ANNEXE 6.3.4.2

概算事業費算出用工事単価

(1) Superstructure des ponts en acier.

- Coûts de construction et de montage pour les ponts en acier (DT par tonne).

	Construction	Montage	Total
Pont à poutre-caisson continue	2.590 (3.280)	1.240 (1.560)	3.830 (4.840)
Pont suspendu à haubans	2.860 (3.620)	1.580 (2.000)	4.440 (5.620)
Pont Nielsen	3.260 (4.120)	2.080 (2.630)	5.340 (6.750)
Pont Lohse	2.960 (3.750)	1.730 (2.190)	4.690 (5.940)
Pont mobile	4.000 (5.060)	1.400 (1.770)	5.400 (6.830)

Note :

Les valeurs supérieures indiquent les coûts totaux de construction et de montage sans ceux des travaux préparatoires, ni de construction des accessoires du pont, alors que les valeurs inférieures entre parenthèses comprennent ces derniers coûts.

(2) Superstructure des ponts en béton précontraint.

Désignation	Unité	Prix unitaire (DT)
1. Béton	m3	72,800
2. Coffrage	m2	38,700
3. Armature	Kg	0,870
4. Câble de précontrainte	Kg	5,300

2) Prix unitaire de montage :

Type de pont	Prix unitaire (DT/m2)	Méthode de montage
1. A poutre-caisson de béton précontraint	250	En porte-à-faux
2. A poutre en T de béton précontraint	1 000	Par grue
3. A tablier en dalle élégie	75	Sur échafaudage

(3)Pieu en béton coulé sur place

Diamètre de pieu	Par mètre d'autres pieux de diamètre (TD/m)
0.80 m	215
1.0 m	290
1.5 m	520
2.0 m	835

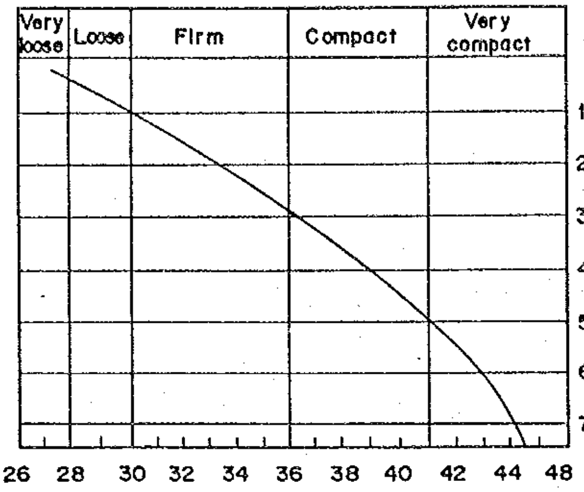
(4)Voie d'accès

Désignation	Unité	Prix unitaire (DT)
1. Remblai	m3	10,200
2. Pavement	m2	24,400
3. Protection des talus	m2	11,800
4. Pont à l'échangeur	m2	970,000
5. Amendement des sols	m2	10,500

ANNEXE 7.2.1.1

土質調査結果

Fig. 1 RELATIVE CHART FOR N-VALUE AND INTERNAL FRICTION ANGLE



INTERNAL FRICTION ANGLE
(DEGREE)

Fig. 2 RELATIVE CHART FOR LOAD AND VOID RATIO OF SANDY SOIL

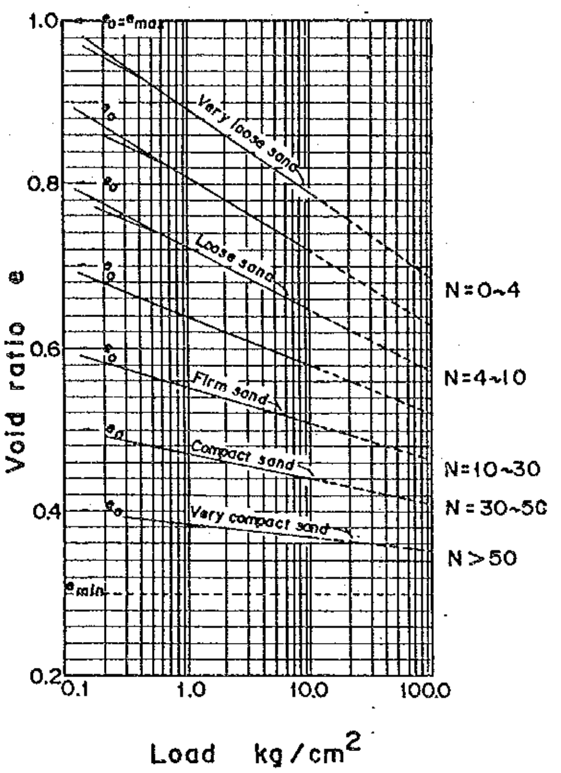
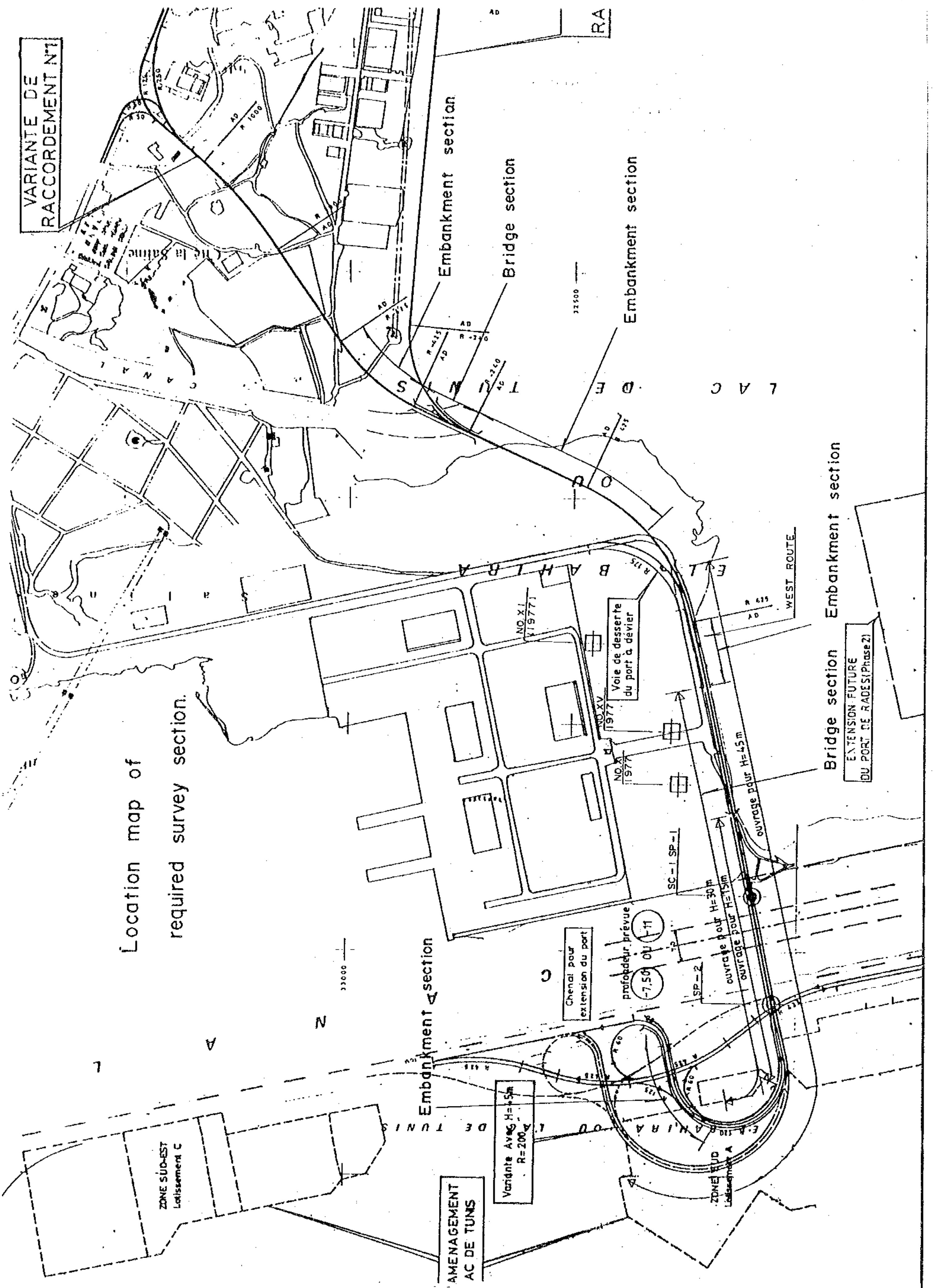


TABLE 1 WET DENSITY OF SOIL

Soil		Condition of Soil		Wet density(t/m^3)	Symbols
Embakment	Sand mixed gravel	Compact		2.0	GW, GP
	Sand	Compact	good gradation	2.0	SW, SP
			No good gradation	1.9	
	Sandy soil	Compact		1.8	SM, SC
	Cohesive soil	Compact		1.7	ML, CL, (MH, CH)
	Volcanic cohesive soil		Compact	1.4	VH
Natural ground	gravel	Compat or good gradation		2.0	GW, GP
		Loose or no good gradation		1.8	
	Sand mixed gravel	Compact		2.1	GW, GP
		Loose		1.9	
	Sand	Compact or good gradation		2.0	SW, SP
		Loose or no good gradation		1.8	
	Sandy soil	Compact		1.9	SM, SC
		Loose		1.7	
	Cohesive soil	hard		1.8	ML, CL
		soft		1.6	
	Silt	hard		1.6	ML
		Soft		1.4	
	Clay	hard		1.7	CH, MH
		Soft		1.5	
	Volcanic cohesive soil			1.4	VH



Location map of required survey section.

Fig. 3 RELATIVE CHART OF PLANING EMBANKMENT HEIGHT (HE) AND NECESSARY EMBANKMENT HEIGHT (HNE)

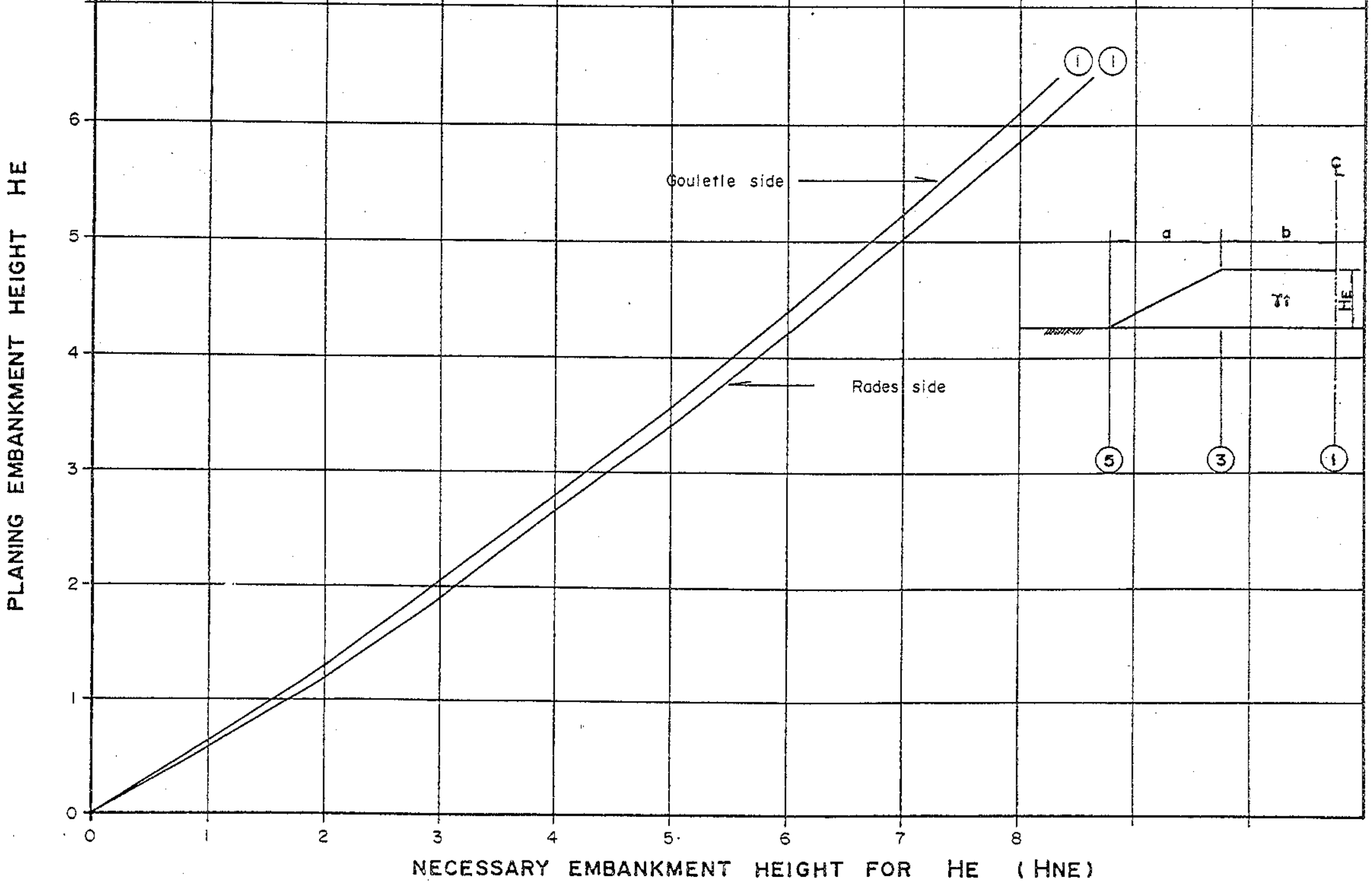


Fig. 4 RELATIVE CHART FOR INTERVAL OF SAND DRAIN AND REMAINING SUBSIDED VALUE

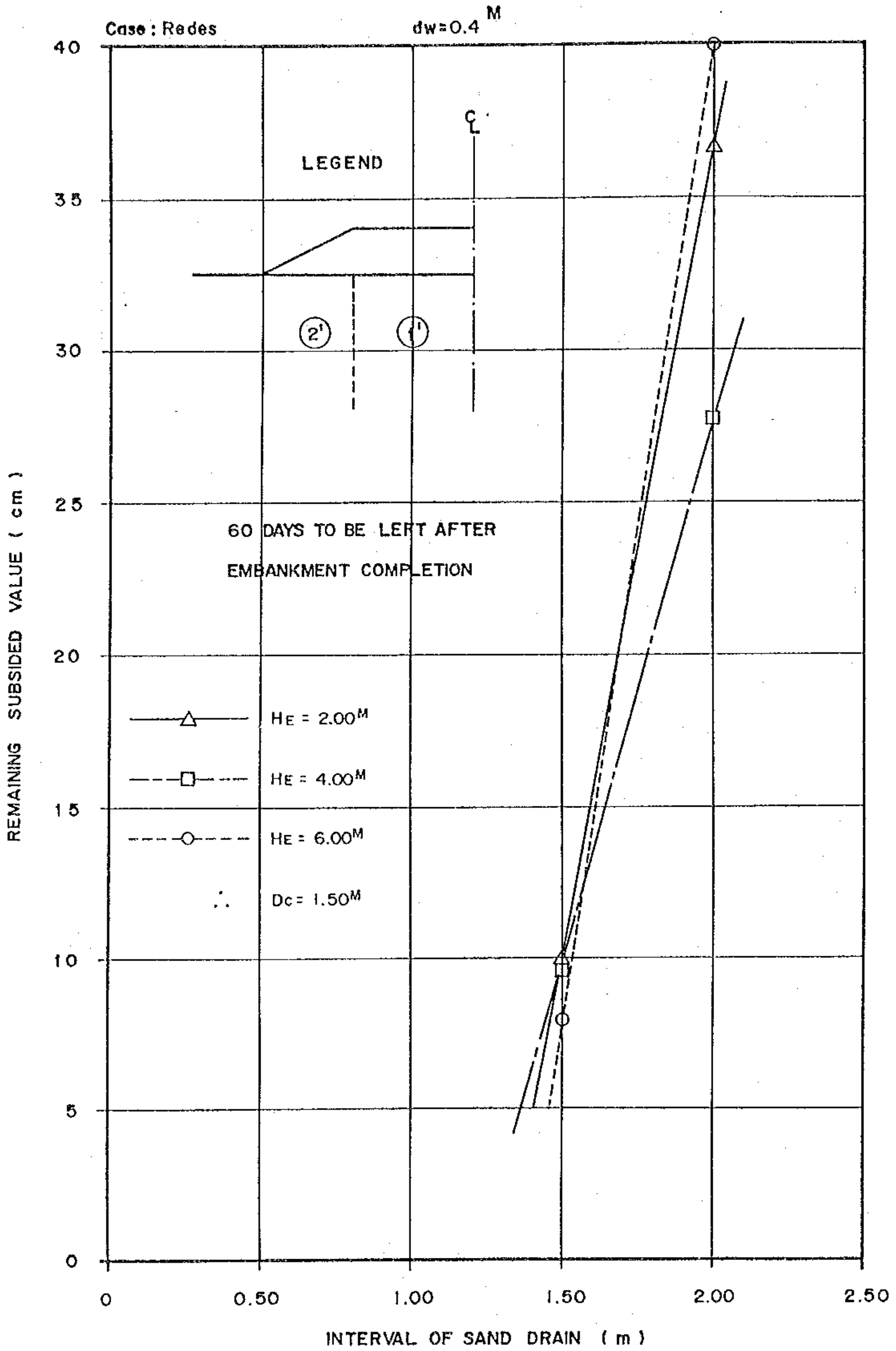


Fig. 5 RELATIVE CHART FOR INTERVAL OF SAND DRAIN AND REMAINING SUBSIDED VALUE

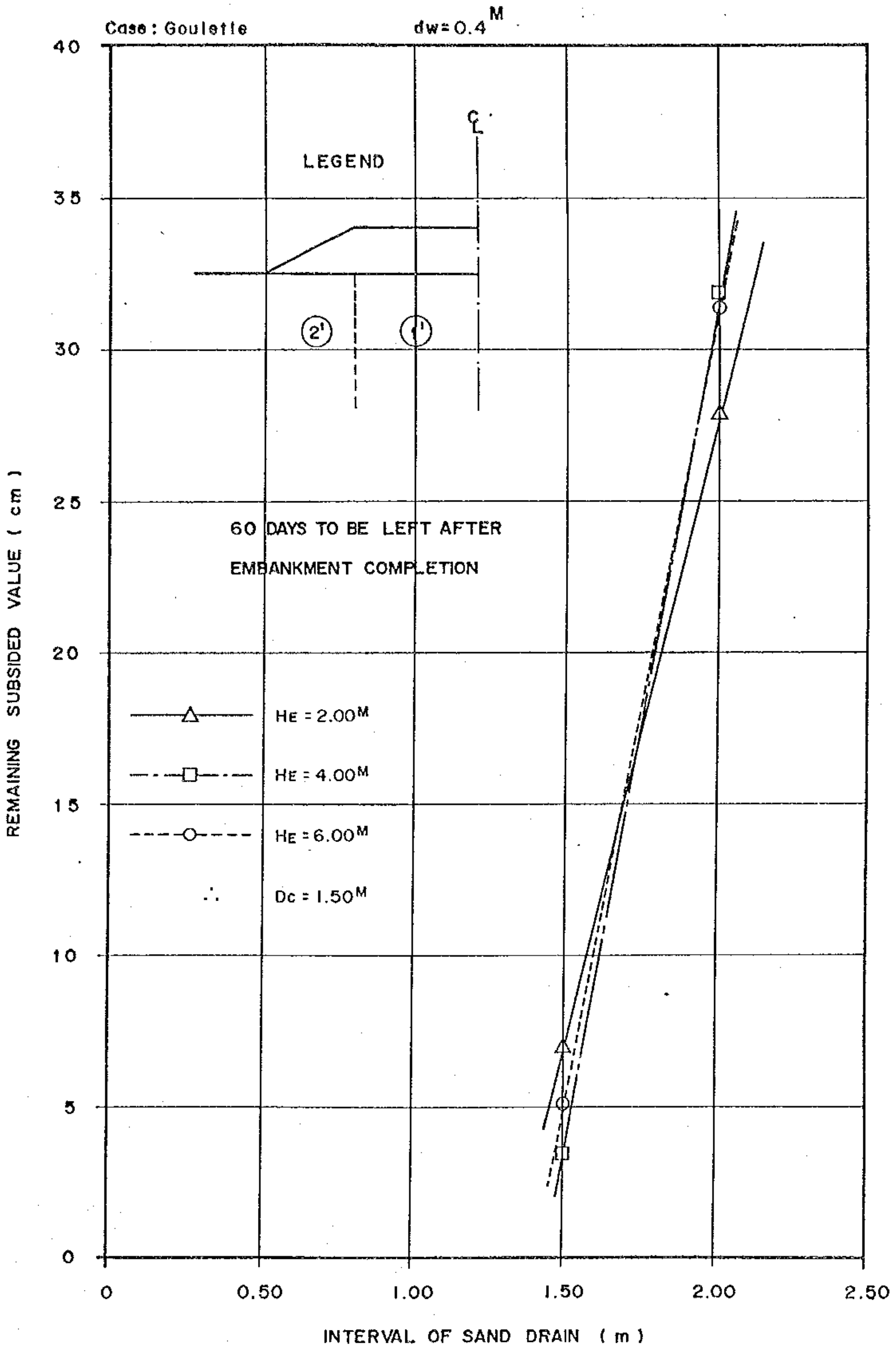


Fig. 6 RELATIVE CHART OF EMBANKMENT HEIGHT AND SAFETY FACTOR

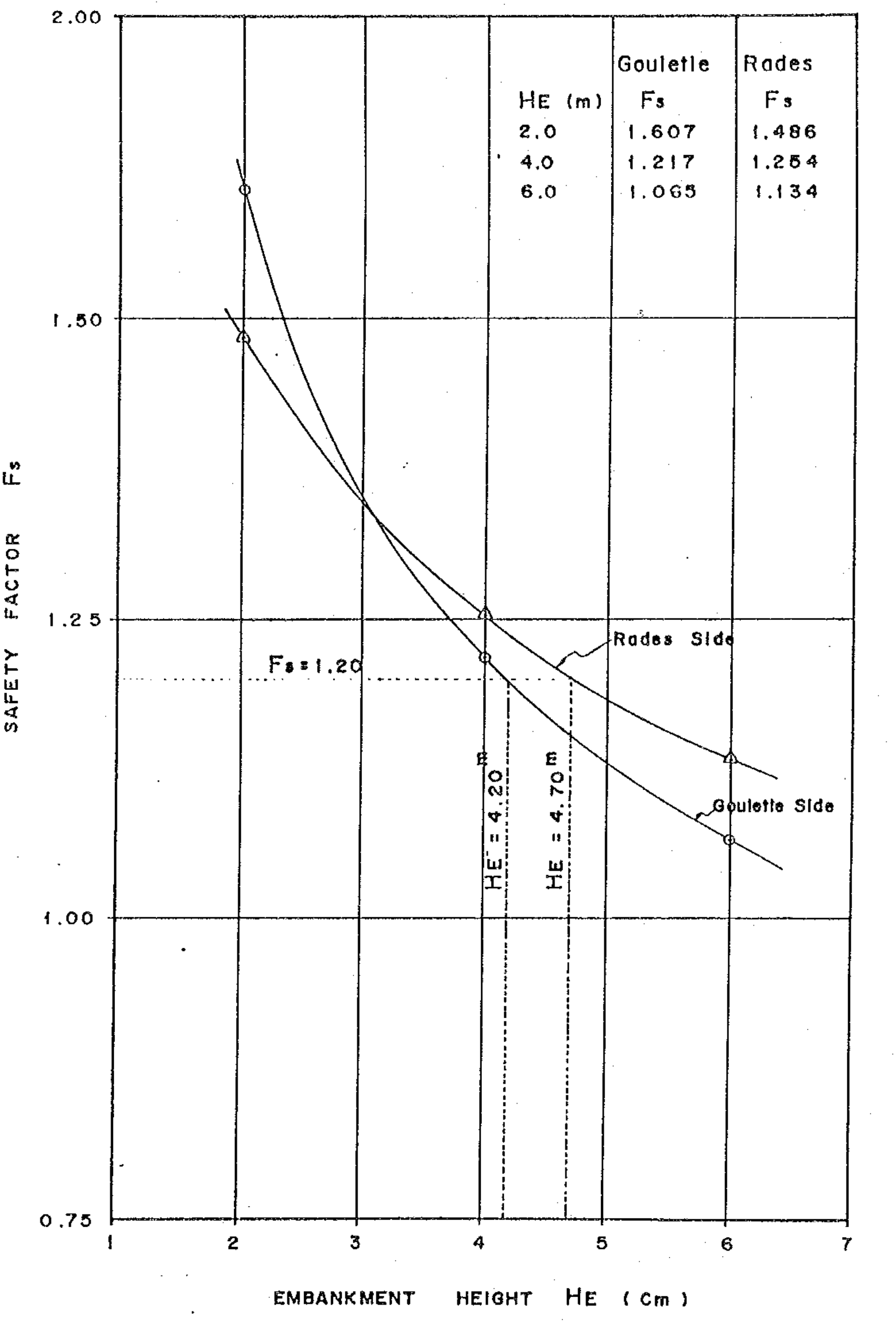


TABLE 2 WORK ITEMS AND QUANTITIES OF SOILS AND MATERIALS INVESTIGATIONS

PURPOSE OF INVESTIGATION	FIELD WORKS								LABORATORY TESTING									NOTES
	MACHINE BORING NO.	NAME OF LOCALITY	DIAMETER OF BORING (m.m)	GROUND ELEVATION (m)	DEPTH OF BORING (m)	STANDARD PENETRATION TEST	THIN WALL-ED TUBE SAMPLING	OTHER TEST	SPECIFIC GRAVITY	NATURAL WATER CONTENT	PARTICLE SIZE GRADATION TEST	LIQUID LIMIT TEST	PLASTIC LIMIT TEST & INDEX	WET DENSITY	UNCONFINED COMPRESSION TEST	TRIAXIAL COMPRESSION TEST	CONSOLIDATION TEST	
1/ 2/	SC. 1	Rades	ø =101mm	1.03	120.10	5	5	–	10	5	10	5	5	5	–	5	5	1/ Laboratory works Pertaining to the Testing of [illegible] [illegible] taken by [illegible] [illegible] study. 2/ Laboratory works Pertaining to the Testing of [illegible] [illegible] for the study of [illegible]. 3/ [illegible] for the study of [illegible]. 4/ Field CBR test [illegible] [illegible]. 5/ [illegible]
1/ 2/	SC. 2	Goulette	ø =101mm	0.64	110.30	4	5	–	9	5	9	5	5	5	–	5	5	
	Sub total				230.40	9	10	–	19	10	19	10	10	10	–	10	10	
							Vane test	Pressio-meter test										
Sounding	SP. 1	Rades	ø = 62mm	1.03	105	–	–	86										
Sounding	SP. 2	Goulette	ø = 62mm	1.62	105	–	6	76										
	Sub total				210	–	6	162										
	TOTAL		ø =101mm ø = 62mm		230.40 210.00	9	10	Pressio meter test 162 Vane test 6	19	10	19	10	10	10	–	10	10	

PURPOSE OF INVESTIGATION	NAME OF MATERIALS	FIELD WORKS					LABORATORY TESTING													NOTES
		TEST PIT NO.	NAME OF LOCALITY	GROUND ELEVATION OR SAMPLING DEPTH	FIELD CBR TEST	DENSITY OF SOIL INPLACE	SPECIFIC GRAVITY	NATURAL WATER CONTENT	PARTICLE SIZE GRADATION TEST	LIQUID LIMIT TEST	PLASTIC LIMIT TEST & INDEX	COMPACTION TEST	CBR TEST	UNCONFIND COMPRESSION TEST	TRIAXIAL COMPRESSION TEST	APPARENT SPECIFIC GRAVITY	ABSORPTION	GRAIN SIZE ANALYSIS	ABRASION RATIO	
5/	Embank material	1	Di Ressas	0.5 - 1.0	–	–	1	1	1	–	–	1	1	–	–	–	–	–	–	
5/	Embank material	2	Corbous	0.5 - 1.0	–	–	1	1	1	–	–	1	1	–	–	–	–	–	–	
	Sub Total				–	–	2	2	2	–	–	2	2	–	–	–	–	–	–	
	TOTAL		Location 2		–	–	2	2	2	–	–	2	2	–	–	–	–	–	–	

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT	Feasibility study on Crossing Construction between Rades and la Goulette			SOILS & MATERIALS ENGINEER S . TAKADA			
SAMPLING LOCATION			Sc−2	Sc−2	Sc−2	Sc−2	Sc−2
SAMPLE NO.			1	2	3	4	5
SAMPLING DEPTH		(m)	12.50~13.00	33.70~34.20	49.00~49.50	70.00~70.50	104.50~105.00
GRADATION	GRAVEL	(%)	0	0	0	0	0
	SAND	(%)	5	5	4	0	5
	SILT−CLAY	(%)	40 55	25 70	21 75	20 80	18 77
	CLASSIFIED GRADING PASS NO. 10 (2.00mm)	(%)	100	100	100	100	100
	NO. 40 (0.425mm)	(%)	99	98	97	100	97
	NO. 200 (0.002mm)	(%)	95 41	95 63	96 63	100 62	95 65
LIQUID LIMIT		LL (%)	81	60	65	67	72
PLASTICITY INDEX		PI	47	33	36	37	41
CLASSIFICATION			CL	CH	CH	CH	CH
SPECIFIC GRAVITY		Gs	2,655	2,745	2,742	2,738	2,745
NATURAL STATE	WATER CONTENT	ω_n (%)	67	36	39	34	34
	WET DENSITY	γ_t (g/cm^3)	1.58	1.87	1.83	1.88	1.88
	VOID RATIO	e	1,990	1,140	1,173	0.990	1,030
	DEGREE OF SATURATION	Sr (%)					
UNCONFINED COMPRESSION	COMPRESSIVE STRENGTH	q_u (kg/cm^2)	—	—	—	—	—
	MODULUS OF ELASTICITY	E_{50}(kg/cm^2)	—	—	—	—	—
	SENSITIVITY RATIO	St	—	—	—	—	—
TRIAXIAL COMPRESSION	TYPE OF TEST		Cu	Cu	uu	uu	Ou
	COHESION	C (kg/cm^2)	0.075	0.115	0.350	0.860	0.60
	ANGLE OF INTERNAL FRICTION	ϕ°	18	20	3	2	15
CONSOLIDATION	YIELD STRESS OF CONSOLIDATION	P_Y (kg/cm^2)					
	COMPRESSION INDEX	C_c	0.56	0.257	0.339	0.279	0.249
		C_v (cm^2/ sec)	1.3×10^{-4}	2.7×10^{-4}	2.7×10^{-4}	4.7×10^{-4}	3.1×10^{-4}
COMPACTION	METHOD OF TEST						
	OPTIMUM MOISTURE CONTENT	ω opt.(%)					
	MAXIMUM DRY DENSITY	γ dmax. (g/cm^3)					
CBR	SAMPLE CONDITION	(%)					
	TEST CONDITION						
	WATER CONTENT	ω (%)					
	DRY DENSITY	γ_d (g/cm^3)					
	CBR	(%)					
REMARKS							

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT	SOILS & MATERIALS ENGINEER
Feasibility study on Crossing Construction between Rades and la Goulette	S . TAKADA

SAMPLING LOCATION			Sc—1	Sc—1	Sc—1	Sc—1	Sc—1
SAMPLE NO.			1	2	3	4	5
SAMPLING DEPTH (m)			7.20~7.70	21.00~21.50	34.75~35.25	42.45~42.95	50.80~51.30
GRADATION	GRAVEL	(%)	0	0	0	0	0
	SAND	(%)	31	1	3	33	1
	SILT—CLAY	(%)	39 30	31 68	57 40	32 35	19 80
	CLASSIFIED GRADING PASS NO. 10	(2.00mm) (%)	100	100	100	100	100
	NO. 40	(0.425mm) (%)	88	100	100	99	100
	NO. 200	(0.002mm) (%)	69 21	99 52	97 25	67 27	99 64
LIQUID LIMIT		LL (%)	74	55	31	26	62
PLASTICITY INDEX		PI	43	29	13	9	34
CLASSIFICATION			CH	CH	CL	CL	CH
SPECIFIC GRAVITY		G_s	2,672	2,671	2,709	2,692	2,693
NATURAL STATE	WATER CONTENT	ωn (%)	83	39	25	18	33
	WET DENSITY	γt (g/cm^3)	1.48	1.85	2.04	2.17	1.87
	VOID RATIO	e	2,880	0.958	0.702	0.610	0.999
	DEGREE OF SATURATION	S_r (%)					
UNCONFINED COMPRESSION	COMPRESSIVE STRENGTH	q_u (kg/cm^2)	—	—	—	—	—
	MODULUS OF ELASTICITY	E_{50}(kg/cm^2)	—	—	—	—	—
	SENSITIVITY RATIO	St	—	—	—	—	—
TRIAXIAL COMPRESSION	TYPE OF TEST		Cu	Cu	uu	uu	uu
	COHESION	C (kg/cm^2)	0.125	0.200	0.360	0.800	0.320
	ANGLE OF INTERNAL FRICTION	$\phi°$	19	15	2	6	5
CONSOLI-DATION	YIELD STRESS OF CONSOLIDATION	P_Y (kg/cm^2)	0.26	1.27	1.37	1.80	1.65
	COMPRESSION INDEX	C_c	0.77	0.30	0.171	0.112	0.302
		Cv (cm^2/sec)	1×10^{-3}	5.6×10^{-4}	8.1×10^{-4}	33×10^{-4}	2.9×10^{-4}
COMPACTION	METHOD OF TEST						
	OPTIMUM MOISTURE CONTENT	ω opt.(%)					
	MAXIMUM DRY DENSITY	γ dmax. (g/cm^3)					
C B R	SAMPLE CONDITION	(%)					
	TEST CONDITION						
	WATER CONTENT	ω (%)					
	DRY DENSITY	γd (g/cm^3)					
	C B R	(%)					

REMARKS

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT	Feasibility study on Crossing Construction between Rades and la Goulette			SOILS & MATERIALS ENGINEER S . TAKADA				
SAMPLING LOCATION				Sc—1	Sc—1	Sc—1	Sc—1	Sc—1
SAMPLE NO.				SPT.1	SPT.2	SPT.3	SPT.4	SPT.5
SAMPLING DEPTH			(m)	11.50~11.95	14.00~14.45	25.50~25.95	28.20~28.65	31.30~31.75
GRADATION	GRAVEL		(%)	0	0	0	0	0
	SAND		(%)	59	22	77	84	75
	SILT–CLAY		(%)	41	72	23	16	25
	CLASSIFIED GRADING PASS	NO. 10 (2.00mm)	(%)	100	100	100	100	100
		NO. 40 (0.425mm)	(%)	99	100	84	92	95
		NO. 200 (0.075mm)	(%)	41	72	23	16	25
LIQUID LIMIT			LL (%)					
PLASTICITY INDEX			PI					
CLASSIFICATION				SM	ML	SM	SM	SM
SPECIFIC GRAVITY			Gs	2,711	2,717	2,694	2,686	2,703
NATURAL STATE	WATER CONTENT		ω_n (%)					
	WET DENSITY		γ_t (g/cm^3)					
	VOID RATIO		e					
	DEGREE OF SATURATION		S_r (%)					
UNCONFINED COMPRESSION	COMPRESSIVE STRENGTH		q_u (kg/cm^2)					
	MODULUS OF ELASTICITY		E_{50}(kg/cm^2)					
	SENSITIVITY RATIO		S_t					
TRIAXIAL COMPRESSION	TYPE OF TEST							
	COHESION		C (kg/cm^2)					
	ANGLE OF INTERNAL FRICTION		ϕ°					
CONSOLI-DATION	YIELD STRESS OF CONSOLIDATION		P_Y (kg/cm^2)					
	COMPRESSION INDEX		C_c					
COMPACTION	METHOD OF TEST							
	OPTIMUM MOISTURE CONTENT		ω opt.(%)					
	MAXIMUM DRY DENSITY		γdmax. (g/cm^3)					
C B R	SAMPLE CONDITION		(%)					
	TEST CONDITION							
	WATER CONTENT		ω (%)					
	DRY DENSITY		γ_d (g/cm^3)					
	C B R		(%)					
REMARKS								

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT Feasibility study on Crossing Construction between Rades and la Goulette	SOILS & MATERIALS ENGINEER . S . TAKADA

SAMPLING LOCATION				Sc−2	Sc−2	Sc−2	Sc−2	
SAMPLE NO.				SPT.1	SPT.2	SPT.3	SPT.4	
SAMPLING DEPTH			(m)	2.70~3.15	25.50~25.95	28.80~29.25	31.50~31.95	
GRADATION	GRAVEL		(%)	0	0	0	0	
	SAND		(%)	79	76	73	41	
	SILT−CLAY		(%)	21	24	27	59	
	CLASSIFIED GRADING PASS	NO. 10 (2.00mm)	(%)	100	100	100		
		NO. 40 (0.425mm)	(%)	91	99	92	100	
		NO. 200 (0.075mm)	(%)	21	24	27	59	
LIQUID LIMIT			LL (%)					
PLASTICITY INDEX			PI					
CLASSIFICATION				SM	SM	SM	ML	
SPECIFIC GRAVITY			Gs	2,661	2,679	2,675	2,699	
NATURAL STATE	WATER CONTENT		ωn (%)					
	WET DENSITY		γt (g/cm^3)					
	VOID RATIO		e					
	DEGREE OF SATURATION		Sr (%)					
UNCONFINED COMPRESSION	COMPRESSIVE STRENGTH		q_u (kg/cm^2)					
	MODULUS OF ELASTICITY		E_{50}(kg/cm^2)					
	SENSITIVITY RATIO		St					
TRIAXIAL COMPRESSION	TYPE OF TEST							
	COHESION		C (kg/cm^2)					
	ANGLE OF INTERNAL FRICTION		ϕ °					
CONSOLIDATION	YIELD STRESS OF CONSOLIDATION		P_Y (kg/cm^2)					
	COMPRESSION INDEX		C_c					
COMPACTION	METHOD OF TEST							
	OPTIMUM MOISTURE CONTENT		ω opt.(%)					
	MAXIMUM DRY DENSITY		γ dmax. (g/cm^3)					
C B R	SAMPLE CONDITION		(%)					
	TEST CONDITION							
	WATER CONTENT		ω (%)					
	DRY DENSITY		γd (g/cm^3)					
	C B R		(%)					

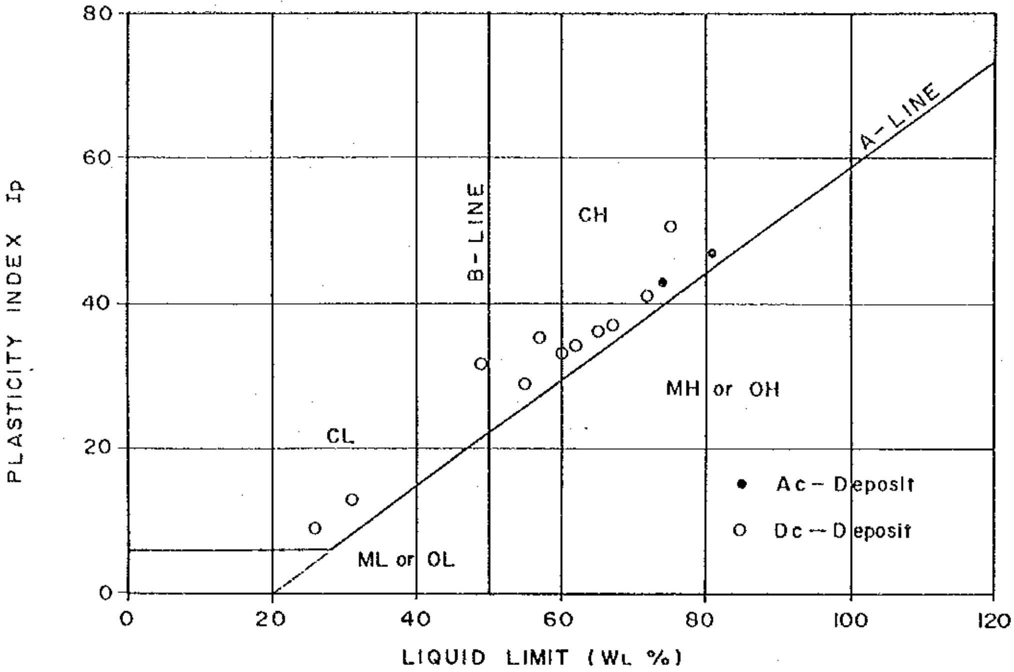
REMARKS

SUMMARY OF LABORATORY TEST OF SOILS

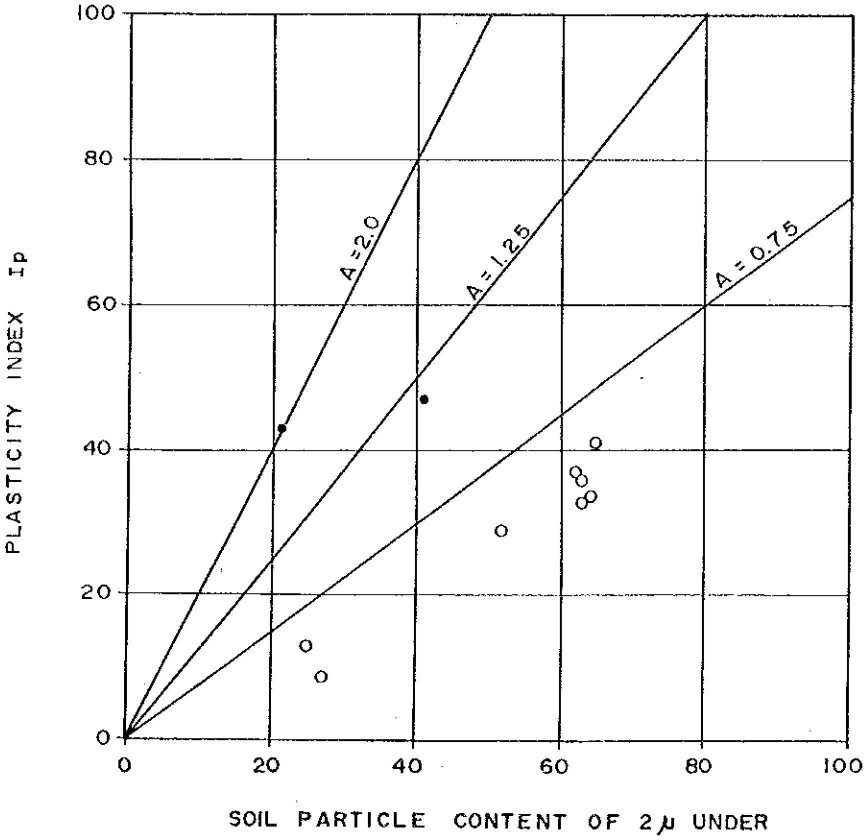
PROJECT Feasibility study on Crossing Construction between Rades and la Goulette	SOILS & MATERIALS ENGINEER S. TAKADA

SAMPLING LOCATION				Korbous	di Ressas			
SAMPLE NO.				1	1			
SAMPLING DEPTH			(m)	0.5	0.5			
GRADATION	GRAVEL		(%)	21	38			
	SAND		(%)	59	51			
	SILT--CLAY		(%)	20	11			
	CLASSIFIED GRADING PASS	NO. 10 (2.00mm)	(%)	79	62			
		NO. 40 (0.425mm)	(%)	58	32			
		NO. 200 (0.075mm)	(%)	20	11			
LIQUID LIMIT			LL (%)					
PLASTICITY INDEX			PI					
CLASSIFICATION				SM	SM			
SPECIFIC GRAVITY			Gs	2,658	2,704			
NATURAL STATE	WATER CONTENT		ωn (%)	4.0	3.0			
	WET DENSITY		γt (g/cm^3)					
	VOID RATIO		e					
	DEGREE OF SATURATION		S_r (%)					
UNCONFINED COMPRESSION	COMPRESSIVE STRENGTH		q_u (kg/cm^2)					
	MODULUS OF ELASTICITY		E_{50}(kg/cm^2)					
	SENSITIVITY RATIO		St					
TRIAXIAL COMPRESSION	TYPE OF TEST							
	COHESION		C (kg/cm^2)					
	ANGLE OF INTERNAL FRICTION		$\phi°$					
CONSOLI-DATION	YIELD STRESS OF CONSOLIDATION		P_Y (kg/cm^2)					
	COMPRESSION INDEX		C_c					
COMPACTION	METHOD OF TEST							
	OPTIMUM MOISTURE CONTENT		ω opt.(%)	10	8			
	MAXIMUM DRY DENSITY		γ dmax. (g/cm^3)	1.87	2.06			
CBR	SAMPLE CONDITION		(%)	Disturbed	Disturbed			
	TEST CONDITION			Soaked	Soaked			
	WATER CONTENT		ω (%)	16.0	8.0			
	DRY DENSITY		γd (g/cm^3)	1,777	1,957			
	CBR		(%)	14	10			

REMARKS



PLASTICITY CHART



COLLOIDIAL ACTIVITY

Fig. 7

CONSISTENCY CHART

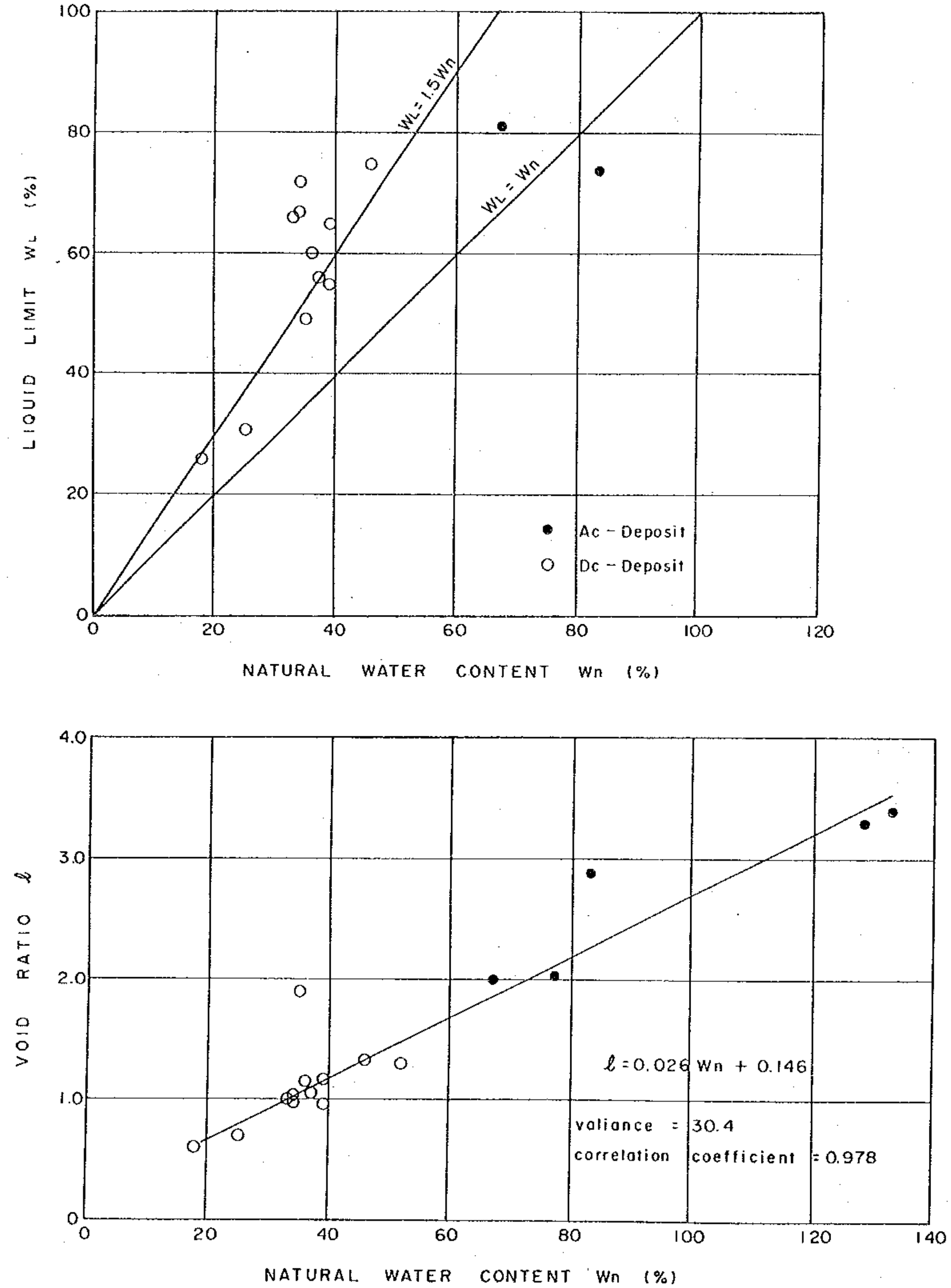


Fig. 8

RELATIVE CHART OF Wn AND WL, e

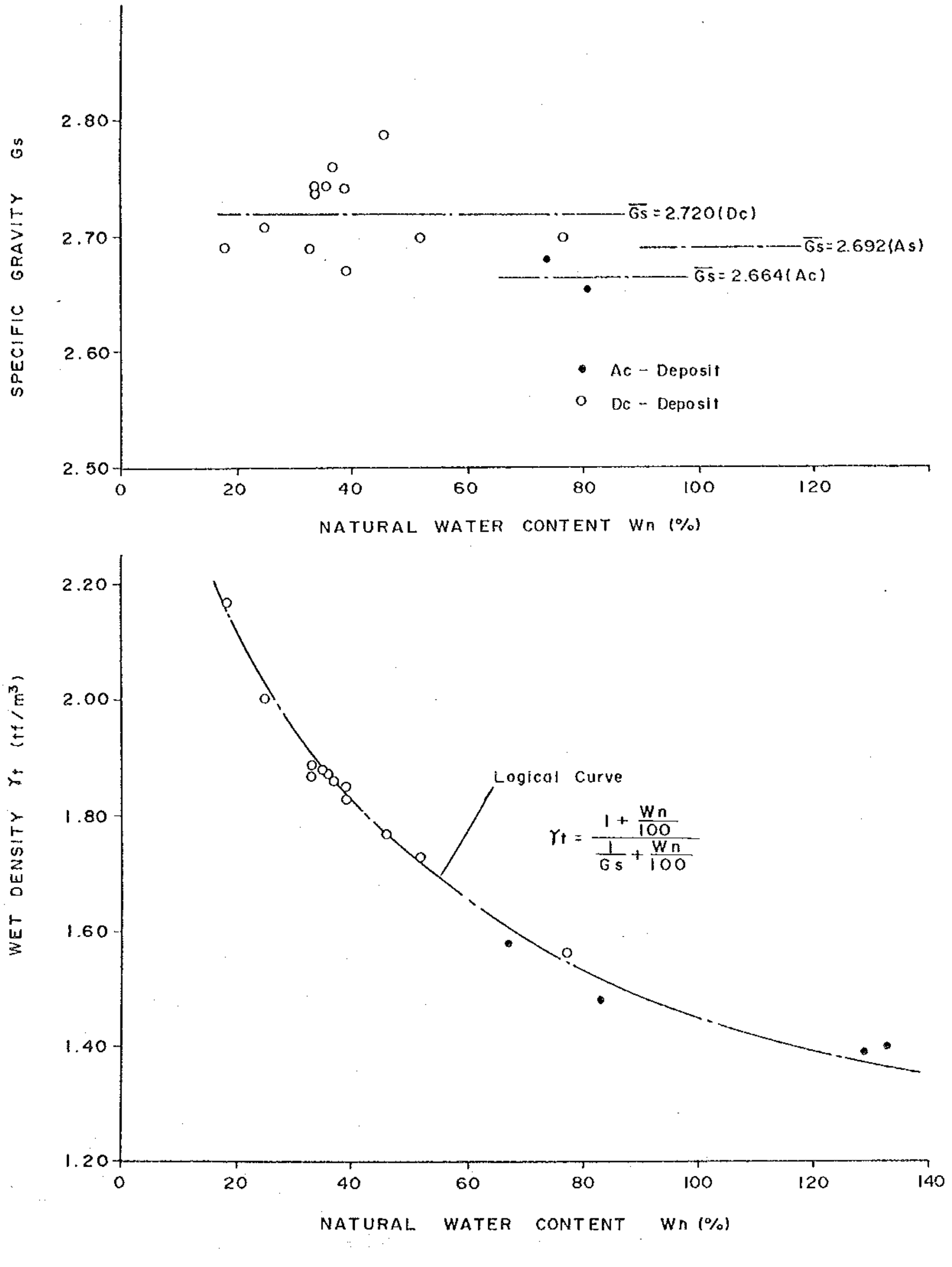


Fig. 9	RELATIVE CHART OF Wn AND Gs, γt

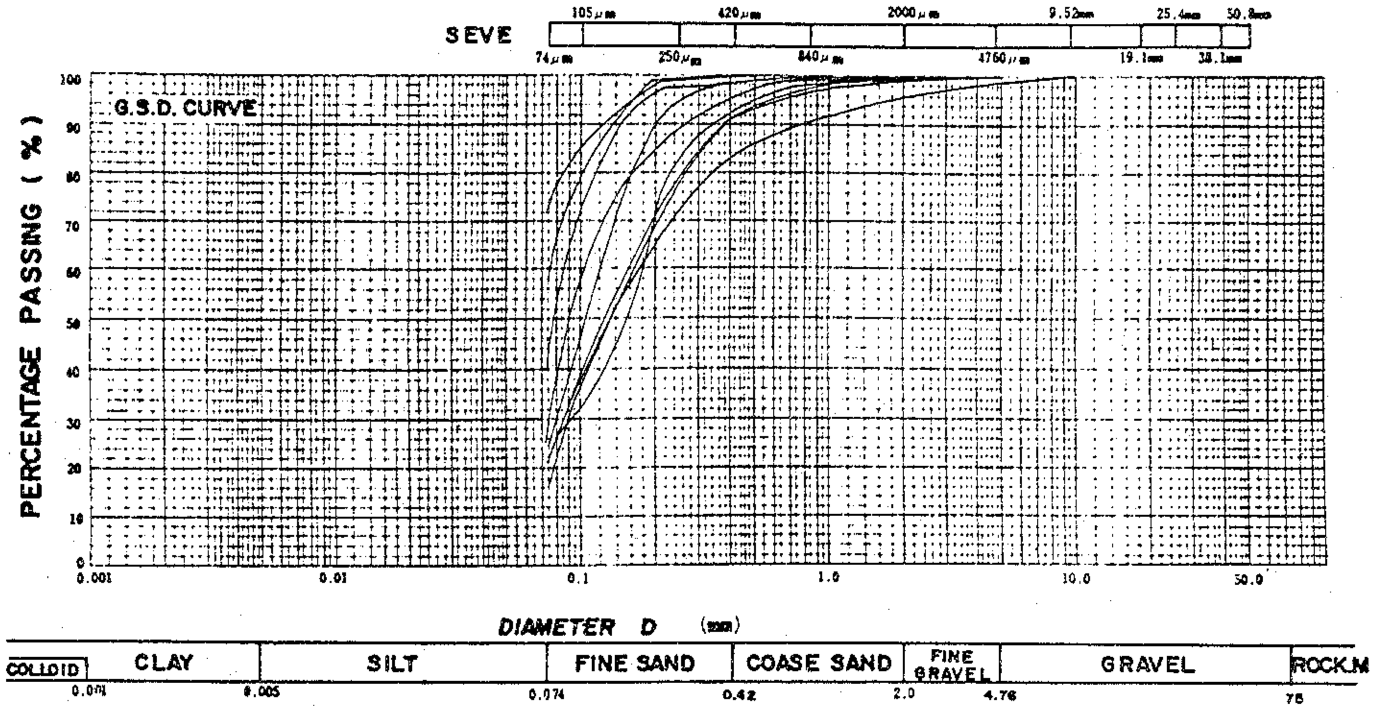


Fig. 10 GRAIN SIZE DISTRIBUTION CURVE (As-deposit)

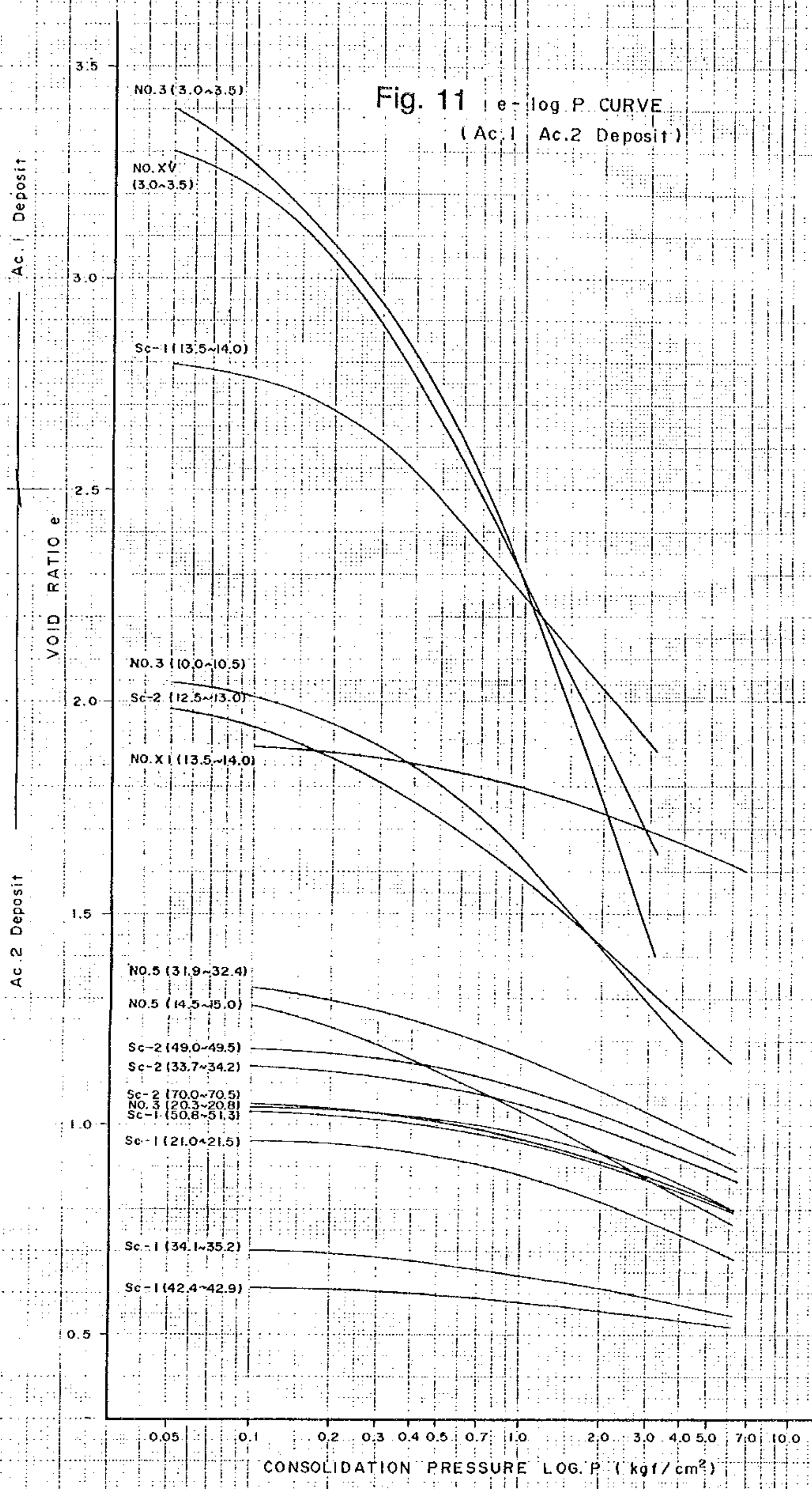


Fig. 11 e-log P CURVE (Ac.1 Ac.2 Deposit)

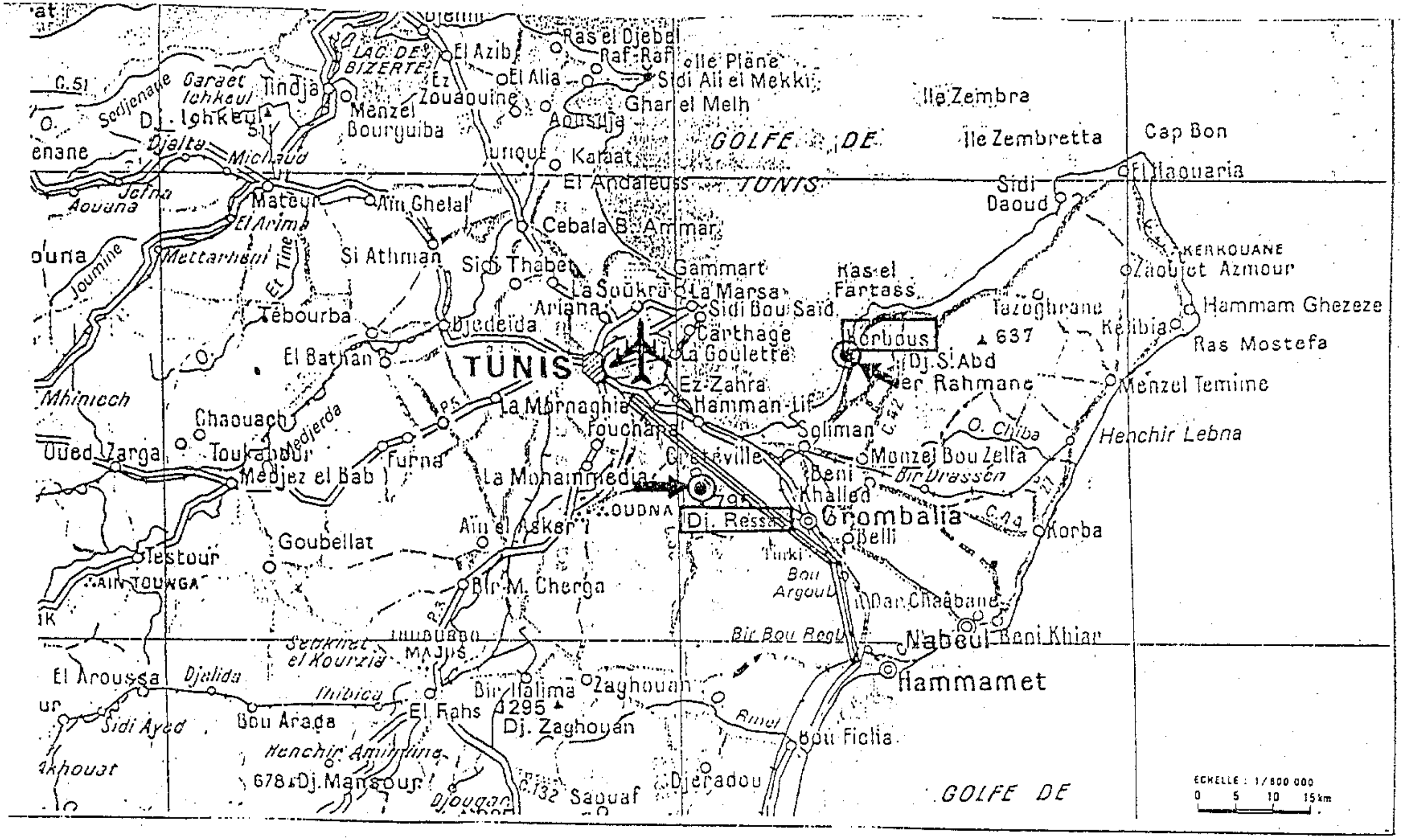


Fig. 12 LOCATION MAP (Embankment Material)

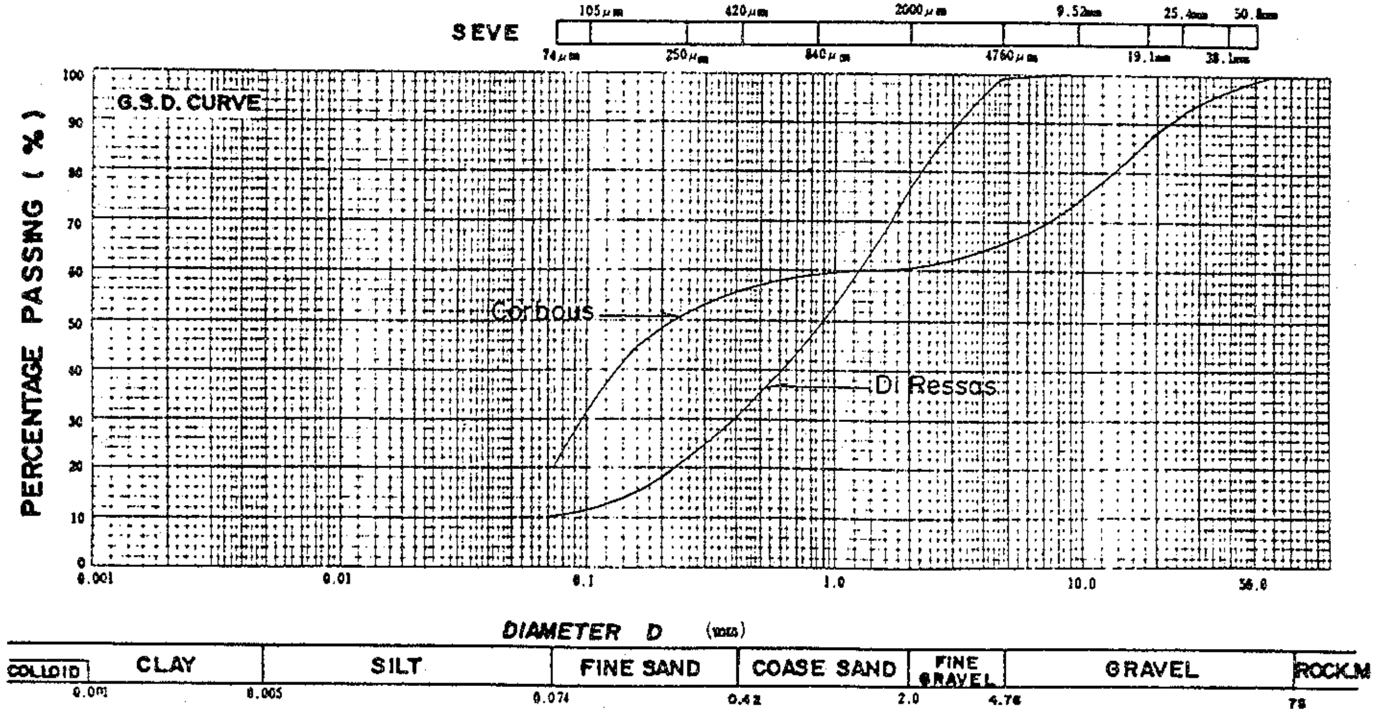
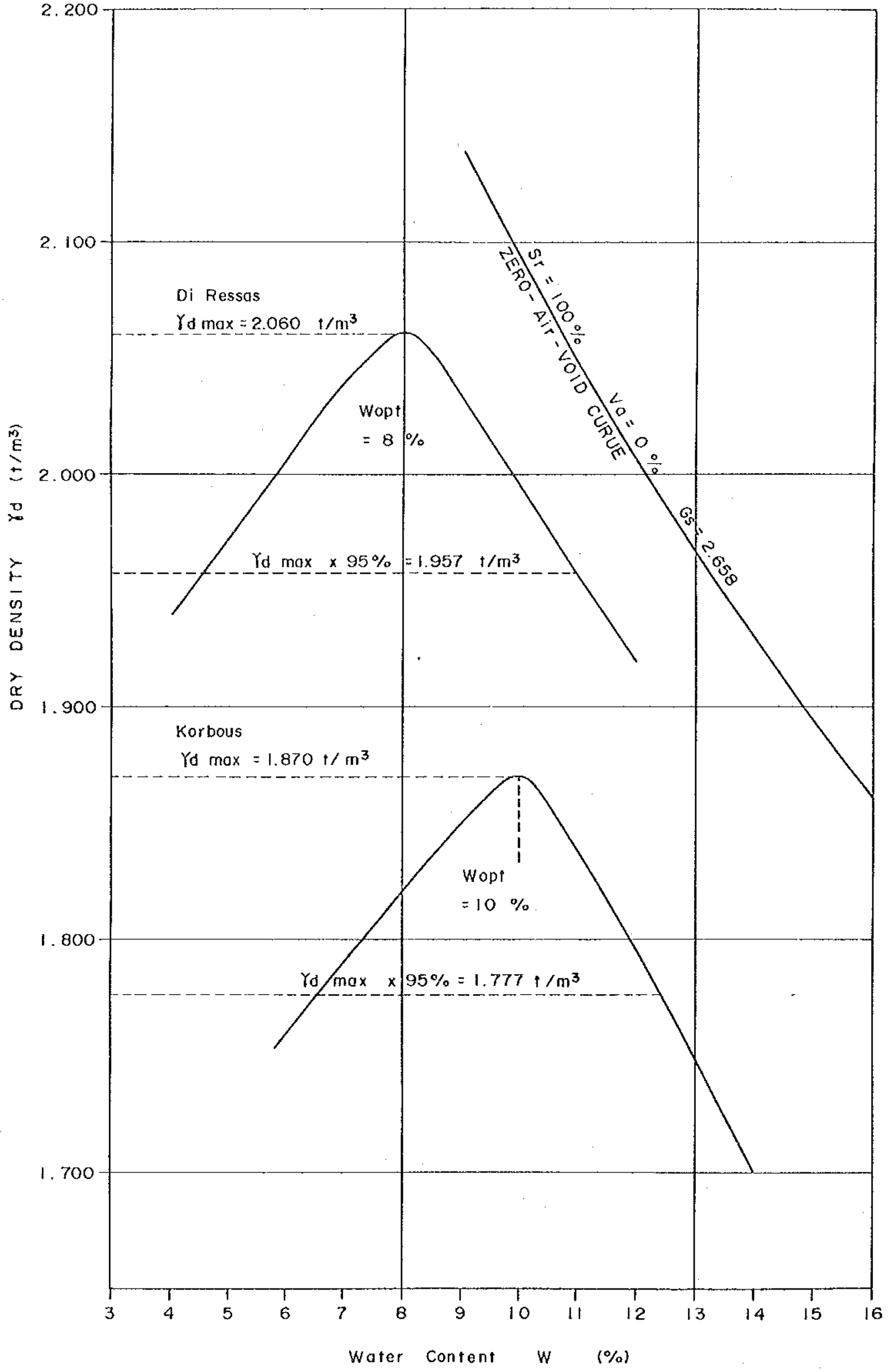
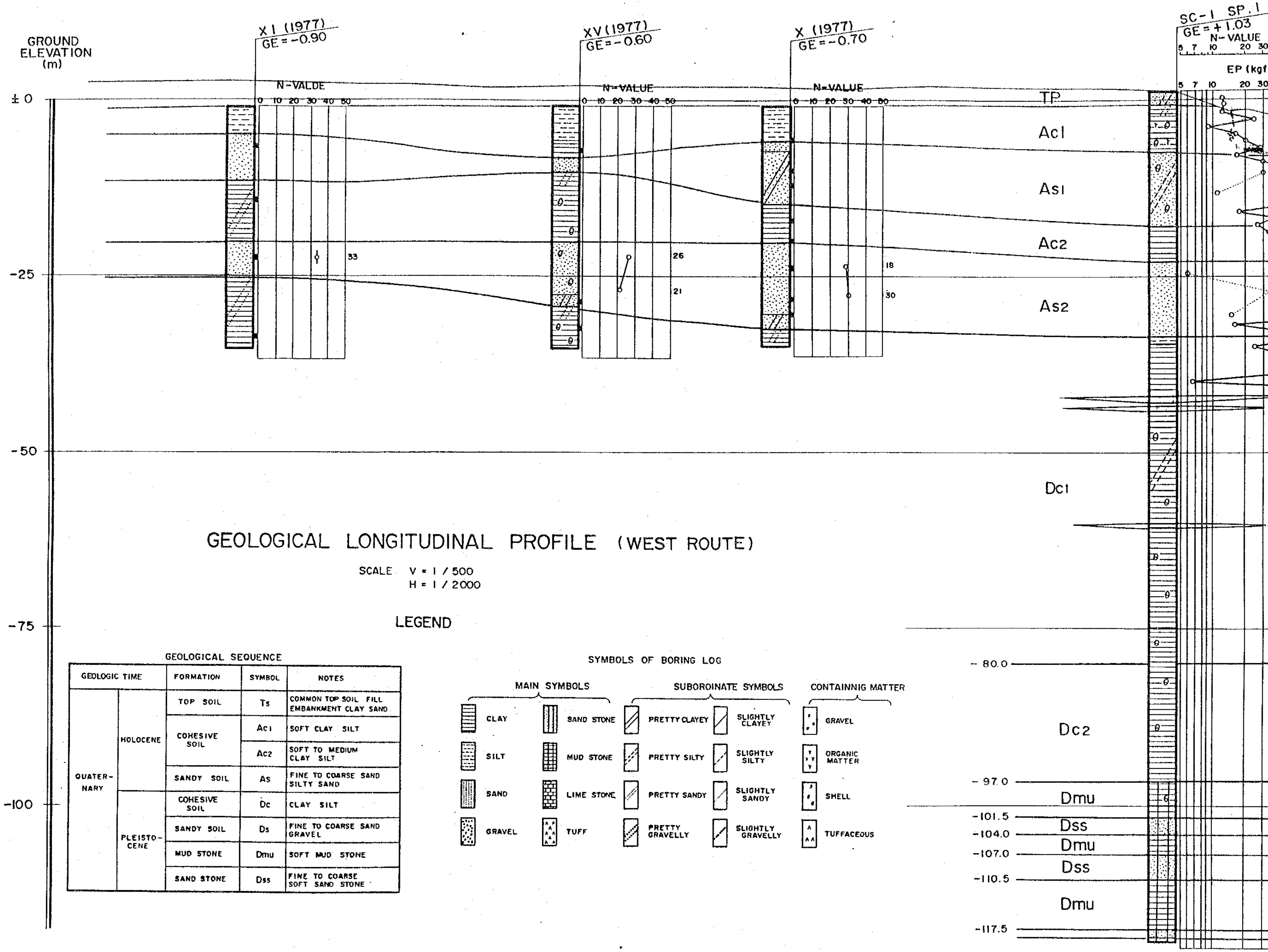


Fig. 13 GRAIN SIZE DISTRIBUTION CURVE (Embankment Material)

Fig. 14 COMPACTION CURVE FOR EMBANKMENT MATERIALS



Geological longitudinal profile



GEOLOGICAL LONGITUDINAL PROFILE (WEST ROUTE)

SCALE V = 1 / 500
H = 1 / 2000

LEGEND

GEOLOGICAL SEQUENCE

GEOLOGIC TIME		FORMATION	SYMBOL	NOTES
QUATERNARY	HOLOCENE	TOP SOIL	Ts	COMMON TOP SOIL FILL EMBANKMENT CLAY SAND
		COHESIVE SOIL	Ac1	SOFT CLAY SILT
			Ac2	SOFT TO MEDIUM CLAY SILT
		SANDY SOIL	As	FINE TO COARSE SAND SILTY SAND
	PLEISTOCENE	COHESIVE SOIL	Dc	CLAY SILT
		SANDY SOIL	Ds	FINE TO COARSE SAND GRAVEL
		MUD STONE	Dmu	SOFT MUD STONE
		SAND STONE	Dss	FINE TO COARSE SOFT SAND STONE

SYMBOLS OF BORING LOG

MAIN SYMBOLS

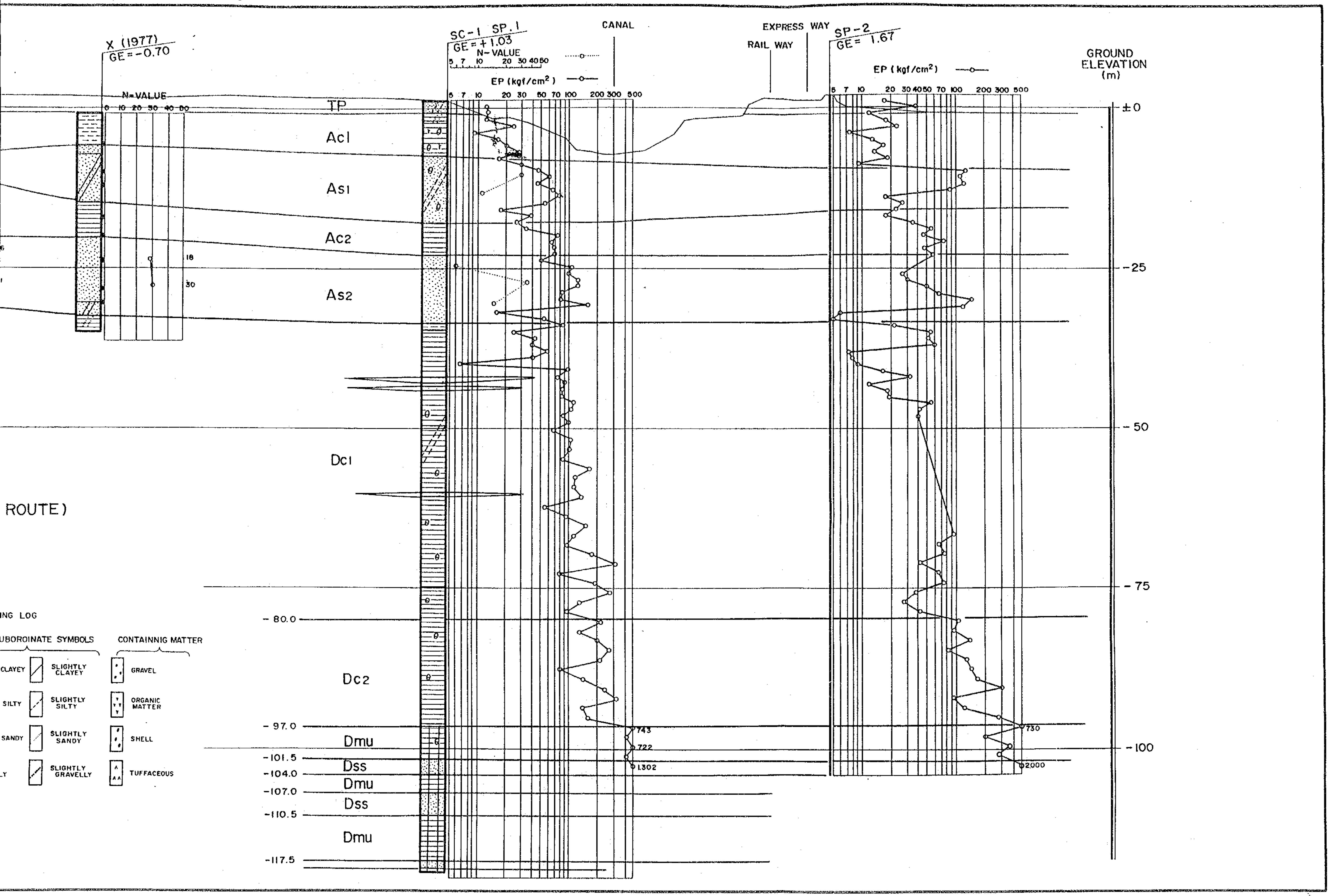
- CLAY
- SILT
- SAND
- GRAVEL
- SAND STONE
- MUD STONE
- LIME STONE
- TUFF

SUBOROINATE SYMBOLS

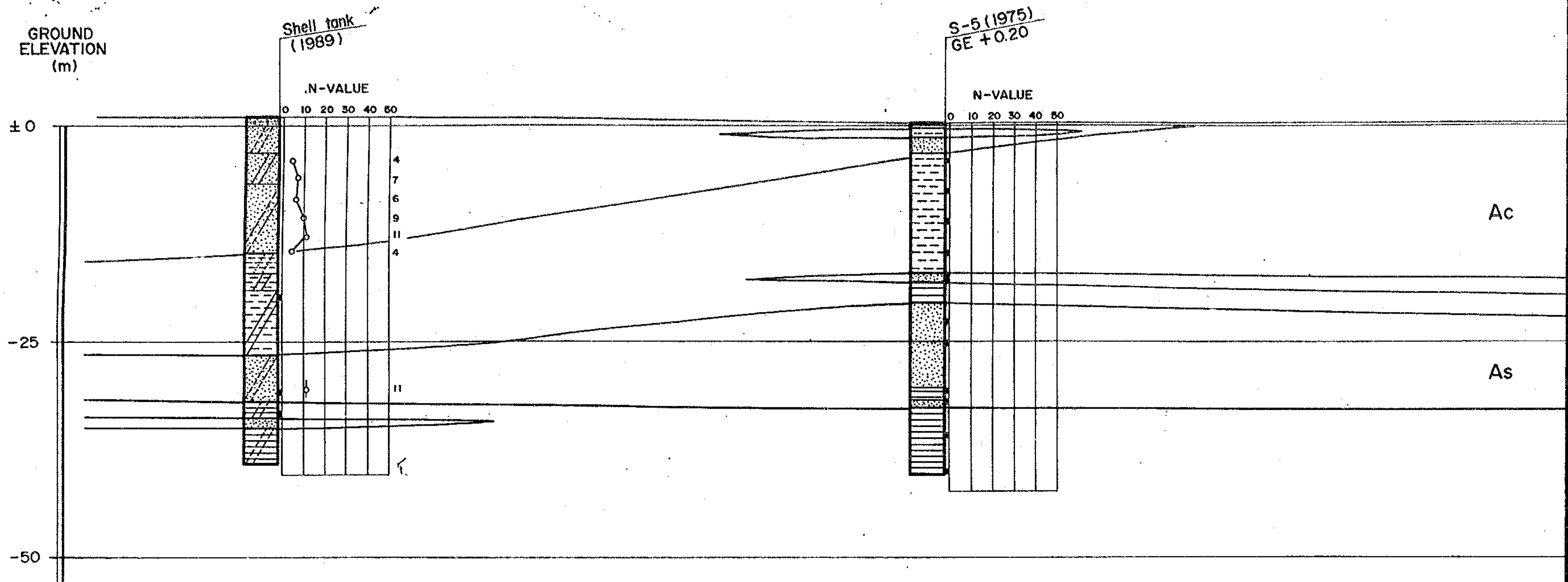
- PRETTY CLAYEY
- PRETTY SILTY
- PRETTY SANDY
- PRETTY GRAVELLY
- SLIGHTLY CLAYEY
- SLIGHTLY SILTY
- SLIGHTLY SANDY
- SLIGHTLY GRAVELLY

CONTAINNIG MATTER

- GRAVEL
- ORGANIC MATTER
- SHELL
- TUFFACEOUS



X (1977)
GE=-0.70
N-VALUE
0 10 20 30 40 50
18
30
SC-1 SP.1
GE=+1.03
N-VALUE
5 7 10 20 30 40 50
EP (kgf/cm²)
5 7 10 20 30 50 70 100 200 300 500
CANAL
EXPRESS WAY
RAIL WAY
SP-2
GE= 1.67
EP (kgf/cm²)
5 7 10 20 30 40 50 70 100 200 300 500
GROUND
ELEVATION
(m)
±0
-25
-50
-75
-100
TP
Ac1
As1
Ac2
As2
Dc1
Dc2
Dmu
Dss
Dmu
Dss
Dmu
- 80.0
- 97.0
-101.5
-104.0
-107.0
-110.5
-117.5
743
722
1302
730
2000
ROUTE)
ING LOG
UBOROINATE SYMBOLS
CONTAINNIG MATTER
CLAYEY
SLIGHTLY CLAYEY
GRAVEL
SILTY
SLIGHTLY SILTY
ORGANIC MATTER
SANDY
SLIGHTLY SANDY
SHELL
LY
SLIGHTLY GRAVELLY
TUFFACEOUS



GEOLOGICAL LONGITUDINAL PROFILE (CENTRAL ROUTE)

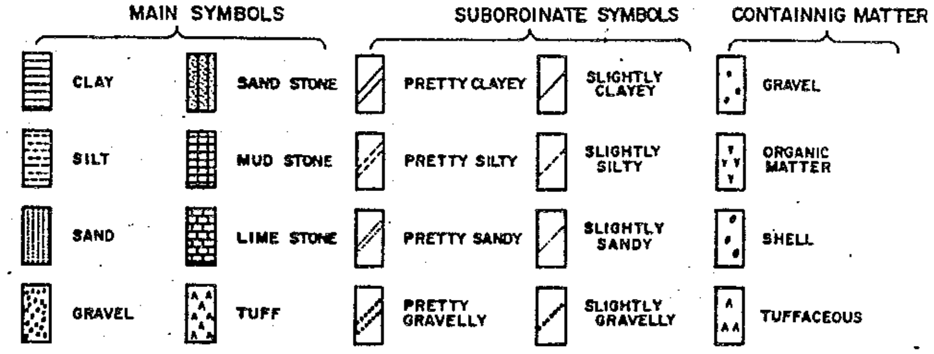
SCALE V = 1 / 500
H = 1 / 2000

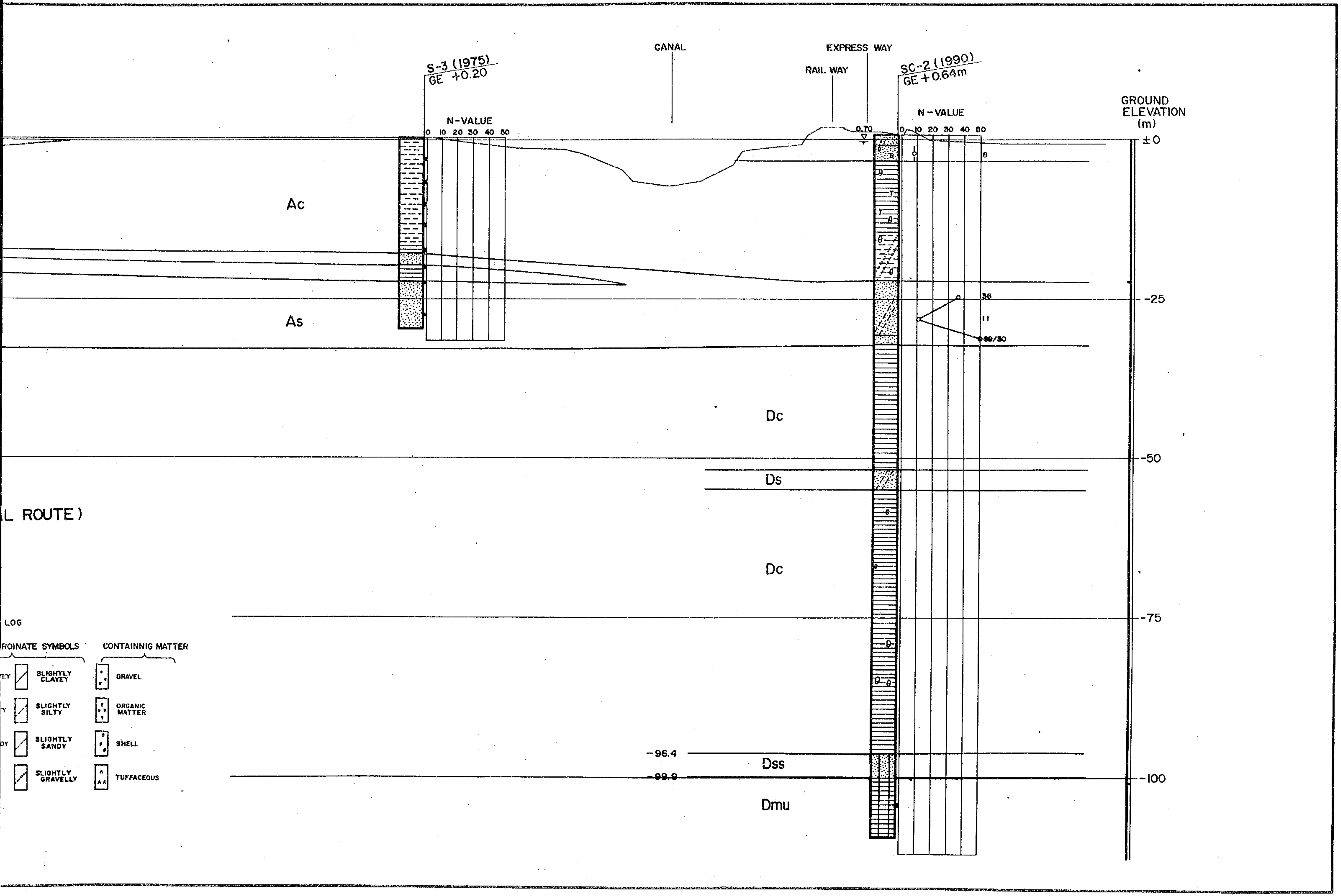
LEGEND

GEOLOGICAL SEQUENCE

GEOLOGIC TIME		FORMATION	SYMBOL	NOTES
QUATER-NARY	HOLOCENE	TOP SOIL	Ts	COMMON TOP SOIL FILL EMBANKMENT CLAY SAND
		COHESIVE SOIL	Ac1	SOFT CLAY SILT
			Ac2	SOFT TO MEDIUM CLAY SILT
		SANDY SOIL	As	FINE TO COARSE SAND SILTY SAND
	PLEISTO-CENE	COHESIVE SOIL	Dc	CLAY SILT
		SANDY SOIL	Ds	FINE TO COARSE SAND GRAVEL
		MUD STONE	Dmu	SOFT MUD STONE
		SAND STONE	Dss	FINE TO COARSE SOFT SAUD STONE

SYMBOLS OF BORING LOG




S-3 (1975)
GE +0.20
N-VALUE
0 10 20 30 40 50
CANAL
RAIL WAY
EXPRESS WAY
0.70
SC-2 (1990)
GE +0.64m
N-VALUE
0 10 20 30 40 50
GROUND ELEVATION (m)
±0
-25
-50
-75
-100
8
36
11
69/30
Ac
As
Dc
Ds
Dc
Dss
Dmu
-96.4
-99.9
L ROUTE)
LOG
ROINATE SYMBOLS
CONTAINNIG MATTER
SLIGHTLY CLAYEY
SLIGHTLY SILTY
SLIGHTLY SANDY
SLIGHTLY GRAVELLY
GRAVEL
ORGANIC MATTER
SHELL
TUFFACEOUS

Boring log

BORING LOG

SCALE : 1 / 200

PROJECT : FEASIBILITY STUDY ON CROSSING CONSTRUCTION BETWEEN RADES AND LA GOULETTE

GROUND ELEVATION + 1. 03 m

DATE 22Th TUN 1990 ~ 12Th FEB 1990
1ST March 1990 ~ 5Th March 1990

HOLE NO. SC – 1 (Ø = 101mm)

GROUNDWATER LEVEL GL – 1.10 m

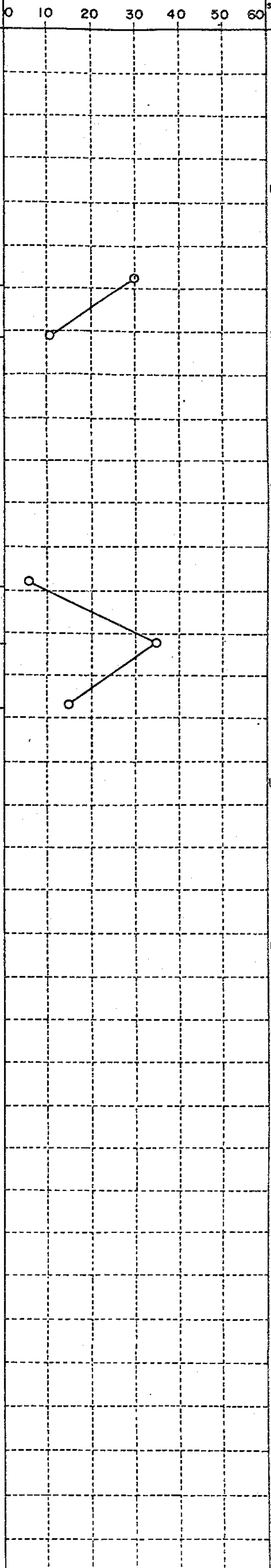
SURVEYED BY S. TAKADA HAMDY

SOIL

ELEVATION m	DEPTH m	THICKNESS OF STRATUM m	VISUAL CLASSIFICATION	COLOR	DESCRIPTION
-0.87	1.90	1.90	SAND	LIGHT BROWN	SILTY, FINE TO COARSE SAND WITH SHELL FRAGMENT
-2.47	3.50	1.60	CLAY	DARK GREY	SOFT SANDY CLAY WITH SHELL FRAGMENT COMESION: NIGH
-7.47	8.50	5.00	CLAY	BLAKKISH GREY	SOFT HUMUS CLAY COHESION : HIGH INCLUDING SHELL AND SHELL FRAGMENT HUMUS WOODS
-12.97	14.00	5.50	SAND	LIGHT BROWN	DEPTH 8.60M COBBLE Ø =10cm SILTY FINE SAND INCLUDING SHELL FRAGMENT
-17.97	19.00	5.00	SAND	BROWNISH GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL FRAGMENT
-23.07	24.10	5.10	CLAY	BROWNISH GREY	SOFT TO MEDIUM CLAY
-25.32	24.35	0.25	SAND	DARK GREEN	SILTY FINE SAND
-29.57	30.60	6.25	SAND	BROWNISH GREY	FINE SAND INCLUDING SHELL FRAGMENT
-33.67	34.70	4.10	SAND	GREY	FINE SAND INCLUDING SHELL FRAGMENT AND SLIGHTLY SILT
-34.72	35.75	1.05	SILT	BROWNISH GREY	SANDY SILT COHESION : LOW
-35.12	36.15	0.40	SAND	GREY	FINE SAND WITH SHELL FRAGMENT
-40.77	41.80	5.65	CLAY	GREY	MEDIUM TO STIFF CLAY INCLUDING SHELL FRAGMENT DEPTH 40 – 42 m SOLID STATUS
-41.92	42.95	1.15	CLAY	LIGHT GREEN	MEDIUM TO STIFF SILTY CLAY WITH SHELL FRAGMENT
-42.47	43.50	0.55	SAND	LIGHT GREEN	FINE SAND
-43.27	44.30	0.80	CLAY	LIGHT GREEN	SANDY CLAY
-43.97	45.00	0.70	CLAY	BLACK	HUMUS CLAY WITH HUMUS WOOD AND SHELL FRAGMENT
-55.97	57.00	12.00	CLAY	DARK GREY LIGHT GREENISH GREY	STIFF SILTY CLAY INCLUDING SHELL FRAGMENT COHESION : HIGH Wn : MEDIUM
-57.97	59.00	2.00	CLAY	BLACKISH GREY	MEDIUM CLAY WITH SHELL FRAGMENT
-60.17	61.20	2.20	CLAY	DARK GREY	STIFF CLAY COHESION : HIGH
-60.57	61.60	0.40	SAND	GREY	SILTY FINE SAND

STANDARD PENETRATION TESTS

DEPTH	NO OF BLOWS / LENGTH OF PENETRATION cm	NO OF BLOWS AT EACH 10cm: 15 cm	30 cm	45 cm
11.50 11.95	30/30	7	16	14
14.00 14.45	11/30	4	5	6
25.50 25.95	6/30	2	3	3
28.20 28.65	35/30	10	17	18
31.30 31.75	15/30	6	7	8



SOIL SAMPLES

NO OF SAMPLE	DEPTH m	SYMBOL OF SAMPLER
UD-1	7.20 7.70	●
S-1	11.50 11.95	○
S-2	14.00 14.45	○
UD-2	21.10 21.50	●
S-3	25.50 25.95	○
S-4	28.20 28.65	○
S-5	31.30 31.75	○
UD-3	34.75 35.25	●
UD-4	42.45 42.95	●
UD-5	50.80 51.30	●

-43.27	44.50	0.80	CLAY	LIGHT GREEN	SANDY CLAY
-43.97	45.00	0.70	CLAY	BLACK	HUMUS CLAY WITH HUMUS WOOD AND SHELL FRAGMENT
-55.97	57.00	12.00	CLAY	DARK GREY LIGHT GREENISH GREY	STIFF SILTY CLAY INCLUDING SHELL FRAGMENT COHESION: HIGH Wn : MEDIUM
-57.97	59.00	2.00	CLAY	BLACKISH GREY	MEDIUM CLAY WITH SHELL FRAGMENT
-60.17	61.20	2.20	CLAY	DARK GREY	STIFF CLAY COHESION : HIGH
-60.57	61.60	0.40	SAND	GREY	SILTY FINE SAND
-96.97	98.00	36.40	CLAY	DARK GREY BROWNISH GREY DARK GREENISH GREY	STIFF CLAY INCLUPING SHELL FRAGMENT COHESION: HIGH Wn : MEDIUM
-98.77	99.80	1.80	CLAY	YELLOWISH GREEN	HARD CLAY WITH SHELL FRAGMENT
-99.17	100.20	0.40	CLAY	GREY	INCLUDING WHITE WEATHERED ROCK
-101.47	102.50	2.30	CLAY	BLAKISH GREY	HARD CLAY
-103.97	105.00	2.50	SOFT SAND STONE	GREY	CLAYEY FINE COMPACT SAND OF SOLID STATUS
-106.97	108.00	3.00	SOFT MUD STONE	YELLOWISH GREEN GREY	HARD CLAY OF SOLID STATUS ALTERNATION OF SANDY CLAY AND FINE SAND
-110.47	111.50	3.50	SOFT SAND STONE	GREY	ALTERNATION OF SAND AND CLAY THICKNESS 1 ~ 3 cm
-113.97	115.00	3.50	CLAY (SOFT MUD STONE)	BLACKISH GREY	HARD HUMUS CLAY WITH MANY SHELL AND SHELL FRAGMENT
-117.47	118.50	3.50	SOFT MUD STONE	YELLOWISH GREEN	HARD CLAY OF SOLID STATUS
-118.72	119.75	1.25	SOFT SAND STONE	GREY TO WHITE	WITH GYPSUM
-119.07	120.10	0.35	SOFT MUD STONE	GREY	WITH GYPSUM FRAGMENT

UD-5 50.80 51.30 ●

44 46 48 50 52 54 56 58 60 62 64 66 68 70 72 74 76 78 80 82 84 86 88 90 92 94 96 98 100 102 104 106 108 110 112 114 116 118 120

REMARKS:

SYMBOLS OF SAMPLER
● THINWALL SAMPLER
○ SPLIT - SPOON SAMPLER
◉ DENISON-TYPE SAMPLER
⊕ FOIL SAMPLER
× OTHER SAMPLER

SC - 1

BORING LOG

SCALE : 1 / 200

PROJECT : FEASIBILITY STUDY ON CROSSING CONSTRUTION BETWEEN RADES AND LA GOULETTE

GROUND ELEVATION + 0. 64 m

DATE 14 Th FEB 1990 - 24 Th FEB 1990

HOLE NO. SC - 2 (ø = 101mm)

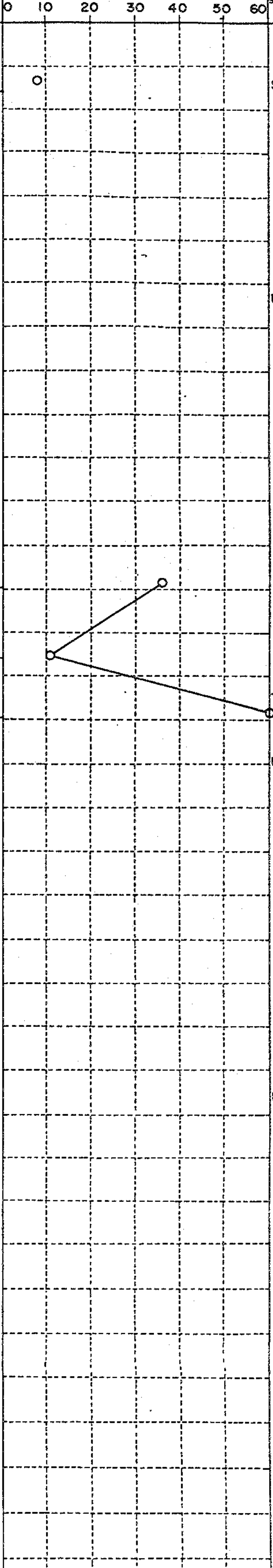
GROUND WATER LEVEL GL - 0.70 m

SURVEYED BY S. TAKAD HAMDY

ELEVATION m	DEPTH m	THICKNESS OF STRATUM m	VISUAL CLASSIFICATION	COLOR	DESCRIPTION
-0.96	1.60	1.60	SAND	LIGHT BROWN	EMBANKMENT MATERIAL SAND WITH GRAUEL
-3.36	4.60	2.40	SAND	LIGHT GREY	FINE SAND WITH SHELL FRAGMENT
-14.56	15.20	11.20	CLAY	BLACKISH GREY	VERY SOFT TO SOFT SILTY CLAY WITH SHELL FRAGMENT AND SEA WEEDS COHESION : HIGH Wn : HIGH
-20.36	21.00	5.80	SILT	GREY YELLOWISH GREEN	MEDIUM SANDY SILT WITH SHELL FRAGMENT COHESION: MEDIUM TO HIGH Wn: MEDIUM
-22.36	23.00	2.00	SILT	GREY	MEDIUM TO STIFF SANDY SILT WITH SHELL FRAGMENT
-25.36	26.00	3.00	SAND	LIGHT BROWN	FINE SAND WITH SLIGHTLY SILT AND SHELL FRAGMENT
-30.86	31.50	5.50	SAND	GREY	SILTY FINE SAND INCLUDING SHELL AND SHELL FRAGMENT
-32.36	33.00	1.50	SAND	GREY	COMPACT FINE SAND
-40.86	41.50	8.50	CLAY	BLACKISH GREY	SOFT TO MEDIUM SILTY CLAY WITH SHEEL FRAGMENT AND HUMUS SEA WEEDS COHESION: HIGH Wn : MEDIUM TO HIGH
-43.36	44.00	2.50	CLAY	YELLOW BLACKISH GREY	HUMUS MEDIUM CLAY WITH MANY CORAL FRAGMENT COHESION: HIGH Wn: MEDIUM TO HIGH
-47.56	48.20	4.20	CLAY	YELLOW BLACKISH GREY	HUMUS MEDIUM CLAY WITH CORAL FRAGMENT
-51.86	52.50	4.30	CLAY	GREENISH GREY	MEDIUM TO STIFF CLAY COHESION: HIGH
-55.06	55.70	3.20	SAND	GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL AND SHELL FRAGMENT
-60.36	61.00	5.30	CLAY	GREY	MEDIUM CLAY WITH SHELL AND SHELL FRAGMENT COHESION: HIGH
				GREY	STIFF CLAY WITH SLIGHTLY SHELL AND CORAL FRAGMENT COHESION : HIGH Wn : MEDIUM DEPTH 71.8m

STANDARD PENETRATION TESTS

DEPTH	NO OF BLOWS / LENGTH OF PENETRATION cm	NO OF BLOWS AT EACH 10 cm: 15 cm	30 cm	45 cm
2.70 3.15	8/30	4	4	4
25.50 25.95	36/30	10	16	20
28.80 29.25	11/30	3	6	5
31.50 31.95	69/30	12	31	38



SOIL SAMPLES

NO OF SAMPLE	DEPTH m	SYMBOL OF SAMPLER
S-1	2.70 3.15	○
UD-1	12.56 13.00	●
S-2	25.56 25.95	○
S-3	28.80 29.25	○
S-4	31.50 31.95	○
UD-2	33.70 34.20	●
UD-3	49.00 49.50	●
UD-4	70.00	●

				YELLOW	COHESION: HIGH Wn: MEDIUM TO HIGH
-47.56	48.20	4.20	CLAY	BLACKISH GREY	HUMUS MEDIUM CLAY WITH CORAL FRAGMENT
-51.86	52.50	4.30	CLAY	GREENISH GREY	MEDIUM TO STIFF CLAY COHESION: HIGH
-55.06	55.70	3.20	SAND	GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL AND SHELL FRAGMENT
-60.36	61.00	5.30	CLAY	GREY	MEDIUM CLAY WITH SHELL AND SHELL FRAGMENT COHESION: HIGH
-84.26	84.90	23.90	CLAY	GREY DARK GREY	STIFF CLAY WITH SLIGHTLY SHELL AND CORAL FRAGMENT COHESION : HIGH Wn : MEDIUM DEPTH 71.8m INCLODING MANY SHELL FRAGMENT
-85.76	86.40	1.50	CLAY	GREENISH YELLOW	HARD CLAY WITH MANY SHELL
-92.36	93.00	6.60	CLAY	BLACKISH GREY	VERY STIFF HOMUS CLAY WITH SHELL FRAGMENT COHESION: HIGH Wn : LOW
-96.36	97.00	4.00	CLAY	GREY	HARD CLAY OF SOLID STATUS WITH SHELL AND SHELL FRAGMENT
-99.86	100.50	3.50	SAND	GREY	SILTY COMPACT SAND OF SOLID STATUS WITH MANY SHELL AND SHELL FRAGMENT
-103.86	104.50	4.00	SOFT MUD STONE	GREENISH YELLOW	VERY HARD CLAY OF SOLID STATUS WITH SLIGHTLY SHELL FRAGMENT
-109.66	110.30	5.80	CLAY	YELLOWISH GREEN GREY	→ 105.3m WITH CORAL FRAGMENT VERY STIFF CLAY WITH SHELL FRAGMENT

UD-3	49.00 49.50	●
UD-4	70.00 70.50	●
UD-5	104.50 105.00	●

REMARKS :

SYMBOLS OF SAMPLER
● THINWALL SAMPLER
○ SPLIT-SPOON SAMPLER
◉ DENISON-TYPE SAMPLER
⊕ FOIL SAMPLER
× OTHER SAMPLER

SC- 2

Pressiometer test log

BORING LOG

SCALE : 1 / 200

PROJECT : FEASIBILITY STUDY ON CROSSING CONSTRUCTION BETWEEN RADES AND LA GOULETTE

GROUND ELEVATION + 1. 03 m

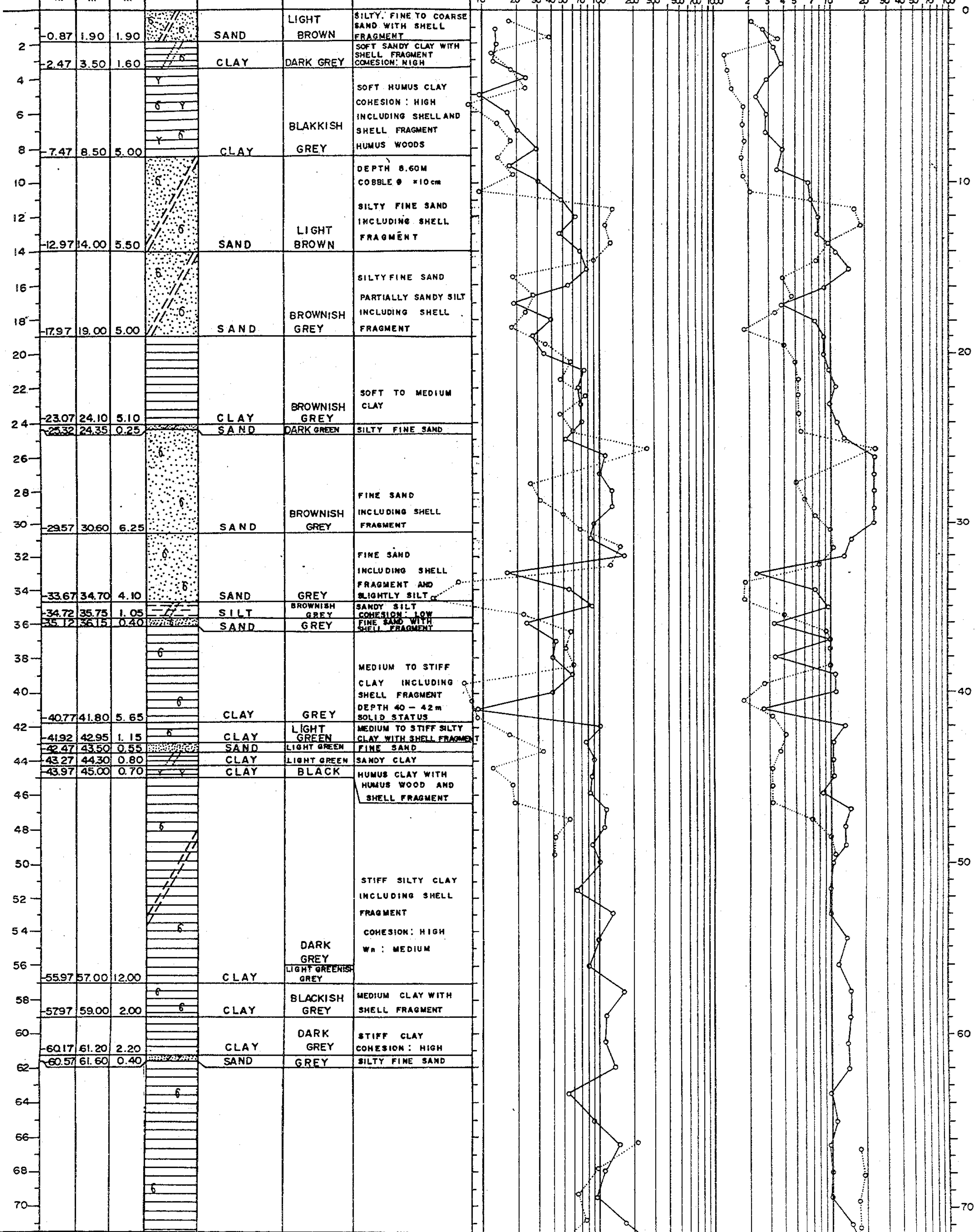
DATE 22Th TUN 1990 ~ 12Th FEB 1990 / 1st March 1990 ~ 5Th March 1990

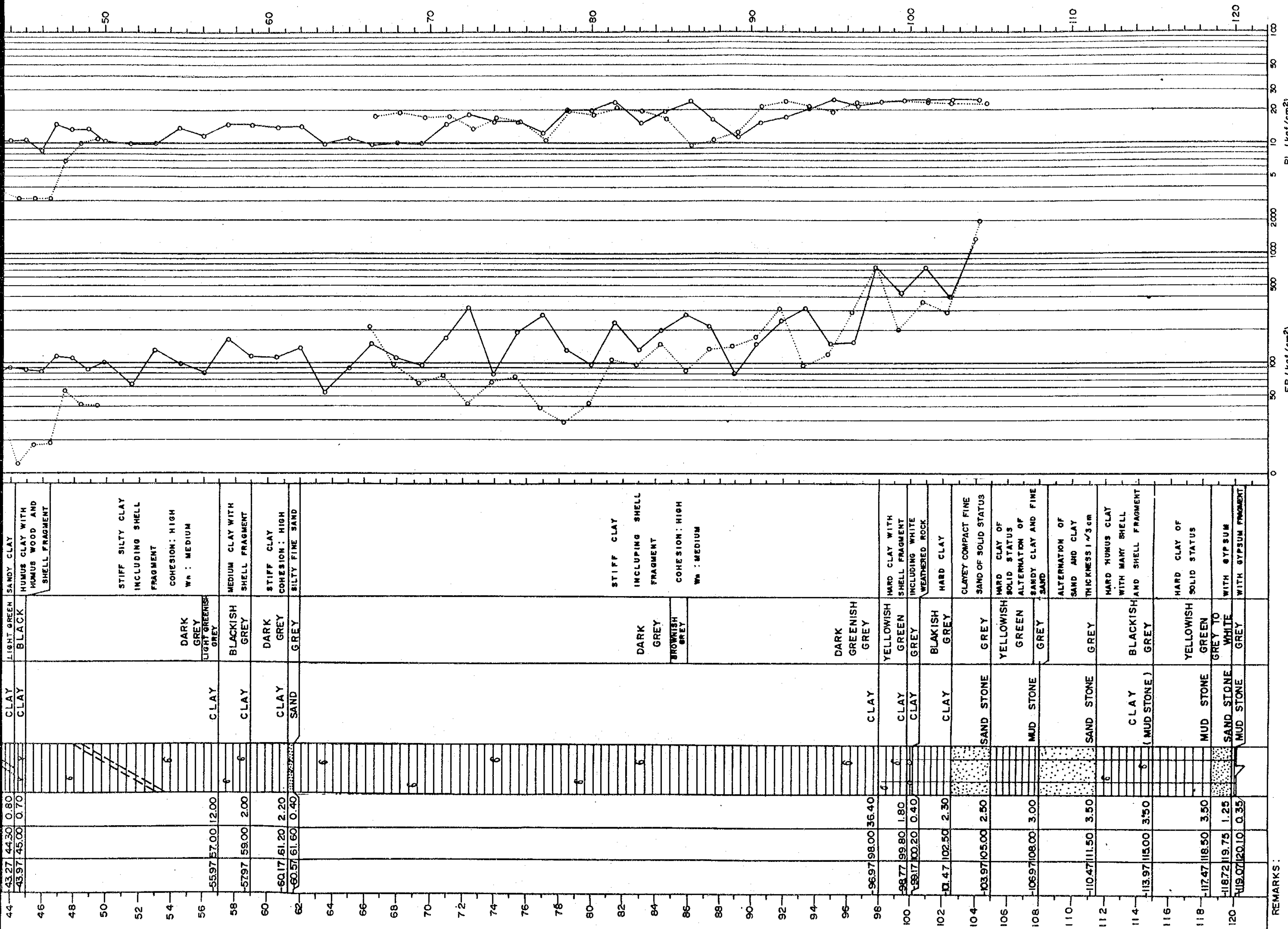
HOLE NO. SC – 1 (ø = 101mm)

GROUND WATER LEVEL GL – 1. 10 m

SURVEYED BY S. TAKADA HAMDY

SCALE	ELEVATION m	DEPTH m	THICKNESS OF STRATUM m	SOIL SYMBOL	VISUAL CLASSIFICATION	COLOR	DESCRIPTION
	-0.87	1.90	1.90		SAND	LIGHT BROWN	SILTY, FINE TO COARSE SAND WITH SHELL FRAGMENT
	-2.47	3.50	1.60		CLAY	DARK GREY	SOFT SANDY CLAY WITH SHELL FRAGMENT COMESION: NIGH
	-7.47	8.50	5.00		CLAY	BLAKKISH GREY	SOFT HUMUS CLAY COHESION : HIGH INCLUDING SHELL AND SHELL FRAGMENT HUMUS WOODS
	-12.97	14.00	5.50		SAND	LIGHT BROWN	DEPTH 8.60M COBBLE ø ×10cm SILTY FINE SAND INCLUDING SHELL FRAGMENT
	-17.97	19.00	5.00		SAND	BROWNISH GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL FRAGMENT
	-23.07	24.10	5.10		CLAY	BROWNISH GREY	SOFT TO MEDIUM CLAY
	-23.32	24.35	0.25		SAND	DARK GREEN	SILTY FINE SAND
	-29.57	30.60	6.25		SAND	BROWNISH GREY	FINE SAND INCLUDING SHELL FRAGMENT
	-33.67	34.70	4.10		SAND	GREY	FINE SAND INCLUDING SHELL FRAGMENT AND SLIGHTLY SILT
	-34.72	35.75	1.05		SILT	BROWNISH GREY	SANDY SILT COHESION : LOW
	-35.12	36.15	0.40		SAND	GREY	FINE SAND WITH SHELL FRAGMENT
	-40.77	41.80	5.65		CLAY	GREY	MEDIUM TO STIFF CLAY INCLUDING SHELL FRAGMENT DEPTH 40 – 42m SOLID STATUS
	-41.92	42.95	1.15		CLAY	LIGHT GREEN	MEDIUM TO STIFF SILTY CLAY WITH SHELL FRAGMENT
	-42.47	43.50	0.55		SAND	LIGHT GREEN	FINE SAND
	-43.27	44.30	0.80		CLAY	LIGHT GREEN	SANDY CLAY
	-43.97	45.00	0.70		CLAY	BLACK	HUMUS CLAY WITH HUMUS WOOD AND SHELL FRAGMENT
	-55.97	57.00	12.00		CLAY	DARK GREY LIGHT GREENISH GREY	STIFF SILTY CLAY INCLUDING SHELL FRAGMENT COHESION : HIGH Wn : MEDIUM
	-57.97	59.00	2.00		CLAY	BLACKISH GREY	MEDIUM CLAY WITH SHELL FRAGMENT
	-60.17	61.20	2.20		CLAY	DARK GREY	STIFF CLAY COHESION : HIGH
	-60.57	61.60	0.40		SAND	GREY	SILTY FINE SAND





SC-1 (SP-1)
PL (kgf/cm2)
EP (kgf/cm2)
LEGEND:
SP. 1 (SC.1 GE = 1.03m)
SP. 2 (GE = 1.67m)
REMARKS:
-43.27 44.30 0.80 CLAY LIGHT GREEN SANDY CLAY
-43.97 45.00 0.70 CLAY BLACK HUMUS CLAY WITH HUMUS WOOD AND SHELL FRAGMENT
-55.97 57.00 12.00 CLAY DARK GREY / LIGHT GREENISH GREY STIFF SILTY CLAY INCLUDING SHELL FRAGMENT COHESION: HIGH Wn: MEDIUM
-57.97 59.00 2.00 CLAY BLACKISH GREY MEDIUM CLAY WITH SHELL FRAGMENT
-60.17 61.20 2.20 CLAY DARK GREY STIFF CLAY COHESION: HIGH
-60.57 61.60 0.40 SAND GREY SILTY FINE SAND
-96.97 98.00 36.40 CLAY DARK GREY / BROWNISH GREY / DARK GREENISH GREY STIFF CLAY INCLUPING SHELL FRAGMENT COHESION: HIGH Wn: MEDIUM
-98.77 99.80 1.80 CLAY YELLOWISH GREEN HARD CLAY WITH SHELL FRAGMENT
-99.17 100.20 0.40 CLAY GREY INCLUDING WHITE WEATHERED ROCK
-101.47 102.50 2.30 CLAY BLAKISH GREY HARD CLAY
-103.97 105.00 2.50 SAND STONE GREY CLAYEY COMPACT FINE SAND OF SOLID STATUS
-106.97 108.00 3.00 MUD STONE YELLOWISH GREEN / GREY HARD CLAY OF SOLID STATUS ALTERNATION OF SANDY CLAY AND FINE SAND
-110.47 111.50 3.50 SAND STONE GREY ALTERNATION OF SAND AND CLAY THICKNESS 1~3cm
-113.97 115.00 3.50 CLAY (MUD STONE) BLACKISH GREY HARD HUMUS CLAY WITH MANY SHELL AND SHELL FRAGMENT
-117.47 118.50 3.50 MUD STONE YELLOWISH GREEN HARD CLAY OF SOLID STATUS
-118.72 119.75 1.25 SAND STONE GREY TO WHITE WITH GYPSUM
-119.07 120.10 0.35 MUD STONE GREY WITH GYPSUM FRAGMENT
44 46 48 50 52 54 56 58 60 62 64 66 68 70 72 74 76 78 80 82 84 86 88 90 92 94 96 98 100 102 104 106 108 110 112 114 116 118 120
50 60 70 80 90 100 110 120
100 50 30 20 10 5
2000 1000 500 100 50 0

ANNEXE 7.5.1.1

NEW SEISMIC DESIGN SPECIFICATION FOR ROAD BRIDGE IN JAPAN

NEW SEISMIC DESIGN SPECIFICATIONS FOR ROAD BRIDGE IN JAPAN

by
Toshio IWASAKI
Kazuhiko KAWASHIMA
and
Kinji HASEGAWA

Public Works Research Institute
Ministry of Construction

22nd Joint Meeting
U.S.—Japan Panel on Wind and Seismic Effects, U.J.N.R.
Gaithersburg, Maryland, U.S.A., May, 1990

NEW SEISMIC DESIGN SPECIFICATIONS OF HIGHWAY BRIDGES IN JAPAN

Toshio IWASAKI[1], Kazuhiko KAWASHIMA[2] and Kinji HASEGAWA[3]

ABSTRACT

Introduced are the new Seismic Design Specifications of Highway Bridges in Japan issued on February 1990. Main items of the revision including the seismic lateral force, evaluation of inertia force considering structural response, check of bearing capacity for lateral load for reinforced concrete pier and dynamic response analysis are presented.

KEY WORDS

Seismic Design, Design Method, Highway Bridge

1. INTRODUCTION

First requirement on lateral force for design of road bridge in Japan was included in "Draft Details of Road Structure" issued in 1926 with considerations of the damage caused by the Kanto Earthquake in 1923. It was stipulated in the draft details that the maximum lateral force expected to develop at the site shall be considered in seismic design. After experiencing significant damages during strong earthquakes seismic regulations were reviewed and amended several times as shown in Table 1. "Design Specifications of Steel Road Bridges (Draft)" were issued in 1914, and "Design Specifications of Steel Road Bridges" and their revised version were issued in 1934 and 1939, respectively. Although a requirement for lateral force was stipulated in these specifications, first comprehensive seismic design stipulations were issued in 1971 in a separate volume exclusively for seismic design as "Seismic Design Specifications of Road Bridges". It was described in the specifications that lateral force shall be determined depending on zone, importance and ground condition in seismic coefficient method and structural response shall be further considered in the modified seismic coefficient method. Evaluation on soil liquefaction was firstly incorporated in view of the damages caused by the Niigata Earthquake in 1964. The 1971 specifications were revised in a form of "Part V Seismic Design" of "Design Specifications of Road Bridges" in 1980. A rational evaluation method for soil liquefaction as well as practical countermeasures against the liquefaction was included in the specifications.

Since 1985 extensive efforts have been undertaken to establish more rational seismic design criteria for road bridges, with consideration of recent advantages of earthquake engineering and lessons gained from recent earthquakes. New specifications were were completed in 1990.

This paper outlines the new specifications (JRA, 1990) for seismic design of road bridges with emphasis on the stipulations revised.

2. BASIC PRINCIPLES FOR DEVELOPING NEW SPECIFICATIONS

Road bridge is a vital component of road, and it has to be safe enough against an earthquake so that function of road be maintained. It can be used without loosing any structural functions against small to moderate earthquakes with high to moderate possibility to occur at the site. Critical failure causing total collapse of the bridge has to be avoided even during a significant earthquake such as the Kanto Earthquake in 1923. Seismic design method has been gradually improved with consideration on damage experiences and advantage of earthquake engineering. Based on such improvement of seismic design method as well as various countermeasures against an earthquake, it is considered that seismic damages of road bridge have been decreasing as a total in recent years as shown in Table 2. However it is also important to note that new damages which have not been identified in the past earthquakes have been seen in recent earthquakes, due to improvement of seismic design method. This obviously shows that the countermeasures aiming only to minimize the damages developed in the past earthquake do not give a credit to avoid new type of damage in the future earthquake. In revising the design specifications, it is required to take account of such new type of

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damage patterns.

In view of the understanding on the damage patterns of road bridges in recent earthquakes and advances of earthquake engineering, the following revisions were included in the New Seismic Design Specifications.

1) modification on classification of ground condition
2) unification of seismic coefficient method and modified seismic coefficient method including the revision of the seismic coefficient
3) evaluation method of inertia force for continuous bridges
4) improvement on strength of sandy soils against liquefaction
5) evaluation of bearing capacity of reinforced concrete piers for lateral force (ductility check)
6) design ground motion for dynamic response analysis as well as an analytical model

Table of content of the new specifications is the following :

Chapter 1	General
Chapter 2	Basic Principle of Seismic Design
Chapter 3	Load and Design Condition in Seismic Design
Chapter 4	Seismic Coefficient
Chapter 5	Bearing Capacity for Lateral Force of Reinforced Concrete Piers
Chapter 6	Dynamic Response Analysis
Chapter 7	Structural Details in Seismic Design
Chapter 8	Device to Expect Reduction of Lateral Force

(Appendix)

Ⅰ. References on Liquefaction
Ⅱ. Examples of Classification of Ground Condition
Ⅲ. References on Design Ground Motion
Ⅳ. Example of Calculation of Natural Period and Inertia Force
Ⅴ. Reference on Bearing Capacity of Reinforced concrete Piers for Lateral Force
Ⅵ. Practices of Structural Details

Table 3 shows the revised part or newly introduced parts in the new specifications, and Fig. 1 shows the flow-chart of seismic design.

3. CLASSIFICATION OF GROUND CONDITION

Classification of ground condition was changed from four groups to three groups as shown in Table 4 by combining the group 2 and group 3 into same group. The classification of the ground condition is made base on the characteristics value at the site specified as

$$T_G = 4 \sum_{i=1} \frac{H_i}{V_{si}} \qquad (1)$$

where

T_G : characteristic value (sec)
H_i : thickness of i-th subsoil layer (m)
V_{si} : shear wave velocity of i-th sublayer (m/sec)
i : sublayer's number counted from ground surface

The change was made because most of the seismic damage of road bridge in the past earthquake was likely to be developed on the group 4 with the difference between the ground group 2 and 3 being practically very small, and because difference of ground motion characteristics in terms of response acceleration spectra is few between the ground group 2 and 3. It was also taken into account that the classification of the ground condition based on the difference of characteristic value of only 0.2 second is practically quite hard in view of the fact that shear wave velocity is generally estimated from an empirical formulae based on N-value of standard penetration test.

4. SEISMIC COEFFICIENT

In the previous specifications, the seismic coefficient method was used for the bridges with pier height less or equal to 15 m and the modified seismic coefficient method was used for the bridges with pier height larger than 15 m. The modified seismic coefficient method was originally intended to apply to bridges with the natural period longer than 0.5 second. However, as various foundations becomes to be adopted on wide range of soil conditions, even bridges with pier height less than 15 m is likely to have the natural period longer than 0.5 second. This always caused confusion in selecting the seismic design method.

Therefore, the seismic coefficient method the substructure subjected to a lateral force equivalent with 80 % of the dead weight of a substructure above the ground surface assumed in seismic design and the dead weight of a part of the superstructure supported by the substructure.

It should be noted here that only the substructure on the ground surface assumed in seismic design shall be subjected to inertia force because the seismic force below the ground surface assumed in seismic design is disregarded in seismic design. The ground surface assumed in seismic design is generally taken as the bottom plane level of the footing in case of pile foundation. However, when the soil layers whose soil constant is assumed as zero due to possible soil failure such as liquefaction, the bottom of these layers is regarded as the ground surface assumed in seismic design.

2)Seismic design structural unit consisting of substructures and a part of superstructure supported by these substructures.

This is the case newly introduced in the new specifications. Natural period and the inertia force shall be evaluated in accordance with Fig. 4, i.e.,

i)idealize a seismic design structural unit by a linear elastic frame model

ii)apply a lateral force equivalent with the dead weight of superstructure and substructure above the ground surface assumed in seismic design, and compute the natural period as

$$T = 2.01 \sqrt{\delta} \qquad (5)$$

$$\delta = \frac{\int w(s)u(s)^2 ds}{\int w(s)u(s) ds} \qquad (6)$$

where

w(s) : dead weight of the seismic design structural unit (superstructure and substructure above the ground surface assumed in seismic design) at point s (tf/m)

u(s) : lateral displacement developed in the seismic design structural unit at point s (m) when subjected to w(s) in a direction considered in design.

iii)determine the seismic coefficient k_h depending on the natural period T.

iv)compute inertia force as

$$F_d = k_h \times F \qquad (7)$$

where

F_d : shear force (tf) / bending moment (tfm) due to inertia force

k_h : seismic coefficient

F : force developed in the seismic design structural unit when subjected to a lateral force equivalent with dead weight of seismic design structural unit above the ground surface assumed in seismic design (tf/tfm)

For substructure supporting girder bridges, shearing force developed at the center of gravity of superstructure shall be regarded as the lateral force for seismic design. However, when the inertia force computed by Eq.(7) is smaller than the inertia force computed by Eq.(3), the latter shall be adopted for design. This needs some explanations. The inertia force computed by Eq.(7) is approximately proportional to the stiffness of each substructure. This implies that the inertia force concentrates to the substructures with high stiffness, while reverse is true for the substructure with low stiffness. Inertia force takes even negative value when the stiffness of the substructure is extremely small as compared with other substrctures. However, when failure of the structure such as bearing supports occurs, the contribution of each substructure for supporting the inertia force of superstructure could be changed from the distribution computed by Eq.(7). Base on such considerations, lower limit for the inertia force evaluated by Eq.(7) was included.

6. STRENGTH OF SANDY SOIL LAYER FOR LIQUEFACTION

For saturated alluvial sandy layers within 20 m from the ground surface which have the water table within 10 m from the ground surface, and have D_{50}-value on the grain size accumulation curve between 0.02 and 2.0 mm, the liquefaction potential during an earthquake has to be checked. For those soil layers required for the check of liquefaction, liquefaction potential shall be evaluated in terms of liquefaction resistance factor F_L as

$$F_L = R / L \qquad (8)$$

where

F_L : liquefaction resistance factor

R : resistance of soil elements against dynamic load

L : dynamic load induced in soil eleand the modified seismic coefficient method was unified, and this is referred to as seismic coefficient method.

In the seismic coefficient method, the design seismic coefficient shall be determined by Eq.(2), but no less than 0.1.

$$k_h = c_z \cdot c_G \cdot c_I \cdot c_T \cdot k_{h0} \qquad (2)$$

where

k_h : design horizontal seismic coefficient,

k_{h0} : standard design horizontal seismic coefficient (=0.2),

c_z : modification factor for zone (refer to Fig. 2),

c_G : modification factor for ground condition (refer to Table 5),

c_I : modification factor for importance (refer to Table 6), and

c_T : modification factor for structural response (refer to Table 7). For computing inertia force associated with the weight of soils and dynamic earth pressure, c_T has to be 1.0.

In Eq.(2) the modification factors c_z and c_I are the same with the value specified in the previous specifications. The modification factor c_G was changed associated with the change of ground condition classification. The modification factor c_T, which represents the difference of lateral force due to structural response, corresponds to the modification factor β in the previous specifications. The modification factor β was formulated based on an analysis of 44 components of strong motion records obtained in Japan. The new modification factor c_T was determined from an analysis of earthquake response spectra base on 394 components of strong motion records obtained in Japan. Considerations that the seismic damage was likely to occur in bridges with short natural period were also included.

Fig. 3 shows an comparison of the seismic coefficient between the previous and new specifications assuming that $c_Z = c_T = 1.0$.

5. EVALUATION METHOD OF INERTIA FORCE

In the previous specification, for computing an inertia force associated with dead weight of superstructure, which is to be used in seismic design of substructure, the bridge is divided into several structural segments consisting of a substructure and the part of superstructure which is vertically supported by the substructure considered. Then, the inertia force (lateral force) used for the seismic design of substructure of the i-th structural segment is assumed to be determined as

$$F_{di} = k_{hi} R_i \qquad (3)$$

where

F_{di} : inertia force associated with dead weight of superstructure for design of i-th substructure

k_{hi} : seismic coefficient considered for i-th structural segment

R_i : reaction force developed at i-th substructure due to dead weight of the part of superstructure supported by the i-th substructure.

However as various type of bridge structures become to be constructed, it became apparent that the inertia force determined by Eq.(3) assuming that the bridge can be divided into several structural segments is inadequate, because the inertia force at each substructure significantly depends on flexibility and height of the substructure and type of the superstructure. Therefore, more rational method to evaluate the inertia force induced in the superstructure for the purpose of seismic design of substructure was required. The same demand also arises in the bridge where the superstructure and the substructure are not separated by means of bearing supports.

Therefore, in stead of the structural segment, a definition of seismic design structural unit, defined as the structural unit which can be considered to develop the same response during an earthquake, was newly introduced in the new specifications. Table 8 shows examples how the seismic design structural unit be selected for typical combinations of structural type and direction for evaluating the inertia force.

Natural period and the inertia force shall be determined as:

1)Seismic design structural unit consisting of a substructure and a part of superstructure supported by the substructure

In this case, there is essentially no difference with the previous specifications. Inertia force shall be determined by Eq.(4), and the natural period shall be computed as

$$T = 2.01 \sqrt{\delta} \qquad (4)$$

where

T : natural period, in second, of the seismic design structural unit

δ : lateral displacement, in meter, of

ments during an earthquake

In the previous specifications the resistance R of soil elements against dynamic load was evaluated from N-value of standard penetration test and D_{50}-value. Because fine sand content is found an important factor, it was included for evaluating R.

7. CHECK OF BEARING CAPACITY OF REINFORCED CONCRETE PIERS FOR LATERAL FORCE

Even for a significant earthquake, the bridge has to avoid critical failure. Reinforced concrete pier which is the most important member for preventing critical failure of bridge has to have the required ductility as well as the strength. The check of bearing capacity for lateral load was introduced to avoid brittle failure of reinforced concrete piers in inelastic range. Check of bearing capacity for lateral load is advised for the reinforced concrete piers with small concrete section.

Fig. 5 shows a flow-chart of the check of bearing capacity of reinforced concrete piers for lateral force. Main steps of this check are the following :

1) Judgement of bearing capacity for lateral force

Bearing capacity for lateral force shall be checked as

$$P_a > k_{he}W \qquad (9)$$

where

P_a : bearing capacity of reinforced concrete pier for lateral force (tf)

k_{he} : equivalent horizontal seismic coefficient for check of bearing capacity for lateral load

W : equivalent dead weight (tf), and shall be determined as

$$W = W_u + c_P W_P \qquad (10)$$

$$c_P = \begin{cases} 0.5 & P_u \leqq P_s \\ 1.0 & P_u > P_s \end{cases} \qquad (11)$$

W_u : dead weight of a part of superstructure supported by the reinforced concrete pier (tf)

W_P : dead weight of the reinforced concrete pier (tf)

P_u : bearing capacity of reinforced concrete pier for flexural failure (tf)

P_s : bearing capacity of reinforced concrete pier for shear failure (tf)

When the check for the bearing capacity for lateral force by Eq.(9) cannot be satisfied, modification of the design has to be made so that the modified section, which is designed in accordance with the seismic coefficient method, satisfies Eq.(9). It is generally advised to increase the concrete section with decreasing the amount of reinforcement.

2)Equivalent horizontal seismic coefficient for check of bearing capacity for lateral force

Equivalent horizontal seismic coefficient for check of bearing capacity for lateral force shall be determined as

$$k_{he} = \frac{k_{ho}}{\sqrt{2\mu - 1}} \qquad (12)$$

$$k_{ho} = c_Z \cdot c_I \cdot c_R \cdot k_{hoo} \qquad (13)$$

where

k_{he} : equivalent horizontal seismic coefficient for check of bearing capacity for lateral force

k_{hc} : seismic horizontal coefficient for check of bearing capacity for lateral force

μ : allowable ductility factor

c_Z : modification factor for zone (refer to Fig. 2)

c_I : modification factor for importance (refer to Table 6)

c_R : modification factor for structural response (refer to Table 9)

K_{hco} : standard horizontal seismic coefficient for check of bearing capacity for lateral force

The standard horizontal seismic coefficient k_{hoo} was determined so that it can represent a realistic ground motion developed during a significant earthquake with magnitude as large as 8.

3)Bearing capacity for lateral force and allowable ductility factor

Bearing capacity for lateral force P_a and the allowable ductility factor μ shall be determined based on the failure mode as :

a)Flexural failure

$$P_a = P_y + \frac{P_u - P_y}{\alpha} \qquad (14)$$

$$\mu = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y} \qquad (15)$$

where

P_u, δ_u : bearing capacity (tf) and ultimate displacement (m) for flexural failure

P_y, δ_y : yielding force (tf) an yielding displament (m) for flexural failure

α : safety factor (=1.5)

b)Shear failure

$$P_e = P_s \quad (16)$$

$$\mu = 1 \quad (17)$$

where

P_s : bearing capacity (tf) for shear failure

8. DYNAMIC RESPONSE ANALYSIS

As number of bridges with complex dynamic response characteristic increases, unified stipulations for dynamic response analysis become required. Although only general consideration for selecting the input ground motion was described in the previous specifications, a simplified but practical input ground motion as well as an analytical model and evaluation of the results of the dynamic response analysis was introduced in the new specifications.

In principle, dynamic response analysis shall be made by means of response spectral analysis with use of an analytical model which simulates dynamic characteristics of the bridge. Acceleration response spectrum for the response spectrum analysis shall be determined as

$$S = c_Z \cdot c_I \cdot c_D \cdot S_0 \quad (18)$$

where

S : response spectrum for response spectrum analysis (gal)

c_Z : modification factor for zone (refer to Fig. 2)

c_I : modification factor for importance (refer to Table 6)

c_D : modification factor for damping, and shall be determined based on modal damping ratio h_i as

$$c_D = \frac{1.5}{40h_i + 1} + 0.5 \quad (19)$$

S_0 : standard response spectrum for response analysis method (gal) (refer to Table 10)

When the time history analysis is required, strong motion records which have the similar characteristics with S by Eq.(18) shall be used with the consideration on site condition and structural response of the bridge. Three ground acceleration records which were modified so that their response characteristics match with S_0 in Eq.(18) are provided in Appendix III. They can be used as the standard ground motions for the time history analysis.

Because the results of the dynamic response analysis highly depend on the analytical model and the properties assumed in the analysis, indiscriminate evaluation of the results is quite difficult. Therefore it is required to use the overall result of the analysis considering accuracy of the properties and the analytical model used in the analysis. It is stipulated in the new specifications that the results obtained by the dynamic response analysis shall be used for the check of seismic safety of the bridge based on the allowable stress and displacement which are referred in the seismic coefficient method.

9. CONCLUDING REMARKS

The proceeding pages presented the outline of the main points of the new Seismic Design Specifications for Highway Bridges issued in February 1990. In the new specifications, seismic coefficient method by combining the previous seismic coefficient method and the modified seismic coefficient method is adopted, and based on the allowable design approach this method is regarded as the basic seismic design method. Besides the design by means of the seismic coefficient method, for those bridges which require the check of ductility, the check of bearing capacity of reinforced concrete piers for lateral force is made. Dynamic response analysis can be made for the bridges which require precise analysis considering structural response.

REFERENCES

1) Kawakami,K., Kuribayashi,E., Iwasaki,T. and Iida,Y.: On Specifications for Earthquake Resistant Design of Highway Bridges, 7th Joint Meeting, Panel on Wind and Seismic Effects, UJNR, Tokyo, Japan, 1975

2) Kuribayashi,E., Iwasaki,T., and Ueda,O.: New Specifications for Earthquake Resistant Design of Highway Bridges in Japan, 12th Joint Meeting, Panel on Wind and Seismic Effects, UJNR, Washington, D.C., U.S.A., 1980

Table 1 History of Design Loads for Highway Bridges in Japan

Year	Name of Regulations	Design Live Loads				Impact Loads	Seismic Loads k: Horizontal Seissmic Coefficient	Major Earthquake
		Class	Truck Roller Loads Streetcar	Uniform Loads	Line Loads			
1) 1886	Order No. 13 Ministry of Home Affairs (MHA)			U-454 kg/m	-	not considered	not considered	1946. Nankai (M=8.1) 1948. Fukui (M=7.3) 1952. Tokachi-oki (M=8.2) 1964 Niigata (M=7.5)
2)	Order MHA		R-13.6 t S-12.7	U-400 600 kg/m (carriage way) U-270 400 kg/m (footway)		not condered	not considered	
3) 1919	Road Laws MHA		R-13.6 t T-11.2 t S-30 t	Same as 2)	-	not considered	not considered	
4) 1926	Specifications for Design of Roads, Road Laws, MHA	1st 2nd 3rd	T-12 t T-8 T T-6 t	U-600 kg/m U-500 kg/m U-500 kg/m	-	considered	Seismic Coefficient Method K=0.15~0.4 depending on location and ground condition ($k \geqq 0.3$ advised in Toyo, Yokohama)	
5) 1939	Specifications for Design of Steel Road Bridges, MHA	1st 2nd	T-13 t T-9 T	U-500 kg/m U-400 kg/m		considered	Seismic Coefficient Method k_s=0.2, k_v=0.1	
6) 1956 (and 1964)	Revision of Specifications for Design of Steel Road Bridges, JRA	1st 2nd	T-20 t T-14 t	- -	L-20 (5t.m) L-14 (3.5 t/m)	considered	Seismic Coefficient Method k=0.1~0.35 depending on location and ground conditions	
7) 1964 to 1971	Specifications for Design of Substructure of Road Bridges, JRA	Same as 6)					Same k as (6) Detailed calculation methods	
8) 1971	Specifications for Earthquake-Resistant Design of Highway Bridges, JRA	Same as 6)					Seismic Coef. method k=0.1~0.24 (Rigid) Modified SCM. K=0.05~0.3 (Flexible)	1978 Miyagiken-oki (M=7.4)
9) 1980 Modified SCM	Part V Seismic Design Specifications for Highway Bridges JRA	Same as 6)					Seismic Coef. Method k=0.1~0.24 (Rigid) k=0.05~0.3 (Flexible) Earthquake Response Analysis (Very Flexible Bridges)	
10) 1990	Same as 9)	Same as 6)					Seismic Coefficient Method k=0.1~0.3 Bearing Capacity of RC Piers for Lateral Load Dynamic Response	1982 Urakawa-oki (M=7.1) 1982 Nihon-kai-chubu(M=7.7)

Table 2 Seismic Damage in Recent Earthquakes

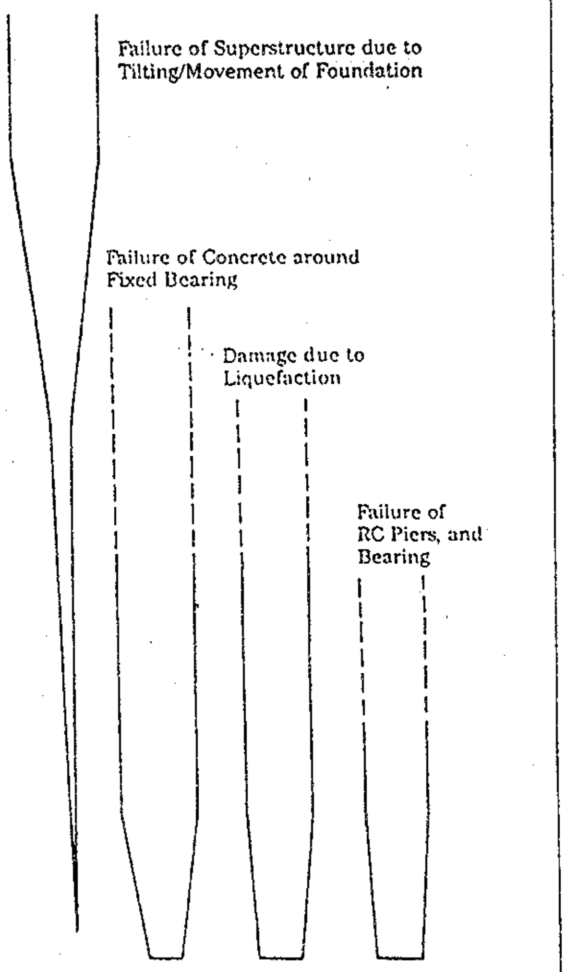
Year	Major Earthquake	Change of Major Seismic Damage	Seismic Design Method	Seismic Inspection and Retrofitting
1920	1923 Kanto Earthquake (M7.9)	 Failure of Superstructure due to Tilting/Movement of Foundation	1926 Initiation of Seismic Design (Stipulations for Road Construction)	
1930		Failure of Concrete around Fixed Bearing	1939 Introduction of Standard Seismic Coefficient (Specifications for Steel Bridge)	
1940	1946 Nankai Earthquake (M8.1) 1948 Fukui Earthquake (M7.3)	Damage due to Liquefaction		
1950	1952 Tokachi-oki Earthquake (M8.1)	Failure of RC Piers, and Bearing	1956 Seismic Coefficient depending on Zone and Ground Condition (Specifications for Steel Bridge)	
1960	1964 Niigata Earthquake (M7.5)			
1970	1978 Miyagi-ken-oki Earthquake (M7.4)		1971 • Seismic Coefficient depending on Zone, Ground Conditions, Importance and Structural Response • Introduction of Evaluation Method for Liquefaction (Specifications for Seismic Design)	1971 Seismic Inspection 1976 Seismic Inspection 1979 Seismic Inspection
1980	1982 Urakawa-oki Earthquake (M7.1) 1983 Nihon-kai-chubu Earthquake (M7.7)		1980 • Part V Seismic Design, Specifications for Design of Highway Bridge • Introduction of New Evaluation Method for Liquefactions	1986 Seismic Inspection
1990			1990 Part V Seismic Design, Specificatins for Design of Road Bridge	

Table 3　Table of Contents of 1990 Seismic Design Specifications of Highway Bridges

	Contents	Same as 1980 Specs.	Modified		Newly Introduced
			Slightly	Completely	
Chapter 1	General 1.1 Scope	○			
	1.2 Definition of Term		○		
Chapter 2	Basic Principle for Seismic Design		○		
Chapter 3	Design Load and Design Condition in Seismic Design 3.1 Design Load and Load Combination in Seismic Design		○		
	3.2 Effect of Earthquake	○			
	3.3 Inertia Force 3.3.1 General			○	
	3.3.2 Calculation of Natural Period			○	○
	3.3.3 Calculation of Inertia Force			○	○
	3.4 Earth Pressure during Earthquake	○			
	3.5 Hydrodynamic Pressure during Earthquake		○		
	3.6 Classification of Ground Condition in Seismic Design		○		
	3.7 Soil Layers Whose Bearing Capacities are Reduced in Seismic Design 3.7.1 General				○
	3.7.2 Sandy Layer Vulnerable for Liquefaction		○		
	3.7.3 Very Soft Clayey and Silty Layer	○			
	3.7.4 Treatment of Soil Layers Whose Bearing Capacities are Reduced in Seismic Design	○			
	3.8 Ground Surface Assumed in Seismic Design	○			
Chapter 4	Design Seismic Coefficient 4.1 General	○			
	4.2 Design Horizontal Seismic Coefficient			○	
	4.3 Factors for Modifying Standard Design Horizontal Seismic Coefficient			○	
Chapter 5	Check of Bearing Capacity of Reinforced Concrete Pier for Lateral Load 5.1 General				○
	5.2 Check of Safety				○
	5.3 Horizontal Seismic Coefficient for Check of Bearing Capacity for Lateral Load 5.3.1 Equivalent Horizontal Seismic Coefficient for Check of Bearing Capacity for Lateral Load				○
Chapter 5 (cont'd)	5.3.2 Design Horizontal Seismic Coefficient for Check of Bearing Capacity for Lateral Load				○
	5.4 Bearing Capacity for Lateral Load 5.4.1 Bearing Capacity for Lateral Load Allowable Ductility and Equivalent Natural Period				○
	5.4.2 Displacement and Lateral Load at Yielding and Ultimate Stage				○
	5.4.3 Bearing Capacity for Shear Failure				○
Chapter 6	Dynamic Response Analysis 6.1 General				○
	6.2 Analytical Method and Analytical Model 6.2.1 Analytical Method				○
	6.2.2 Analytical Model				○
	6.3 Input Motion for Dynamic Response Analysis 6.3.1 Acceleration Response Spectral Value for Response Spectral Analysis				○
	6.3.2 Grond Motion for Time History Analysis				○
	6.4 Check of Safety				○
Chapter 7	General Provisions for Structural Details in Seismic Design 7.1 General		○		
	7.2 Devices for Preventing Superstructures from Falling 7.2.1 General		○		
	7.2.2 Stoppers at Movable Bearing		○		
	7.2.3 Length between End of Girder and Edge of Substructure		○		
	7.2.4 Devices for Preventing Superstructure from Falling		○		
	7.3 Structural Details at Bearing		○		
Chapter 8	Devices Expecting to Reduce Seismic Effects			(○)	
Appendices	I. References on Liquefaction		○		
	II. Examples of Classification of Ground Condition		○		
	III. Reference on Design Ground Motion			○	
	IV. Example of Calculation of Natural Period and Inertia Force			○	
	V. Reference on Bearing Capacity of Reinforced Concrete Pier for Lateral Load and Example of Calculation				○
	VI. Practices of Structural Details		○		

Table 4 Classification of Ground Conditions

1980 Specifications			1990 Specifications	
1	$T_G < 0.2$	(Rock)	I	$T_G < 0.2$
2	$0.2 \leqq T_G < 0.4$	(Diluvium)	II	$0.2 \leqq T_G < 0.6$
3	$0.4 \leqq T_G < 0.6$	(Alluvium)		
4	$0.6 \leqq T_G$	(Soft Alluvium)	III	$0.6 \leqq T_G$

Table 5 Ground Condition Factor c_G

Ground Group	I	II	III
c_G	0.8	1.0	1.2

Table 6 Importance Factor c_I

Group	c_I	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road. Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 7 Structual Response Factor c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$ $c_T = 2.69T^{1/3} \geqq 1.00$	$0.1 \leqq T \leqq 1.1$ $c_T = 1.25$	$1.1 < T$ $c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$ $c_T = 2.15T^{1/3} \geqq 1.00$	$0.2 \leqq T \leqq 1.3$ $c_T = 1.25$	$1.3 < T$ $c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$ $c_T = 1.80T^{1/3} \geqq 1.00$	$0.34 \leqq T \leqq 1.5$ $c_T = 1.25$	$1.5 < T$ $c_T = 1.64T^{-2/3}$

Table 8 Seismic Design Structual Unit

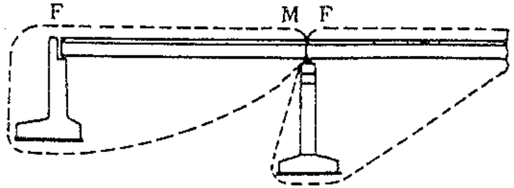
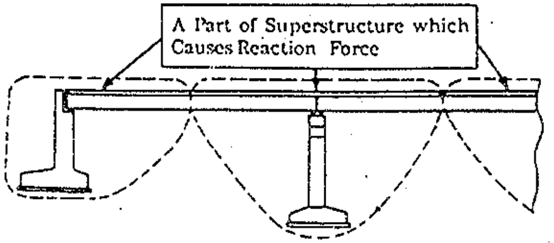
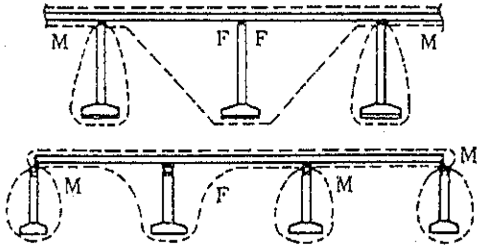
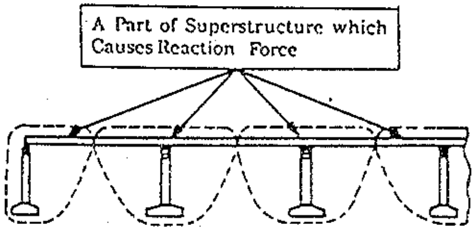
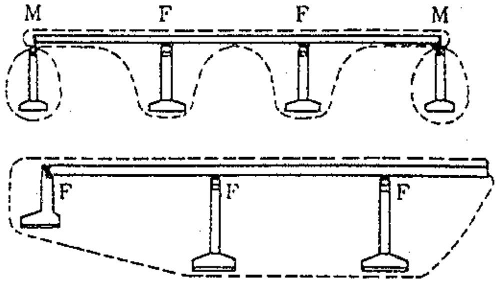
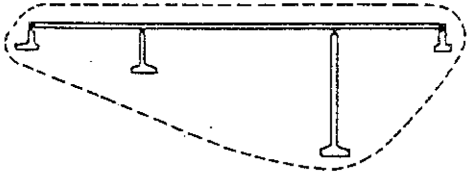
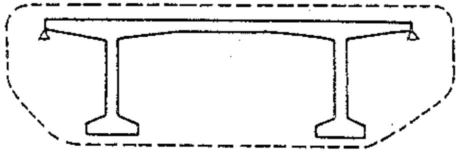
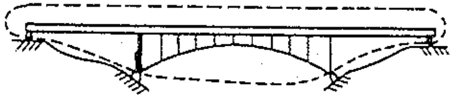
<table>
<tr><td></td><td colspan="3">Longitudinal</td><td colspan="3">Transverse</td><td>Seismic Design Unit</td></tr>
<tr><td>Simple Girder</td><td colspan="3"> </td><td colspan="3"> </td><td rowspan="2">Regarded as A Unit Consisting of A Substructure and A Part of Superstructure Supported by the Substructure</td></tr>
<tr><td rowspan="2">Continuous Girder</td><td rowspan="2">Support Condition in Longitudinal Direction</td><td>Fixed at One Support</td><td> </td><td rowspan="2">Difference of Natural Period between Piers</td><td>Small</td><td> </td></tr>
<tr><td>Fixed at Multiple Supports</td><td> </td><td>Large</td><td></td><td rowspan="2">Regarded as A Unit Consisting of Substructures and the Superstructure Supported by the Substructures</td></tr>
<tr><td>Arch, Frame and Others</td><td colspan="3"></td><td colspan="3"></td></tr>
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Table 9 Structual Response Factor c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leqq 1.4$ $c_R = 0.7$		$1.4 < T_{EQ}$ $c_R = 0.876 T_{EQ}^{-2/3}$
Group II	$T_{EQ} < 0.18$ $c_R = 1.51 \ T_{EQ}^{1/3} \geqq 0.7$	$0.18 \leqq T_{EQ} \leqq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16 T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51 T_{EQ}^{1/3} \geqq 0.7$	$0.29 \leqq T_{EQ} \leqq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59 T_{EQ}^{-2/3}$

Table 10 Structual Response Spectral Value S_o

Ground Condition	S_o (gal)		
Group I	$T_i < 0.1$ $S_o = 431 T_i^{1/3} \geqq 160$	$0.1 \leqq T_i \leqq 1.1$ $S_o = 200$	$1.1 < T_i$ $S_o = 220/T$
Group II	$T_i < 0.2$ $S_o = 427 T_i^{1/3} \geqq 200$	$0.2 \leqq T_i \leqq 1.3$ $S_o = 250$	$1.3 < T_i$ $S_o = 325/T_i$
Group III	$T_i < 0.34$ $S_o = 430 T_i^{1/3} \geqq 240$	$0.34 \leqq T_i \leqq 1.5$ $S_o = 300$	$1.5 < T_i$ $S_o = 450/T_i$

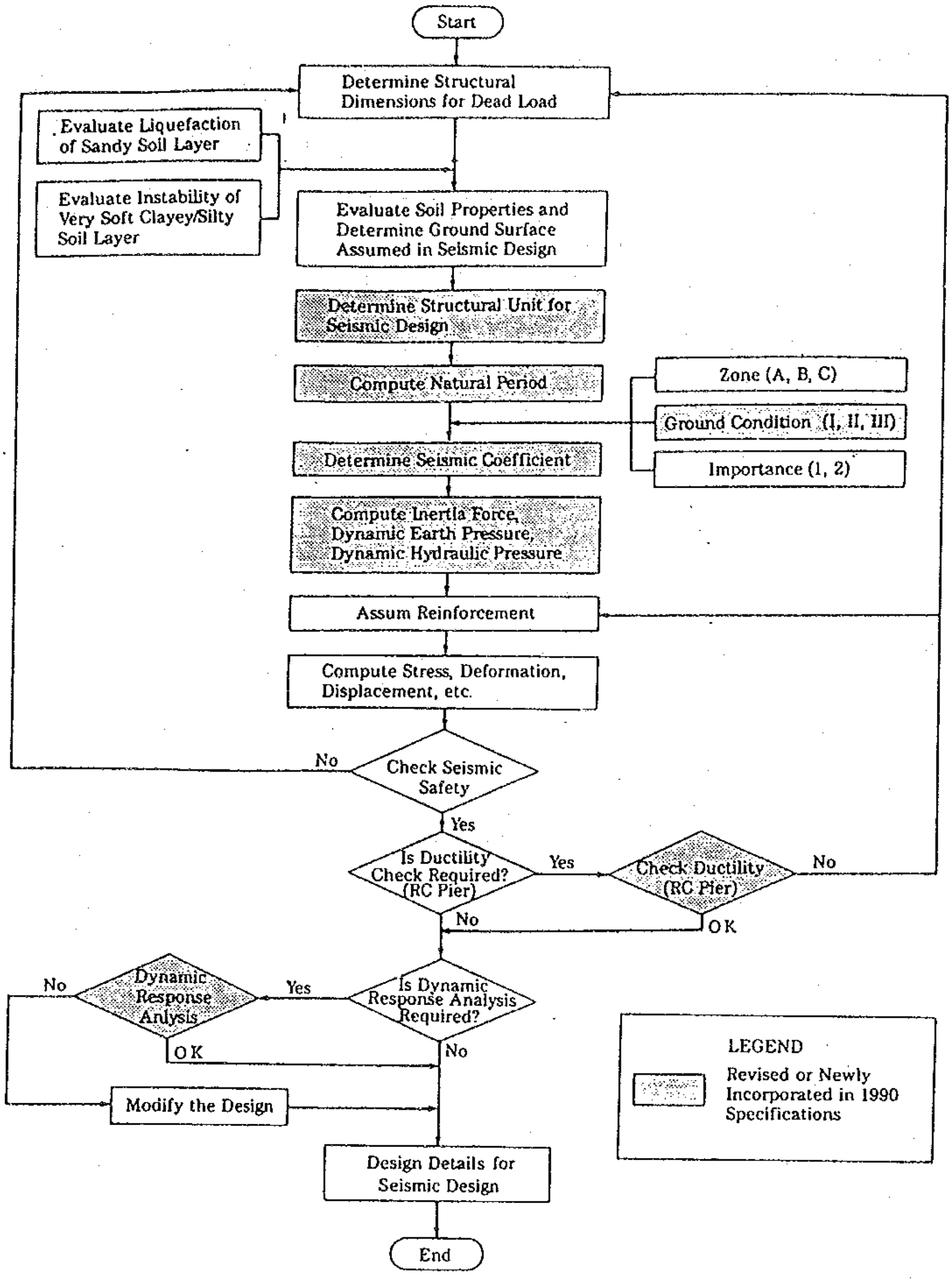


Fig.1 Flow of Seismic Design of Highway Bridge

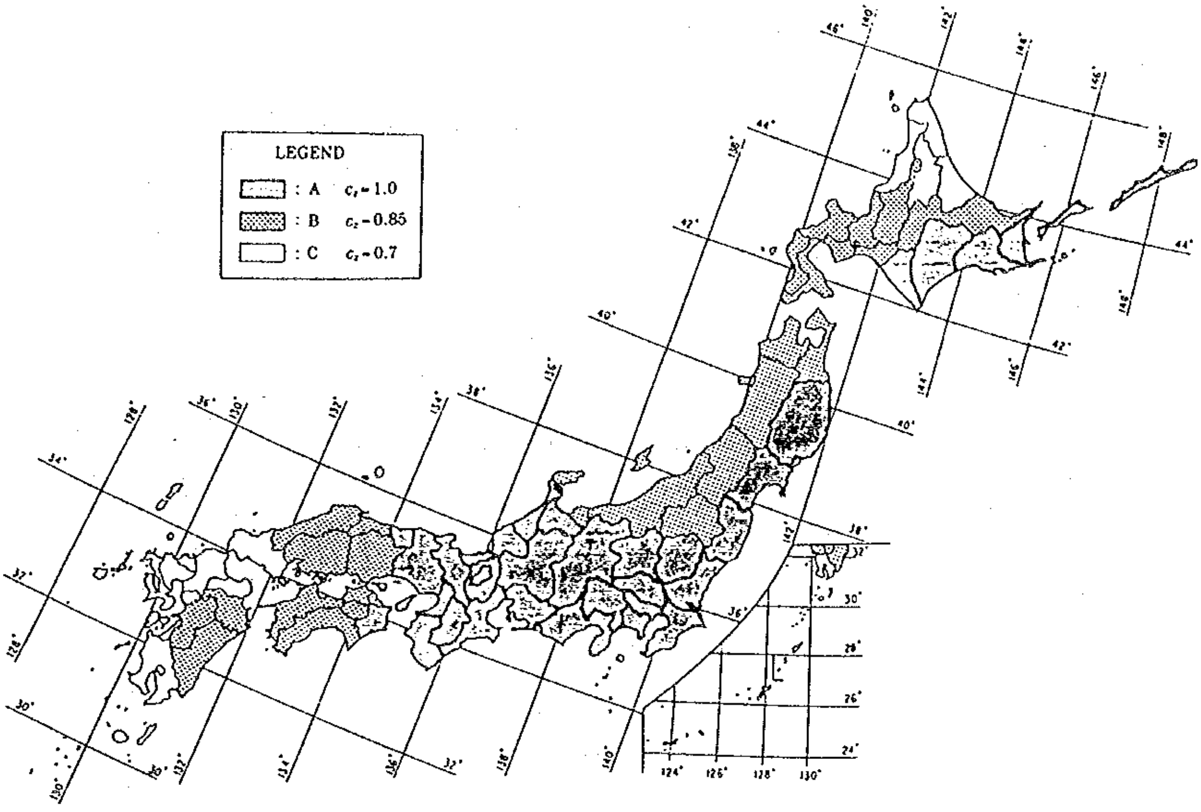
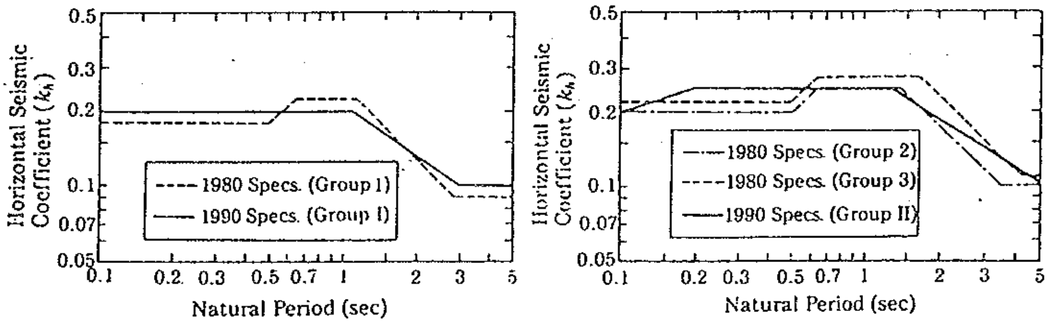
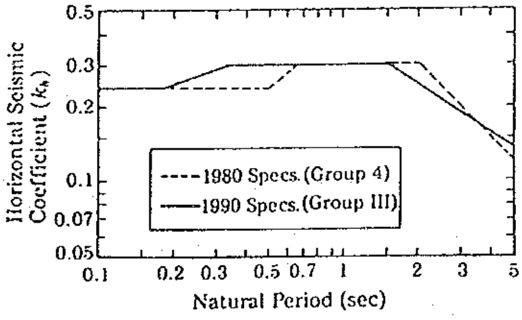


Fig.2 Seismic Zoning Map and Modification Coefficient c_z



(a) Group I

(b) Group II



(c) Group III

Fig.3 Comparison of Seismic Coefficient between 1980 and 1990 Specifications

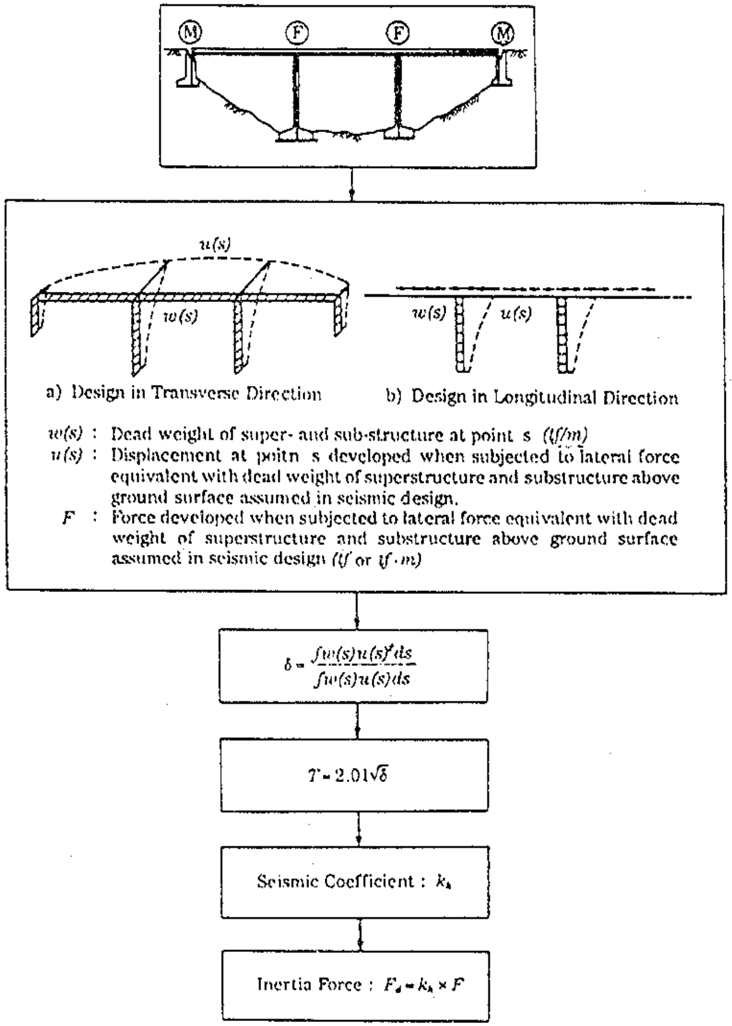


Fig.4 Determination of Inertia Force

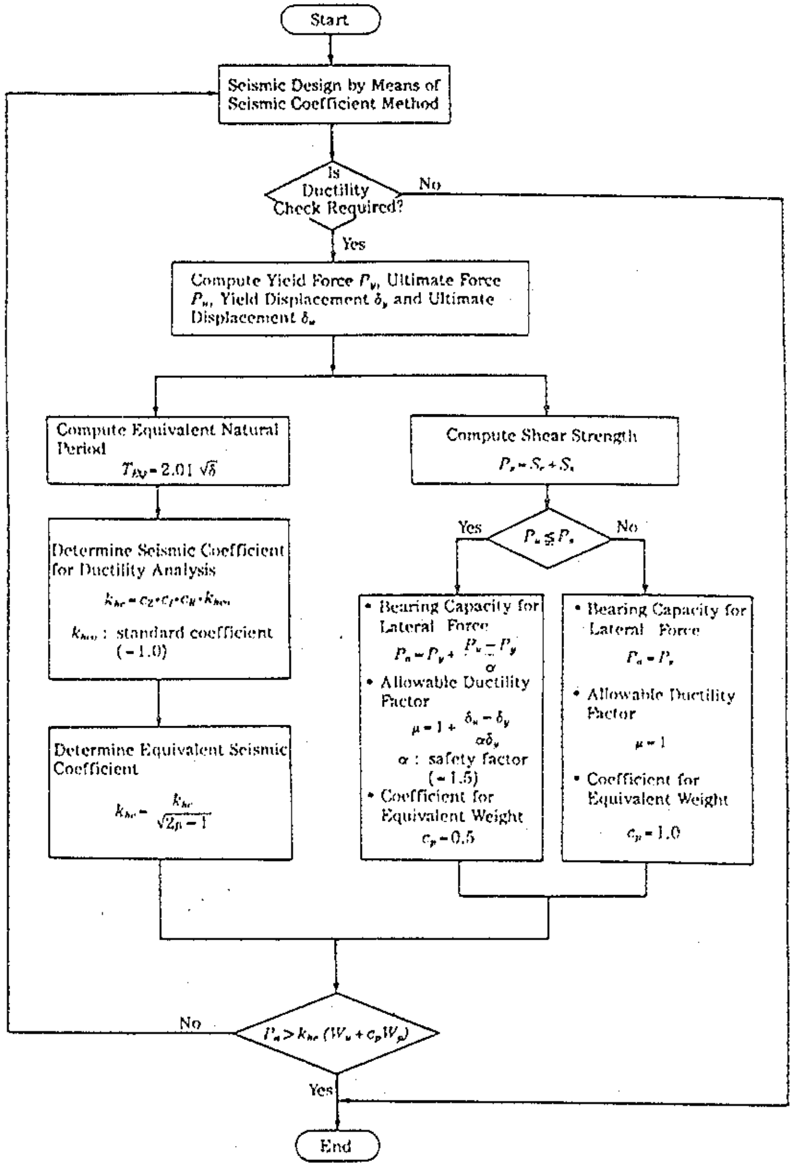


Fig.5 Check of Bearing Capacity of Reinforced Concrete Pier for Lateral Force

ANNEXE 7.6.6.1

エアーリフト設備概要図

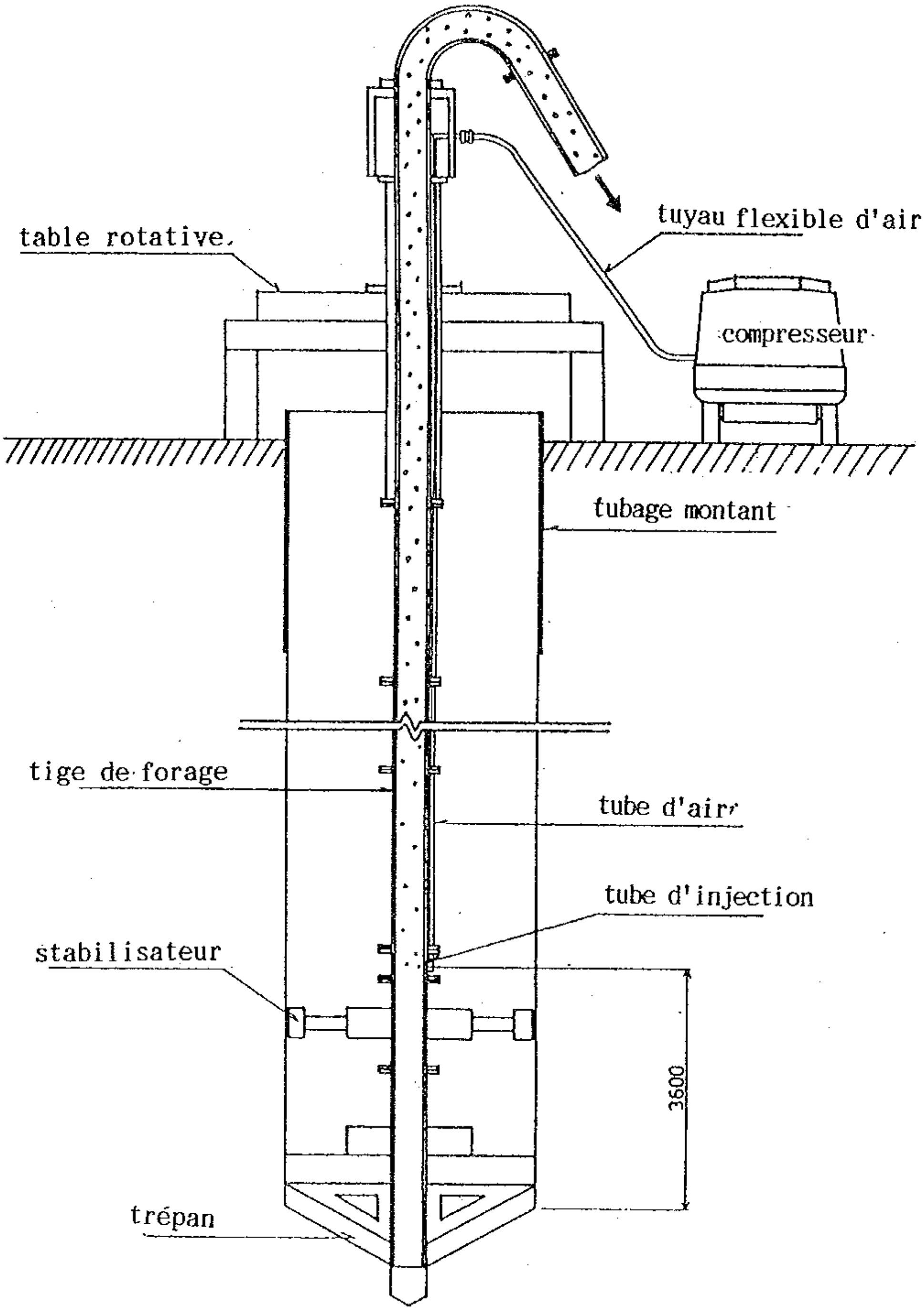
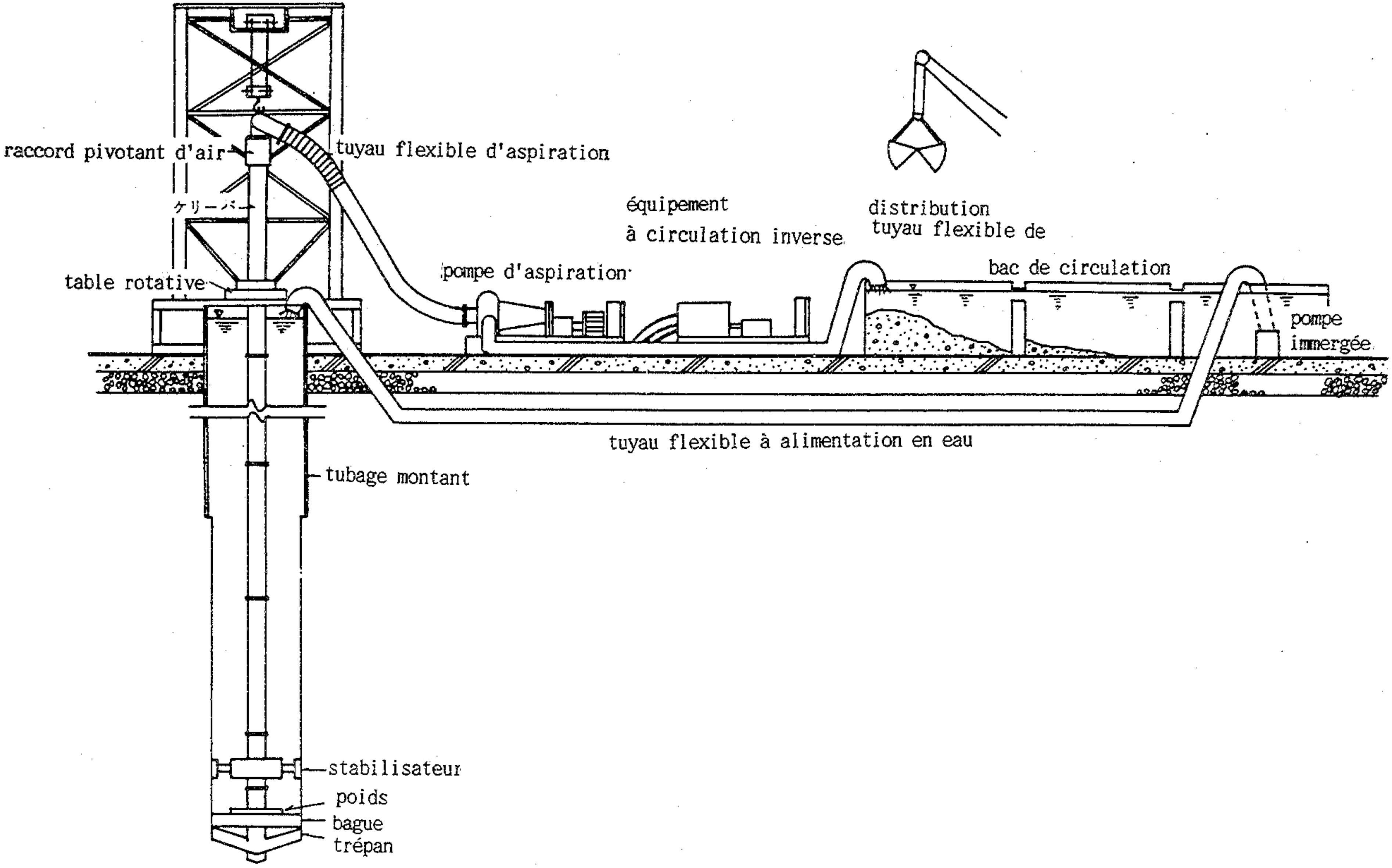


table rotative
tuyau flexible d'air
compresseur
tubage montant
tige de forage
tube d'air
tube d'injection
stabilisateur
3600
trépan

ANNEXE 7.6.6.2

リバース工法施工要領図


raccord pivotant d'air
tuyau flexible d'aspiration
ケリーバー
table rotative
pompe d'aspiration
équipement
à circulation inverse
distribution
tuyau flexible de
bac de circulation
pompe
immergée
tuyau flexible à alimentation en eau
tubage montant
stabilisateur
poids
bague
trépan

7.6.6.2-1

ANNEXE 8.2.1.1

工事費算出根拠

Répartition en parts en devise, en monnaie locale et impôts.

(1) Matériaux à importer par l'entrepreneur

Câbles de précontrainte, haubans, ancrages de précontrainte, poutres en H, toles d'acier. etc....

Prix CAF x Impôts 55% = Prix local

Part en devise: Part en monnaie locale: Part d'impôts:

(2) Matériaux en cours dans le marché tunisien (ceux qui ont été importés par les sociétés dépendantes à l'Administration)

Contre-plaqués, adjuvants, asphaltes, bois, etc...

Prix de base x Coefficient de régulation = Prix de marché

Note) Le prix de base correspond à la valeur CAF qui constitue la part en devise, alors que le coefficient de régulation correspond à la part en monnaie locale dont la commission, les impôts, etc.

Part en devise: 100/151 = 0,66 (66%)

Part en monnaie locale: 51/151 = 0,34 (34%)

La T.V.A occupe 17% de la part en monnaie locale de 34%.

(3) Matériaux importés à usage public (ceux qui ont été importés par les sociétés dépendantes de l'Administration)

Prix de base x Coefficient de régulation = Prix de marché

Note) Le prix de base correspond à la valeur CAF qui constitue la part en devise, alors que le coefficient de régulation correspond à la part en monnaie locale dont la commission, les impôts, etc.

Part en devise: 100/151 = 0,66 (66%)

Part en monnaie locale: 51/151 = 0,34 (34%)

La T.V.A occupe 6% de la part en monnaie locale de 34%.

(4) Matériaux produits en Tunisie

Les matériaux produits en Tunisie tels que le ciment, l'armature, etc... sont divisés en frais de l'amortissement pour la mise en service d'une usine, des matériaux et de la gestion du personnel, qui sont chacun subdivisés en pourcentage en parts en devise, en monnaie locale et impôts.

Sur les frais de l'amortissement pour la mise en service d'une usine, les frais de l'amortissement de cette usine (occupant 80% des frais de l'amortissement pour la mise en service de cette usine) constitue sa part en devise puisque l'usine est construite en devise, alors que les frais de l'entretien de cette usine (occupant 20% du frais de l'amortissement pour la mise en service de cette usine) constituent sa part en mannaie locale. Le rapport des parts en devise et en monnaie locale sur les frais des matériaux est déterminé par répartition en ceux d'origine étranger et local. Pour le ciment, le clinker (40% du frais de ciment) est importé et le calcaire (60% de ce frais) est d'orgine locale. Pour l'armature, le coke (40% du frai d'armature) est importé et le minerai (60% de ce frais) est d'orgine locale. Quant aux frais de la gestion du personnel, il sont constitué 100 pourcents en monnaie locale.

Ciment et armature

Rapport sur l'ensemble Part en devise Part en monnaie locale

1. Frais de l'amortissement pour la mise en service d'une usine
2. Frais des matériaux
3. Frais de la gestion du personnel

Note) La T.V.A occupe 17% de la part en monnaie locale de 52%.

(5) Matériaux à fournir en Tunisie

Sables, matériaux de remblai, etc...

Pour ce qui concerne les sables et les matériaux de remblais capables d'être fournis en Tunisie, leurs prix sont divisés en frais du forage, du transport et des droits, et impôts, qui sont chacun subdivisés en pourcentage en parts en devise et en monnaie locale. Sur les frais du forage et du transport, les frais des matériels mécaniques (occupant 75%

des frais indiqués ci-dessus) constituent la part en devise, alors que les frais du personnel (occupant 25% des frais indiqués ci-dessus) constituent la part en monnaie locale. D'autre part, les frais des droits ainsi que les impôts constituent 100% la part en monnaie locale.

Rapport sur l'ensemble Part en devise Part en monnaie locale

1. Frais du forage
2. Frais du transport
3. Frais des droits
4. Impôts (T.V.A)

Note) La T.V.A occupe 17% de la part en monnaie locale de 45%.

Agrégats grossiers

Pour ce qui concerne les rochers concassés et les pierres pour la défense à la mer capables d'être fournis en Tunisie, leurs prix sont divisés en frais du forage et du transport, celui des droits et impôts, qui sont chacun subdivisés en pourcentage en parts en devise et en monnaie locale. Sur les frais du forage et du transport, les frais des matériels mécaniques (occupant 75% des frais indiqués ci-dessus) constituent la part en devise, alors que les frais du personnel (occupant 25% des frais indiqués cidessus) constituent la part en monnaie locale. D'autre part, le frais des droits ainsi que les impôts constituent 100% la part en monnaie locale.

Rapport sur l'ensemble Part en devise Part en monnaie locale

1. Frais du forage et du transport
2. Frais des droits
3. Impôts (T.V.A)

Note) La T.V.A occupe 17% de la part en monnaie locale de 49%.

Béton prémélangé

Rapport sur l'ensemble Part en devise Part en monnaie locale

1. Mélange (y compris matériaux)
2. Frais du transport
3. Impôts (T.V.A)

Note) La T.V.A occupe 17% de la part en monnaie locale de 51%.

(6) Coût des matériels de construction

Les matériels de construction sont en principe à apporter de l'étranger et à remporter après la construction par les entrepreneurs.

Part en devise: 100% du frais de l'endommagement mécanique, 70% des frais de l'entretien et du dépannage réservés aux pièces de rechange et 100% des frais du contrôle mécanique.

Part en monnaie locale: 30% du frais de l'entretien et du dépannage, réservés pour le frais des mécaniciens locaus.

Rapport sur l'ensemble Part en devise Part en monnaie locale

1. Frais de l'endommagement mécanique

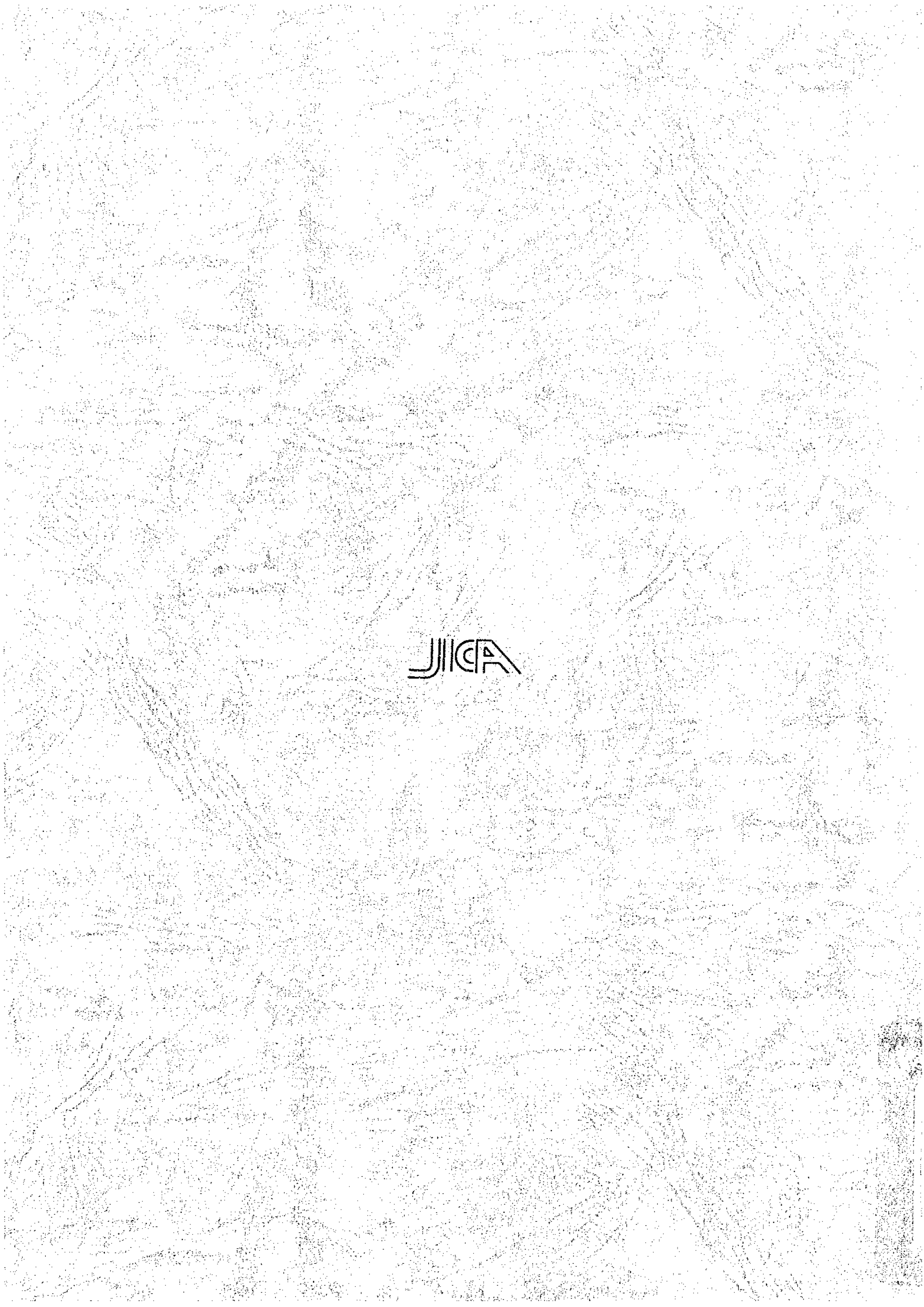
2. Frais de l'entretien et du dépannage

3. Frais du contrôle mécanique

Les impôts (T.V.A) sont nuls (0%) puisque les matériels ont pour principe d'être remportés à l'étranger.

(7) Frais du personnel local

Ils constitue 100% la part en monnaie locale, dont 17% sont occupés par la T.V.A.



JICA