

ANNEXE 6.3.4.2

PRIX UNITAIRES APPLIQUES
POUR L'ESTIMATION DU COUT DU PROJET

(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) Pieux en béton coulé sur place

Diamètre de pieu	Par mètre d'autres pieux de diamètre (T Φ /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

RESULTATS DE L'ETUDE GEOTECHNIQUE

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

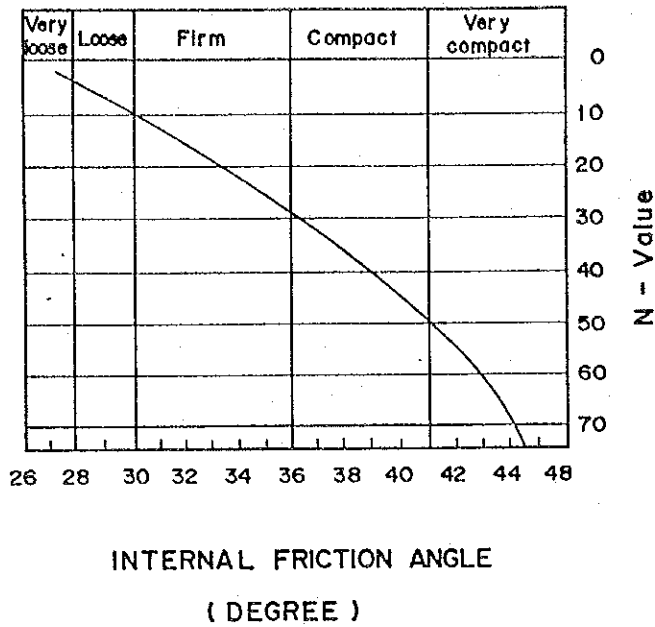


Fig. 2 RELATIVE CHART
FOR LOAD AND VOID RATIO
OF SANDY SOIL

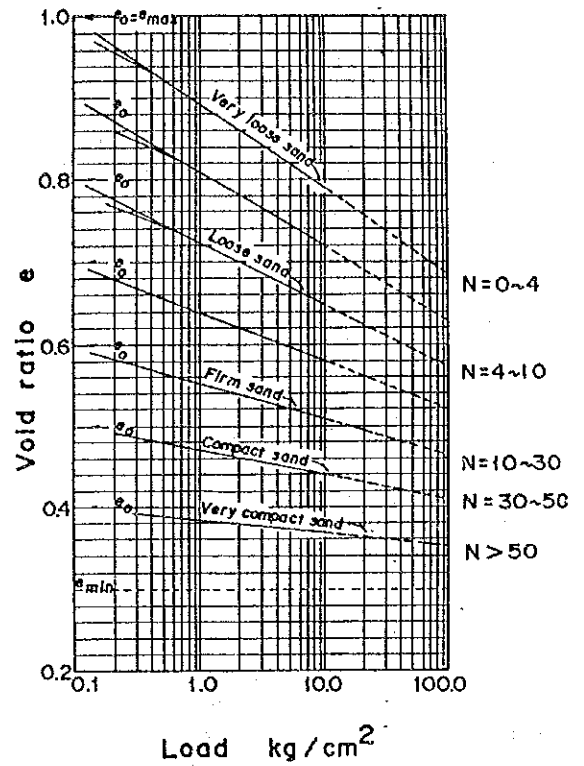


TABLE 1 WET DENSITY OF SOIL

Soil		Condition of Soil	Wet density(t/m ³)	Symbols	
Embankment	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		

VARIANTE DE RACCORDEMENT N°1

Location map of required survey section.

33000

ZONE SUD-EST Lotissement C

AMENAGEMENT AC DE TUNIS

Variante Aves H=1.5m R=20m

Embankment section

Embankment section

Bridge section

Embankment section

Embankment section

Bridge section EXTENSION FUTURE DU PORT DE RADES(Phase 2)

Voie de desserte du port à devier

Chenal pour extension du port

profondeur prévue -7.50 CV -11

ouvrage pour H=30m

ouvrage pour H=15m

ouvrage pour H=4.5m

WEST ROUTE

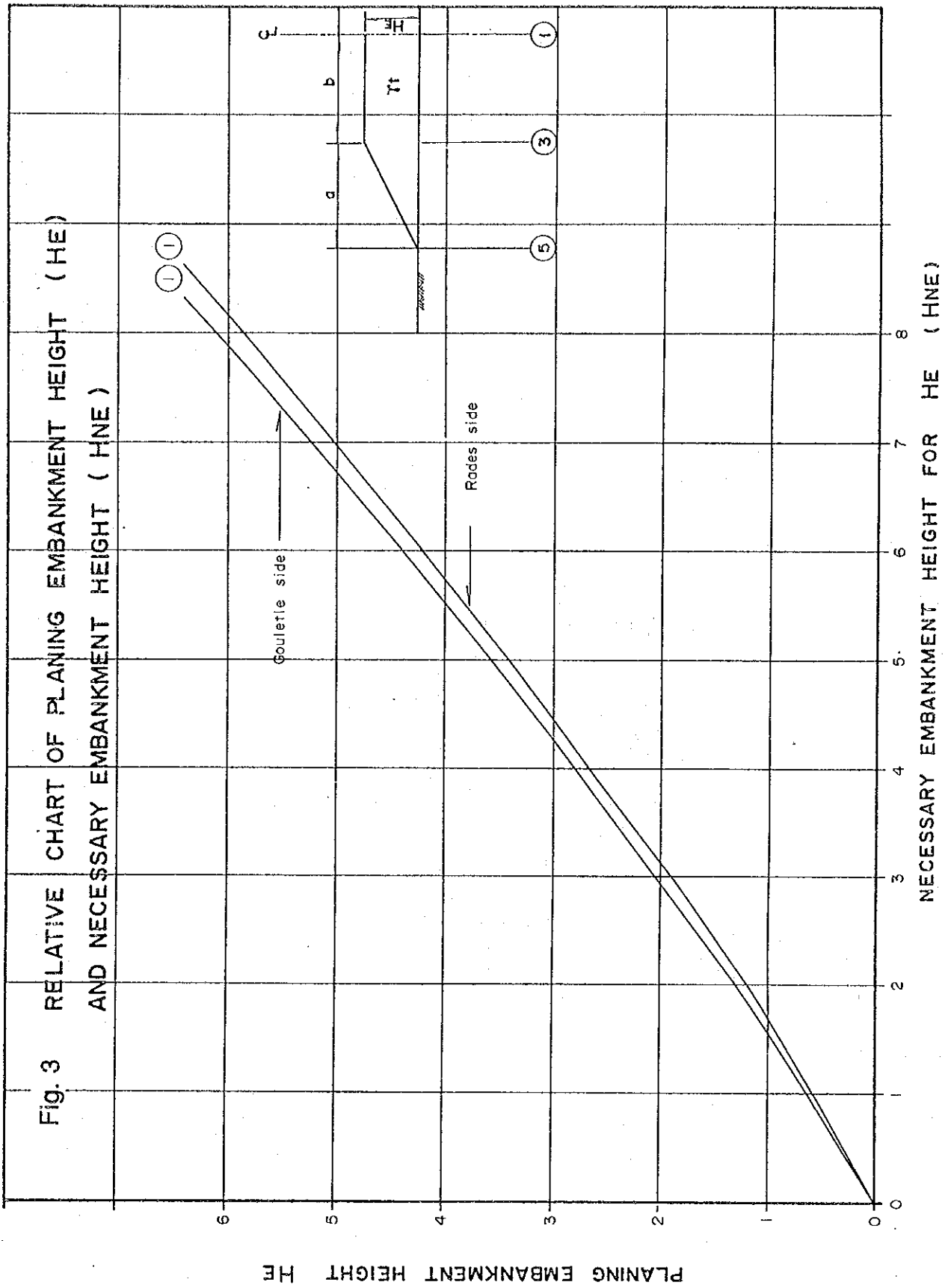


Fig. 4 RELATIVE CHART FOR INTERVAL OF SAND DRAIN AND REMAINING SUBSIDED VALUE

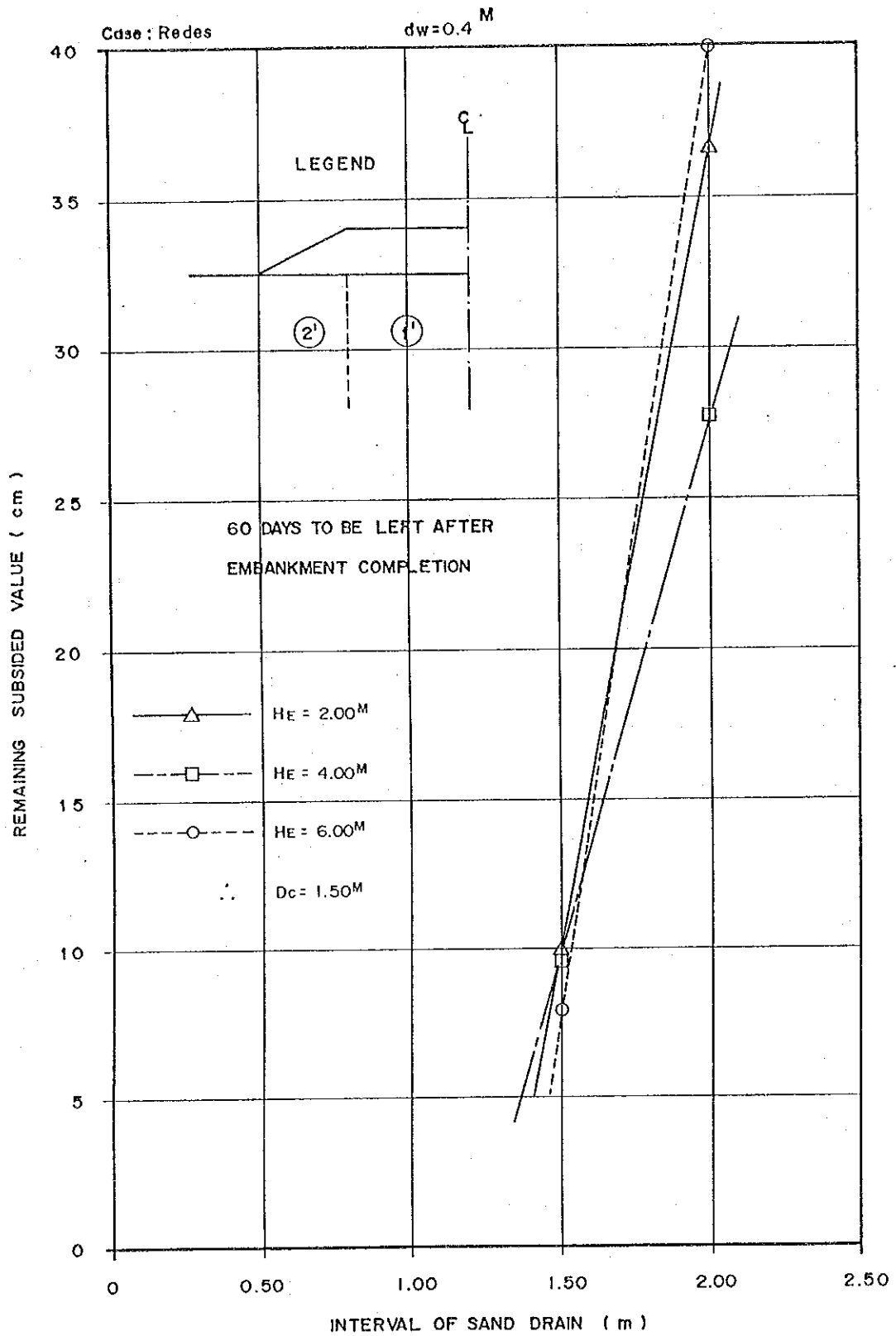


Fig. 5 RELATIVE CHART FOR INTERVAL OF SAND DRAIN AND REMAINING SUBSIDED VALUE

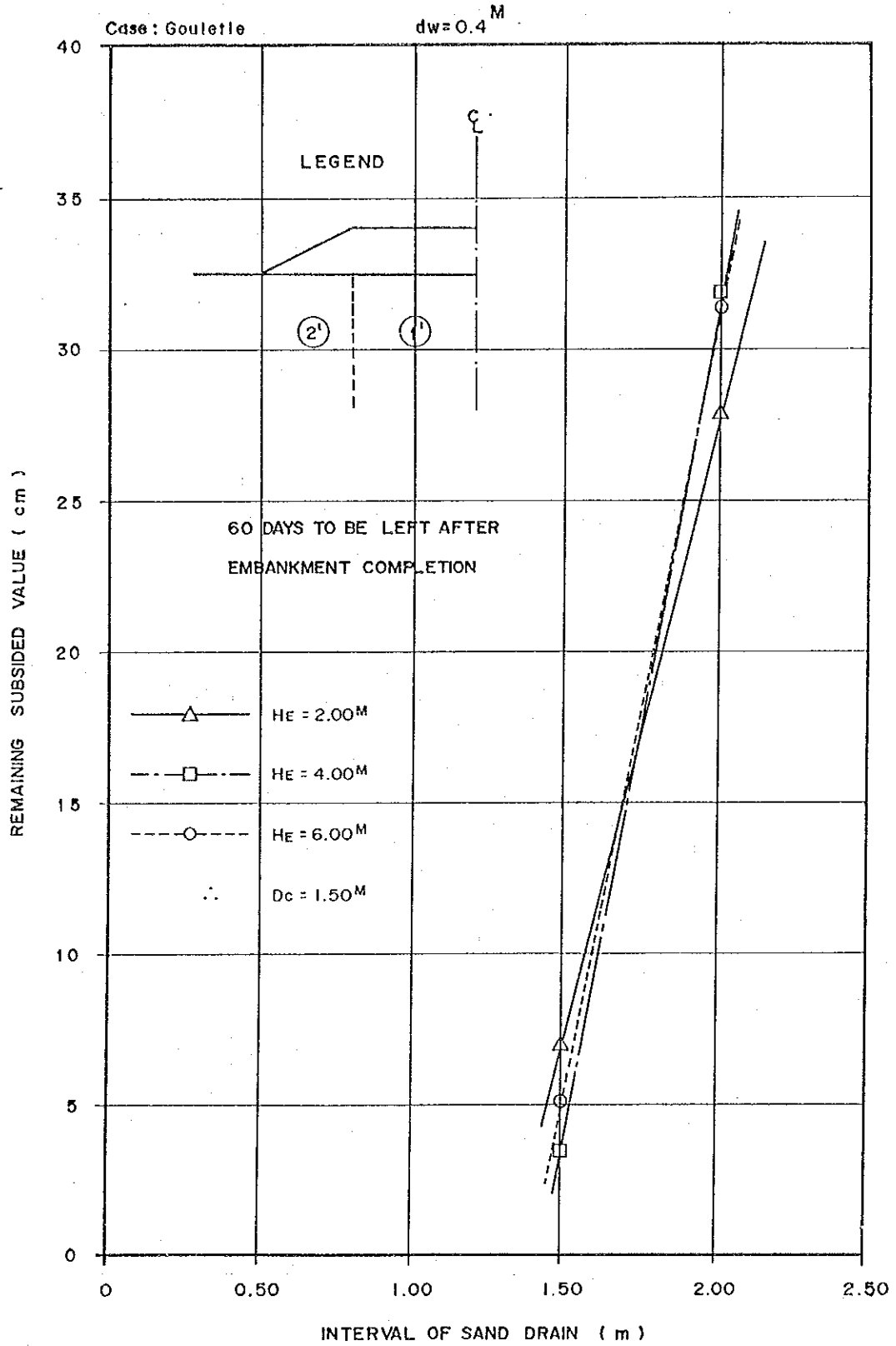


Fig. 6 RELATIVE CHART OF EMBANKMENT HEIGHT AND SAFETY FACTOR

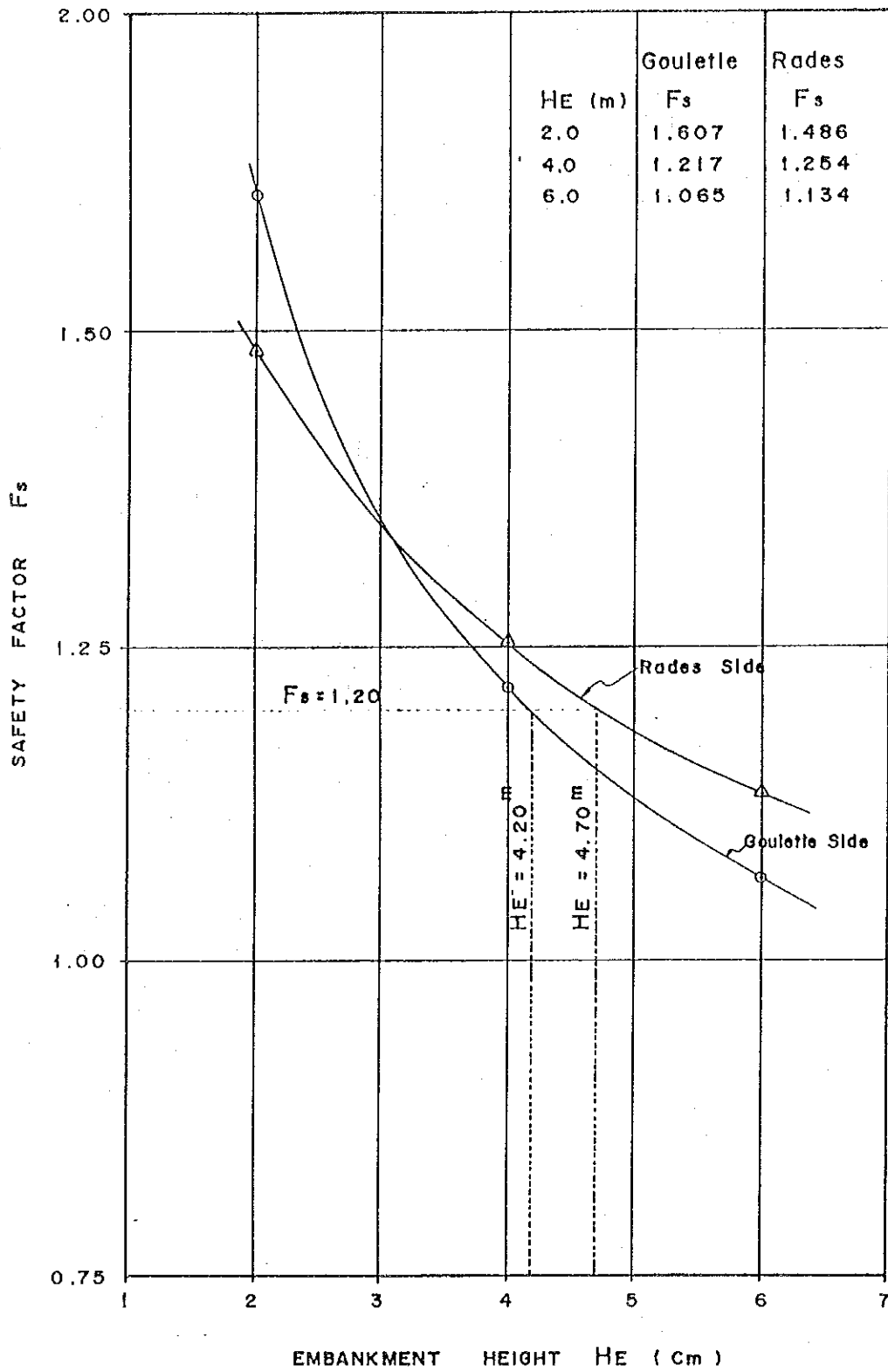


TABLE 2 WORK ITEMS AND QUANTITIES OF SOILS AND MATERIALS INVESTIGATIONS

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 TEST	OTHER TEST	SPECIFIC GRAVITY	NATURAL WATER CONTENT	PARTICLE SIZE GRAVIMETRY TEST	LIQUID LIMIT TEST	PLASTIC LIMIT TEST	UNCONFINDED COMPRESSION TEST	TRIAXIAL COMPRESSION TEST	WET DENSITY	UNCONFINDED COMPRESSION TEST		TRIAxIAL COMPRESSION TEST	CONSOLIDATION TEST
↓ 2	SC. 1	Rades	φ = 101mm	1.03	120.10	5	5	-	10	5	10	5	5	-	5	5	5	5	5	2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
↓ 2	SC. 2	Goulette	φ = 101mm	0.64	110.30	4	5	-	9	5	9	5	5	-	5	5	5	5	5	2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
	Sub total				230.40	9	10	-	19	10	19	10	10	-	10	10	10	10	10	2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
	SP. 1	Rades	φ = 62mm	1.03	105	-	-	86												2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
	SP. 2	Goulette	φ = 62mm	1.62	103	-	6	76												2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
	Sub total				210	-	6	162												2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
	TOTAL				230.40	9	10	162	19	10	19	10	10	-	10	10	10	10	10	2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
					210.00			6												2/ Laboratory tests performed in the State of Louisiana. The results of these tests are subject to the same conditions as those stated in the report.
PURPOSE OF INVESTIGATION	FIELD WORKS					LABORATORY TESTING												NOTES		
	NAME OF MATERIALS	TEST PIT NO.	NAME OF LOCALITY	GROUND ELEVATION OR SAMPLING DEPTH	FIELD CBR TEST	DENSITY OF SOIL IN PLACE	SPECIFIC GRAVITY	NATURAL WATER CONTENT	PARTICLE SIZE GRAVIMETRY TEST	LIQUID LIMIT TEST	PLASTIC LIMIT TEST	UNCONFINDED COMPRESSION TEST	TRIAXIAL COMPRESSION TEST	APPARENT SPECIFIC GRAVITY	ABSORPTION	GRAIN SIZE ANALYSIS	ABRASION RATIO			
5/	Embank material	1	Di Resses	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	G _s	2,655	2,745	2,742	2,738	2,745			
NATURAL STATE	WATER CONTENT	w _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	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	C _u	C _u	uu	uu	C _u			
	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	w _{opt.} (%)							
	MAXIMUM DRY DENSITY	γ _{dmax.} (g/cm ³)							
C B R	SAMPLE CONDITION (%)								
	TEST CONDITION								
	WATER CONTENT	w (%)							
	DRY DENSITY	γ _d (g/cm ³)							
	C B R (%)								
REMARKS									

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT		Feasibility study on Crossing Construction between Bades and la Goulette			SOILS & MATERIALS ENGINEER 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 w/n (%)	83	39	25	18	33	
	WET DENSITY γ_t (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 S_t	-	-	-	-	-	
TRIAXIAL COMPRESSION	TYPE OF TEST	C_u	C_u	uu	uu	uu	
	COHESION C (kg/cm ²)	0.125	0.200	0.360	0.800	0.320	
	ANGLE OF INTERNAL FRICTION ϕ°	19	15	2	6	5	
CONSOLIDATION	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	
	C_v (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 $w_{opt.}$ (%)						
	MAXIMUM DRY DENSITY $\gamma_{dmax.}$ (g/cm ³)						
C B R	SAMPLE CONDITION (%)						
	TEST CONDITION						
	WATER CONTENT w (%)						
	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-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
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 G _s		2,711	2,717	2,694	2,686	2,703
NATURAL STATE	WATER CONTENT w _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°					
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 γ _d (g/cm ³)					
	C B R (%)					
REMARKS						

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT			Feasibility study on Crossing Construction between Rades and Ia 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 G _s			2,661	2,679	2,675	2,699	
NATURAL STATE	WATER CONTENT ω (%)						
	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°						
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 γ _d (g/cm ³)						
	C B R (%)						
REMARKS							

SUMMARY OF LABORATORY TEST OF SOILS

PROJECT		Feasibility study on Crossing Construction between Rades and Ia 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 G_s		2,658	2,704			
NATURAL STATE	WATER CONTENT w/n (%)	4.0	3.0			
	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°					
CONSOLIDATION	YIELD STRESS OF CONSOLIDATION P_y (kg/cm ²)					
	COMPRESSION INDEX C_c					
COMPACTION	METHOD OF TEST					
	OPTIMUM MOISTURE CONTENT w_{opt} (%)	10	8			
	MAXIMUM DRY DENSITY γ_{dmax} (g/cm ³)	1.87	2.06			
C B R	SAMPLE CONDITION (%)	Disturbed	Disturbed			
	TEST CONDITION	Soaked	Soaked			
	WATER CONTENT w (%)	16.0	8.0			
	DRY DENSITY γ_d (g/cm ³)	1,777	1,957			
	C B R (%)	14	10			
REMARKS						

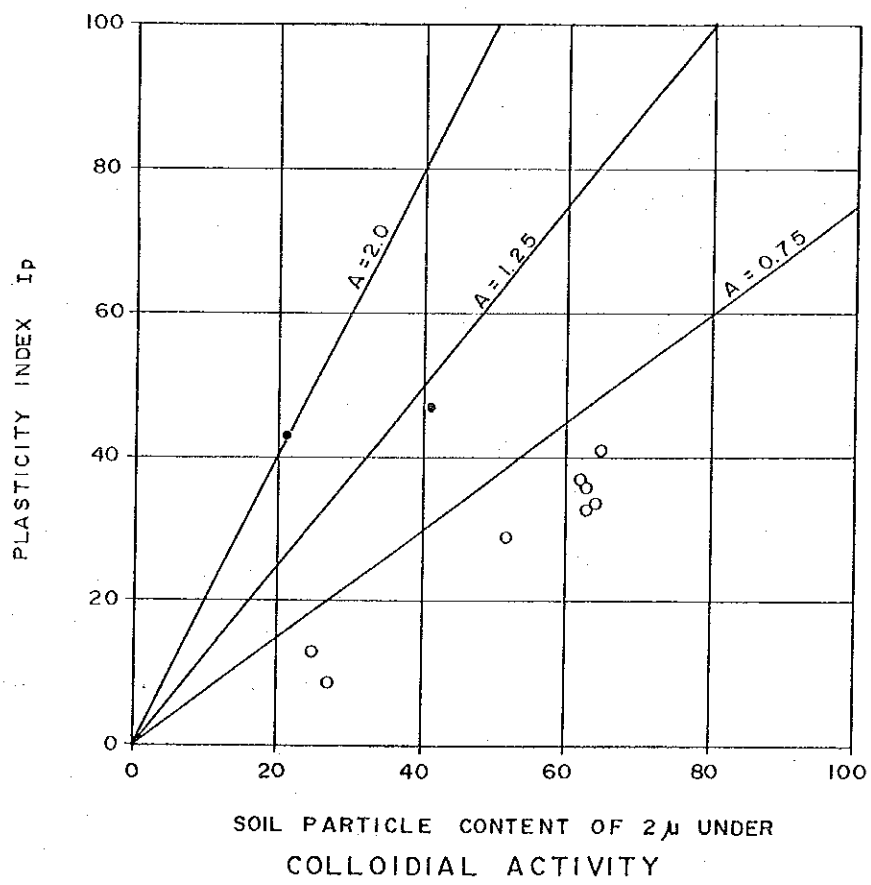
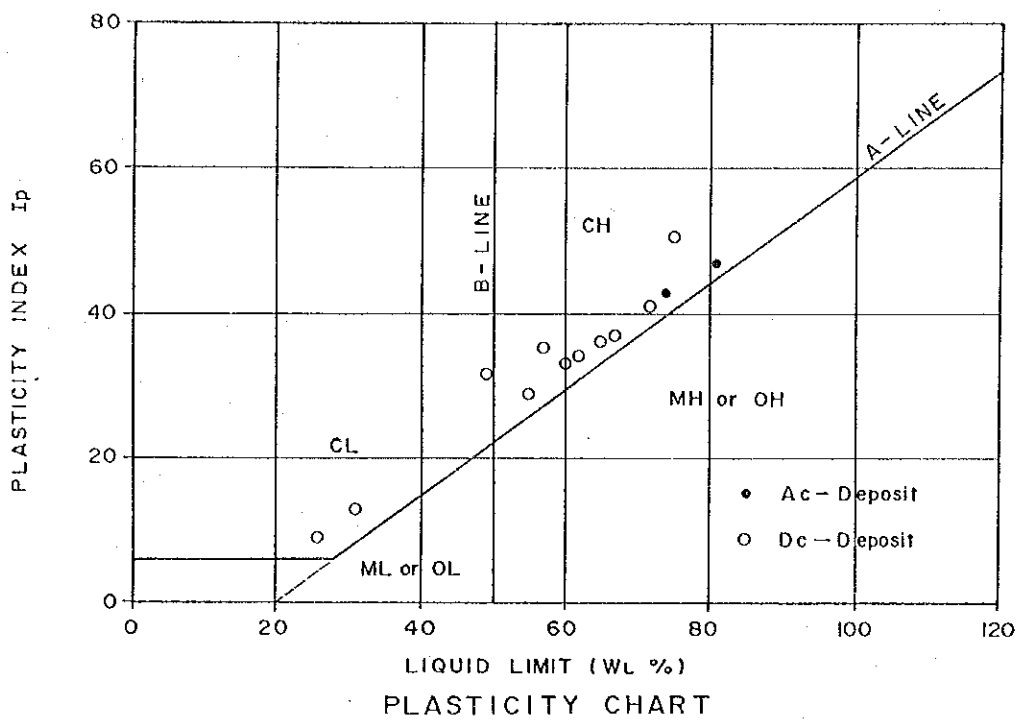


Fig. 7

CONSISTENCY CHART

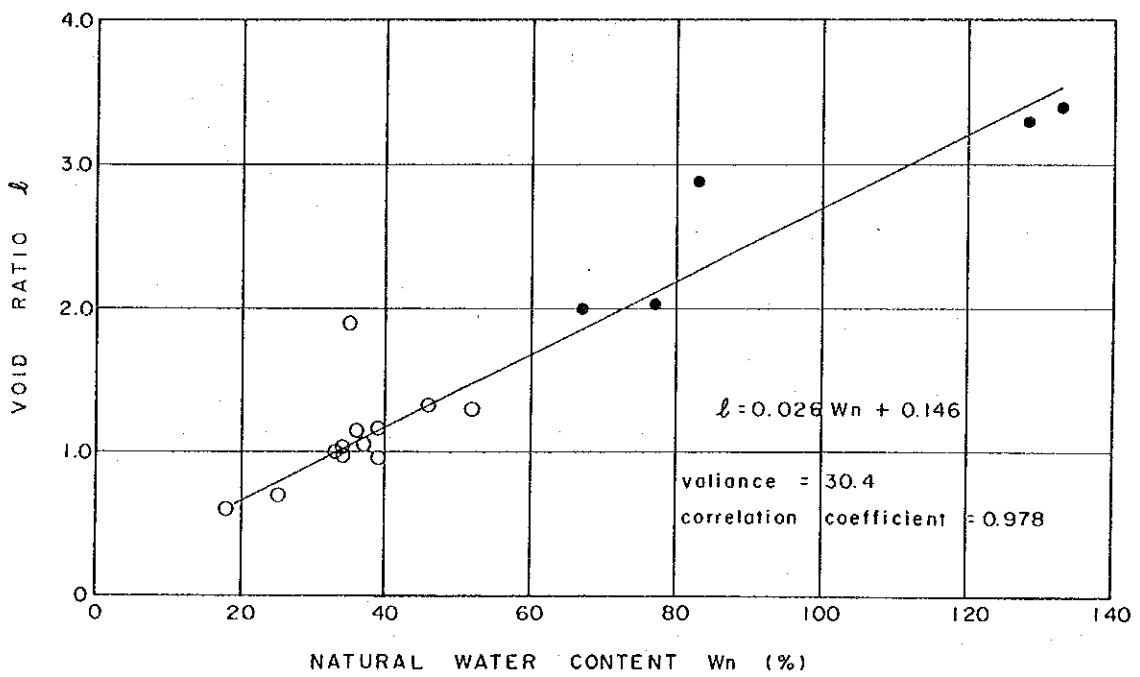
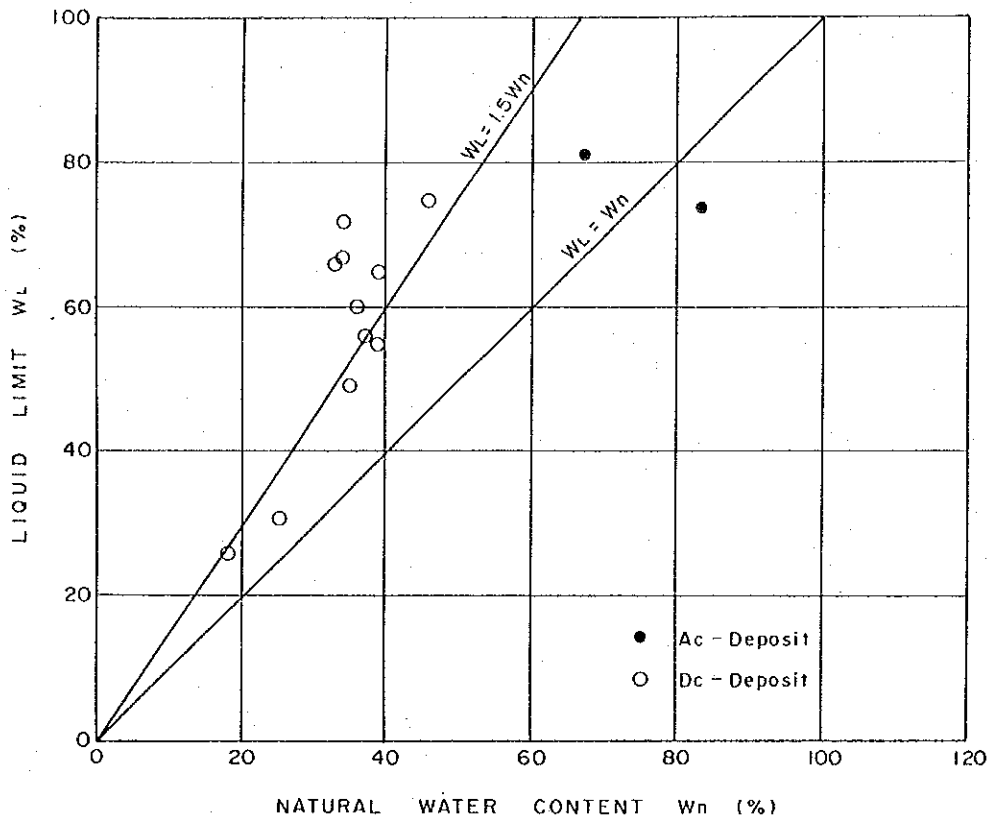


Fig. 8

RELATIVE CHART OF W_n AND W_L , e

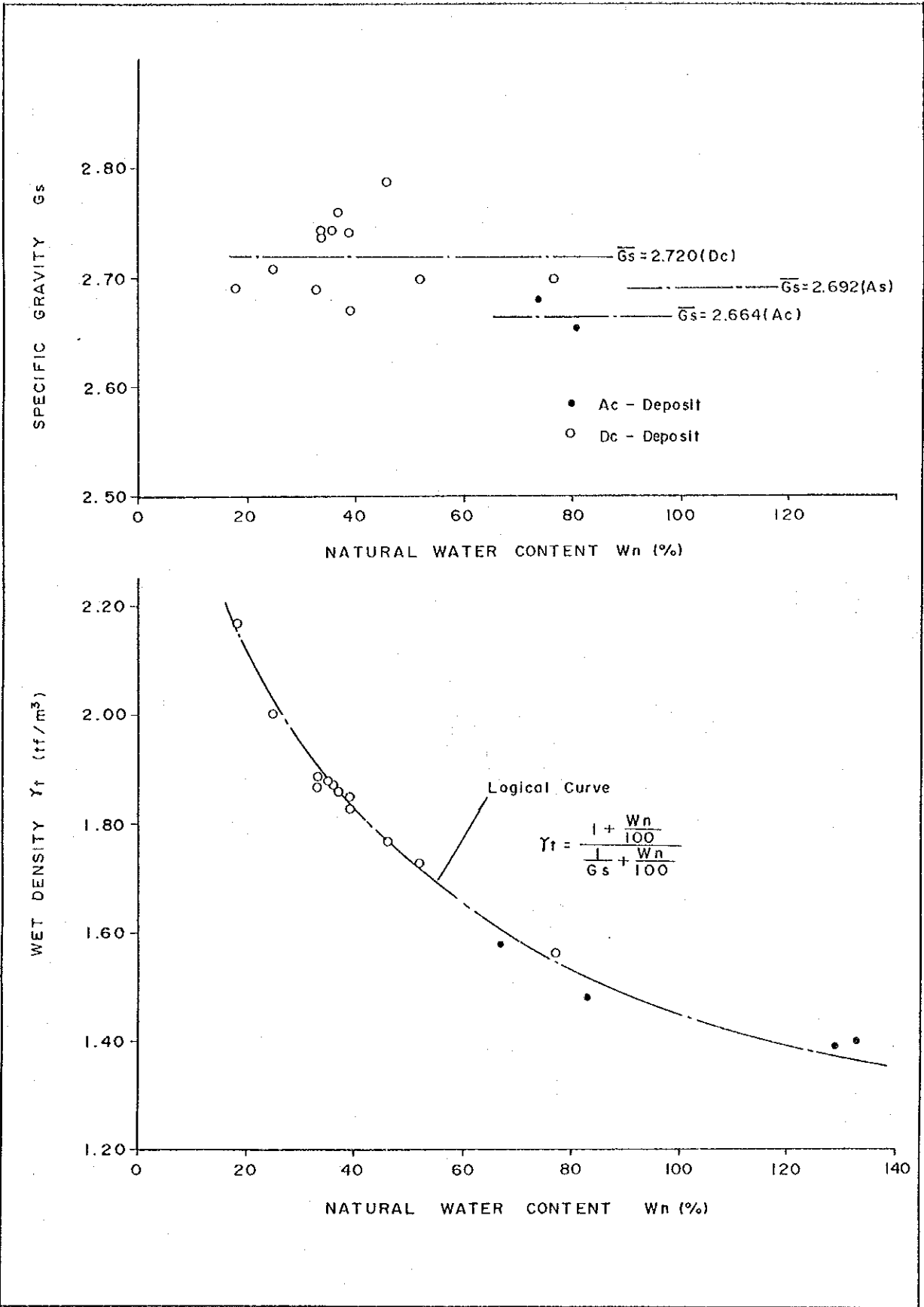


Fig. 9

RELATIVE CHART OF W_n AND G_s , γ_t

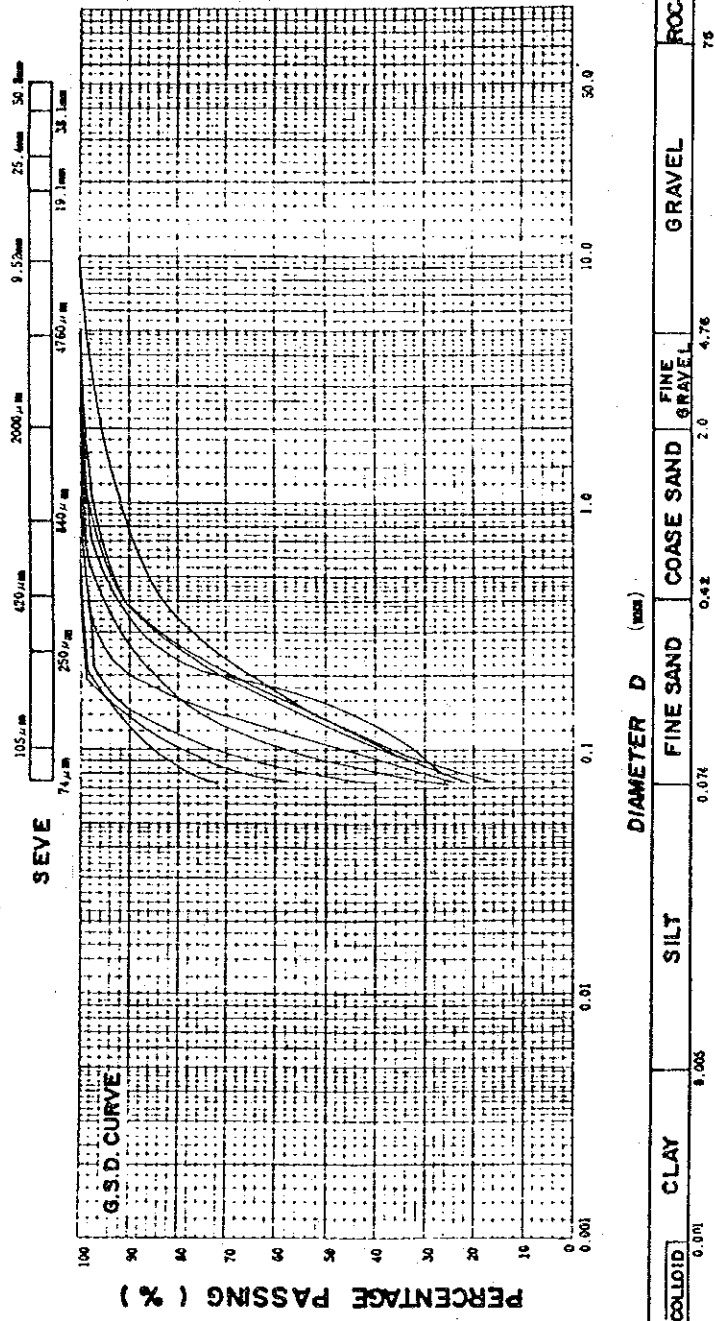
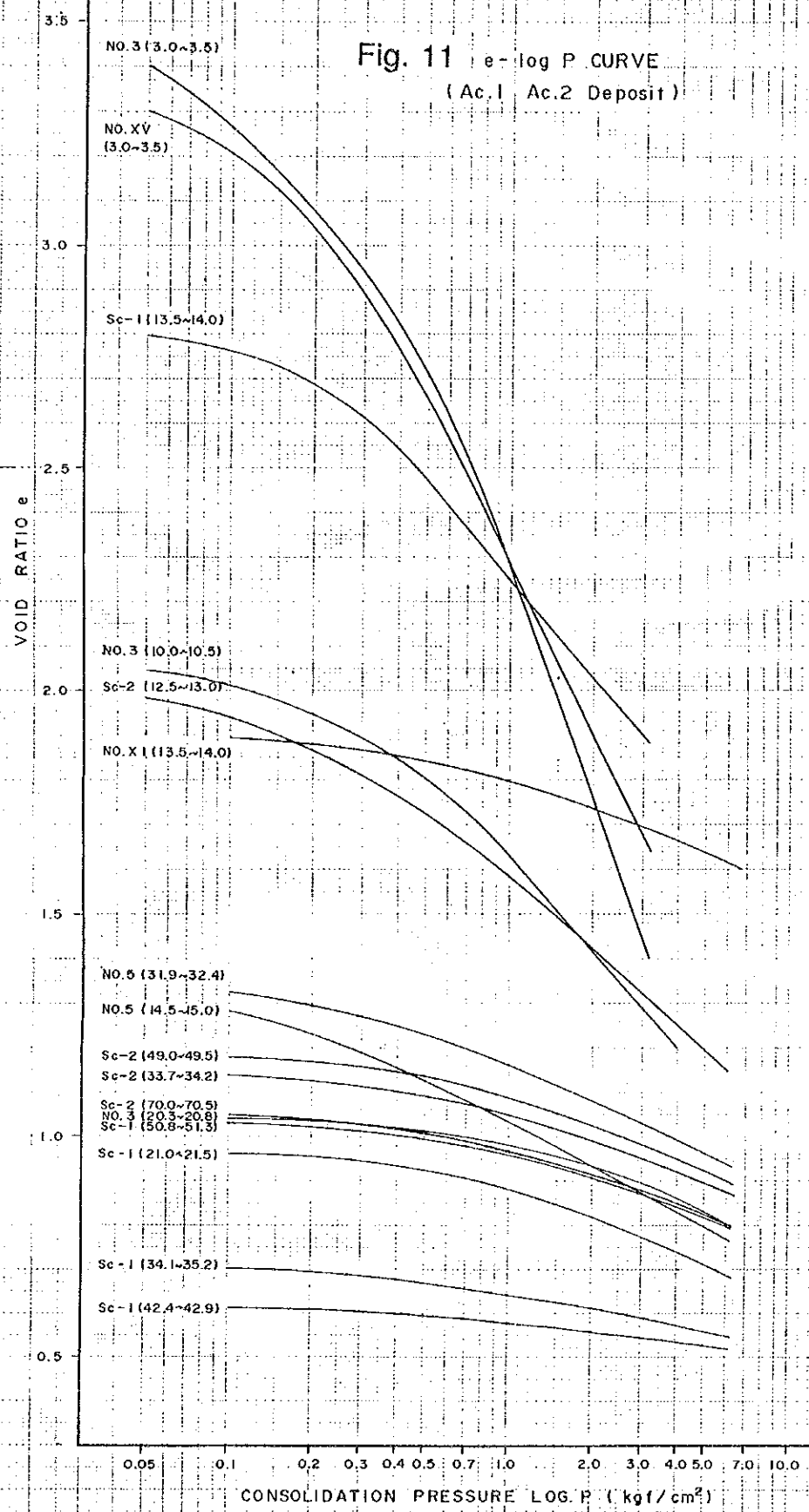


Fig. 10 GRAIN SIZE DISTRIBUTION CURVE (As-deposit)

Ac. 1 Deposit

Ac. 2 Deposit

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



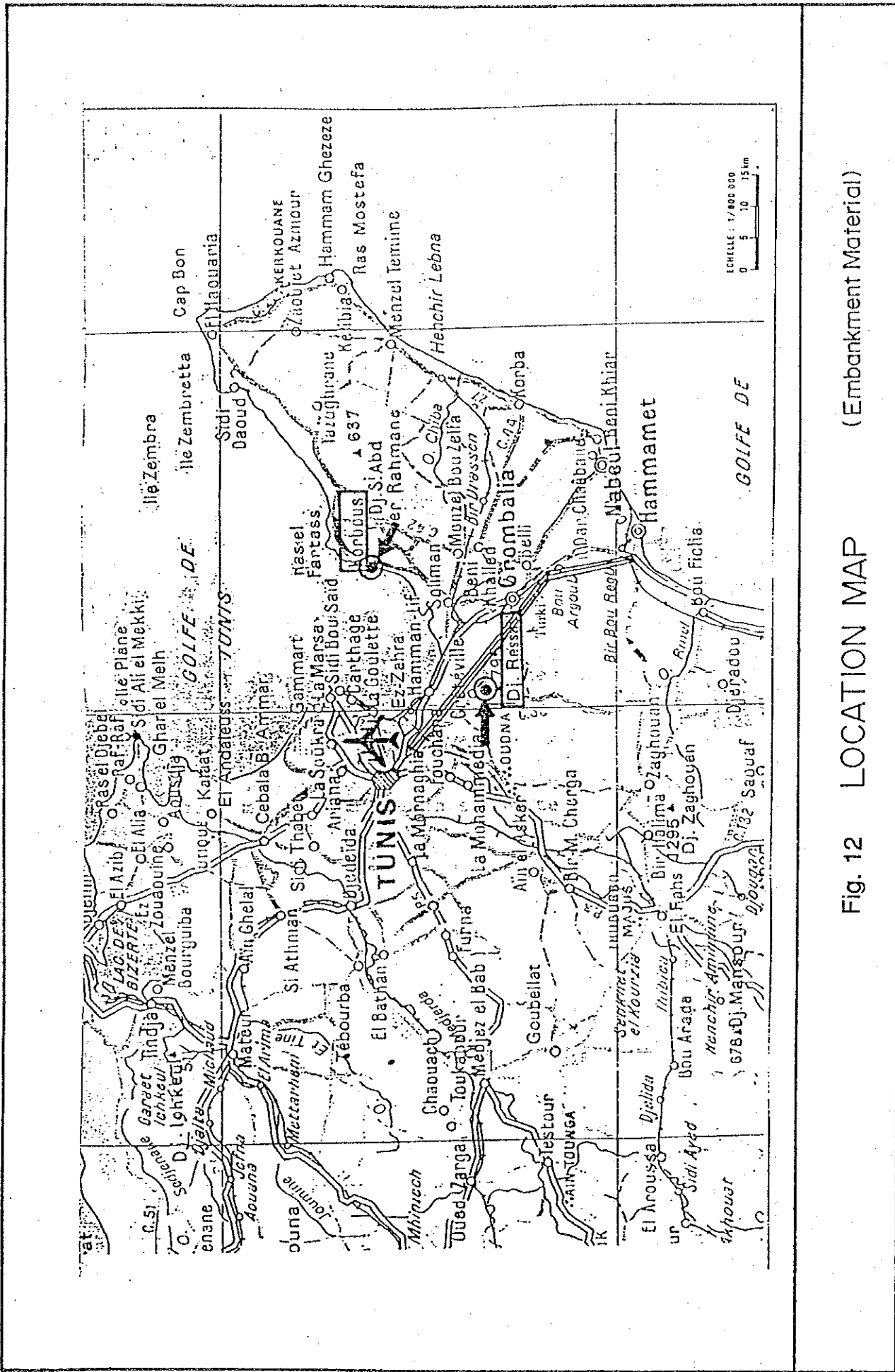


Fig. 12 LOCATION MAP (Embankment Material)

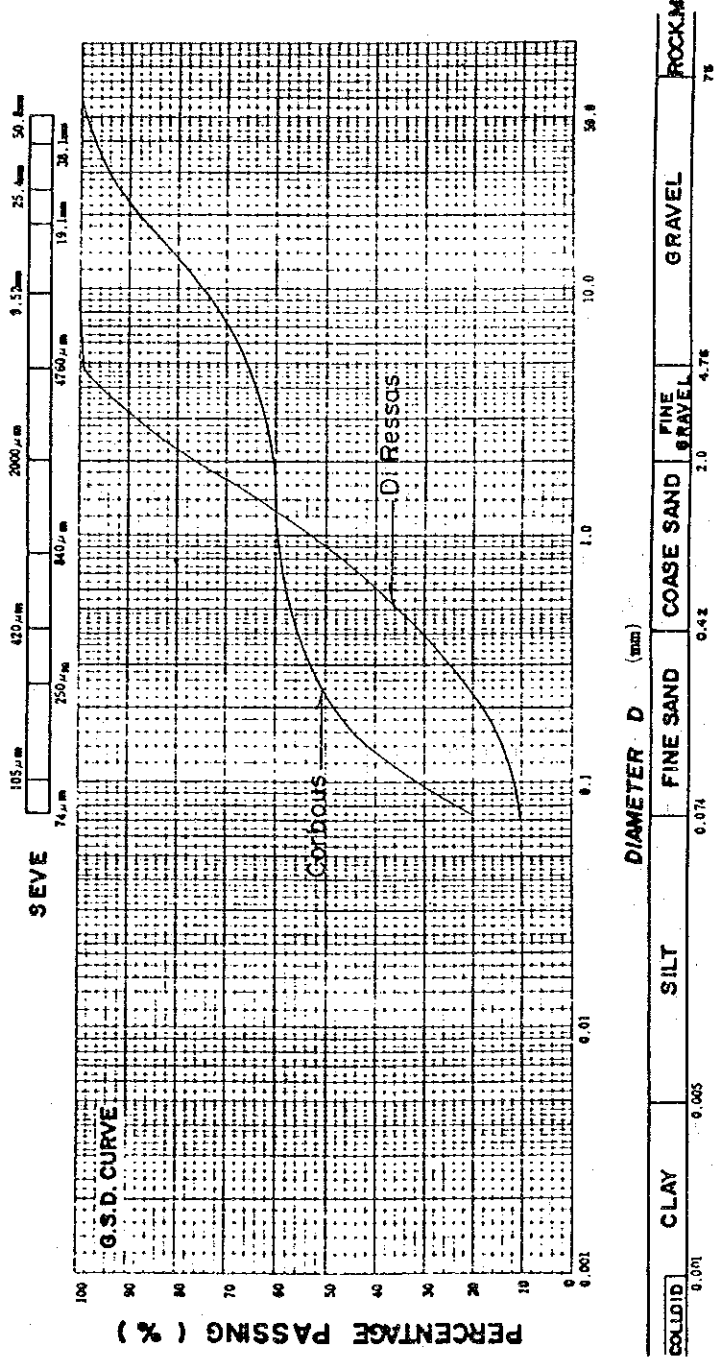
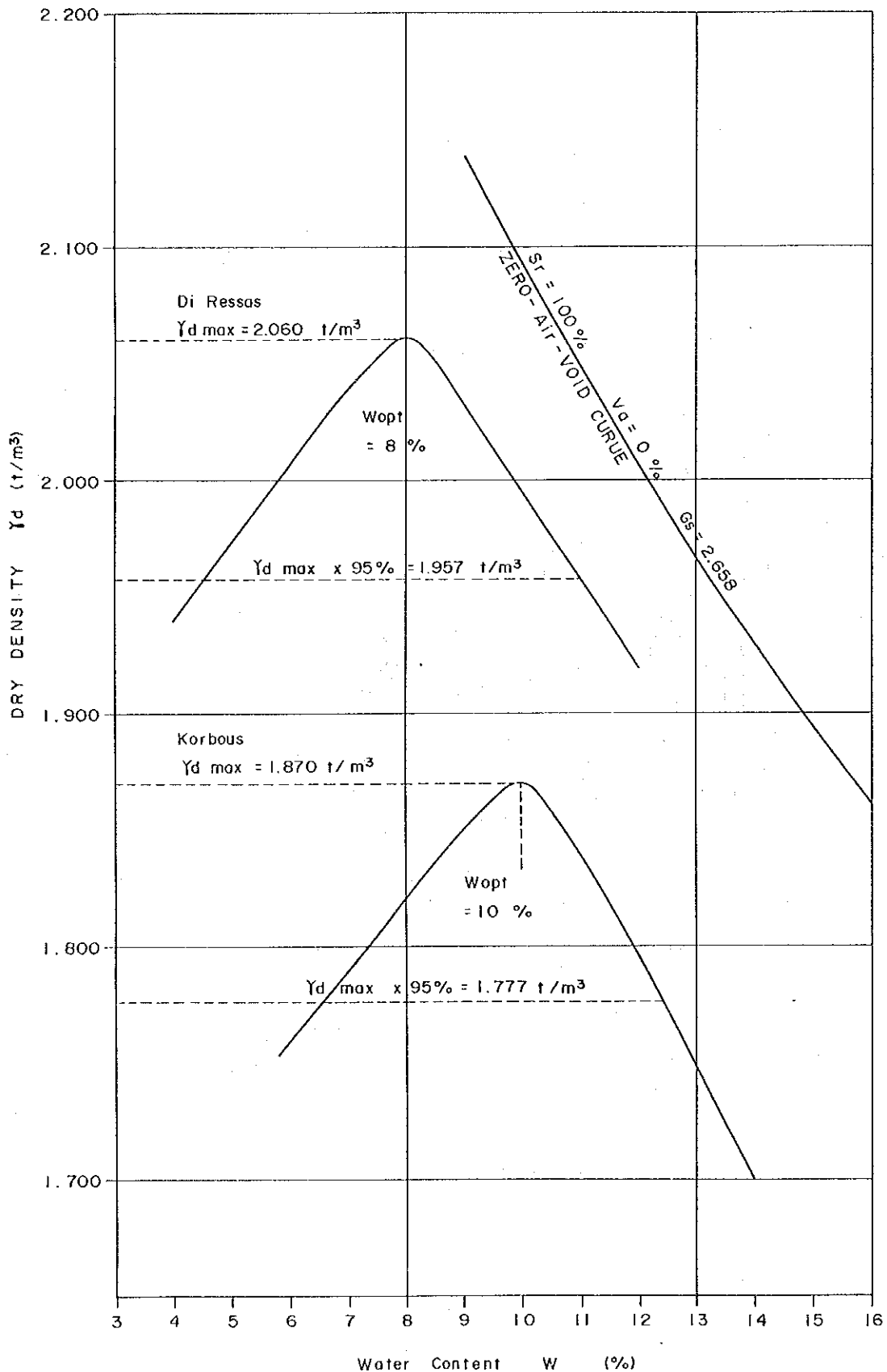
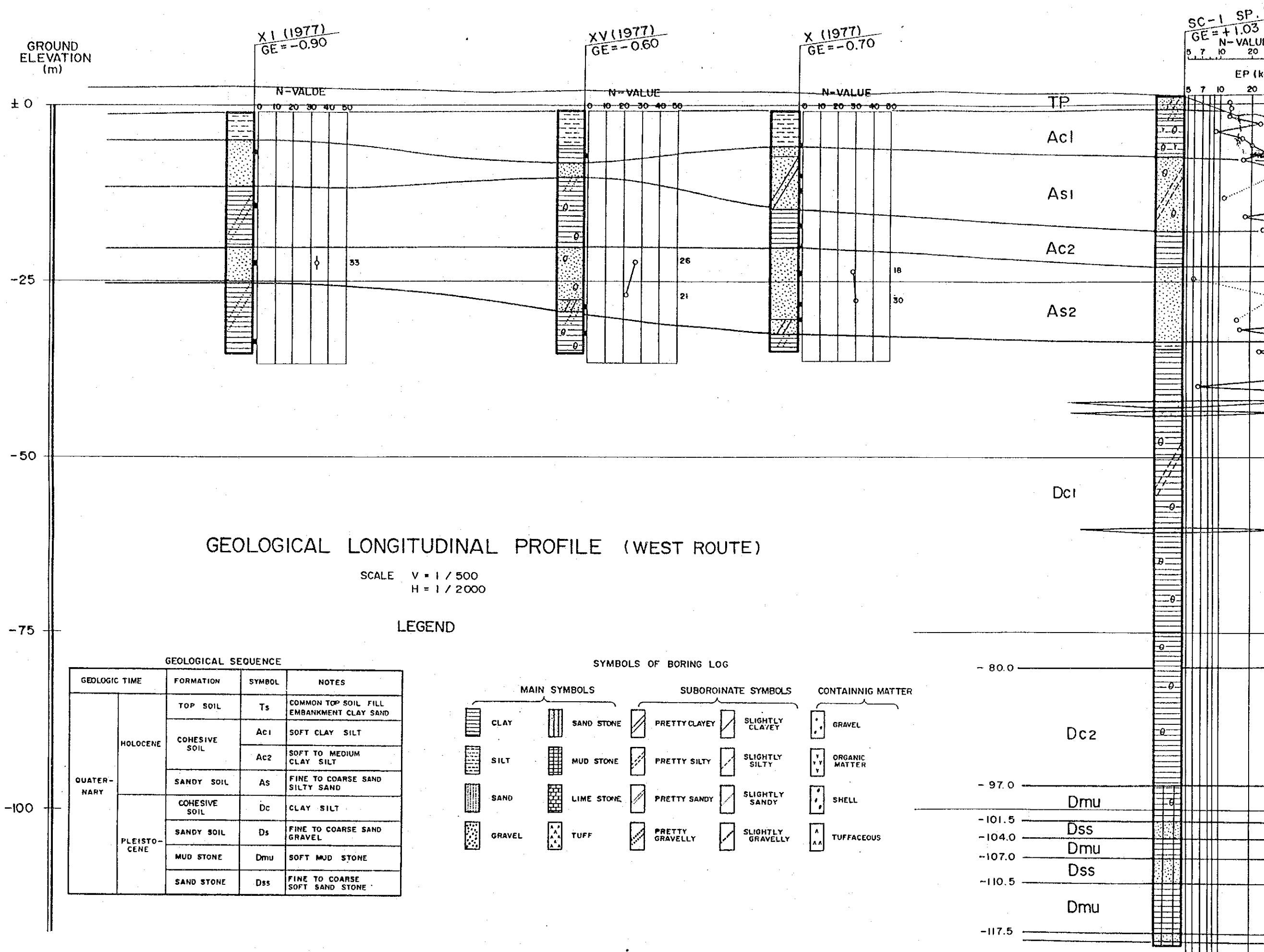


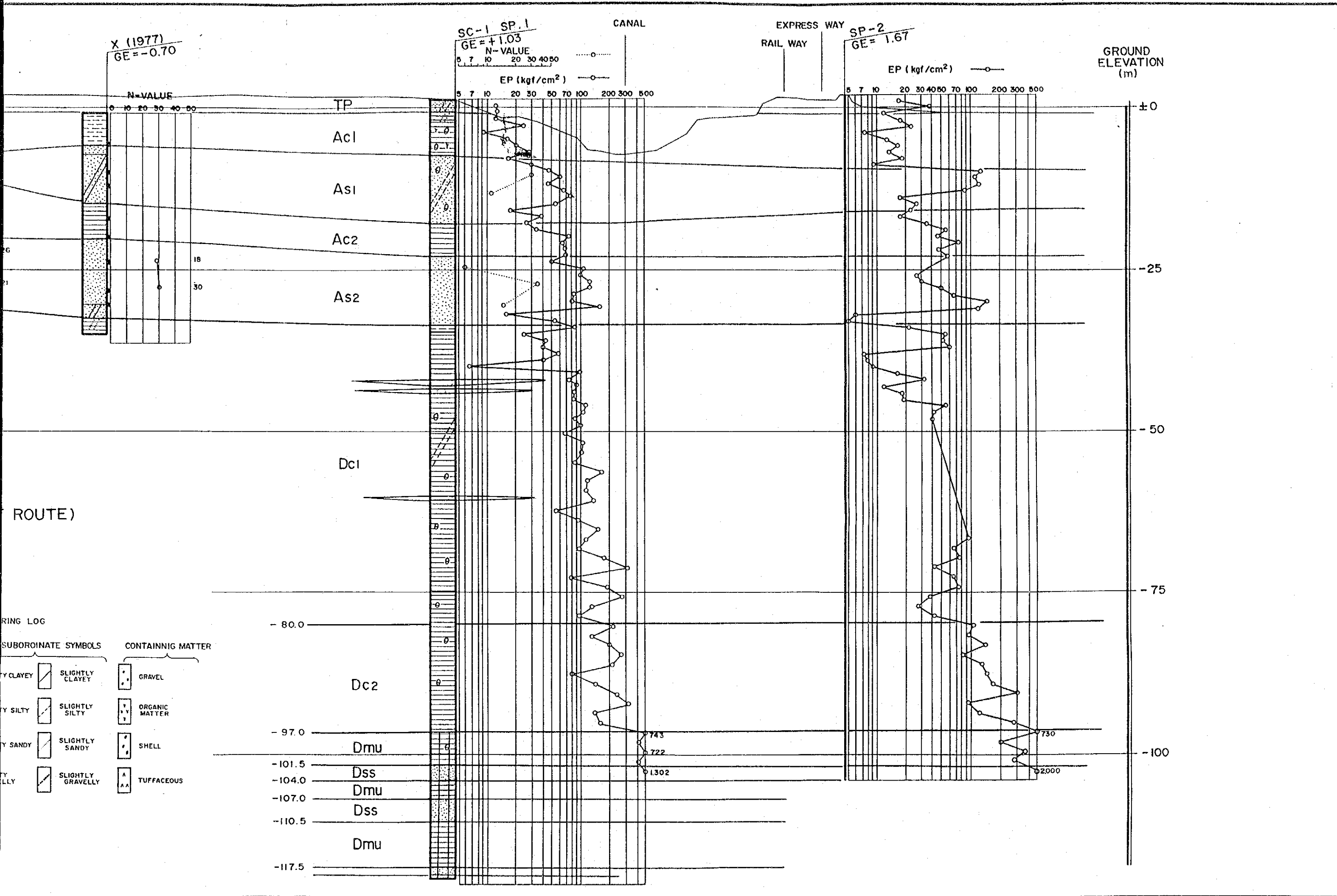
Fig. 13 GRAIN SIZE DISTRIBUTION CURVE (Embankment Material)

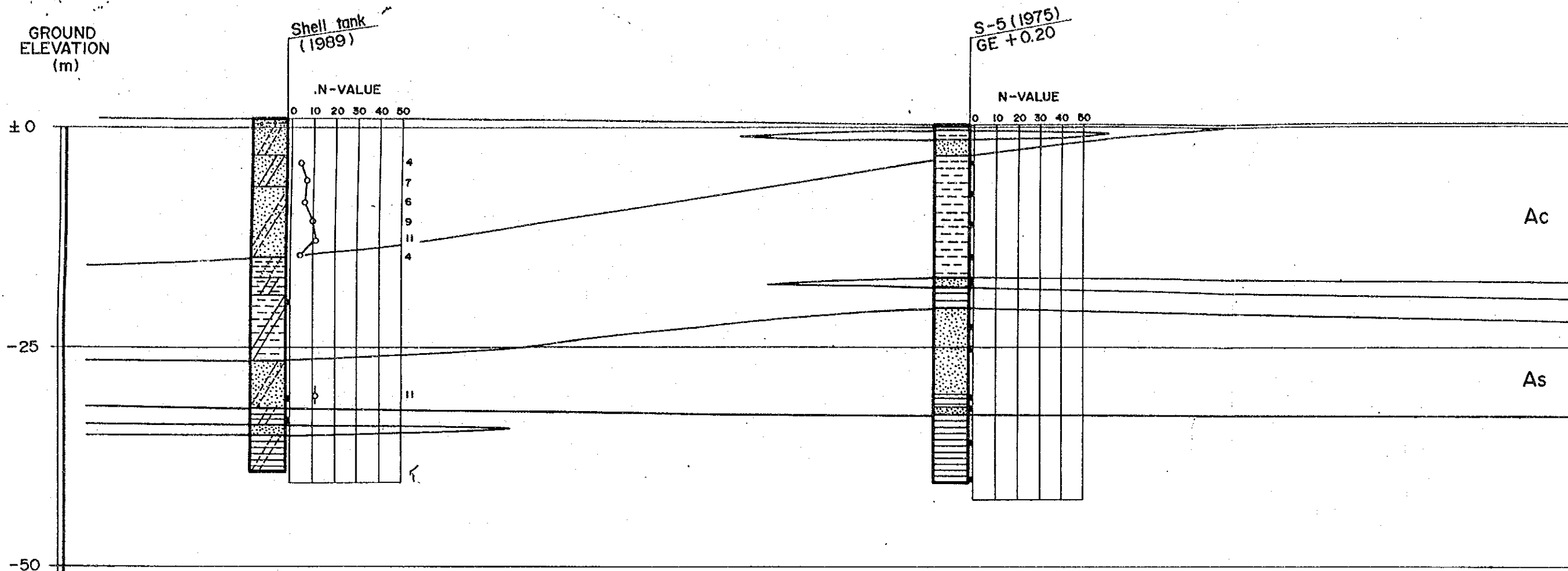
Fig. 14 COMPACTION CURVE FOR EMBANKMENT MATERIALS



Geological longitudinal profile







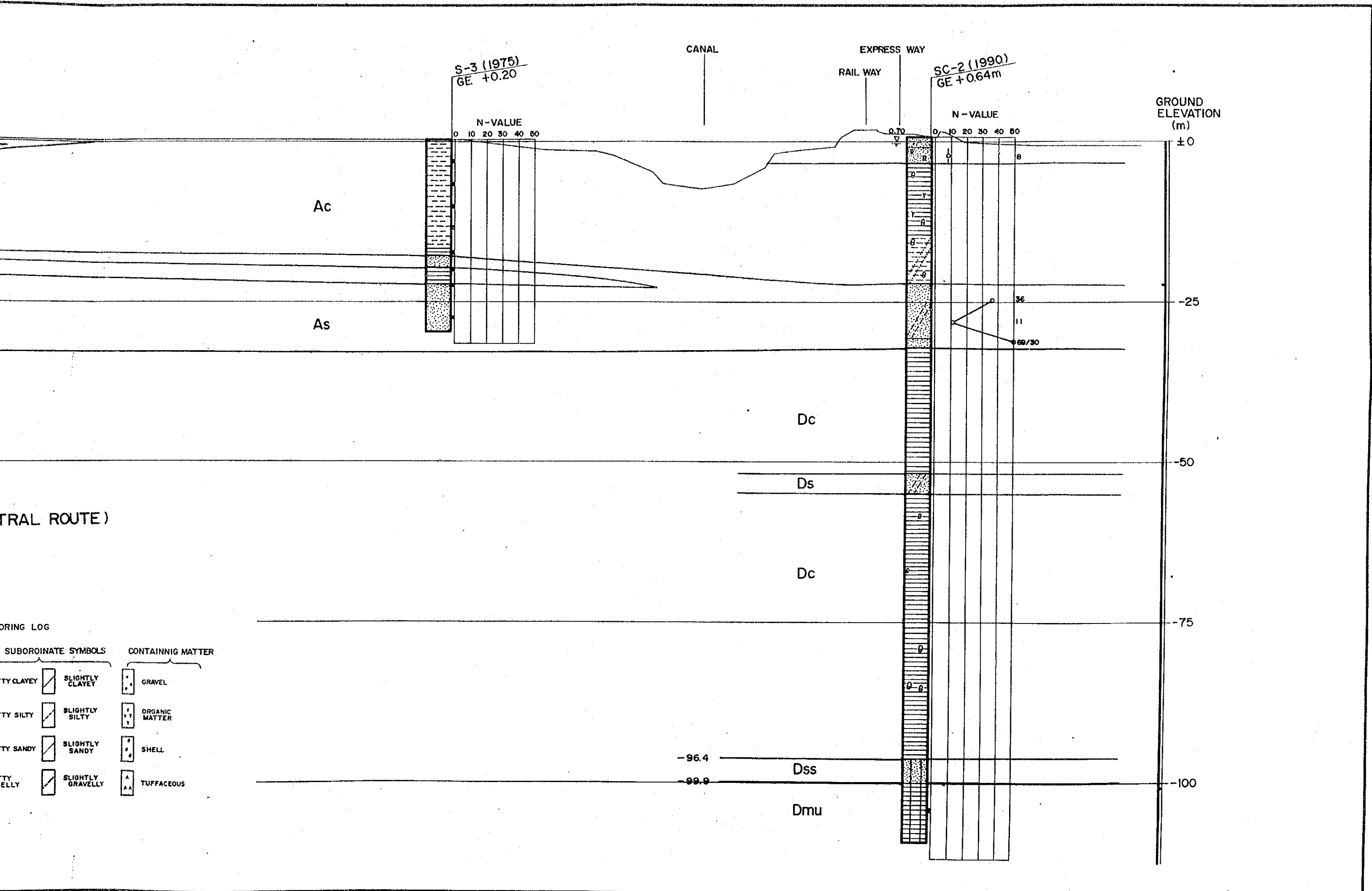
GEOLOGICAL LONGITUDINAL PROFILE (CENTRAL ROUTE)

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		Dm	SOFT MUD STONE	
	SAND STONE	Dss	FINE TO COARSE SOFT SAND STONE	

SYMBOLS OF BORING LOG		
MAIN SYMBOLS	SUBORDINATE SYMBOLS	CONTAINING MATTER
CLAY	SAND STONE	PRETTY CLAYEY
SILT	MUD STONE	PRETTY SILTY
SAND	LIME STONE	PRETTY SANDY
GRAVEL	TUFF	PRETTY GRAVELLY
		SLIGHTLY CLAYEY
		SLIGHTLY SILTY
		SLIGHTLY SANDY
		SLIGHTLY GRAVELLY
		GRAVEL
		ORGANIC MATTER
		SHELL
		TUFFACEOUS



TRAL ROUTE)

BORING LOG

SUBORDINATE SYMBOLS		CONTAINING MATTER	
CLAYEY	SLIGHTLY CLAYEY	GRAVEL	
SILTY	SLIGHTLY SILTY	ORGANIC MATTER	
SANDY	SLIGHTLY SANDY	SHELL	
VELLY	SLIGHTLY GRAVELLY	TUFFACEOUS	

Boring log

44	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			
46									
48									
50									
52									
54									
56					DARK GREY	STIFF SILTY CLAY INCLUDING SHELL FRAGMENT			
					LIGHT GREENISH GREY	COHESION: HIGH W _n : MEDIUM			
58	55.97	57.00	12.00	CLAY	BLACKISH GREY	MEDIUM CLAY WITH SHELL FRAGMENT			
60	57.97	59.00	2.00	CLAY					
62	60.17	61.20	2.20	CLAY	DARK GREY	STIFF CLAY			
	60.57	61.60	0.40	SAND	GREY	COHESION: HIGH SILTY FINE SAND			
64									
66									
68									
70									
72									
74									
76									
78									
80									
82									
84					DARK GREY	STIFF CLAY INCLUDING SHELL FRAGMENT			
86					BROWNISH GREY	COHESION: HIGH W _n : MEDIUM			
88									
90									
92									
94									
96									
98	96.97	98.00	36.40	CLAY	DARK GREENISH GREY				
100	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			
102	101.47	102.50	2.30	CLAY	BLACKISH GREY	HARD CLAY			
104	103.97	105.00	2.50	SOFT SAND STONE	GREY	CLAYE FINE COMPACT SAND OF SOLID STATUS			
106				SOFT MUD STONE	YELLOWISH GREEN	HARD CLAY OF SOLID STATUS			
108	106.97	108.00	3.00	SOFT MUD STONE	GREY	ALTERNATION OF SANDY CLAY AND FINE SAND			
110				SOFT SAND STONE	GREY	ALTERNATION OF SAND AND CLAY THICKNESS 1/3 cm			
112	110.47	111.50	3.50	CLAY (SOFT MUD STONE)	BLACKISH GREY	HARD HUMUS CLAY WITH MANY SHELL AND SHELL FRAGMENT			
114	113.97	115.00	3.50	SOFT MUD STONE	GREY TO WHITE				
116				SOFT MUD STONE	GREY				
118	117.47	118.50	3.50	SOFT MUD STONE	YELLOWISH GREEN	HARD CLAY OF SOLID STATUS			
120	118.72	119.75	1.25	SOFT MUD 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

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 CONSTRUCTION BETWEEN RADES AND LA GOULETTE

DATE : 14TH FEB. 1990 - 24TH FEB. 1990

GROUND ELEVATION : + 0.64 m

HOLE NO. : SC - 2 (φ = 101mm)

SURVEYED BY : S. TAKAD HAMDY

GROUNDWATER LEVEL : GL - 0.70 m

SCALE	ELEVATION m	DEPTH m	THICKNESS OF STRATUM m	SYMBOL	VISUAL CLASSIFICATION	COLOR	DESCRIPTION	DEPTH m	STANDARD PENETRATION TESTS					N VALUE	NO OF SAMPLE	DEPTH m	SYMBOL OF SAMPLER
									NO OF BLOWS LENGTH OF PENETRATION cm	15 cm	30 cm	45 cm	60 cm				
2	-0.96	1.60	1.60	X	SAND	LIGHT BROWN	EMBANKMENT MATERIAL SAND WITH GRAVEL	2.70	8 / 30	4	4	4	4		S-1	2.70	○
4	-3.36	4.60	2.40	6	SAND	LIGHT GREY	FINE SAND WITH SHELL FRAGMENT	3.15									○
6				6			VERY SOFT TO SOFT SILTY CLAY WITH SHELL FRAGMENT AND SEA WEEDS										
8				6			COHESION : HIGH										
10				6			WR : HIGH										
12				6													
14	-14.56	15.20	11.20	6	CLAY	BLACKISH GREY	MEDIUM SANDY SILT WITH SHELL FRAGMENT										
16	-20.36	21.00	5.80	6	SILT	YELLOWISH GREEN	COHESION : MEDIUM TO HIGH										
18				6			WR : MEDIUM										
20	-22.36	23.00	2.00	6	SILT	GREY	MEDIUM TO STIFF SANDY SILT WITH SHELL FRAGMENT										
22				6													
24	-25.36	26.00	3.00	6	SAND	LIGHT BROWN	FINE SAND WITH SLIGHTLY SILT AND SHELL FRAGMENT	25.50	36 / 30	10	16	20			S-2	25.56	○
26				6													
28				6			SILTY FINE SAND INCLUDING SHELL AND SHELL FRAGMENT	25.95									
30				6													
32	-30.86	31.50	5.50	6	SAND	GREY	COMPACT FINE SAND	28.80	11 / 30	3	6	5			S-3	28.80	○
34	-32.36	33.00	1.50	6	SAND	GREY		29.25									
36				6													
38				6			SOFT TO MEDIUM SILTY CLAY WITH SHELL FRAGMENT AND HUMUS SEA WEEDS										
40				6			COHESION : HIGH										
42	-40.86	41.50	8.50	6	CLAY	BLACKISH GREY	WR : MEDIUM TO HIGH	31.50	69 / 30	12	31	38			S-4	31.50	○
44				6													
46	-43.36	44.00	2.50	6	CLAY	YELLOW	HUMUS MEDIUM CLAY WITH MANY CORAL FRAGMENT	31.95									
48				6			COHESION : HIGH										
50	-47.56	48.20	4.20	6	CLAY	BLACKISH GREY	HUMUS MEDIUM CLAY WITH CORAL FRAGMENT										
52	-51.86	52.50	4.30	6	CLAY	GREENISH GREY	MEDIUM TO STIFF CLAY										
54				6			COHESION : HIGH										
56	-55.06	55.70	3.20	6	SAND	GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL AND SHELL FRAGMENT										
58				6													
60	-60.36	61.00	5.30	6	CLAY	GREY	MEDIUM CLAY WITH SHELL AND SHELL FRAGMENT										
62				6			COHESION : HIGH										
64				6													
66				6			STIFF CLAY WITH SLIGHTLY SHELL AND CORAL FRAGMENT										
68				6			COHESION : HIGH										
70				6			WR : MEDIUM										

DEPTH (m)	DEPTH (ft)	SOIL TYPE	COLOUR	REMARKS	LABORATORY	DEPTH (m)	DEPTH (ft)	SOIL TYPE	COLOUR	REMARKS	LABORATORY
44	144.35		GREY			48	157.56	CLAY	BLACKISH GREY		UD-3
46			YELLOW			50	161.86	CLAY	GREENISH GREY		49.50
48	157.56	CLAY	BLACKISH GREY			52	166.06	SAND	GREY		
50	161.86	CLAY	GREENISH GREY			54	170.26				
52	166.06					56	174.46				
54	170.26					58	178.66				
56	174.46					60	182.86	CLAY	GREY		UD-4
58	178.66					62	187.06				70.00
60	182.86	CLAY	GREY			64	191.26				70.50
62	187.06					66	195.46				
64	191.26					68	199.66				
66	195.46					70	203.86				
68	199.66					72	208.06				
70	203.86					74	212.26				
72	208.06					76	216.46				
74	212.26					78	220.66				
76	216.46					80	224.86				
78	220.66					82	229.06				
80	224.86					84	233.26	CLAY	DARK GREY		
82	229.06					86	237.46	CLAY	GREENISH YELLOW		
84	233.26	CLAY	DARK GREY			88	241.66				
86	237.46	CLAY	GREENISH YELLOW			90	245.86				
88	241.66					92	250.06				
90	245.86					94	254.26				
92	250.06					96	258.46				
94	254.26					98	262.66				
96	258.46					100	266.86	SAND	GREY		
98	262.66					102	271.06				
100	266.86	SAND	GREY			104	275.26	SOFT MUD STONE	GREENISH YELLOW		
102	271.06					106	279.46				
104	275.26	SOFT MUD STONE	GREENISH YELLOW			108	283.66				
106	279.46					110	287.86	CLAY	GREY		UD-5
108	283.66					112	292.06				104.50
110	287.86	CLAY	GREY			114	296.26				105.00
112	292.06					116	300.46				
114	296.26					118	304.66				
116	300.46					120	308.86				

REMARKS :

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

SC - 2

Pressiometer test log

BORING LOG

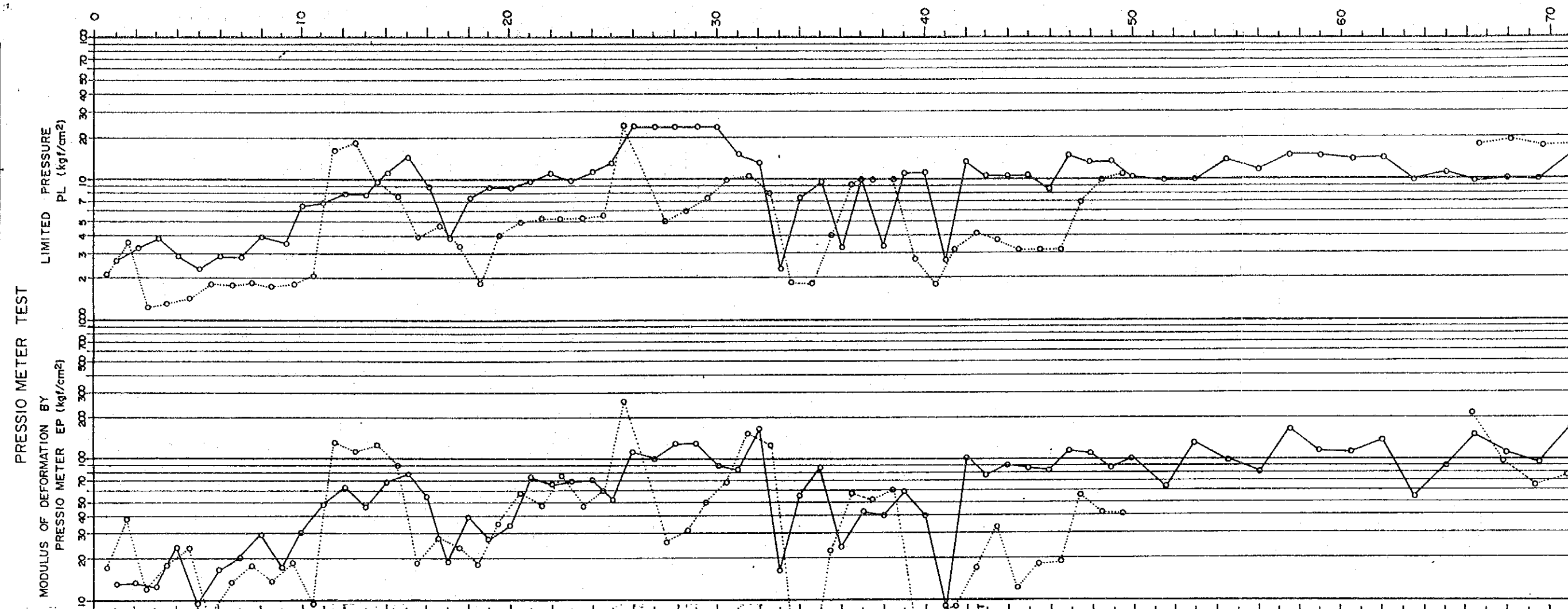
SCALE : 1 / 200

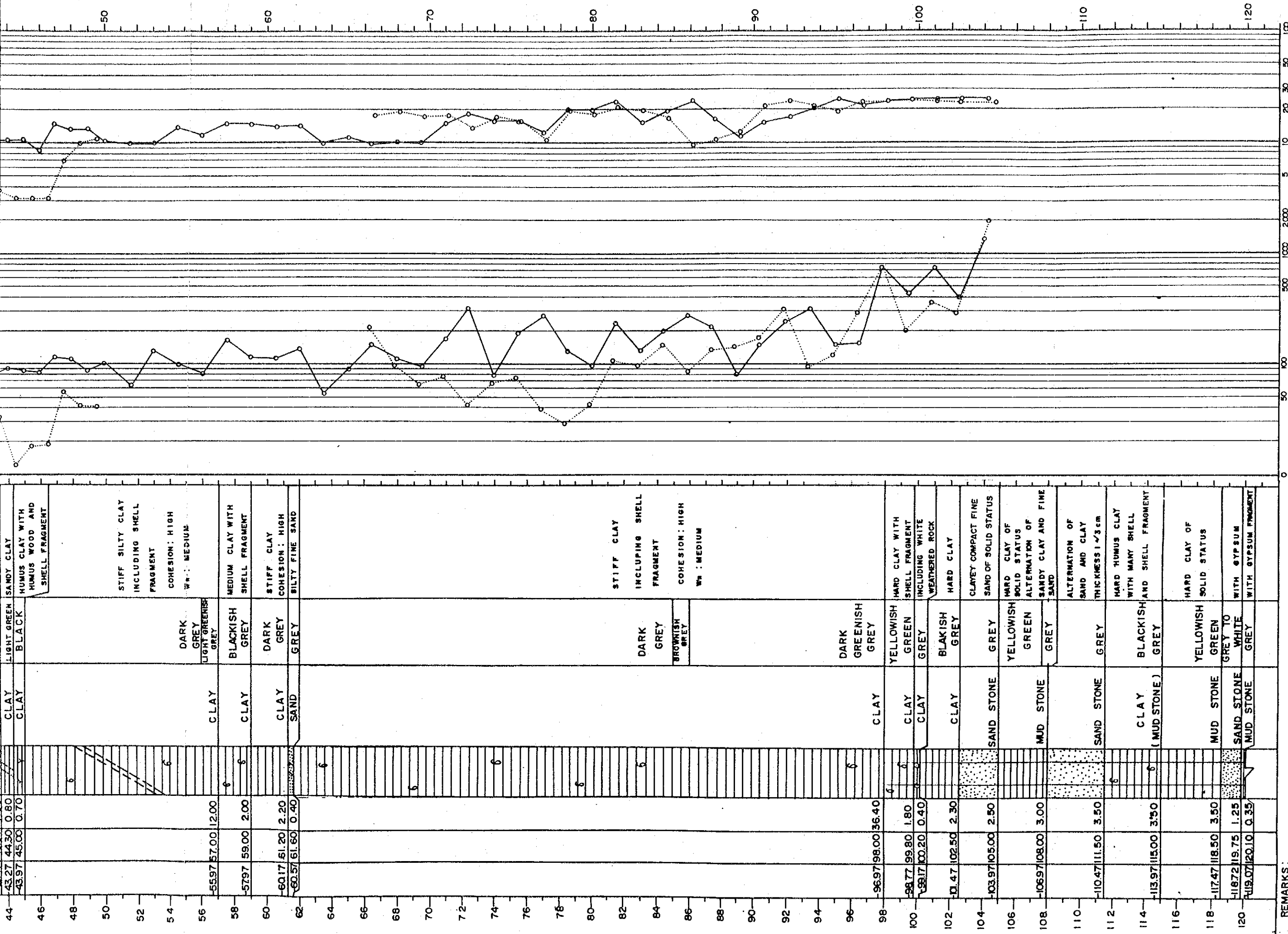
FEASIBILITY STUDY ON CROSSING CONSTRUCTION

PROJECT : BETWEEN RADES AND LA GOULETTE GROUND ELEVATION : + 1.03 m DATE : 22TH JUN 1990 ~ 12TH FEB 1990
 HOLE NO. : SC - 1 (Ø = 10 [mm]) GROUND WATER LEVEL : GL - 1.10 m SURVEYED BY : S. TAKADA HAMDY

SC - 1 (Ø = 10 [mm])

SCALE	ELEVATION m	DEPTH m	THICKNESS OF STRATUM m	SYMBOL	VISUAL CLASSIFICATION	COLOR	DESCRIPTION
2	-0.87	1.90	1.90	SAND	SAND	LIGHT BROWN	SILTY, FINE TO COARSE SAND WITH SHELL FRAGMENT
4	-2.47	3.50	1.60	CLAY	CLAY	DARK GREY	SOFT SANDY CLAY WITH SHELL FRAGMENT COHESION: HIGH
6	-7.47	6.50	5.00	CLAY	CLAY	BLAKKISH GREY	SOFT HUMUS CLAY COHESION: HIGH INCLUDING SHELL AND SHELL FRAGMENT HUMUS WOODS
10	-12.97	14.00	5.50	SAND	SAND	LIGHT BROWN	DEPTH 8.60M COBBLE Ø ± 10 cm SILTY FINE SAND INCLUDING SHELL FRAGMENT
16	-17.97	19.00	5.00	SAND	SAND	BROWNISH GREY	SILTY FINE SAND PARTIALLY SANDY SILT INCLUDING SHELL FRAGMENT
24	-23.07	24.10	5.10	CLAY SAND	CLAY SAND	BROWNISH GREY	SOFT TO MEDIUM CLAY
26	-25.32	24.35	0.25	SAND	SAND	DARK GREEN	SILTY FINE SAND
30	-29.57	30.60	6.25	SAND	SAND	BROWNISH GREY	FINE SAND INCLUDING SHELL FRAGMENT
34	-33.67	34.70	4.10	SAND	SAND	GREY	FINE SAND INCLUDING SHELL FRAGMENT AND FRAGMENT AND
36	-34.72	35.75	1.05	SILT SAND	SILT SAND	BROWNISH GREY	BRIGHTLY SILTY SANDY SILT
38	-35.12	36.15	0.40	SAND	SAND	GREY	COHESION: LOW FINE SAND WITH SHELL FRAGMENT
40	-40.77	41.80	5.65	CLAY	CLAY	GREY	MEDIUM TO STIFF CLAY INCLUDING SHELL FRAGMENT
42	-41.92	42.95	1.15	CLAY	CLAY	LIGHT GREEN	DEPTH 40 - 42 m SOLID STATUS
44	-42.47	43.50	0.55	SAND	CLAY SAND	LIGHT GREEN	MEDIUM TO STIFF SILTY CLAY WITH SHELL FRAGMENT
46	-43.27	44.30	0.80	CLAY	CLAY	LIGHT GREEN	FINE SAND
48	-43.97	45.00	0.70	CLAY	CLAY	BLACK	SANDY CLAY HUMUS CLAY WITH HUMUS WOOD AND SHELL FRAGMENT
50	-55.97	57.00	12.00	CLAY	CLAY	DARK GREY	STIFF SILTY CLAY INCLUDING SHELL FRAGMENT
52	-57.97	59.00	2.00	CLAY	CLAY	LIGHT GREENISH GREY	COHESION: HIGH W _n : MEDIUM
58	-60.17	61.20	2.20	CLAY SAND	CLAY SAND	BLACKISH GREY	MEDIUM CLAY WITH SHELL FRAGMENT
60	-60.57	61.60	0.40	SAND	SAND	DARK GREY	STIFF CLAY COHESION: HIGH
62						GREY	SILTY FINE SAND





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

REMARKS:

LEGEND: ○ — SP. 1 (SC.1 GE = 1.03m)
○..... SP. 2 (GE = 1.67m)

SC-1 (SP-1)

ANNEXE 7.5.1.1

NEW SEISMIC DESIGN SPECIFICATIONS
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 IWASAKI¹⁾, Kazuhiko KAWASHIMA²⁾ and Kinji HASEGAWA³⁾

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 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 losing 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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- 3) Research Engineer, ditto

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

$$T_G = 4 \sum_{i=1} \frac{H_i}{V_{oi}} \quad (1)$$

where

T_G : characteristic value (sec)

H_i : thickness of i -th subsoil layer (m)

V_{oi} : 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

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

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

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

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

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 substructures. 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). Based 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_{60} -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

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

where

F_L : liquefaction resistance factor

R : resistance of soil elements against dynamic load

L : dynamic load induced in soil ele-

and 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.

$$k_h = c_z \cdot c_g \cdot c_i \cdot c_T \cdot k_{hd} \quad (2)$$

where

- k_h : design horizontal seismic coefficient,
- k_{hd} : 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 a 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

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

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

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

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

$$P_o > k_{ho} W \quad (9)$$

where

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

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

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

$$W = W_u + C_P W_p \quad (10)$$

$$C_P = \begin{cases} 0.5 & P_u \leq P_s \\ 1.0 & P_u > P_s \end{cases} \quad (11)$$

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

$$k_{ho} = \frac{k_{ho0}}{\sqrt{2\mu - 1}} \quad (12)$$

$$k_{ho0} = C_z \cdot C_1 \cdot C_R \cdot k_{ho00} \quad (13)$$

where

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

k_{ho0} : 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_1 : modification factor for importance (refer to Table 6)

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

k_{ho00} : standard horizontal seismic coefficient for check of bearing capacity for lateral force

The standard horizontal seismic coefficient k_{ho00} 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_o and the allowable ductility factor μ shall be determined based on the failure mode as :

a) Flexural failure

$$P_o = P_y + \frac{P_u - P_y}{a} \quad (14)$$

$$\mu = 1 + \frac{\delta_u - \delta_y}{a \delta_y} \quad (15)$$

where

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

P_y, δ_y : yielding force (tf) and yielding

displament (m) for flexural
 failure
 α : safety factor (=1.5)

b) Shear failure

$$P_o = P_u \quad (16)$$

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

where

P_u : 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

$$S = C_z \cdot C_1 \cdot C_D \cdot S_0 \quad (18)$$

where

S : response spectrum for response spectrum analysis (gal)

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

C_1 : 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.

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- 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 Seismic 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		
2)	Order MHA		R-13.6 t S-12.7	U-400 600 kg/m (carriage way) U-270 400 kg/m (footway)		not considered		
3) 1919	Road Laws MHA		R-13.6 t T-11.2 t S-30 t	Same as 2)	-	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	-	Seismic Coefficient Method K=0.15-0.4 depending on location and ground condition (K \geq 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		Seismic Coefficient Method K \geq 0.2, k \geq 0.1	1946. Nankai (M=8.1) 1948, Fukui (M=7.3) 1952. Tokachi-oki (M=8.2) 1964 Niigata (M=7.5)	
6) 1956 (and 1964)	Revision of Specifica- tions for Design of Steel Road Bridges, JRA	1st 2nd	T-20 t T-14 t	- -	L-20 (51m) L-14 (3.5 t/m)	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)		
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)	1978 Miyagi-ken- oki (M=7.4)	
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)	<p>Failure of Superstructure due to Tilting/Movement of Foundation</p> <p>Failure of Concrete around Fixed Bearing</p> <p>Damage due to Liquefaction</p> <p>Failure of RC Piers, and Bearing</p>	1926 Initiation of Seismic Design (Stipulations for Road Construction)		
1930					
1940				1939 Introduction of Standard Seismic Coefficient (Specifications for Steel Bridge)	
1950	1946 Nankai Earthquake (M8.1) 1948 Fukui Earthquake (M7.3)				
1960	1952 Tokachi-oki Earthquake (M8.1)			1956 Seismic Coefficient depending on Zone and Ground Condition (Specifications for Steel Bridge)	
1970	1964 Niigata Earthquake (M7.5)				1971 Seismic Inspection
1980	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)	1976 Seismic Inspection
1980	1982 Urakawa-oki Earthquake (M7.1)			1980 • Part V Seismic Design, Specifications for Design of Highway Bridge • Introduction of New Evaluation Method for Liquefactions	1979 Seismic Inspection
1980	1983 Nihon-kai-chubu Earthquake (M7.7)				1986 Seismic Inspection
1990				1990 Part V Seismic Design, Specifications for Design of Road Bridge	

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

Chapter	Contents	Same at 1980 Seper.	Modified		Newly Intro-duced
			Slightly	Com-pletely	
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.)	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 Model for Dynamic Response Analysis 6.3.1 Acceleration Response Spectral Value for Response Spectral Analysis				○
	6.3.2 Ground 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	$0.2 \leq T_G < 0.4$	(Diluvium)	II	$0.2 \leq T_G < 0.6$
3	$0.4 \leq T_G < 0.6$	(Alluvium)		
4	$0.6 \leq T_G$	(Soft Alluvium)	III	$0.6 \leq T_G$

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 Structural Response Factor c_T

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

Table 8 Seismic Design Structural Unit

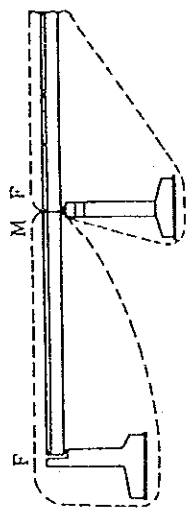
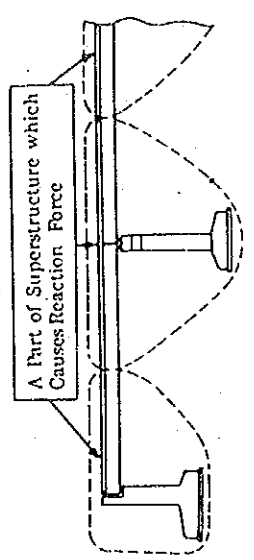
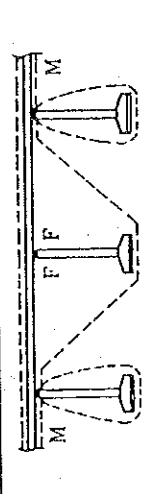

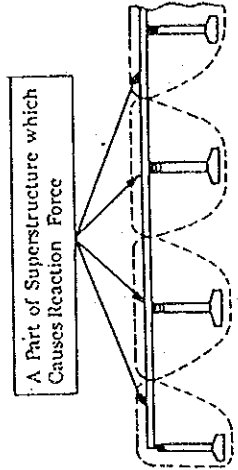
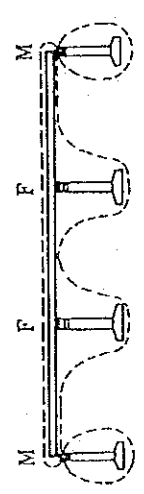
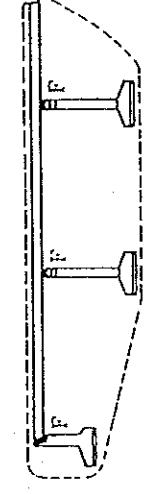
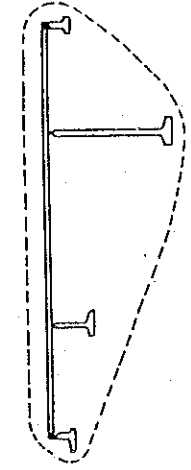
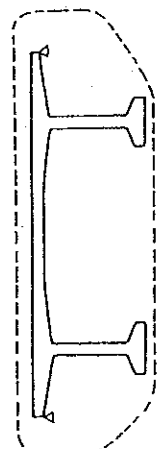
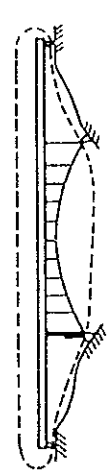
	Longitudinal	Transverse	Seismic Design Unit
Simple Girder			Regarded as A Unit Consisting of A Substructure and A Part of Superstructure Supported by the Substructure
Continuous Girder	Support Condition in Longitudinal Direction		Regarded as A Unit Consisting of Substructures and the Superstructure Supported by the Substructures
	Fixed at One Support		
	Fixed at Multiple Supports		
	Difference of Natural Period between Piers		
	 	 <p>Small</p>	
	 	 <p>Large</p>	
Arch, Frame and Others			

Table 9 Structural Response Factor c_R

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

Table 10 Structural Response Spectral Value S_o

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

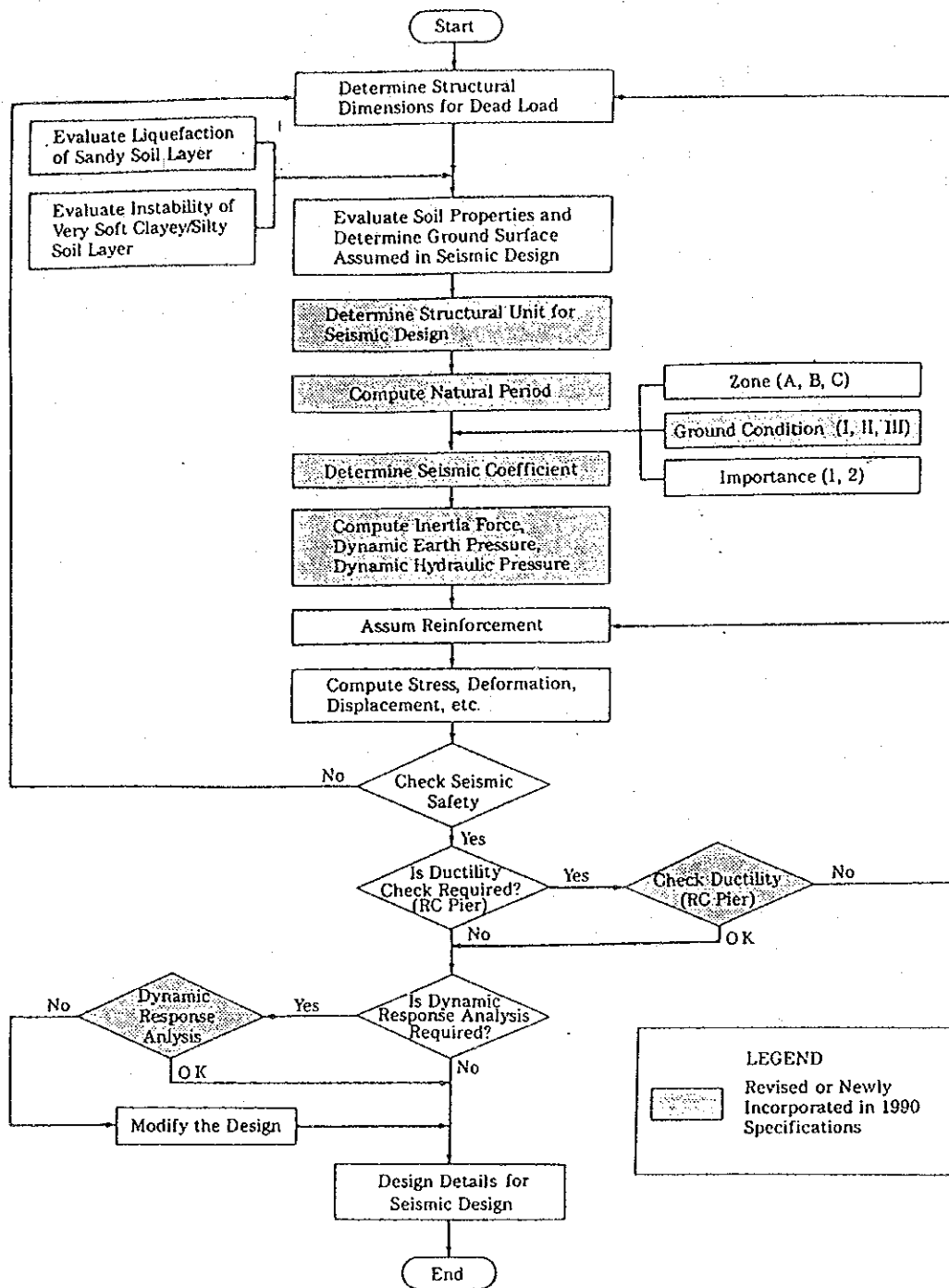


Fig.1 Flow of Seismic Design of Highway Bridge

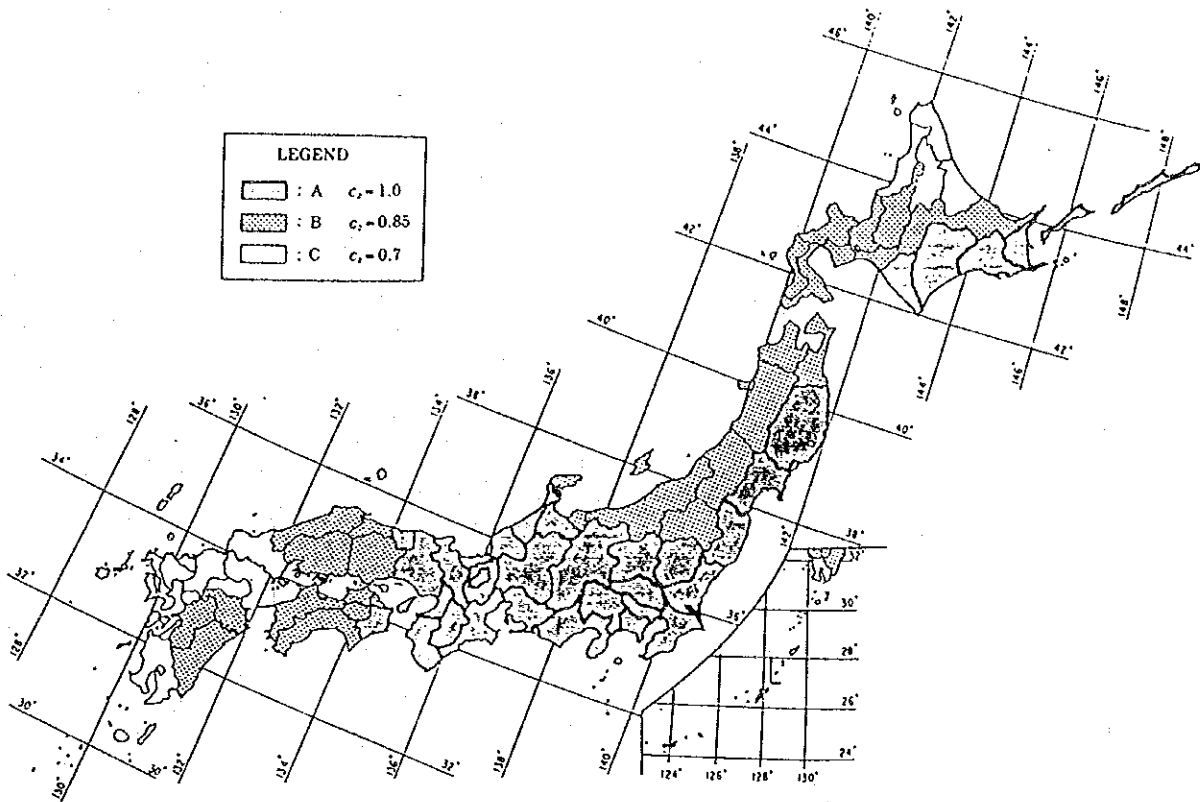


Fig.2 Seismic Zoning Map and Modification Coefficient c_z

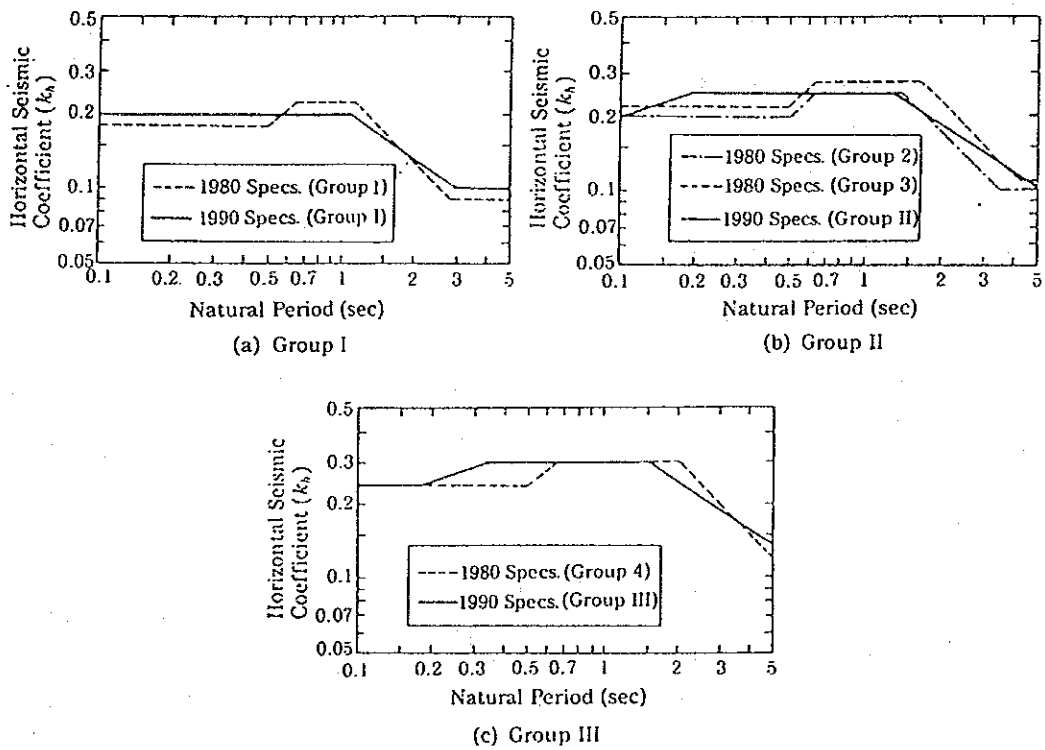


Fig.3 Comparison of Seismic Coefficient between 1980 and 1990 Specifications

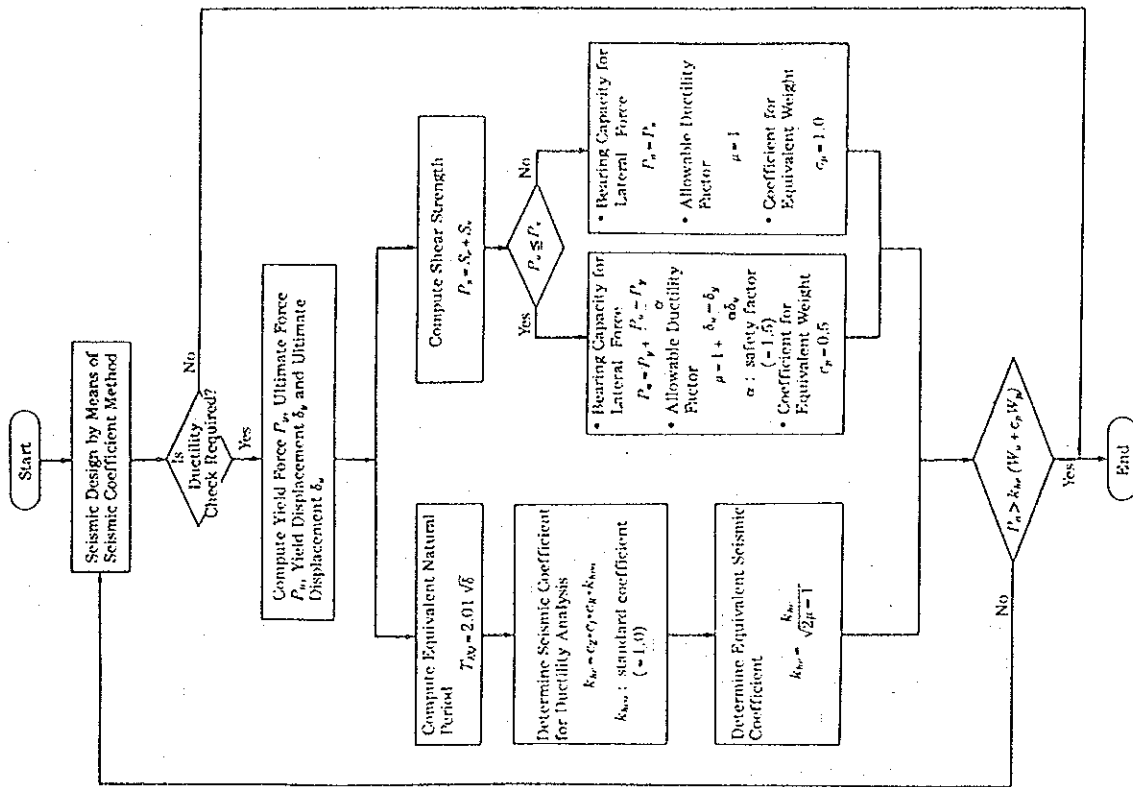


Fig.5 Check of Bearing Capacity of Reinforced Concrete Pier for Lateral Force

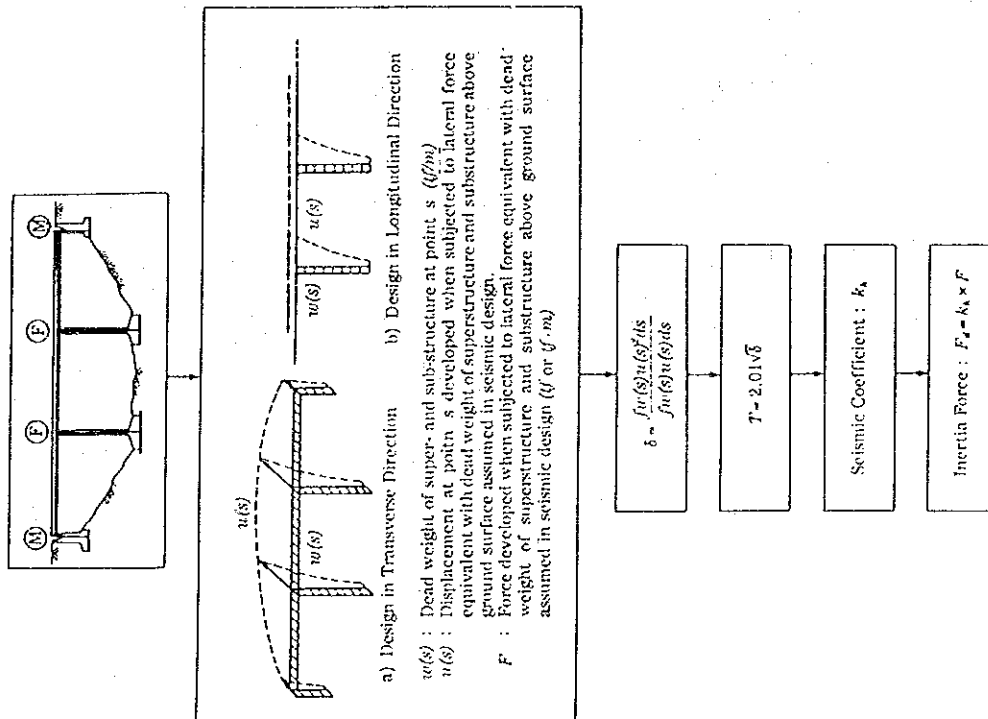
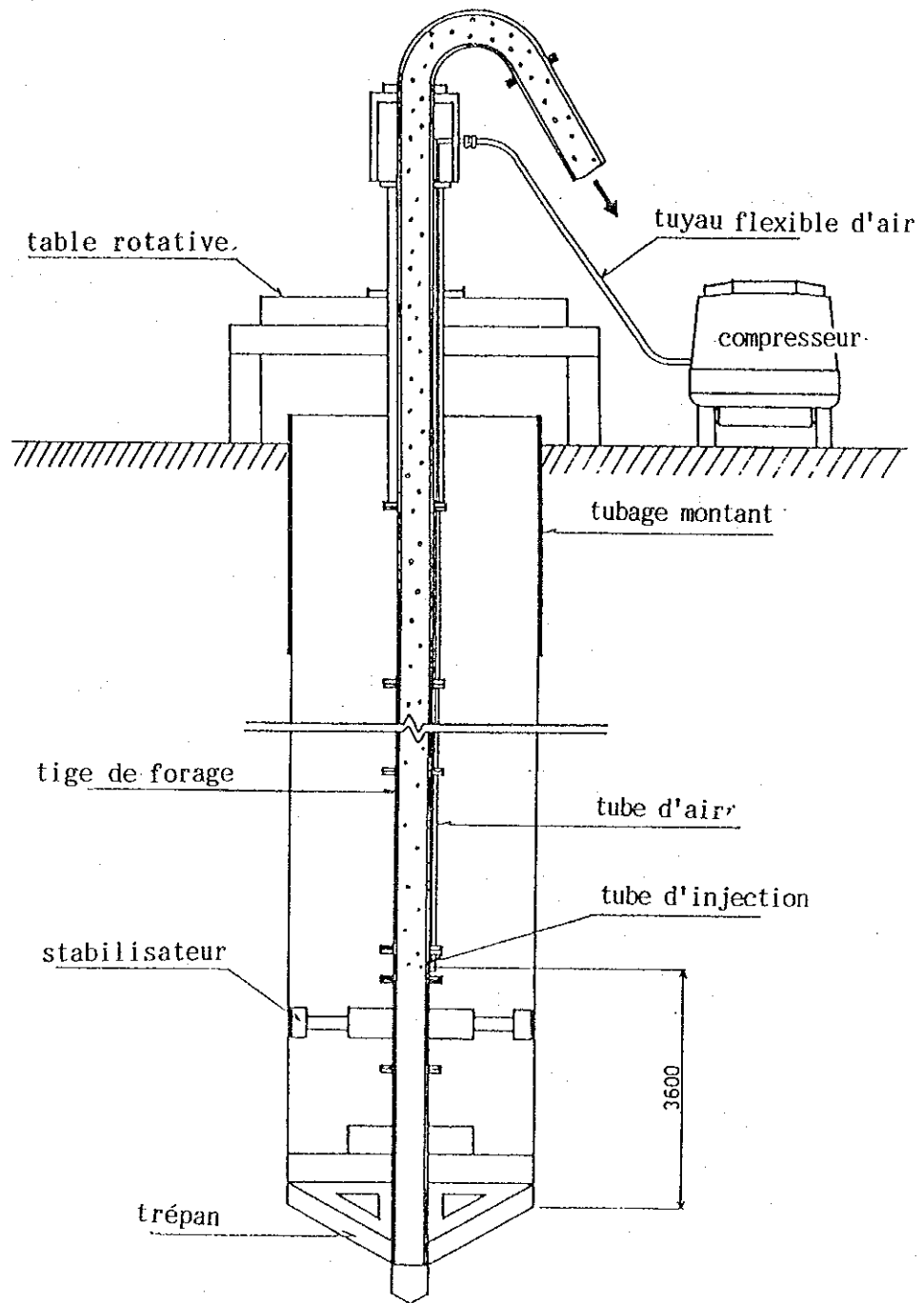


Fig.4 Determination of Inertia Force

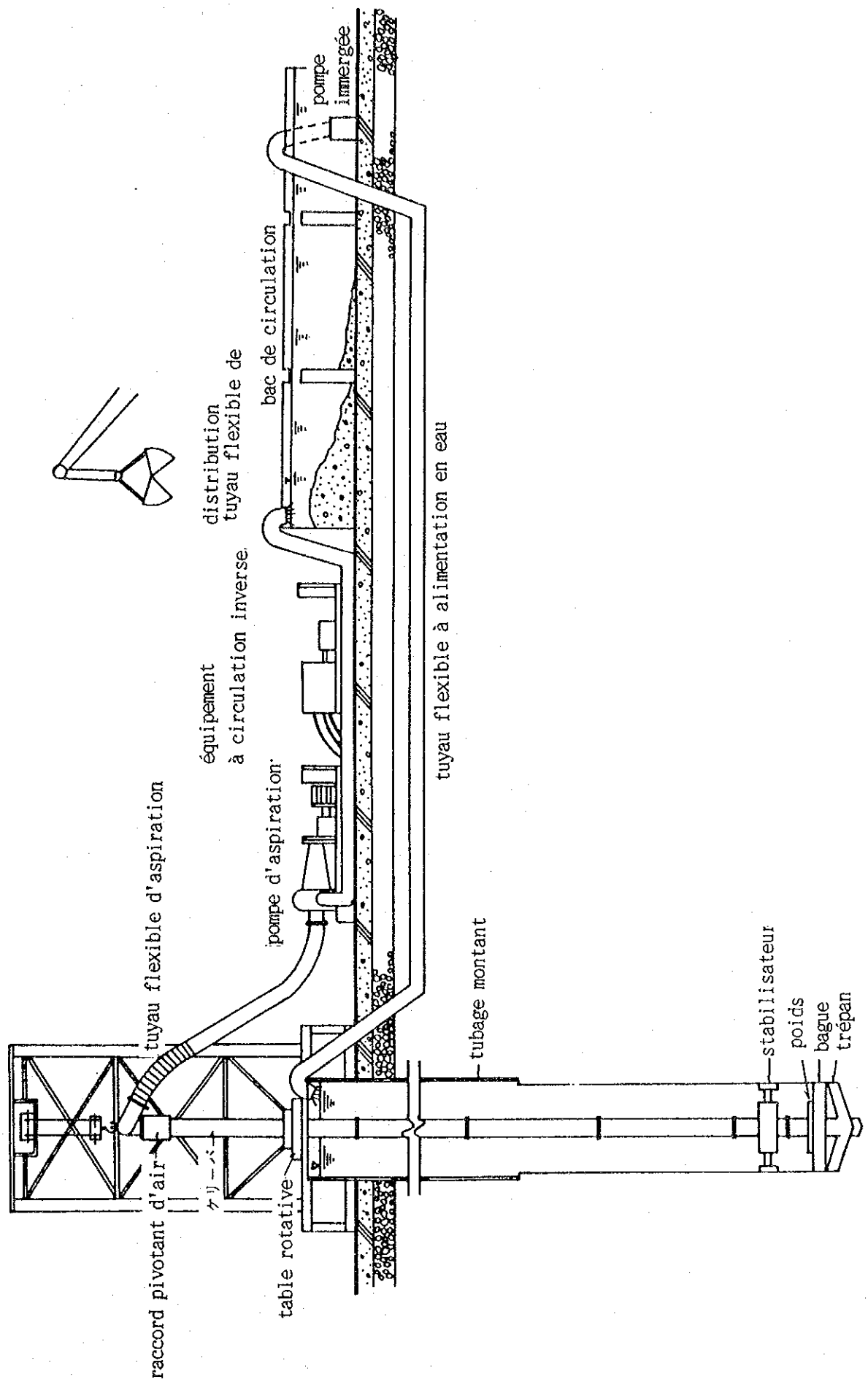
ANNEXE 7.6.6.1

ELEVATEUR PNEUMATIQUE



ANNEXE 7.6.6.2

PROCEDE DE FORAGE A CIRCULATION INVERSE



ANNEXE 8.2.1.1

REPARTITION EN DEVISE, MONNAIE LOCALE ET IMPOTS

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 monnaie 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'origine locale. Pour l'armature, le coke (40% du frais d'armature) est importé et le minerai (60% de ce frais) est d'origine 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 locaux.

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