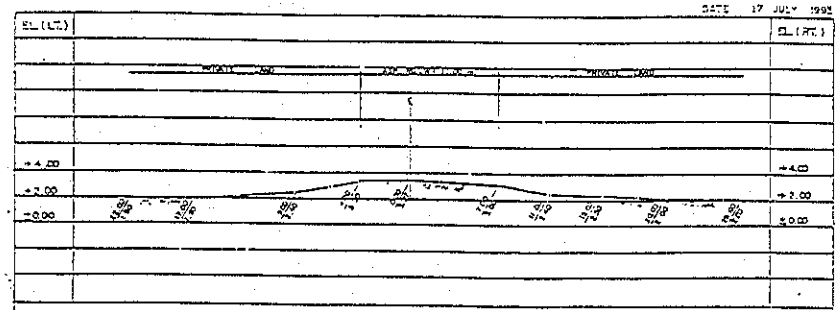
DATE 17 JULY 1993



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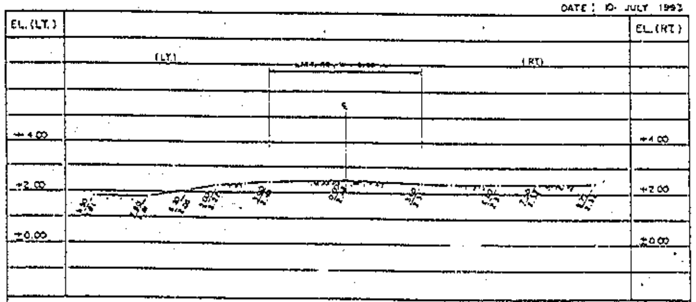

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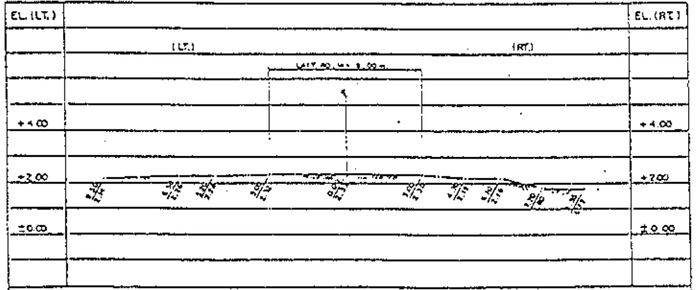

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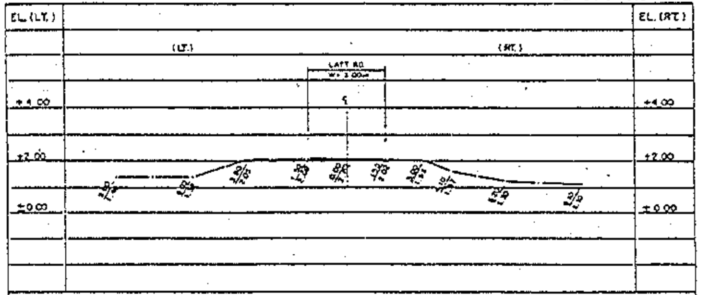

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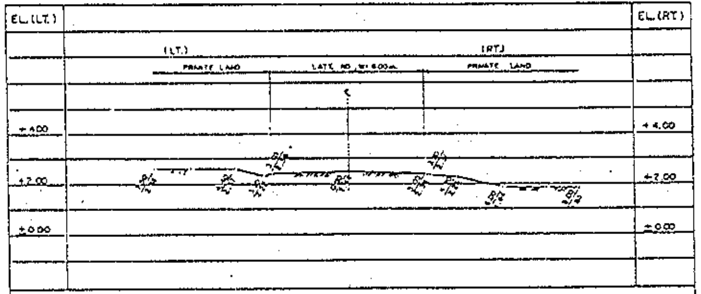

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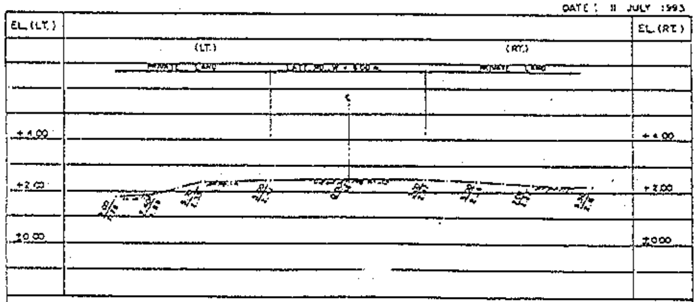

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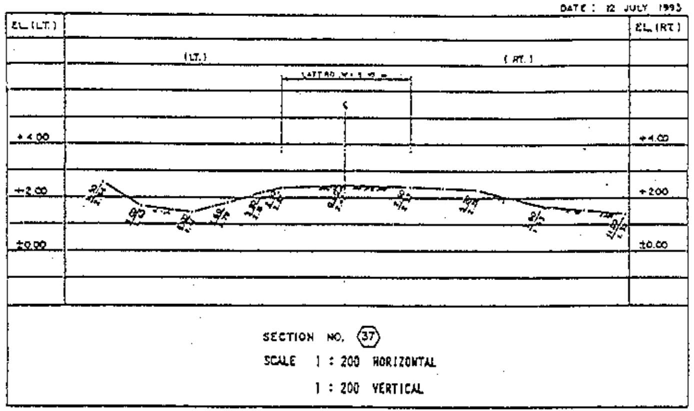

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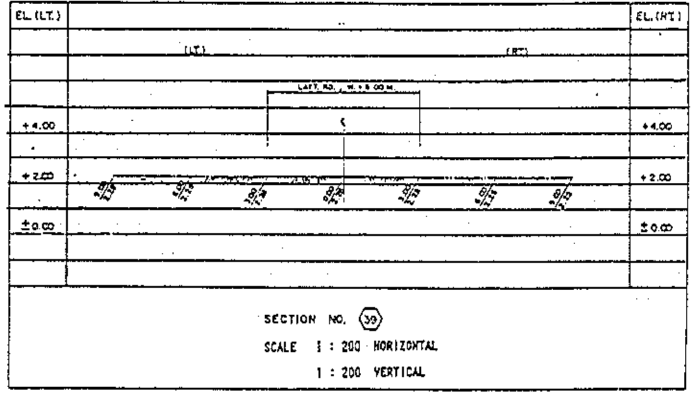

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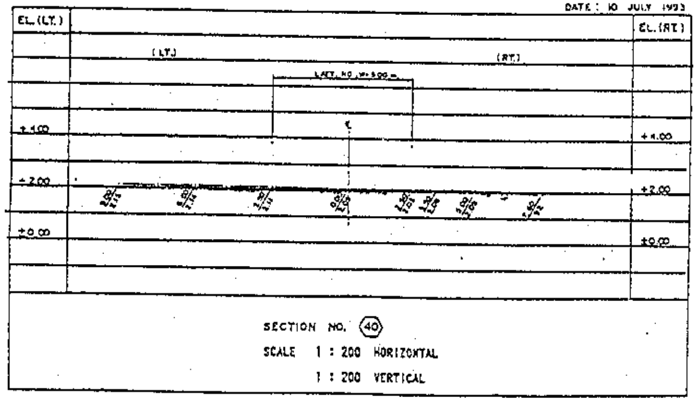

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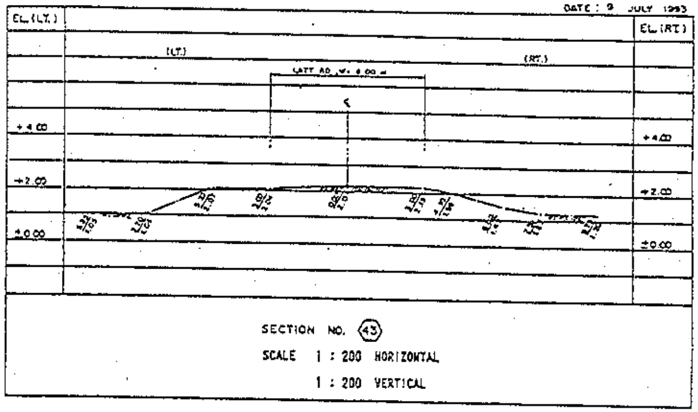

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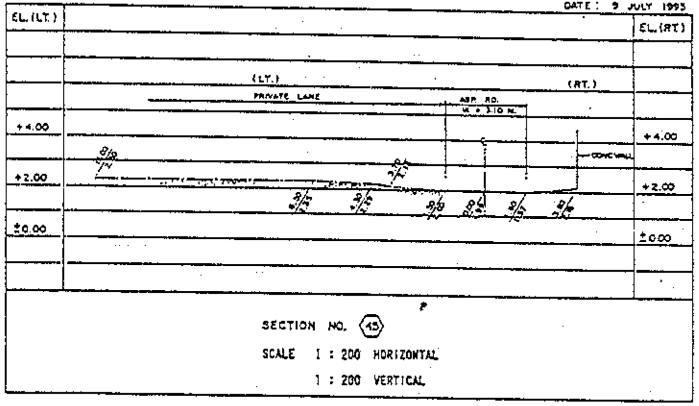

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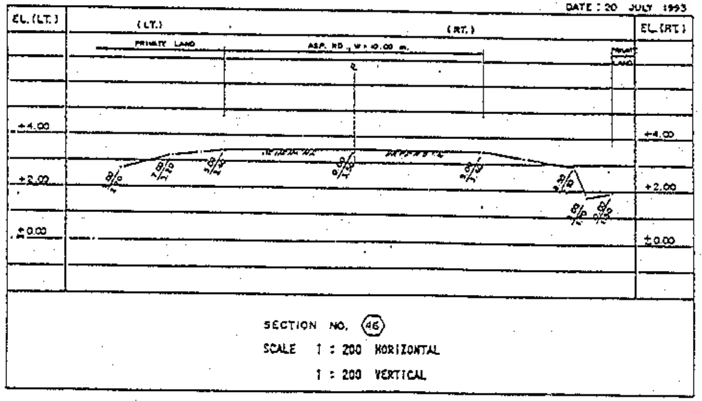

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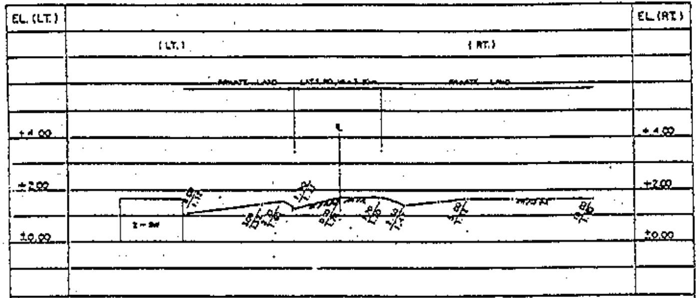

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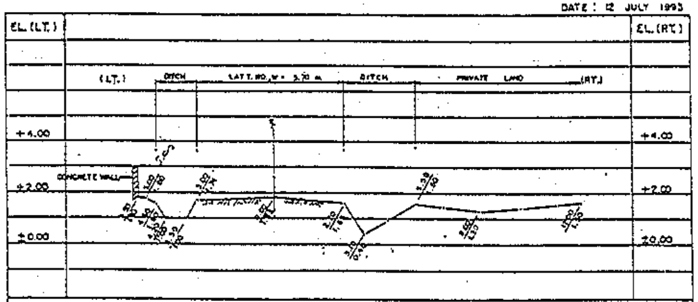

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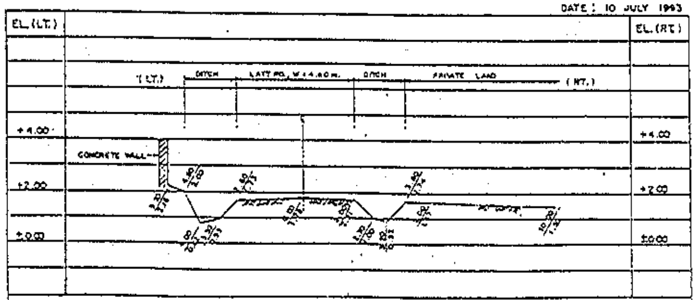

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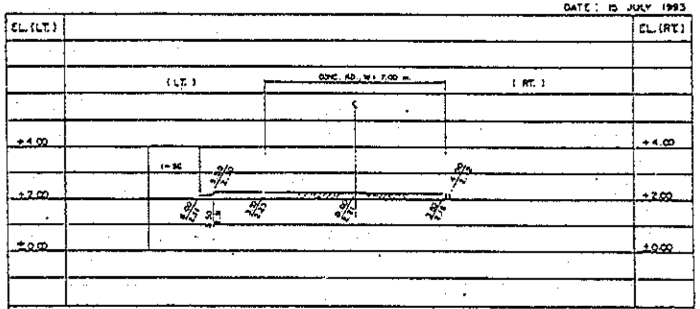

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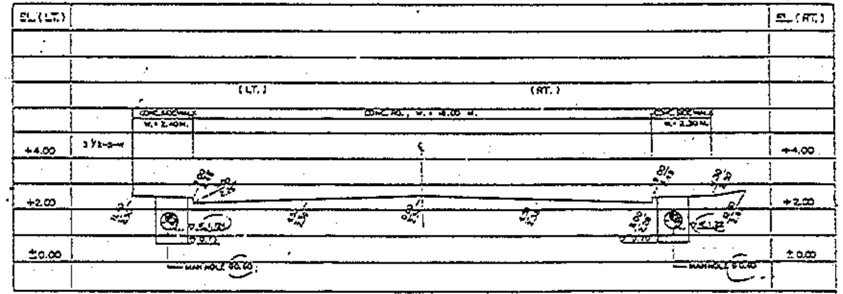

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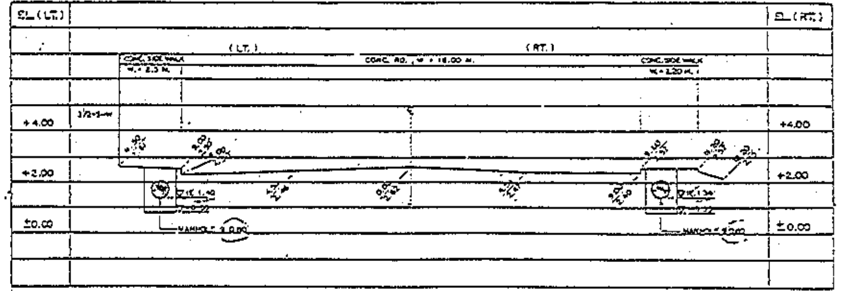

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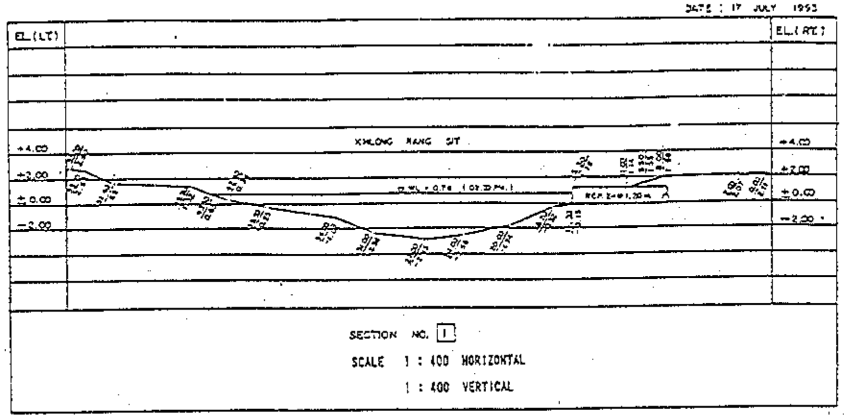

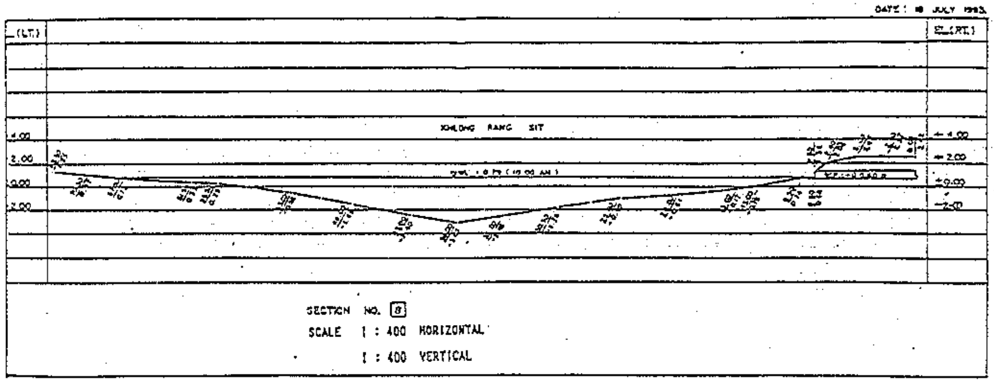

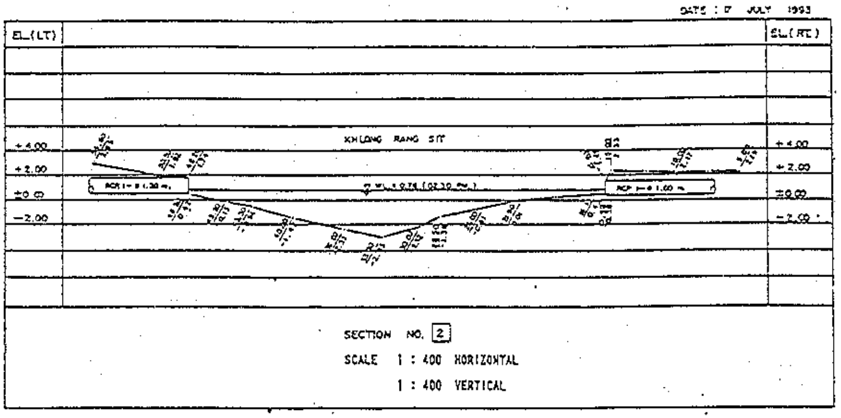

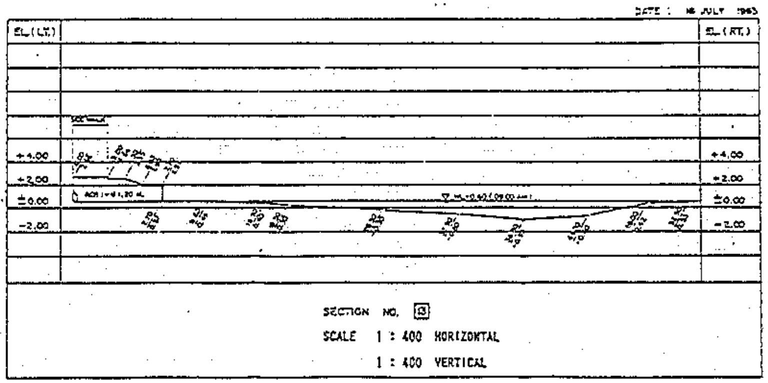

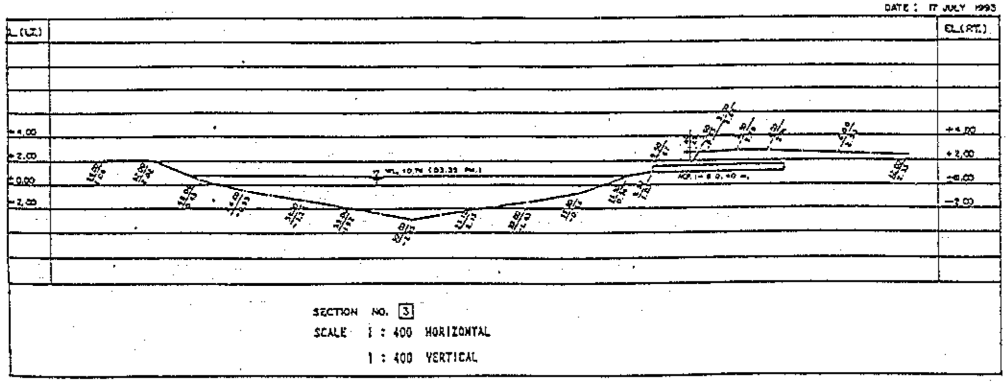

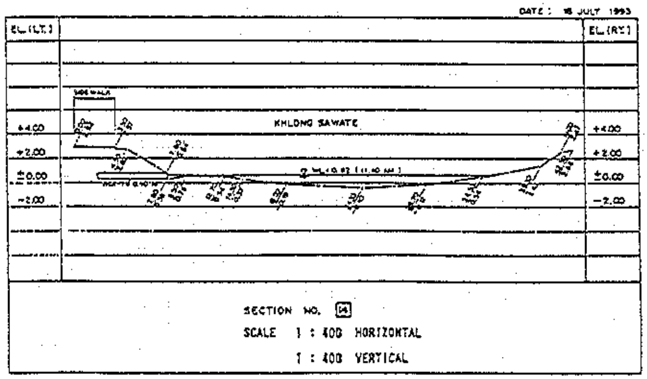

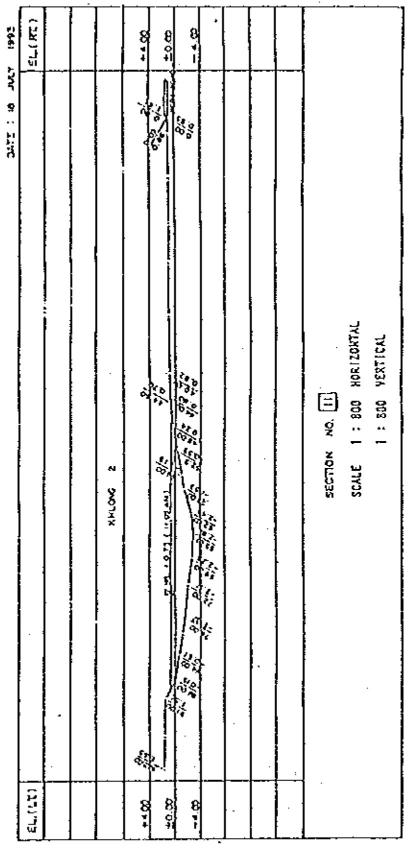

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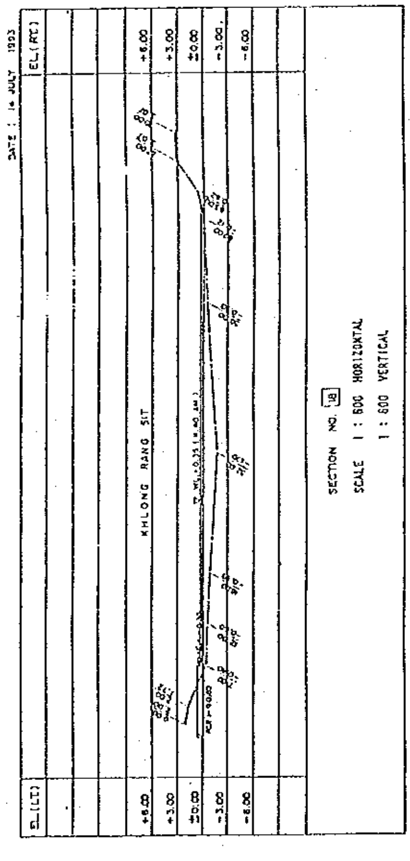

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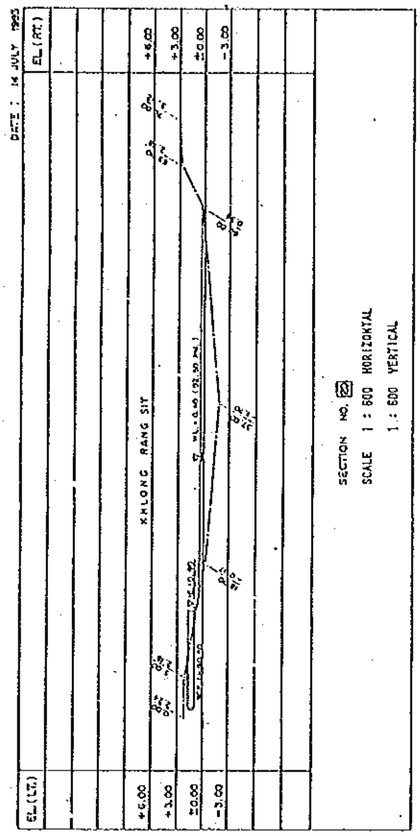

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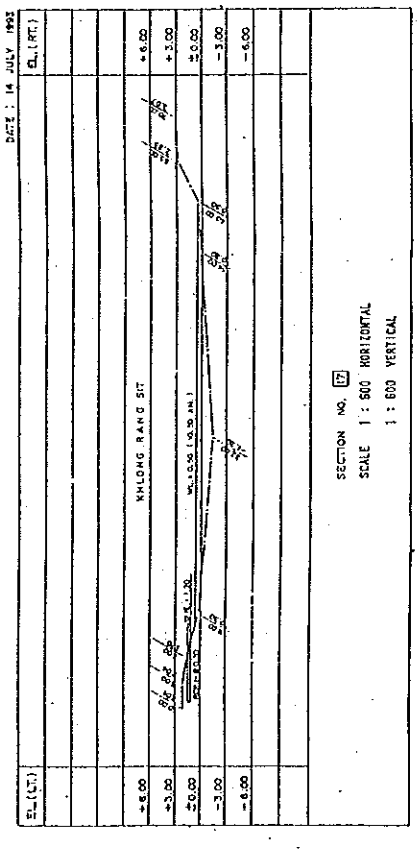

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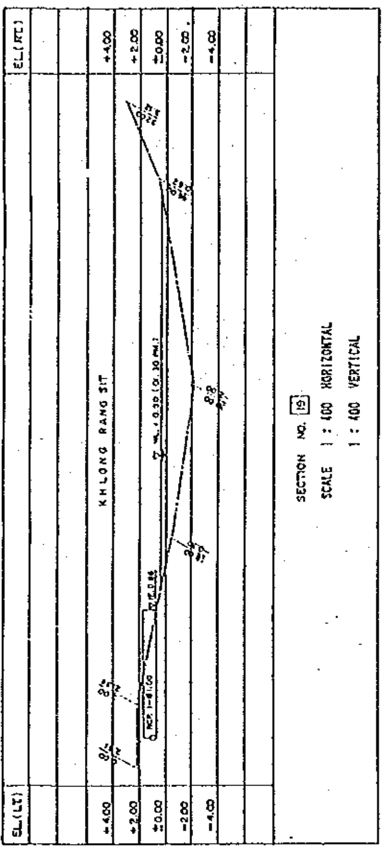

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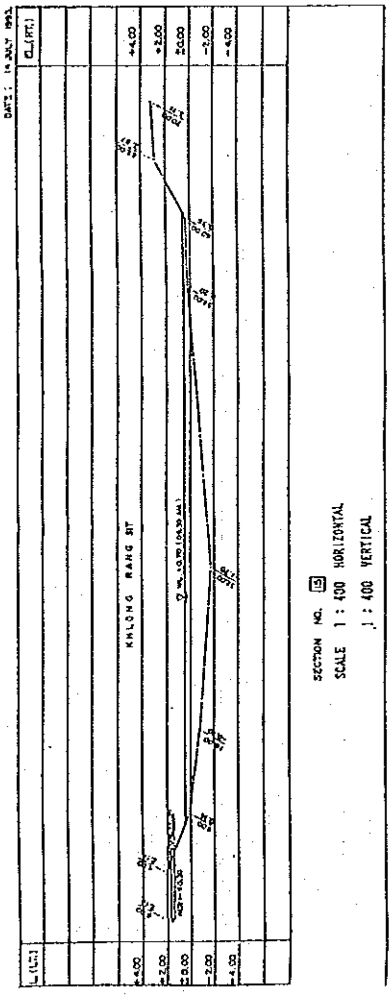

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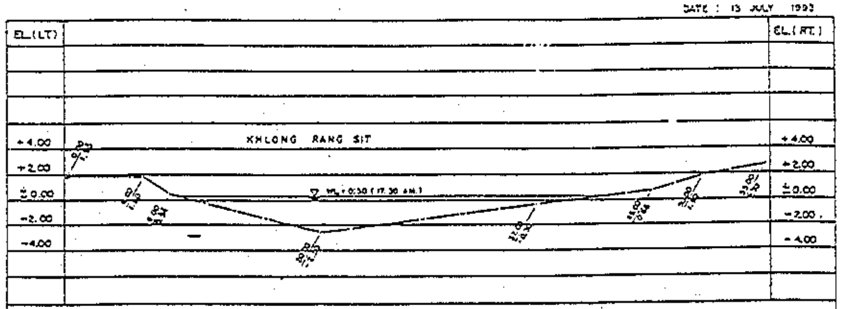
(3) Cross section of Klongs



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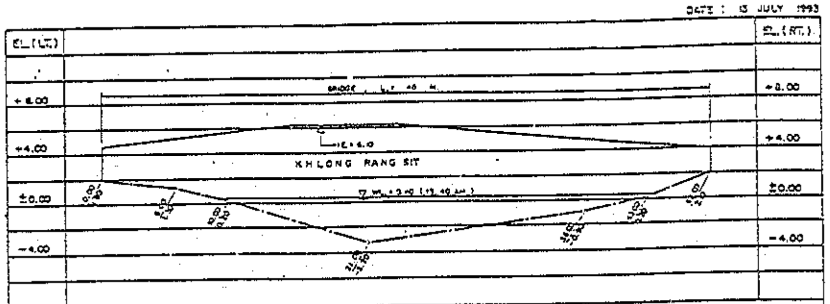

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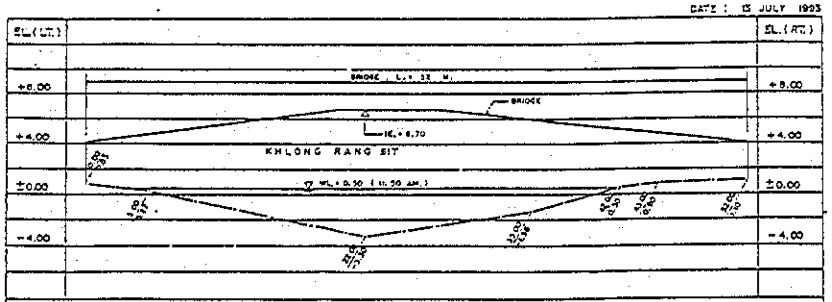

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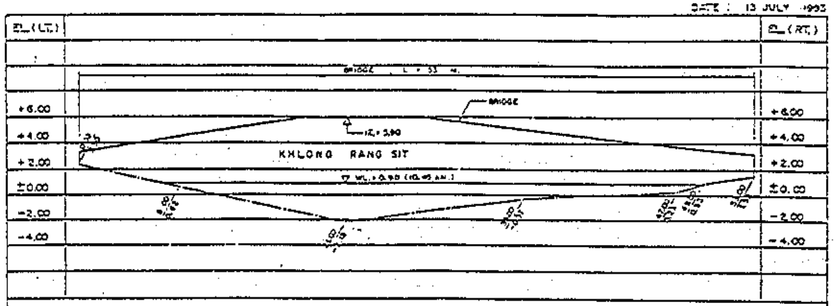

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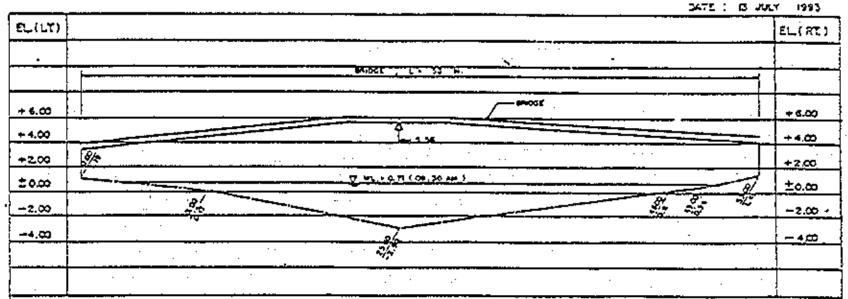

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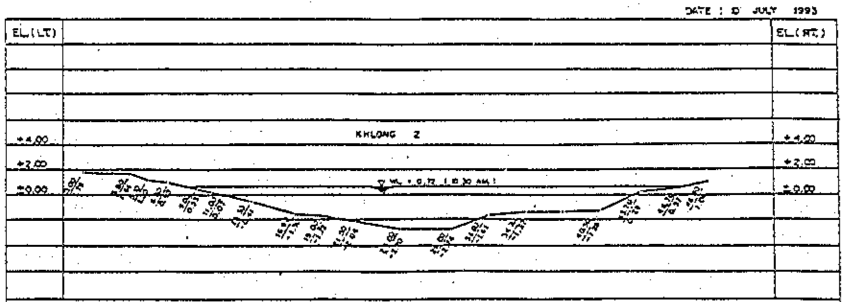

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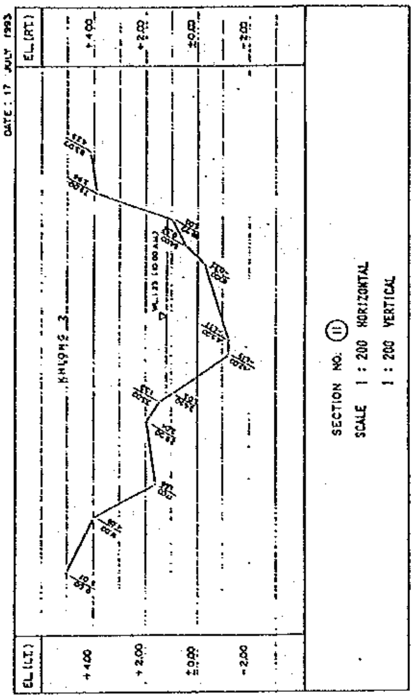

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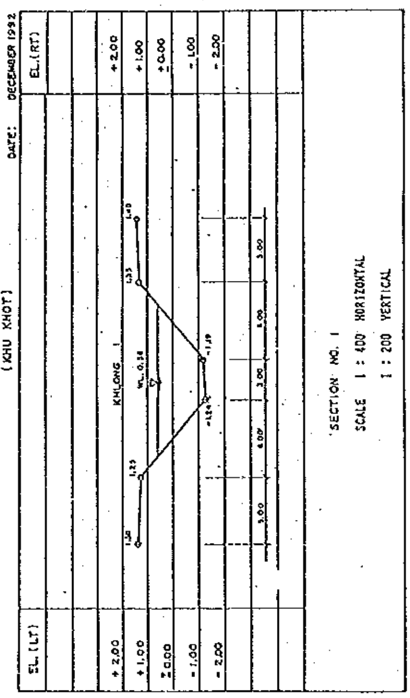

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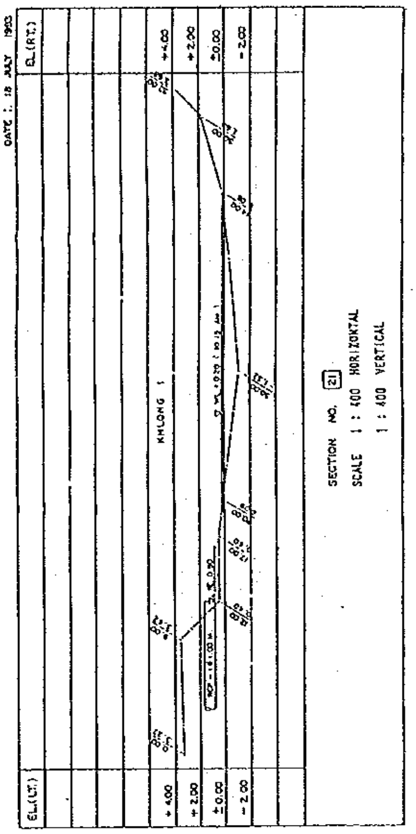

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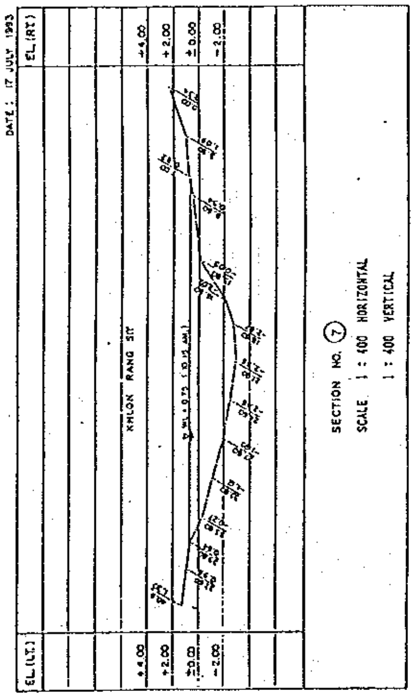

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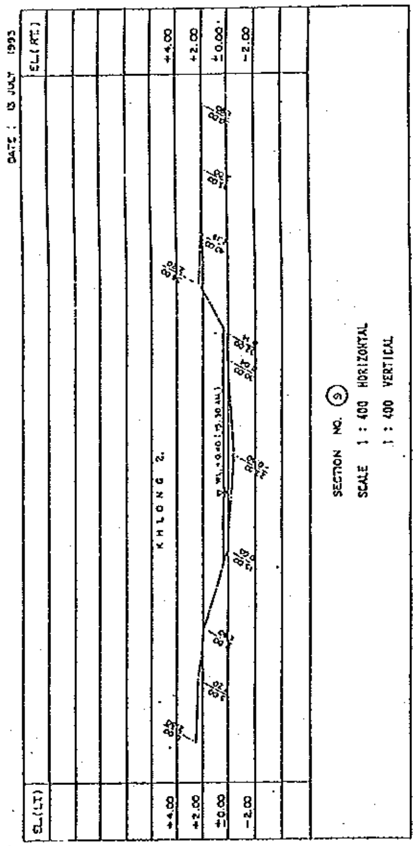

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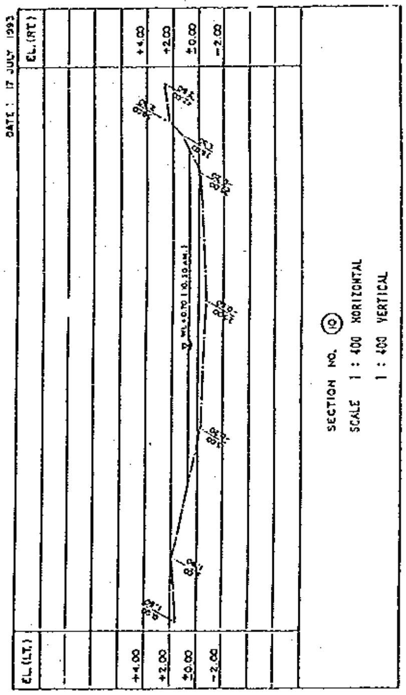

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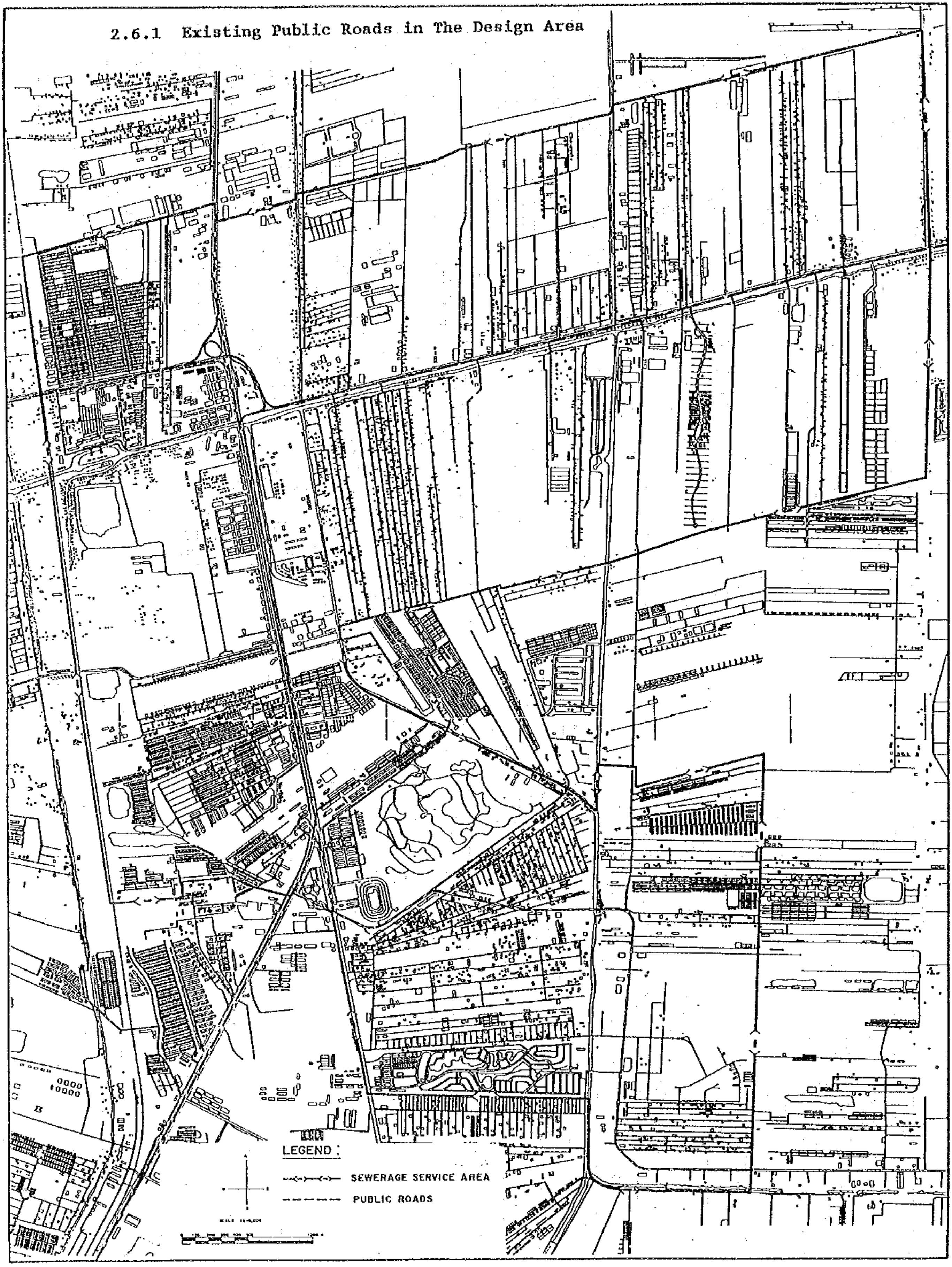
2.6.1 Existing Public Roads in The Design Area



2.7.1 Soil Boring Test Results

1. INTRODUCTION

This report described the scope and results of the soil investigation carried out on the site of the proposed Water Treatment Plant which is located in Amphoe Klong Luang, Pathum Thani Province. The purpose of the investigation was to estbalish the types and characteristics of the subsoil strata, and to enable recommendations to be made concerning foundation types, founding level, safe bearing capacity for the range of structures envisaged for storage services.

The investigation was carried out during the month of July, the dry season so that it may be said that dry conditions prevailing in the subsoils.

2. SITE DESCRIPTION

The site was located in Amphoe Klong Luang, Pathum Thani Province which is in an area of the Chao Phraya River Basin of Thailand as shown in Figure 1. Most area of the site was covered with the organic top soil about 0.30 metre thick.

3. FIELD WORKS

3.1 Locations of Boreholes

Three boreholes were required to drill to about the maximum depth of 21.45 metres below existing ground surface. The exact locations of the boreholes were shown in Figure 2.

3.2 Field Tests

Standard methods for field testing soils for engineering purposes have been established by the American Society for Testing and Materials (ASTM,1958). The basic concepts are presented belows:

Standard penetration test (SPT)

In the standard penetration test a soils for engineering purposes spoon is used. It is an open-ended steel cylinder which splits longitudinally into two halves. These two halves are held together by a cutting shoe at the lower and a coupling which connects the sampler to the drill rod. The split spoon is driven 45 cm. into the ground by means of a 63.5 kg. weight (hammer) falling a free height of 76 cm. The number of hammer blows for each 15 cm. penetration is recorded. The total number of blows required to drive the second and third 15 cm. of penetration is called the standard penetration resistance N which represents mumber of blows per 30 cm. (Terzaghi and Peck, 1948). After the blow counts are recorded, the spoon is withdrawn from the borehole and a representative sample is secured. These samples are kept in airtight jars with proper identification for visual examination and or laboratory tests.

Thin-walled Tube Sampling (Shelby Tube)

For moderate to large jobs the shear strength of the cohesive soil should be determined from relatively undisturbed samples. This is usually done by taking samples from the borehole by means of a seamless thin-walled steel tube commonly known as a Shelby tube. The tube is 5.08 cm. or 7.6 cm. in deameter and has a bevelled butting edge at the lower end. It is connected to the drill rod and pushed by static force into the bottom of the hole. When the tube is almost full (avoid over-penetration), it is withdrawn from the hole, removed from the drill rod, sealed at both ends with parafin, and shipped to soil laboratory for tests.

Vane Test

A vane tester consists of a pair of thin steel blades connected to a vertical shaft. The tester is pushed into the ground or into the bottom of a borehole and a torque is applied on the shaft. If the shaft is kept free from the surrounding soil by means of a casing, the torque required to shear the soil along the cylindrical surface with blades size (Skempton, 1950).

3.3 Drilling

The rig used was a skid-mounted type, provided with equipment for carrying out augering, wash boring, vane tests and penetration tests. In all cases,the hole was cased to about 10.00 metres depth to prevent washing out of the holes and to permit more efficient water recirculation during wash boring. The procedure in the field at each borehole location commenced with an in situ vane shear test in soft soil by using a standard vane types at a depth of 1.50 metre intervals and also a standard penetration test at the same depth intervals. These results were plotted on the boring log, Figure 3 to 5.

4. SOIL TPYES ENCOUNTERED

Boreholes over the site were taken to a maximum depth of about 21.45 metres. Over this range of depth, three broad and distinct strata of materials were encountered, and are described in general terms belows:

Stratum A: This material usually appeared directly below organic top soil and extended to the depth between 10.00 to 16.00 metres below existing ground surface around an area of Boreholes. It consists of silty clay, with occasional organic matters and very fine sand, varying in color from brown to dark grey. The consistency as measured by a vane device, was in the range very soft to medium. The soil in this stratum was classified as CL, ML-OL, MH-OH & CH.

Stratum B: This material was encountered beneath Stratum A and extended to a depth between 17.50 to 19.40 metres below existing ground surface around an area of Boreholes. It consists of silty clay, with occasional very fine sand and decomposed rock, varying in color from dark to light grey, yellow to brown. The consistency as measured by a pocket penetrometer, was in the range stiff to hard. The soil in this stratum was classified as CL and CH.

Stratum C: Lying beneath Stratum B and extended to the end of the boreholes at a maximum depth of about 21.45 metres below existing ground surface around an area of the boreholes. It consists of very to fine sand with occasional coarse sand and gravel, varying in color from light grey, yellow to brown. Relative density as measured by a split spoon, was in the range dense to very dense. The soil in this stratum was classified as SM & SW-SM.

5. LABORATORY WORKS

Standard methods for testing soils for engineering purposes have been established by the American Society for Testing and Materials (ASTM,1958). The basic concepts of the more common tests are presented below. Details of the testing programme carried out in the laboratory, using selected samples, are listed below. Results of the tests are given in Section 6 and Appendix.

5.1 Unit weight

Unit weight of a granular soil is difficult to determine except where the soil is at the ground surface. Granular soil recovered by a sampler is highly disturbed and gives no indication whatsoever of its original unit weight. In practice, the unit weight of soils is estimated from the results of penetration tests.

Unit weight of a cohesive soil however, can be readily determened by measuring the weight and volume of the soil sample. The unit weight of a plastic clay may be computed on the assumption that the clay is 100 per cent saturated.

5.2 Grain size analysis

Grain size distribution of a soil can be determined by sieve analysis down to the size of No. 200 sieve. For determination of smaller fractoins, the wet method must be used. A soil sample is dispersed thoroughly in distilled water. The soil-water mixture is well shaken so that all soil grains are in suspension. By means of a hydrometer, the density of the suspension can be determined. Correlation between the density of the suspension and the diameter of the grains has been worked out on the assumption that all grains are spherical. Results are given in Section 6 and Appendix.

5.3 Water content

The natural water content of a soil sample is determined by weighting the sample before and after it is dried in the oven under controlled temperature.

The natural moisture contents of the majority of the samples recovered were measured in order to determine a moisture content profile. Values are given in Section 6 and Appendix.

5.4 Atterberg Limits

Liquid Limit

The liquid limit of a soil is the water content at the boundary between the liquid and plastic states. A soil sample (with grains passing Nó. 40 sieve) is thoroughly mixed with water and is placed in the dish to a thickeness of 2.54 cm. at the bottom of the dish. A groove of 1.27 cm. width is cut in the middle of the sample. The dish is lifted and dropped by turning the crank. The number of drops required to closed this 1.27 cm. groove is recorded. The liquid limit is the water content at which 25 drops of the dish will close the 1.27 cm. groove.

Plastic limit

The plastic limit of a soil is the water content at the boundary between the plastic and semisolid states. The water content at the boundary is arbitrarily defined as the lowest water content at which the soil can be rolled into threads 3.2 mm. in the diameter without the threads breaking into pices.

Plastic and liquid liumits of the majority of the cohesive soil samples recovered were measured in order to determine the different states of soils along the depth. Values are given in Section 6 and Appendix.

5.5 Specific Gravity

The specific gravity of a substance is the ratio of its weight to the weight of an equal volumn of water. The specific gravity of a mass of soil or rock (including air, water, and solids) is termed mass specific gravity or apparent specific gravity.

Measurement using a standard density bottle were taken on the majority of the recovered samples. Results are shown in Section 6.

5.6 Unconfined compression test

A relatively undistubed soil sample, usually secured by means of a thin-walled tube, is subjected to an axial compression in a manner similar to the test of a concrete cyinder. For plastic clays, the unconfined compression strength is taken at 20 percent strain of the sample. The sample of a stiff soil, however, will break before reaching the 20 percent strain, for most practical cases,the shear strength of a cohesive soil may be taken as one-half of its unconfined compression strength.

5.7 Direct shear test

The test is conducted by means of a shear box or other variations of this aparatus. A shear box is a sample container which is split in the mid-height. When a normal force N is applied the forec required to start the movement of the upperhalf of the sample withrespect to the lower half is measured. This test is very useful in measuring the relationship between the shear strength and the angle of internal friction of granular soil. The sand samples were selected for the Direct shear test. Results of all the above tests are given in Section 6 and Appendix.

6. TEST RESULTS

A summary of values obtained from laboratory works, are given belows:

6.1 Natural Water Contents

	Range(%)	Average(%)
Stratum A:	31.51-124.80	72.72
Stratum B:	15.26-48.74	28.34
Stratum C:	12.31-19.17	14.93

6.2 Atterberg Limits

	Range(%)		Average(%)	
	LL	PL	LL	PL
Stratum A:	41.08-64.50	19.24-30.46	52.19	25.80

	Range(%)		Average(%)	
	LL	PL	LL	PL
Stratum B:	29.13-64.35	12.05-21.48	45.84	17.09
Stratum C:	-	N.P.	-	N.P.

6.3 Specific Gravity

	Range(%)	Average(%)
Stratum A:	2.51-2.60	2.56
Stratum B:	2.55-2.65	2.58
Stratum C:	2.57-2.63	2.60

6.4 Bulk Density

	Range(cu.m.)	Average(cu.m.)
Stratum A:	1.55-1.90	1.64
Stratum B:	1.71-2.18	2.03
Stratum C:	2.10-2.29	2.22

6.5 Strength Parameters

	Range	Average
Stratum A:		
N,blows/30 cm.	-	-
Vp,tsm.	1.220-1.830	1.500
Uc,ksc.	0.100-0.750	0.320
Stratum B:		
N,blows/30 cm.	14-52	35.4
Up,ksc.	1.250-4.500	3.290
Uc,ksc.	0.130-2.960	1.770
Stratum C:		
N,blows/30 cm.	45-121	84.1
*C,Tsm	0.00	0.00
*φ,Degree	33.8	33.8

*....Direct Shear Test

7. GROUND WATER OBSERVATION

Measurement obtained at twenty four hours after drilling ground water level to exist at the depth between 0.62 to 0.85 metres below existing ground surface around an area of boreholes. Significant fluctuations in the ground water table should be anticipated throughout the year depending upon the amount of precipitation, evaporation and surface ren off.

8. CONCLUSIONS

8.1 General Consideration

In establishing a suitable foundation system for the principal building proposed for the storage services, it is necessary to consider two espects of foundation design namely stability and settlement.

8.2 Allowable Pile Load Capacity

As shown above, the thick layer of soft silty clay of Stratum A is a high compressible layer. The foundation should not be spread footings or raft foundations because it would lead to excessive settlement so that the long pile foundations are recommended.

Study of the SPT-values measured in the borehole showns that at a depth of about 16.00 metres below existing ground surface, the SPT-values are in the range 33 to 52 blows/30 cm. which are high enough to support structures. The pile's tip should be founded at this depth. Exception was on Borehole 1, the pile's tip should be seated at least at the depth of about 19.00 metres below existing ground surface. However, the consolidation of underlying compressible soils, should be taken in to consideration for the foundations design.

8.2.1 Ultimate Pile Load Capacity

The ultimate load capacity of an individual precast concrete pile can be evaluated by the following equation:

$$Q_u = (C_a + K_s.q.\tan\phi)A_s + (C_p.N_c + q.N_q)A_p$$

where Q_u = ultimate load bearing of the pile

C_a = adhesive force (from Figure C.)

= 4.50 + 0.3 (C - 5.0) Tsm.

K_s = lateral pressure coefficient (from Table b)

tan ϕ = coeff. of friction of cohesionless soil (from Table c)

A_s = embedded area

q = effective overburden pressure (from Table a)

N_c & N_q = bearing capacity factors (from Figures A or B)

A_p = cross sectional area of the pile's tip

C = average shear strength

C_p = shear strength at pile tip

The ultimate load capacity of various size piles where as the pile's tops are at 1.00 metre below existing ground surface, are as follows:

I-Section Piles

			Ultimate Load/Pile(tons)			
			I.22	I.26	I.30	I.40
		Perimeter, cm	88	104	120	160
		Tip Area ,sq.cm	310	460	660	1240
	Pile tip elev.	Pile length m.				
BH_1	14.00	13.00	13.97	16.65	19.41	-
	15.00	14.00	15.28	18.20	21.19	-
	16.00	15.00	16.59	19.75	22.98	-
	17.00	16.00	22.30	26.82	31.59	-
	18.00	17.00	-	46.26	61.10	110.04
	19.00	18.00	-	65.70	91.07	176.66
	20.00	19.00	-	74.85	104.05	199.53
	21.00	20.00	-	80.82	111.85	212.97
BH_2	12.00	11.00	21.78	26.56	31.77	-
	13.00	12.00	28.75	35.04	41.90	-

I-Section Piles

	Pile tip elev.	Pile length m.	Ultimate Load/Pile(tons)			
			I.22	I.26	I.30	I.40
BH_2	(cont.)					
	14.00	13.00	36.10	43.94	52.45	-
	15.00	14.00	44.39	53.94	64.29	-
	16.00	15.00	-	65.40	77.91	110.69
	17.00	16.00	-	77.96	92.86	131.91
	18.00	17.00	-	88.88	105.46	148.71
	19.00	18.00	-	99.80	118.06	165.51
	20.00	19.00	-	136.50	176.12	278.07
	21.00	20.00	-	143.71	185.42	294.50
BH_3	12.00	11.00	24.58	29.97	35.86	-
	13.00	12.00	31.57	38.44	45.92	-
	14.00	13.00	38.78	47.12	56.14	-
	15.00	14.00	48.76	59.33	70.81	-
	16.00	15.00	-	71.21	84.89	120.77
	17.00	16.00	-	82.79	98.43	139.34
	18.00	17.00	-	93.71	111.03	156.14
	19.00	18.00	-	104.63	123.63	172.94
	20.00	19.00	-	134.65	171.53	284.32
	21.00	20.00	-	140.57	179.32	298.58

8.2.2 Negative Skin Friction

where a pile is driven through compressible soil layer to found in relatively firm strata, there is a tendency for the compressible soil around the pile tends to "hang up" on the pile, exerting a downward drag force called the negative skin friction. This force has been calculated according to the effective stress method, using the same basic formula as for positive side friction (ref.8) as follows:

$$Qn = B.Nv.Asn$$

where Qn = maximum negative skin friction

B = negative skin friction coefficient

= K.tan ϕ

= 0.30 for normally consolidated clay with high plasticity

Nv = average effective stress along the pile shaft

Asn = pile shaft area

Substituting these values, the negative skin friction can be determined as follows:

	Negative Skin Friction.Tons/Pile			
	I.22	I.26	I.30	I.40
BH_1 Qn =	12.94	15.29	17.64	23.52
BH_2 Qn =	11.05	13.06	15.07	20.10
BH_3 Qn =	9.21	10.88	12.56	16.74

8.2.3 Allowable Pile Load Capacity

Where the maximum value of the negative skin friction is taken into account, the safety coefficient on this force may be 1.1, while the safety coefficient on the pile working load varying between 2.0 and 2.5. In this report, we adopted 2.5 as a factor of safety. The allowable pile load capacity can be calculated as follows:

$$Qa = (Qu-1.1Qn)/2.5$$

where Qa = allowable pile load capacity,tons

Qn = maximum value of negative skin friction

The allowable load capacity of the precast concrete piles of various size and lengths where as the pile's tops are at 1.00 metre below existing ground surface, are as follows:

I-Section Piles

	Allowable Load/Pile(tons)			
	I.22	I.26	I.30	I.40

I-Section Piles

			Allowable Load/Pile(tons)			
	Pile tip elev.	Pile length m.	I.22	I.26	I.30	I.40
BH$_1$	14.00	13.00	1.71	2.07	2.47	-
	15.00	14.00	2.23	2.69	3.18	-
	16.00	15.00	2.76	3.31	3.90	-
	17.00	16.00	5.04	6.14	7.34	-
	18.00	17.00	-	11.78	16.68	33.67
	19.00	18.00	-	19.55	28.67	60.32
	20.00	19.00	-	23.21	33.86	69.46
	21.00	20.00	-	25.60	36.98	74.84
BH$_2$	12.00	11.00	5.40	6.70	8.19	-
	13.00	12.00	8.18	10.10	12.24	-
	14.00	13.00	11.12	13.65	16.46	-
	15.00	14.00	14.44	17.66	21.19	-
	16.00	15.00	-	22.24	26.64	38.25
	17.00	16.00	-	27.26	32.62	46.73
	18.00	17.00	-	31.63	37.66	53.45
	19.00	18.00	-	36.00	42.70	60.17
	20.00	19.00	-	50.68	65.93	105.20
	21.00	20.00	-	53.56	69.65	111.77
BH$_3$	12.00	11.00	7.07	8.73	10.58	-
	13.00	12.00	9.87	12.11	14.60	-
	14.00	13.00	12.75	15.58	18.69	-
	15.00	14.00	16.74	20.47	24.56	-
	16.00	15.00	-	25.22	30.19	43.29
	17.00	16.00	-	29.85	35.61	50.71
	18.00	17.00	-	34.22	40.65	57.43
	19.00	18.00	-	38.59	45.69	64.15
	20.00	19.00	-	50.59	64.84	108.70
	21.00	20.00	-	52.96	67.96	114.41

I-Section Piles

Allowable Load/Pile(tons)			
I.22	I.26	I.30	I.40
+20.92	+31.05	+44.55	+83.70
*13.95	+20.70	+29.70	+55.80

\+ Pile Bearing Capacity based on Concrete Strength (fc'=300 ksc.)

* Pile Bearing Capacity based on Concrete Strength (fc'=200 ksc.)

() Pile Bearing Capacity based on Underlying Soil Strength

8.3 Conlusion and Recommendations

8.3.1 The subsoils consist generally of silty clay and very fine to coarse sand. These soils are variable in colour and texture extending to a depth of 21.45 metres below existing ground surface.

8.3.2 These soils exist in very soft to medium consistency of silty clay, stiff to hard consistency of underlying silty clay and dense state to very dense state of very fine to coarse sand at the end of boreholes.

8.3.3 The recommended foundation system for the proposed building, consists of the pile foundation.

8.3.4 The allowable load capacity of piles are recommended in item 8.2.

8.3.5 The allowable pile load capacity is determined by the conventional equation with theoretical concern. If the higher pile load capacity is required, the pile load test must be performed and evaluated their possibility.

8.3.6 The hard driving may occur in sand sothat water jetting should be needed to avoid damage in reaching full desired embedment. Final sets should be obtained by driving without water jetting, when dynamic formulas might be used.

8.3.7 The safety factor 2.5 is adopted.

8.3.8 The working load of a desing pile must not be higher than the allowable load of a pile's structures.

8.3.9 The consolidation of underlying compressible soils should be taken into consideration for foundation design.

8.3.10 The concept of design and construction enclosed herewith must be taken into consideration.

8.4 Concept of Design and Construction

8.4.1 In evaluation of a foundation stability, the designer should take into consideration the following:

8.4.1.1 To take in to consideration, the effect of grouping on pile bearing capacity.

8.4.1.2 To determine a vertical stress in the soil surrounding and below the pile tips by using elastic theory. If the calculated stress exceeds the ultimate bearing capacity of the layer, the design should be revised by reducing the pile load capacity, by increasing the pile spacing or by extending the pile to another layer below.

8.4.1.3 To determine the differntial and total settlement of the foundation. If the ultimate differential and total settlement are found to be excessive, the design should be revised by trying the other type of foundations,by increasing the pile spacing, or by reducing the pile load capacity.

8.4.2 All piles should be driven to sufficient depth to carry the purposed load. Final penetration refusal criteria should be determined in the field, dependent upon the type of pile driving equipment used.

8.4.3 To obtain the required penetration, piles heavier than 3.5 tons should be driven by a hammer with a rated energy of at least 4.10 metre-tons. To obtain the required penetration and to prevent pile breakage, piles lighter than 3.50 tons should be driven by a hammer with a rated energy greater than 2.77 metre-tons, and less than 4.10 metre-tons.

8.4.4 All pile driving operations should be carefully inspected with complete and accurate records kept for each pile driven, such as final penetration,pile heavy and amount of downward movement on redriving tip and top elevation, length of pile and amount of cut off.

8.4.5 To minimize ground quake to the adjacent structure, all pile within 12.00 metres of the existing structures should be set in holes prebored to a depth of 6.00 to 10.00 metres below the existing ground level of other method which the engineer approved to be effective.

8.4.6 The design of ground floor structures must take the problem of land settlement due to the load of fill, structures and other into consideration.

9. General

The analysis and recommendation submitted in this report are based upon available information. Since it is possible for variation in soil conditions to exist between boring locations, it is recommended that the soil excavation and the pile driving should be inspected by an experienced soil engineer to assure that the construction is operated conformance with recommendation and foundation are seated upon suiiable materials.

This report has been prepared in order to aid in the evaluation of this site and to assist the engineers in the design of the project, based on our understanding of design details, criteria and utilization of the project as outlined herein. Also, if our understanding of the design and utilization is not correct, we should be promptly informed of the correct data sothat we may revise our recommendations as appropriate.

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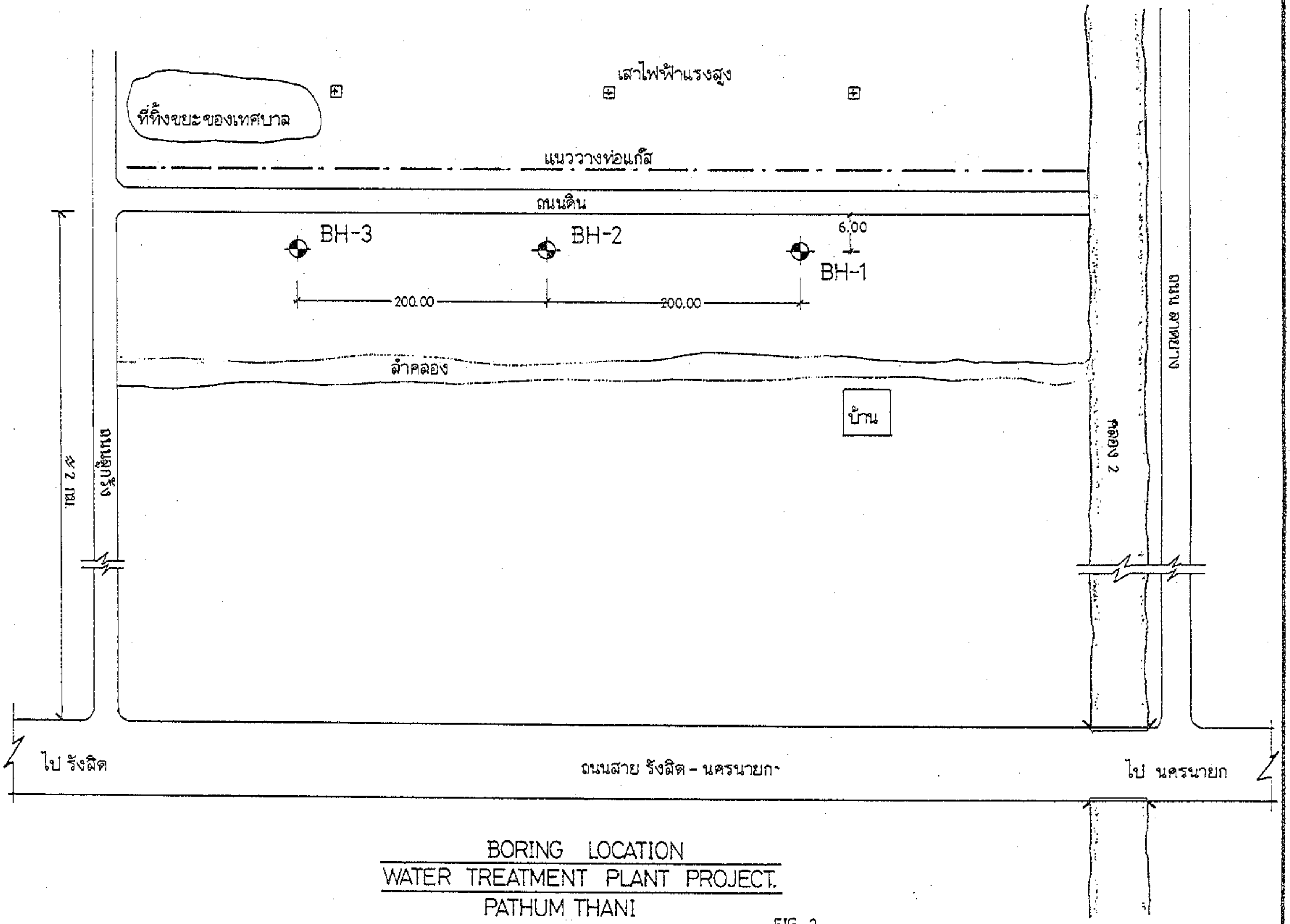


BORING LOCATION
WATER TREATMENT PLANT PROJECT.
PATHUM THANI

FIG. 2

PROJECT.

WATER TREATMENT PLANT

FIG. 3

W.A.C. BORING LOG

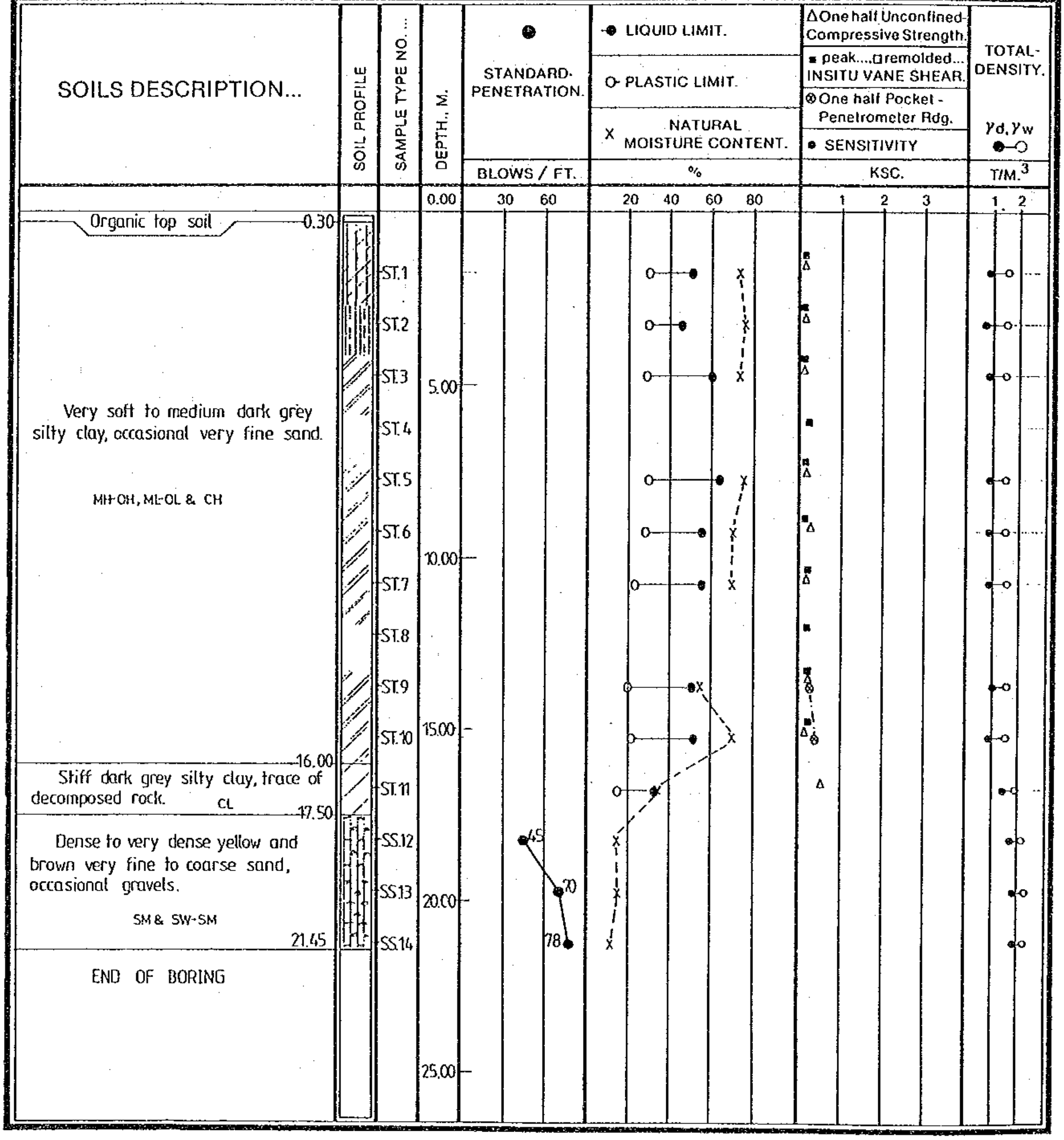
GROUND WATER OBSERVATION.

DATE	TIME	EL. of HOLE	EL.of WATER

24 HR.AFTER BORING. 0.85 M.

LOCATION. AMPHOE KLONG LUANG
PATHUM THANI

BORING NO. 1
SURFACE ELV.
DATE START 24 / 7 / 36
DATE FINISH 25 / 7 / 36



Organic top soil 0.30

Very soft to medium dark grey silty clay, occasional very fine sand.

MH-OH, ML-OL & CH

16.00

Stiff dark grey silty clay, trace of decomposed rock. CL

17.50

Dense to very dense yellow and brown very fine to coarse sand, occasional gravels.

SM & SW-SM

21.45

END OF BORING

PROJECT.

WATER TREATMENT PLANT

FIG. 4

W.A.C. BORING LOG

GROUND WATER OBSERVATION.			
DATE	TIME	EL. of HOLE	EL.of WATER
24 HR.AFTER BORING.		0.67 M.	

LOCATION: AMPHOE KLONG LUANG
PATHUM THANI

BORING NO. 2
SURFACE ELV.
DATE START 25/ 7/ 36
DATE FINISH 26/ 7/ 36

SOILS DESCRIPTION...

Organic top soil. 0.20

Soft grey and brown silty clay, trace of organic matters. CL 2.10

Very soft to soft dark grey silty clay, occasional very fine sand.

CH, ML-OL & CL 10.00

Stiff dark grey silty clay.

CL 12.00

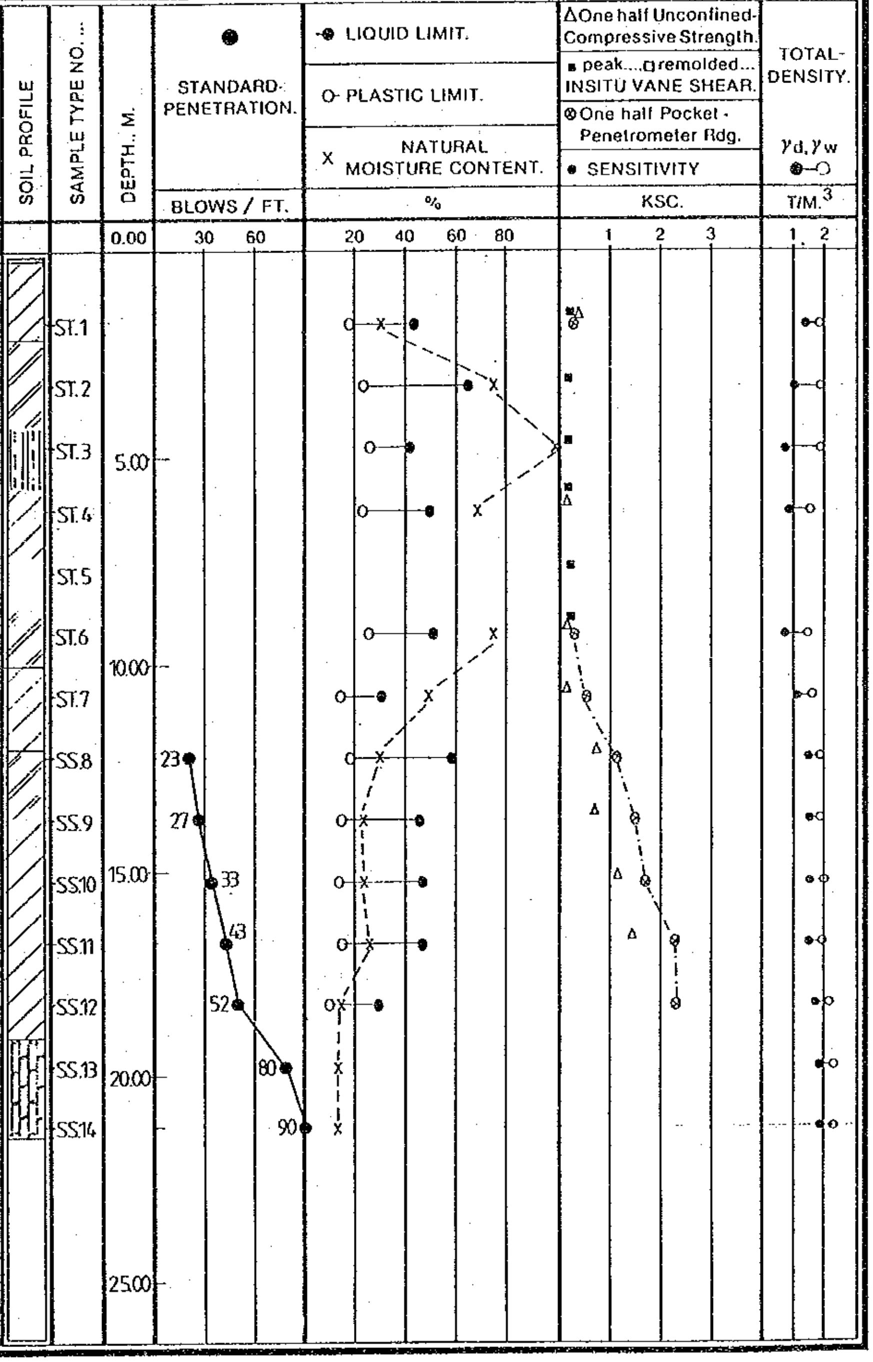
Very stiff to hard light grey, yellowish brown and brown silty clay, occasional very fine sand and decompose rock.

CH & CL 19.00

Very dense grey and yellow very fine to coarse sand, trace of gravels.

SM 21.45

END OF BORING



PROJECT.

WATER TREATMENT PLANT

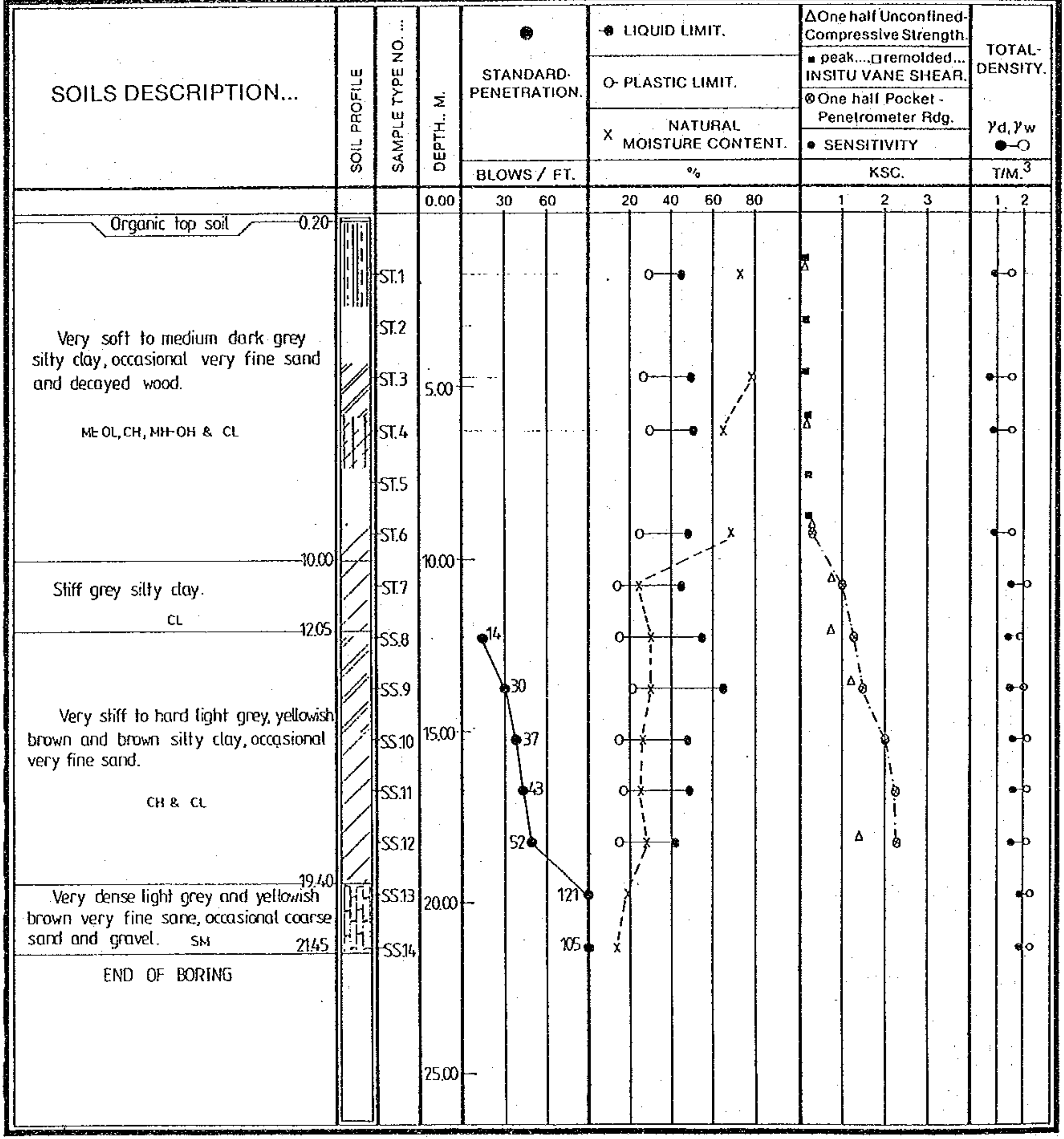
FIG. 5

W.A.C. BORING LOG

GROUND WATER OBSERVATION.			
DATE	TIME	EL. of HOLE	EL.of WATER
24 HR.AFTER BORING.		0.62 M.	

LOCATION. AMPHOE KLONG LUANG
PATHUM THANI

BORING NO. 3
SURFACE ELV.
DATE START 27/7/36
DATE FINISH 28/7/36



SOILS DESCRIPTION...

Organic top soil 0.20

Very soft to medium dark grey silty clay, occasional very fine sand and decayed wood.

ML OL, CH, MH-OH & CL

10.00

Stiff grey silty clay.

CL

12.05

Very stiff to hard light grey, yellowish brown and brown silty clay, occasional very fine sand.

CH & CL

19.40

Very dense light grey and yellowish brown very fine sane, occasional coarse sand and gravel. SM 21.45

END OF BORING

GRADING CURVES

TEST BY.

DATE.

SAMPLE BY.

DATE.

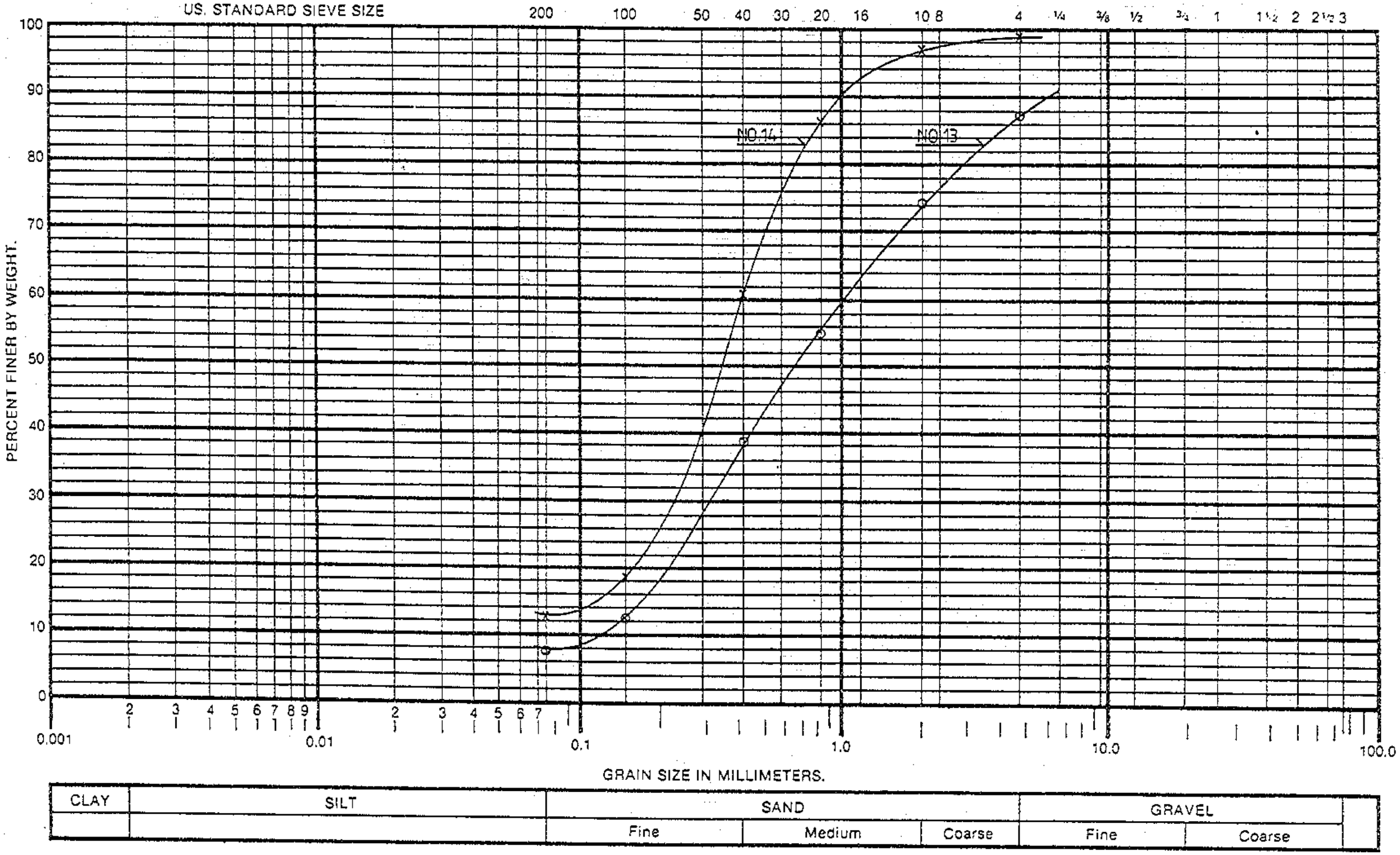
SAMPLE NO. 13 , 14

DEPTH. 19.50-19.95 , 21.00-21.45 m.

BORING NO. 1

STATION. A. Klong luang , PATHUM THANI

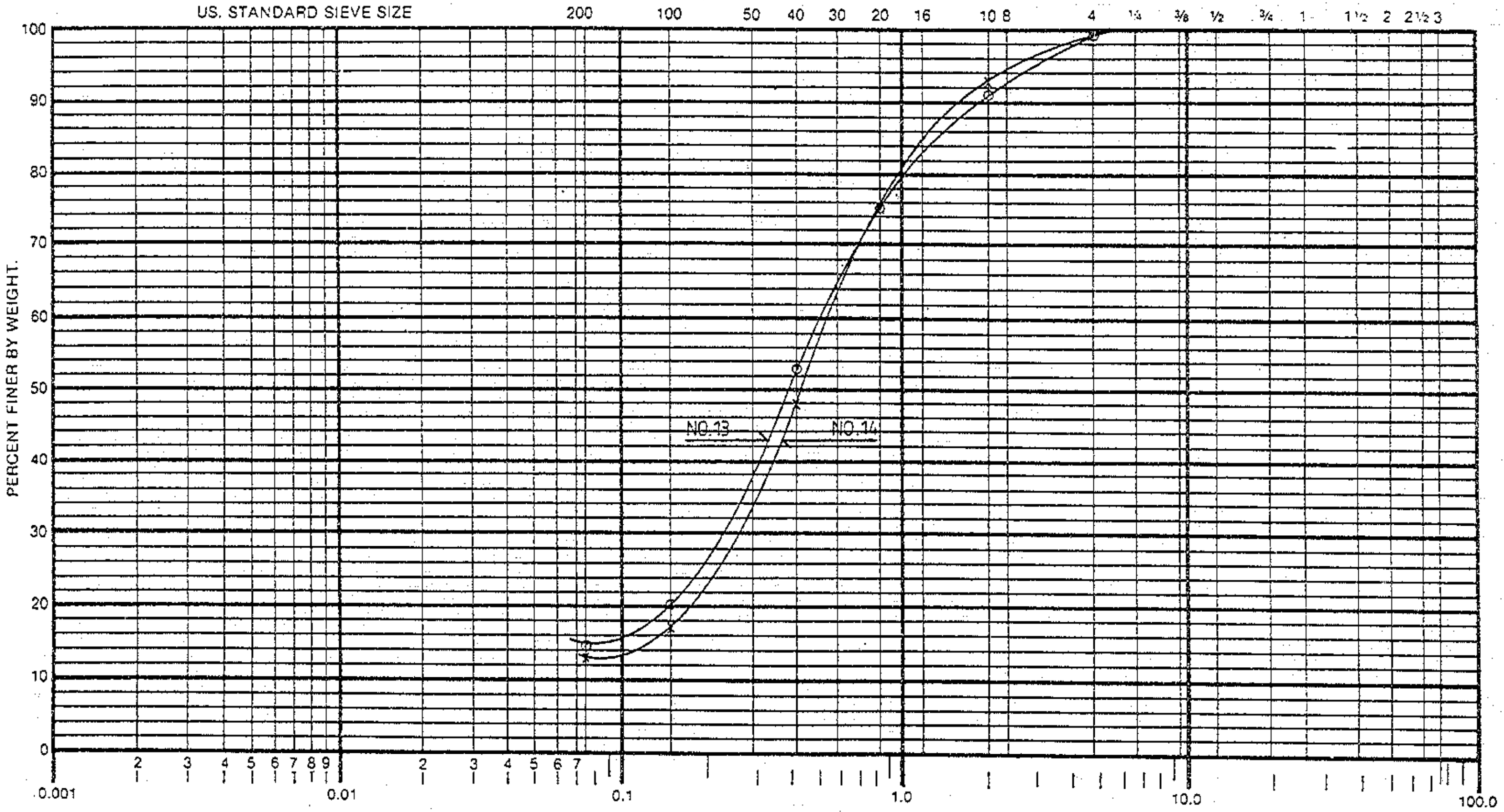
PROJECT. WATER TREATMENT PLANT.



GRADING CURVES

TEST BY.		DATE.	
SAMPLE BY.		DATE.	
SAMPLE NO.	13 , 14	DEPTH.	19.50-19.95 , 21.00-21.45 m.
BORING NO.	2	STATION.	A. Klong luang , PATHUM THANI

PROJECT. WATER TREATMENT PLANT.



CLAY	SILT	SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

GRADING CURVES

TEST BY.

DATE.

SAMPLE BY.

DATE.

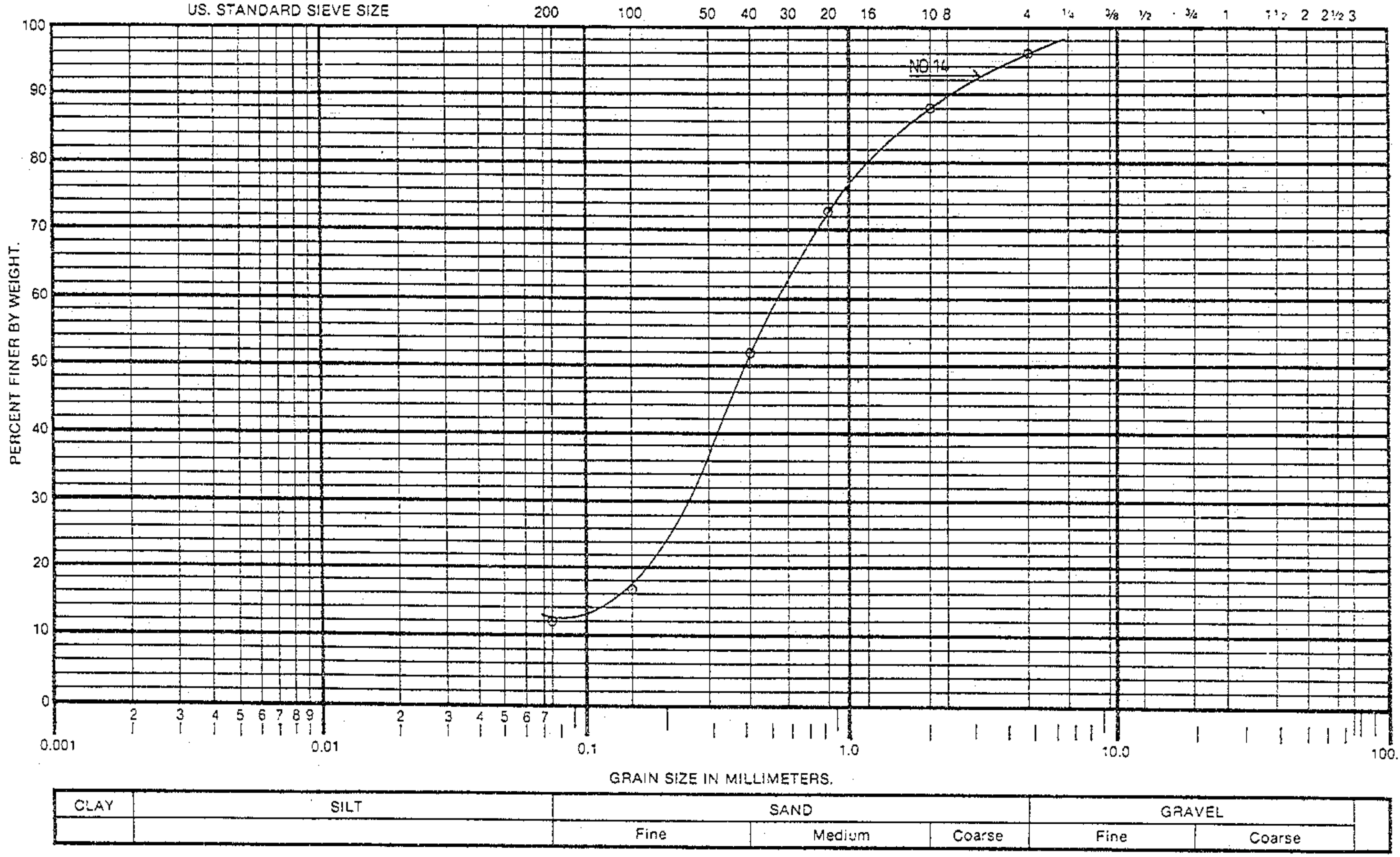
SAMPLE NO. 14

DEPTH. 21.00-21.45 m.

BORING NO. 3

STATION. A. Klong luang , PATHUM THANI

PROJECT. WATER TREATMENT PLANT.

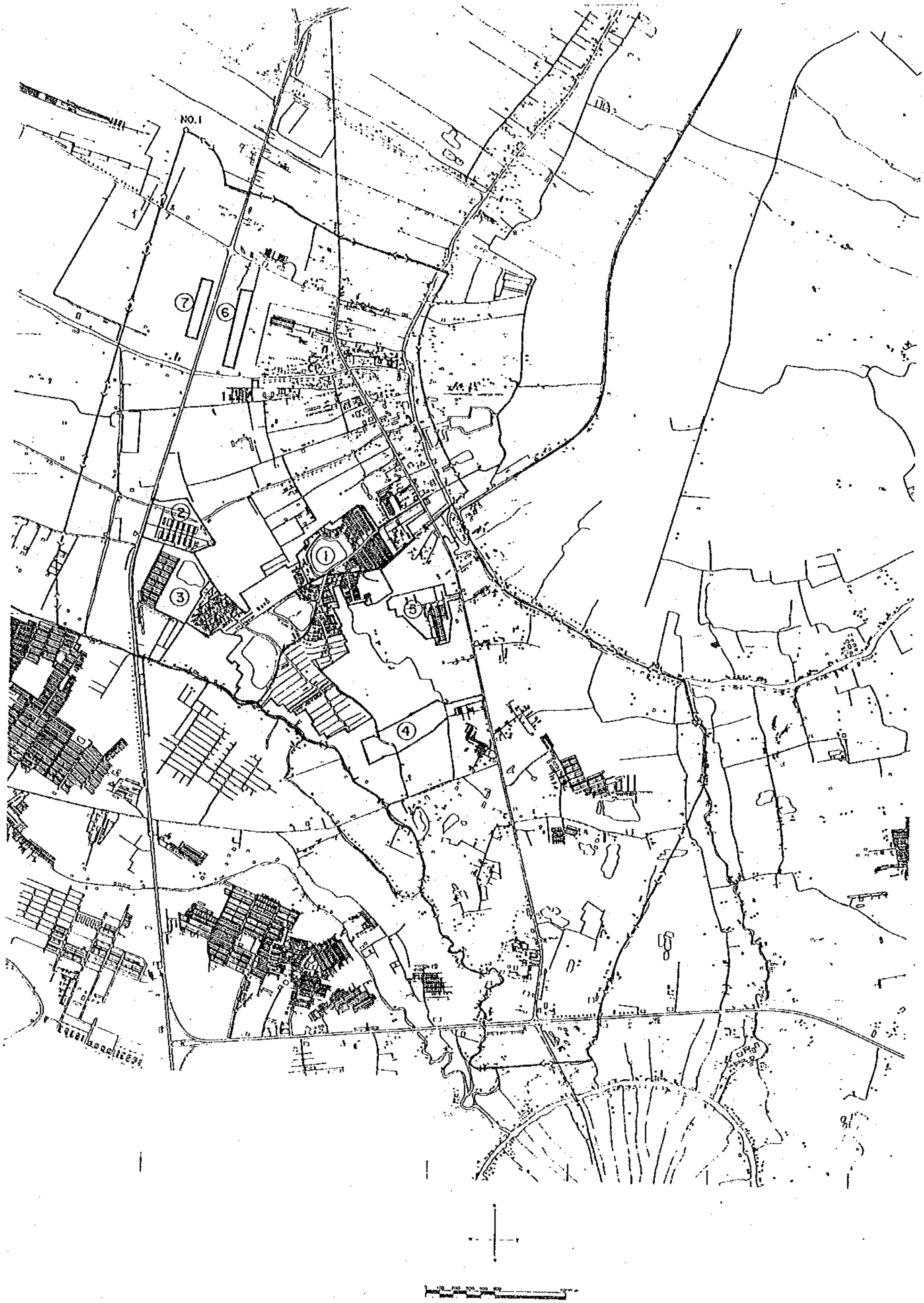


3.1.1 Housing Estates in Bang Bua Thong

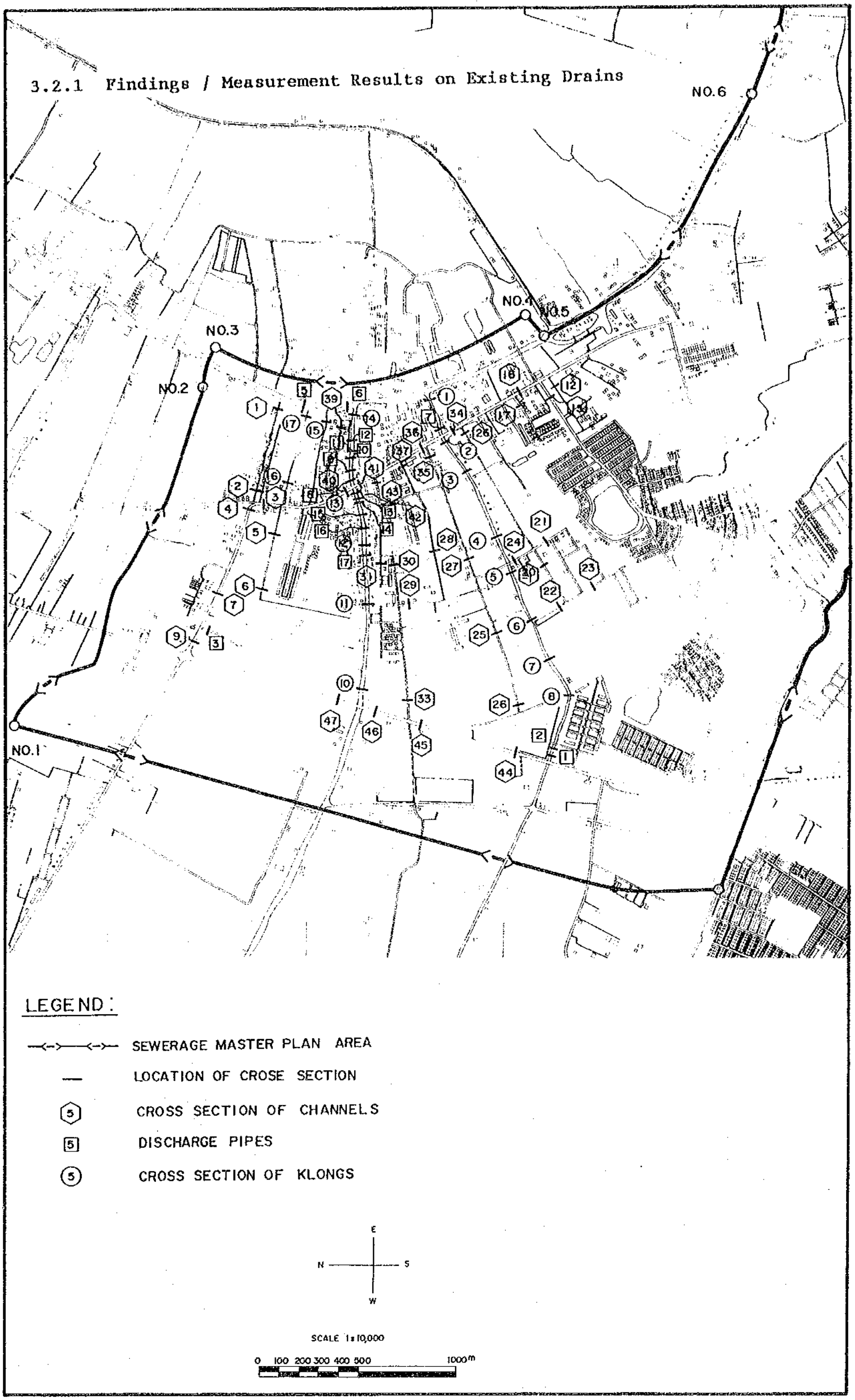
Table 3.1.1.1 Housing Complexs

No.	Name of H.C	No. of H.H.	Population	Pop. Density (per./ha)	Area Coverage (ha)	No. of S.T.P	Remarks
1.	Chollada	2,012	8,048	73	109.92	3 (1–Under Construction 2–Planned)	Future estimation 2,112
2.	P. Phasuk	700	2,800	292	9.60	1	
3.	Chantima	1,000	4,000	208	19.20	1	
4.	Busarin (Under–Construction)	1,251	5,004	255	17.76	3	
5.	C. Rungraung	500	2,000	150	13.28	1	
6.	Busakorn (Under–Construction)	336	1,344	280	4.80	None (Planned)	
7.	Pichada (Planned)	585	2,340	357	6.56	None	

Figure 3.1.1.1 LOCATION OF HOUSING ESTATES IN BANG BUA THONG

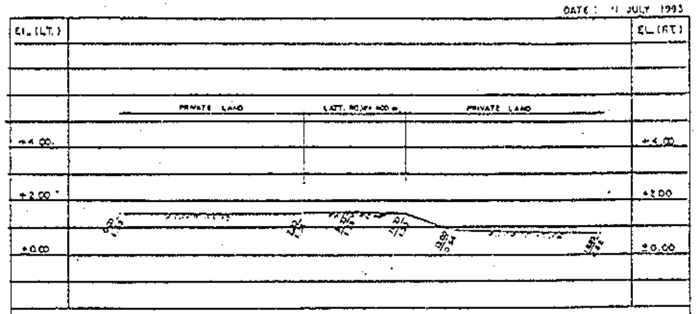


3.2.1 Findings / Measurement Results on Existing Drains



(1) Cross Section of Channels

DATE : 11 JULY 1993



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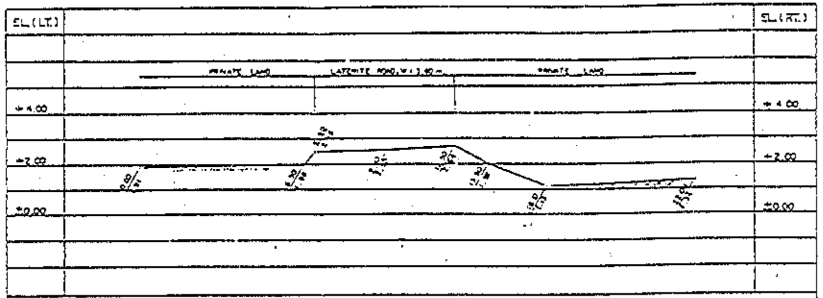
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1993



SECTION NO. ②
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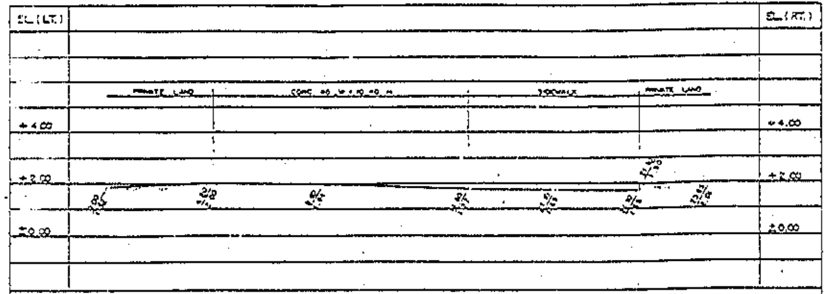
1 : 200 VERTICAL

DATE : 11 JULY 1993



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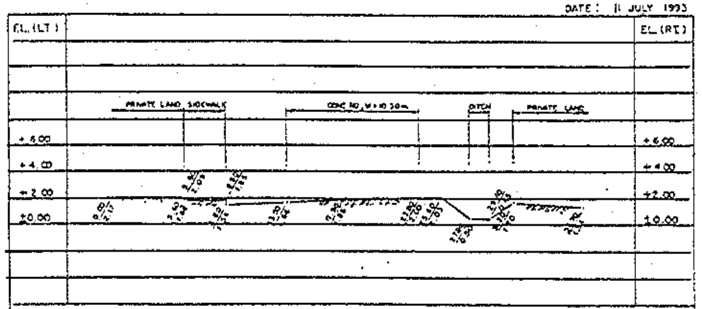
1 : 200 VERTICAL

DATE : 11 JULY 1993



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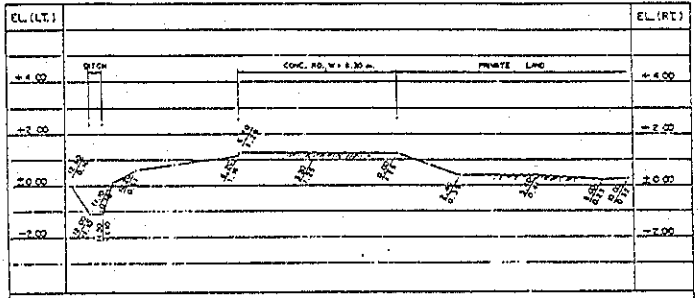

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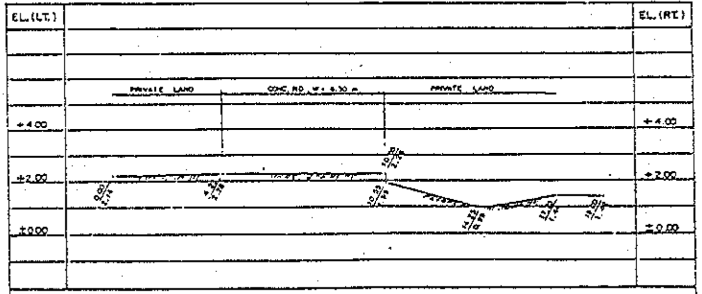
1 : 200 VERTICAL

DATE : 1 JULY 1993



SECTION NO. ⑥
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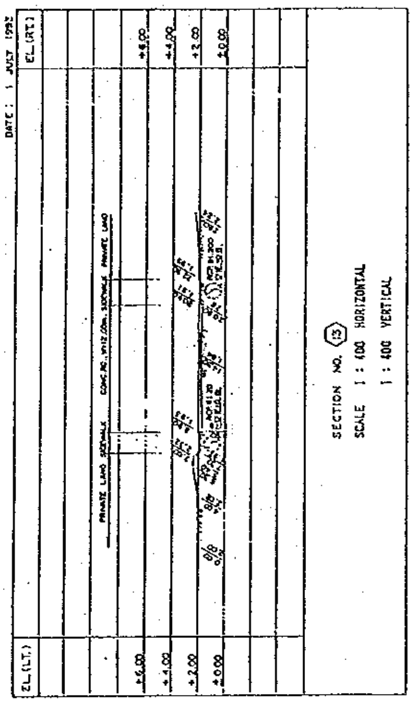
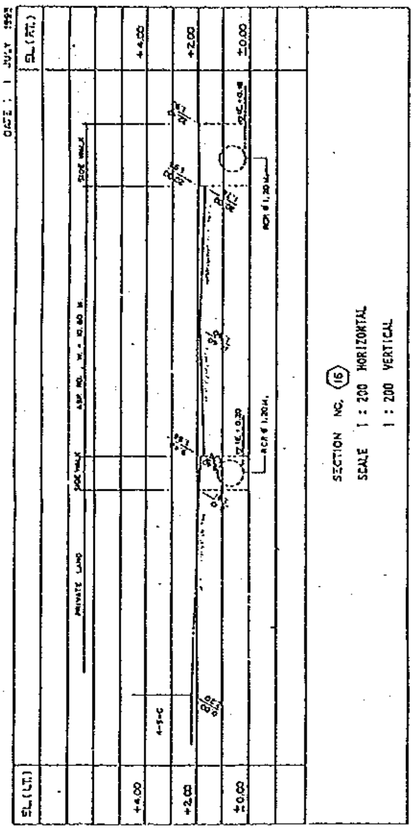
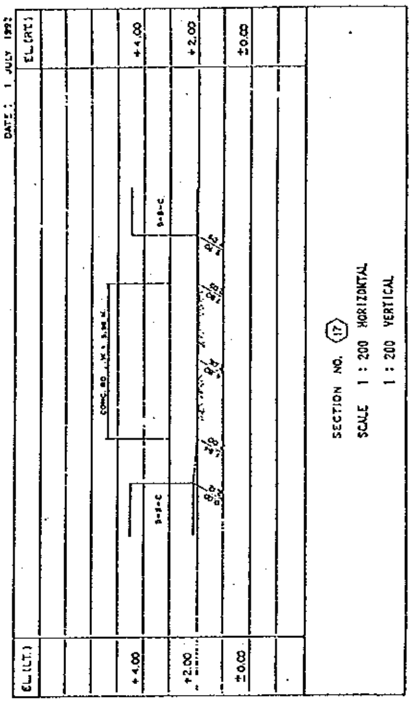
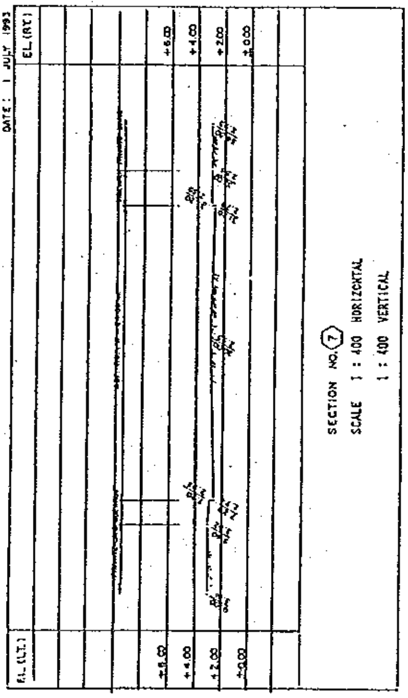
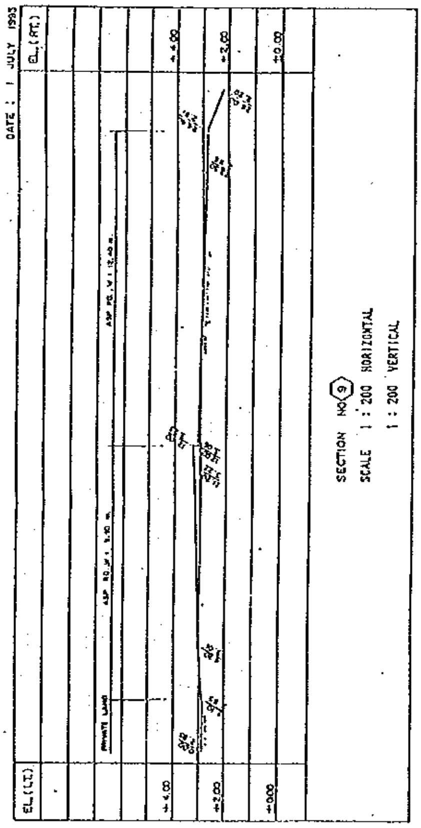
1 : 200 VERTICAL

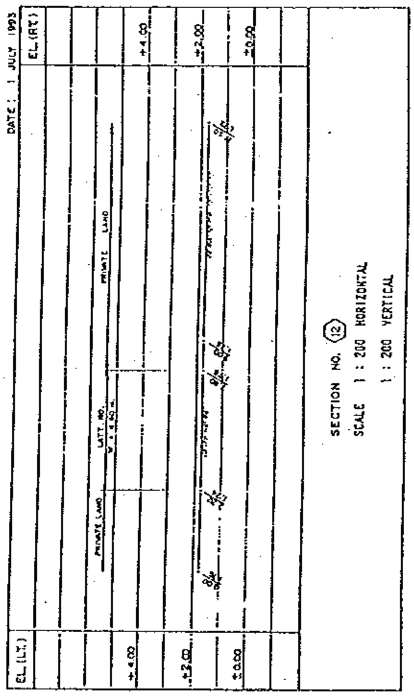

DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+6.00
+4.00
+2.00
±0.00
PRIVATE LAND
SIDEWALK
CONC. RD.
SIDEWALK
PRIVATE LAND
SECTION NO. ⑬
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL


DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
PRIVATE LAND
SIDEWALK
ASP. RD.
SIDEWALK
SECTION NO. ⑮
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

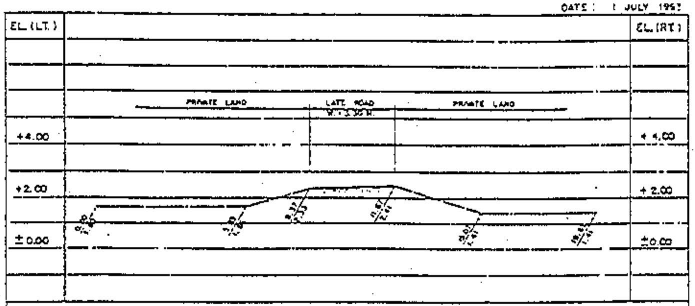

DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
CONC. RD.
SECTION NO. ⑰
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+6.00
+4.00
+2.00
±0.00
SECTION NO. ⑦
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL


DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
PRIVATE LAND
ASP. RD.
ASP. RD.
SECTION NO. ⑨
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 1 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
PRIVATE LAND
LATT. RD.
PRIVATE LAND
SECTION NO. ⑫
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1992

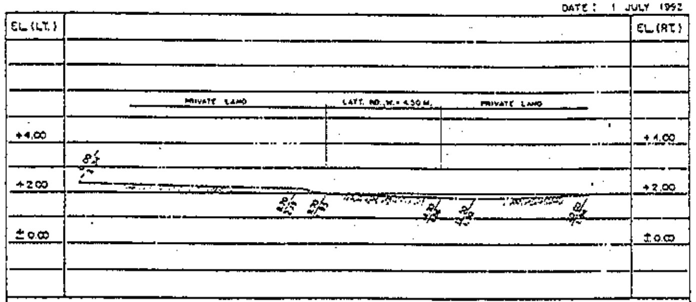


SECTION NO. 20

SCALE 1 : 200 HORIZONTAL

1 : 200 VERTICAL

DATE : 1 JULY 1992

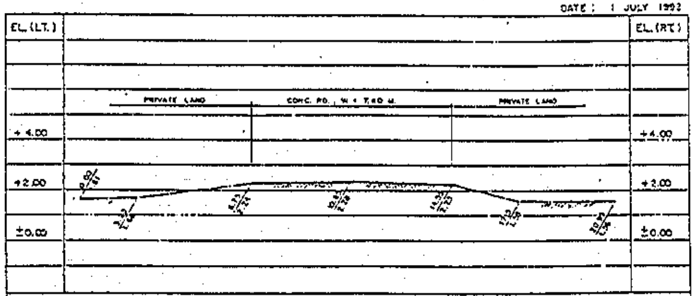


SECTION NO. 20'

SCALE 1 : 200 HORIZONTAL

1 : 200 VERTICAL

DATE : 1 JULY 1992

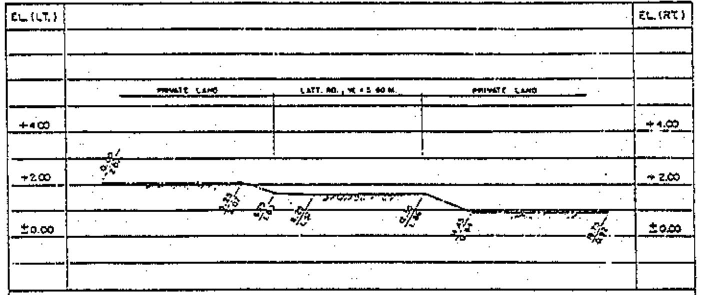


SECTION NO. 21

SCALE. 1 : 200 HORIZONTAL

1 : 200 VERTICAL

DATE : 1 JULY 1992

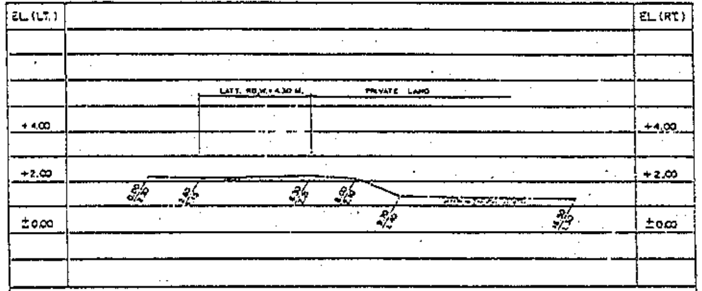


SECTION NO. 22

SCALE 1 : 200 HORIZONTAL

1 : 200 VERTICAL

DATE : 1 JULY 1992

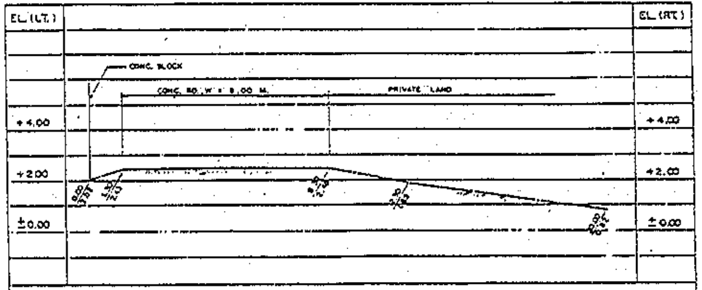


SECTION NO. 23

SCALE 1 : 200 HORIZONTAL

1 : 200 VERTICAL

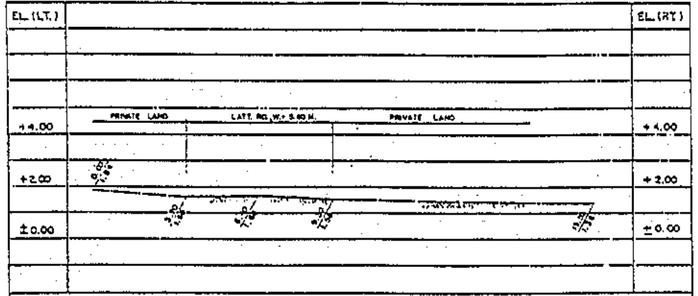
DATE : 1 JULY 1992



SECTION NO. 24

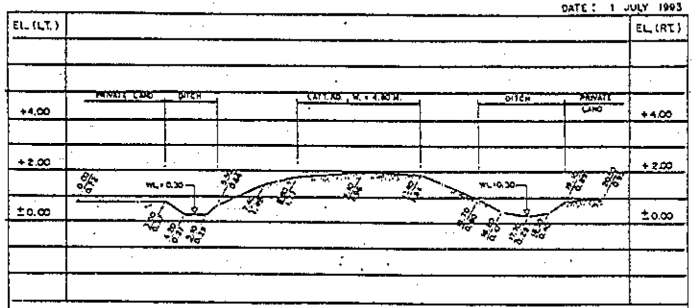
SCALE 1 : 200 HORIZONTAL

1 : 200 VERTICAL



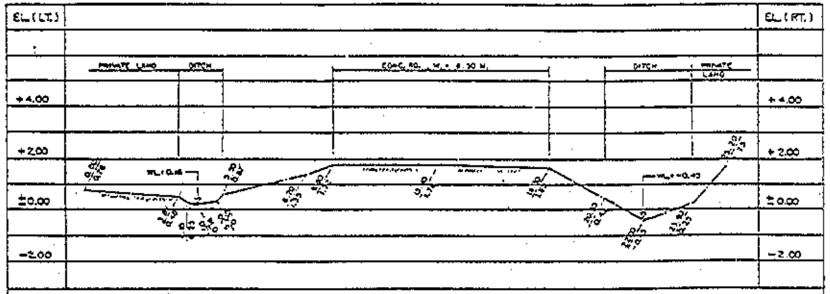
DATE : 1 JULY 1993

SECTION NO. ㉕
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



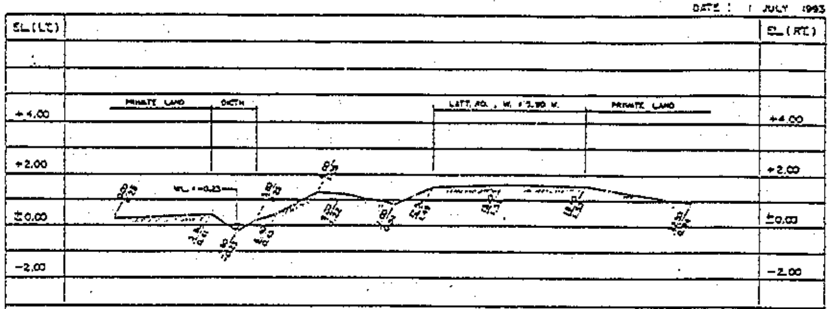
DATE : 1 JULY 1993

SECTION NO. ㉖
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



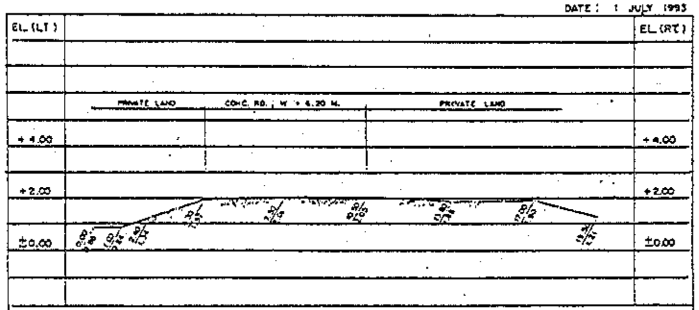
DATE : 1 JULY 1993

SECTION NO. ㉗
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



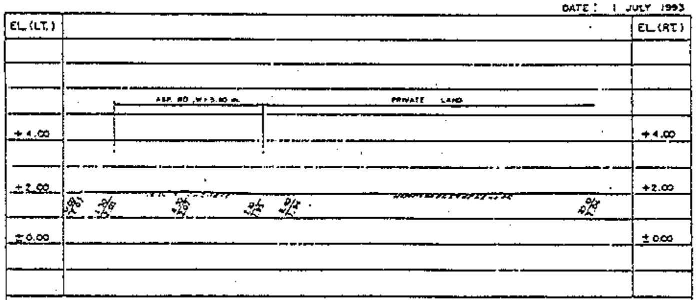
DATE : 1 JULY 1993

SECTION NO. ㉘
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



DATE : 1 JULY 1993

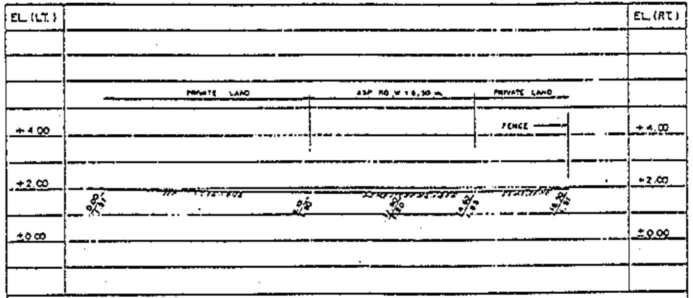
SECTION NO. ㉙
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



DATE : 1 JULY 1993

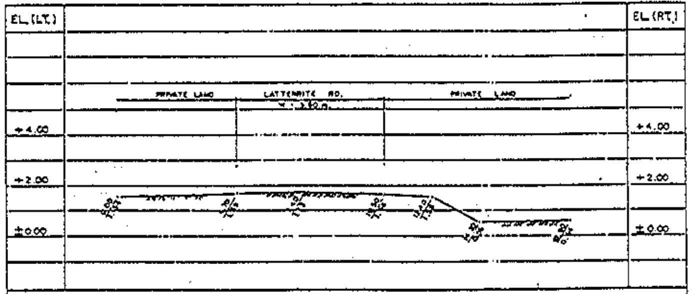
SECTION NO. ㉚
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1993



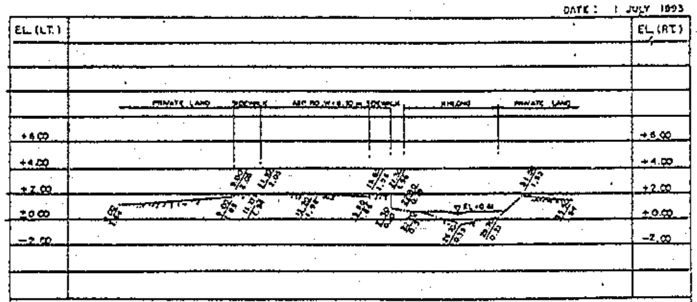
SECTION NO. ㉛
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1993



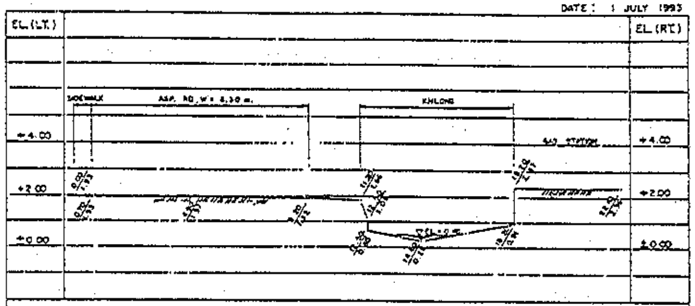
SECTION NO. ㉝
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1993



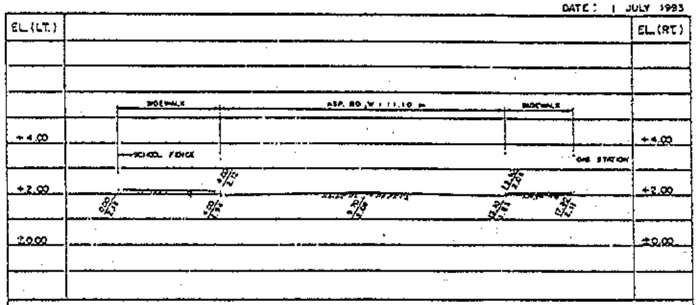
SECTION NO. ㉞
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL

DATE : 1 JULY 1993



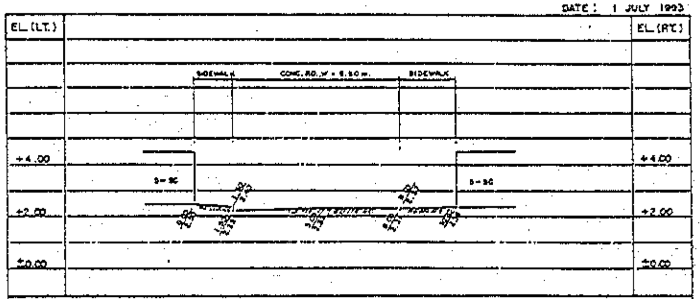
SECTION NO. ㉟
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

DATE : 1 JULY 1993

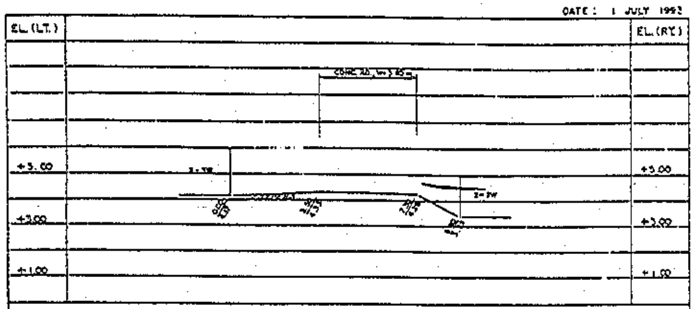


SECTION NO. ㊱
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

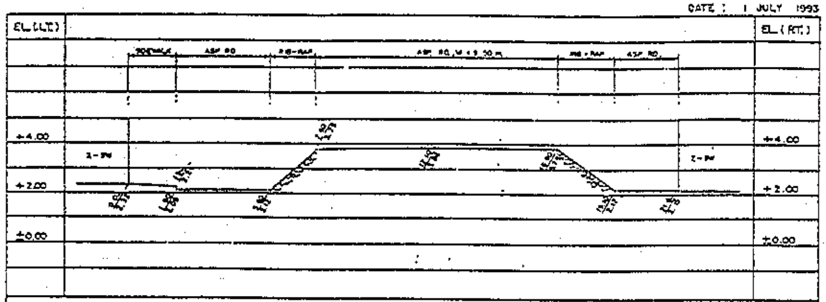
DATE : 1 JULY 1993



SECTION NO. ㊲
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



SECTION NO. ㊴
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



SECTION NO. ㊵
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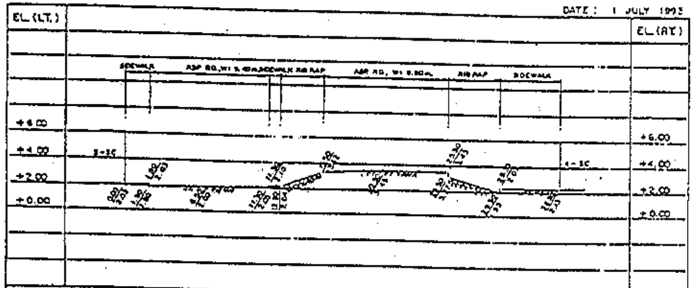
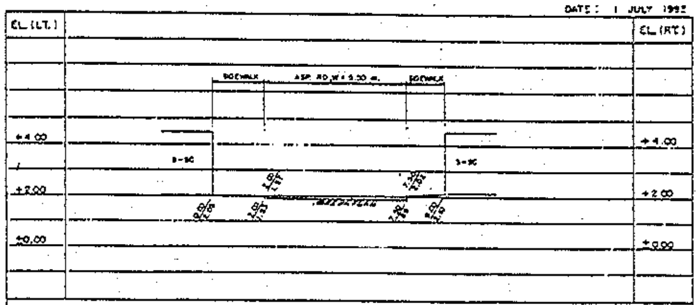
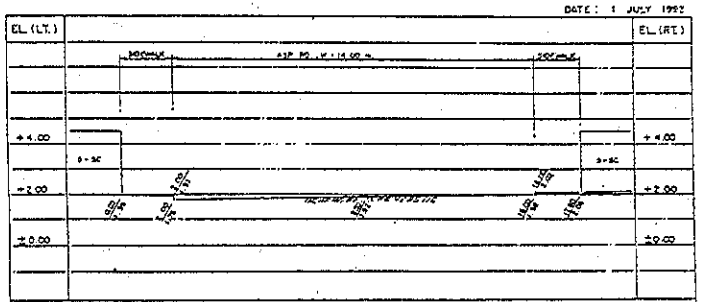
1 : 200 VERTICAL



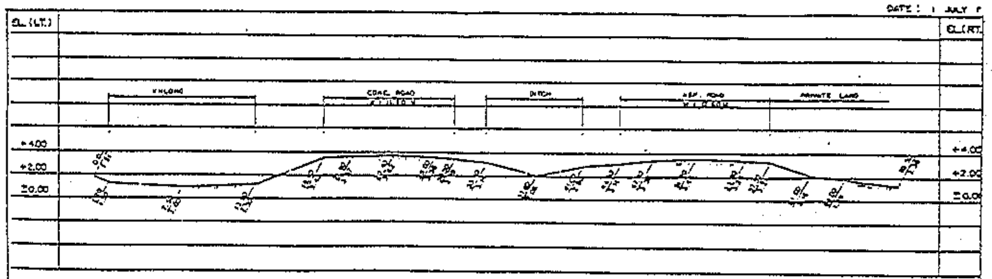
SECTION NO. ㊶
SCALE 1 : 400 · HORIZONTAL
1 : 400 VERTICAL



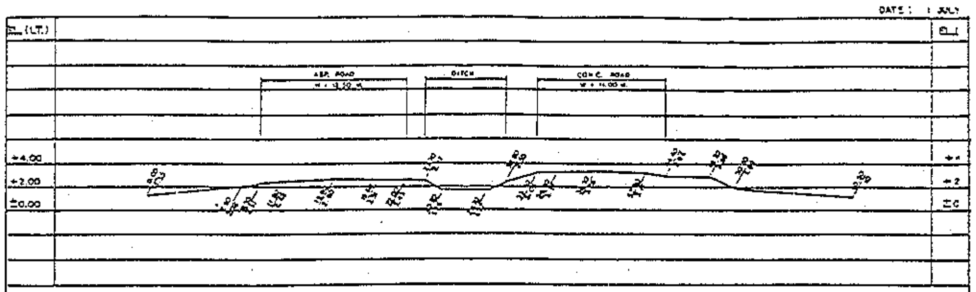
SECTION NO. ㊷
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



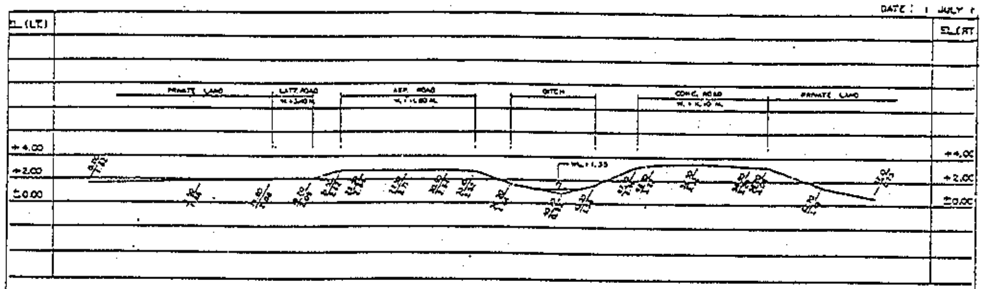
SECTION NO. ㊸
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



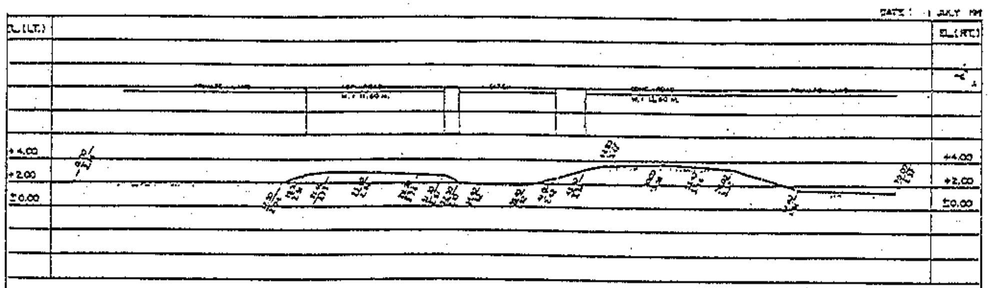
SECTION NO. ㊹
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL



SECTION NO. ㊺
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL

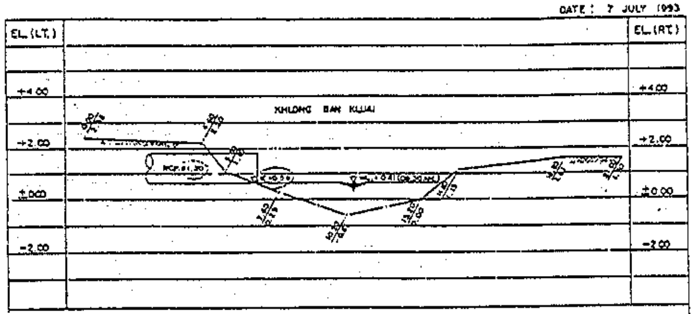


SECTION NO. ㊻
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL

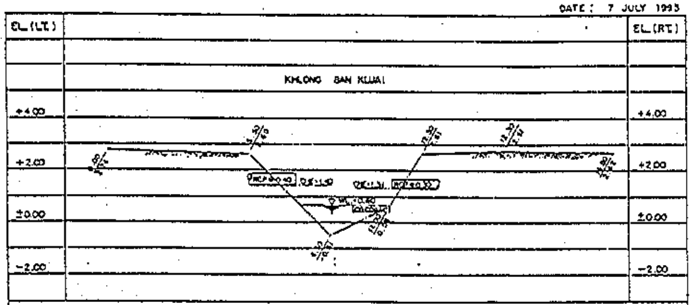


SECTION NO. ㊼
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL

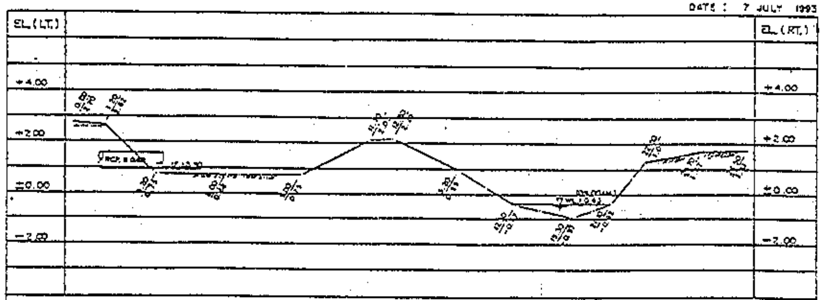
(2) Discharge Pipes



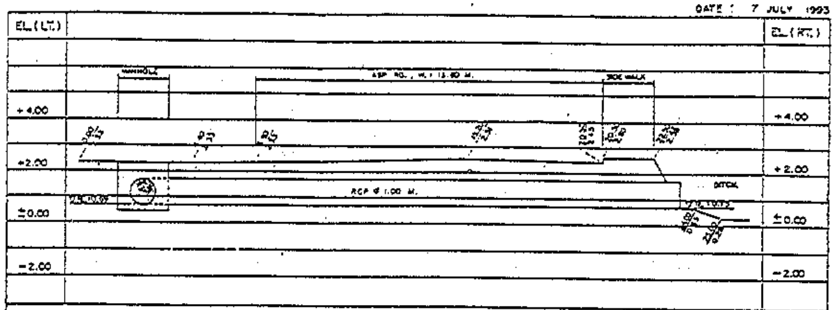
SECTION NO. 1
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



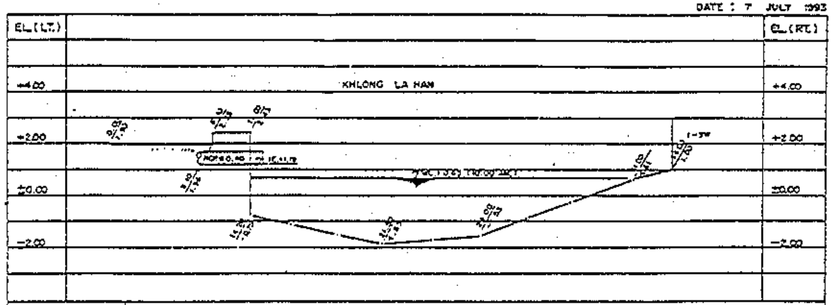
SECTION NO. 2
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



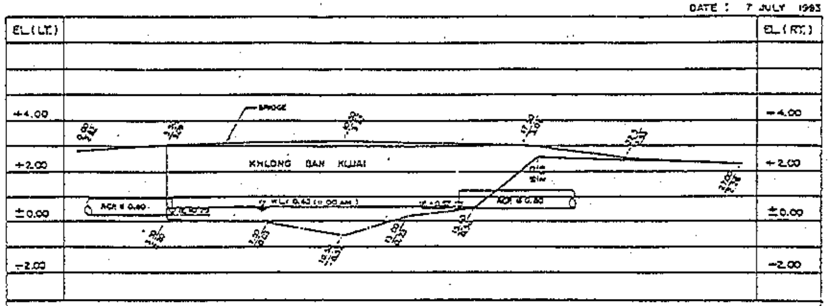
SECTION NO. 3
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



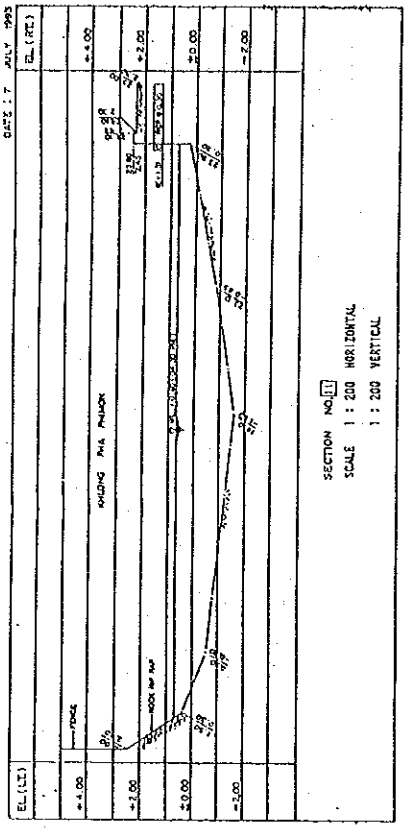
SECTION NO. 5
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

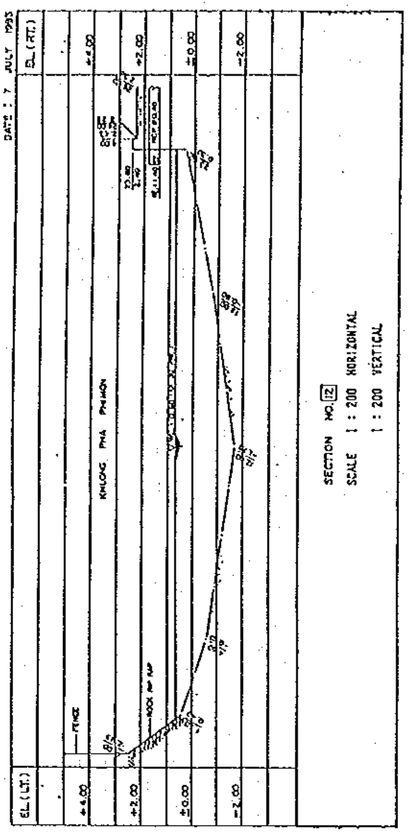


SECTION NO. 6
SCALE 1 : 400 HORIZONTAL
1 : 200 VERTICAL



SECTION NO. 7
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 7 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
-2.00
KHLONG PHA PHIMON
FENCE
ROCK RIP RAP
SECTION NO. 11
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 7 JULY 1993
EL.(LT.)
EL.(RT.)
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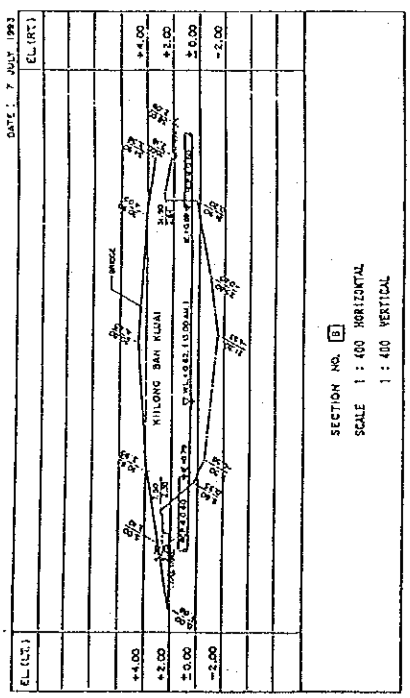
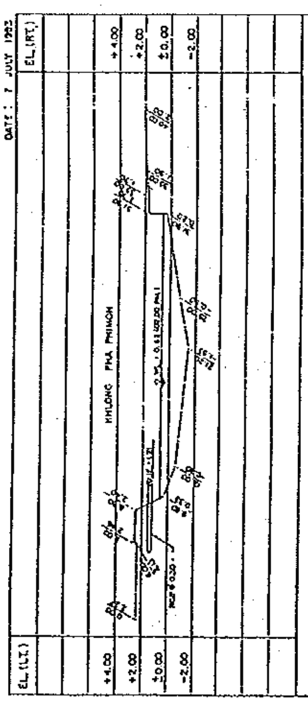
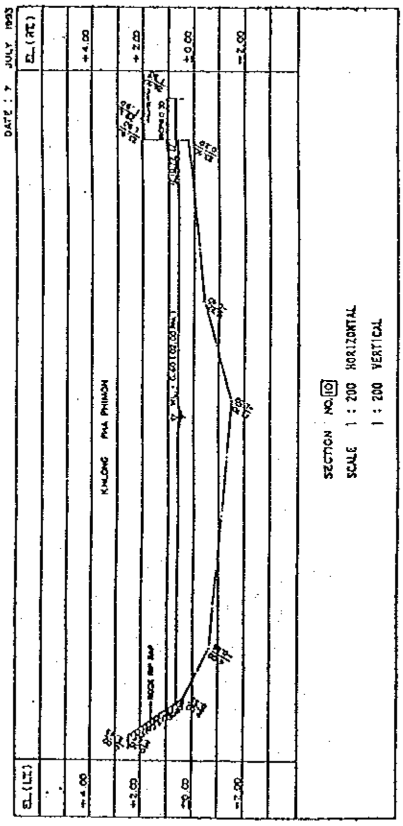
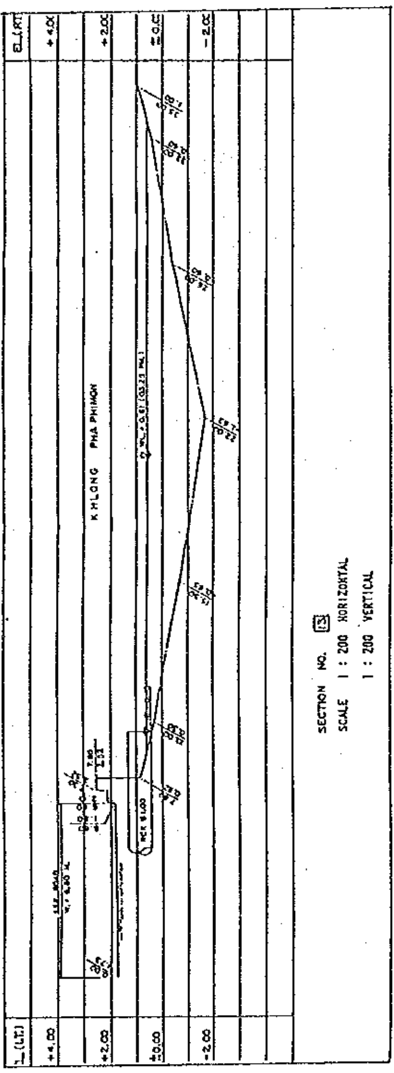
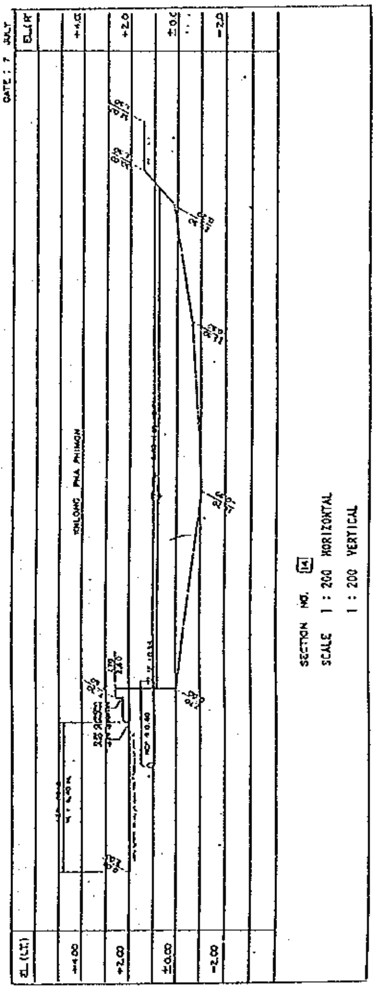
+2.00
±0.00
-2.00
KHLONG PHA PHIMON
FENCE
ROCK RIP RAP
SECTION NO. 12
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

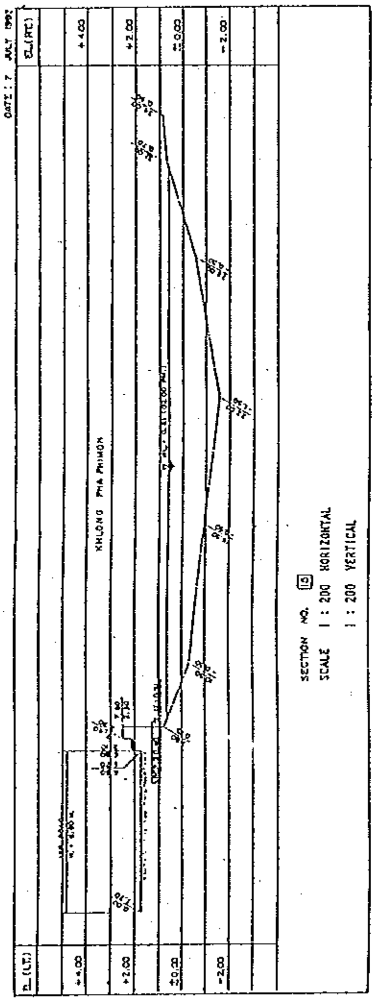

DATE : 7 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
-2.00
KHLONG BAN KLUAI
BRIDGE
SECTION NO. 8
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL

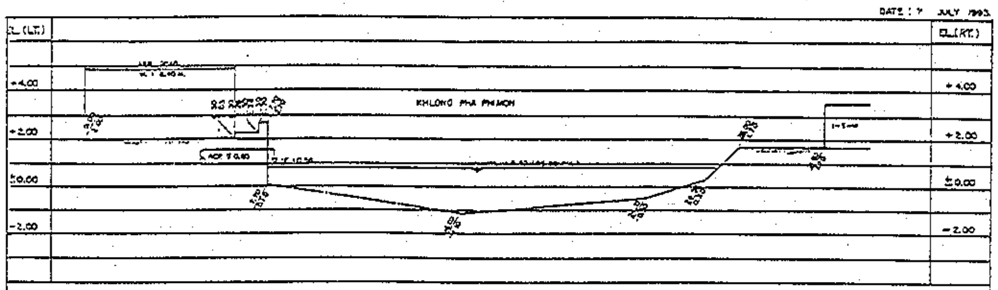

DATE : 7 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
-2.00
KHLONG PHA PHIMON
SECTION NO. 9
SCALE 1 : 400 HORIZONTAL
1 : 400 VERTICAL


DATE : 7 JULY 1993
EL.(LT.)
EL.(RT.)
+4.00
+2.00
±0.00
-2.00
KHLONG PHA PHIMON
ROCK RIP RAP
SECTION NO. 10
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

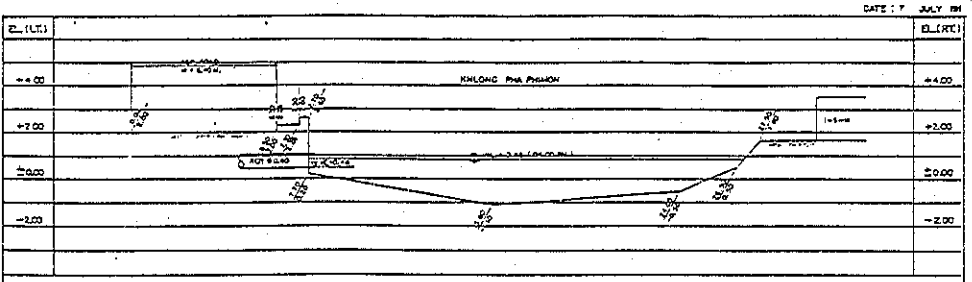

DATE : 7 JULY
EL.(RT)
+4.00
+2.00
±0.00
-2.00
EL.(LT)
KHLONG PHA PHIMON
SECTION NO. 13
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 7 JULY
EL.(RT)
+4.00
+2.0
±0.00
-2.0
EL.(LT)
KHLONG PHA PHIMON
SECTION NO. 14
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL


DATE : 7 JULY 1992
EL.(RT)
+4.00
+2.00
±0.00
-2.00
EL.(LT)
KHLONG PHA PHIMON
SECTION NO. 15
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

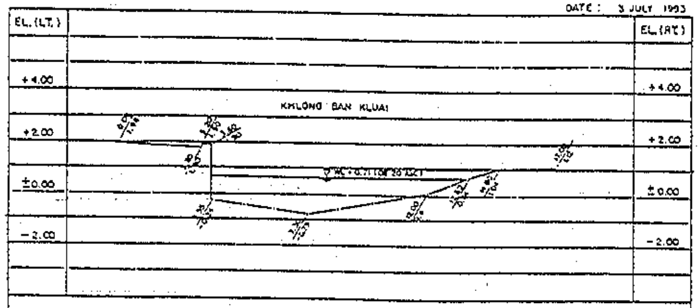


SECTION NO. 16
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

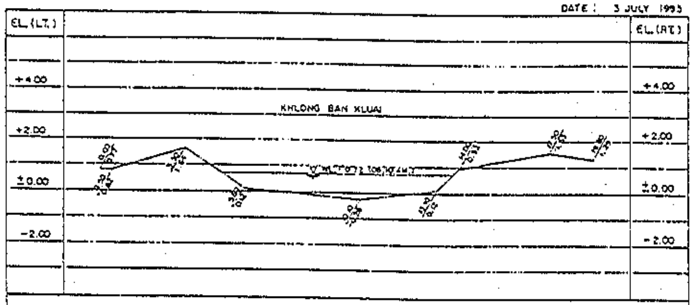


SECTION NO. 17
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL

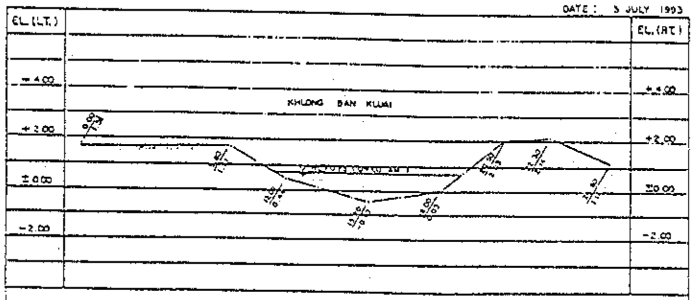
(3) Cross section of Klongs



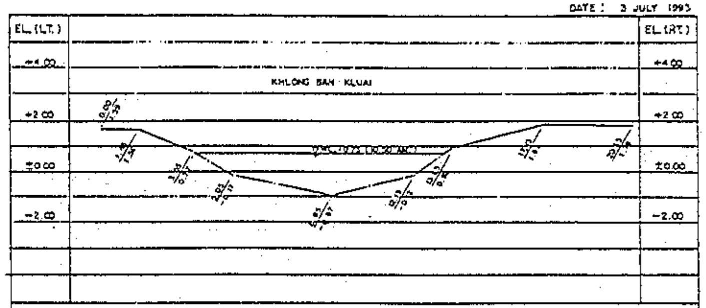
SECTION NO.①
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



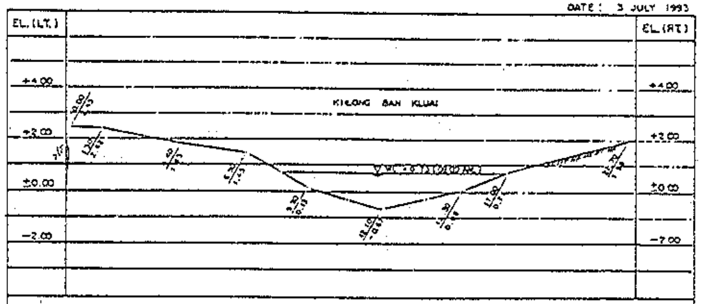
SECTION NO.②
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



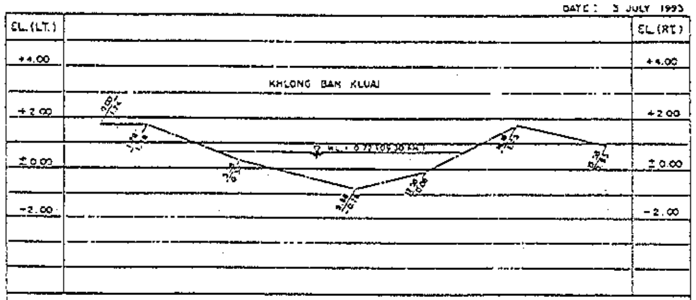
SECTION NO.③
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



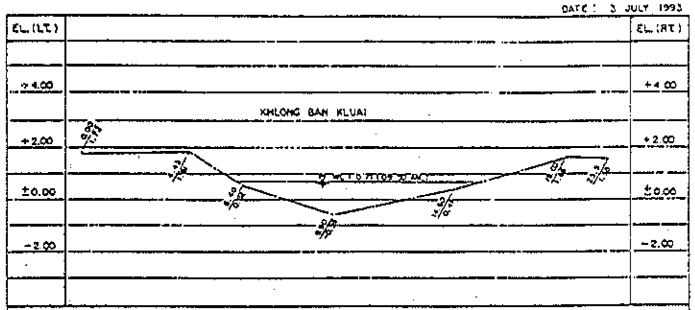
SECTION NO.④
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



SECTION NO.⑤
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



SECTION NO.⑥
SCALE 1 : 200 HORIZONTAL
1 : 200 VERTICAL



SECTION NO.⑦
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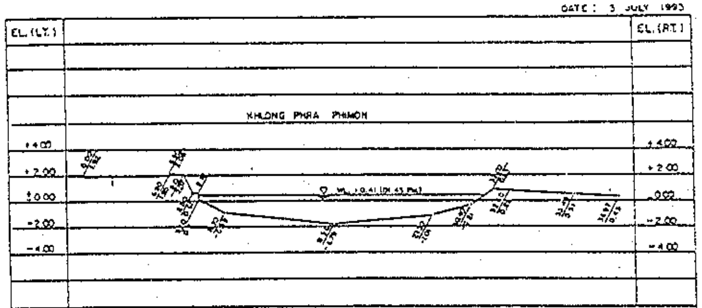

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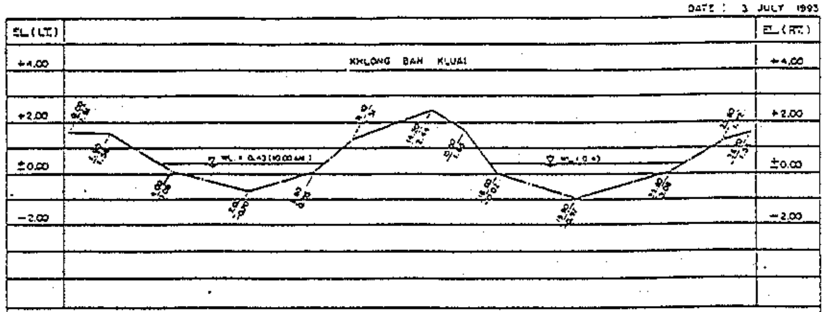

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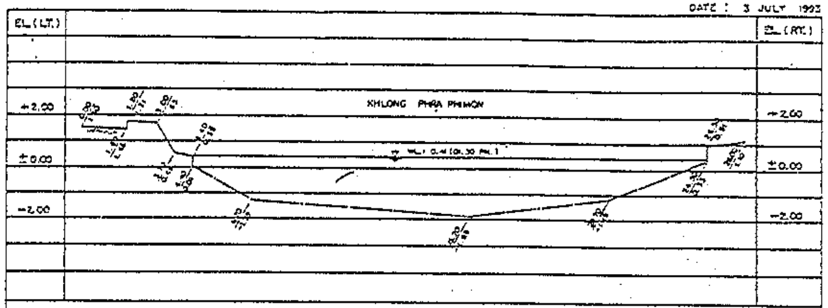

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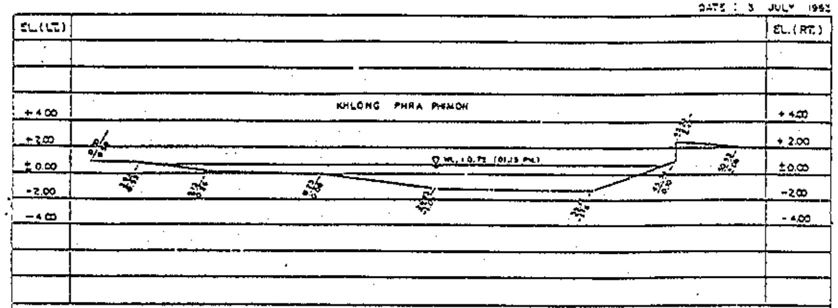

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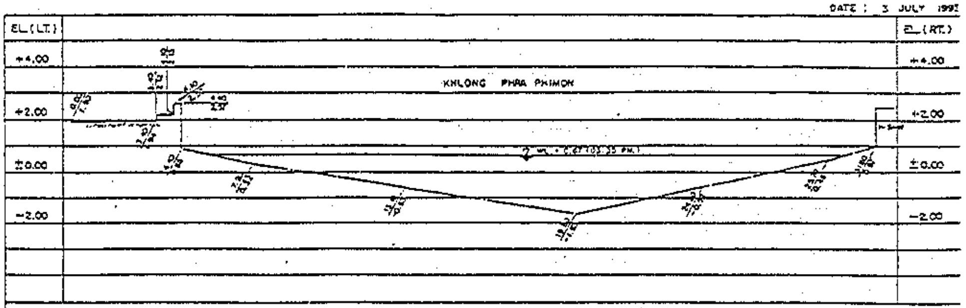

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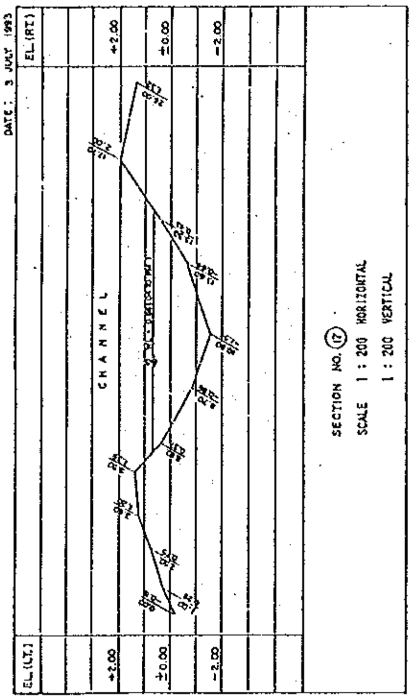
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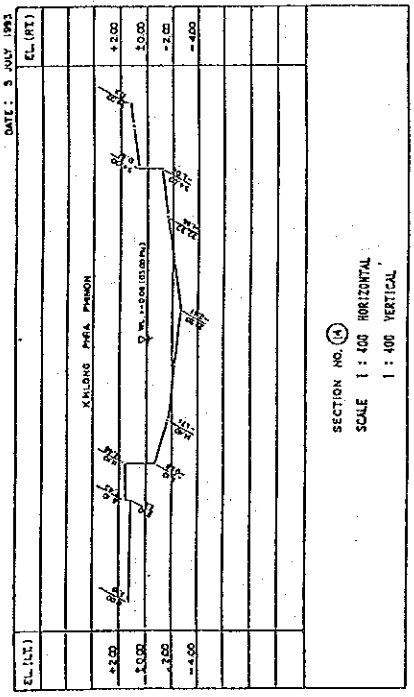
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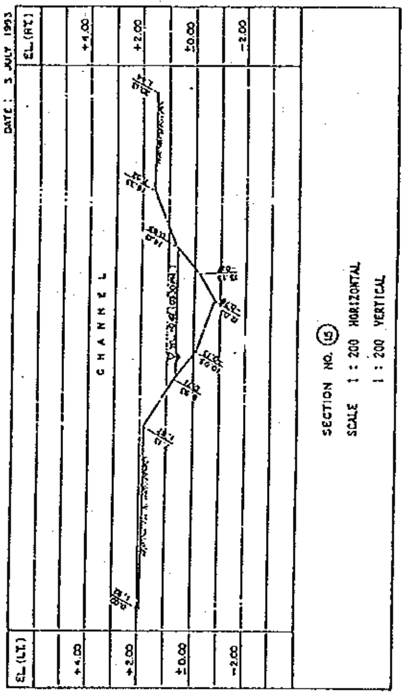
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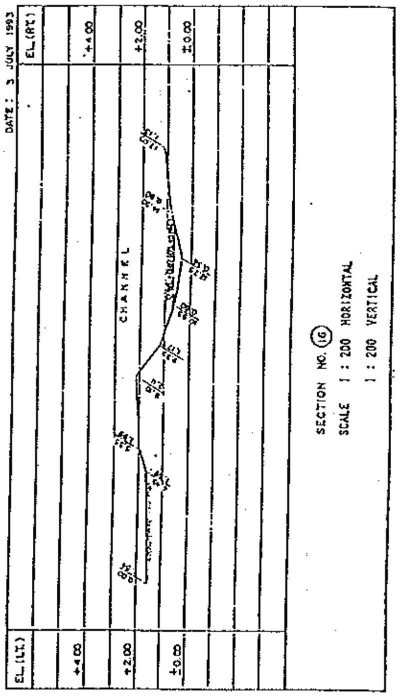
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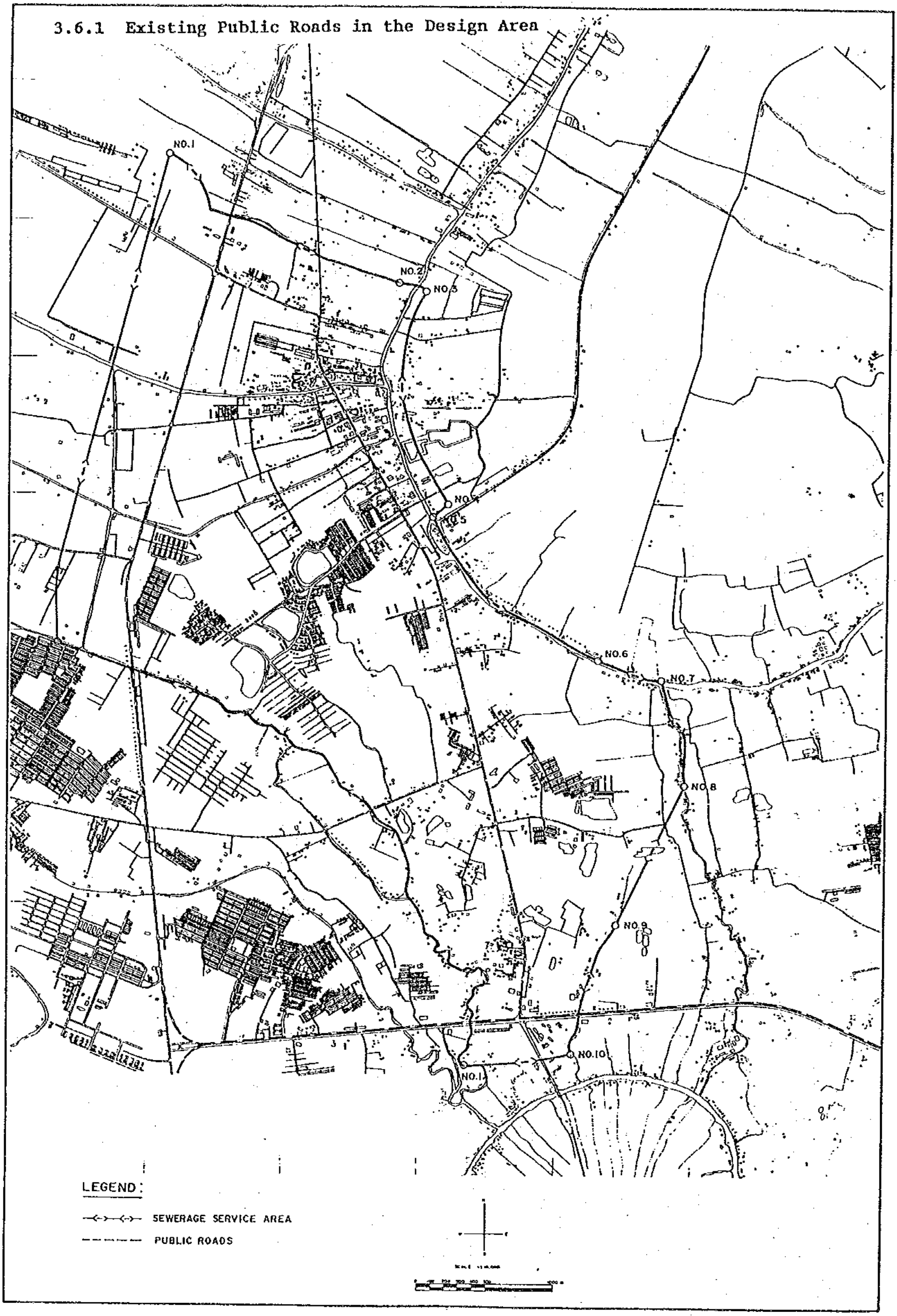
3.6.1 Existing Public Roads in the Design Area



3.7.1 Soil Boring Test Results

1. INTRODUCTION

This report described the scope and results of the soil investigation carried out on the site of the proposed Water Treatment Plant which is located in Amphoe Bang Bua Thong, Nonthaburi Province. The purpose of the investigation was to estbalish the types and characteristics of the subsoil strata, and to enable recommendations to be made concerning foundation types, founding level, safe bearing capacity for the range of structures envisaged for storage services.

The investigation was carried out during the month of July, the dry season so that it may be said that dry conditions prevailing in the subsoils.

2. SITE DESCRIPTION

The site was located in Amphoe Bang Bua Thong, Nonthaburi Province which is in an area of the Chao Phraya River Basin of Thailand as shown in Figure 1. Most area of the site was covered with the fill soil about 1.00 metre thick. Exception was on Borehole 3, the site was covered with an organic top soil about 0.30 metre thick.

3. FIELD WORKS

3.1 Locations of Boreholes

Three boreholes were required to drill to about the maximum depth of 21.45 metres below existing ground surface. The exact locations of the boreholes were shown in Figure 2.

3.2 Field Tests

Standard methods for field testing soils for engineering purposes have been established by the American Society for Testing and Materials (ASTM,1958). The basic concepts are presented belows:

Standard penetration test (SPT)

In the standard penetration test a soils for engineering purposes spoon is used. It is an open-ended steel cylinder which splits longitudinally into two halves. These two halves are held together by a cutting shoe at the lower and a coupling which connects the sampler to the drill rod. The split spoon is driven 45 cm. into the ground by means of a 63.5 kg. weight (hammer) falling a free height of 76 cm. The number of hammer blows for each 15 cm. penetration is recorded. The total number of blows required to drive the second and third 15 cm. of penetration is called the standard penetration resistance N which represents mumber of blows per 30 cm. (Terzaghi and Peck, 1948). After the blow counts are recorded, the spoon is withdrawn from the borehole and a representative sample is secured. These samples are kept in airtight jars with proper identification for visual examination and or laboratory tests.

Thin-walled Tube Sampling (Shelby Tube)

For moderate to large jobs the shear strength of the cohesive soil should be determined from relatively undisturbed samples. This is usually done by taking samples from the borehole by means of a seamless thin-walled steel tube commonly known as a Shelby tube. The tube is 5.08 cm. or 7.6 cm. in deameter and has a bevelled butting edge at the lower end. It is connected to the drill rod and pushed by static force into the bottom of the hole. When the tube is almost full (avoid over-penetration), it is withdrawn from the hole, removed from the drill rod, sealed at both ends with parafin, and shipped to soil laboratory for tests.

Vane Test

A vane tester consists of a pair of thin steel blades connected to a vertical shaft. The tester is pushed into the ground or into the bottom of a borehole and a torque is applied on the shaft. If the shaft is kept free from the surrounding soil by means of a casing, the torque required to shear the soil along the cylindrical surface with blades size (Skempton, 1950).

3.3 Drilling

The rig used was a skid-mounted type, provided with equipment for carrying out augering, wash boring, vane tests and penetration tests.

In all cases,the hole was cased to about 10.00 metres depth to prevent washing out of the holes and to permit more efficient water recirculation during wash boring. The procedure in the field at each borehole location commenced with an in situ vane shear test in soft soil by using a standard vane types at a depth of 1.50 metre intervals and also a standard penetration test at the same depth intervals. These results were plotted on the boring log, Figure 3.

4. SOIL TPYES ENCOUNTERED

Boreholes over the site were taken to a maximum depth of about 21.45 metres. Over this range of depth, three broad and distinct strata of materials were encountered, and are described in general terms belows:

Stratum A: This material usually appeared directly below fill soil and extended to the depth between 10.50 to 11.70 metres below existing ground surface around an area of Boreholes. It consists of silty clay, with occasional organic matters and very fine sand, varying in color from brown to dark grey. The consistency as measured by a vane device, was in the range very soft to medium. The soil in this stratum was classified as CL, ML-OL & CH.

Stratum B: This material was encountered beneath Stratum A and extended to a depth between 13.40 to 14.80 metres below existing ground surface around an area of Boreholes. It consists of silty clay, with occasional very fine sand, varying in color from dark grey to brown. The consistency as measured by a pocket penetrometer, was in the range medium to very stiff. The soil in this stratum was classified as CL.

This material did not appear around an area of Borehole 3.

Stratum C: Lying beneath Stratum B and extended to the end of the boreholes at a maximum depth of about 21.45 metres below existing ground surface around an area of the boreholes. It consists of silty very fine sand with occasional coarse sand and gravel, varying in color from grey, yellow to brown. Relative density as measured by a split spoon, was in the range loose to very dense. The soil in this stratum was classified as SM & SP-SM.

There was a thin substratum of very dense light grey and brown clayey very fine sand (C1) in the middle depth of this stratum. This soil was classified as SC.

5. LABORATORY WORKS

Standard methods for testing soils for engineering purposes have been established by the American Society for Testing and Materials (ASTM,1958). The basic concepts of the more common tests are presented below. Details of the testing programme carried out in the laboratory, using selected samples, are listed below. Results of the tests are given in Section 6 and Appendix.

5.1 Unit weight

Unit weight of a granular soil is difficult to determine except where the soil is at the ground surface. Granular soil recovered by a sampler is highly disturbed and gives no indication whatsoever of its original unit weight. In practice, the unit weight of soils is estimated from the results of penetration tests.

Unit weight of a cohesive soil however, can be readily detemened by measuring the weight and volume of the soil sample. The unit weight of a plastic clay may be computed on the assumption that the clay is 100 per cent saturated.

5.2 Grain size analysis

Grain size distribution of a soil can be determined by sieve analysis down to the size of No. 200 sieve. For determination of smaller fractoins, the wet method must be used. A soil sample is dispersed thoroughly in distilled water. The soil-water mixture is well shaken so that all soil grains are in suspension. By means of a hydrometer, the density of the suspension can be determined. Correlation between the density of the suspension and the diameter of the grains has been worked out on the assumption that all grains are spherical. Results are given in Section 6 and Appendix.

5.3 Water content

The natural water content of a soil sample is determined by weighting the sample before and after it is dried in the oven under controlled temperature.

The natural moisture contents of the majority of the samples recovered were measured in order to determine a moisture content profile. Values are given in Section 6 and Appendix.

5.4 Atterberg Limits

Liguid Limit

The liquid limit of a soil is the water content at the boundary between the liquid and plastic states. A soil sample (with grains passing No. 40 sieve) is thoroughly mixed with water and is placed in the dish to a thickeness of 2.54 cm. at the bottom of the dish. A groove of 1.27 cm. width is cut in the middle of the sample. The dish is lifted and dropped by turning the crank. The number of drops required to closed this 1.27 cm. groove is recorded. The liquid limit is the water content at which 25 drops of the dish will close the 1.27 cm. groove.

Plastic limit

The plastic limit of a soil is the water content at the boundary between the plastic and semisolid states. The water content at the boundary is arbitrarily defined as the lowest water content at which the soil can be rolled into threads 3.2 mm. in the diameter without the threads breaking into pices.

Plastic and liquid liumits of the majority of the cohesive soil samples recovered were measured in order to determine the different states of soils along the depth. Values are given in Section 6 and Appendix.

5.5 Specific Gravity

The specific gravity of a substance is the ratio of its weight to the weight of an equal volumn of water. The specific gravity of a mass of soil or rock (including air, water, and solids) is termed mass specific gravity or apparent specific gravity.

Measurement using a standard density bottle were taken on the majority of the recovered samples. Results are shown in Section 6.

5.6 Unconfined compression test

A relatively undistubed soil sample, usually secured by means of a thin-walled tube, is subjected to an axial compression in a manner similar to the test of a concrete cyinder. For plastic clays, the unconfined compression strength is taken at 20 percent strain of the sample. The sample of a stiff soil, however, will break before reaching the 20 percent strain, for most practical cases,the shear strength of a cohesive soil may be taken as one-half of its unconfined compression strength.

5.7 Direct shear test

The test is conducted by means of a shear box or other variations of this aparatus. A shear box is a sample container which is split in the mid-height. When a normal force N is applied the forec required to start the movement of the upperhalf of the sample withrespect to the lower half is measured. This test is very useful in measuring the relationship between the shear strength and the angle of internal friction of granular soil. The sand samples were selected for the Direct shear test. Results of all the above tests are given in Section 6 and Appendix.

6. TEST RESULTS

A summary of values obtained from laboratory works, are given belows:

6.1 Natural Water Contents

	Range(%)	Average(%)
Stratum A:	36.47-90.06	66.20
Stratum B:	23.48-52.02	39.97
Stratum C:	14.13-25.88	18.85

	Range(%)	Average(%)
C_1:	14.40	14.40

6.2 Atterberg Limits

	Range(%)		Average(%)	
	LL	PL	LL	PL
Stratum A:	24.95-48.92	16.12-26.25	36.72	21.62
Stratum B:	27.84-42.00	15.16-19.04	33.97	17.65
Stratum C:	20.55	N.P.	20.55	N.P.
C_1:	24.60	12.06	24.60	12.06

6.3 Specific Gravity

	Range(%)	Average(%)
Stratum A:	2.48-2.68	2.62
Stratum B:	2.59-2.70	2.63
Stratum C:	2.57-2.61	2.59
C_1:	2.61	2.61

6.4 Bulk Density

	Range(cu.m.)	Average(cu.m.)
Stratum A:	1.50-1.84	1.64
Stratum B:	1.71-2.05	1.83
Stratum C:	1.38-2.27	2.08
C_1:	2.18	2.18

6.5 Strength Parameters

	Range	Average
Stratum A:		
N,blows/30 cm.	-	-
Vp,tsm.	1.220-2.440	1.540
Uc,ksc.	0.220-0.640	0.400
Stratum B:		
N,blows/30 cm.	3-28	13
Up,ksc.	1.500-3.500	2.330
Uc,ksc.	0.770-1.520	2.630

	Range	Average
Stratum C:		
N,blows/30 cm.	3-110	55.6
*C,Tsm	0.00	0.00
*ϕ,Degree		
C_1:		
N,blows/30 cm.	59	59

*....Direct Shear Test

7. GROUND WATER OBSERVATION

Measurement obtained at twenty four hours after drilling ground water level to exist at the depth between 0.35 to 1.75 metres below existing ground surface around an area of boreholes. Significant fluctuations in the ground water table should be anticipated throughout the year depending upon the amount of precipitation, evaporation and surface ren off.

8. CONCLUSIONS

8.1 General Consideration

In establishing a suitable foundation system for the principal building proposed for the Water Treatment, it is necessary to consider two espects of foundation design namely stability and settlement.

8.2 Allowable Pile Load Capacity

As shown above, the thick layer of soft silty clay of Stratum A is a high compressible layer. The foundation should not be spread footings or raft foundations because it would lead to excessive settlement so that the long pile foundations are recommended.

Study of the SPT-values measured in the borehole showns that at a depth of about 16.00 metres below existing ground surface, the SPT-values are in the range 49 to 110 blows/30 cm. which are high enough to support structures. However, the consolidation of underlying compressible soils, should be taken in to consideration for the foundations design.

8.2.1 Ultimate Pile Load Capacity

The ultimate load capacity of an individual precast concrete pile can be evaluated by the following equation:

$$Q_u = (C_a + K_s.q.\tan \phi)A_s + (C_p.N_c + q.N_q)A_p$$

where Q_u = ultimate load bearing of the pile

C_a = adhesive force (from Figure C.)

= 4.50 + 0.3 (C - 5.0) Tsm.

K_s = lateral pressure coefficient (from Table b)

$\tan \phi$ = coeff. of friction of cohesionless soil (from Table c)

A_s = embedded area

q = effective overburden pressure (from Table a)

N_c & N_q = bearing capacity factors (from Figures A or B)

A_p = cross sectional area of the pile's tip

C = average shear strength

C_p = shear strength at pile tip

The ultimate load capacity of various size piles where as the pile's tops are at 1.00 metre below existing ground surface, are as follows:

I-Section Piles

			Ultimate Load/Pile(tons)			
			I.22	I.26	I.30	I.40
		Perimeter, cm	88	104	120	160
		Tip Area ,sq.cm	310	460	660	1240
	Pile tip elev.	Pile length m.				
BH_1	12.00	11.00	17.26	20.65	24.19	-
	13.00	12.00	24.16	29.20	34.59	-
	14.00	13.00	32.94	40.03	47.72	-

I-Section Piles

			Ultimate Load/Pile(tons)			
	Pile tip elev.	Pile length m.	I.22	I.26	I.30	I.40
BH_1	(cont.)					
	15.00	14.00	47.17	60.58	76.97	-
	16.00	15.00	-	91.69	124.72	207.91
	17.00	16.00	-	99.71	134.89	224.68
	18.00	17.00	-	109.41	145.86	241.46
	19.00	18.00	-	116.80	155.32	258.24
	20.00	19.00	-	122.52	163.74	275.02
	21.00	20.00	-	132.89	175.77	291.80
BH_2	12.00	11.00	18.21	21.95	25.90	-
	13.00	12.00	22.17	26.63	31.30	-
	14.00	13.00	28.28	35.85	45.12	-
	15.00	14.00	42.19	58.87	81.40	-
	16.00	15.00	-	72.45	102.01	198.31
	17.00	16.00	-	78.31	109.79	212.65
	18.00	17.00	-	83.95	117.21	226.99
	19.00	18.00	-	89.84	125.05	241.33
	20.00	19.00	-	95.93	133.21	255.68
	21.00	20.00	-	101.79	141.01	270.02
BH_3	12.00	11.00	14.43	17.72	21.39	-
	13.00	12.00	16.71	21.16	26.38	-
	14.00	13.00	19.64	25.55	32.72	-
	15.00	14.00	26.40	36.27	49.05	-
	16.00	15.00	-	53.30	75.36	152.41
	17.00	16.00	-	64.73	92.42	189.52
	18.00	17.00	-	70.02	99.42	201.82
	19.00	18.00	-	71.34	99.89	197.96
	20.00	19.00	-	69.96	95.92	183.10
	21.00	20.00	-	78.96	109.01	210.50

8.2.2 Negative Skin Friction

where a pile is driven through compressible soil layer to found in relatively firm strata, there is a tendency for the compressible soil around the pile tends to "hang up" on the pile, exerting a downward drag force called the negative skin friction. This force has been calculated according to the effective stress method, using the same basic formula as for positive side friction (ref.8) as follows:

$$Qn = B.Nv.Asn$$

where Qn = maximum negative skin friction

B = negative skin friction coefficient

= K.tan ϕ

= 0.30 for normally consolidated clay with high plasticity

Nv = average effective stress along the pile shaft

Asn = pile shaft area

Substituting these values, the negative skin friction can be determined as follows:

		Negative Skin Friction.Tons/Pile			
		I.22	I.26	I.30	I.40
BH_1	Qn =	16.31	19.27	22.24	29.65
BH_2	Qn =	10.35	12.23	14.12	18.82
BH_3	Qn =	10.22	12.07	13.93	18.58

8.2.3 Allowable Pile Load Capacity

Where the maximum value of the negative skin friction is taken into account, the safety coefficient on this force may be 1.1, while the safety coefficient on the pile working load varying between 2.0 and 2.5. In this report, we adopted 2.5 as a factor of safety. The allowable pile load capacity can be calculated as follows:

Qa = (Qu-1.1Qn)/2.5

where Qa = allowable pile load capacity,tons

Qn = maximum value of negative skin friction

The allowable load capacity of the precast concrete piles of various size and lengths where as the pile's tops are at 1.00 metre below existing ground surface, are as follows:

I-Section Piles

			Allowable Load/Pile(tons)			
	Pile tip elev.	Pile length m.	I.22	I.26	I.30	I.40
BH$_1$	12.00	11.00	2.01	2.48	3.00	-
	13.00	12.00	4.77	5.90	7.17	-
	14.00	13.00	8.28	10.23	12.42	-
	15.00	14.00	13.98	18.45	24.12	-
	16.00	15.00	-	28.20	40.10	70.11
	17.00	16.00	-	31.40	44.17	76.83
	18.00	17.00	-	35.29	48.56	83.54
	19.00	18.00	-	38.24	52.34	90.25
	20.00	19.00	-	40.53	55.71	96.96
	21.00	20.00	-	44.68	60.52	103.67
BH$_2$	12.00	11.00	4.18	5.11	6.13	-
	13.00	12.00	5.76	6.98	8.29	-
	14.00	13.00	8.21	10.67	13.81	-
	15.00	14.00	13.77	19.88	28.33	-
	16.00	15.00	-	23.60	34.59	71.04
	17.00	16.00	-	25.94	37.71	76.78
	18.00	17.00	-	28.20	40.67	82.52
	19.00	18.00	-	30.55	43.81	88.25
	20.00	19.00	-	32.99	47.07	93.99
	21.00	20.00	-	35.33	50.19	99.73

I-Section Piles

			Allowable Load/Pile(tons)			
			I.22	I.26	I.30	I.40
BH$_3$	12.00	11.00	2.71	3.47	4.38	-
	13.00	12.00	3.62	4.84	6.37	-
	14.00	13.00	4.79	6.60	8.91	-
	15.00	14.00	7.49	10.88	15.44	-
	16.00	15.00	-	16.01	24.01	52.79
	17.00	16.00	-	20.58	30.84	67.63
	18.00	17.00	-	22.69	33.64	72.55
	19.00	18.00	-	23.22	33.83	71.01
	20.00	19.00	-	22.67	32.24	65.07
	21.00	20.00	-	26.27	37.48	76.03
			+20.92	+31.05	+44.55	+83.70
			*13.95	+20.70	+29.70	+55.80

\+ Pile Bearing Capacity based on Concrete Strength (fc'=300 ksc.)

* Pile Bearing Capacity based on Concrete Strength (fc'=200 ksc.)

() Pile Bearing Capacity based on Underlying Soil Strength

8.3 Conlusion and Recommendations

8.3.1 The subsoils consist generally of silty clay and silty very fine to coarse sand. These soils are variable in colour and texture extending to a depth of 21.45 metres below existing ground surface.

8.3.2 These soils exist in very soft to medium consistency of silty clay, stiff to very stiff consistency of underlying silty clay and medium state to very dense state of silty very fine to coarse sand at the end of boreholes.

8.3.3 The recommended foundation system for the proposed building, consists of the pile foundation.

8.3.4 The allowable load capacity of piles are recommended in item 8.2.

8.3.5 The allowable pile load capacity is determined by the conventional equation with theoretical concern. If the higher pile load capacity is required, the pile load test must be performed and evaluated their possibility.

8.3.6 The hard driving may occur in sand sothat water jetting should be needed to avoid damage in reaching full desired embedment. Final sets should be obtained by driving without water jetting, when dynamic formulas might be used.

8.3.7 The safety factor 2.5 is adopted.

8.3.8 The working load of a desing pile must not be higher than the allowable load of a pile's structures.

8.3.9 The consolidation of underlying compressible soils should be taken into consideration for foundation design.

8.3.10 The concept of design and construction enclosed herewith must be taken into consideration.

8.4 Concept of Design and Construction

8.4.1 In evaluation of a foundation stability, the designer should take into consideration the following:

8.4.1.1 To take in to consideration, the effect of grouping on pile bearing capacity.

8.4.1.2 To determine a vertical stress in the soil surrounding and below the pile tips by using elastic theory. If the calculated stress exceeds the ultimate bearing capacity of the layer, the design should be revised by reducing the pile load capacity, by increasing the pile spacing or by extending the pile to another layer below.

8.4.1.3 To determine the differntial and total settlement of the foundation. If the ultimate differential and total settlement are found to be excessive, the design should be revised by trying the other type of foundations,by increasing the pile spacing, or by reducing the pile load capacity.

8.4.2 All piles should be driven to sufficient depth to carry the purposed load. Final penetration refusal criteria should be determined in the field, dependent upon the type of pile driving equipment used.

8.4.3 To obtain the required penetration, piles heavier than 3.5 tons should be driven by a hammer with a rated energy of at least 4.10 metre-tons. To obtain the required penetration and to prevent pile breakage, piles lighter than 3.50 tons should be driven by a hammer with a rated energy greater than 2.77 metre-tons, and less than 4.10 metre-tons.

8.4.4 All pile driving operations should be carefully inspected with complete and accurate records kept for each pile driven, such as final penetration,pile heavy and amount of downward movement on redriving tip and top elevation, length of pile and amount of cut off.

8.4.5 To minimize ground quake to the adjacent structure, all pile within 12.00 metres of the existing structures should be set in holes prebored to a depth of 6.00 to 10.00 metres below the existing ground level of other method which the engineer approved to be effective.

8.4.6 The design of ground floor structures must take the problem of land settlement due to the load of fill, structures and other into consideration.

9. General

The analysis and recommendation submitted in this report are based upon available information. Since it is possible for variation in soil conditions to exist between boring locations, it is recommended that the soil excavation and the pile driving should be inspected by an experienced soil engineer to assure that the construction is operated conformance with recommendation and foundation are seated upon suiiable materials.

This report has been prepared in order to aid in the evaluation of this site and to assist the engineers in the design of the project, based on our understanding of design details, criteria and utilization of the project as outlined herein. Also, if our understanding of the design and utilization is not correct, we should be promptly informed of the correct data sothat we may revise our recommendations as appropriate.

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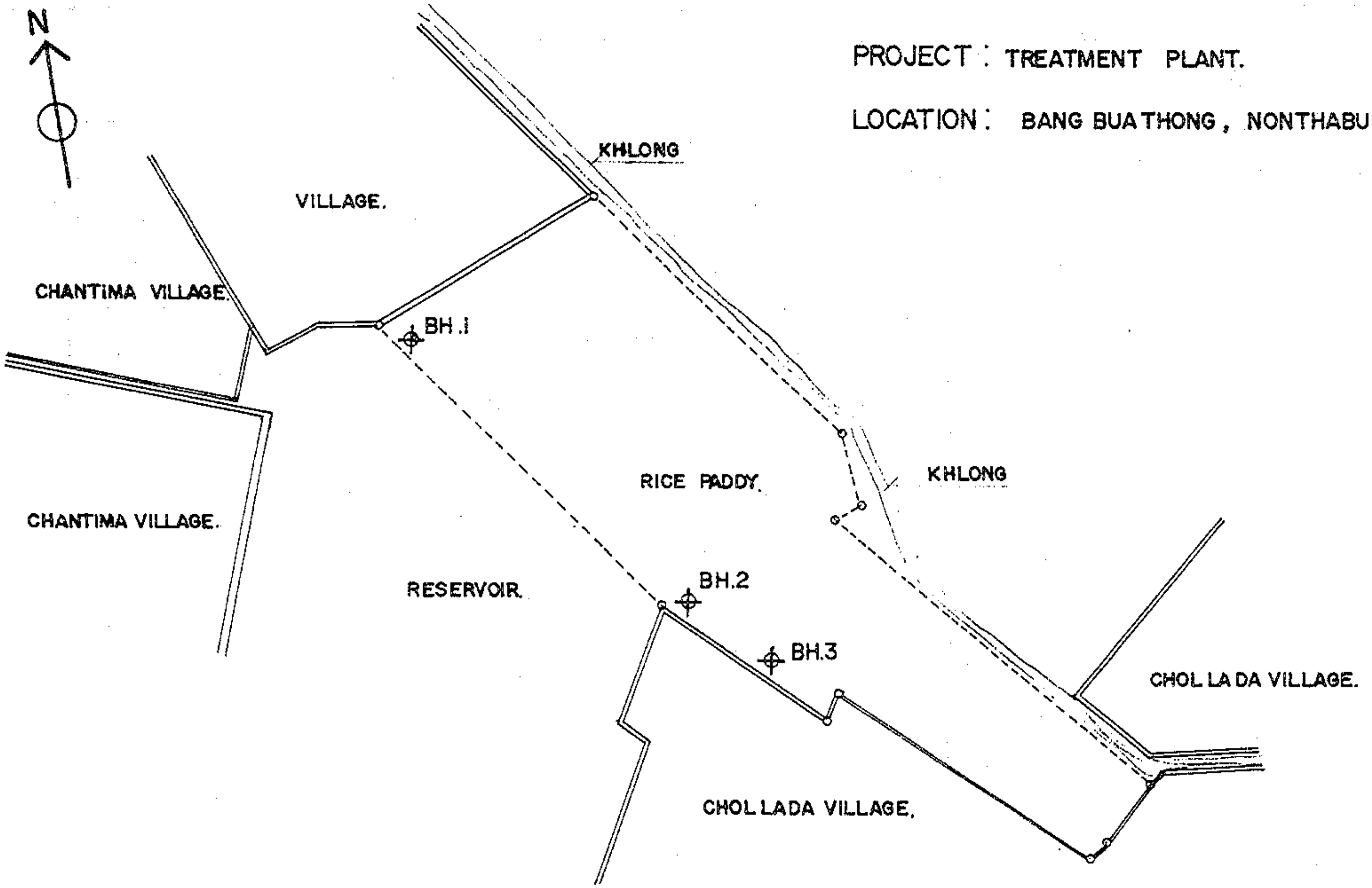
PROJECT : TREATMENT PLANT.

LOCATION : BANG BUATHONG , NONTHABURI.



ผังบริเวณตำแหน่งหลุมเจาะ

PROJECT.

TREATMENT PLANT

FIG. 3

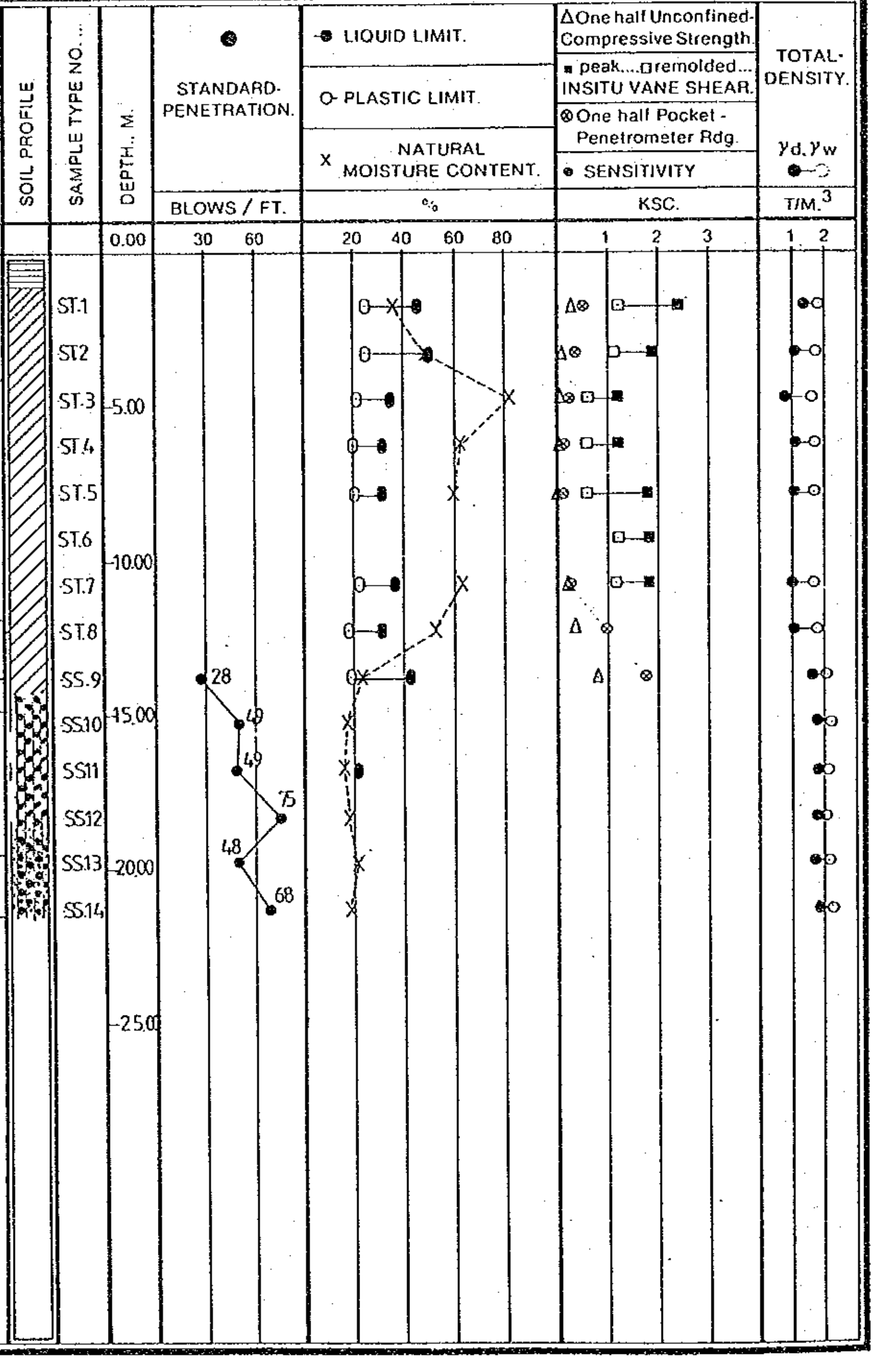
GROUND WATER OBSERVATION.			
DATE	TIME	EL. of HOLE	EL.of WATER
24 HR.AFTER BORING.		1.75 M.	

W.A.C. BORING LOG

LOCATION. Bang buatong, NONTHABURI.

BORING NO. 1
SURFACE ELV.
DATE START 31 / 7 / 36
DATE FINISH 31 / 7 / 36

SOILS DESCRIPTION...	
	0.00
Fill soil	1.00
Medium brown and brownish grey silt clay CL.	4.00
Very soft to soft dark grey silty clay, occasional very fine sand. CL.	11.70
Very stiff brownish dark grey silty clay. CL.	13.60
Very stiff brown silty clay. CL	14.80
Dense to very dense yellowish brown silty very fine sand. SM.	19.50
Dense to very dense grey and brown very fine to coarse sand. SP-SM	21.45
END OF BORING	



3-36

PROJECT.

TREATMENT PLANT

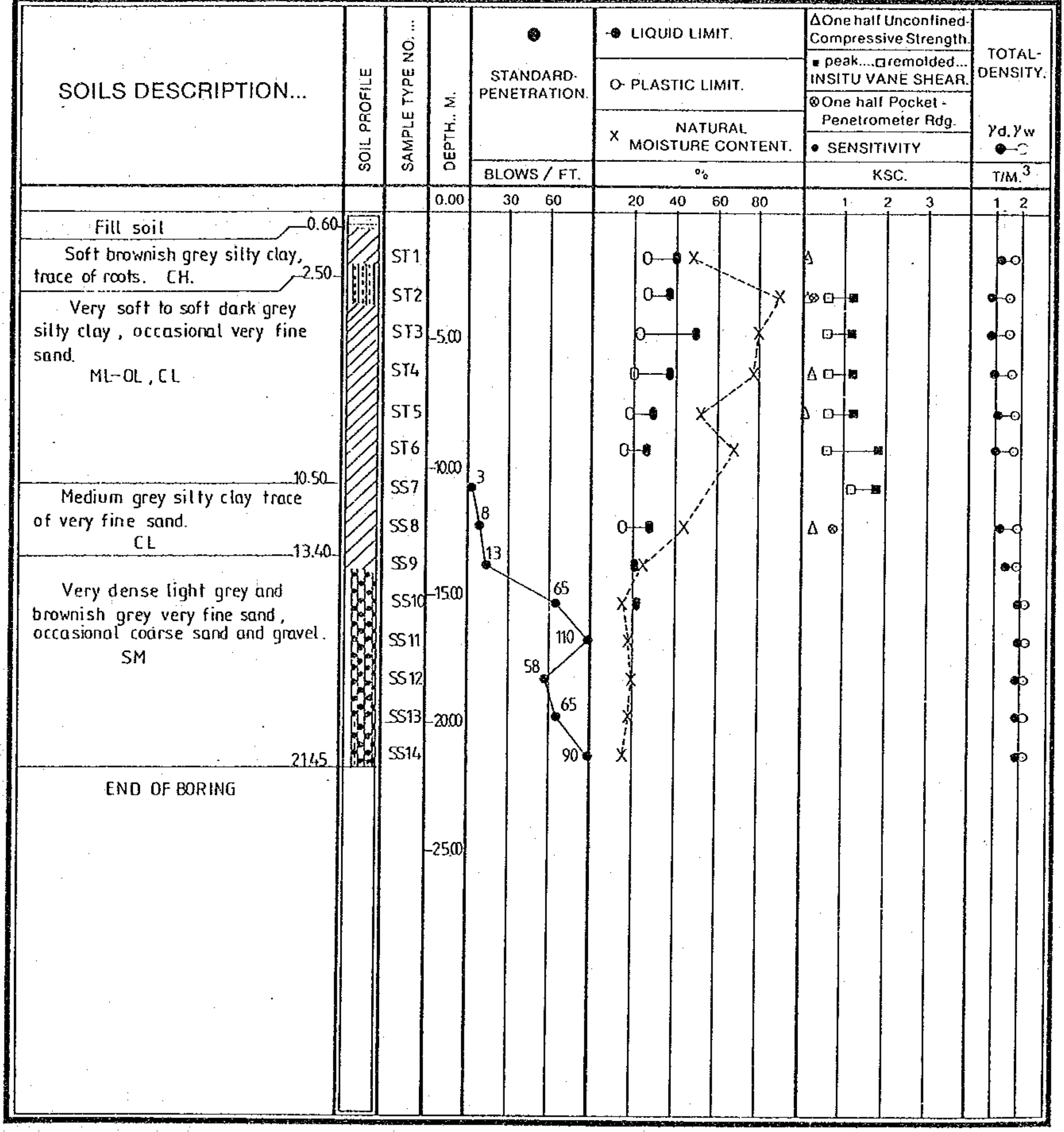
FIG. 4

GROUND WATER OBSERVATION.			
DATE	TIME	EL. of HOLE	EL.of WATER
24 HR.AFTER BORING.			0.55 M.

W.A.C. BORING LOG

LOCATION. BANG BUATHONG,NONTHABUR

BORING NO. 2
SURFACE ELV. -
DATE START 31/7/36
DATE FINISH 31/7/36



SOILS DESCRIPTION...

Fill soil 0.60

Soft brownish grey silty clay, trace of roots. CH. 2.50

Very soft to soft dark grey silty clay, occasional very fine sand. ML-OL, CL 10.50

Medium grey silty clay trace of very fine sand. CL 13.40

Very dense light grey and brownish grey very fine sand, occasional coarse sand and gravel. SM 21.45

END OF BORING

PROJECT.

TREATMENT PLANT

FIG. 5

W.A.C. BORING LOG

GROUND WATER OBSERVATION.			
DATE	TIME	EL. of HOLE	EL.of WATER
24 HR.AFTER BORING.		0.35 M.	

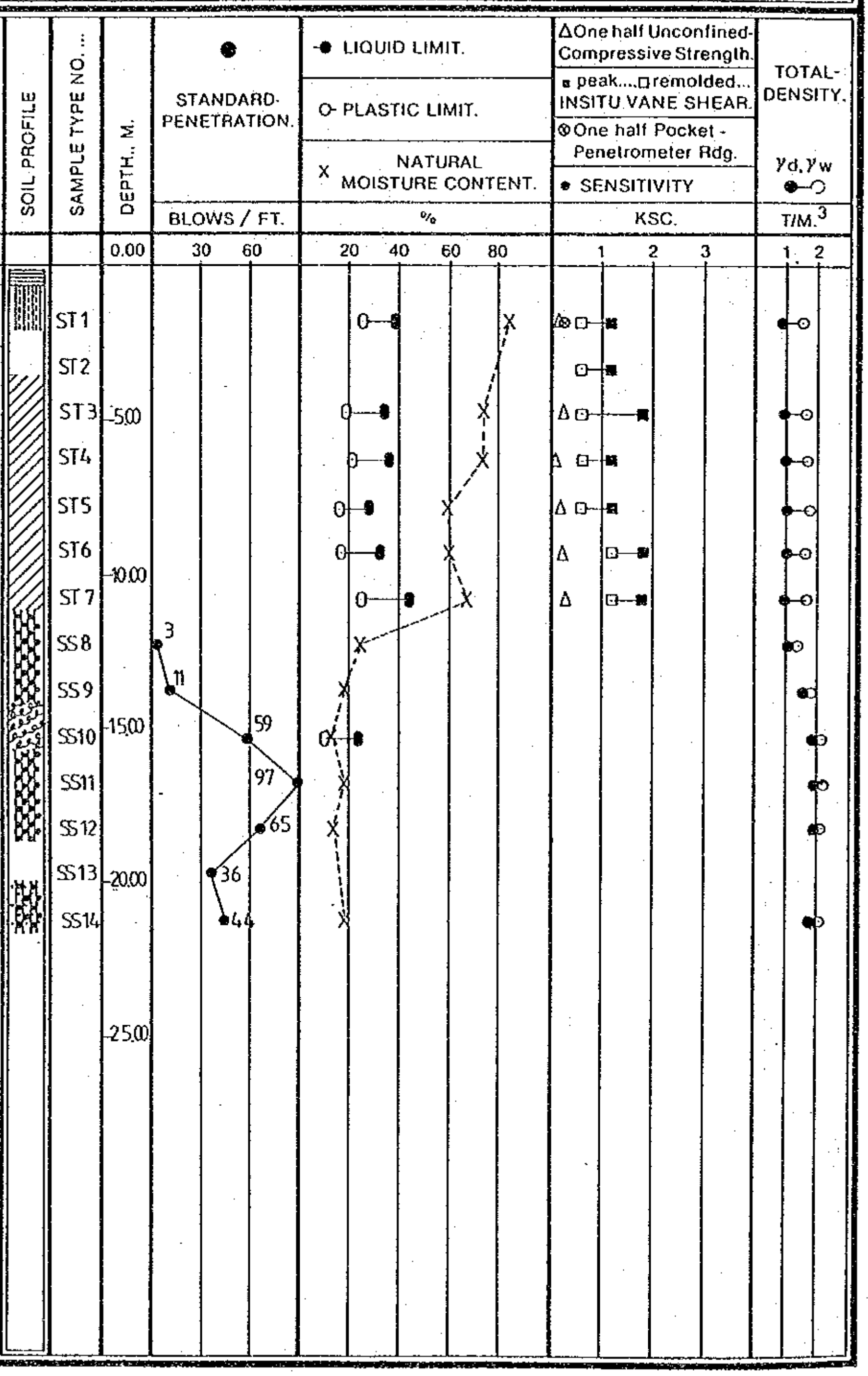
LOCATION.Bang bua thong,NONTHABURI

BORING NO. 3
SURFACE ELV.
DATE START 31/7/36
DATE FINISH 31/7/36

SOILS DESCRIPTION...

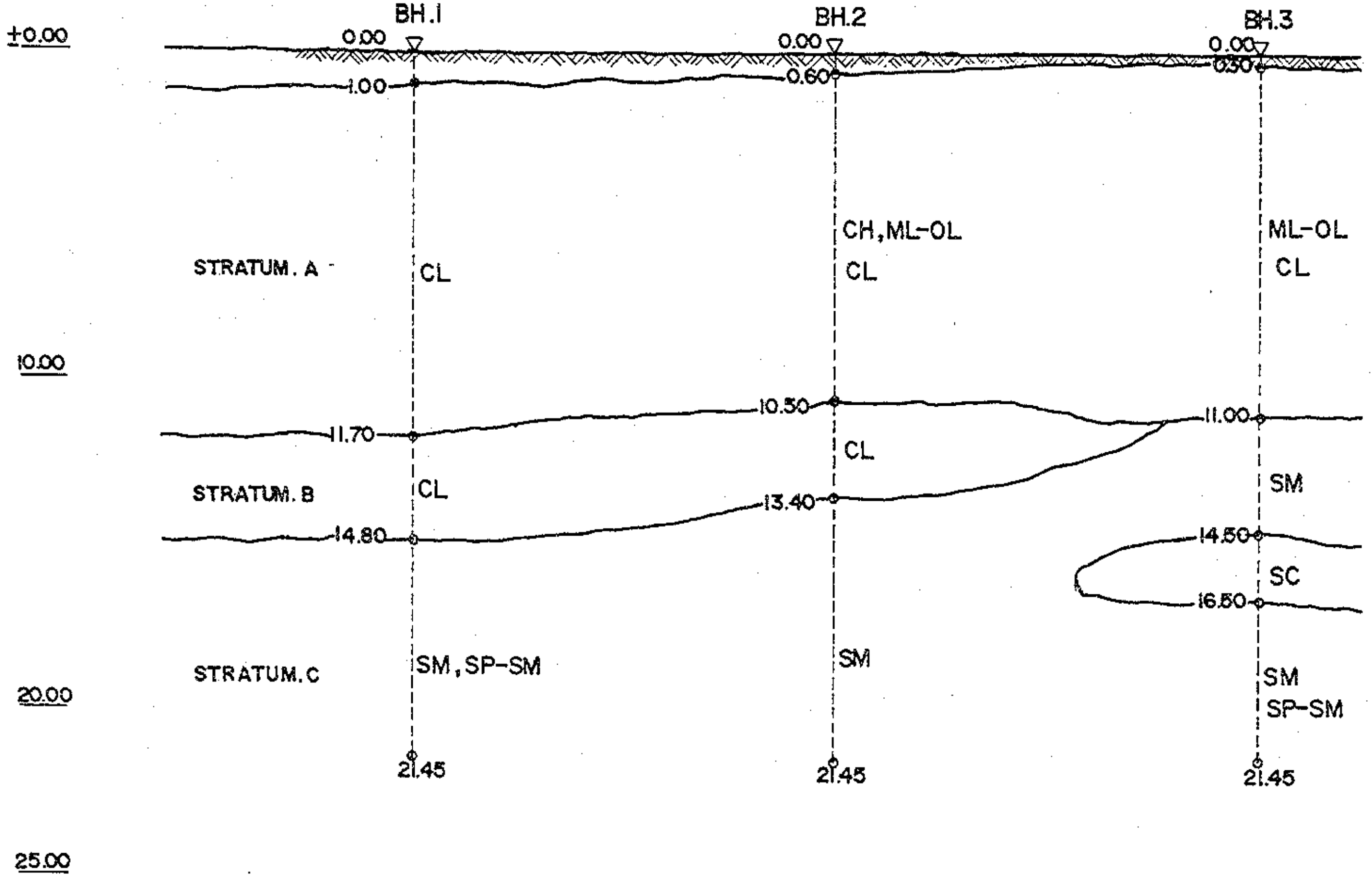
Organic top soil	0.30
Soft brownish dark grey silty clay, occasional organic matters ML-OL.	2.30
Very soft to soft dark grey silty clay, occasional very fine sand. CL.	
Loose to medium grey silty very fine sand. SM.	11.00
Very dense light grey and brown clayey very fine sand. SC.	14.50
Very dense light grey and brownish grey very fine sand. SM.	16.50
Very dense to dense yellow very fine sand	18.00
Dense brown very fine sand, occasional coarse sand and gravel. SP–SM.	21.00 21.45

END OF BORING



SUB SOIL PROFILE

WATER TREATMENT PLANT.
BANG BUA THONG.
NONTHABURI.



GRADING CURVES

TEST BY.

DATE.

SAMPLE BY.

DATE.

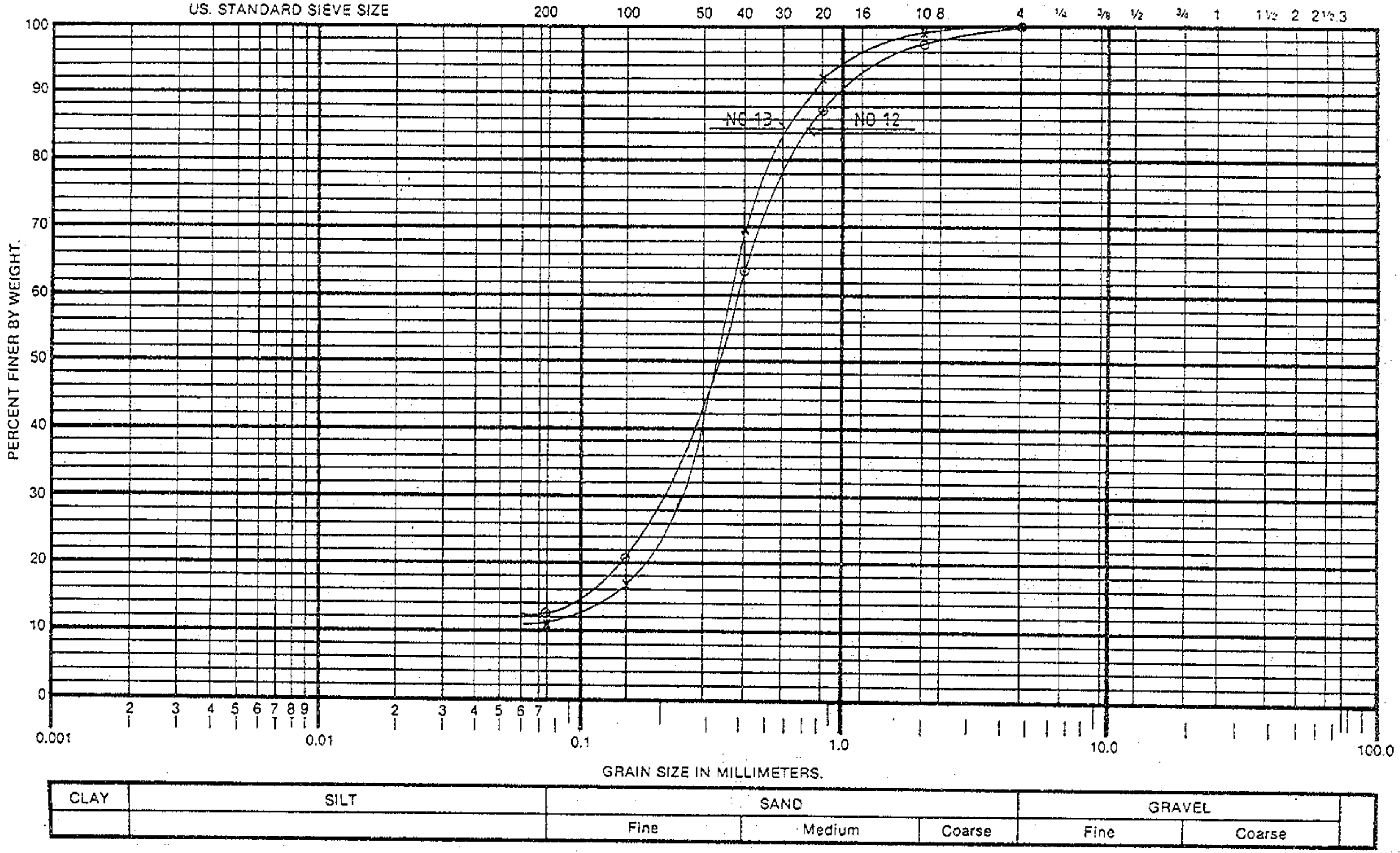
SAMPLE NO. 12, 13

DEPTH. 18.00–18.45 , 19.50–19.95 m.

BORING NO. 1

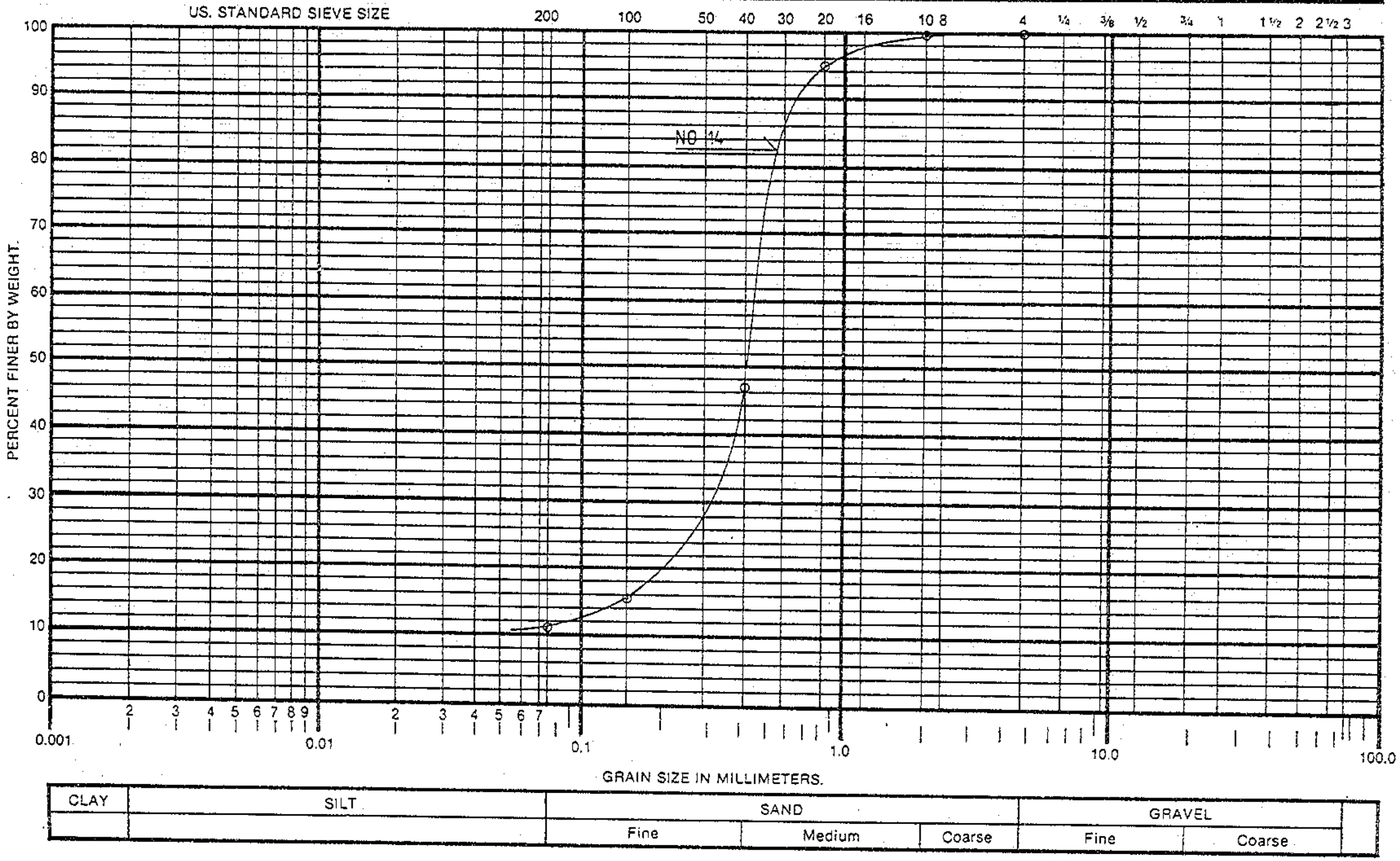
STATION. BANG BUA THONG, NON THABURI.

PROJECT. TREATMENT PLANT.



GRADING CURVES

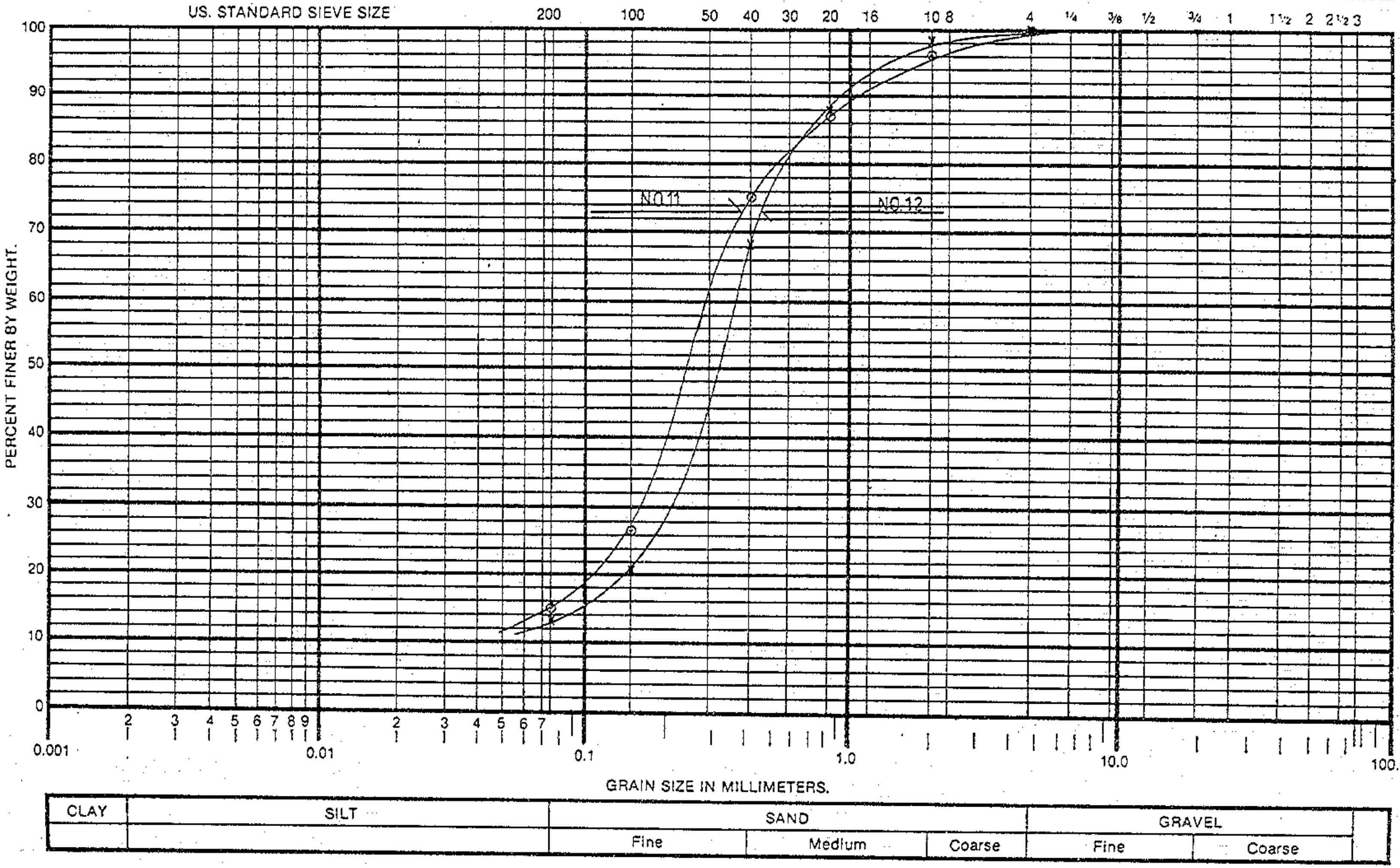
TEST BY.
DATE.
SAMPLE BY.
DATE.
SAMPLE NO. 14
DEPTH. 21.00–21.45 m.
BORING NO. 1
STATION. BANG BUA THONG, NONTHABURI
PROJECT. TREATMENT PLANT.



GRADING CURVES

TEST BY.	DATE.
SAMPLE BY.	DATE.
SAMPLE NO. 11, 12	DEPTH. 16.50 – 16.95 , 18.00 – 18.45 m.
BORING NO. 2	STATION. BANG BUA THONG , NONTHABURI.

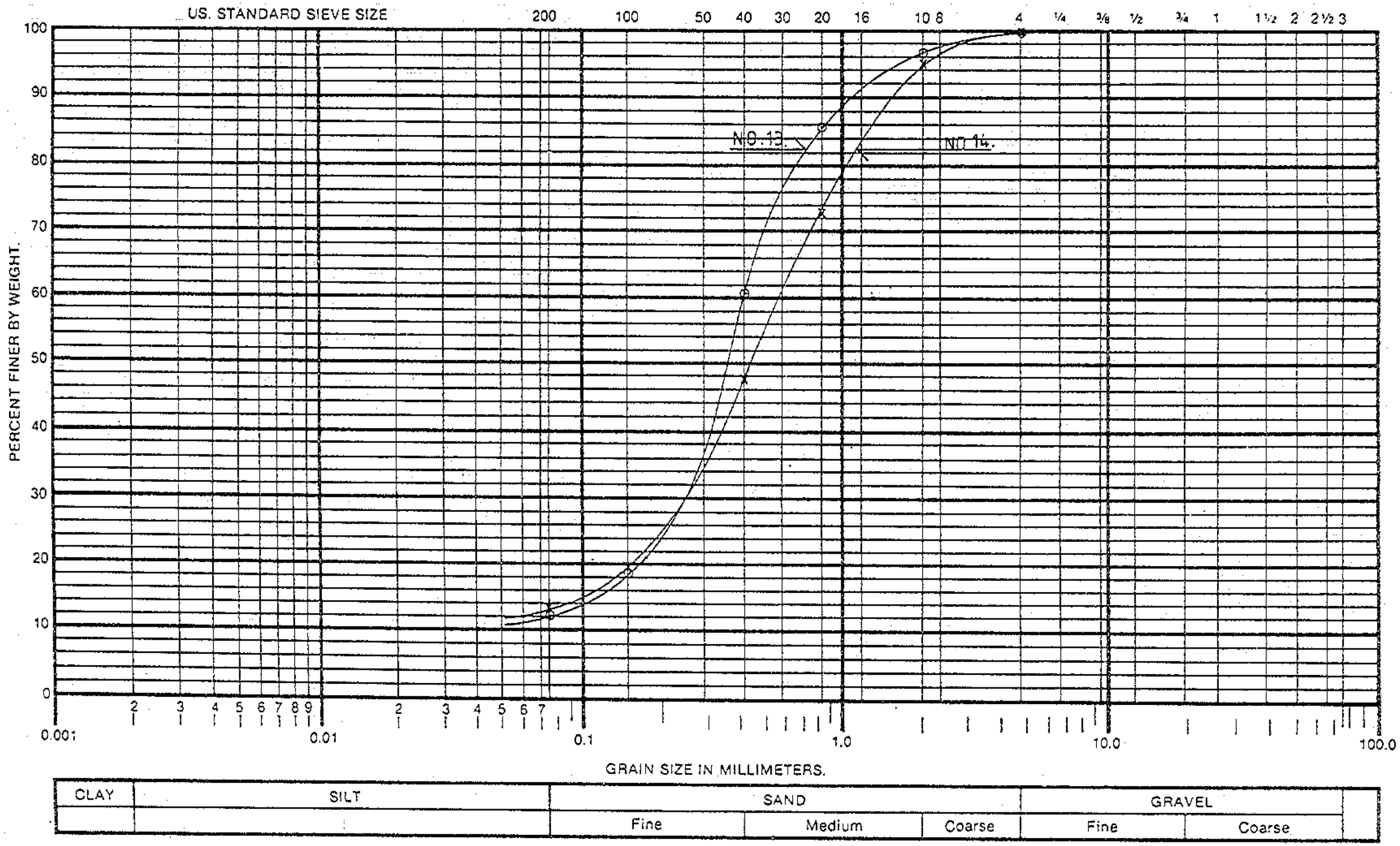
PROJECT. TREATMENT PLANT.



GRADING CURVES

TEST BY.	DATE.
SAMPLE BY.	DATE.
SAMPLE NO. 13 , 14	DEPTH. 19.50-19.95 , 21.00 - 21.45 m.
BORING NO. 2	STATION. BANG BUATHONG , NONTHABURI.

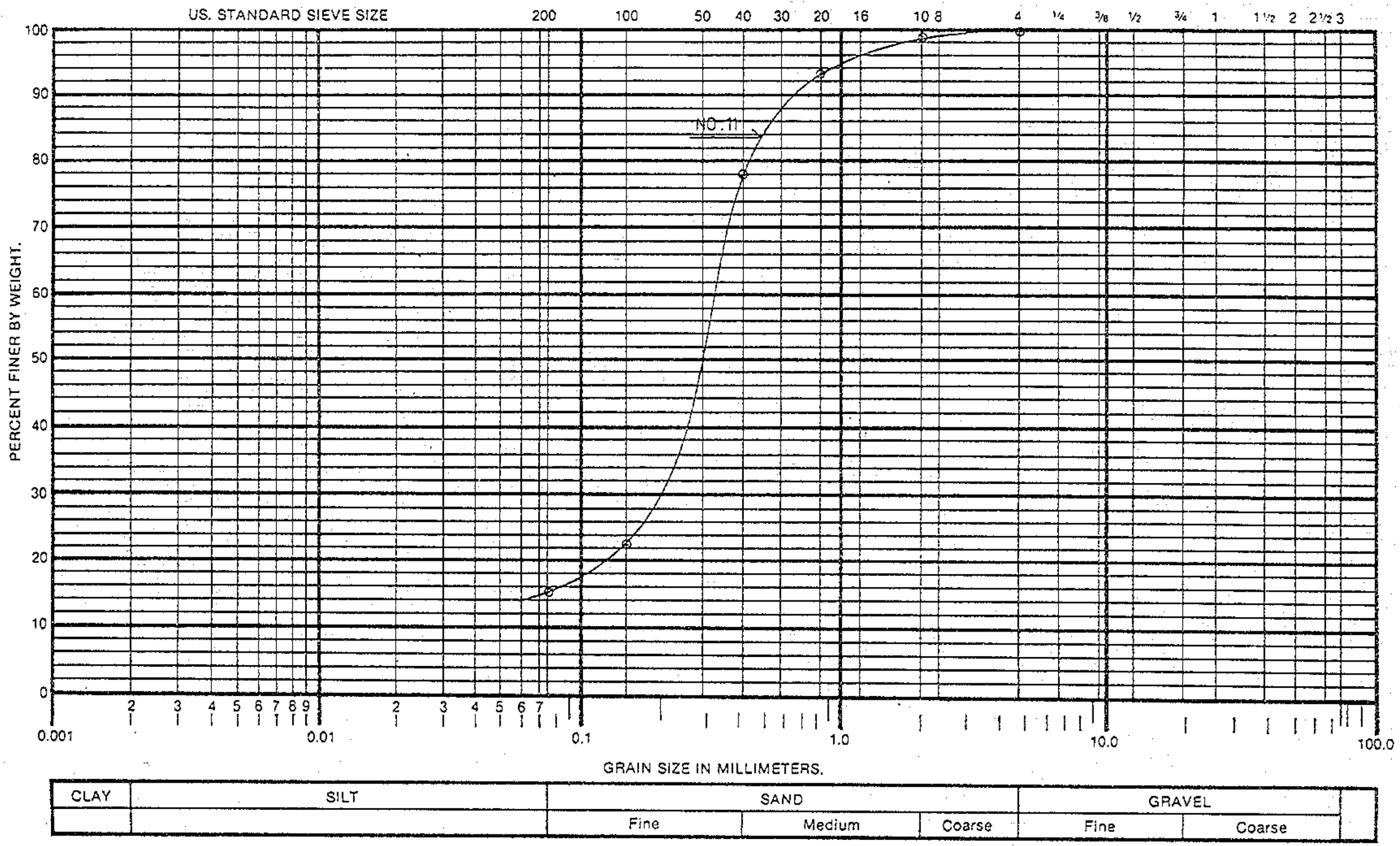
PROJECT. TREATMENT PLANT.



GRADING CURVES

TEST BY.	DATE.
SAMPLE BY.	DATE.
SAMPLE NO. 11	DEPTH. 16.50 – 16.95 m.
BORING NO. 3	STATION. BANG BUA THONG, NONTHABURI.

PROJECT. TREATMENT PLANT.



GRADING CURVES

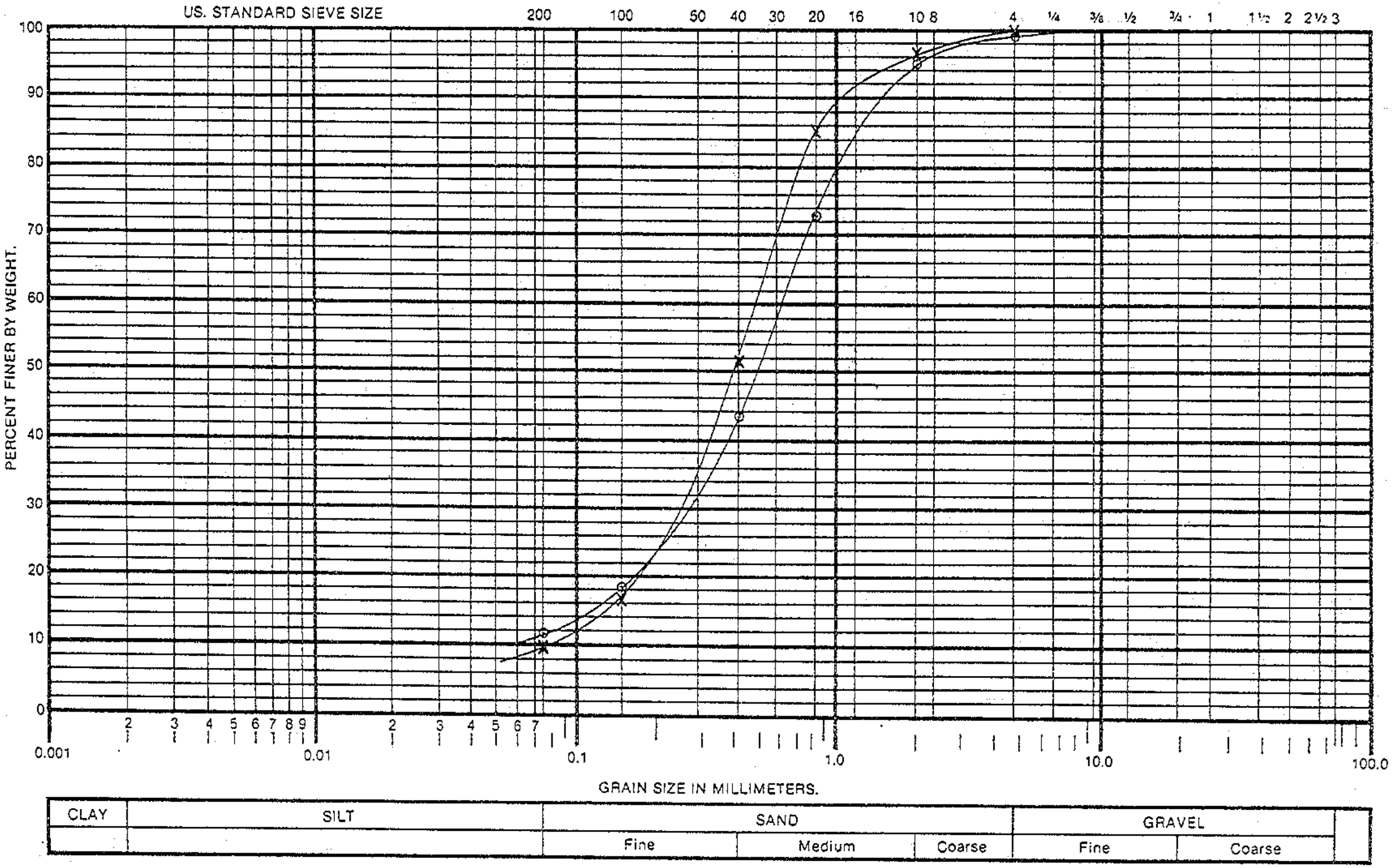
TEST BY. DATE.

SAMPLE BY. DATE.

SAMPLE NO. 12 , 14 DEPTH. 18.00 - 18.45 , 21.00 - 21.45 m.

BORING NO. 3 STATION. BANG BUA THONG , NONTHABURI.

PROJECT. TREATMENT PLANT.



3.13.4 Financial Study

Table 3.13.4.1 Selected Characteristics of Financial Roles

Financial Support from the Central Gov't	1. Population criterion 2. Emergency criterion
Nature of Support	Specifically designated
Deficit Financing	Arrangements could be made
Capital Expenditure	
Land & Construction	Provincial/Central
Equipment	Provincial/Central
Priority of Expenditure	Committee
Determination of Fees	Committee & the Executive Approval
Major Roles of Municipality	Local road, construction, education, health and sanitation

Table 3.13.4.2 IMPLEMENTATION ARRANGEMENTS FOR SEWAGE WORKS

Item	Government		
	Nation	Province	Municipality/S.D
Establishment of sewerage plan Decision on the priority projects	- Investigation & planning by PWD - Approval by cabinet meeting 100 %	0 %	- Willingness on participation 0 %
Land acquisition	- Recommendation on location and land size by PWD 0 % 70 % 1/ 60 % 2/ 75 % 3/	0 %	100 % 30 % 1/ 40 % 2/ 25 % 3/
Design & engineering work	Design Committee		
	PWD staff Designing (by consultants) 100 %	Representative 0 %	Representative 0 %
Construction	Advisory Committee		
	PWD staff Supervision work 100 % 70 % 1/ 60 % 2/ 75 % 3/	Representative 0 %	Representative 0 % 30 % 1/ 40 % 2/ 25 % 3/
Staff training	Training Plan by PWD 100 %	0 %	0 %
Operation and Maintenance of facilities	Technical assistance 0 %	0 %	Responsible 100 %

Note: percentage (%) indicates budgetary arrangements
1/ Fifth National Economic and Social Development Plan
2/ Sixth National Economic and Social Development Plan
3/ Seventh National Economic and Social Development Plan (NESDB)

Table 3.13.4.3 Budgets by Municipality (Unit: 1000 Baht)

Year	Chai 1991	Nat 1990	Sing Buri 1991	Lop Buri 1992	Ang 1992	Thong 1991	Pa 1992	Mok 1991	Se 1992	na 1991	Rang 1991	sit 1990	Bang Bua 1991	Thong 1990
Total Income	24,588	25,866	33,810	85,559	35,278	29,747	27,306	27,691	35,503	25,434	26,406	26,968	41,300	35,383
Income Tax	12,137	10,102	153	52,014	2,783	2,518			17,285	15,312	18,861	14,214	22,752	22,129
Licensing Fees	896	802	2,664	4,566	4,860	4,382			537	411	2,985	2,925	804	589
Property Tax	835	789	1,424	6,919	1,494	1,332	10,450	8,779	3,266	2,601	761	1,114	3,450	2,538
Utility Tax	357	297	476	1,488	1,584	933			236	419	1,059	944	325	300
Miscellaneous	319	256	16,197	890	0	0			287	190	70	480	1,517	516
Support(C Gov't)	10,042	8,963	7,814	17,682	0	0	16,856	18,912	7,663	4,393	2,620	2,046	10,688	7,569
Loan	2	4,657	5,082	0	24,557	20,582			6,229	2,108		5,245	1,764	1,742
Total Expenditures	22,111	23,433	30,810	76,124	31,659	28,373	25,488	26,753	30,163	19,564	19,136	21,464	38,736	33,221
Expenditure(C Gov't)	359	270	6,516	5,280			15,924	18,369	1,384	944	601	421	821	1,114
Office Exp.	8,936	10,281	20,227	34,272			9,564	8,384	15,430	11,776	6,247	5,499	18,040	15,720
Fixed Investment	1,211	229	0	23,094					6,129	4,735	9,668	8,996	8,543	7,660
C Gov't Investment	9,036	7,996	0	13,478					990		2,620	2,046	7,497	6,250
Balance for Investment	2,569	4,657	4,067	0					6,230	2,109		4,502	1,960	622
Investment for Materials	0	0	0	0									1,764	1,748
													111	87

Table 3.13.4.4 Average Monthly Household Income and Expenditures of Municipalities in Central Region, 1988

Total Income	6,145	Total Expenditure	5,735
Money Income	5,056	Consumption Expenditures	5,131
Non Money Income	1,008	Food & Beverage	2,055
Other Money Receipts	81	Apparel & Footwear	254
		Household Operation	1,331
Saving	410	Piped water	55
		Ground water	5
		Electricity & fuel	282
		Medical Care & Services	341
		Personal Care	158
		Transport & Communication	672
		Recreation & Reading	150
		Education	87
		Miscellaneous	83
		Non Consumption Expenditures	604

Source : Report of the 1988 Household Socio-Economic Survey Central Region

Table 3.13.4.5 Average Monthly Household Expenditures in Central Region, 1986 and 1990

	1986	Percent	1990	Percent
Total Expenditure	4,187	100	5,931	100.0
Consumption Expenditures	3,849	91.9	5,366	90.5
Food & Beverages	1,631	38.9	2,201	37.1
Alcoholic Beverages	54		91	
Tobacco Products	75		90	
Apparel & Footwear	216		298	
Housing	1,002	23.9	1,452	24.5
Medical Care	148		175	
Personal Care	113		144	
Transport & Communication	360	8.6	642	10.8
Recreation & Reading	97		121	
Education	57		72	
Miscellaneous	96		80	
Non Consumption Expenditures	338	8.1	565	9.5
Average Household Size	4.2		4.1	

Source : Statistical Yearbook of Thailand 1992, Table 15.3

Table 3.13.4.6 Selected Housing Characteristics, 1990
(Percentage of Households)

	Municipal	Non-Municipal
Chai Nat		
Tap water	76.5	9.2
Electric lighting	98.0	89.8
Sanitary toilet *	94.0	99.5
Refrigerator	-	-
Sing Buri		
Tap water	75.6	14.7
Electric lighting	98.7	97.5
Sanitary toilet	98.4	99.5
Refrigerator	64.2	45.3

Note : * Flush latrine and inovlded brecket latrine
Source : 1990 Population and Housing Census : Changwat Chai Nat

Percentage of Employed Population Aged 13 Years and Over, 1990

	Municipal	Non-Municipal	Total
Chai Nat			
1. Agri, forest fishermen, hunters	-	83.4	80.0
2. Sales	31.6	4.8	6.0
3. Craftsmen, Production worker	12.8	5.3	5.6
4. Professional	18.5	2.6	3.3
5. Services	10.7	-	-
Sing Buri			
1. Agri, forest fishermen, hunters	12.1	71.2	65.9
2. Sales	15.9	12.0	12.3
3. Craftsmen, Production worker	33.7	7.6	10.0
4. Professional	15.8	4.0	5.1
5. Services	-	-	-

Source : 1990 Population and Housing Census, Table C

Table 3.13.4.7 BWA WATER TARIFF

Category 1: Residence		Category 2: Business State Enterprise Government Agency & Others		Category 3: Industrial	
Volume (Cu.M.)	Rate (Baht/Cu.M.)	Volume (Cu.M.)	Rate (Baht/Cu.M.)	Volume (Cu.M.)	Rate (Baht/Cu.M.)
0–30	4.00 (But not less than 20 Baht)	0–10	Package Rate 50 Baht	0–10	Package Rate 50 Baht
31–40	4.25	11–20	6.20	11–20	6.20
41–50	4.50	21–30	6.45	21–30	6.45
51–60	4.75	31–40	6.70	31–40	6.70
61–70	5.00	41–50	6.95	41–50	6.95
71–80	5.25	51–60	7.20	51–60	7.20
81–90	6.15	61–80	7.45	61–80	7.45
91–100	6.40	81–100	7.70	81–100	7.70
101–120	6.65	101–120	7.95	101–120	7.95
121–160	6.90	121–160	8.20	121–160	8.20
161–200	7.15	161–200	8.45	161–200	8.45
201–over	7.65	201–over	8.70	201–2,000	8.60
				2,001–4,000	8.40
				4,001–6,000	8.00
				6,001–10,000	7.50
				10,001–20,000	7.00
				20,001–30,000	6.50
				30,001–40,000	6.00
				40,001–50,000	5.50
				50,001–over Cu.M.	5.00

Table 3.13.4.8 Provincial Waterworks Authority
Water Tariff

Consumption (cum/month)	Connection Categories			
	1. Residence	2. Government agencies State Enterprises and Others	3. Business and Industrial	
			Developed Area	The Rest
0 - 10	3.75 But not less than 15	5.00 But not less than 30	8.00 But not less than 50	8.00 But not less than 50
11 - 20	4.50	6.00	12.00	10.00
21 - 30	6.50	7.25	15.00	12.00
31 - 50	8.50	8.50	18.00	14.00
51 - 80	9.00	9.00	19.00	16.00
81 - 100	9.50	9.50	20.00	18.00
101 - 300	10.00	10.00	21.00	19.00
301 - 1000	10.25	10.25	22.00	20.00
1001 - 2000	10.50	10.50	21.00	19.00
2001 - 3000	10.75	10.75	20.00	18.00
3001 and over	11.00	11.00	19.00	17.00

Effective Date : October 1, 1992

Table 3.13.4.9 BOD Inflow and Discharge by Factories, Nontaburi, 1991

Factory Activity	Treatment Method	Wastewater m³/day	BOD (mg/liter)		BOD Load (kg/day)	
			Inflow	Discharge	Inflow	Discharge
1. Laundry	Chemical & Biological	250	350	18	84	4.5
2. Juice Maker	Chemical & Biological	400	650	3	260	1.2
3. Laundry	Chemical & Biological	200	350	62	70	1.2
4. Icecream	Chemical & Biological	100	650	22	65	2.2
5. Food	Pond	2	650	20	1.3	0.04
6. Fish Sauce	Pond	2	350	15	0.7	0.03
7. Glue	Chemical	10	250	0.3	2.5	0.003
8. Drinking Water	Pond	200	150	18	30	3.6
9. Brewery	Pond	200	350	14	17	2.8
10. Noodles	Biological	50	350	39	17	1.9
11. Spectacle Lens	Chemical	10	250	71	2.5	0.7
12. Icecream	Biological	100	650	18	65	1.8
13. Paper	Chemical & Biological	3000	650	13	1950	41
14. Chocolate	Pond	3	350	-	1.5	-
15. Dying	Chemical & Biological	1000	650	32	650	33
16. Dying	Biological	900	650	11	585	10
17. Chicken Soup	Biological	30	350	5	10	0.16
18. Vegetable Oil	Pond	-	-	-	-	-
19. Vegetable Oil	Biological	10	350	7	3.5	0.07
20. Dying	Chemical	300	450	17	135	5.3
	Total	6767			4007	121

Table 3.14.4.10 BOD Discharge by Phrannungkiao Hospital, 1992

Method	BOD Discharge (mg/liter)	Suspended Solid (mg/liter)	DO (mg/liter)
Oxidization	12.3	9	21

Table 3.14.4.11 Salmonella by Source in Isolation, Thailand, January - December 1987

Source of Specimens

Total	Human	Food	Animal	Animal Food	Water	Environment
6,734	4,217	752	537	115	132	981
100%						14%

Source : NIH Research Paper, Page 100

Table 3.14.4.12 Pattaya Municipality

Fee Rate for Wastewater and Sludge

Type	Wastewater Baht/unit	Sludge Baht/unit	Unit
1. Hotel	672.0	67.2	room
2. Appartment, Flat	360.0	36.0	room
3. Restaurant, Food shop	36.0	3.6	area of building/m^2
4. Building	6.0	0.6	-ditto-
5. House	3.6	0.36	-ditto-
6. Government Office	-	-	-

Table 3.13.4.13 Patong Sewerage Plant, 1993 (Started 1989, July)

(1) Connection Fees and User Charge

	Connection fee	User charge/year
House	100	400
House 2 stories	100	500
House 3 and above stories	200	500
Restaurant	10/m2	40/m2
Hotel	50/Room	600/Room

Connections:	Hotel	31 units	1683 room	@600 =	1,009,800
	Commercial Building	65 units	135 room	@500 =	67,500
	Restaurant	14 units		@500 =	7,000
					1,084,300

(2) Income and Expenses

Year	Income		Expenses	
1991	991,980		881,604	
		Electricity	46,400	
		Oil	7,100	monthly
		Staff	19,467	
			73,467	
		Yearly	73,467 x 12	
		=	881,604	

(3) Wastewater Treatment Expenses

Treated Wastewater	2,250 m3 x 365 days =	821,250 m3/year
Expenses		881,604/year
Unit O&M Cost of Treating Wastewater		1.073/m3

(4) Revenue Collection Efficiency

Collected Revenue	Collection Efficiency
991,980	91.48 %

Table 3.13.4.14 Monthly Revenues & Expenditures for Garbage Collection 1993

	Pathum Thani	Khu Kot	Bang Bua Thong
Revenue from garbage collection	96,500	71,530	65,000 *
Revenue from septic tank cleaning	–	–	150,000 *
Expenditures Personnel	95,000	94,200	327,000
6 Drivers (3,000 Bht.)	18,000	6 (4,000) = 24,000	3 (4,500) = 13,500
24 Helpers (3,000 Bht.)	72,000	20 (3,000) = 60,000	52 (3,000) = 156,000
1 Officers (5,000 Bht.)	5,000	1 (5,200) = 5,200	6 (6,000) = 36,000
Major source of water pollution	–	–	–
Daily Collection	60 tons/day	175 m^3/day	18 tons/day
Future Plan	more trucks	more trucks	more trucks
Finance	province	province	province

Note : * Garbage collection and septic tank fee will be increased to 300% in a few month. This will amount to Baht 645,000 a month

Table 3.13.4.15 SANITATION FEES (BAHT)

Khu Khot

Garbage Collection Fees for Household Per Month

	Baht
20 liter or less	20
20	30
40	50
60	70
80	100
100	150
200	300
300	350
400	450

Commercial and Industrial Fees Per Month

1 cubic meter	1,500
In excess of 1 cubic meter	1,500

Khu Khot

- * Number of Garbage Trucks : 4 to start operation in 1993 for the first time
- * Sludge Vacuuming : Not done
- * Septic tank maintenance : private sector

Table 3.13.4.16 SANITATION FEES (BAHT)

Bang Bua Thong

Garbage Collection Fees for Household Per Month

	Baht
20 liter or less	8
20	12
40	16
60	20
80	24
100	44
200	64
300	84
400	104

Commercial and Industrial Fees Per Month

1 cubic meter	400
In Excess of 1 Cubic Meter	400

Toilet Water Suction Fee

1st cubic meter	$60/m^3$
In Excess of 1 Cubic Meter	$40/m^3$

Prachatipat

* Number of Garbage Trucks : 6

* Sludge Vacuuming in cooperation with Patum Thani province

* Septic Tank Maintenance : private sector

Table 3.13.4.17 Sanitation Expenditures, Ang Thong, 1991-1992

	1992	1991
Total Revenues	35,278	23,005
Total Expenditures	31,659	22,794
Expenditures		
Cleaning	2,349	1,789
Road Construction & Drainage	2,655	250
Budget for land Acquisition	5,000	-
Total Revenue	35,278	23,005
Local Revenues	10,538	12,512
Income Tax	2,783	2,519
Licensing Fees	4,680	4,382
Advertising Fees	1,495	1,332
Other Revenue	1,580	4,279

Table 3.13.4.18 Property Value

(Unit : 1,000 Baht)

KHU KHOT

High Priced Area

Road Side A 20 meter	16,000
Road Side B 20 meter	12,000
Road Side C 40 meter	12,000

Low Priced Area

Road Side A 40 meter	800
Road Side B	60

BANG BUA THONG

Land Price

Central	15,000 / wah^2
Outside Central	13,000 / wah^2

ANG THONG

Property Value/rai	Tax Rate/rai
50,000	120
2,000,000	4,995
3,600,000	8,995
600,000	1,495
280,000	695
100,000	245
220,000	545

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