THE USE OF CONE PENETRATION TESTING TO INVESTIGATE SAND FILL SUBSIDENCE AT CROSS-DRAIN LOCATIONS

FINAL REPORT

VOLUME I

BY

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Research Report No. 170

Research Project No. 83-3S

Conducted By

GEOGULF, INC.
(Subsidiary of Fugro Inter, Inc.)

AND

LOUISIANA DEPARTMENT OF TRANSPORTATION
AND DEVELOPMENT
Research and Development Section
In Cooperation With
U. S. Department of Transportation
FEDERAL HIGHWAY ADMINISTRATION

MARCH 1984

DISCLAIMER

ACKNOWLEDGEMENT

The field cone penetrometer testing, analysis of test data and preparation of report was performed by Messrs Recep Yilmaz and Bert Lakeman of Geogulf, Inc. Mr. James L. Melancon of the Louisiana D.O.T.D. provided the liaison between the Louisiana D.O.T.D. and Geogulf and supervision of test site selection and preparation and the Department's laboratory and field testing programs. The efforts of Mr. Steve Bokun and other personnel of Louisiana D.O.T.D. is also acknowledged and appreciated.

METRIC CONVERSION FACTORS*

To Convert from	To	Multiply by
	Length	
foot	meter (m)	0.3048
inch	millimeter (mm)	25.4
yard	meter (m)	0.9144
mile (statute)	kilometer (km)	1.609
	Area	
square foot	square meter (m^2)	0.0929
square inch	square centimeter (cm^2)	6.451
square yard	square meter (m^2)	0.8361
	Volume (Capacity)	
cubic foot	cubic meter (m^3)	0.02832
gallon (U.S. liquid)**	cubic meter (m^3)	0.003785
gallon (Can. liquid)**	cubic meter (m^3)	0.004546
ounce (U.S. liquid)	cubic centimeter (cm^3)	29.57
	Mass	
ounce-mass (avdp)	gram (g)	28.35
pound-mass (avdp)	kilogram (kg)	0.4536
ton (metric)	kilogram (kg)	1000
ton (short, 2000 lbs)	kilogram (kg)	907.2
	Mass per Volume	
pound-mass/cubic foot	kilogram/cubic meter (kg/m^3)	16.02
pound-mass/cubic yard	kilogram/cubic meter (kg/m^3)	0.5933
pound-mass/gallon (U.S.)**	kilogram/cubic meter (kg/m^3)	119.8
pound-mass/gallon (Can.)**	kilogram/cubic meter (kg/m^3)	99.78
	Pressure	
pounds per square inch	kilograms per square centimeters (kg/cm^2)	0.7030
pounds per square inch	mega pascal (MPa)	0.006894

*The reference source for information on SI units and more exact conversion factors is "Metric Practice Guide" ASTM E 380.

**One U.S. gallon equals 0.8327 Canadian gallon.

ABSTRACT

During construction of Interstate I-10 between Baton Rouge and LaPlace, Louisiana, highly organic swamp deposits were excavated and replaced with hydraulically pumped river sand. Recently, excessive settlement was encountered at numerous cross-drain and bridge end locations within the hydraulically placed sand fills.

The main objective of this study was to evaluate the subsurface conditions and investigate the reason(s) for sand fill subsidence at cross-drain locations and to determine the possibility of utilizing the electric cone penetration test for such investigation. Sounding data was collected at sites both with and without cross-drains in a grid pattern to a minimum depth of two feet in the Pleistocene clay beneath the hydraulically placed sand fill. All soundings were evaluated within each site location first, and subsequently compared with the representative average values from the other sites.

Numerous conclusions based on the study are presented although the major evidence for subsidence is loosely compacted sand at the cross-drain locations. It was also noted that the electric cone penetration test is a valuable tool and should be utilized both immediately after construction, as well as in checking time related effects in subsidence and compaction studies.

THE USE OF CONE PENETRATION TESTING
TO INVESTIGATE SAND FILL SUBSIDENCE
AT CROSS-DRAIN LOCATIONS

VOLUME 1

CONTENTS

INTRODUCTION

Four sites located between Baton Rouge and LaPlace on East-bound I-10 were tested by the electric cone penetration method (Table 1 and Plate 1).

SITE	STATION	MILES EAST FROM ST. JAMES AND ST. JOHN PARISH LINE
2A	1953+00	2.20
2	1961+00	2.35
1	1991+00	2.92
1A	1995+50	3.01

TABLE 1

Sites 1 and 2 are at cross-drain locations which contain two 63" x 43" asbestos bonded corrugated metal pipe arches.

As stated in State Project No. 700-04-57, the site originally consisted of very soft and compressible swamp deposits from the surface to a depth of 16 to 20 ft. The swamp deposits are underlain by a zone of weathered Pleistocene materials on the order of 3 to 5 ft. thick. The weathered zone overlies silty, firm clays of Pleistocene age.

The highly organic swamp soils were totally excavated and the area was backfilled with hydraulically pumped river sand. This sand fill is now the foundation of route I-10.

Purpose and Scope of Investigation

Excessive settlement occurs at numerous cross drain and bridge end locations within hydraulically placed sand fills along route I-10.

The scope of this study included evaluation of the subsurface conditions at the sites and investigation of the reason(s) for sand fill subsidence. Future recommendations are based on field exploration and engineering analyses.

This investigation consisted of electric cone penetration soundings at four selected sites on East-bound I-10 in order to analyze the sand fill subsidence at cross-drain locations. Two sites were selected at cross-drain locations; two were chosen at sites without cross-drains. Sounding data were collected at each site in a grid pattern to a minimum depth of 2 feet into the Pleistocene clay.

FIELD INVESTIGATION

In-situ exploration data were obtained by electric cone penetration soundings (CPT). The CPT method for evaluating stratigraphy and obtaining engineering soil properties has the advantage of providing more detailed, precise and continuous data in comparison with other in-situ exploration methods. The following is a description of the equipment and field activities.

Electric Cone Penetrometer

The CPT system is housed in an environmentally controlled cabin which is mounted on a four-wheel drive truck. Subsurface information is obtained by hydraulically pushing the penetrometer, a cone shaped instrument, into the soil. The standard penetrometer has a 60° apex angle, a 10 cm^2 (1.55 sq. in.) cone with a straight cylindrical shaft (150 cm^2) above the cone having the same diameter as the cone. With the use of a hydraulically operated system, the penetrometer is able to penetrate the ground at a constant rate of 2 cm/sec (0.8 in/sec.). During penetration, continuous measurements of tip resistance (q_c) and side-friction (f_s) are taken by strain-gauge load cells located inside the penetrometer. This information is recorded on a strip chart directly. Also, all field data are recorded in digitized form on tapes.

The results of the Cone Penetration Tests can be used to obtain the following geotechnical soil informations:

1. Stratification of the subsoil: The continuous measurements of both tip resistance and friction resistance along the sleeve provides a very accurate profile of the soil stratification.

2. Bearing Capacity of soils (soft, loose versus hard, dense): Layers with different bearing capacities are reflected in changes in tip resistance and can easily be detected from the CPT graph.

3. Type of soil in each stratum: The ratio of the sleeve friction to the tip resistance provides an accurate means to identify the soil types penetrated. As shown on Plate 20, a soil classification chart for the standard electric friction cone after Douglas and Olsen (1981), is presented. Also, a modified soil classification chart by Tumay and Chan (1983), is presented on Plate 23.

4. Density and angle of internal friction of granular soils: As a guide, to convert the tip resistance to sand properties, the following table (Table 2) given by Meyerhof (1965) can be used.

COMPACTNESS OF FINE SAND	RELATIVE DENSITY (Dr)	SPT (N)	CONE TIP RESISTANCE (q_c in tsf)	ANGLE OF INTERNAL FRICTION (ϕ, in degrees)
Very Loose	< 0.2	< 4	< 21	< 30
Loose	0.2 - 0.4	4 - 10	21 - 42	30 - 35
Medium Dense	0.4 - 0.6	10 - 30	42 - 125	35 - 40
Dense	0.6 - 0.8	30 - 50	125 - 209	40 - 45
Very Dense	0.8 - 1.0	> 50	> 209	> 45

TABLE 2

Based on the above values, in general, the first 3 meters can be classified as very dense to dense sand. From 3 to 3.5 meters, the sand is medium dense, and below 3.5 meters, loose sand is present. For comparison of relative density, Dr, and angle of internal friction, ϕ, Locations 1 and 1A are chosen. Average cone results at cross-sections AA and BB are used to determine Dr and based on work done by Schmertmann (1978). Results are presented on Plates 21 and 22 for Locations 1 and 1A, respectively. As stated by Robertson and Campanella (1982), it should be kept in mind that the estimation of sand properties by cone results is empirical. Also, it should be noted that the relationship between tip resistance and sand properties given by Schmertmann (1978) and others is for saturated, uncemented, and normally consolidated sands.

5. Shear strength of cohesive soils: In general, the undrained shear strength, C_u of cohesive soils can be estimated with the following expression:

$$q_c = C_u . N_k$$

where: q_c = tip resistance

C_u = undrained shear strength

N_k = cone factor, depending on type of soil, stress history and stress-strain relation of soil

For normally consolidated clays in South Louisiana, a cone factor of $N_k = 19$ has proven to give reasonably accurate results corresponding with unconfined compression tests (see Tumay and Yilmaz, 1981). Estimated undrained shear strength values are also shown on Plate 21 and 22.

Detailed descriptions of equipment and applications of cone penetration tests are given by Schmertmann (1978) and de Ruiter (1982). All tests were performed in accordance with ASTM D-3441-75T: "Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests".

In-Situ Testing and Data Collection

Prior to site study, approximately 4 inch diameter holes were cored through the continuously reinforced concrete pavement by Louisiana Department of Transportation and Development (DOTD). As seen on Plate 2, a grid pattern is constructed with cored holes at each selected test site. Per site, 27 holes were cored through the pavement and 20 tests were performed from the shoulders of the road on both sides of the pavement. In order to reduce the complexity, the location of soundings is numbered in the same manner for all sites.

Under the supervision of DOTD's field crew, the cone truck was driven to the center of the hole and the penetrometer was pushed hydraulically through the sand fill into the underlying Pleistocene clay to a minimum depth of 2 feet. During testing, measurements were recorded on a strip chart for on-site visual information. From this chart, it was possible to decide quickly the completion depth of sounding.

A daily average of 22 soundings was carried out during the in-situ testing plan. In this way, it was possible to complete all 47 of cone soundings in a couple of days at each site.

GENERAL SOIL CONDITIONS

Soil conditions and geological soil formations along route I-10 have been investigated by numerous authors, most notably Walters, et. al. (1964), Moore, Walters and Lasserre (1965), Arman (1969), Arman and Munfakh (1973). References directly related to this investigation are listed in State Project No. 700-04-57 and Kinchen and Melancon (1973).

ANALYSIS OF RESULTS

During in-situ testing, 188 soundings with a standard cone penetrometer were conducted. For engineering interpretation, in-house data reduction and processing methods were applied to the digital CPT data. With the aid of the moving average method, friction ratios were calculated for every corresponding sounding. Computer plots of these results are presented in Volume II. The results are drawn with the existing top of pavement or groundlevel as zero level.

Discussion of Test Results

In general, the subsoil conditions at the test sites can be described as follows:

- Underneath the pavement, the hydraulically placed sand fill extends to a depth of approximately 6.5 to 7.5 meters with a friction ratio of 1%.

- Beneath the sand layer, there exists a Pleistocene clay with a friction ratio of 4-6%.

- Locally, soft layers of organic clay with friction ratio of 6-10% (see CPT Number 15/1), and pieces of wood with high tip resistance, friction and friction ratio (see CPT Number 28/1A) were encountered throughout the site. Also, several sounding logs present existence of silty sand, particularly within the upper sand layer with friction ratio of 2-4% (see CPT Number 29/1A at depth of 6.7 to 7.1 meters).

Location 1

A preliminary examination of individual soundings at Location 1 suggests a noticeable difference in CPT results between the locations around the drainage pipes and those further away from the pipes. For this reason, the sounding results at the nearby drainage pipes (on lines II, HH, BB on Plate 2) are compared to those away from the pipe locations, such as in Line AA (Plate 2). CPT's of cross-sections AA, BB, HH and II are drawn on top of one another, as shown in the Appendix (Plates A-1 through A-4), in order to see the profile scatter. Average results are indicated on each plate by a darker line.

Plate 3 indicates that the average of cone results in the top 3m (10 ft.) at cross-sections BB, HH and II are much less than at the location of AA. However, over the same depth interval, similar results in friction ratios are observed. The friction ratio suggests that the soil type throughout the area is sand. One can conclude that the sand around the pipes is loose in nature. At a depth slightly below 3 meters (plate 3), the situation is reversed. AA results are slightly less dense than those of the cross-sections BB, HH and II. This indicates further densification of the sand fill immediately below the pipes. That densification is possibly due to the weight of the pipes, the vibratory effect of piles and/or operations of bulldozers prior to and during placement of drainage pipes in the trench. This effect is pronounceable to a depth of 4.5 to 5 meters. In deeper layers, sand density and clay stiffness are very similar on every cross-section. At approximately 7.5 meters, a transition from sand to clayey soil occurs. The top of the Pleistocene clay starts at about 8 meters. Plate 4 depicts the above conditions and the existence of mucky material with variable thickness on top of the clay.

Cross-section GG is located on the outer shoulder of Location 1 (see Plate 2) and is shown on Plate 5. In the center of the cross-section (CPT's 41 through 43) the tip resistance in the upper 3m is very low as compared to either side. Possible because of the weight of the overlying pipes, the tip resistance of CPT 42 is higher than the tip resistance registered in CPT 41 and CPT 43.

Location 1A

Plate 6 presents the CPT locations and representative elevations of the roadway cross-section at Location 1A. On Plate A-5 through A-11 in the Appendix, soundings along the lines AA, BB, CC, DD, EE, FF and GG are drawn on top of each other with adjusted elevations. Preliminary examination of the results showed that the averages of the cross-sections are symmetrical relative to the center of the roadway, EE, with minor differences. For that reason, only cross-sections from the center line EE to the right shoulder GG are considered on Plate 7. As seen on this plate, the average tip resistances are the highest at the center EE and gradually decrease toward the outer berm (CPT 47) for the first 1.5 meters in depth. Below 1.5 meters, results are similar with minor differences which reflect uniform soil densification and type. CPT 47 (Plate 7) is different than the rest of the cross-sections within the upper 1.5 meters. This is due to the existence of top soil (silty clay) on top of the sand fill along the berms. In the topsoil, the tip resistance is relatively low, and the high friction ratio of 4-6% indicates the amount of fine silt and clay particles in the soil. Cross-section BB on Plate 8 shows top soil on both berms. Also, higher sand densities are pointed out under the pavement, which is expected due to traffic loads and vibrations. The maximum influence of the traffic loads and weight of the pavement appears to extend to a depth of approximately 1.5 meters.

Location 2

The soundings lay-out plan for Location 2 is shown on Plate 9. This site is basically the same as Location 1. For that reason, the analysis is identical to that of Location 1. The CPT results along cross-sections and their averages are depicted in Plates A-12 to A-15 in the Appendix.

On these plates, cone results largely fluctuate within the first 1.5 meters. Because of density differences across the roadway, this fluctuation is expected. Under the traffic load and weight of the structure, the pavement can be assumed as a loaded strip footing. In this case, a pressure bulb forms across the pavement in such a way that the pressure distribution increases toward the center of the loaded area. Therefore, sounding results across the roadway reflect density fluctuations (see the top 1.5 meters sounding results on Plates A-12 to A-15).

Plate 10 demonstrates the comparison of averages at cross-section AA, BB, HH and II for Location 2. As in the case of Location 1, this plate also reflects the influence zone of drainage pipes. The location of cross-drains, their influence zone and the existence of soft material are shown in cross-section BB and GG in Plates 11 and 12.

Location 2A

A CPT lay-out plan is drawn on Plate 13 for Location 2A. CPT results shown in Plates A-16 through A-22 in the Appendix and Plates 14 and 15 demonstrate the same characteristics of those at location 1A. As pointed out earlier, Plate A-17 and A-18 are good examples of density comparisons across and along the roadway. It can be concluded that densities along the roadway cross-section are constant.

COMPARISON OF LOCATIONS

Up to this point, sounding results have been examined for each site. In the following section, representative results from each site will be compared to other sites. The comparison process will be done first between similar sites and then will extend between good to problem sites.

Average sounding results from Locations 1 and 2 are drawn on Plate 16. Basically, the results from both sites are similar except that the top of Pleistocene clay is at a shallower depth at Location 2. Even though there is a large distance between the two locations, sand densities and drainage pipe influences are the same.

Findings of Locations 1A and 2A are presented on Plate 17. In spite of the trend similarities, 50 to 100 kg/cm^2 differences in tip resistance exist between the sites. Many factors, such as gradation difference in sand, stiffness and type differences in underlying clay layers might cause these variations.

Illustrations on Plates 18 and 19 clearly demonstrate the difference between sites with and without cross-drains. In summary, cone penetration results on the four sites demonstrate a similar pattern in densities in the sand fill layer, except at locations where cross-drains exist. Lower densities at both cross-drain areas are similar.

CONCLUSIONS AND RECOMMENDATIONS

Four selected sites were investigated based on 188 individual cone soundings. Sites were first examined independently then comparisons were extended to include all sites. Conclusions are summarized as follows:

1. Soft, compressible mucky material exists in varying thickness on top of the natural, Pleistocene clay and within hydraulically placed sand.

2. Density profiles on cross-sections along the roadway are very similar; however, they are different in magnitude crosswise. Densities are the greatest at the center of the road and decrease toward the shoulders with a symmetrical pattern. These fluctuations are within a depth of 1.5 meters from the surface, then the profiles become constant for the entire site.

3. Sand around drainage pipes is in a loose to very loose state in the zone between depths of 0.5 to 3 meters. The possible reasons for low tip resistances (loose conditions) are:

   - bad quality of backfilling/compaction

   - loosening effects of traffic vibration transported through rigid pipes to sand.

   - deformations of pipes under traffic load would also cause loosening of the sand around the pipes.

4. Settlement and differential settlement at the investigated sites are caused by:

   - Settlement in soft organic mucky material under the present loading conditions. For example, settlements in a layer of soft material with a thickness of 3 feet can be calculated at 8-15 inches. We estimate 95-98 percent of this type of settlement has already occurred. The variation in thickness of the soft material is a cause of differential settlement at all locations.

   - Settlement in underlying stiff Pleistocene clay. Differential setttlement for this case should be minor.

- Compaction of the sandfill along any part of the roadway, mainly in the top layers, due to traffic loads and weight of the pavement. Due to the lack of data immediately following construction, it is not possible to compare present in-situ conditions of the sandfill with initial conditions and, therefore, the order of magnitude of settlement and differential settlement. Results of CPT's made in the shoulders of the road indicate that the top sand layers were probably already dense prior to the installation of the pavement and thus settlements of the sand fill outside the cross-drain locations will have been relatively small. Although the sand around the cross-drains has shown to be in a very loose to loose state at present, it is to be expected that densification of the sand has resulted in larger settlements here than outside the cross-drain areas.

Due to differential settlement under the continuously reinforced pavement, it will behave as a bridge. The length of the bridging effect will depend on the above factors. It is just a matter of time, under traffic load and vibrations before the pavement will fail at bridging locations.

5. Based on the cone results, there is no evidence of failure planes and horizontal movement in the road embankment. Even on the shoulders and berms, sand reflects high density in the upper 3-3.5 meters.

It is clearly shown during this investigation that cone penetration testing is a superior method over other in-situ testing methods for the following reasons:

- the cone provides continuous data during profiling,
- it detects seams and soft layers,
- it can distinguish density differences,
- it can determine geotechnical soil properties, and
- it provides a quick and economical soil investigation.

Because of the above reasons, this method is extensively used for compaction control.

Based on our previous experience and this study, it is strongly recommended that this type of investigation should be carried out during compaction control and immediately following construction in order to detect the presence of any remaining soft material and to determine in-situ density. Also, following a certain period of time, testing should be carried out in order to find out the effects of traffic and vibration on the density of the sandfill, both within and outside cross-drain locations.

Based upon the results of the CPT's and our previous experiences, it is our feeling that the hydraulically placed sand outside cross-drain locations has reached a final densification and settlement state. Further settlement will most likely not occur. However, if a gap between the sand and pavement has already formed and the pavement acts as a bridge, then these areas, depending on the size of the gap, might fail in the future.

At cross-drain locations, sand is in a loose state. It might be expected to further densify which will produce additional settlement on the pavement. During construction, it is necessary to backfill these locations with compacted lifts. The decision of lift thickness and compaction effort should be based on available equipment. Also, backfilling should be carried out in such a way that densification would be uniform along the roadway embankment.

To prevent future subsidence at cross-drain locations, there are two options:

1. Excavation of the loose sand, backfilling in lifts and adequate compaction of each lift. As stated earlier in this chapter, loosening of the sand might occur after compaction due to traffic vibrations or possible deformations of the pipes. Therefore, this solution is not recommended.

2. Stabilization of the loose sands around the pipes in such a way that future densification or loosening of the sand is prevented. We recommend grouting of the area where loose sands are present without significantly increasing the weight of the grouted area, in order to prevent additional settlement.

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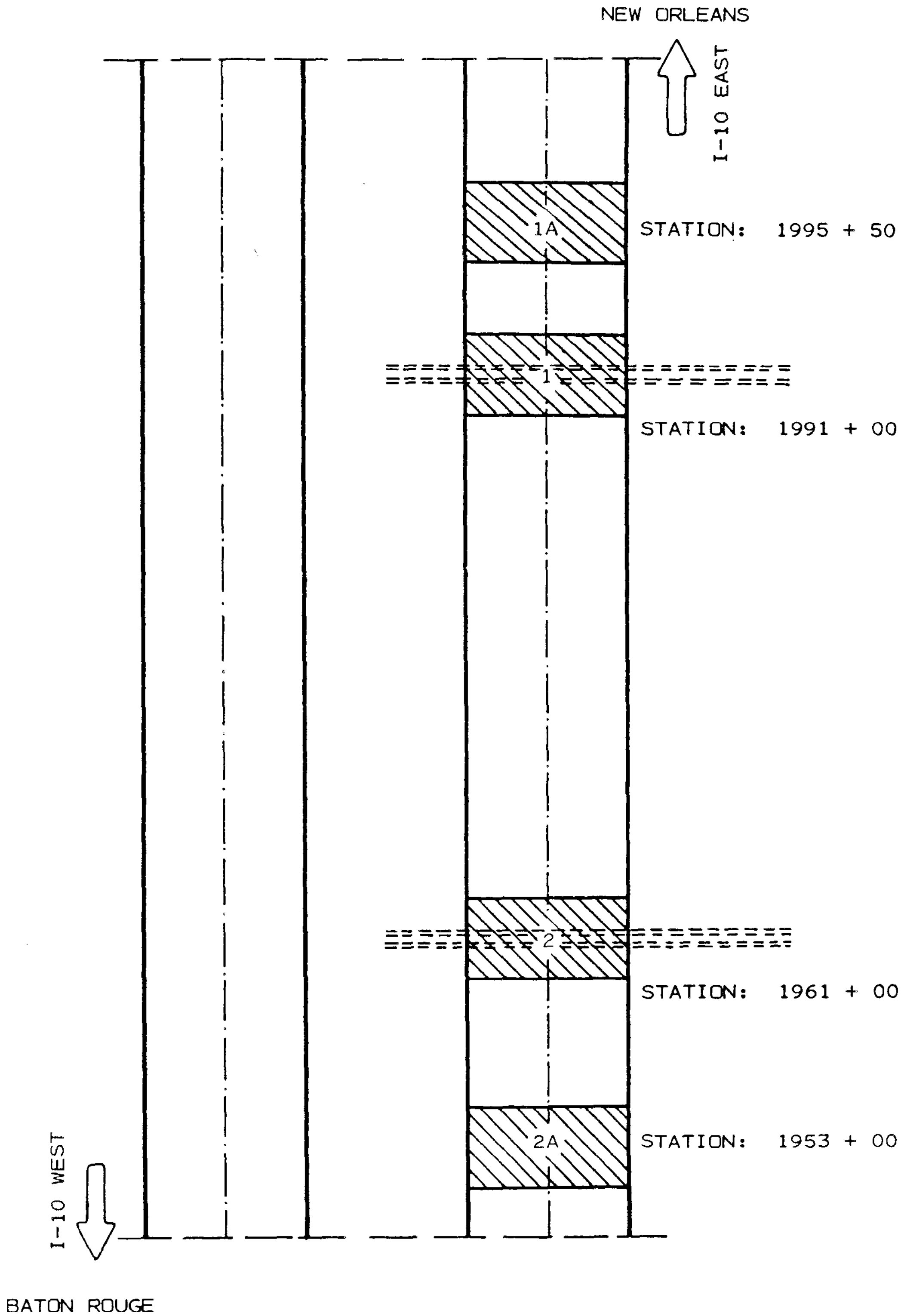
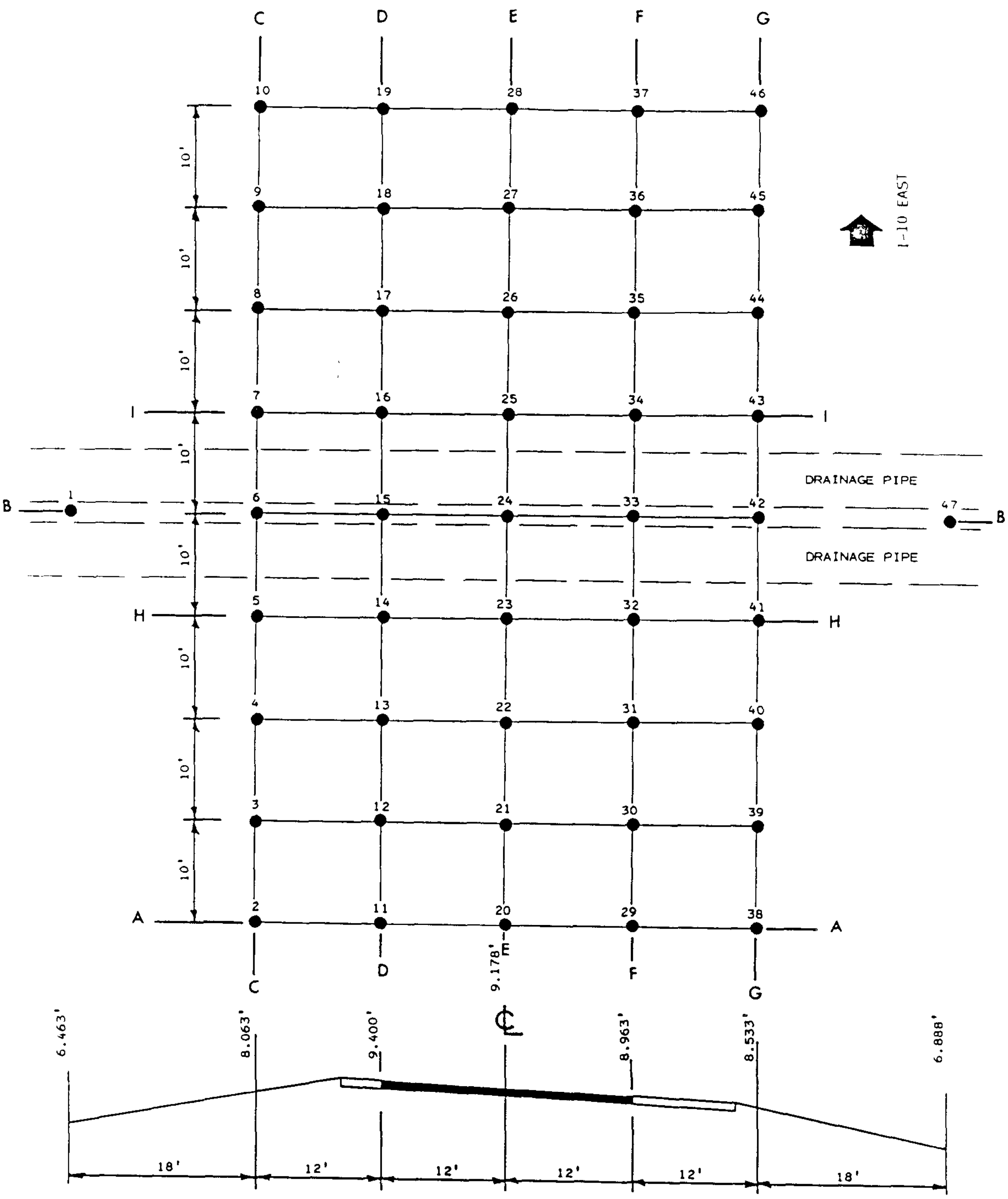
ILLUSTRATIONS



CONE PENETRATION TEST SITE PLAN



LOCATION: 1
STATION: 1991+00

● CPT LOCATION

CPT LAY OUT PLAN

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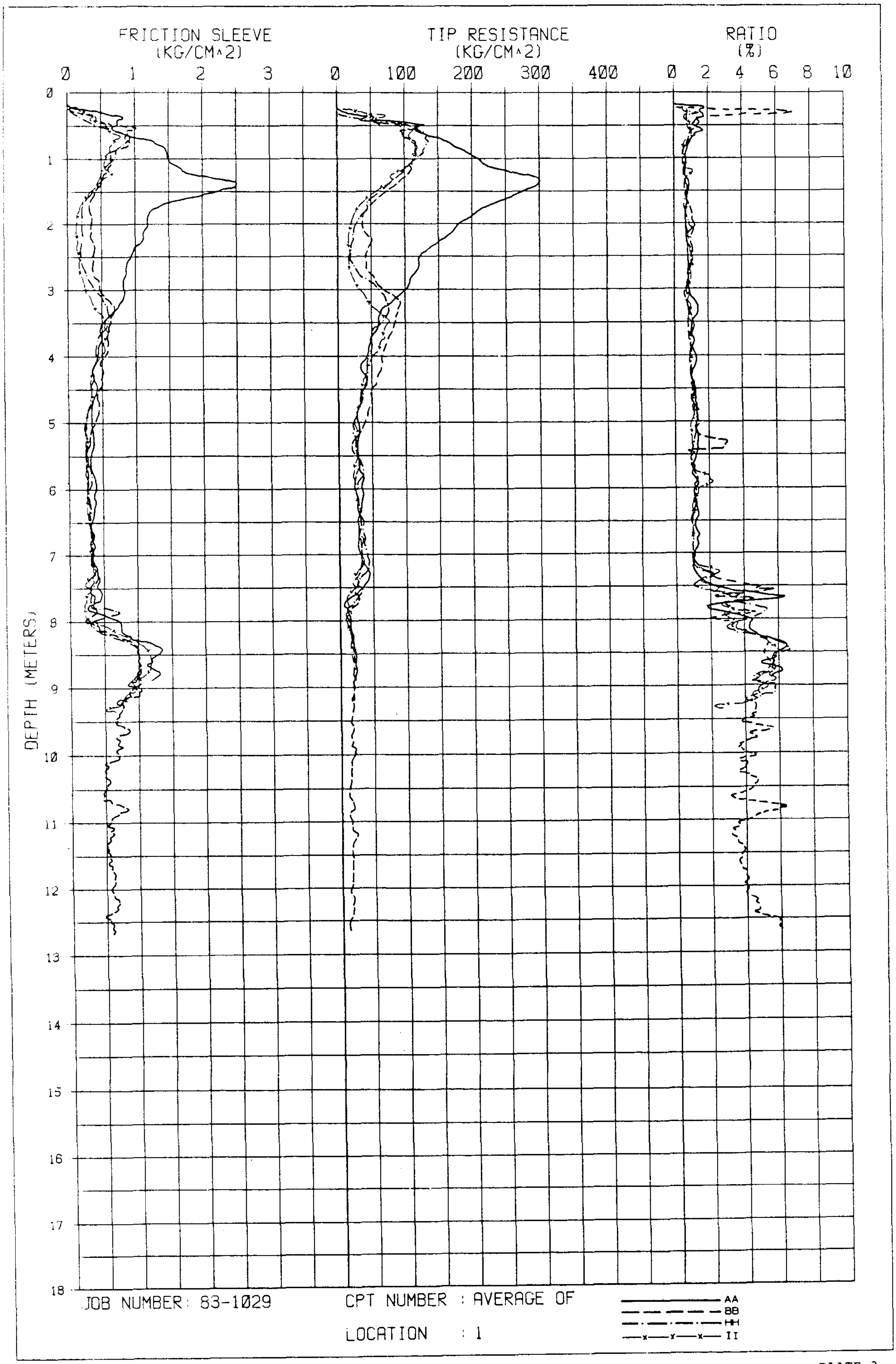
FRICTION SLEEVE
(KG/CM^2)
TIP RESISTANCE
(KG/CM^2)
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(%)
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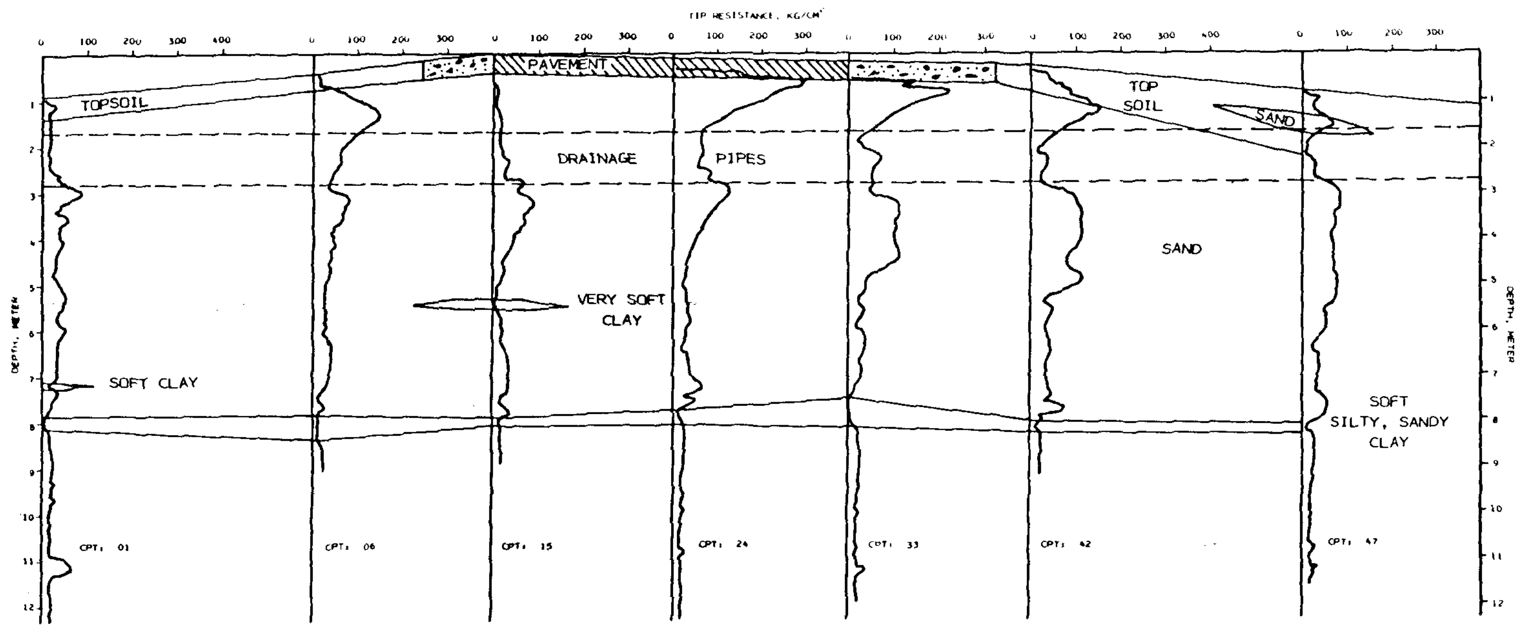
CPT NUMBER : AVERAGE OF
LOCATION : 1
AA
BB
HH
II



LOCATION: 1
CROSS SECTION: B-B
PAVEMENT ELEVATION: 2.87M (9.400 FT.)
OPEN WATER LEVEL: 0.85M (2.783 FT.)

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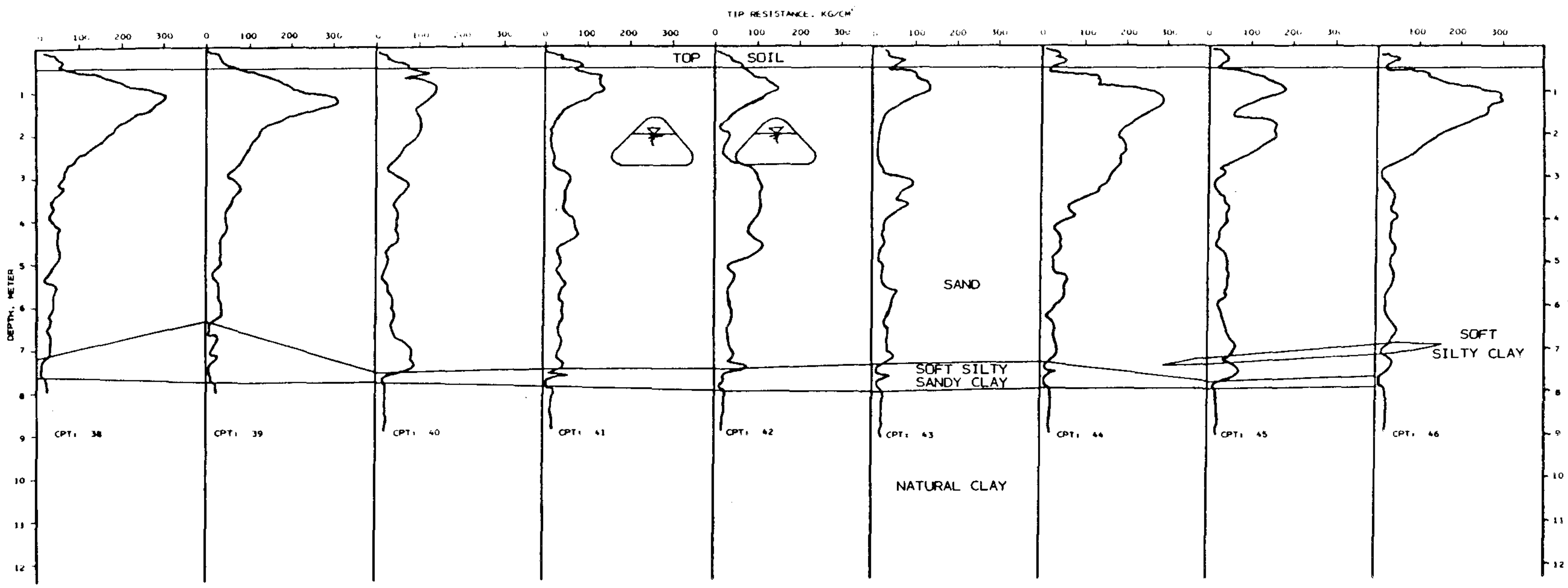
PLATE 4



LOCATION: 1

CROSS SECTION: G-G

PAVEMENT ELEVATION: 2.87M (9.400 FT.)

OPEN WATER LEVEL: 0.85M (2.783 FT.)

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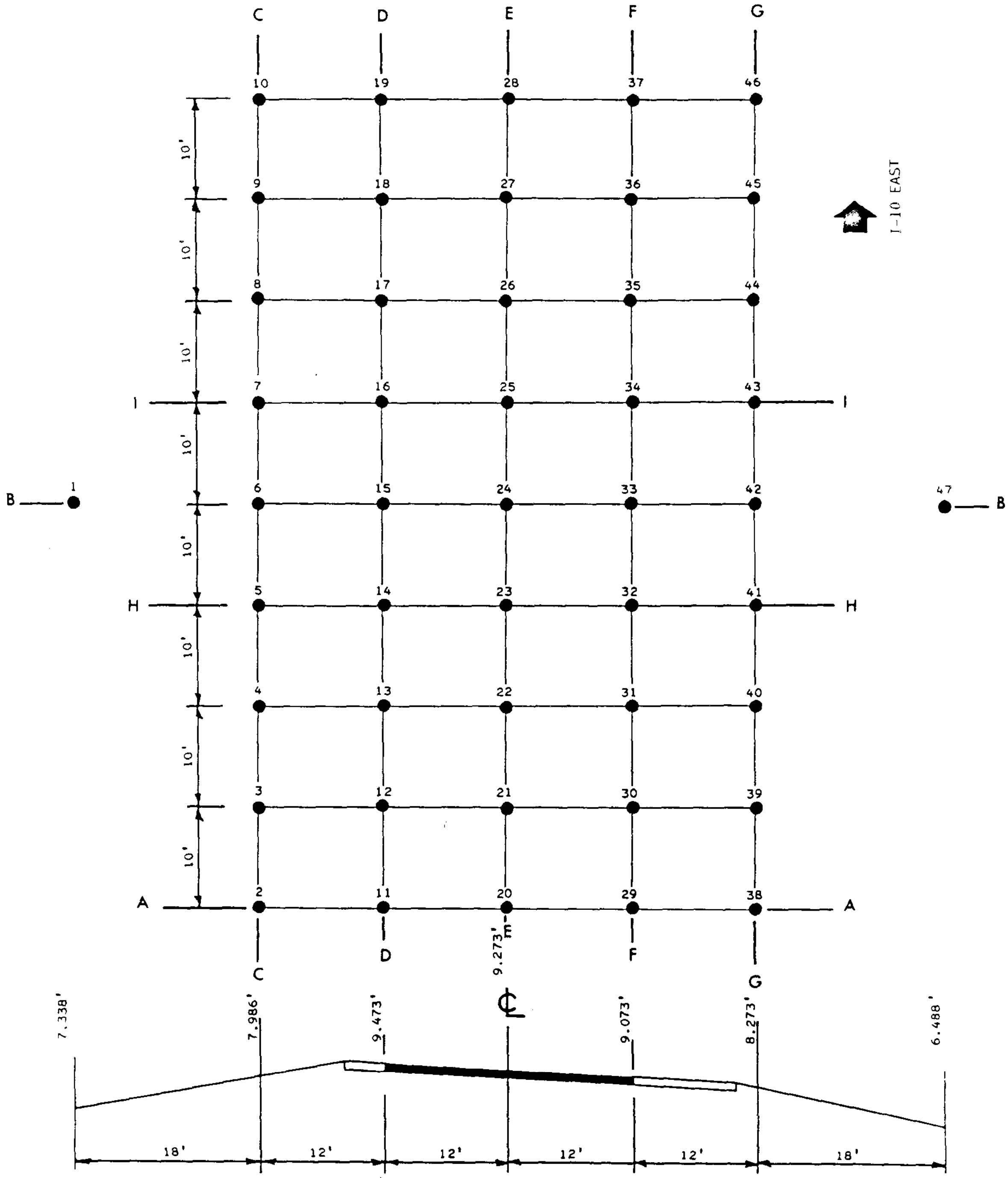

LOCATION: 1A

STATION: 1995+50

● CPT LOCATION

CPT LAY OUT PLAN

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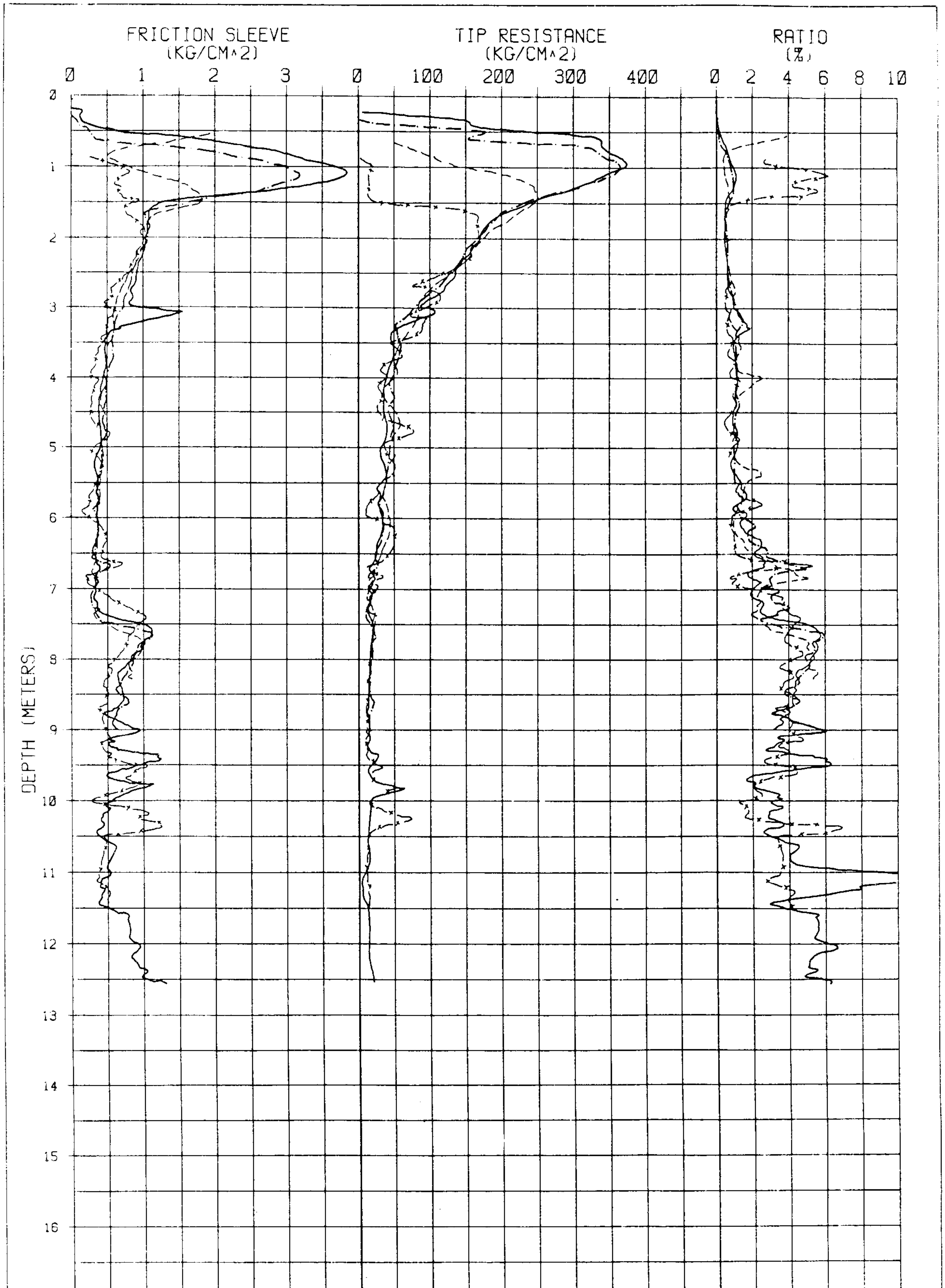
FRICTION SLEEVE
(KG/CM^2)
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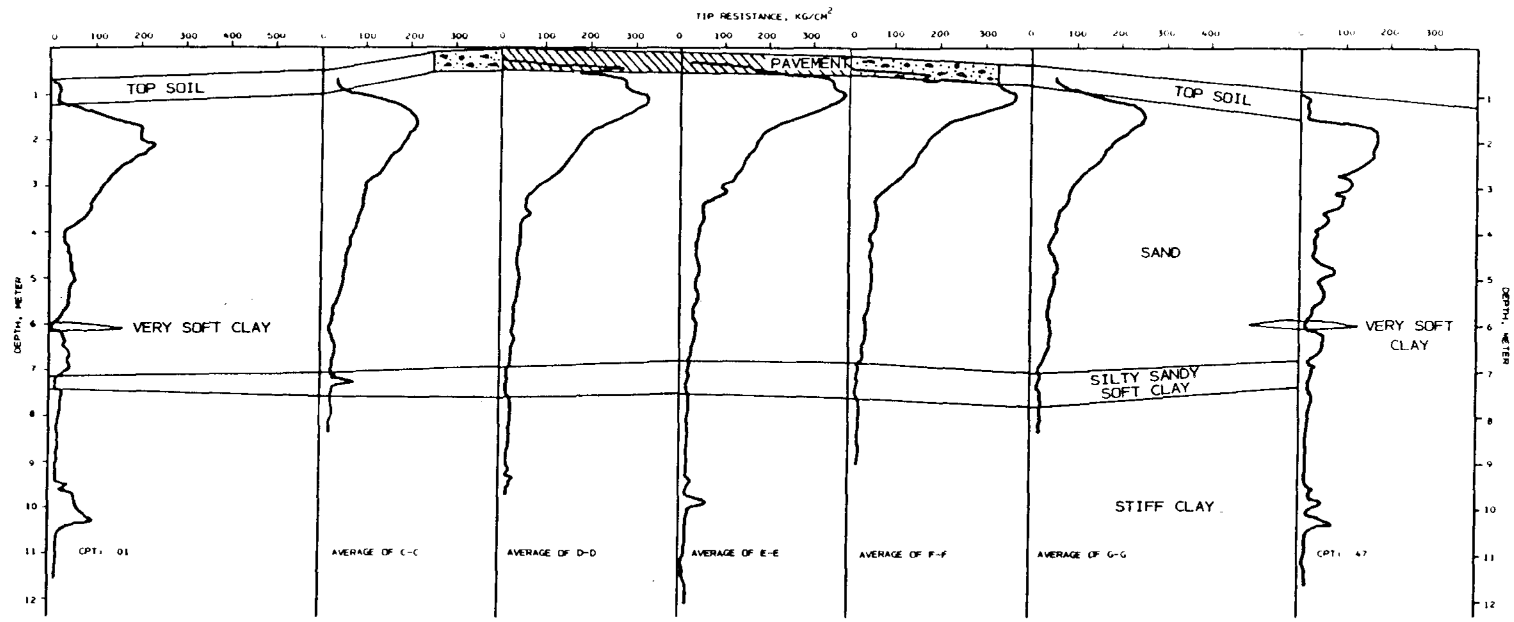
TIP RESISTANCE
(KG/CM^2)
0 100 200 300 400
RATIO
(%)
0 2 4 6 8 10
DEPTH (METERS)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
JOB NUMBER: 83-1029
CPT NUMBER : AVERAGE OF
LOCATION : 1A
EE
FF
GG
CPT 47



LOCATION: 1A

CROSS SECTION: B-B

PAVEMENT ELEVATION: 2.89M (9.473 FT.)

OPEN WATER LEVEL: 0.85M (2.783 FT.)

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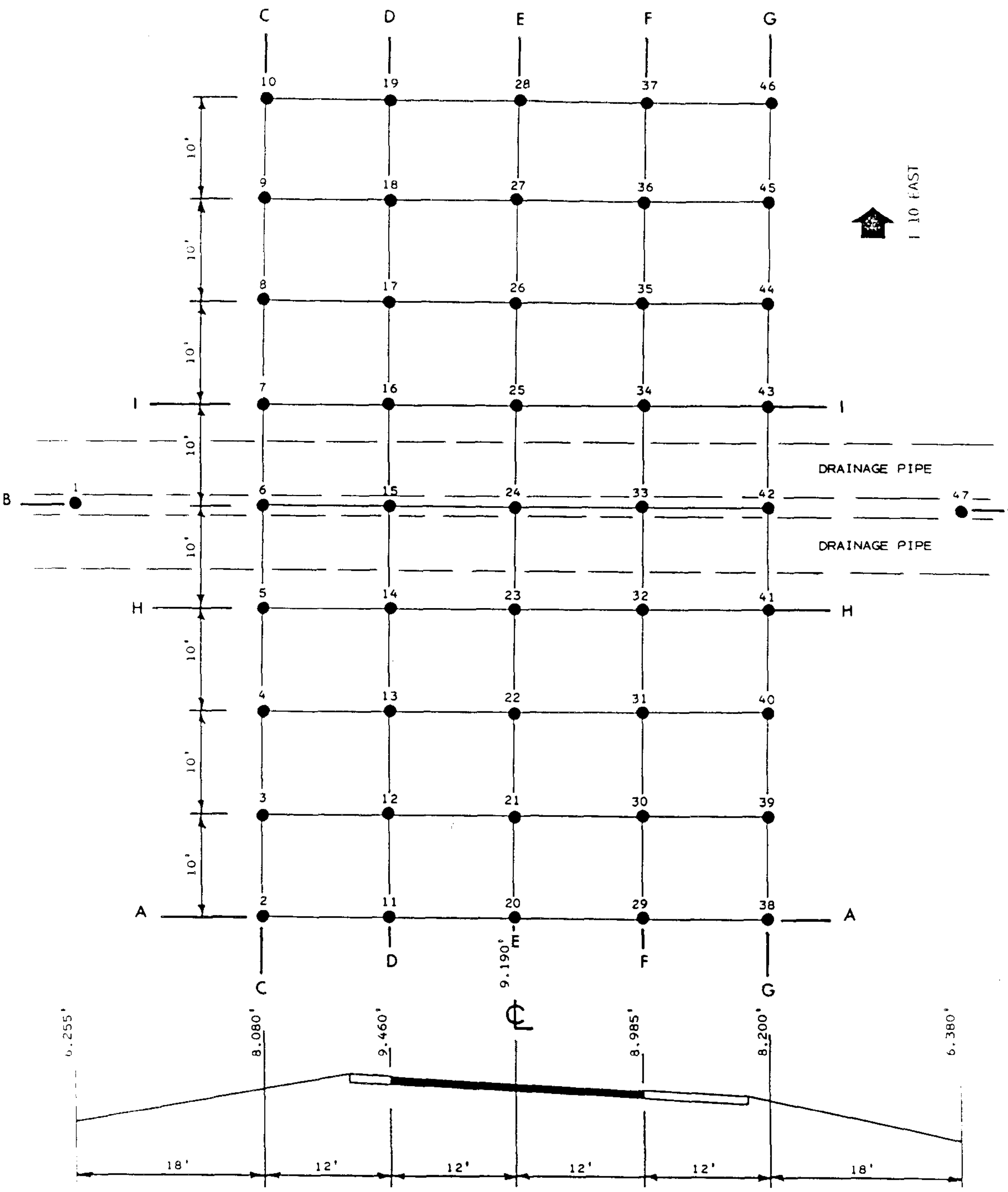
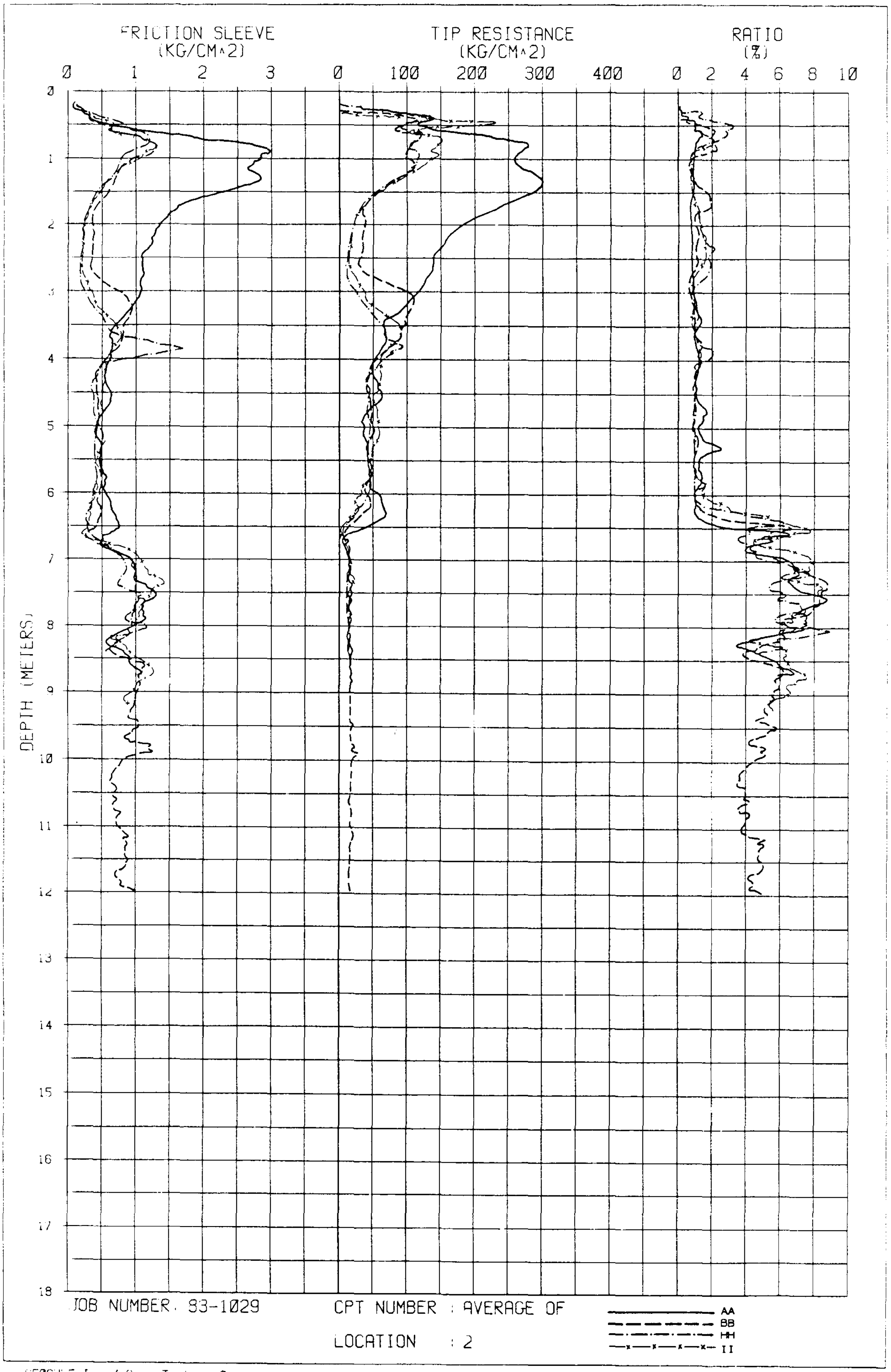


LOCATION: 2

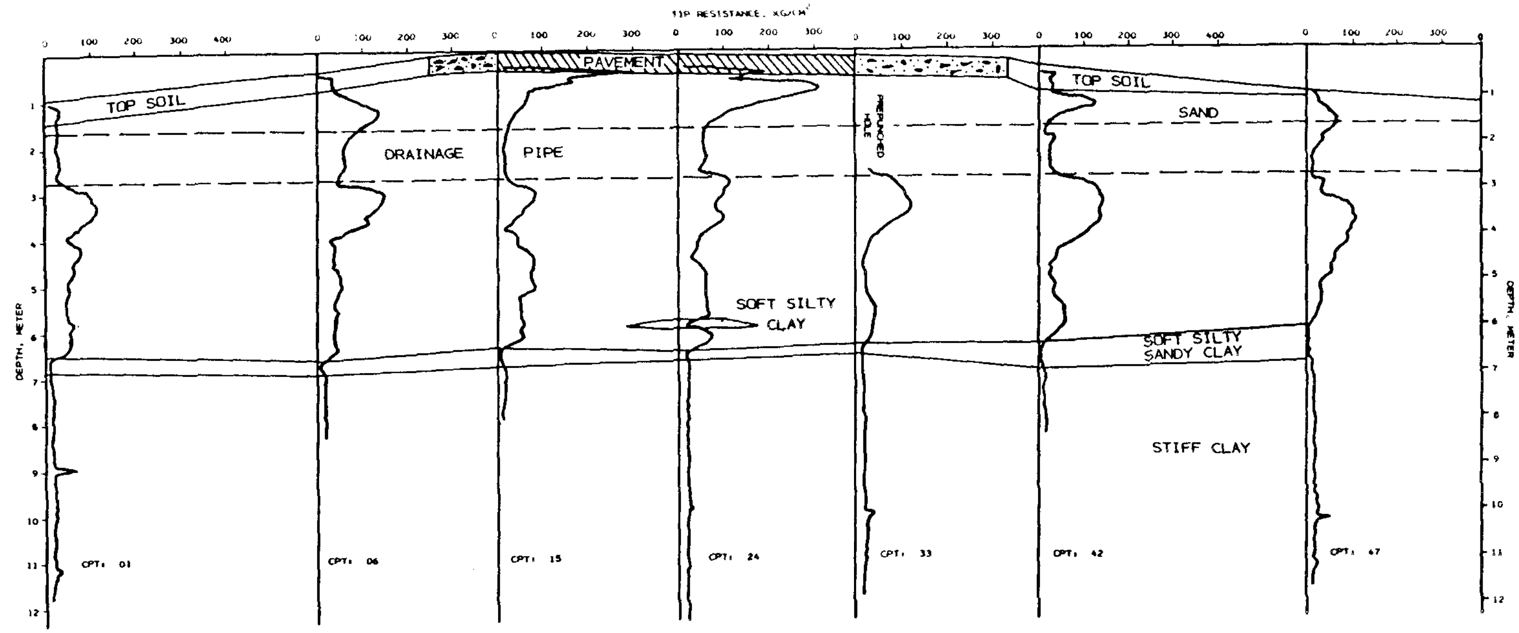
STATION: 1961+00

● CPT LOCATION

CPT LAY OUT PLAN



GEØGULF Inc. / Cone Testing Services



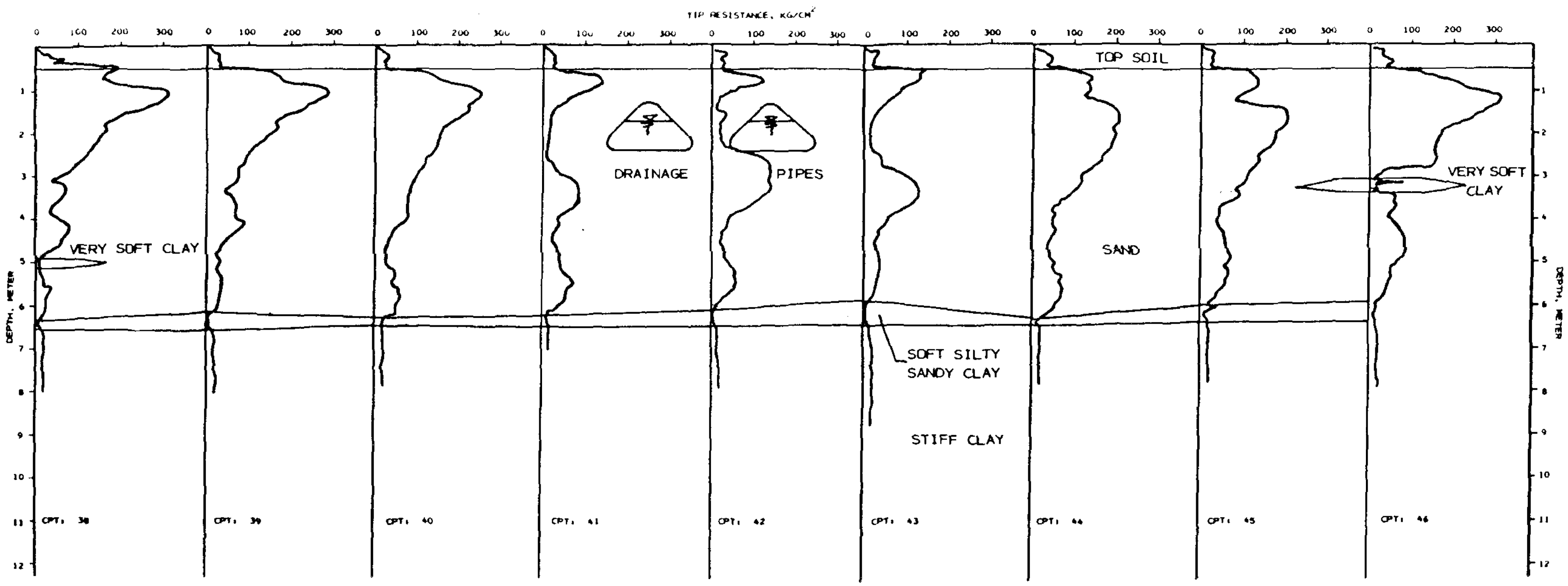
LOCATION: 2

CROSS SECTION: B-B

PAVEMENT ELEVATION: 2.88M (9.460 FT.)

OPEN WATER LEVEL: 0.81M (2.670 FT.)

GEOGULF, INC.
CONE TESTING SERVICES



LOCATION: 2

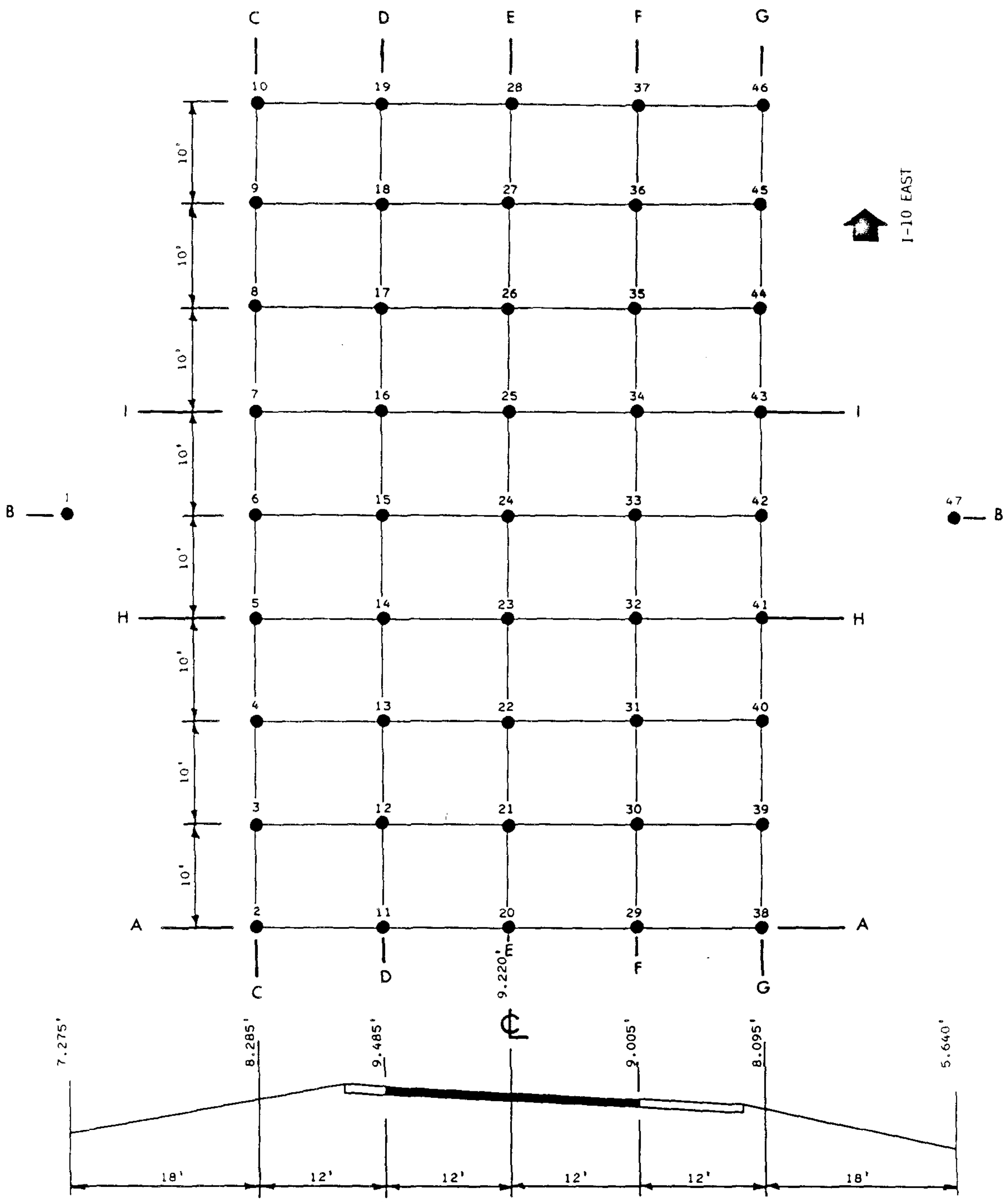
CROSS SECTION: G-G

PAVEMENT ELEVATION: 2.50M (8.200 FT.)

OPEN WATER LEVEL: 0.81M (2.670 FT.)

GEOGULF, INC.
CONE TESTING SERVICES

PLATE 12

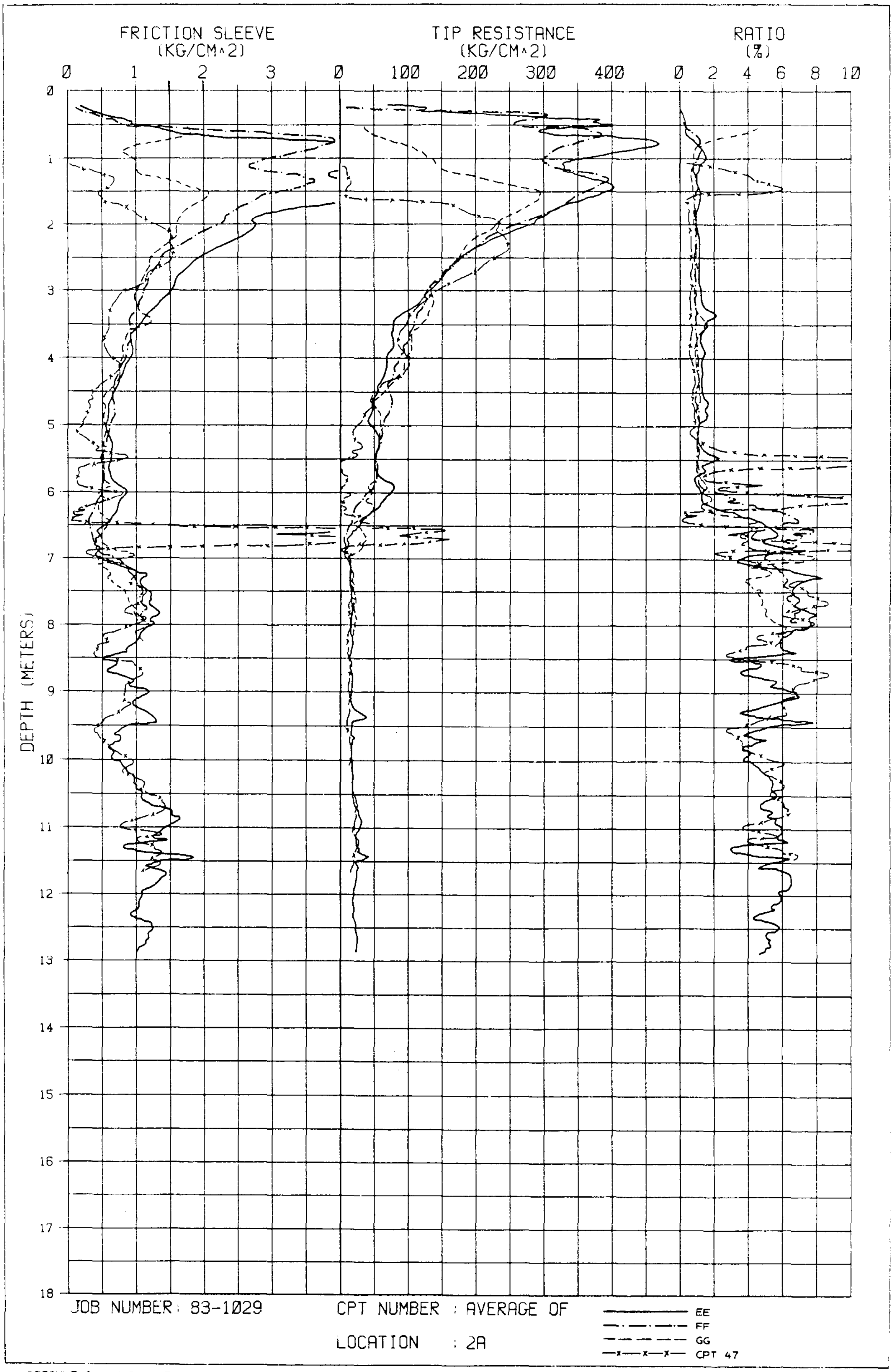


LOCATION: 2A
STATION: 1953+00

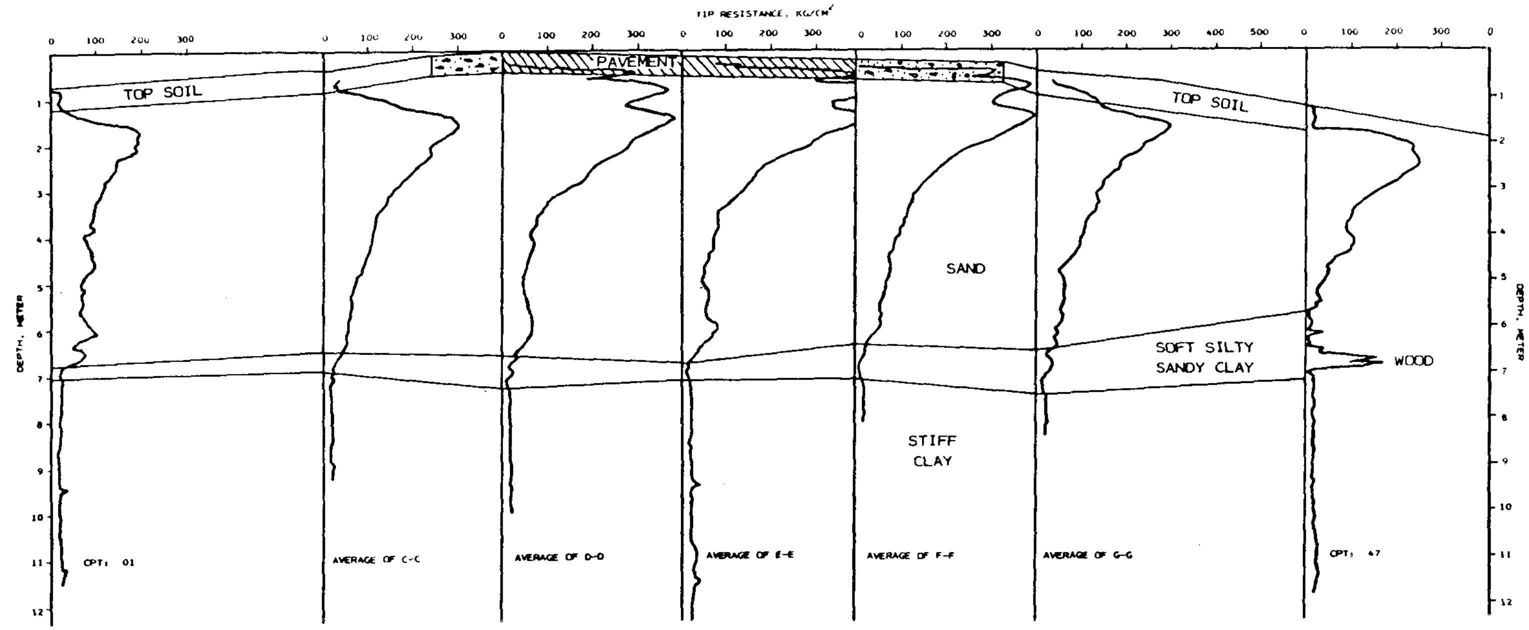
● CPT LOCATION

CPT LAY OUT PLAN

GEOGULF, INC.
CONE TESTING SERVICES



GEOGULF Inc./ Cone Testing Services

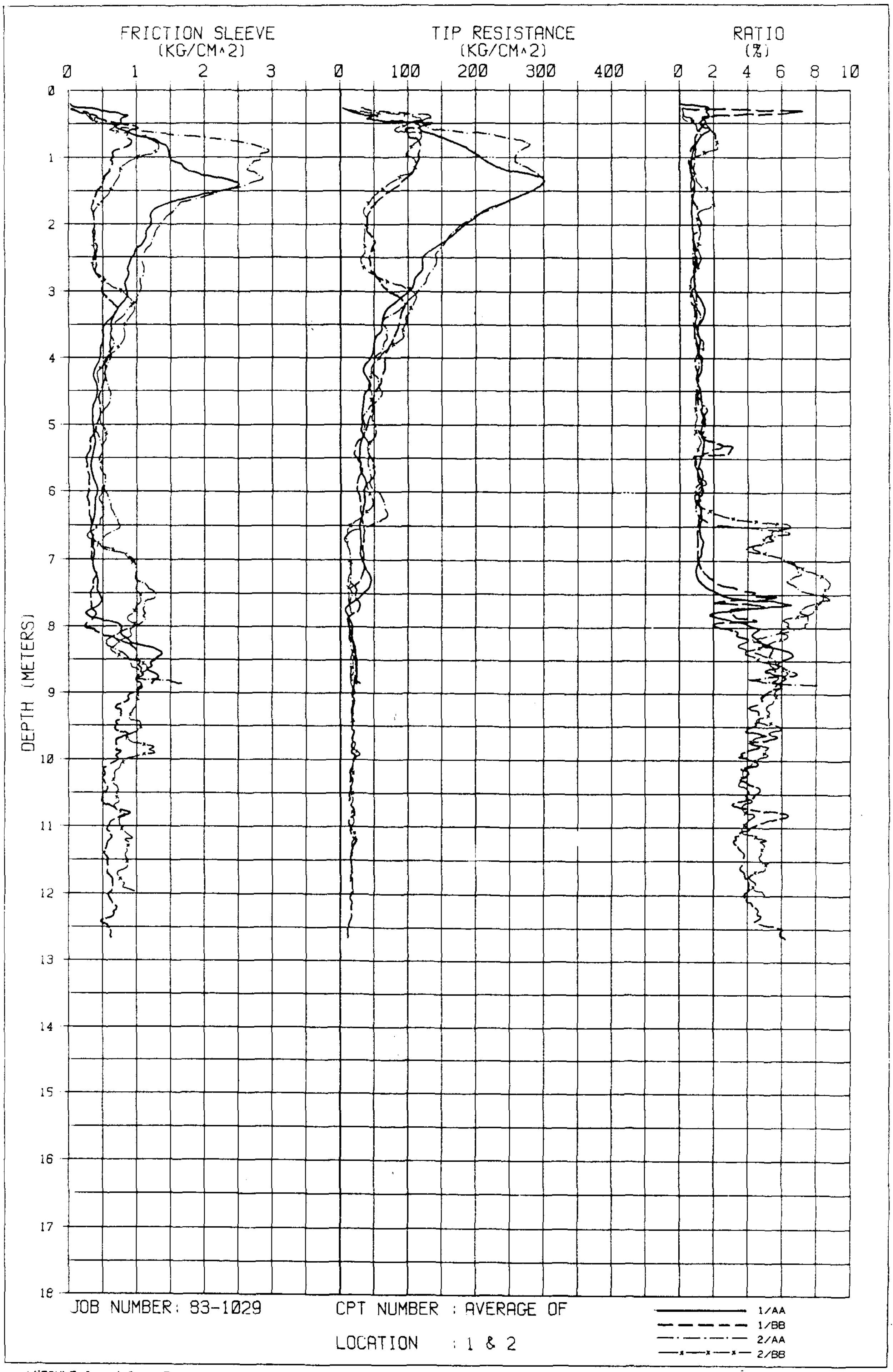


LOCATION: 2A

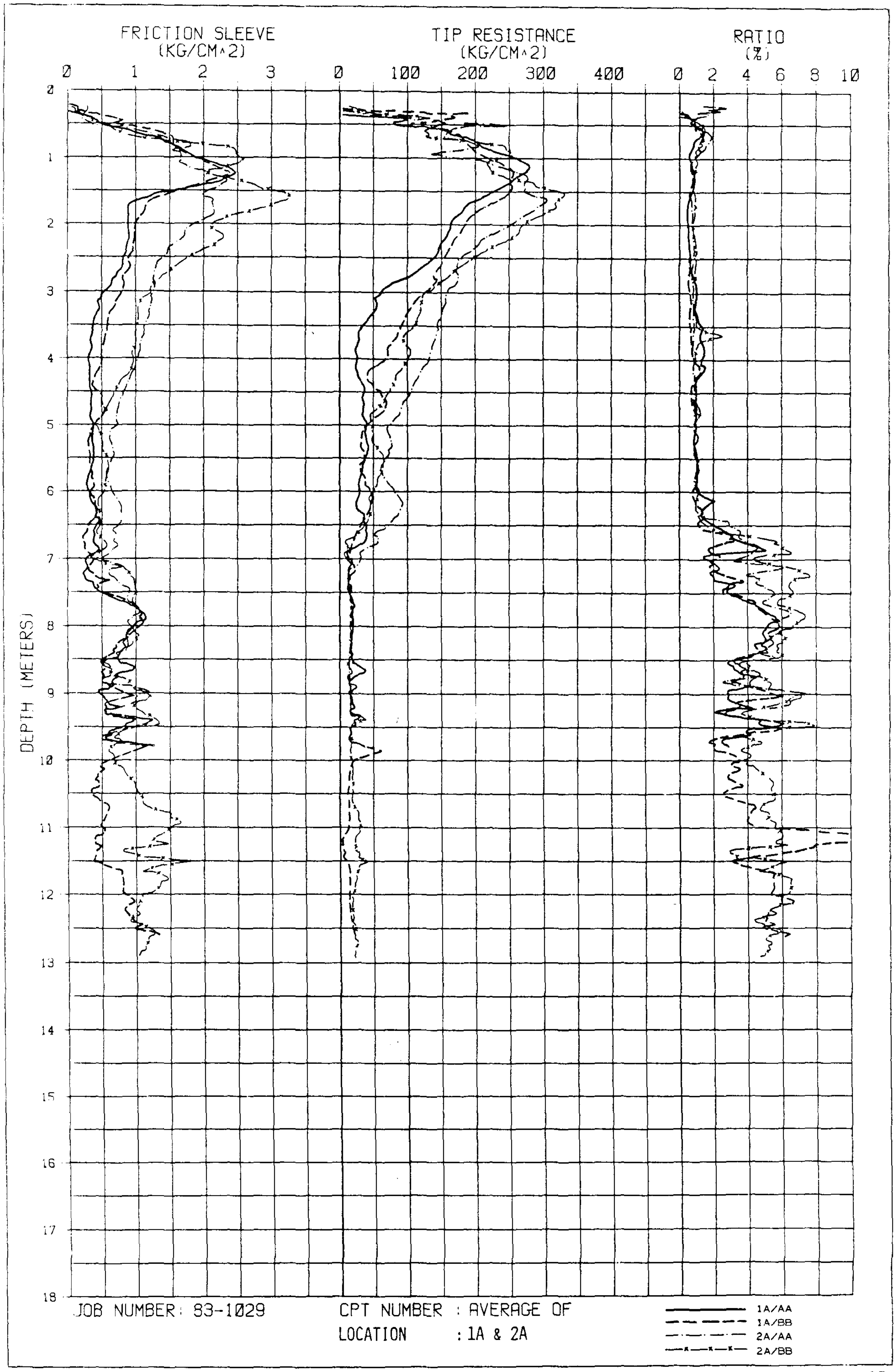
CROSS SECTION: B-B

PAVEMENT ELEVATION: 2.89M (9.485 FT.)

GROUND WATER LEVEL: 0.81M (2.670 FT.)

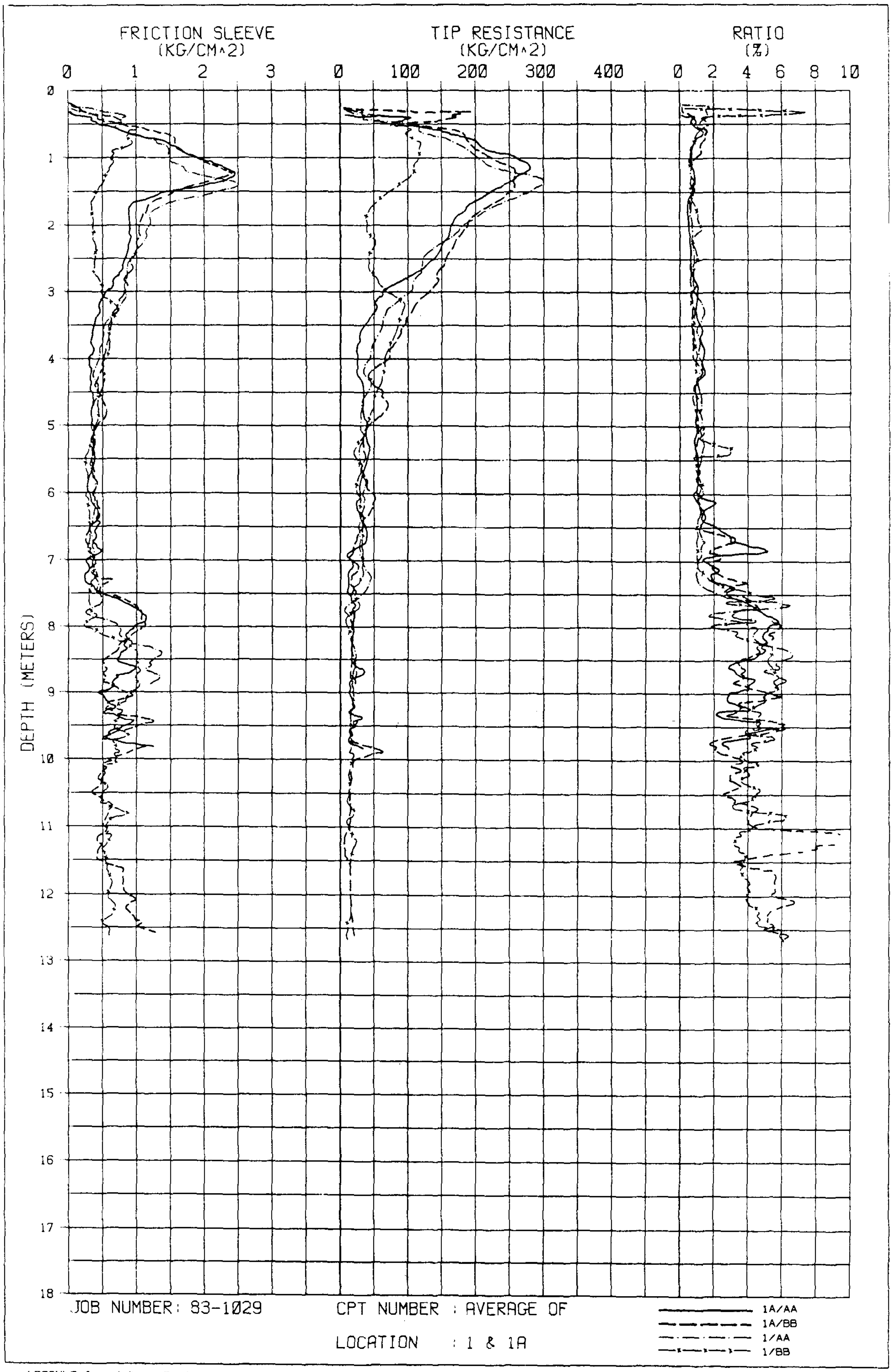


GEOGULF Inc./ Cone Testing Services

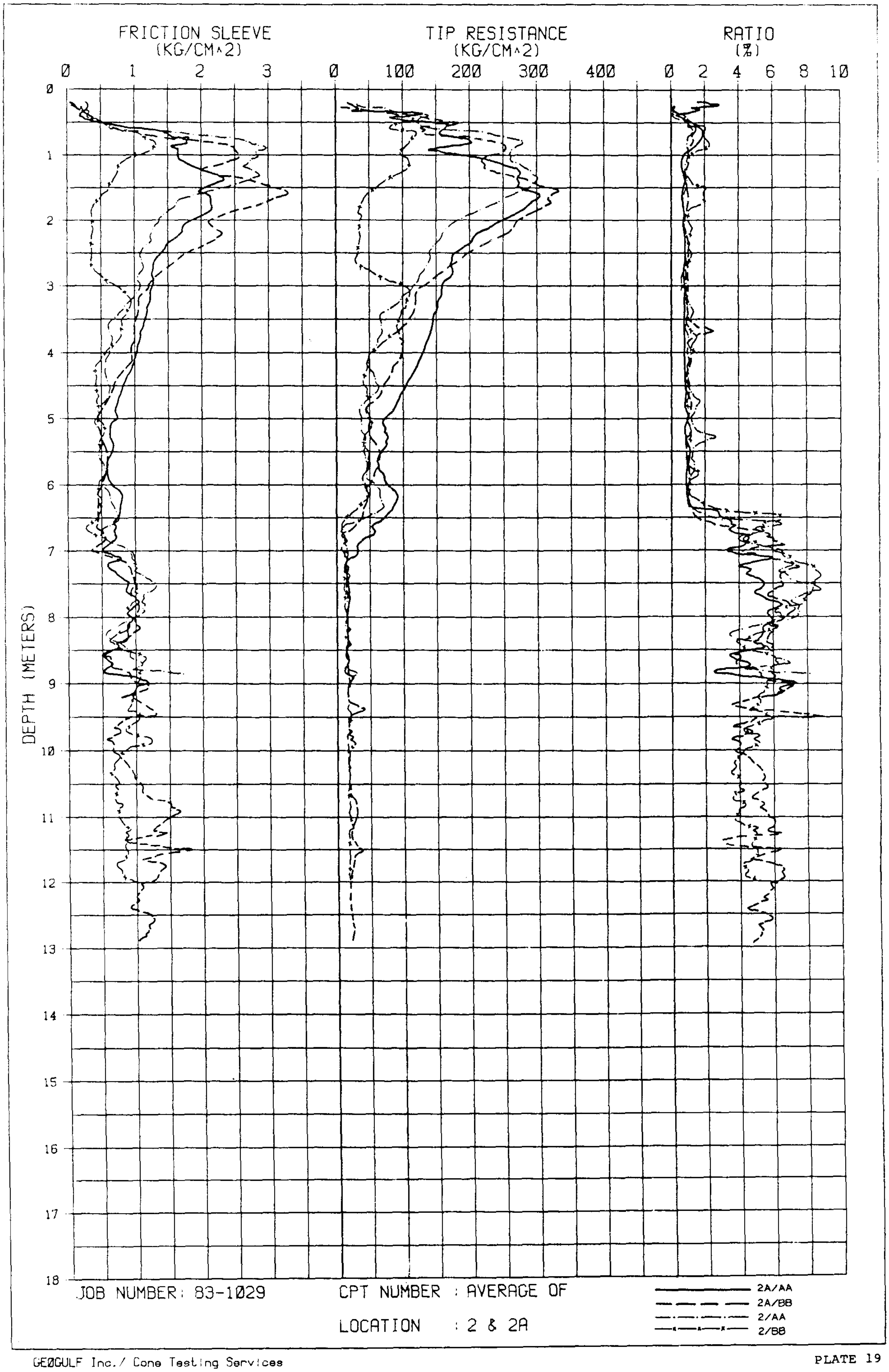


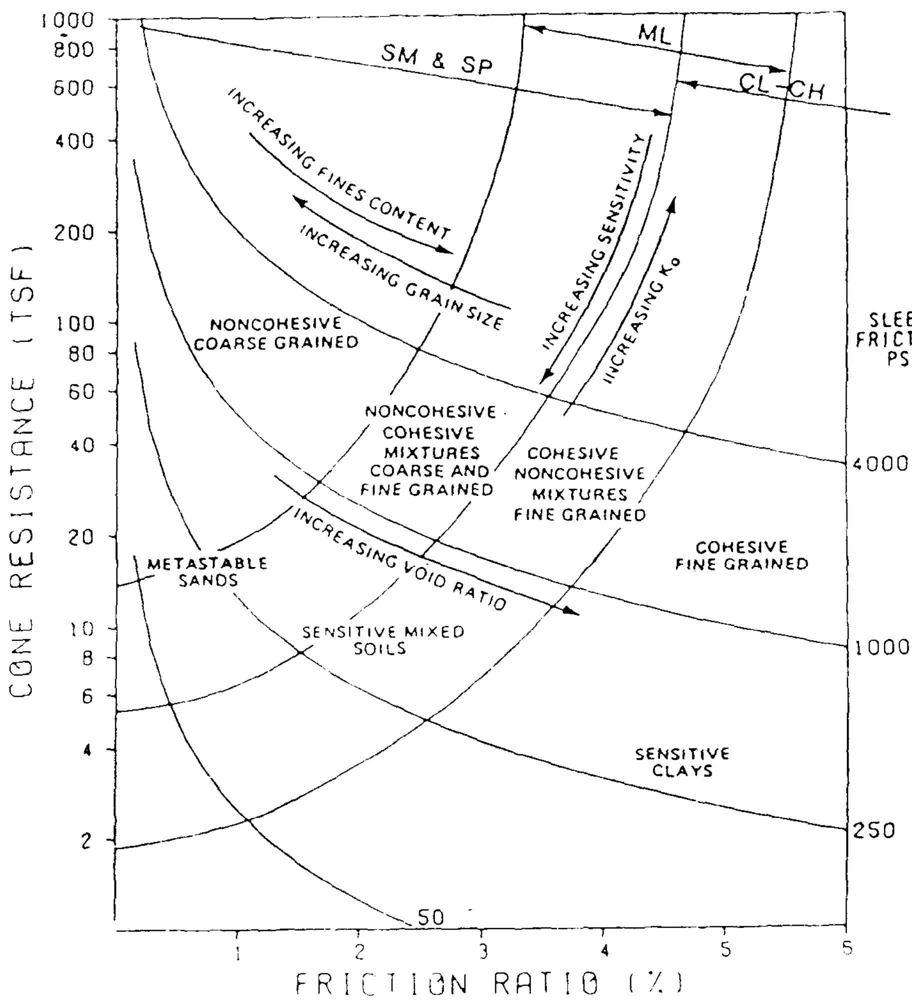
FRICTION SLEEVE
(KG/CM^2)
0
1
2
3
TIP RESISTANCE
(KG/CM^2)
0
100
200
300
400
RATIO
(%)
0
2
4
6
8
10
DEPTH (METERS)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
JOB NUMBER: 83-1029
CPT NUMBER : AVERAGE OF
LOCATION : 1A & 2A
1A/AA
1A/BB
2A/AA
2A/BB

PLATE 17



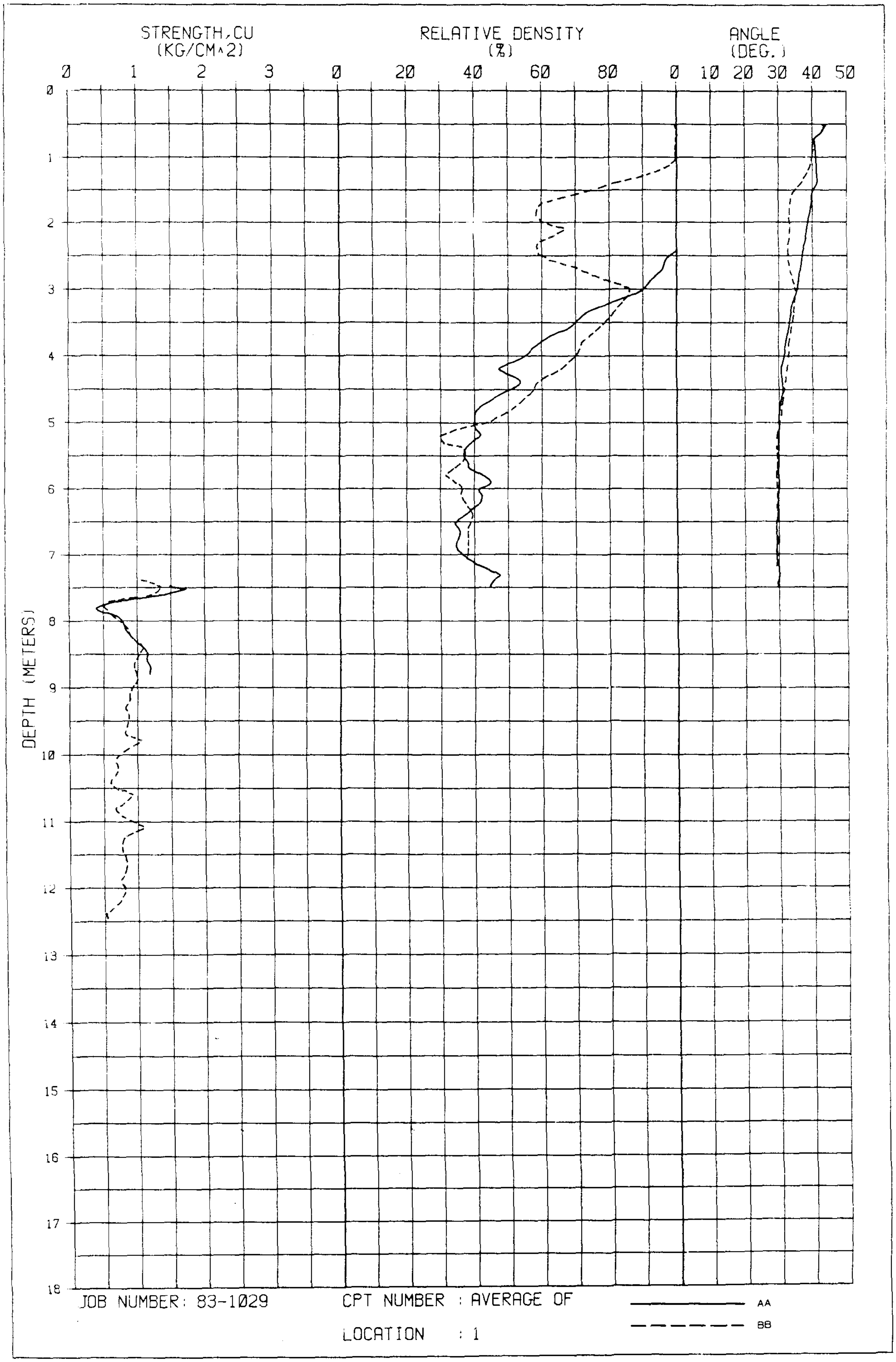
FRICTION SLEEVE
(KG/CM^2)
0 1 2 3
TIP RESISTANCE
(KG/CM^2)
0 100 200 300 400
RATIO
(%)
0 2 4 6 8 10
DEPTH (METERS)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
JOB NUMBER: 83-1029
CPT NUMBER : AVERAGE OF
LOCATION : 1 & 1A
1A/AA
1A/BB
1/AA
1/BB

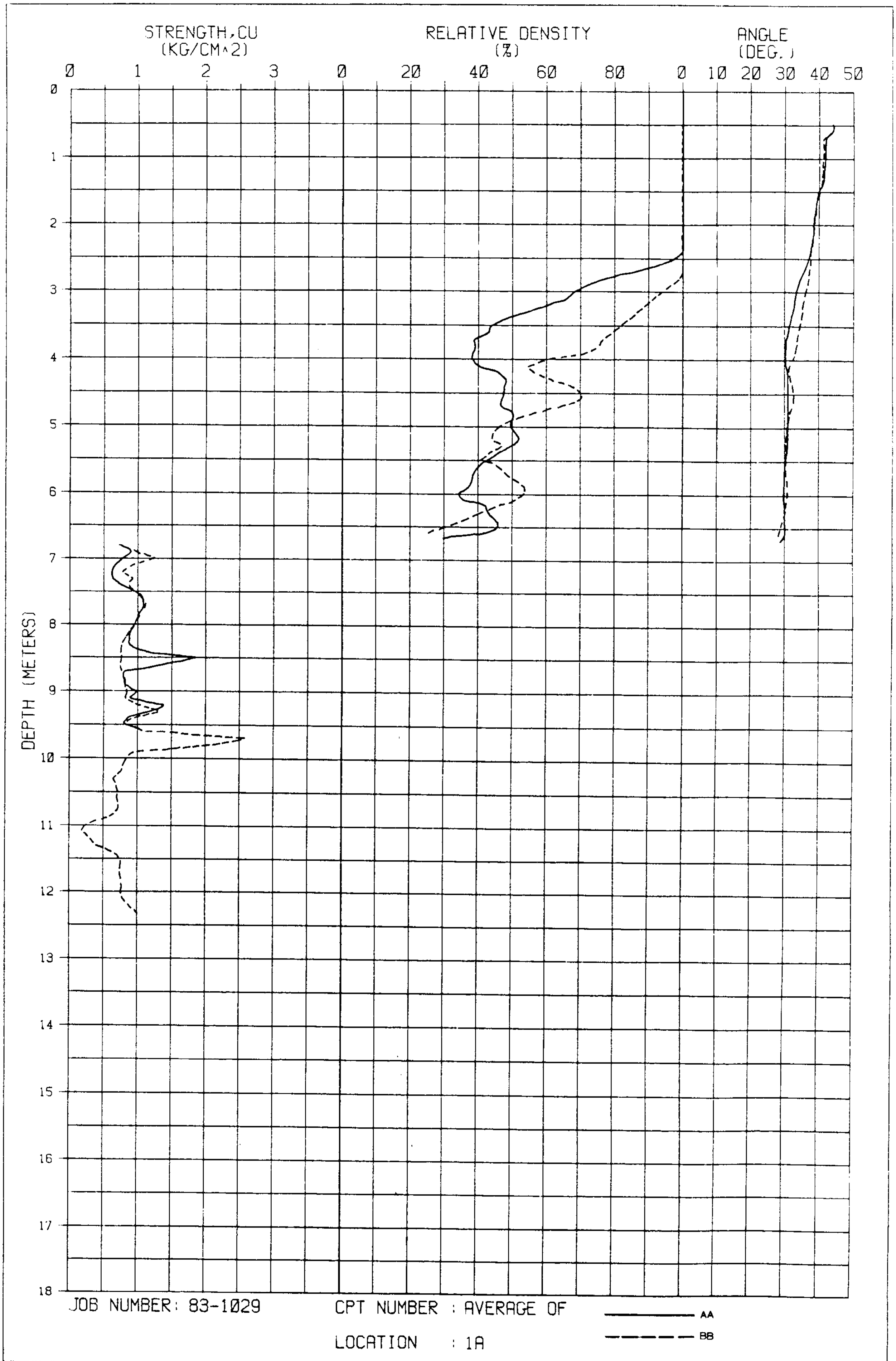

FRICTION SLEEVE
(KG/CM^2)
0 1 2 3
TIP RESISTANCE
(KG/CM^2)
0 100 200 300 400
RATIO
(%)
0 2 4 6 8 10
DEPTH (METERS)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
JOB NUMBER: 83-1029
CPT NUMBER : AVERAGE OF
LOCATION : 2 & 2A
2A/AA
2A/BB
2/AA
2/BB



SOIL CLASSIFICATION CHART (AFTER DOUGLAS AND OLSEN, 1981)

Job No.

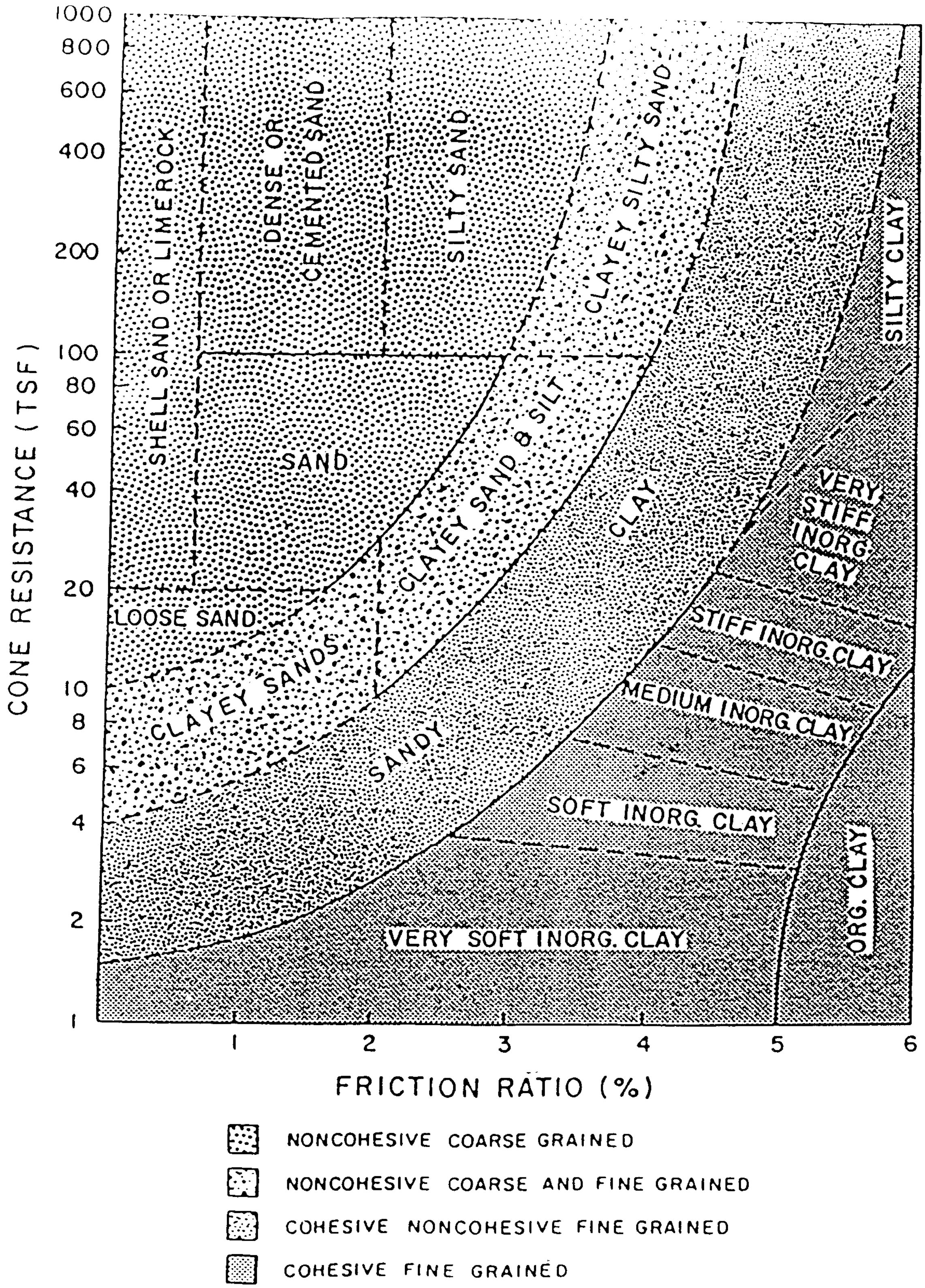

STRENGTH,CU
(KG/CM^2)
0 1 2 3
RELATIVE DENSITY
(%)
0 20 40 60 80
ANGLE
(DEG.)
0 10 20 30 40 50
DEPTH (METERS)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
JOB NUMBER: 83-1029
CPT NUMBER : AVERAGE OF
LOCATION : 1
AA
BB



GEOGULF Inc./ Cone Testing Services

PLATE 22

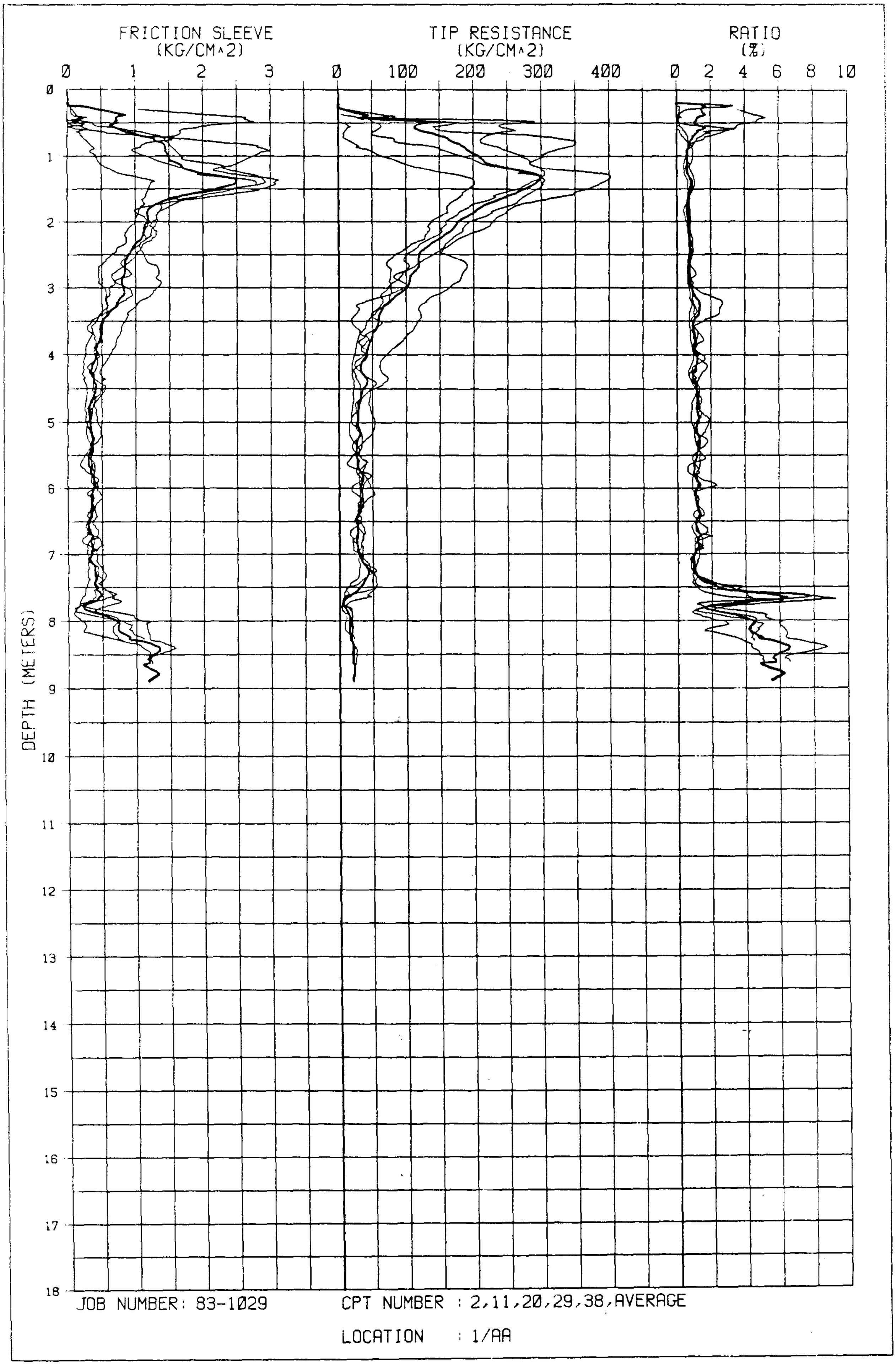
Job No.______

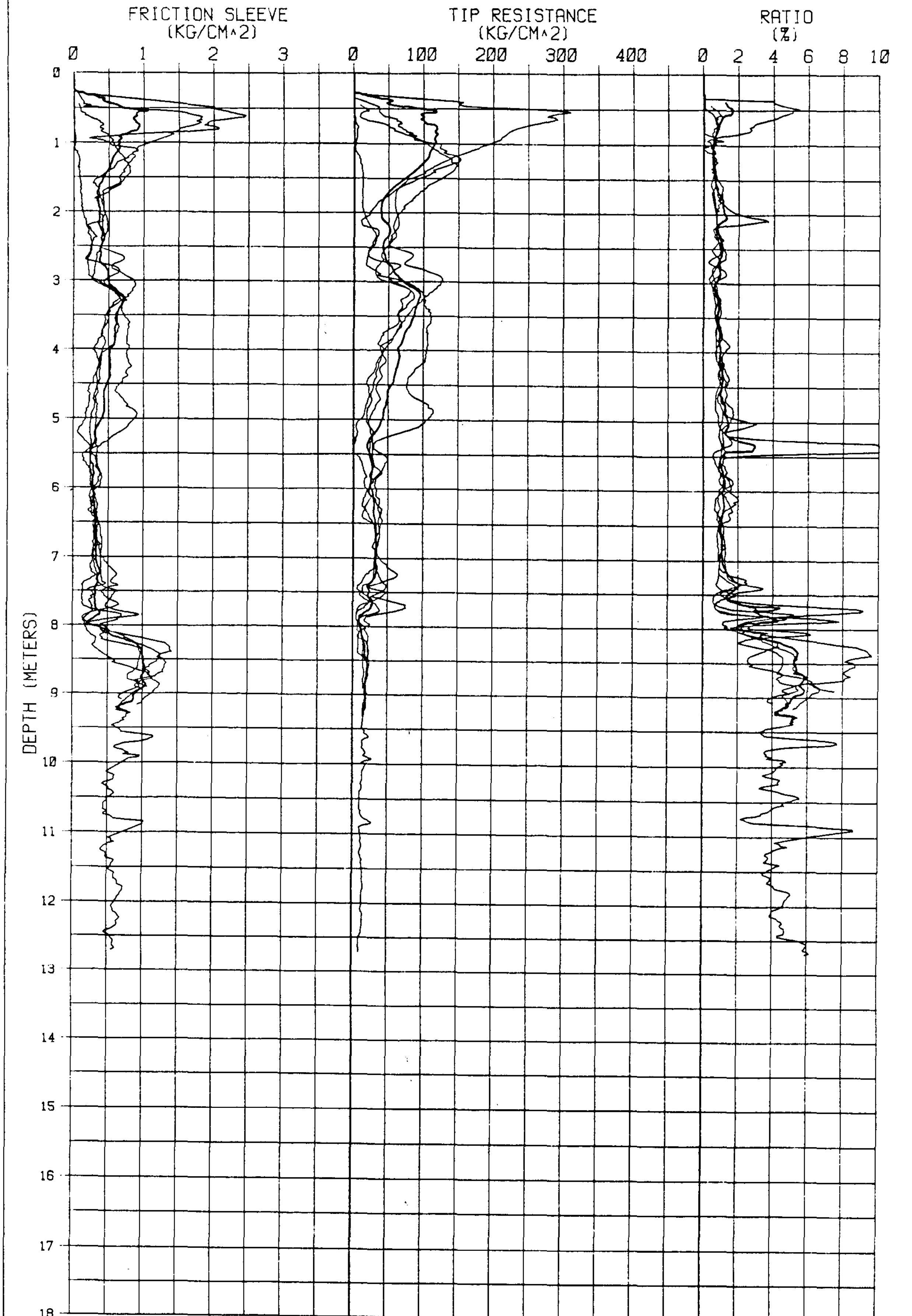


MODIFIED SOIL CLASSIFICATION CHART (AFTER TUMAY AND CHAN, 1983)

A P P E N D I X

AVERAGE CONE RESULTS


FRICTION SLEEVE
(KG/CM^2)
TIP RESISTANCE
(KG/CM^2)
RATIO
(%)
DEPTH (METERS)
JOB NUMBER: 83-1029
CPT NUMBER : 2,11,20,29,38,AVERAGE
LOCATION : 1/AA



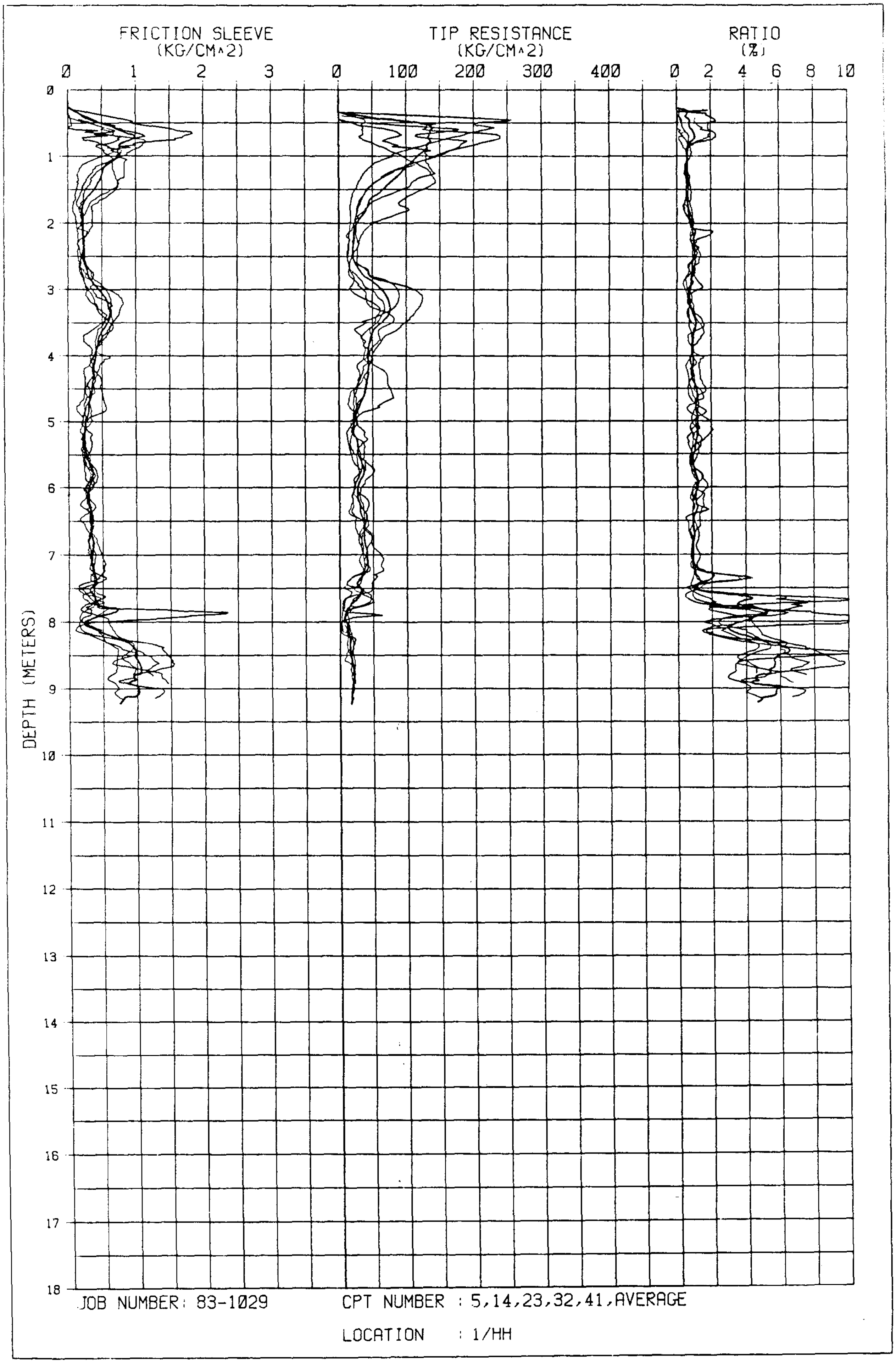
JOB NUMBER: 83-1029

CPT NUMBER : 6,15,24,33,42,AVERAGE

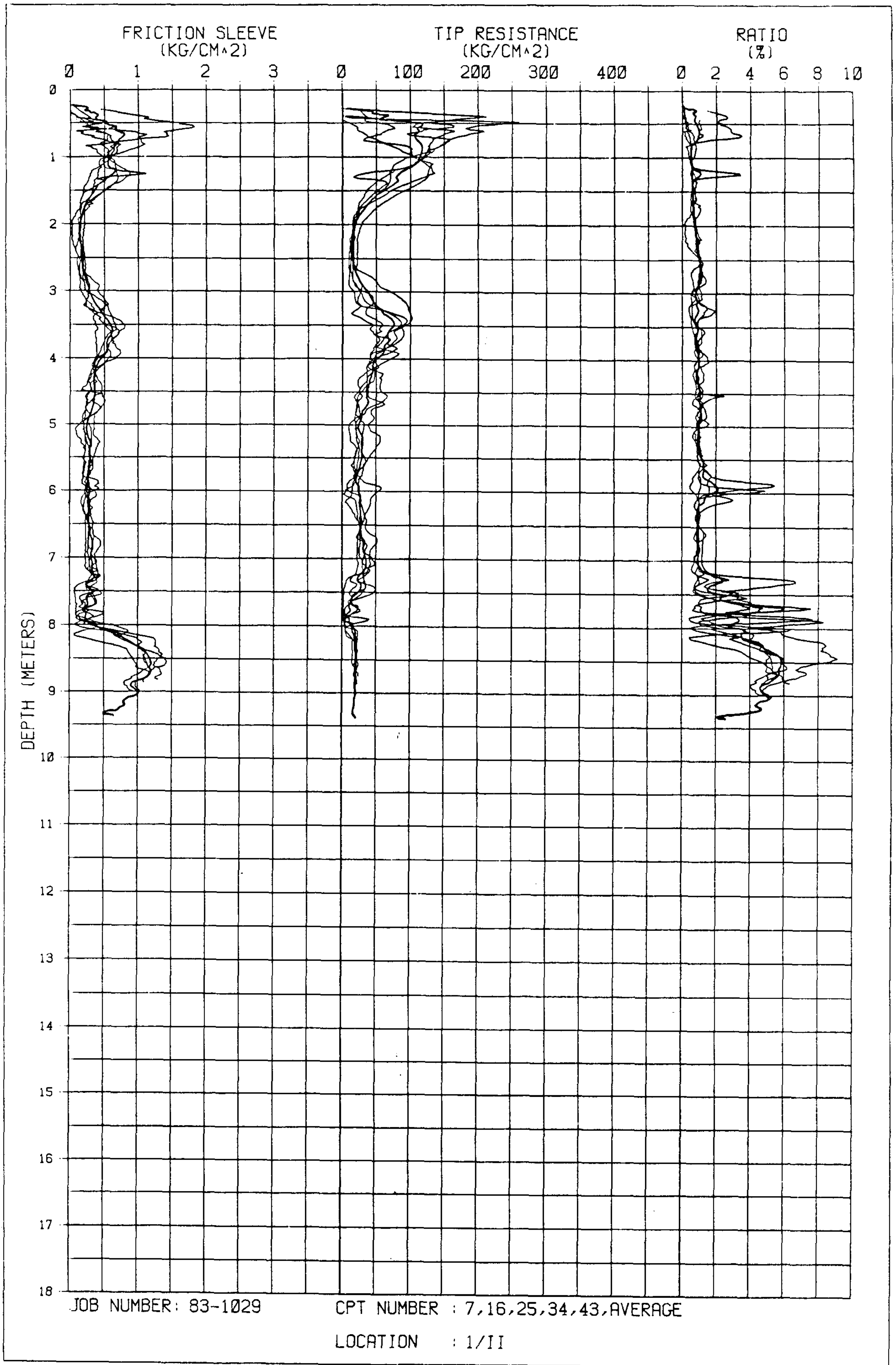
LOCATION : 1/BB

GEOGULF Inc./ Cone Testing Services

PLATE A-2

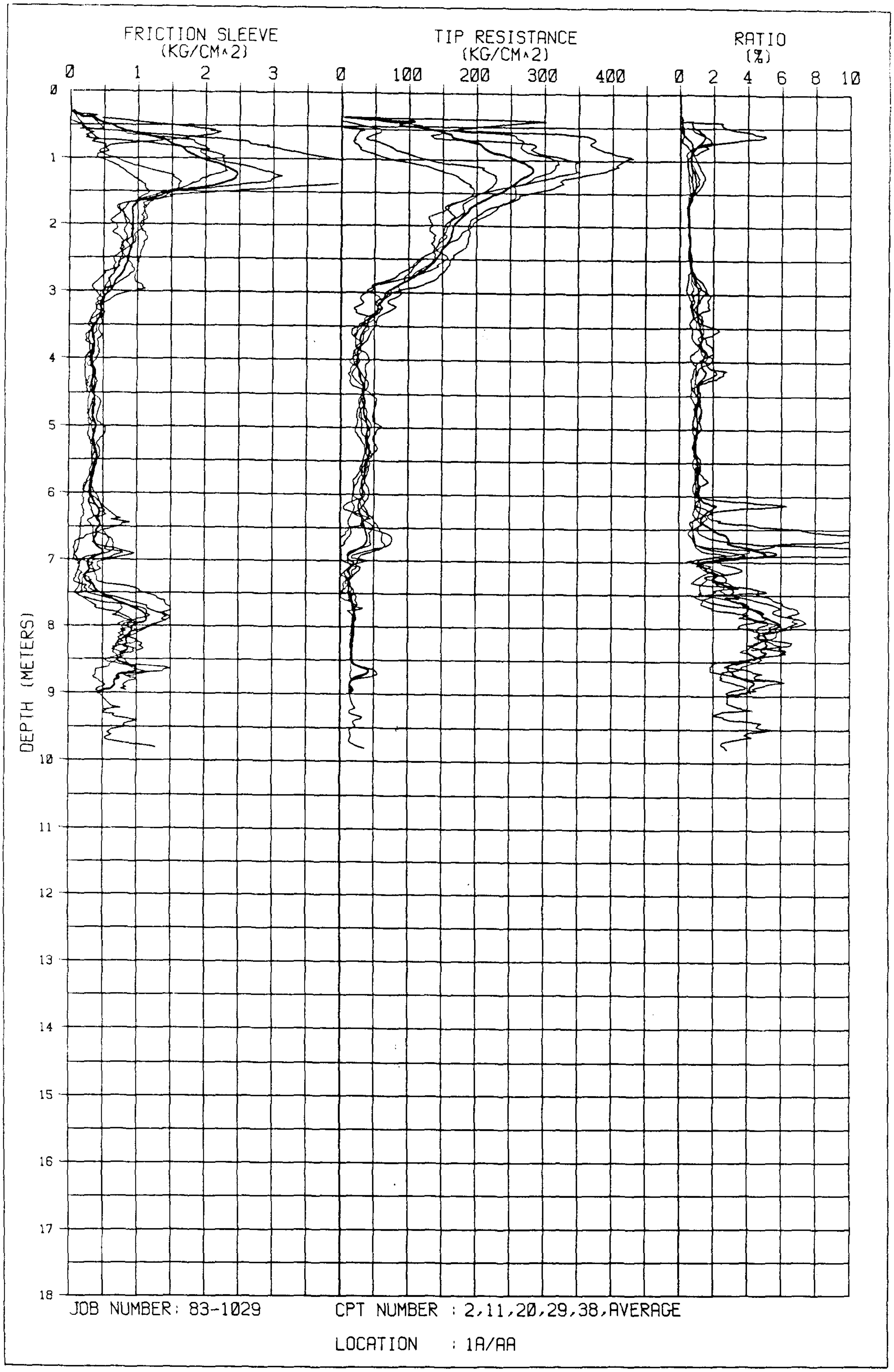


GEØGULF Inc./ Cone Testing Services

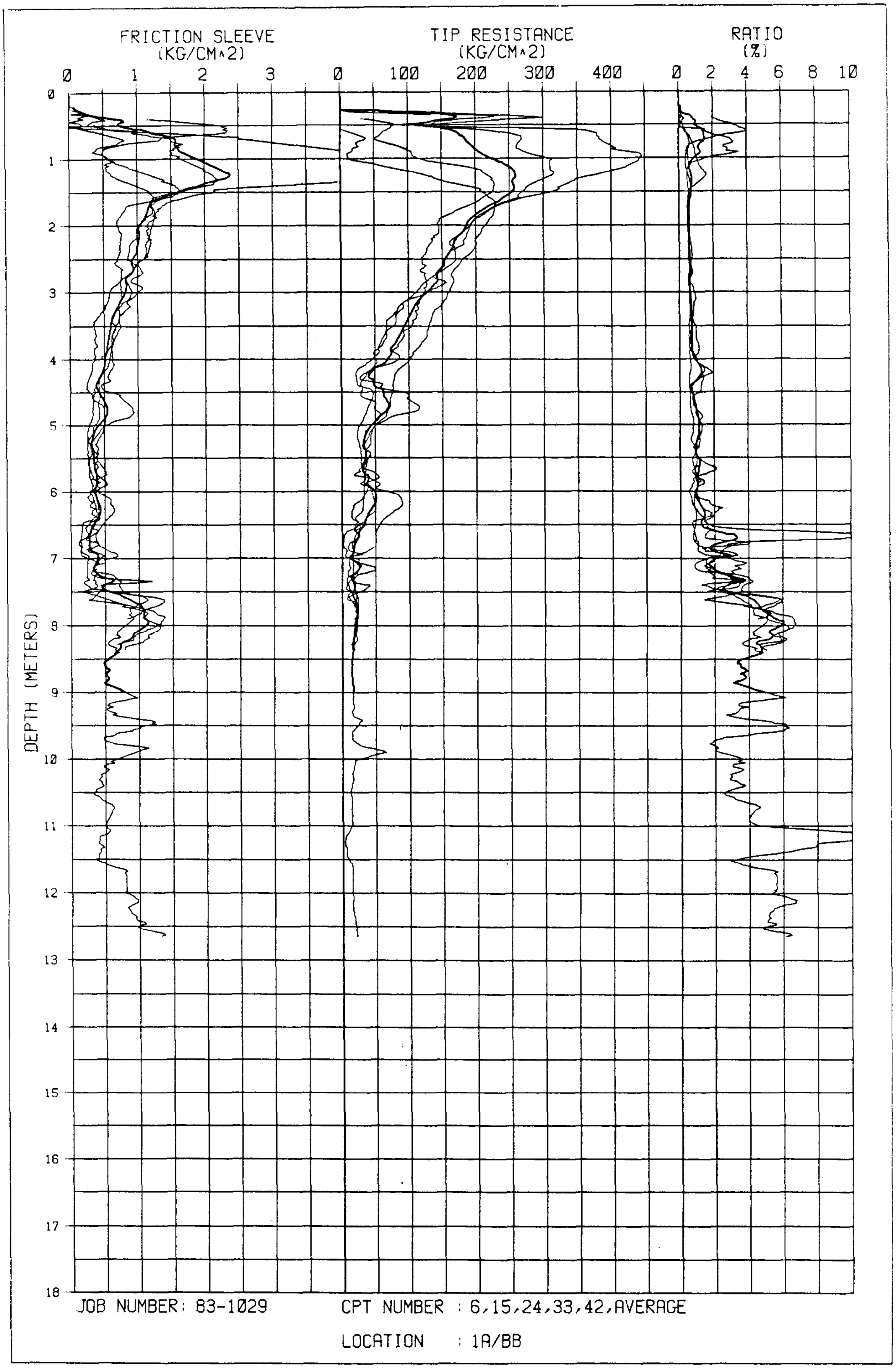


GEØGULF Inc./ Cone Testing Services

PLATE A-4

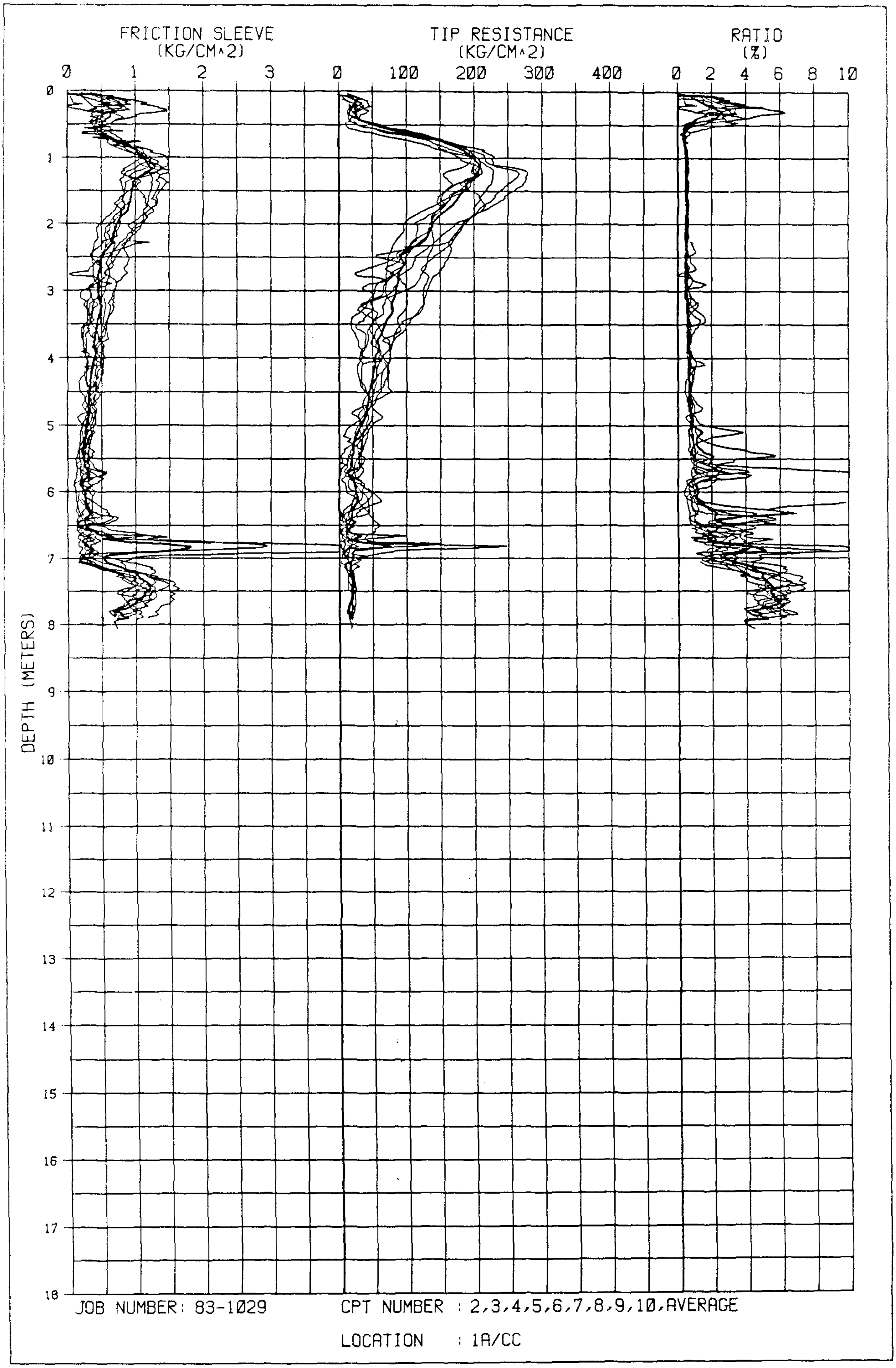


GEØGULF Inc./ Cone Testing Services

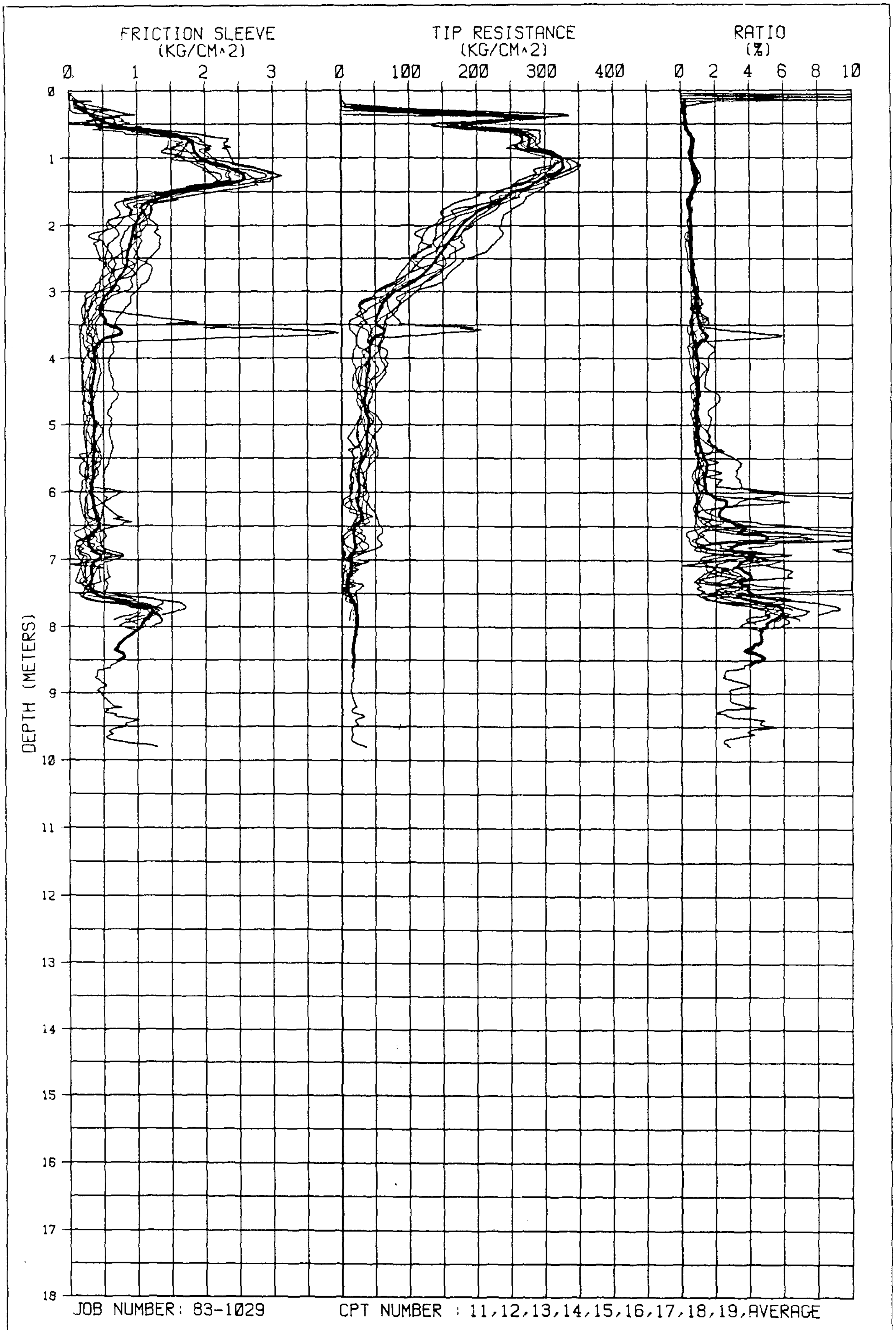


GEØGULF Inc./ Cone Testing Services

PLATE A-6



GEOGULF Inc./ Cone Testing Services

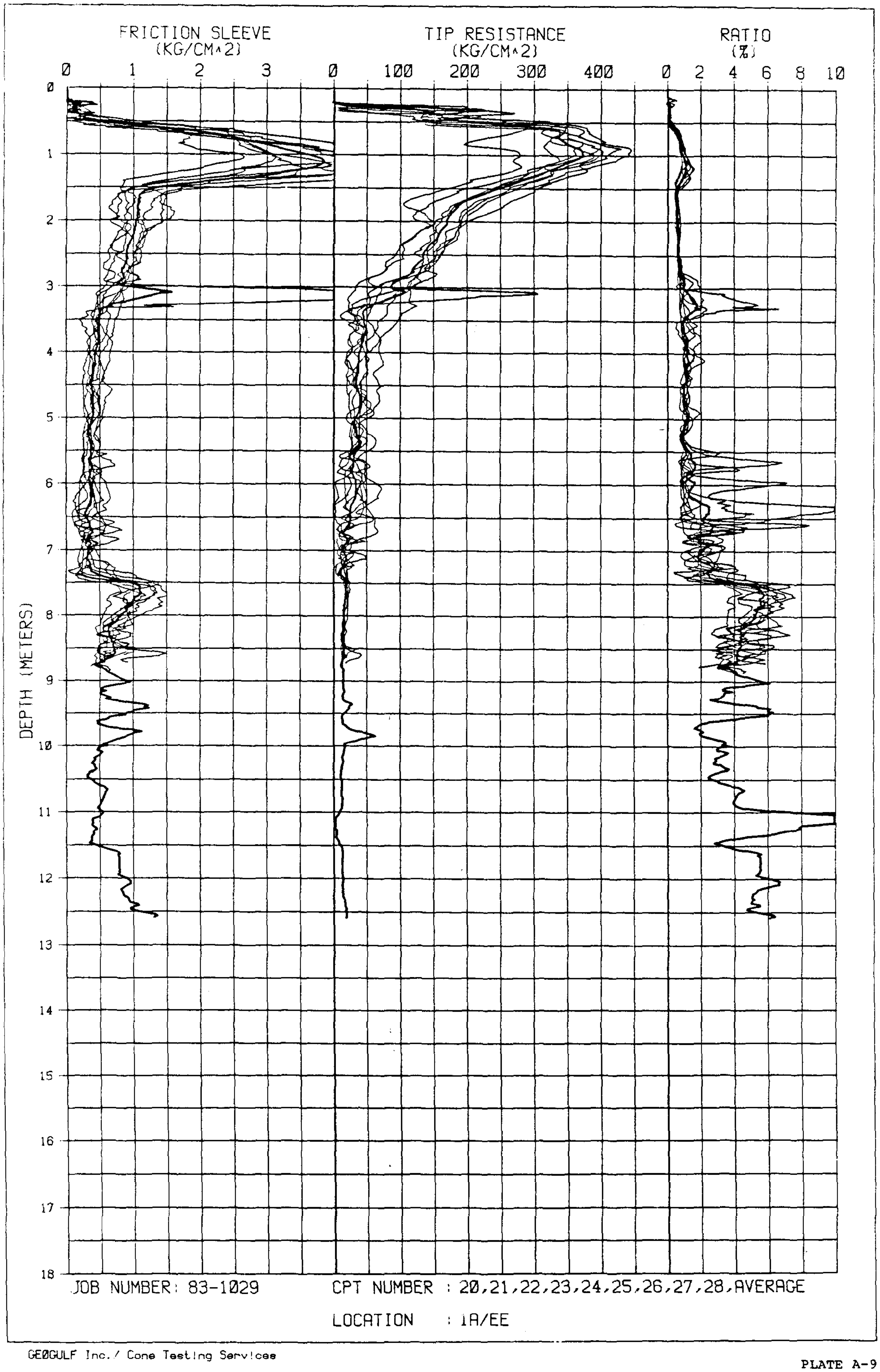


JOB NUMBER: 83-1029

CPT NUMBER : 11,12,13,14,15,16,17,18,19,AVERAGE

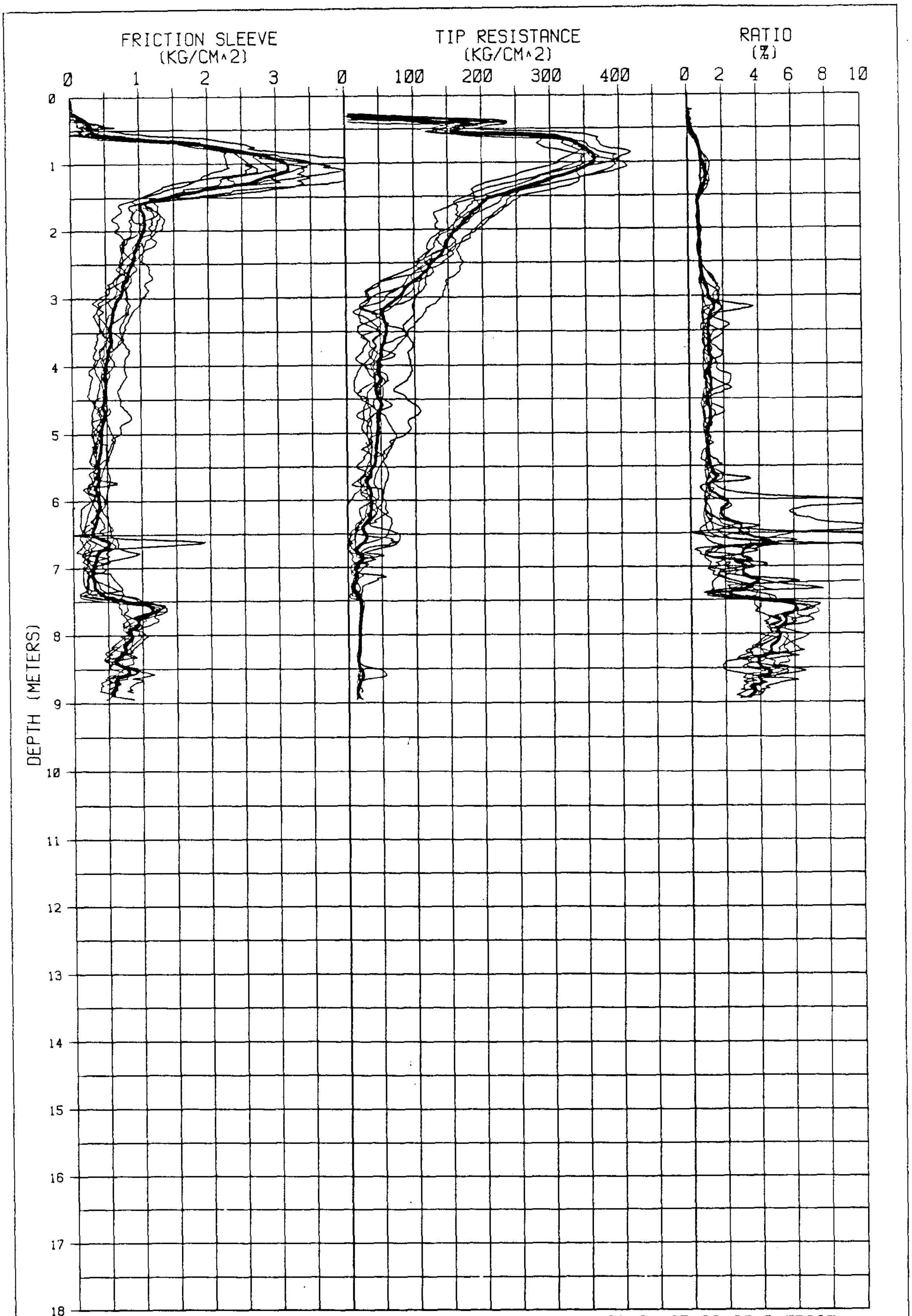
LOCATION : 1A/DD

GEØGULF Inc./ Cone Testing Services



GEØGULF Inc./ Cone Testing Services

PLATE A-9



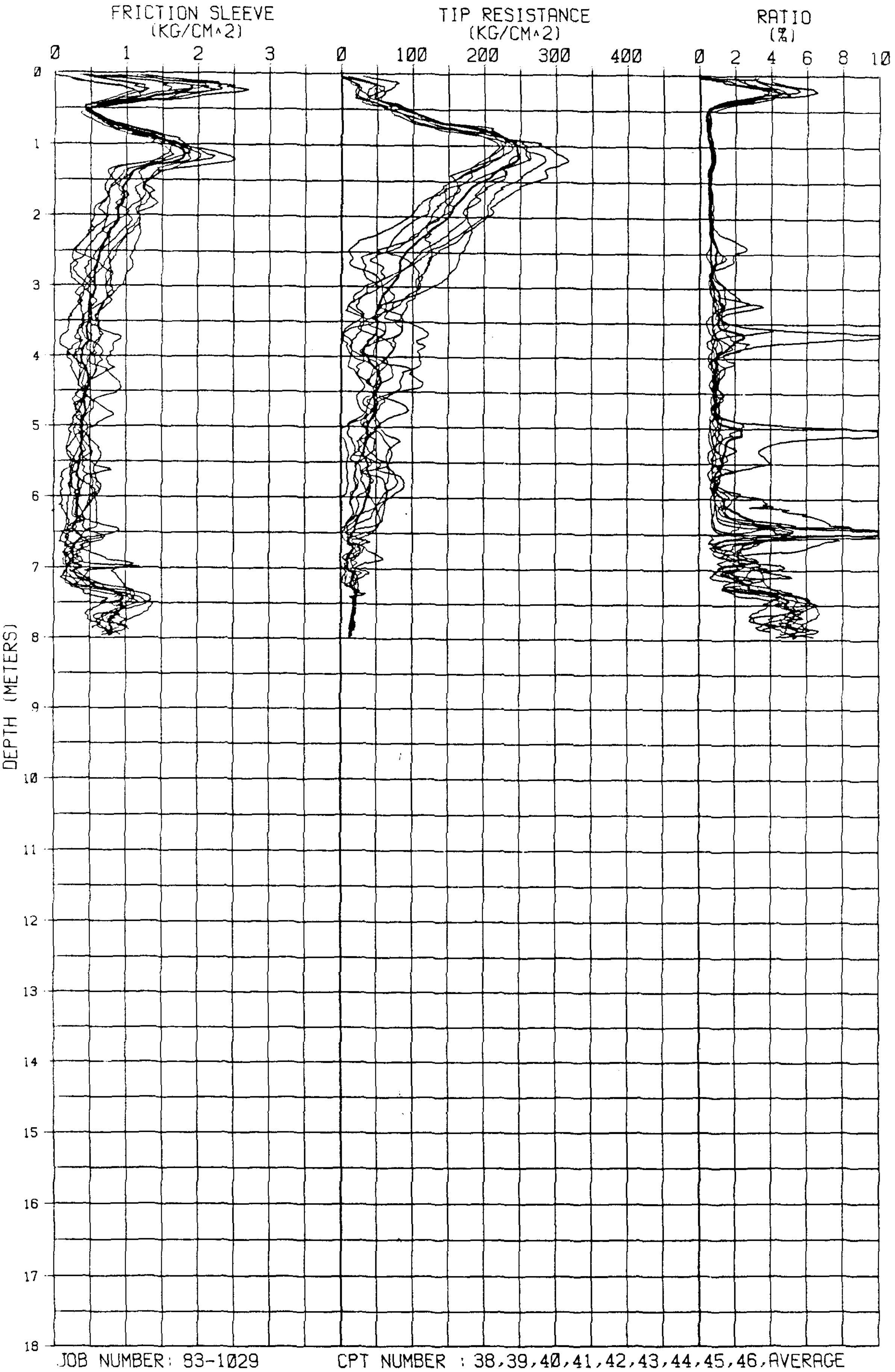
JOB NUMBER: 83-1029

CPT NUMBER : 29,30,31,32,33,34,35,36,37,AVERAGE

LOCATION : 1A/FF

GEØGULF Inc./ Cone Testing Services

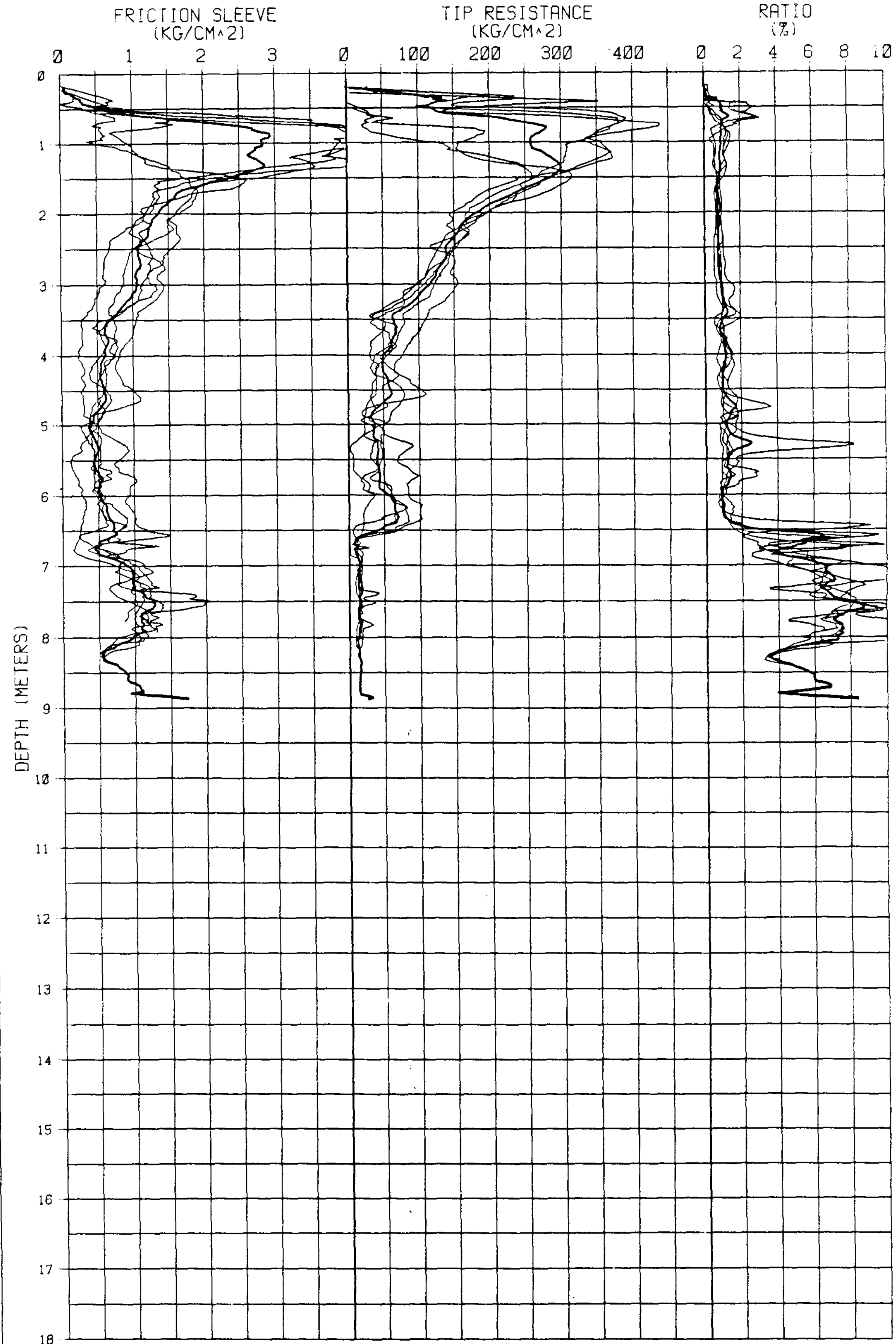
PLATE A-10



JOB NUMBER: 83-1029

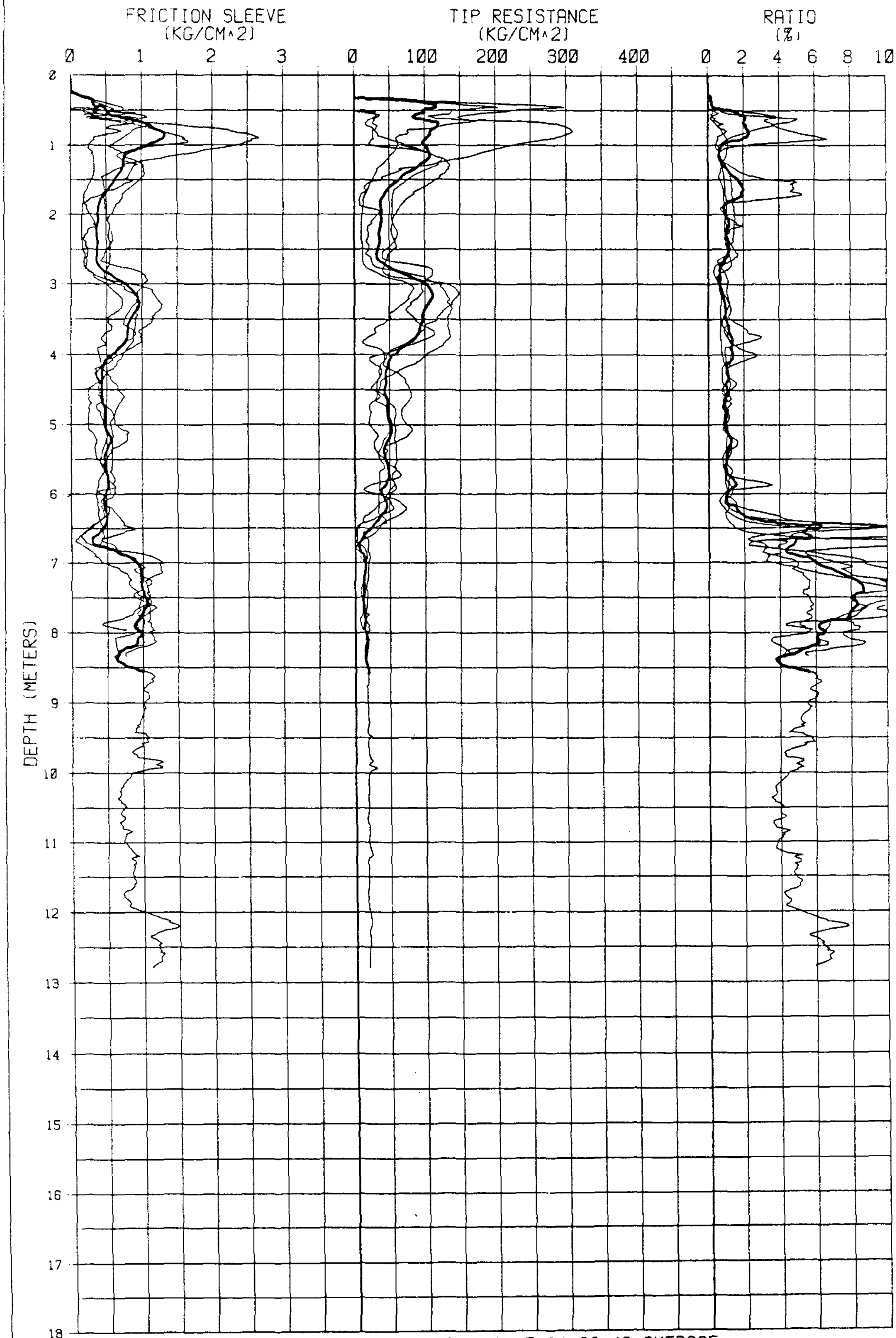
CPT NUMBER : 38,39,40,41,42,43,44,45,46,AVERAGE

LOCATION : 1A/GG



JOB NUMBER: 83-1029 CPT NUMBER : 2,11,20,29,38,AVERAGE

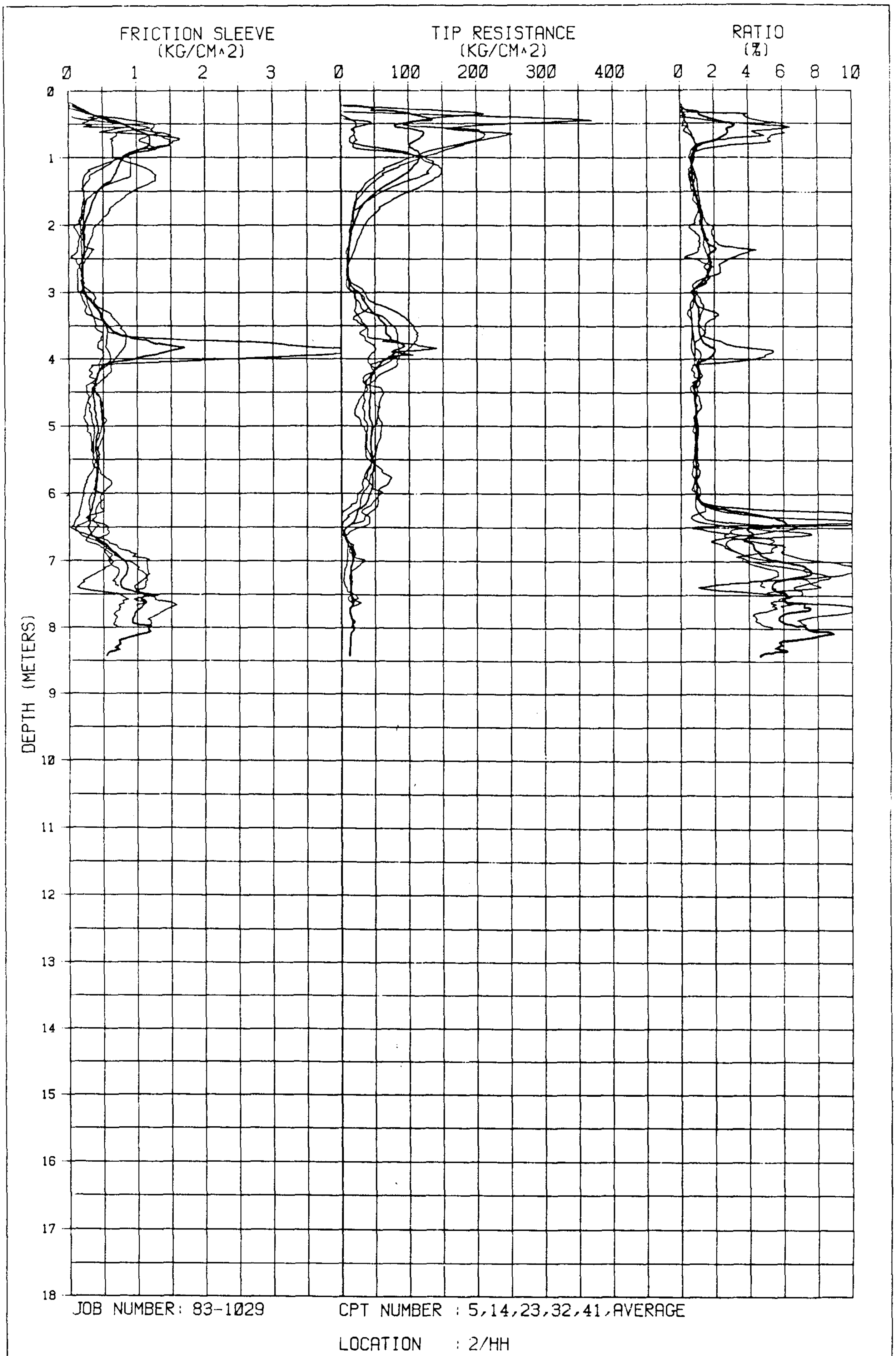
LOCATION : 2/AA

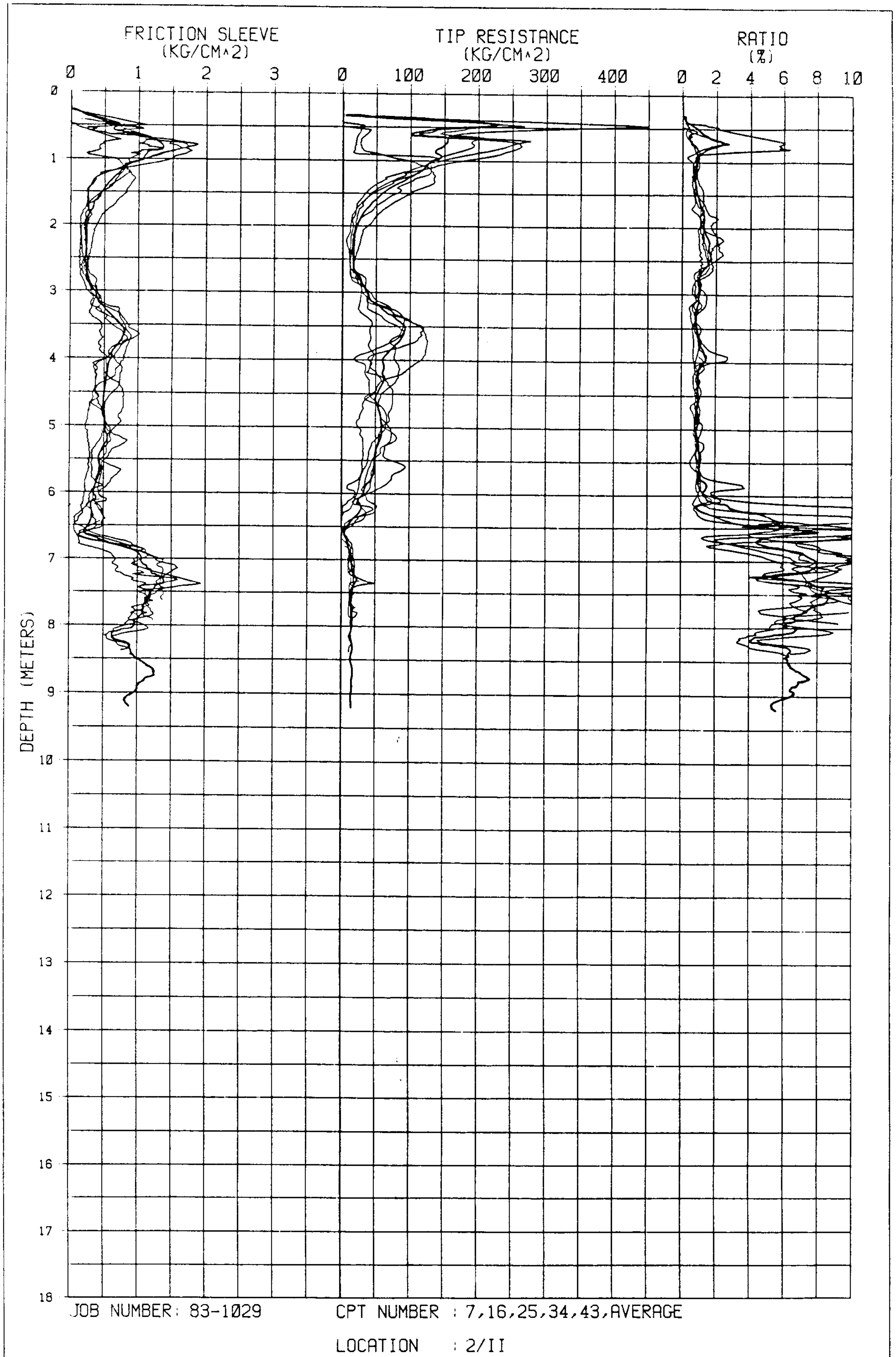


JOB NUMBER: 83-1029

CPT NUMBER : 6,15,24,33,42,AVERAGE

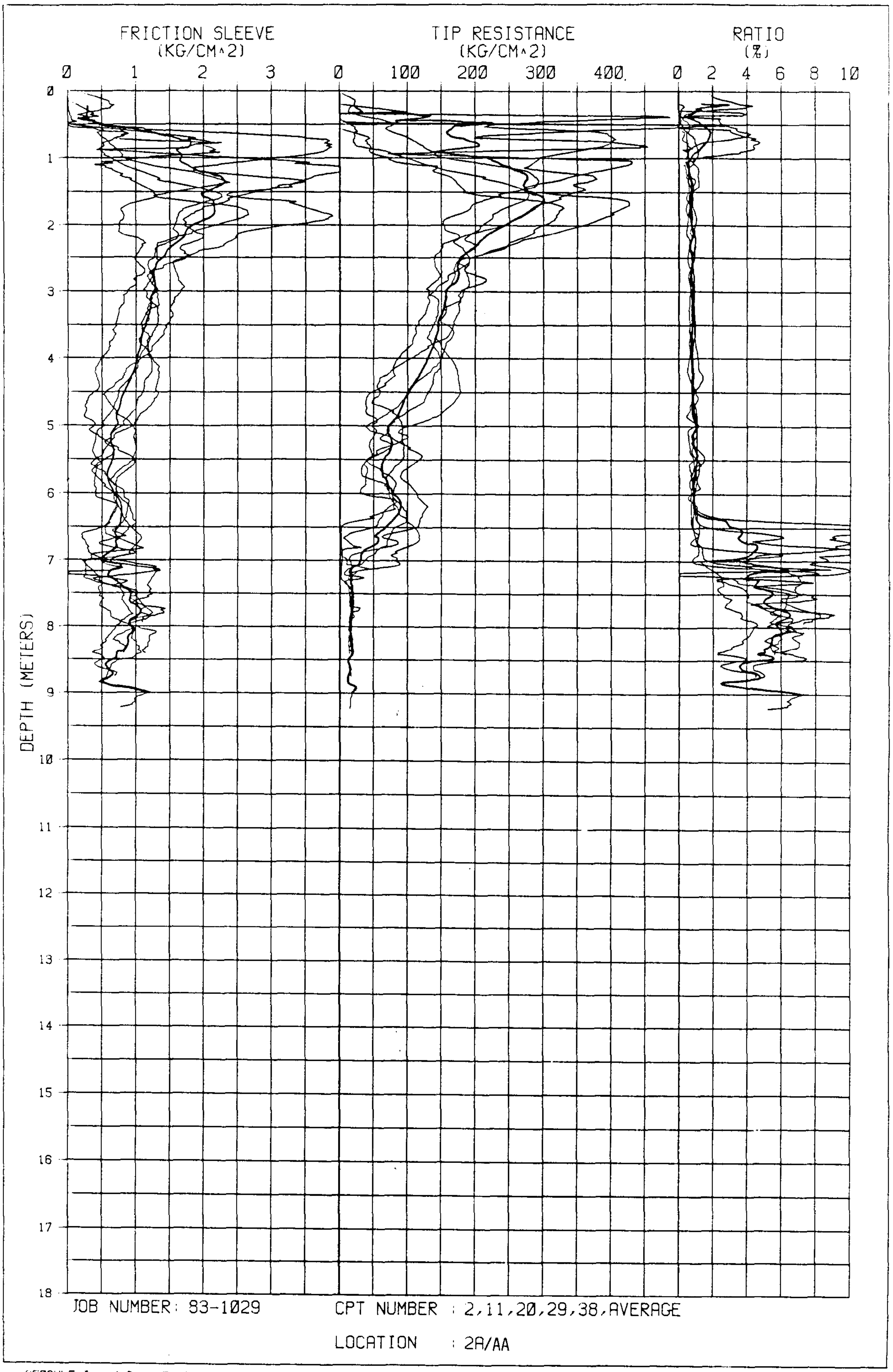
LOCATION : 2/BB


FRICTION SLEEVE
(KG/CM^2)
TIP RESISTANCE
(KG/CM^2)
RATIO
(%)
DEPTH (METERS)
JOB NUMBER: 83-1029
CPT NUMBER : 5,14,23,32,41,AVERAGE
LOCATION : 2/HH

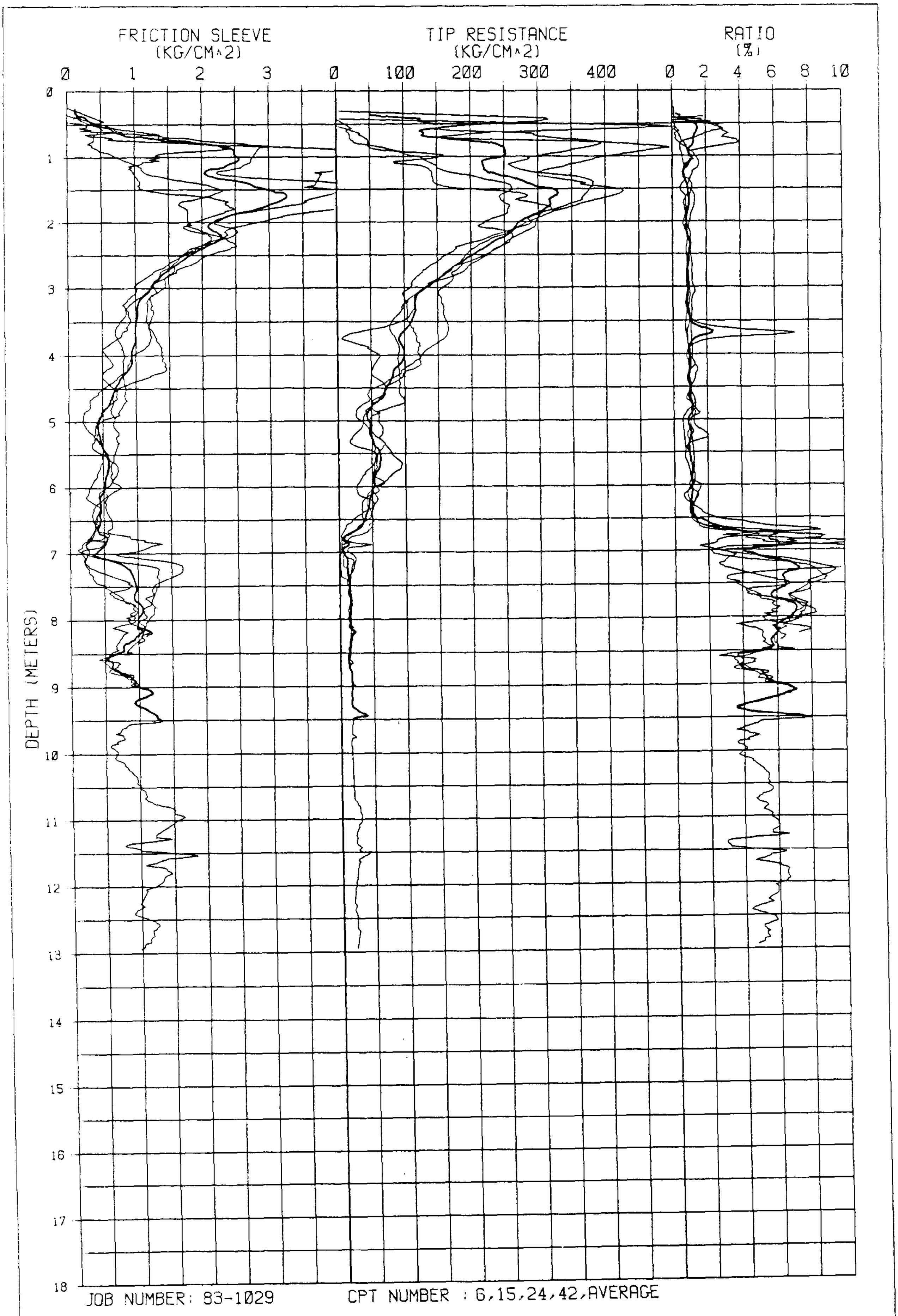


GEØGULF Inc./ Cone Testing Services

PLATE A-15

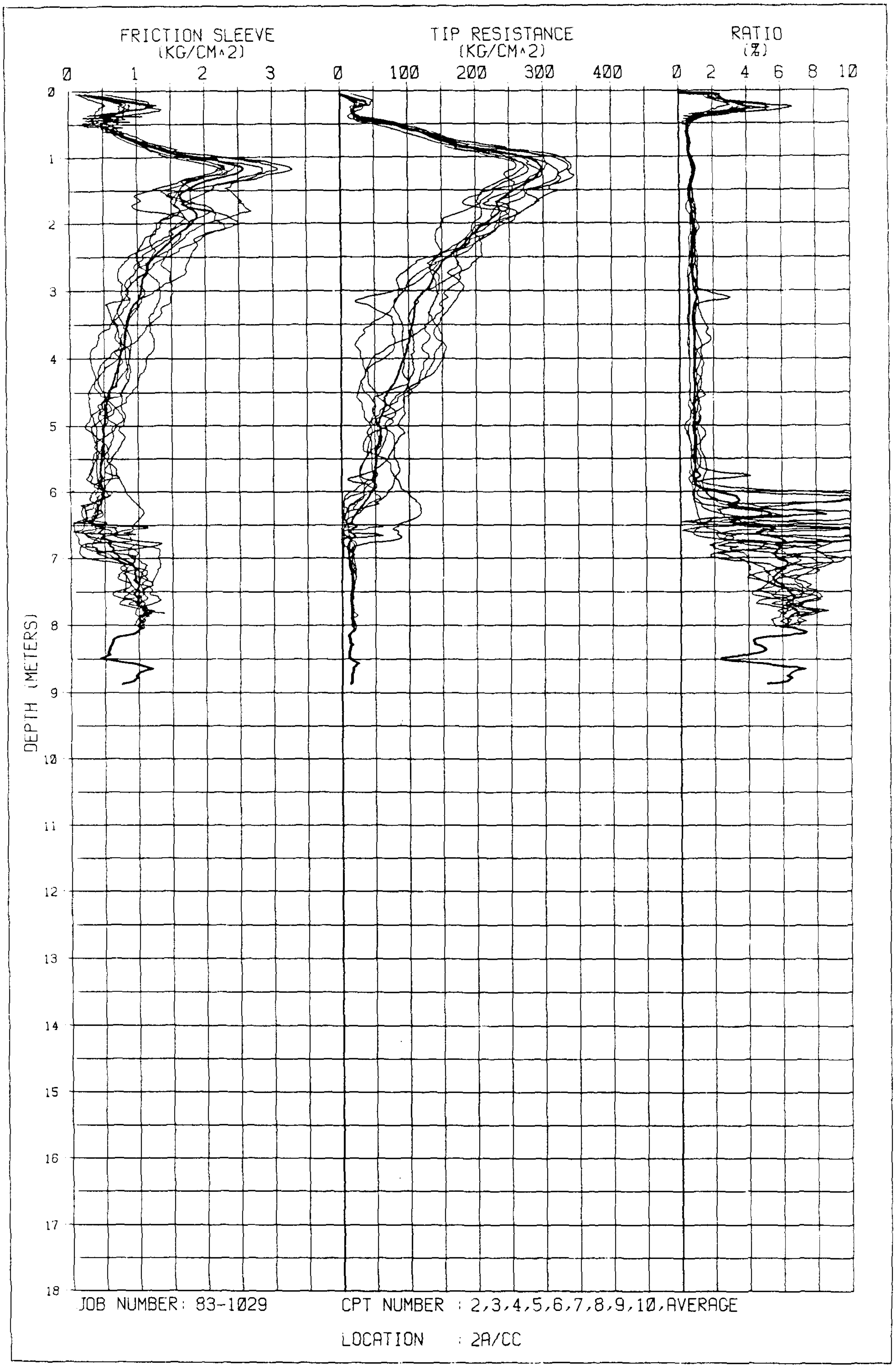


GEOGULF Inc./ Cone Testing Services



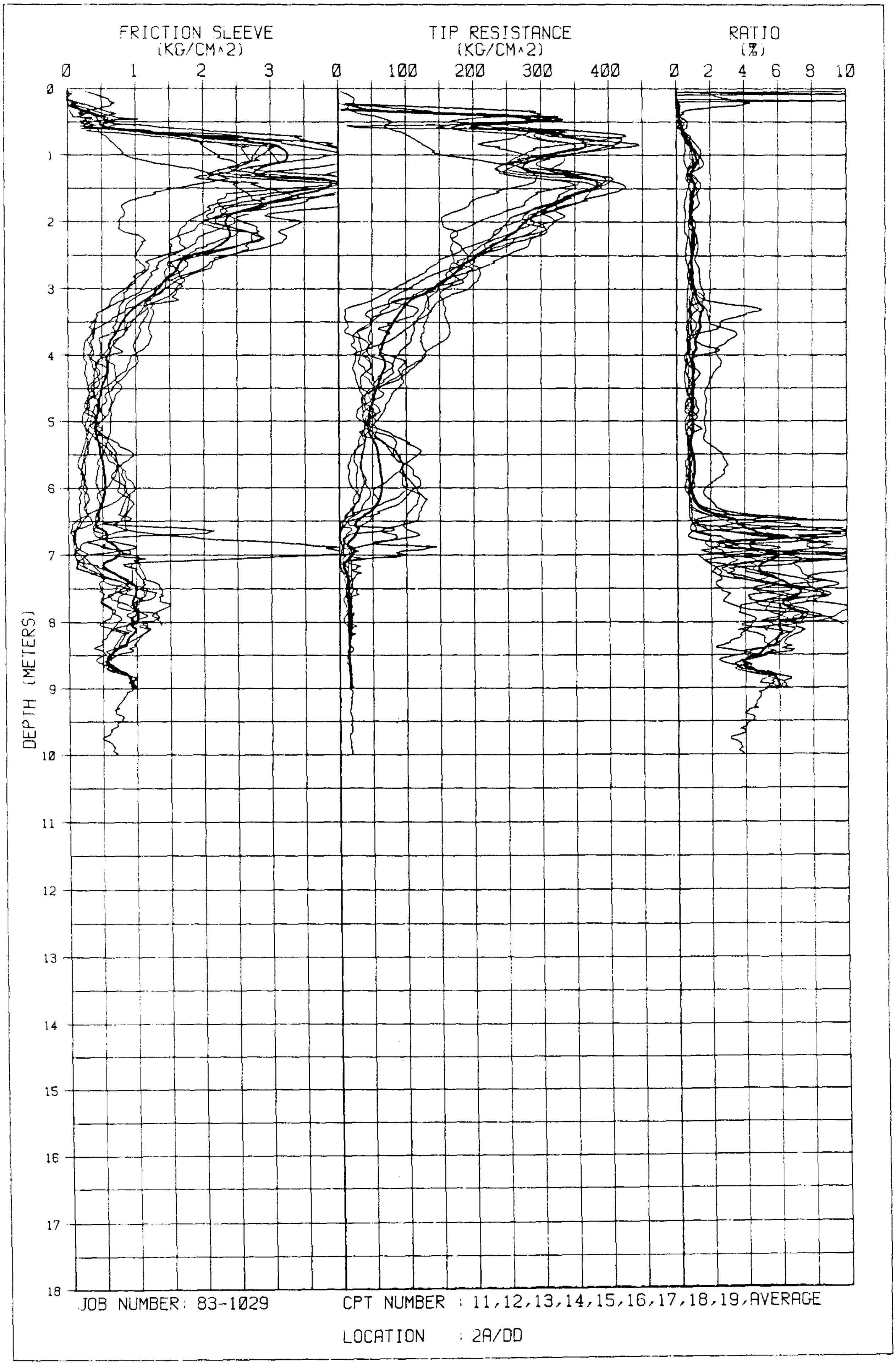
GEØGULF Inc. / Cone Testing Services

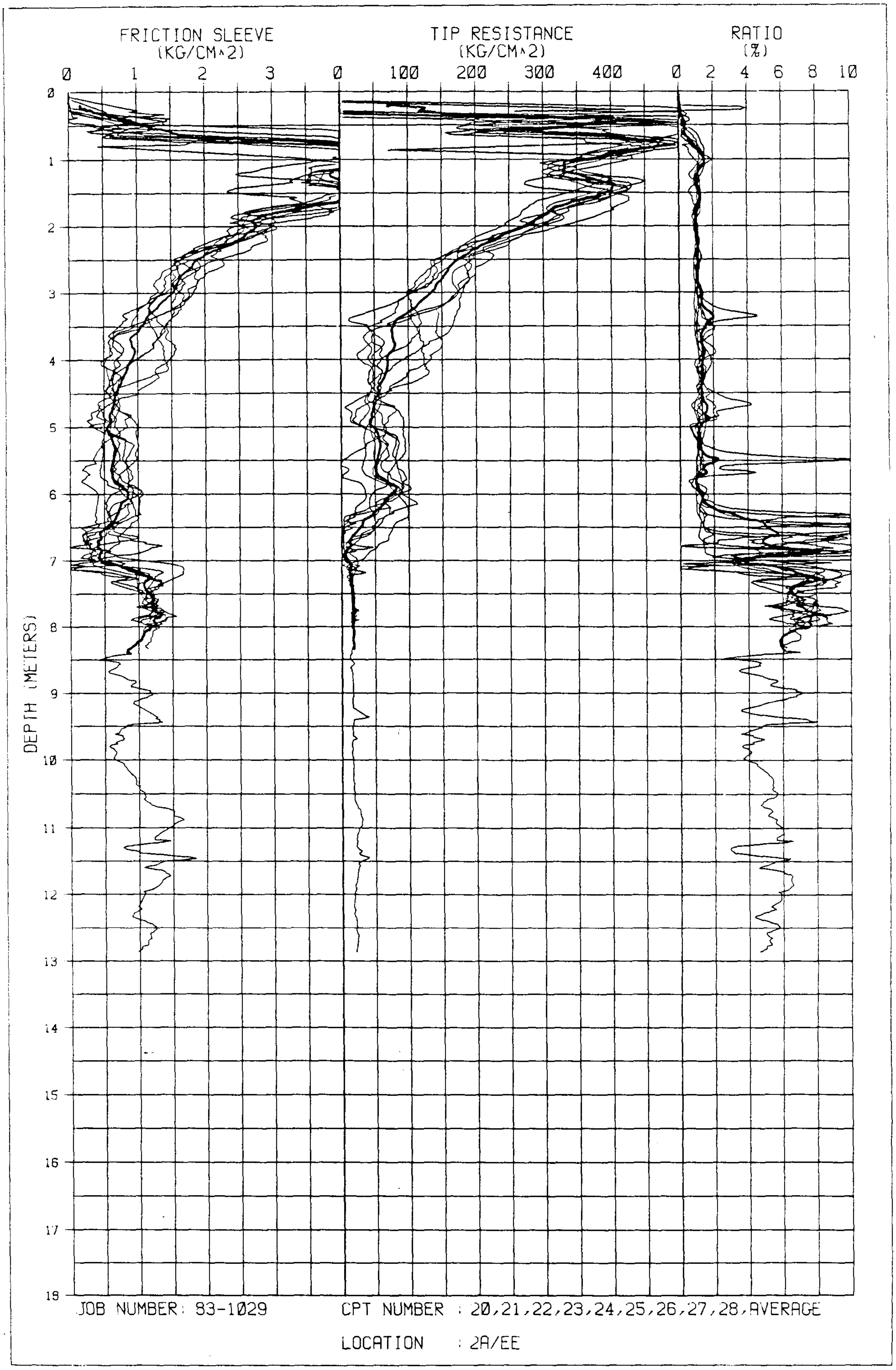
PLATE A-17



GEOGULF Inc. / Cone Testing Services

PLATE A-18


FRICTION SLEEVE
(KG/CM^2)
0
1
2
3
TIP RESISTANCE
(KG/CM^2)
0
100
200
300
400
RATIO
(%)
0
2
4
6
8
10
DEPTH (METERS)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
JOB NUMBER: 83-1029
CPT NUMBER : 11,12,13,14,15,16,17,18,19,AVERAGE
LOCATION : 2A/DD



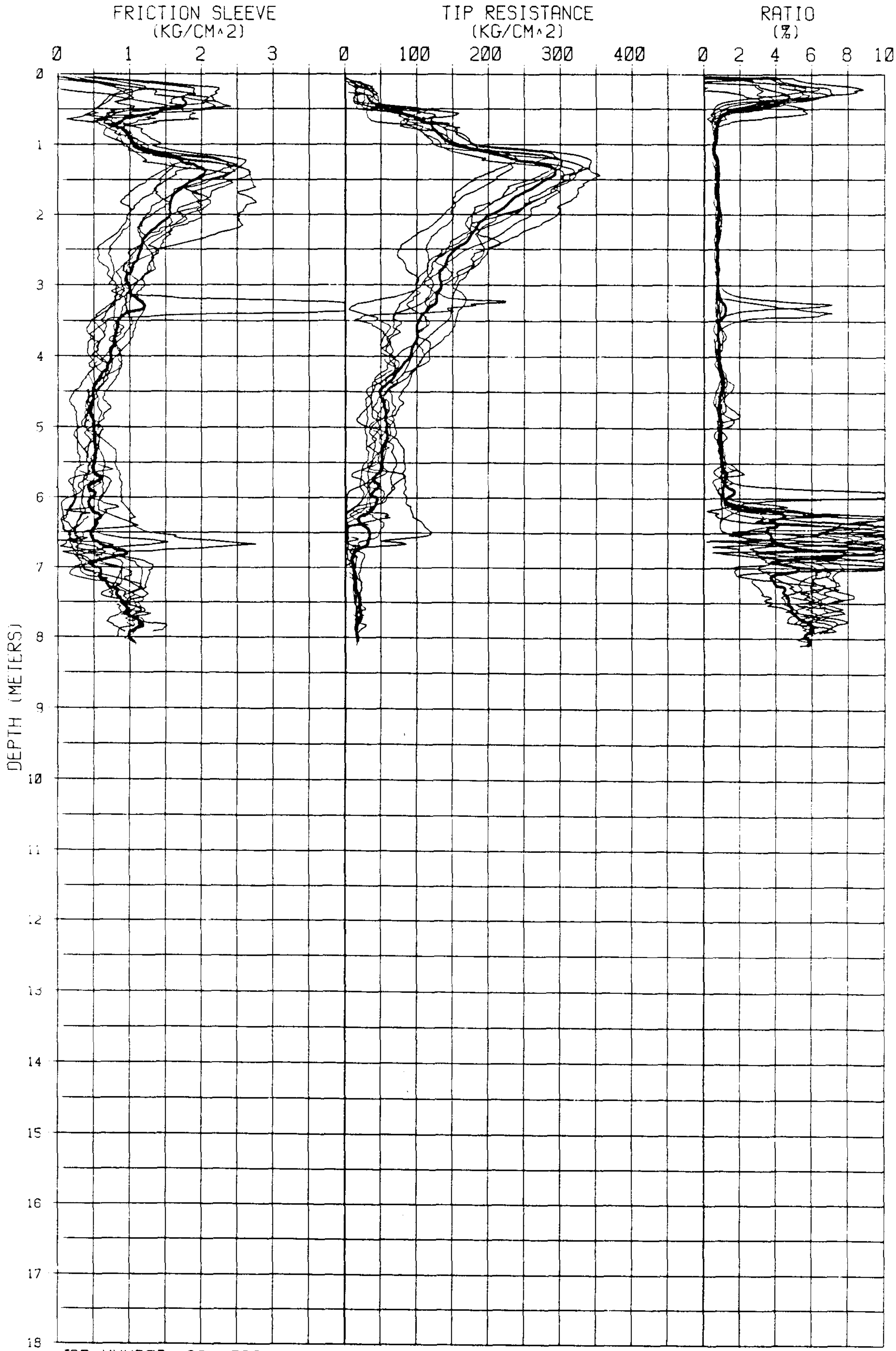
GEØGULF Inc. / Cone Testing Services

PLATE A-20

JOB NUMBER: 83-1029

CPT NUMBER : 29,30,31,32,33,34,35,36,37,AVERAGE

LOCATION : 2A/FF



JOB NUMBER: 83-1029

CPT NUMBER : 38,39,40,41,42,43,44,45,46,AVERAGE

LOCATION : 2A/GG

GEOGULF Inc./ Cone Testing Services