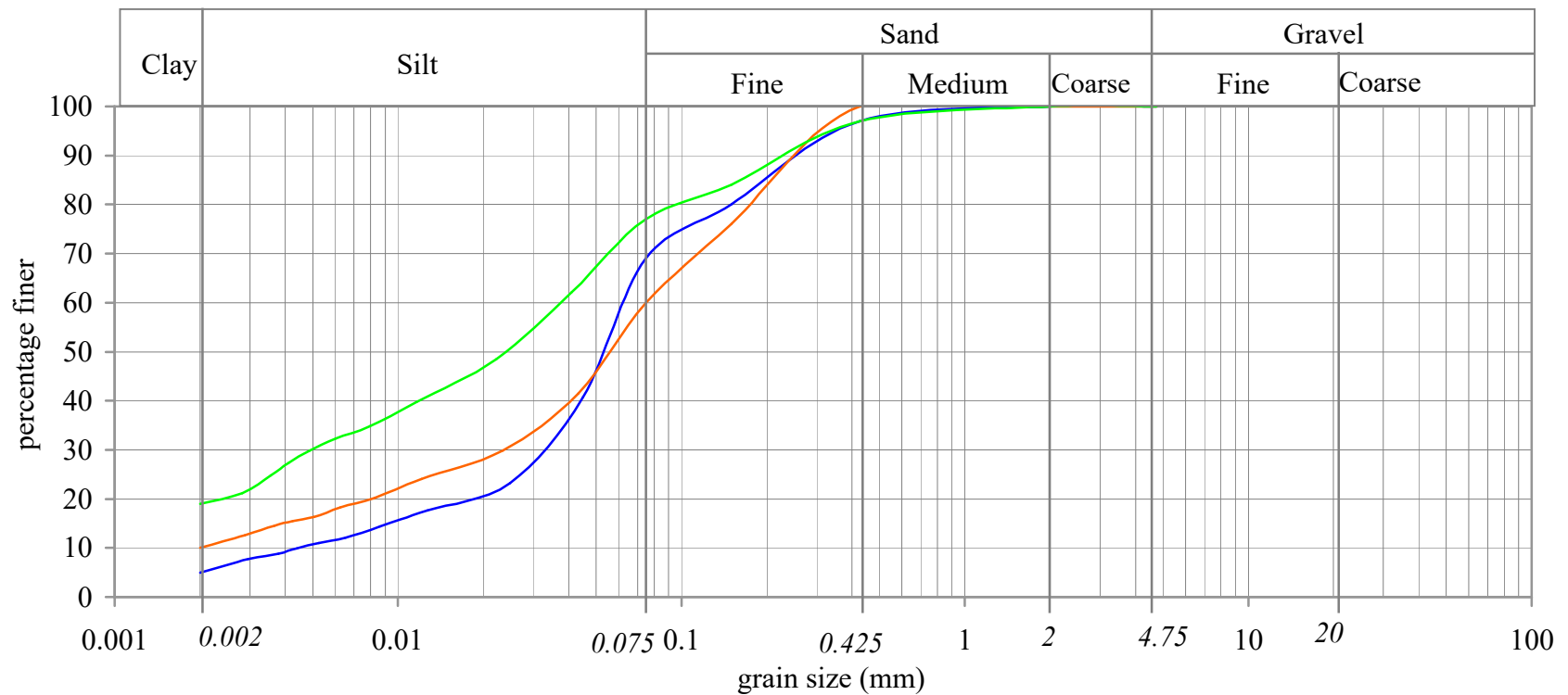


Line Style	CBR	Depth (m)	Description	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d ₆₀	d ₁₀	C _u
	CBR1		Clayey sandy silt	0	18	66	16	0.031	-	-
	CBR2		Clayey sandy silt	0	31	51	18	0.045	-	-
	CBR3		Sandy clayey silt	0	13	71	16	0.038	-	-
	CBR4		Clayey sandy silt	0	28	60	12	0.053	-	-
	CBR5		Clayey sandy silt	0	41	52	7	0.081	0.0028	28.9
	CBR6		Clayey sandy silt	0	21	67	12	0.048	-	-
	CBR7		Clayey sandy silt	0	32	60	8	0.061	0.0026	23.5
	CBR8		Silty sand	0	73	27	0	0.19	0.031	6.1

Job No. : G(D)4527

Sheet No. : 9u

Grain Size Analysis Curves (*refer paragraph no. 7.3.1*)

[illegible]

Analysis of Susceptibility to Liquefaction for Borehole BH1

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.000	0.999	0.234	4	1.70	1	1	0.75	1	5	97	5.00	1.200	11	0.122	0.52
2.4	0.36	1.314	0.998	0.307	8	1.52	1	1	0.8	1	10	78	5.00	1.200	17	0.181	0.59
3.9	0.36	1.519	0.997	0.354	2	1.31	1	1	0.85	1	2	98	5.00	1.200	7	0.088	0.25
5.4	0.36	1.630	0.996	0.380	5	1.17	1	1	0.95	1	6	92	5.00	1.200	12	0.131	0.35
6.9	0.36	1.700	0.995	0.396	9	1.06	1	1	0.95	1	9	85	5.00	1.200	16	0.170	0.43
8.4	0.36	1.747	0.994	0.406	9	0.98	1	1	0.95	1	8	32	4.83	1.171	14	0.150	0.37
9.9	0.36	1.781	0.910	0.379	18	0.92	1	1	1	1	17	23	4.06	1.100	23	0.257	0.68
11.4	0.36	1.808	0.870	0.368	23	0.87	1	1	1	1	20	7	0.12	1.009	20	0.215	0.59
12.9	0.36	1.828	0.830	0.355	24	0.82	1	1	1	1	20	4	0.00	1.000	20	0.215	0.61
14.4	0.36	1.845	0.790	0.341	16	0.78	1	1	1	1	13	3	0.00	1.000	13	0.141	0.41
15.9	0.36	1.859	0.749	0.326	11	0.75	1	1	1	1	8	72	5.00	1.200	15	0.160	0.49
17.4	0.36	1.870	0.709	0.310	9	0.72	1	1	1	1	6	85	5.00	1.200	12	0.131	0.42
20	0.36	1.886	0.640	0.282	8	0.67	1	1	1	1	5	95	5.00	1.200	11	0.122	0.43



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Job No. :
G(D)4527

Sheet No. : **C1**

Analysis of Susceptibility to Liquefaction for Borehole BH2

Depth (m)	$a_{\max}/g$	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
1.5	0.36	1.000	0.999	0.234	5	1.70	1	1	0.8	1	7	89	5.00	1.200	13	0.141	0.60
3	0.36	1.321	0.998	0.308	14	1.39	1	1	0.85	1	16	33	4.88	1.180	24	0.273	0.89
4.5	0.36	1.495	0.997	0.349	6	1.22	1	1	0.95	1	7	90	5.00	1.200	13	0.141	0.40
6	0.36	1.598	0.995	0.372	4	1.10	1	1	0.95	1	4	70	5.00	1.200	10	0.113	0.30
7.5	0.36	1.666	0.994	0.388	7	1.01	1	1	0.95	1	7	47	5.00	1.200	13	0.141	0.36
9	0.36	1.714	0.993	0.398	10	0.94	1	1	1	1	9	78	5.00	1.200	16	0.170	0.43
10.5	0.36	1.750	0.894	0.366	16	0.89	1	1	1	1	14	9	0.56	1.017	15	0.160	0.44
12	0.36	1.778	0.854	0.355	14	0.84	1	1	1	1	12	4	0.00	1.000	12	0.131	0.37
13.5	0.36	1.801	0.814	0.343	17	0.80	1	1	1	1	14	6	0.03	1.005	14	0.150	0.44
15	0.36	1.819	0.774	0.329	10	0.76	1	1	1	1	8	3	0.00	1.000	8	0.096	0.29
16.5	0.36	1.834	0.733	0.315	6	0.73	1	1	1	1	4	85	5.00	1.200	10	0.113	0.36
18	0.36	1.847	0.693	0.300	34	0.70	1	1	1	1	24	42	5.00	1.200	34	0.500	1.67



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Job No. :
G(D)4527

Sheet No. : **C2**

Analysis of Susceptibility to Liquefaction for Borehole BH3

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.000	0.999	0.234	10	1.70	1	1	0.75	1	13	79	5.00	1.200	21	0.228	0.98
2.4	0.36	1.182	0.998	0.276	11	1.50	1	1	0.8	1	13	74	5.00	1.200	21	0.228	0.83
3.9	0.36	1.412	0.997	0.329	3	1.27	1	1	0.85	1	3	98	5.00	1.200	9	0.104	0.32
5.4	0.36	1.541	0.996	0.359	7	1.14	1	1	0.95	1	8	92	5.00	1.200	15	0.160	0.45
6.9	0.36	1.623	0.995	0.378	7	1.04	1	1	0.95	1	7	67	5.00	1.200	13	0.141	0.37
8.4	0.36	1.681	0.994	0.391	9	0.97	1	1	0.95	1	8	62	5.00	1.200	15	0.160	0.41
9.9	0.36	1.723	0.910	0.367	11	0.91	1	1	1	1	10	6	0.03	1.005	10	0.113	0.31
11.4	0.36	1.755	0.870	0.357	15	0.85	1	1	1	1	13	5	0.00	1.000	13	0.141	0.39
12.9	0.36	1.781	0.830	0.346	12	0.81	1	1	1	1	10	3	0.00	1.000	10	0.113	0.33
14.4	0.36	1.802	0.790	0.333	14	0.77	1	1	1	1	11	7	0.12	1.009	11	0.122	0.37
15.9	0.36	1.819	0.749	0.319	3	0.74	1	1	1	1	2	88	5.00	1.200	7	0.088	0.27
17.4	0.36	1.833	0.709	0.304	21	0.71	1	1	1	1	15	79	5.00	1.200	23	0.257	0.84
20	0.36	1.853	0.640	0.278	30	0.67	1	1	1	1	20	61	5.00	1.200	29	0.410	1.48



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Job No. :
G(D)4527

Sheet No. : **C3**

Analysis of Susceptibility to Liquefaction for Borehole BH4

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
1.5	0.36	1.119	0.999	0.262	8	1.70	1	1	0.8	1	11	80	5.00	1.200	18	0.192	0.73
3	0.36	1.448	0.998	0.338	9	1.43	1	1	0.85	1	11	92	5.00	1.200	18	0.192	0.57
4.5	0.36	1.598	0.997	0.373	6	1.25	1	1	0.95	1	7	95	5.00	1.200	13	0.141	0.38
6	0.36	1.684	0.995	0.392	7	1.13	1	1	0.95	1	7	91	5.00	1.200	13	0.141	0.36
7.5	0.36	1.739	0.994	0.405	7	1.03	1	1	0.95	1	7	95	5.00	1.200	13	0.141	0.35
9	0.36	1.778	0.993	0.413	18	0.96	1	1	1	1	17	29	4.64	1.146	24	0.273	0.66
10.5	0.36	1.807	0.894	0.378	13	0.90	1	1	1	1	12	5	0.00	1.000	12	0.131	0.35
12	0.36	1.829	0.854	0.365	22	0.85	1	1	1	1	19	2	0.00	1.000	19	0.203	0.56
13.5	0.36	1.847	0.814	0.352	22	0.81	1	1	1	1	18	2	0.00	1.000	18	0.192	0.55
15	0.36	1.861	0.774	0.337	8	0.77	1	1	1	1	6	86	5.00	1.200	12	0.131	0.39
16.5	0.36	1.873	0.733	0.322	10	0.74	1	1	1	1	7	82	5.00	1.200	13	0.141	0.44
18	0.36	1.883	0.693	0.306	15	0.71	1	1	1	1	11	91	5.00	1.200	18	0.192	0.63



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Job No. :
G(D)4527

Sheet No. : **C4**

Analysis of Susceptibility to Liquefaction for Borehole BH5

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.000	0.999	0.234	7	1.70	1	1	0.75	1	9	63	5.00	1.200	16	0.170	0.73
2.4	0.36	1.245	0.998	0.291	8	1.50	1	1	0.8	1	10	60	5.00	1.200	17	0.181	0.62
3.9	0.36	1.464	0.997	0.341	4	1.29	1	1	0.85	1	4	95	5.00	1.200	10	0.113	0.33
5.4	0.36	1.584	0.996	0.369	8	1.15	1	1	0.95	1	9	80	5.00	1.200	16	0.170	0.46
6.9	0.36	1.661	0.995	0.387	7	1.05	1	1	0.95	1	7	94	5.00	1.200	13	0.141	0.36
8.4	0.36	1.713	0.994	0.398	14	0.97	1	1	0.95	1	13	65	5.00	1.200	21	0.228	0.57
9.9	0.36	1.752	0.910	0.373	16	0.91	1	1	1	1	15	3	0.00	1.000	15	0.160	0.43
11.4	0.36	1.781	0.870	0.362	8	0.86	1	1	1	1	7	3	0.00	1.000	7	0.088	0.24
12.9	0.36	1.804	0.830	0.350	10	0.82	1	1	1	1	8	3	0.00	1.000	8	0.096	0.27
14.4	0.36	1.823	0.790	0.337	24	0.78	1	1	1	1	19	2	0.00	1.000	19	0.203	0.60
15.9	0.36	1.838	0.749	0.322	6	0.74	1	1	1	1	4	75	5.00	1.200	10	0.113	0.35
17.4	0.36	1.851	0.709	0.307	16	0.72	1	1	1	1	11	24	4.18	1.108	16	0.170	0.55
20	0.36	1.870	0.640	0.280	35	0.67	1	1	1	1	24	7	0.12	1.009	24	0.273	0.98



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Job No. :
G(D)4527

Sheet No. : **C5**

Analysis of Susceptibility to Liquefaction for Borehole BH6

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	(N) ₆₀	FC (%)	α	β	(N) _{60CR}	CRR	FOS
1.5	0.36	1.382	0.999	0.323	6	1.70	1	1	0.8	1	8	85	5.00	1.200	15	0.160	0.50
3	0.36	1.640	0.998	0.383	13	1.50	1	1	0.85	1	17	71	5.00	1.200	25	0.292	0.76
4.5	0.36	1.746	0.997	0.407	4	1.30	1	1	0.95	1	5	96	5.00	1.200	11	0.122	0.30
6	0.36	1.804	0.995	0.420	10	1.16	1	1	0.95	1	11	85	5.00	1.200	18	0.192	0.46
7.5	0.36	1.840	0.994	0.428	10	1.06	1	1	0.95	1	10	95	5.00	1.200	17	0.181	0.42
9	0.36	1.865	0.993	0.433	11	0.98	1	1	1	1	11	96	5.00	1.200	18	0.192	0.44
10.5	0.36	1.883	0.894	0.394	17	0.91	1	1	1	1	16	70	5.00	1.200	24	0.273	0.69
12	0.36	1.897	0.854	0.379	20	0.86	1	1	1	1	17	11	1.21	1.026	19	0.203	0.54
13.5	0.36	1.908	0.814	0.363	19	0.82	1	1	1	1	16	4	0.00	1.000	16	0.170	0.47
15	0.36	1.917	0.774	0.347	15	0.78	1	1	1	1	12	4	0.00	1.000	12	0.131	0.38
16.5	0.36	1.924	0.733	0.330	17	0.75	1	1	1	1	13	84	5.00	1.200	21	0.228	0.69
18	0.36	1.930	0.693	0.313	17	0.72	1	1	1	1	12	83	5.00	1.200	19	0.203	0.65



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Job No. :
G(D)4527

Sheet No. : **C6**

Analysis of Susceptibility to Liquefaction for Borehole BH7

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.000	0.999	0.234	11	1.70	1	1	0.75	1	14	64	5.00	1.200	22	0.242	1.03
2.4	0.36	1.314	0.998	0.307	8	1.52	1	1	0.8	1	10	74	5.00	1.200	17	0.181	0.59
3.9	0.36	1.519	0.997	0.354	4	1.31	1	1	0.85	1	4	96	5.00	1.200	10	0.113	0.32
5.4	0.36	1.630	0.996	0.380	5	1.17	1	1	0.95	1	6	85	5.00	1.200	12	0.131	0.35
6.9	0.36	1.700	0.995	0.396	6	1.06	1	1	0.95	1	6	88	5.00	1.200	12	0.131	0.33
8.4	0.36	1.747	0.994	0.406	15	0.98	1	1	0.95	1	14	43	5.00	1.200	22	0.242	0.60
9.9	0.36	1.781	0.910	0.379	16	0.92	1	1	1	1	15	4	0.00	1.000	15	0.160	0.42
11.4	0.36	1.808	0.870	0.368	13	0.87	1	1	1	1	11	3	0.00	1.000	11	0.122	0.33
12.9	0.36	1.828	0.830	0.355	16	0.82	1	1	1	1	13	3	0.00	1.000	13	0.141	0.40
14.4	0.36	1.845	0.790	0.341	15	0.78	1	1	1	1	12	3	0.00	1.000	12	0.131	0.38
15.9	0.36	1.859	0.749	0.326	5	0.75	1	1	1	1	4	56	5.00	1.200	10	0.113	0.35
17.4	0.36	1.870	0.709	0.310	37	0.72	1	1	1	1	27	30	4.71	1.154	36	0.500	1.61
20	0.36	1.886	0.640	0.282	40	0.67	1	1	1	1	27	4	0.00	1.000	27	0.338	1.20



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Job No. :
G(D)4527

Sheet No. : **C7**

Analysis of Susceptibility to Liquefaction for Borehole BH8

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
1.5	0.36	1.037	0.999	0.242	6	1.70	1	1	0.8	1	8	79	5.00	1.200	15	0.160	0.66
3	0.36	1.382	0.998	0.323	11	1.41	1	1	0.85	1	13	75	5.00	1.200	21	0.228	0.71
4.5	0.36	1.545	0.997	0.360	4	1.24	1	1	0.95	1	5	96	5.00	1.200	11	0.122	0.34
6	0.36	1.640	0.995	0.382	5	1.12	1	1	0.95	1	5	81	5.00	1.200	11	0.122	0.32
7.5	0.36	1.702	0.994	0.396	8	1.02	1	1	0.95	1	8	80	5.00	1.200	15	0.160	0.40
9	0.36	1.746	0.993	0.406	14	0.95	1	1	1	1	13	91	5.00	1.200	21	0.228	0.56
10.5	0.36	1.778	0.894	0.372	14	0.89	1	1	1	1	13	5	0.00	1.000	13	0.141	0.38
12	0.36	1.804	0.854	0.360	23	0.84	1	1	1	1	19	6	0.03	1.005	19	0.203	0.56
13.5	0.36	1.824	0.814	0.347	14	0.80	1	1	1	1	11	3	0.00	1.000	11	0.122	0.35
15	0.36	1.840	0.774	0.333	2	0.77	1	1	1	1	2	83	5.00	1.200	7	0.088	0.26
16.5	0.36	1.854	0.733	0.318	8	0.73	1	1	1	1	6	86	5.00	1.200	12	0.131	0.41
18	0.36	1.865	0.693	0.303	17	0.71	1	1	1	1	12	15	2.50	1.048	15	0.160	0.53



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Job No. :
G(D)4527

Sheet No. : **C8**

Analysis of Susceptibility to Liquefaction for Borehole BH9

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.000	0.999	0.234	7	1.70	1	1	0.75	1	9	44	5.00	1.200	16	0.170	0.73
2.4	0.36	1.351	0.998	0.316	4	1.53	1	1	0.8	1	5	83	5.00	1.200	11	0.122	0.39
3.9	0.36	1.549	0.997	0.361	2	1.32	1	1	0.85	1	2	96	5.00	1.200	7	0.088	0.24
5.4	0.36	1.654	0.996	0.385	4	1.17	1	1	0.95	1	4	91	5.00	1.200	10	0.113	0.29
6.9	0.36	1.720	0.995	0.400	6	1.07	1	1	0.95	1	6	86	5.00	1.200	12	0.131	0.33
8.4	0.36	1.764	0.994	0.410	12	0.99	1	1	0.95	1	11	96	5.00	1.200	18	0.192	0.47
9.9	0.36	1.797	0.910	0.382	6	0.92	1	1	1	1	6	97	5.00	1.200	12	0.131	0.34
11.4	0.36	1.821	0.870	0.371	16	0.87	1	1	1	1	14	41	5.00	1.200	22	0.242	0.65
12.9	0.36	1.841	0.830	0.357	23	0.82	1	1	1	1	19	7	0.12	1.009	19	0.203	0.57
14.4	0.36	1.856	0.790	0.343	18	0.78	1	1	1	1	14	2	0.00	1.000	14	0.150	0.44
15.9	0.36	1.869	0.749	0.328	11	0.75	1	1	1	1	8	87	5.00	1.200	15	0.160	0.49
17.4	0.36	1.879	0.709	0.312	14	0.72	1	1	1	1	10	78	5.00	1.200	17	0.181	0.58
20	0.36	1.894	0.640	0.284	24	0.68	1	1	1	1	16	61	5.00	1.200	24	0.273	0.96



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Job No. :
G(D)4527

Sheet No. : **C9**

Analysis of Susceptibility to Liquefaction for Borehole BH10

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	(N) ₆₀	FC (%)	α	β	(N) _{60CR}	CRR	FOS
1.5	0.36	1.000	0.999	0.234	9	1.70	1	1	0.8	1	12	73	5.00	1.200	19	0.203	0.87
3	0.36	1.351	0.998	0.315	3	1.40	1	1	0.85	1	4	91	5.00	1.200	10	0.113	0.36
4.5	0.36	1.519	0.997	0.354	7	1.23	1	1	0.95	1	8	97	5.00	1.200	15	0.160	0.45
6	0.36	1.619	0.995	0.377	8	1.11	1	1	0.95	1	8	95	5.00	1.200	15	0.160	0.42
7.5	0.36	1.684	0.994	0.392	13	1.02	1	1	0.95	1	13	91	5.00	1.200	21	0.228	0.58
9	0.36	1.730	0.993	0.402	17	0.95	1	1	1	1	16	63	5.00	1.200	24	0.273	0.68
10.5	0.36	1.764	0.894	0.369	13	0.89	1	1	1	1	12	3	0.00	1.000	12	0.131	0.36
12	0.36	1.791	0.854	0.358	19	0.84	1	1	1	1	16	2	0.00	1.000	16	0.170	0.48
13.5	0.36	1.812	0.814	0.345	20	0.80	1	1	1	1	16	3	0.00	1.000	16	0.170	0.49
15	0.36	1.829	0.774	0.331	15	0.76	1	1	1	1	11	5	0.00	1.000	11	0.122	0.37
16.5	0.36	1.844	0.733	0.316	14	0.73	1	1	1	1	10	86	5.00	1.200	17	0.181	0.57
18	0.36	1.856	0.693	0.301	29	0.70	1	1	1	1	20	52	5.00	1.200	29	0.410	1.36



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Job No. :
G(D)4527

Sheet No. : **C10**

Analysis of Susceptibility to Liquefaction for Borehole BH11

Depth (m)	a_{max}/g	σ_{vo}/σ'_{vo}	r_d	CSR	N_{obs}	C_N	C_E	C_B	C_R	C_S	$(N)_{60}$	FC (%)	α	β	$(N)_{60CR}$	CRR	FOS
0.9	0.36	1.302	0.999	0.304	7	1.70	1	1	0.75	1	9	71	5.00	1.200	16	0.170	0.56
2.4	0.36	1.673	0.998	0.391	4	1.65	1	1	0.8	1	5	93	5.00	1.200	11	0.122	0.31
3.9	0.36	1.786	0.997	0.417	4	1.39	1	1	0.85	1	5	91	5.00	1.200	11	0.122	0.29
5.4	0.36	1.841	0.996	0.429	7	1.22	1	1	0.95	1	8	95	5.00	1.200	15	0.160	0.37
6.9	0.36	1.874	0.995	0.436	8	1.11	1	1	0.95	1	8	88	5.00	1.200	15	0.160	0.37
8.4	0.36	1.895	0.994	0.441	14	1.02	1	1	0.95	1	14	64	5.00	1.200	22	0.242	0.55
9.9	0.36	1.910	0.910	0.407	23	0.95	1	1	1	1	22	4	0.00	1.000	22	0.242	0.60
11.4	0.36	1.922	0.870	0.391	17	0.89	1	1	1	1	15	2	0.00	1.000	15	0.160	0.41
12.9	0.36	1.931	0.830	0.375	23	0.84	1	1	1	1	19	5	0.00	1.000	19	0.203	0.54
14.4	0.36	1.938	0.790	0.358	18	0.80	1	1	1	1	14	2	0.00	1.000	14	0.150	0.42
15.9	0.36	1.943	0.749	0.341	9	0.76	1	1	1	1	7	91	5.00	1.200	13	0.141	0.41
17.4	0.36	1.948	0.709	0.323	14	0.73	1	1	1	1	10	76	5.00	1.200	17	0.181	0.56
20	0.36	1.955	0.640	0.293	7	0.69	1	1	1	1	5	71	5.00	1.200	11	0.122	0.42



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Job No. :
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