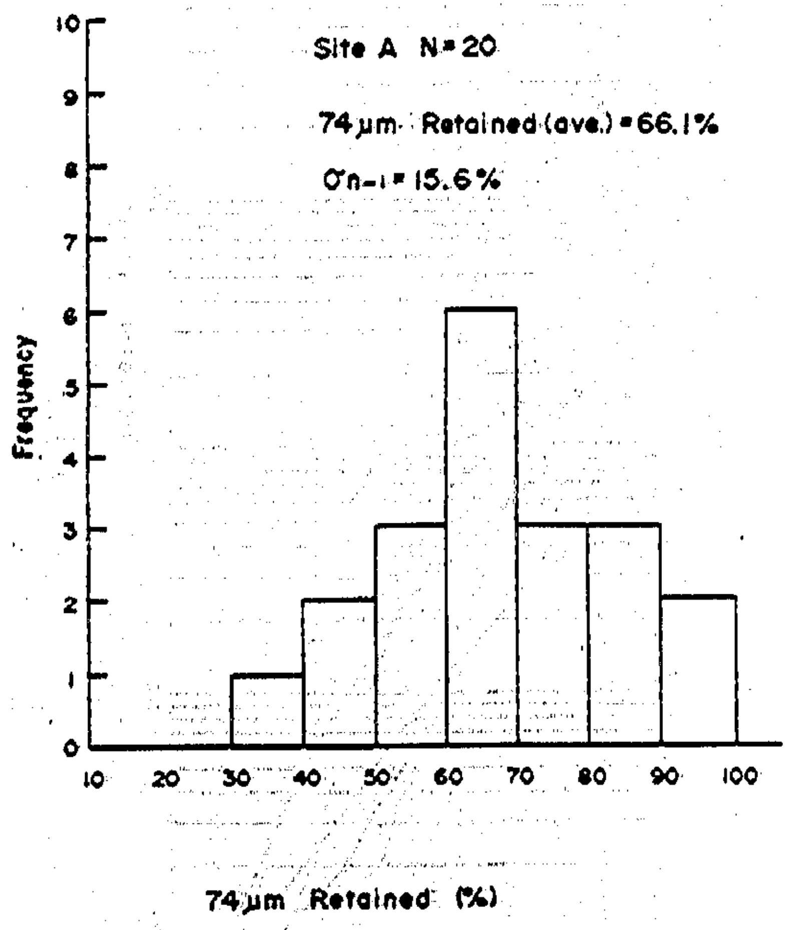
Fig. 6.5 Distribution of 74 μm Retained



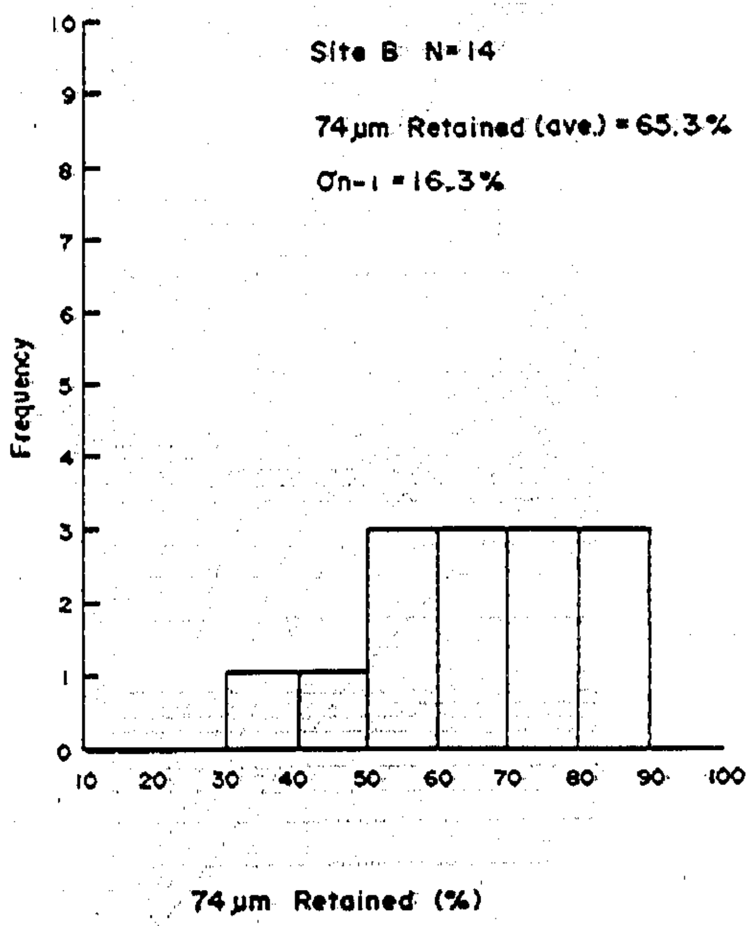


Fig.6.6 Moisture Content – Permeability, Density Relation

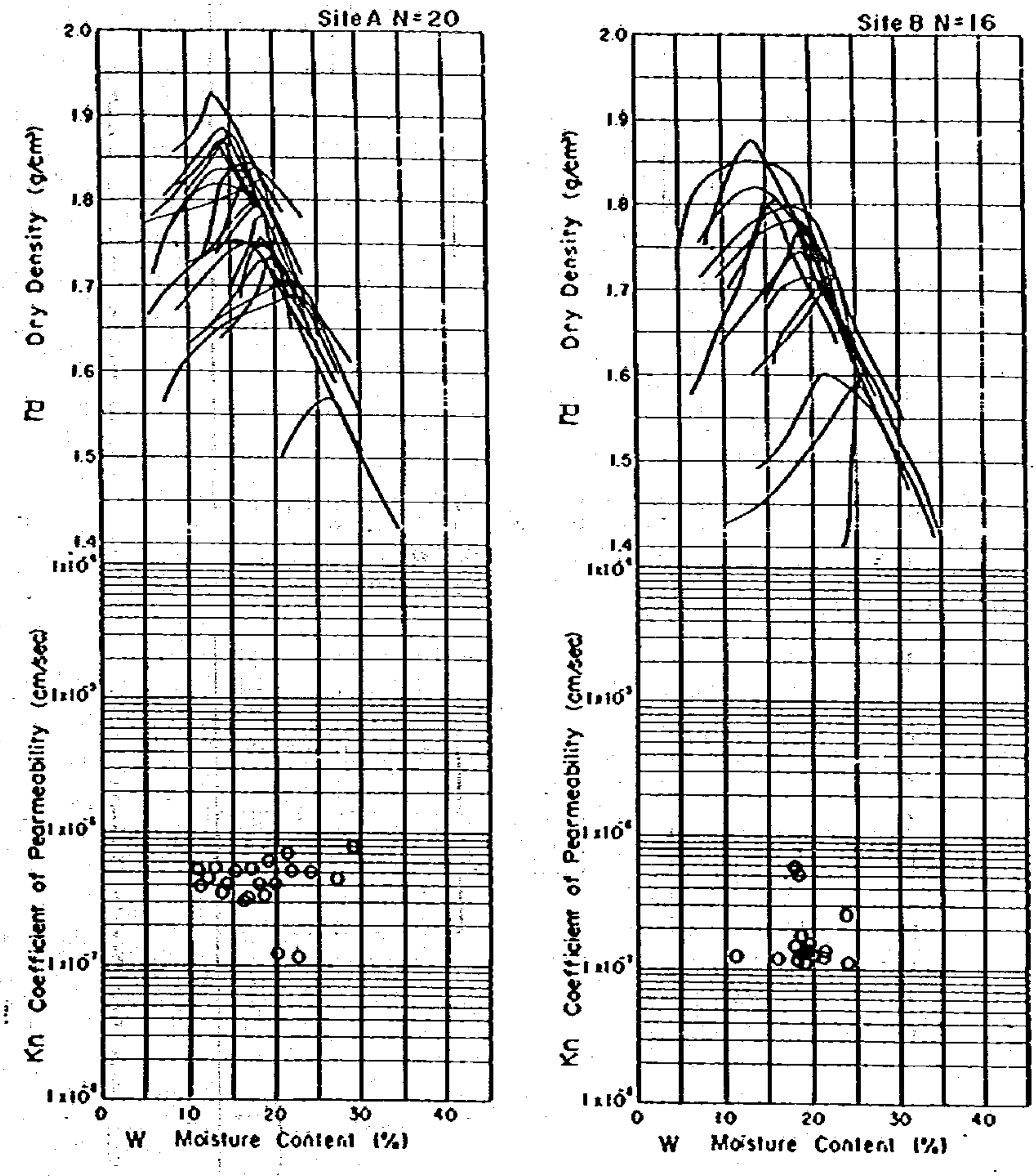


Fig. 6.7 Distribution of Maximum Dry Density

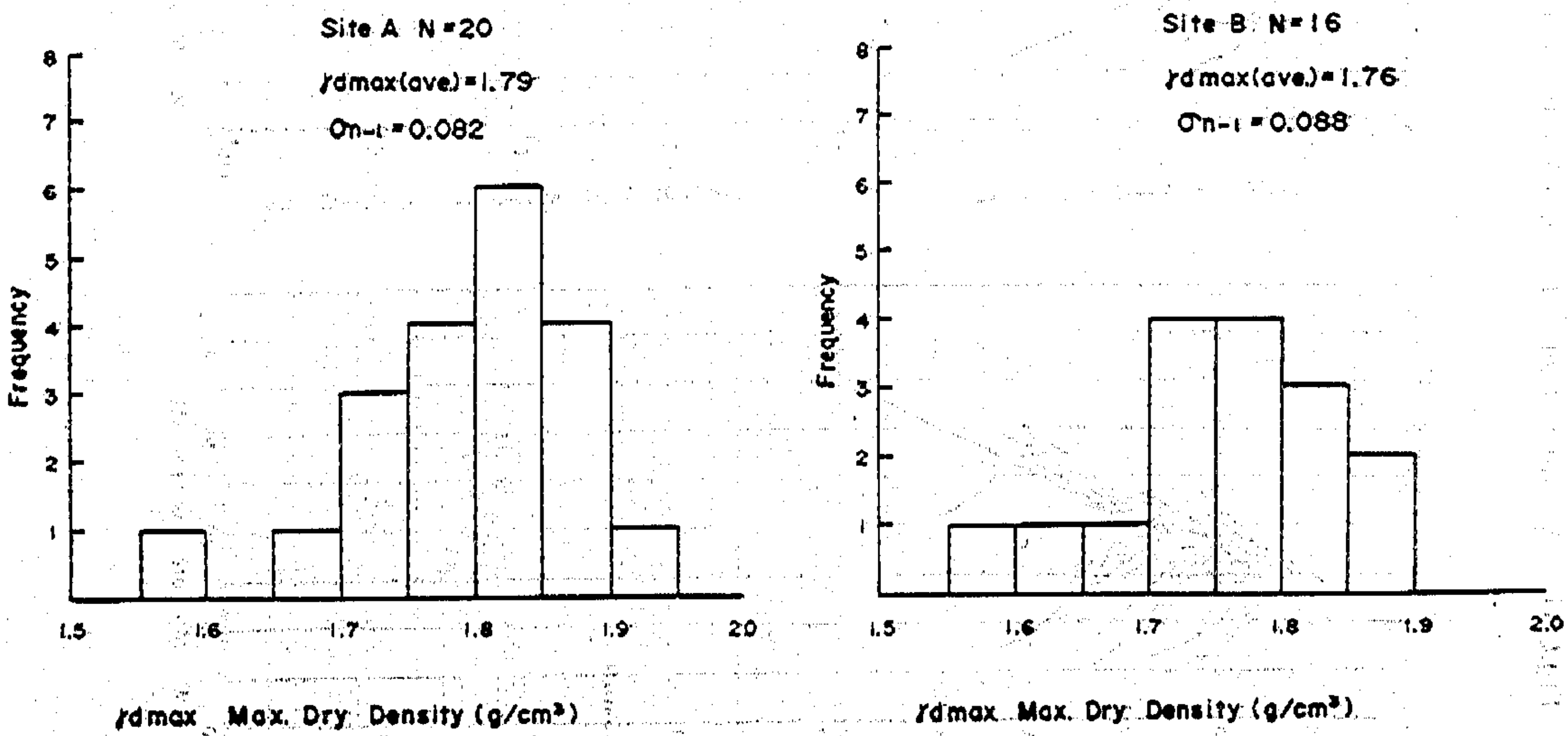


Fig.6.8.1 Relation of Wopt and γdmax

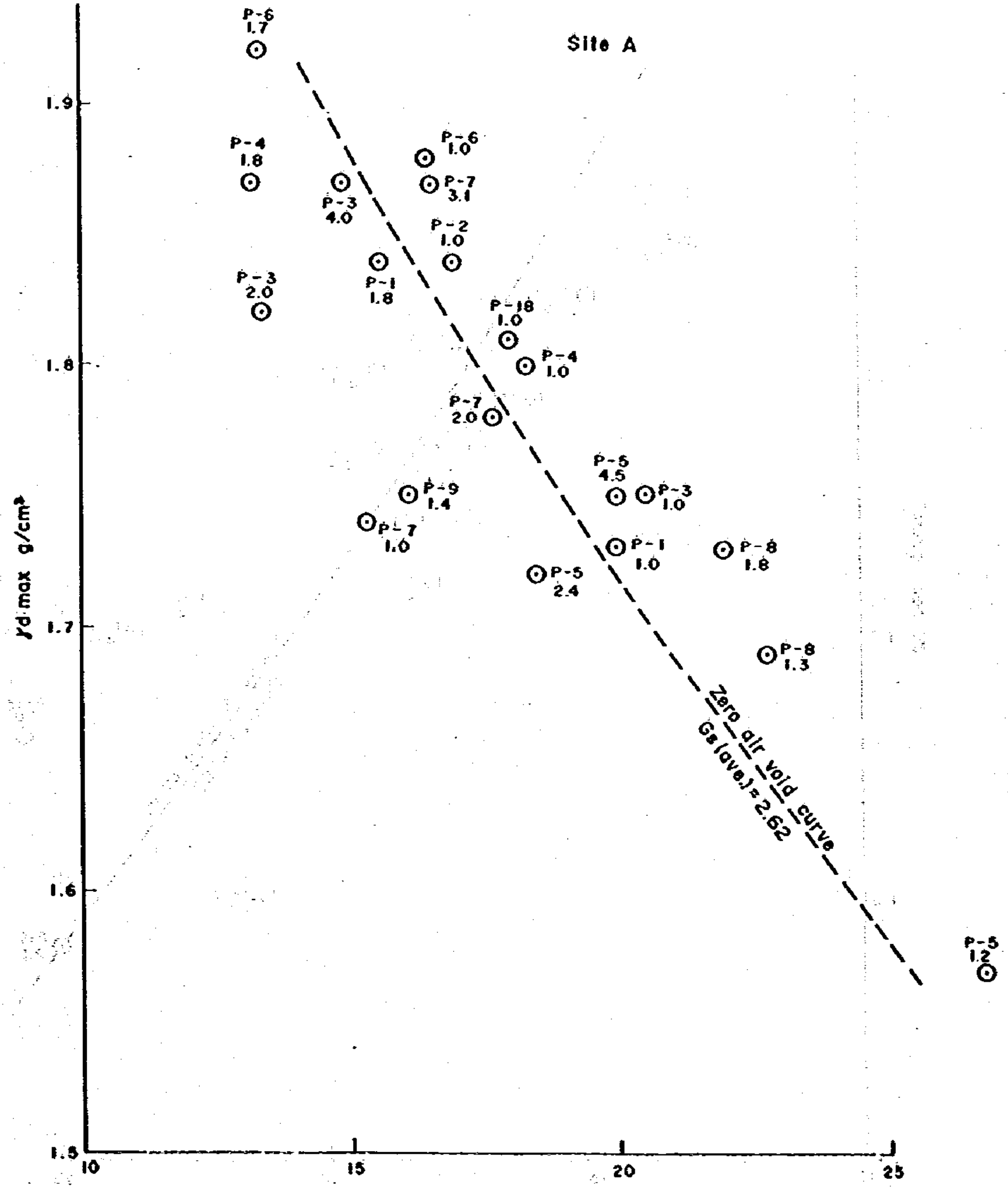


Fig.6.8.2 Relation of Wopt and γd max

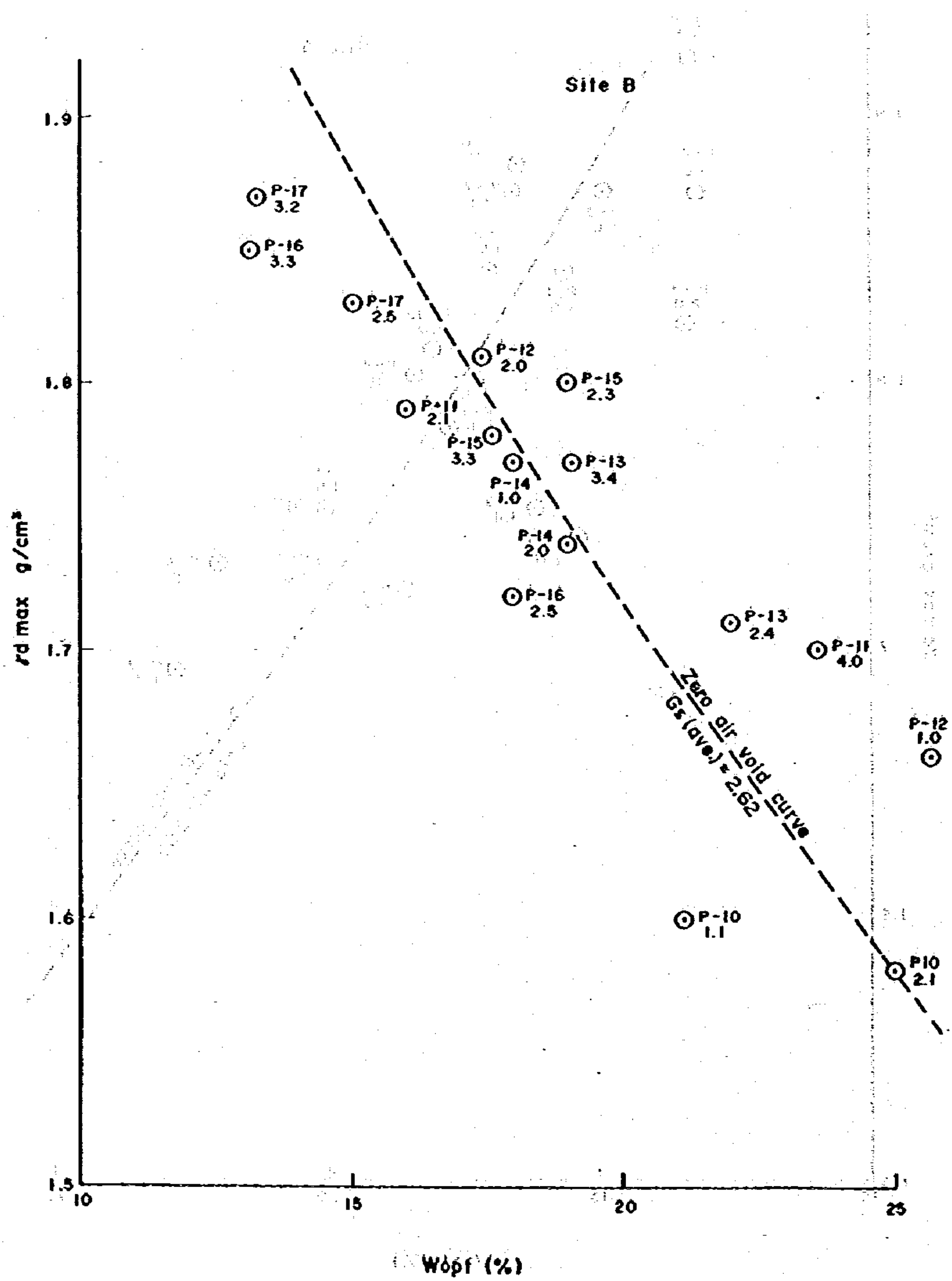


Fig.6.9 Wn ~ Wopt Relation

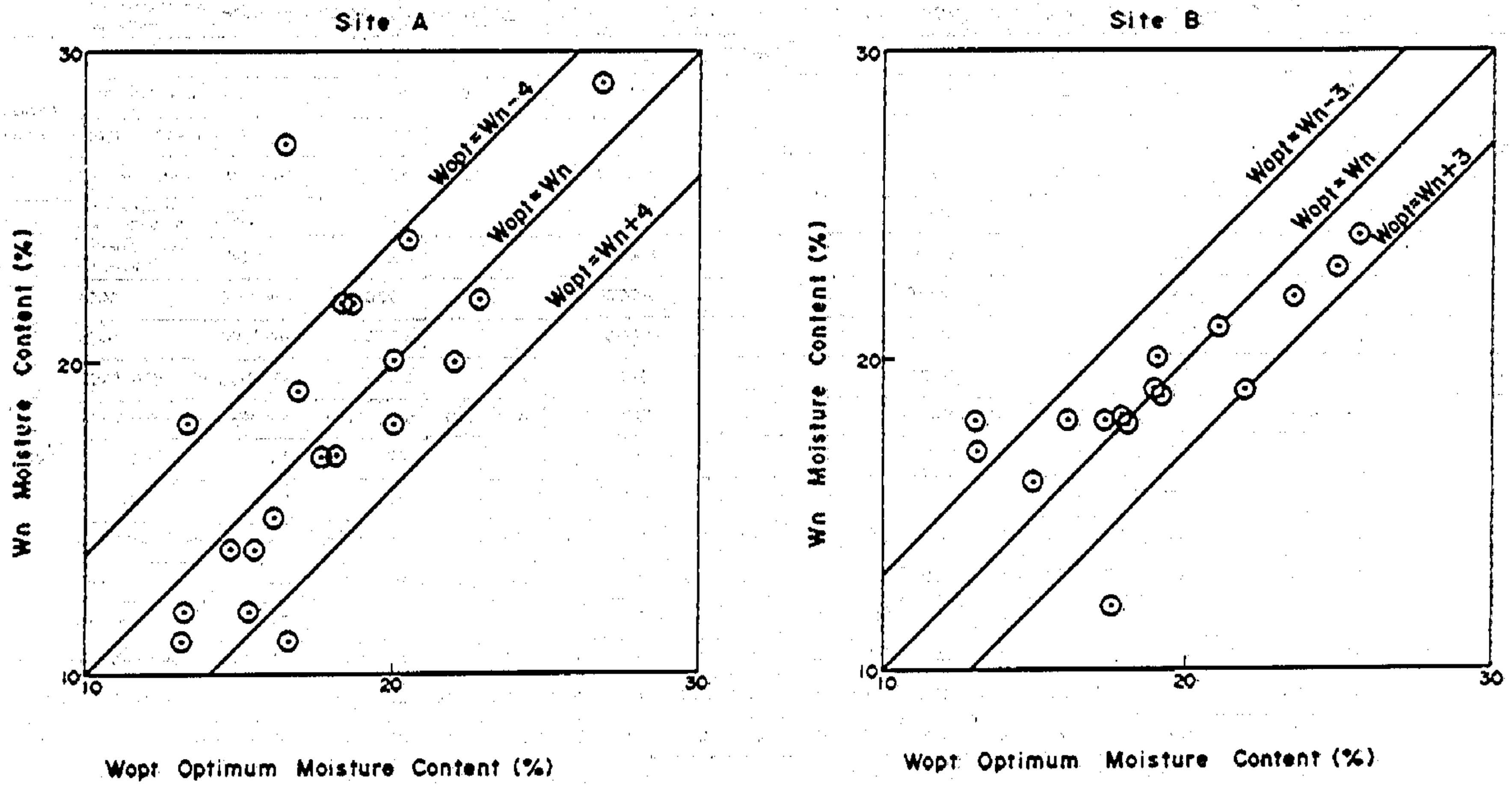


Table 6.3.1 Result of Soil Test (Upper Tekai Borrow Area (Site A))

Pit				P-1		P-2	P-3			P-4		P-5			P-6		P-7			P-8		P-9	
Depth			m	1.0	1.8	1.0	1.0	2.0	4.0	1.0	1.8	1.2	2.4	4.5	1.0	1.7	1.0	2.0	3.1	1.3	1.8	1.4	1.9
Specific Gravity		GS		2.69	2.70	2.69	2.58	2.52	2.68	2.58	2.59	2.61	2.69	2.51	2.55	2.59	2.62	2.55	2.55	2.68	2.70	2.70	2.65
Moisture Content		Wn	%	18	14	19	24	18	14	22	11	29	22	20	11	12	12	17	27	22	20	15	17
Grading	Gravel		%	6	1	14	1	0	0	0	1	2	2	1	2	9	4	0	4	28	37	38	4
	Sand		%	13	8	14	37	47	66	52	50	19	37	47	5	10	23	33	27	11	6	5	12
	Silt and Clay		%	81	91	72	62	53	34	48	49	79	61	52	93	81	73	67	67	61	57	57	84
	(Silt)		%		58				20					33				42					
	(Clay)		%		33				14					19				25					
	Max. Diameter	Dmax	mm	6.7	3.35	6.7	3.35	1.18	1.18	1.18	4.75	6.7	6.7	3.35	4.75	6.7	4.75	0.60	6.7	6.7	6.7	6.7	4.75
	60% grain size	D60	mm		0.019				0.13					0.090				0.062					
	30% 〃	D30	mm		0.0037				0.0058					0.016				0.0076					
	10% 〃	D10	mm		0.0011				0.0014					0.002				0.0013					
	Uniformity -Coefficient	UC			17.3				92.9					45.0				47.7					
	Coefficient of Curvature	UC'			0.73				18.5					1.42				0.72					
Consis -tency	Liquid Limit	Wl	%		34	48			25		26			36		32		35			41		35
	Plastic Limit	Wp	%		20	35			18		21			21		19		17			20		20
	Plasticity Index	Ip			10	13			7		5			15		13		18			21		15
	Classification				CL	OL			SC		CL-ML			CL		CL		CL			CL		CL
Moisture-Density Relations	Optimum Moisture Content	wopt	%	20.0	15.5	16.9	20.5	13.3	14.7	18.3	13.1	26.8	18.5	20.0	16.6	13.2	15.3	17.7	16.5	22.8	22.0	16.1	18.0
	Wopt − Wn		%	+2.0	+1.5	−2.1	−3.5	−4.7	+0.7	+3.7	+2.1	−2.2	−3.5	±0.0	+5.6	+1.2	+3.3	+0.7	−10.5	+0.8	+2.0	+1.1	+1.0
	Max. Dry Density	γd max	g/cm³	1.73	1.84	1.84	1.75	1.82	1.87	1.80	1.87	1.57	1.72	1.75	1.88	1.92	1.84	1.78	1.87	1.69	1.73	1.75	1.81
	Void Ratio	e		0.56	0.47	0.46	0.47	0.39	0.43	0.43	0.39	0.66	0.56	0.43	0.36	0.35	0.42	0.43	0.36	0.59	0.56	0.54	0.46
	Degree of Saturation	Sr	%	97	90	98	100	87	91	100	88	100	88	100	100	98	95	100	100	100	100	80	100
Coefficient of Permeability		Kn	$\times 10^{-7}$ cm/sec	4.33	4.07	6.92	500	3.85	4.04	5.46	5.32	8.14	7.04	4.26	3.85	4.47	5.39	5.04	4.33	1.15	1.21	4.94	3.46

Table 6.3.2 Result of Soil Test (Upper Tekai Quarry and Borrow Area (Site B))

	Pit			P-10		P-11		P-12		P-13		P-14		P-15		P-16		P-17					
Depth			m	1.1	2.1	2.1	4.0	1.0	2.0	2.4	3.4	1.0	2.0	2.3	3.3	2.5	3.3	2.5	3.2				
Specific Gravity		GS		2.51	2.50	2.59	2.52	2.65	2.68	2.59	2.70	2.62	2.63	2.60	2.68	2.69	2.68	2.60	2.68				
Moisture Content		Wn	%	21	23	18	22	24	18	19	20	18	19	19	12	18	18	16	17				
Grading	Gravel		%		1	0	0	3	2	20	13		13	27	2	4	3	0	8				
	Sand		%		39	48	36	9	13	23	15		9	20	11	27	26	63	51				
	Silt and Clay		%		60	52	64	88	85	57	72		78	53	87	69	71	37	41				
	(Silt)		%		15		28		41		22		49		42		19		17				
	(Clay)		%		45		36		44		50		29		45		52		24				
	Max. Diameter	Dmax	mm		4.75	1.18	2.36	4.75	6.7	6.7	6.7		6.7	6.7	6.7	6.7	6.7	2.36	6.7				
	60% grain size	D60	mm																				
	30% 〃	D30	mm																				
	10% 〃	D10	mm																				
	Uniformity Coefficient	UC																					
	Coefficient of Curvature	UC'																					
Consistency	Liquid Limit	Wl	%		65		47		49		54		49		55		71		34				
	Plastic Limit	Wp	%		26		19		22		25		33		23		22		30				
	Plasticity Index	Ip			39		28		27		29		16		32		49		4				
	Classification				CH		CL		CL		CH		OL		CH		CH		SM				
Moisture-Density Relations	Optimum Moisture Content	wopt	%	21.1	25.0	16.2	23.6	25.7	17.4	22.0	19.1	19.0	18.0	17.6	17.5	18.0	13.1	15.0	13.2				
	Wopt − Wn		%	+0.1	+20	−1.8	+1.6	+1.7	−0.6	+3.0	−0.9	+1.0	−1.0	−1.4	+5.5	±0.0	−4.9	−1.0	−3.8				
	Max. Dry Density	$\gamma_{d\,max}$	g/cm³	1.60	1.58	1.79	1.70	1.66	1.81	1.71	1.77	1.74	1.81	1.78	1.86	1.72	1.85	1.83	1.87				
	Void Ratio	e		0.57	0.58	0.48	0.58	0.60	0.48	0.52	0.53	0.51	0.45	0.46	0.44	0.56	0.45	0.42	0.43				
	Degree of Saturation	Sr	%	93	100	89	100	100	97	100	98	98	100	99	100	86	78	93	82				
Coefficient of Permeability		Kn	$\times 10^{-7}$ cm/sec	1.23	2.66	5.76	1.46	1.25	1.24	1.10	1.44	1.29	1.69	1.41	1.45	1.38	1.92	1.22	5.53				

6.6 SUITABILITY OF CORE MATERIAL

Generally speaking, the following properties are required for core material:

1) Impermeability
2) Density and shear strength
3) Resistance against seepage
4) No expansive components
5) Workability and traficability
6) No organic substances
7) Others

We will discuss each of the above subjects in the following pages.

6.6.1 Impermeability

As a result of the permeability test, the coefficient of permeability was found to be 10^{-7} cm/sec order in both A and B Sites at natural moisture content and regular moisture-density relationship. This represents a satisfactory level of impermeability. Moreover, at the time of actual construction, the moisture content will be adjusted to an optimal level and compaction will be performed more solidly than at the time of testing, causing the coefficient of permeability to further decline. These factors lead us to the conclusion that there will be no problem at all concerning impermeability.

6.6.2 Density and Shear Strength

As a result of the moisture-density relationship test, the maximum dry density was found to be 1.79 g/cm^3 in Site A and 1.76 g/cm^3 in Site B. In view of Unified Soil Classification Chart (Table 6.2), these values are not low. The core material in both sites is thus considered well usable.

The shear strength is considered to be satisfactorily high, since it normally increases in proportion to the dry density. The values of the cohesion strength and internal friction angle turned out to be $C = 0.9$ kg/cm^2 and $\phi = 14.5°$ respectively as a result of the triaxial compression test on an undisturbed specimen (P-5, 2.4 m, 35 cm in specimen diameter) collected from Site A. The dry density of this sample is 1.47 g/cm^3, considerably lower than that of core material (1.77 g/cm^3). At the time of actual construction, therefore, the shear strength of core material is expected to be greater than at the time of testing.

6.6.3 Resistance against Seepage

Seepage causes perticles of core material run out. This phenomenon is affected by the velocity of seepage, size of particles, strength of cohesion, interlocking, and so on. The velocity of seepage is normally defined as follows:

$V = Ki$ V: Velocity of seepage (cm/s)

k: Coefficient of permeability (1.0×10^{-6} cm/s).

i: Hydraulic grade line $= H/L$

H: Dam height (100m)

L: Core base thichness (60m)

When the permeability coefficient for designing core material is set at 1.0×10^{-6} cm/s, the velocity of seepage is calculated as follows:

$$V = 1.0 \times 10^{-6} \text{ cm/s} \times 100\text{m}/60\text{m} = 1.67 \times 10^{-6} \text{ cm/s}$$

The above velocity is considered to be within the range of safety since it is sufficiently lower than the value of critical velocity which is calculated by the Justin Formula below:

Diameter of Particle (mm)	Critical Velocity (cm/sec)
5.00	22.86
3.00	17.71
1.00	10.22
0.80	9.14
0.50	7.23
0.30	5.60
0.10	3.23
0.08	2.89
0.05	2.29
0.03	1.77
0.01	1.02

6.6.4 No expansive components

From among the core material collected at Sites A and B, we conducted an X-ray analysis on clay mineral which was assumed to be of an expansive nature. This analysis was intended for studying the expansion of the crystal lattices of the clay minerals both in a natural state as well as during expansion (when water is added).

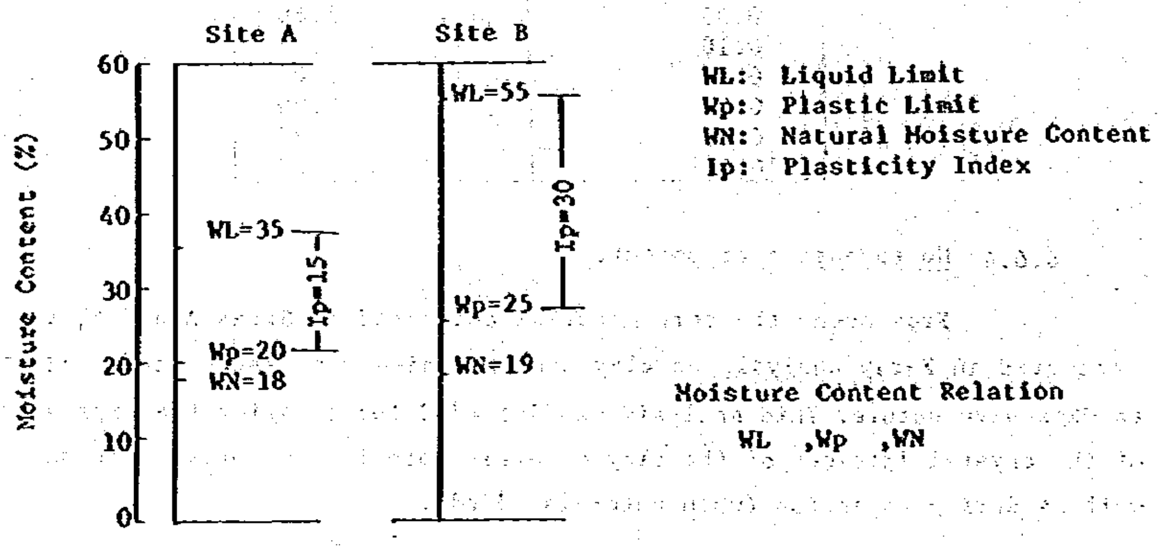
As shown in the X-Ray Analysis Chart (Fig. 6.10)*, the sample used in this analysis did not contain clay mineral of an expasive nature (montmorillonite, kaolinite, halloysite, etc.). This implies that, as evident from Table 6.5, no expansion was seen in the sample. Therefore, the sample should present no problem as core material.

Table 6.5 Results of X-Ray Analysis

Site	Pit No.	Test No.	Lattice spacing x10^{-8}cm Natural (A)	Poured water (B)	$\frac{(B)-(A)}{(B)}$ (%) (C)	$(1+(C))^3$	Remarks
Site A	P-3	1	7.18	7.18	0.000	1.000	Chlorite
		2	9.97	9.97	0.000	1.000	Sericite
	P-8	3	7.12	7.17	0.702	1.021	Chlorite
		4	9.95	9.97	0.201	1.006	Sericite
	P-9	5	7.15	7.15	0.000	1.000	Chlorite
		6	9.97	9.97	0.000	1.000	Sericite
Site B	P-10	7	7.19	7.19	0.000	1.000	Chlorite
		8	9.95	9.97	0.201	1.006	Sericite
	P-13	9	7.18	7.18	0.000	1.000	Chlorite
	P-14	10	7.17	7.17	0.000	1.000	Chlorite

6.5.5 Workability and traficability

The natural moisture content and the Atterberg Limit in Sites A and B are shown below:



Moisture Content Relation

WL ,Wp ,WN

As the schematic chart makes clear, there is little difference between natural moisture content and plastic limit. In the case of fine-grained soil, the optimum moisture content is nearly the same as the plastic limit in general, indicating that the soil is well compacted. It is expected, therefore, that good traficability will be ensured for road ,banking area and borrow site at the time of normal construction work.

6.6.6 No organic substance

In the test pits, top soil and organic soil form a layer about 1 meter thick, as illustrated in Fig. 4.15. For this reason, tests were conducted on core material taken from the weathered zone, after removing these top and organic soils. Among the samples collected from the pits, we found that only p-7/2.00m and P-14/2.00m contained OL (organic silt in terms of soil classification), but other samples contained no organic substance at all. This indicates that the weathered zone contains little organic substance and therefore is suitable for cor material.

6.6.7 Others

The following observations are based on actual results.

(1) Soil Classification

Generally speaking, soil has the following features, according to the "Earth Manual" (Bureau of Reclamation, Second Edition, 1974) given in Table 6.6:

- Site A is composed of CL, which attains adequate impermeability and strength after compaction.

- Site B is composed of CL and CH. The latter attains adequate impermeability after a compaction, but further studies will be required to determine their strength.

(2) Grading Distribution

Shown in Fig. 6.11 is an example of grading distribution comparing the actual results in Sites A and B with those in foreign countries.

As is obvious in Fig. 6.11, the comparison with actual results in other areas shows that Tekai is well qualified as a source of core material.

(3) Various Properties of Core Material

Table 6.7 shows various properties of core material available from various parts of the world. It is clear from this table that properties of core material from Tekai are more or less equal to those from other areas.

The proposed borrow sites contain soils of CL and CH classification. Therefore, from the standpoint of soil classification, the sites are also suitable.

Table 6.6 EARTH MANUAL PROPERTIES OF SOILS

TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	IMPORTANT ENGINEERING PROPERTIES: PERMEABILITY WHEN COMPACTED	IMPORTANT ENGINEERING PROPERTIES: SHEAR STRENGTH WHEN COMPACTED AND SATURATED	IMPORTANT ENGINEERING PROPERTIES: COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	RELATIVE DESIRABILITY FOR VARIOUS USES (NO. 1 IS CONSIDERED THE BEST): ROLLED EARTHFILL DAMS: HOMOGENEOUS EMBANKMENT	ROLLED EARTHFILL DAMS: CORE	ROLLED EARTHFILL DAMS: SHELL	CANAL SECTIONS: EROSION RESISTANCE	CANAL SECTIONS: COMPACTED EARTH LINING	FOUNDATIONS: SEEPAGE IMPORTANT	FOUNDATIONS: SEEPAGE NOT IMPORTANT	ROADWAYS: FILLS: FROST HEAVE NOT POSSIBLE	ROADWAYS: FILLS: FROST HEAVE POSSIBLE	ROADWAYS: SURFACING
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	-	-	1	1	-	-	1	1	1	3
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD		-	2	2			3	3	3	
SILTY GRAVELS, POORLY GRADED SAND-SILT MIXTURES	GM	SEMI PERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	2	4	-	4	4	1	4	4	9	5
CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES.	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1	1	-	3	1	2	6	5	5	1
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	-	-	3 IF GRAVELLY	6	-	-	2	2	2	4
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.	SP	PERVIOUS	GOOD	VERY LOW	FAIR		-	4 IF GRAVELLY	7 IF GRAVELLY		-	5	6	4	
SILTY SANDS, POORLY, GRADED SAND-SILT MIXTURES.	SM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	4	5	-	8 IF GRAVELLY	5 EROSION CRITICAL	3	7	8	10	6
CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES.	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	3	2	-	5	2	4	8	7	6	2
INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY.	ML	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	6	6	-	-	6 EROSION CRITICAL	6	9	10	11	-
INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS SILTY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	5	3	-	9	3	5	10	9	7	7
ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY.	OL	SEMI-PERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR	8	8	-	-	7 EROSION CRITICAL	7	11	11	12	-
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS.	MH	SEMI-PERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	9	9	-	-	-	8	12	12	13	-
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS.	CH	IMPERVIOUS	POOR	HIGH	POOR	7	7	-	10	8 VOLUME CHANGE CRITICAL	9	13	13	8	-
ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	10	10	-	-	-	10	14	14	14	-
PEAT AND OTHER HIGHLY ORGANIC SOILS.	PT	-	-	-	-	-	-	-	-	-	-	-	-	-	-

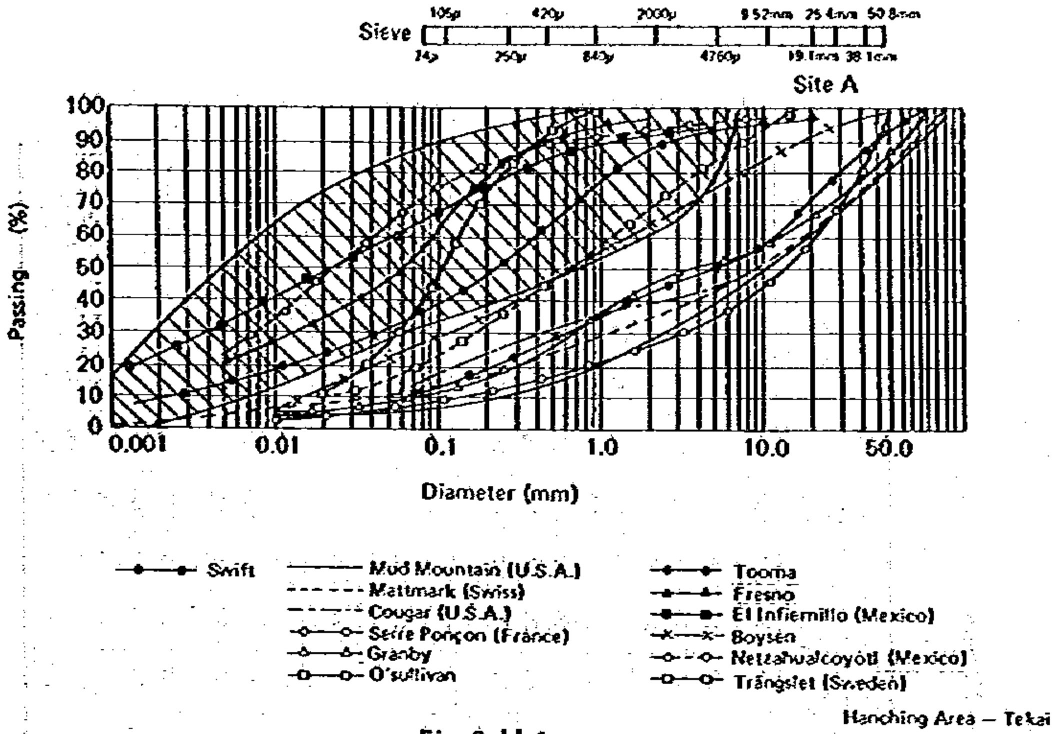


Fig. 6.11.1

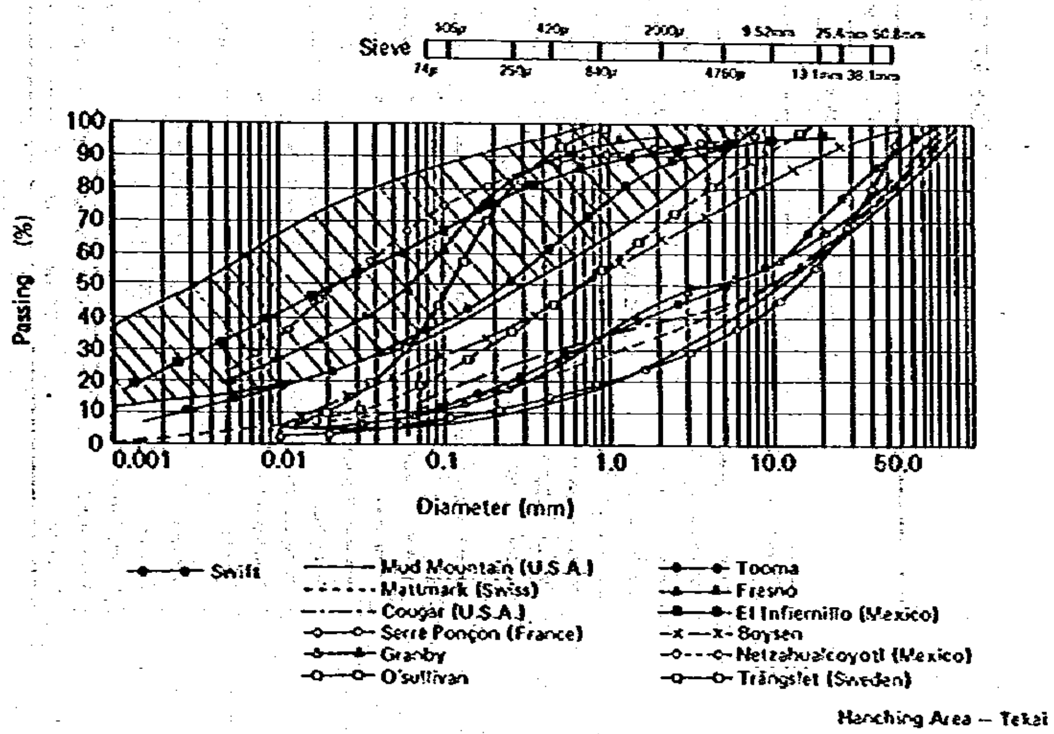


Fig. 6.11.2

Table 6.7 Example of Core Material

	Unified Soil Classification	Gs	Moisture Content: Wn (%)	Moisture Content: Wopt (%)	Moisture Content: Construction	Max. Dry Density δdmax (g/cm³)	Atterberg: Wl (%)	Atterberg: Wp (%)	Atterberg: PI	Grading (mm): 75 (%)	Grading (mm): 5 (%)	Grading (mm): 0.080 (%)	Grading (mm): 0.020 (%)	Permeability k (cm/sec)
Oroville (U.S.A.)	CL, CL-ML		8				28	21	7	100	56	25	8	
Kremasta (Greek)	ML	2.69		16	Wopt +1		27	19	8		95	75	25	5×10^{-8}
Göschenenalp (Swiss)		2.72	6	6	Wopt 0	2.16	20	15	5	93	52	24	5	5×10^{-8}
Gepatsch (Austria)	CL	2.66	8.8	8.8	Wopt 0		31	21	10	100	50	20	1	2.5×10^{-7}
Swift (U.S.A.)	SC-SM		10	12	Wopt +2	1.9			5	92	50	12	1	2×10^{-8}
El Infiernillo (Mexico)	CL	2.76	23	19	Wopt +4	1.69	49	24	25	100	95	65	25	5×10^{-8}
Netzahualcoyotl (Mexico)	ML		25	18	Wopt +7	1.68	55	35	20	100	97	71	15	
Cougar (U.S.A.)	ML		15	14	Wopt +1	1.89	40	30	10	90	76	16	1	10^{-7}
Furnas (Brazil)	CH, CL		23				50	25	25	100	100	73	38	
Serre-Ponçon (France)	ML		14 11	12 10	Wopt +2 +1		26.4	19.7	6.7	95 97	73 54	41 34	7 12	3×10^{-7} 2×10^{-6}
Trinity (U.S.A.)	CL-ML		13	15	Wopt −2	1.90	28	22	6	100	36	25	7	
Akosombo	CL		14	13	Wopt +0.5		40	18	22					10^{-8}
Messaure (Sweden)	SC-SM				Wopt +4			22	5	100	85	26	3	5×10^{-6}
Ojatiluhur	CH	2.73	24	22	Wopt +2	1.63	64	26	38	100	81	64	30	10^{-8}
Hyttejuvet (Norway)	CL-ML	2.76	8	8	0	2.17	21	15	6	82	59	33	5	7×10^{-8}
Présidente Aleman (Mexico)	CH		24	21	Wopt +3		70			100	87	67	30	
Tooma	ML			18	Wopt −2~+1	1.73	30	27	9	100	93	36	5	0.5×10^{-6}
Shek Pik	CL		18	16	Wopt +2	1.76	37	25	12	100	95	60	20	2.5×10^{-7}
Gréoux	ML		11				27	17	10	100	70	50		
Round Butte (U.S.A.)			12.7		Wopt −1			~20	5	100	70	30		
Tekai Site A	CL	2.62	18	18	Wopt +4	1.79	35	20	15	100	90	67	40	4.5×10^{-7}
Tekai Site B	CL, CH	2.62	19	19	Wopt +3	1.76	55	25	30	100	95	65	40	1.5×10^{-7}

6.6.8 Conclusion

The foregoing findings and observations lead us to conclude that samples available from both Sites A and B are well qualified as core materials. These sites in particular, provide materials with degree of impermeability which is one of basic properties essential for a core material.

7. ROCK TESTING

7. ROCK TESTING

7.1 GENERAL

Rock tests, using drilling cores, were carried out in order to evaluate rock material for the upper dam and concrete aggregate for both the upper and lower dams. These tests resulted in the determination of the most suitable rock material and aggregate and the selection of a quarry site.

The study found that hard and fresh rock taken from Site B, a sandstone-rich area close to the upper dam site, would satisfy all conditions in terms of both quality and quantity.

7.2 TEST ITEMS

In general, the tests performed focused on those items usually covered when drilling cores are used. The tests listed below afforded a grasp of the rock's basic properties and grade of weathering, thus helping to determine its suitability as construction material.

1) Specific gravity, absorption
2) Unconfined compressive strength
3) Ultrasonic wave velocity

Quantities used in the tests are given in Table 7.1.

Table 7.1 Quantity of Samples

Site	Sample			
	Sandstone	Shale	Limestone	Total
Upper Dam Site	19	3	0	22
Site A	2	2	0	4
Site B	10	0	0	10
Lower Dam Site	14	6	0	20
Site C	11	1	0	12
Site D	1	0	3	4
Total	57	12	3	72

7.3 TEST METHODS

The rock tests listed below were performed according to specifications of the American Society for Testing and Materials. (A.S.T.M.)

1) Specific Gravity and Absorption
ASTM C127-81
"Specific Gravity and Absorption of Coarse Aggregates"

2) Unconfined Compressive Strength
ASTM D2938
"Test for Unconfined Compressive Strength of Intact Rock Core Specimens"

3) Ultrasonic Wave Velocity
ASTM D2845
"Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock"

7.4 TEST RESULTS

Test results are given in Tables 7.2.1 and 7.2.2. Frequency distribution of each test, classfied according to site and type of rock, are shown in Fig. 7.1 through Fig. 7.7.

Frequency distribution of each test for the entire Tekai Site are shown in Fig. 7.8 through Fig.7.14.

Modules of Elasticity Dynamic (E) and Poissons Ratio Dynamic (ν) is calculated from the following formula (Vp and Vs of the formula is obtained by Hypersonic Testing).

E: Modulus of Elasticity Dynamic (kg/cm2)
ν: Poissons Ratio Dynamic
rs: Dry density (g/cm^3)
Vp (m/s)
Vs (m/s)

$$\nu = \frac{(\frac{Vp}{Vs})^2 - 2}{2((\frac{Vp}{Vs})^2 - 1)}$$

$$E = \frac{2(1+\nu)\ rs\ (Vs)^2}{9.8 \times 10}$$

i) Specific Gravity, Absorption

Fig. 7.1 shows the frequency distribution of dry specific gravity at each dam site and each quarry site. Fig. 7.8 shows the frequency distribution of dry specific gravity for each type of rock. According to Fig. 7.8, the specific gravity of limestone is the highest at 2.701 ± 0.003. Sandstone has a specific gravity of 2.546 ± 0.094, while shale has the lowest at 2.384 ± 0.152.

Fig. 7.2 shows the frequency distribution of absorption at each dam site and each quarry site. Fig. 7.9 shows the frequency distribution of absorption for each type of rock. According to Fig. 7.9, limestone has the lowest absorption rate at 0.15 ± 0.06%. Sandstone has an absorption rate of 1.58 ± 1.35%, while shale has the highest at 3.59 ± 1.52%.

ii) Unconfined Compressive Strength

Fig. 7.3 shows the frequency distribution of unconfined compressive strength for rock specimens occurring at each dam site and each quarry site. Fig. 7.10 shows the frequency distribution of unconfined compressive strength for each type of rock. According to Fig. 7.10, shale has the lowest frequency of unconfined compressive strength at 279 ± 208 kgf/cm^2. The frequency of unconfined compressive strength of limestone is nearly fixed at 954 ± 149 kgf/cm^2, while that of sandstone shows great disparity at 1,230 ± 781 kgf/cm^2. With the exception of some of the shale, these rock specimens exhibit good conditions for the foundation of a dam.

iii) Ultrasonic Wave Velocity

Figs. 7.6 and 7.7 show the P wave velocity and S wave velocity of rock specimens taken from each dam site and each quarry site. Figs. 7.13 and 7.14 show the frequency distribution of the P wave velocity and S wave velocity of each type of rock. Limestone has a high P wave velocity of 6,627 ± 92 m/sec. The P wave velocity of sandstone is 4,228 ± 1,008 m/sec, a normal value for Mesozoic sedimentary rock. The P wave velocity of shale is a relatively low 3,309 ± 977 m/sec. The relationship of unconfined compressive strength and P wave velocity is shown in Fig. 7.17. The unconfined compressive strength is low, while its P wave velocity is low, suggesting that the shale in this area has a great deal of cleavage and is thus, in general, unsuitable as rock material.

Figs. 7.4 and 7.5 show the dynamic modulus of elasticity and Poisson ratio of each dam site and each quarry site, calculated by means of P wave velocity, S wave velocity, and density. The dynamic modulus of elasticity and the Poisson ratio of each type of rock are shown in Fig. 7.11 and Fig. 7.12 respectively.

Table 7.2.1 RESULT OF ROCK TESTS (UPPER TEKAI SITE)

Site	Sample No.	Hole No.	Depth (m)	Rock		Weathering	P. of W.A. (%)	Specific Gravity			U.C.S. (kgf/cm²)	Ultrasonic Wave Velocity (x10³ m/sec)		M. of E. Dynamic (x10⁵ kgf/cm²)	Poisson Ratio Dynamic	Remarks	
								S.S.	S.A.	D.S.		P wave	S wave				
Upper Tekai Dam Site	UR- 1	U - 5	26.40~26.55	Sandstone	(Medium sandstone)	Slightly weathered	0.60	2.649	2.646	2.633	918	5.353	2.940	60.2	0.28	Intake	Dam site
	UR- 2	UD- 4	14.30~14.45	Sandstone	(Fine quartzose sandstone)	Moderately weathered	2.52	2.559	2.528	2.496	924	3.268	1.527	16.6	0.36	Dam center	
	UR- 3	UD- 5	9.14~ 9.30	Sandstone	(Fine quartzose sandstone)	Moderately weathered	1.48	2.616	2.605	2.578	1,185	4.348	2.078	31.3	0.35		
	UR- 4	UD- 6	4.13~ 4.27	Sandstone	(Medium sandstone)	Moderately weathered	0.85	2.625	2.618	2.603	2,109	4.827	2.525	42.5	0.31		
	UR- 5	UD- 7	5.23~ 5.38	Sandstone	(Coarse quartzose sandstone)	Highly weathered	0.99	2.427	2.459	2.447	580	4.419	2.190	33.2	0.34		
	UR- 6	UD- 8	4.55~ 4.70	Sandstone	(Coarse quartzose sandstone)	Moderately weathered	1.15	2.592	2.579	2.562	1,565	3.504	1.728	20.9	0.34	Coffer dam	
	UR- 7	UD- 1	3.17~ 3.32	Sandstone	(Medium quartzose sandstone)	Highly weathered	1.99	2.447	2.430	2.399	56.2	2.614	1.190	10.0	0.37		
	UR- 8	UD- 2	6.32~ 6.47	Sandstone	(Medium quartzose sandstone)	Moderately weathered	0.80	2.641	2.636	2.620	1,630	4.828	2.623	48.0	0.29		
	UR- 9	UD- 3	7.55~ 7.70	Sandstone	(Coarse quartzose sandstone)	Slightly weathered	1.21	2.563	2.548	2.532	684	4.588	2.531	43.4	0.28		
	UR-10	UD- 9	9.70~ 9.85	Sandstone	(Shaly sandstone)	Fresh	0.97	2.678	2.676	2.653	563	5.165	2.693	52.9	0.31		
	UR-11	UD- 4	23.05~23.20	Shale	(Sandy shale)	Moderately weathered	3.34	2.401	2.353	2.324	115	2.656	1.326	11.8	0.33	Spillway	
	UR-12	UD-14	6.30~ 6.47	Sandstone	(Fine sandstone)	Slightly weathered	1.32	2.668	2.660	2.633	456	4.845	2.674	49.9	0.28		
	UR-13	UD-15	15.00~15.15	Sandstone	(Shaly sandstone)	Slightly weathered	1.32	2.649	2.634	2.614	867	4.830	2.668	49.3	0.28		
	UR-14	UD-10	15.60~15.75	Sandstone	(Medium quartzose sandstone)	Moderately weathered	0.91	2.610	2.599	2.586	1,089	4.604	2.624	45.8	0.26	Diversion tunnel	
	UR-15	UD-11	28.08~28.24	Shale	(Shale)	Slightly weathered	3.26	2.480	2.438	2.401	390	3.208	1.506	15.2	0.36		
	UR-16	UD-11	45.25~45.50	Sandstone	(Medium sandstone)	Slightly weathered	0.73	2.624	2.615	2.605	2,030	5.379	3.018	62.2	0.27		
	UR-17	UD-12	25.38~25.50	Sandstone	(Shaly sandstone)	Very fresh	1.63	2.649	2.639	2.606	613	4.821	2.731	51.6	0.26		
	UR-18	UD-13	17.22~17.40	Sandstone	(Medium sandstone)	Slightly weathered	1.11	2.623	2.610	2.595	880	4.858	2.715	50.1	0.27		
	UR-19	UD-16	30.10~30.25	Sandstone	(Shaly sandstone)	Slightly weathered	1.28	2.664	2.653	2.631	661	5.005	2.834	54.6	0.26	Power tunnel	
	UR-20	UD-17	39.00~39.15	Shale	(Silty shale)	Very fresh	0.77	2.712	2.711	2.691	574	4.858	2.305	40.6	0.35		
	UR-21	UD-18	2.15~ 2.25	Sandstone	(Medium quartzose sandstone)	Moderately weathred	0.58	2.618	2.609	2.603	2,462	5.653	3.053	64.9	0.29	Power station	
	UR-22	UD-18	13.80~13.95	Sandstone	(Coarse quartzose sandstone)	Slightly weathered	1.00	2.578	2.563	2.553	1,641	4.802	2.753	49.5	0.26		
Quarry Site B	UR-23	UQ- 1	23.05~23.20	Sandstone	(Medium quartzose sandstone)	Highly weathered	1.39	2.589	2.568	2.554	1,283	2.436	1.209	10.1	0.34	Quarry area	
	UR-24	UQ- 1	49.56~49.73	Sandstone	(Medium sandstone)	Slightly weathered	0.72	2.617	2.608	2.559	2,086	4.386	2.257	35.8	0.32		
	UR-25	UQ- 2	15.27~15.45	Sandstone	(Medium quartzose sandstone)	Moderately weathered	0.96	2.614	2.602	2.589	1,986	3.727	1.865	24.9	0.33		
	UR-26	UQ- 2	42.00~42.17	Sandstone	(Coarse quartzose sandstone)	Fresh	0.44	2.649	2.648	2.638	2,518	5.917	4.368	79.8	0.24		
	UR-27	UQ- 3	14.70~14.80	Sandstone	(Medium quartzose sandstone)	Moderately weathered	1.03	2.439	2.426	2.414	1,718	3.231	1.791	21.5	0.28		
	UR-28	UQ- 3	48.00~48.15	Sandstone	(Medium quartzose sandstone)	Slightly weathered	1.03	2.605	2.592	2.578	672	3.956	1.791	22.0	0.38		
	UR-29	UQ- 4	5.85~ 6.00	Sandstone	(Coarse quartzose sandstone)	Highly weathered	1.17	2.477	2.458	2.448	754	1.294	0.594	2.45	0.37		
	UR-30	UQ- 4	50.00~50.17	Sandstone	(Medium quartzose sandstone)	Slightly weathered	0.33	2.643	2.642	2.635	2,592	5.840	3.403	78.3	0.24		
	UR-31	UQ- 5	25.20~25.35	Sandstone	(Medium quartzose sandstone)	Slightly weathered	1.23	2.622	2.612	2.591	488	4.325	2.258	35.9	0.31		
	UR-32	UQ- 5	37.10~37.13	Sandstone	(Medium quartzose sandstone)	Slightly weathered	0.70	2.637	2.628	2.618	2,688	4.862	2.809	52.5	0.25		
Quarry Site A	UR-33	UB- 1	19.00~19.13	Shale	(Shale)	Slightly weathered	4.35	2.838	2.320	2.283	165	2.427	1.283	10.2	0.31	Borrow area	
	UR-34	UB- 2	14.45~14.58	Sandstone	(Medium sandstone)	Moderately weathered	1.23	2.609	2.599	2.577	1,373	4.649	2.562	44.5	0.28		
	UR-35	UB- 3	15.56~15.70	Shale	(Shale)	Fresh	3.77	2.421	2.362	2.333	51.9	3.878	2.017	27.2	0.31		
	UR-36	UB- 4	28.70~28.85	Sandstone	(Medium quartzose sandstone)	Slightly weathered	1.61	2.599	2.575	2.558	301	4.156	2.076	30.7	0.33		

LEGEND
D.S. ..... Dried Specimen
S.S. ..... Soaked Specimen
S.A. .... Specimen in Air
P. of W.A. ..... Percentage of Water Absorption
U.C.S. ...... Unconfined Compression Strength
M. of E. ..... Modulus of Elasticity

Table 7.2.2 RESULT OF ROCK TESTS (LOWER TEKAI SITE)

Site	Sample No.	Hole No.	Depth (m)	Rock		Weathering	P. of W.A. (%)	Specific Gravity			U.C.S. (kgf/cm²)	Ultrasonic Wave Velocity($\times 10^3$ m/sec)		M. of E. Dynamic ($\times 10^4$ kgf/cm²)	Poisson Radio Dynamic	Remarks	
								S.S.	S.A.	D.S.		P wave	S wave				
Lower Tekai Dam Site	LR- 1	L - 1	16.05~16.18	Shale	(Shale)	Slightly weathered	3.72	2.582	2.538	2.489	206	3.012	1.455	14.8	0.35	Dam center	Dam site
	LR- 2	L - 1	20.50~20.71	Sandstone	(Medium sandstone)	Slightly weathered	1.62	2.589	2.571	2.548	1,158	3.750	1.944	25.8	0.32		
	LR- 3	L - 2	9.24~ 9.37	Sandstone	(Medium sandstone)	Fresh	0.33	2.620	2.617	2.611	2,902	5.761	3.503	78.9	0.21		
	LR- 4	L - 3	12.80~13.00	Sandstone	(Medium quartzose sandstone)	Slightly weathered	0.94	2.608	2.595	2.584	851	5.022	2.765	52.2	0.28		
	LR- 5	L - 4	16.80~17.00	Sandstone	(Shaly sandstone)	Slightly weathered	3.14	2.459	2.406	2.383	846	3.749	1.936	24.2	0.32		
	LR- 6	LD- 2	14.20~14.40	Shale	(Shale)	Slightly weathered	5.80	2.323	2.234	2.196	56.3	2.008	0.919	5.21	0.37		
	LR- 7	LD- 3	7.20~ 7.40	Sandstone	(Shaly sandstone)	Moderately weathered	1.89	2.611	2.504	2.563	961	4.748	2.252	37.3	0.35		
	LR- 8	LD- 3	11.57~11.73	Shale	(Silty shale)	Moderately weathered	1.98	2.646	2.629	2.595	436	3.942	1.728	22.4	0.38		
	LR- 9	LD- 4	14.83~15.00	Sandstone	(Fine quartzose sandstone)	Slightly weathered	0.24	2.640	2.637	2.633	3,222	5.491	2.795	54.6	0.33		
	LR-10	LD- 5	9.80~10.00	Sandstone	(Fine quartzose sandstone)	Moderately weathered	1.34	2.606	2.591	2.572	1,393	4.128	2.134	31.3	0.32		
	LR-11	LD- 6	8.05~ 8.20	Sandstone	(Medium quartzose sandstone)	Moderately weathered	1.82	2.564	2.536	2.518	1,197	2.229	1.168	9.42	0.31		
	LR-12	LD- 9	18.26~18.40	Sandstone	(Medium sandstone)	Slightly weathered	1.64	2.592	2.578	2.550	381	4.534	2.324	37.9	0.32	Spillway	
	LR-13	LD- 1	6.00~ 6.20	Shale	(Shale)	Slightly weathered	1.55	2.532	2.516	2.494	489	4.185	2.266	36.1	0.29	Coffer dam	
	LR-14	LD-10	8.70~ 8.80	Sandstone	(Fine quartzose sandstone)	Highly weathered	1.91	2.588	2.583	2.539	634	4.157	2.190	32.4	0.31		
	LR-15	LD-11	17.17~17.33	Sandstone	(Fine quartzose sandstone)	Moderately weathred	1.18	2.600	2.586	2.570	1,079	4.236	2.165	33.1	0.32	Diversion tunnel	
	LR-16	LD-12	42.78~42.95	Sandstone	(Medium sandstone)	Slightly weathered	0.88	2.639	2.634	2.616	2,050	5.230	2.695	51.2	0.32		
	LR-17	LD-13	18.80~18.95	Sandstone	(Medium sandstone)	Moderately weathered	2.22	2.567	2.543	2.511	536	3.527	1.766	21.6	0.33		
	LR-18	LD- 7	9.45~ 9.55	Shale	(Sandy shale)	Highly weathered	4.40	2.408	2.344	2.307	182	2.172	1.057	7.26	0.34	Penstock	
	LR-19	LD- 8	12.65~12.80	Sandstone	(Medium sandstone)	Slightly weathered	1.02	2.607	2.594	2.581	2.272	4.884	2.568	45.7	0.31	Power station	
	LR-20	LD-14	6.00~ 6.15	Shale	(Mudstone)	Highly weathered	5.32	2.367	2.305	2.247	73.6	2.265	0.969	3.10	0.39	Saddle	
Quarry Site C	LR-21	LQ- 1	14.85~15.00	Sandstone	(Fine sandstone)	Moderately weathered	2.88	2.535	2.494	2.464	754	2.864	1.389	1.30	0.35	Quarry area	
	LR-22	LQ- 1	25.60~25.75	Sandstone	(Fine quartzose sandstone)	Moderately weathered	1.54	2.583	2.559	2.544	1,780	3.679	2.020	27.7	0.28		
	LR-23	LQ- 1	41.20~41.35	Sandstone	(Medium quartzose sandstone)	Moderately weathered	2.23	2.572	2.543	2.516	336	4.058	2.032	29.1	0.33		
	LR-24	LQ- 2	15.30~15.46	Sandstone	(Medium quartzose sandstone)	Moderately weathered	1.26	2.594	2.575	2.562	2,129	3.680	1.945	25.6	0.31		
	LR-25	LQ- 2	29.10~29.25	Sandstone	(Medium quartzose sandstone)	Moderately weathered	2.04	2.574	2.548	2.523	513	3.651	1.917	25.0	0.31		
	LR-26	LQ- 2	47.60~47.80	Sandstone	(Medium quartzose sandstone)	Moderately weathered	1.23	2.605	2.595	2.574	1,449	4.051	2.261	34.4	0.27		
	LR-27	LQ- 3	14.70~14.90	Sandstone	(Shaly sandstone)	Highly weathered	8.86	2.257	2.102	2.074	-	-	-	-	-		
	LR-28	LQ- 3	21.00~21.15	Sandstone	(Medium quartzose sandstone)	Highly weathered	1.70	2.584	2.562	2.541	660	2.391	1.236	10.4	0.32		
	LR-29	LQ- 4	18.30~18.47	Sandstone	(Fine sandstone)	Moderately weathered	4.06	2.447	2.390	2.351	258	3.125	1.548	16.0	0.34		
	LR-30	LQ- 5	13.70~13.85	Shale	(Shale)	Highly weathered	4.83	2.365	2.297	2.256	613	1.855	0.802	4.16	0.39		
	LR-31	LQ- 5	30.45~30.65	Sandstone	(Medium sandstone)	Moderately weathered	2.38	2.578	2.549	2.518	217	3.818	2.025	28.0	0.30		
	LR-32	LQ- 6	21.60~21.75	Sandstone	(Medium quartzose sandstone)	Moderately weathered	1.46	2.594	2.574	2.557	1,717	5.215	2.955	58.8	0.26		
Quarry Site D	LR-33	DH- 1	17.20Ft ~17.80Ft	Limestone	(Limestone)	Fresh	0.09	2.706	2.705	2.704	863	6.725	3.920	105.5	0.24	Kg. Lubuk payong	
	LR-34	DH- 2	23.55Ft ~24.00Ft	Limestone	(Limestone)	Fresh	0.20	2.704	2.701	2.699	1,126	6.543	3.402	85.2	0.31		
	LR-35	DH- 3	43.60Ft ~44.10Ft	Limestone	(Limestone)	Fresh	0.16	2.704	2.701	2.699	873	6.613	3.487	86.9	0.31		
	LR-36	DH- 4	38.70Ft ~39.20Ft	Sandstone	(Sandstone)	Moderately weathered	5.98	2.543	2.468	2.399	205	2.289	1.251	10.5	0.29		

LEGEND
D.S. ..... Dried Specimen S.A. .... Specimen in Air U.C.S. .... Unconfined Compression Strength
S.S. ..... Soaked Specimen P. of W.A. ..... Percentage of Water Absorption M. of E. ..... Modulus of Elasticity

Fig.7.1 Specific Gravity (Dried Specimen)

(Gs)

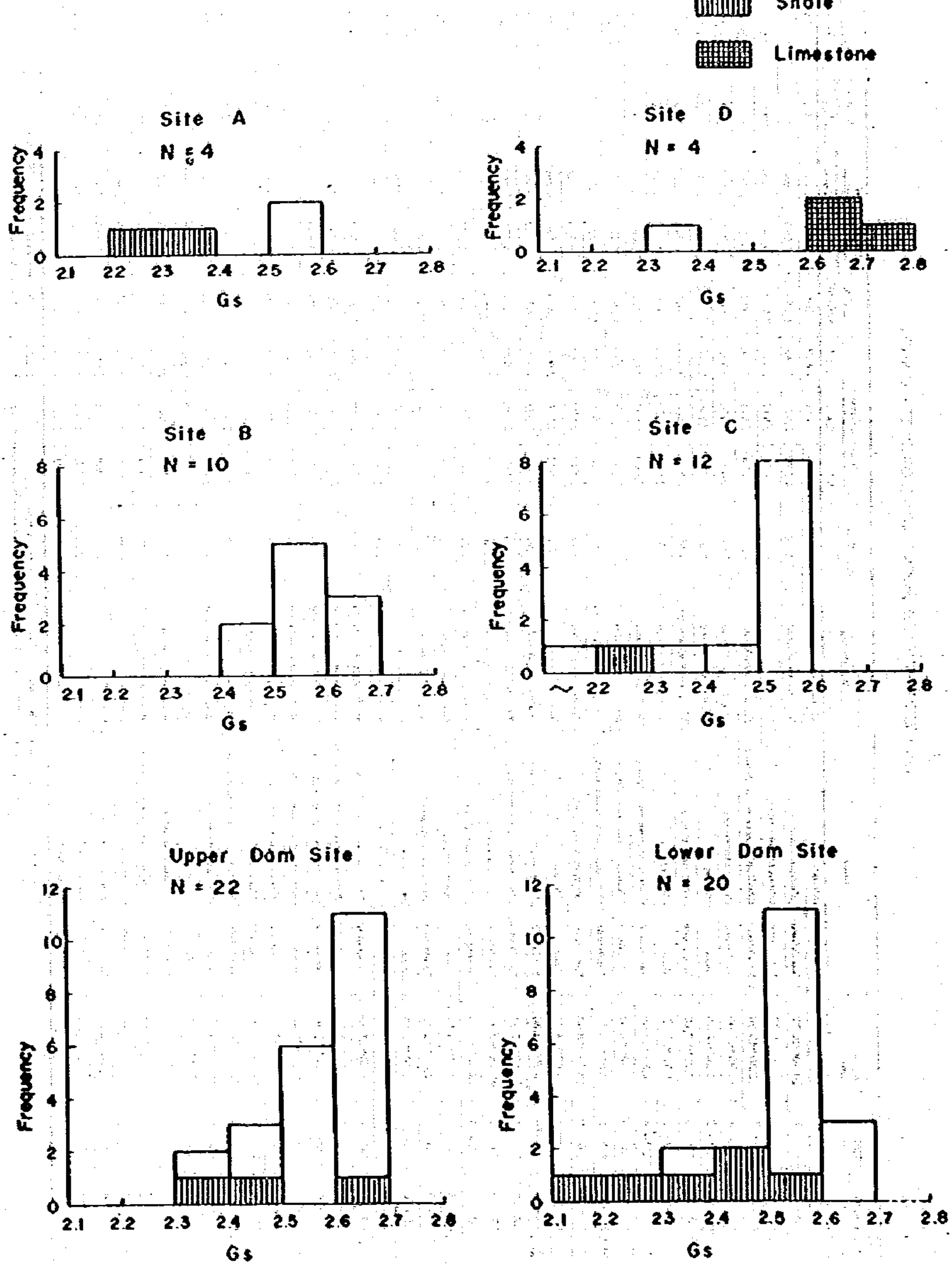


Fig.7.2 Percentage of Water Absorption

(P. of W. A)

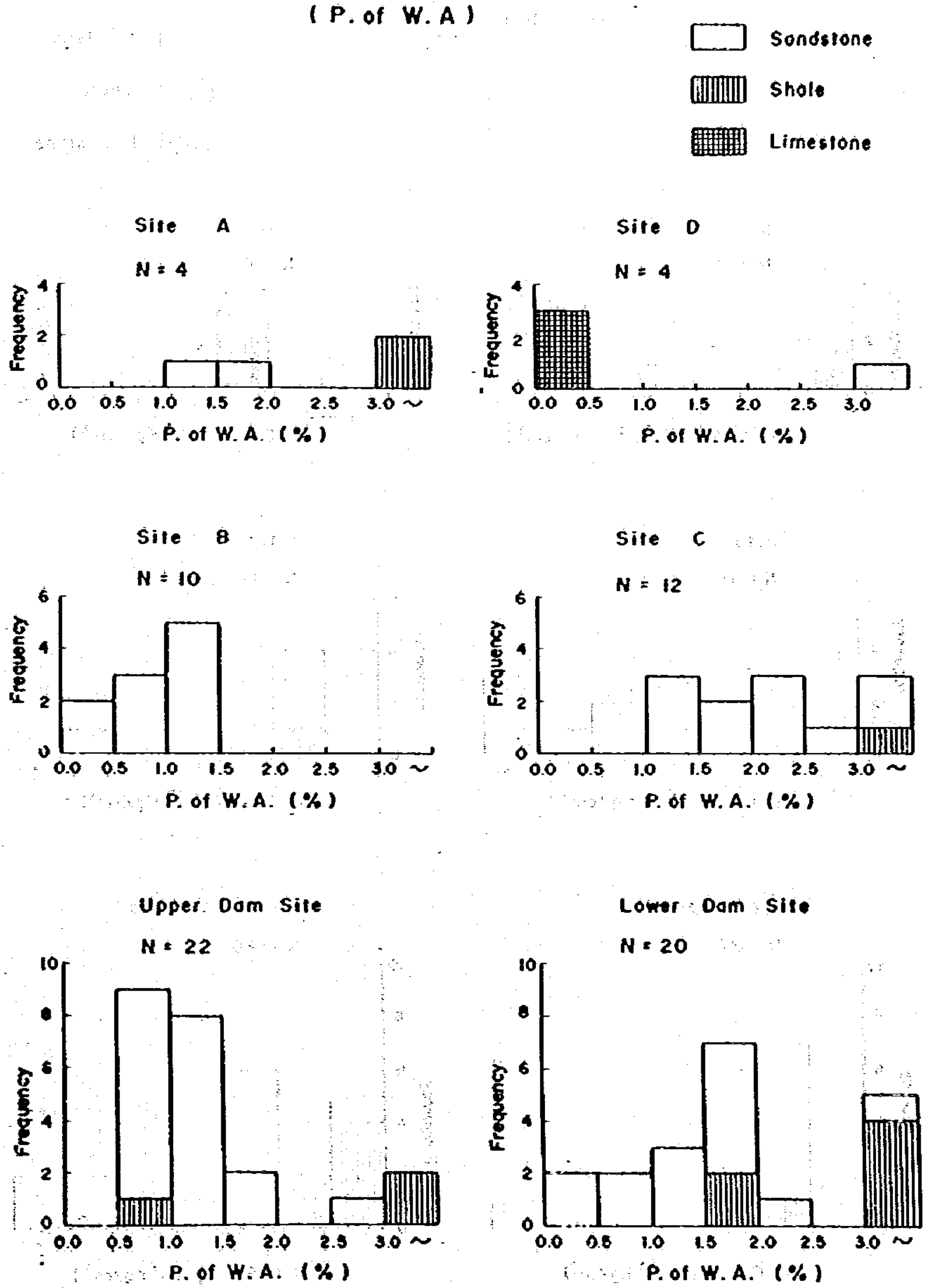


Fig.7.3 Unconfined Compression Strength (U.C.S)

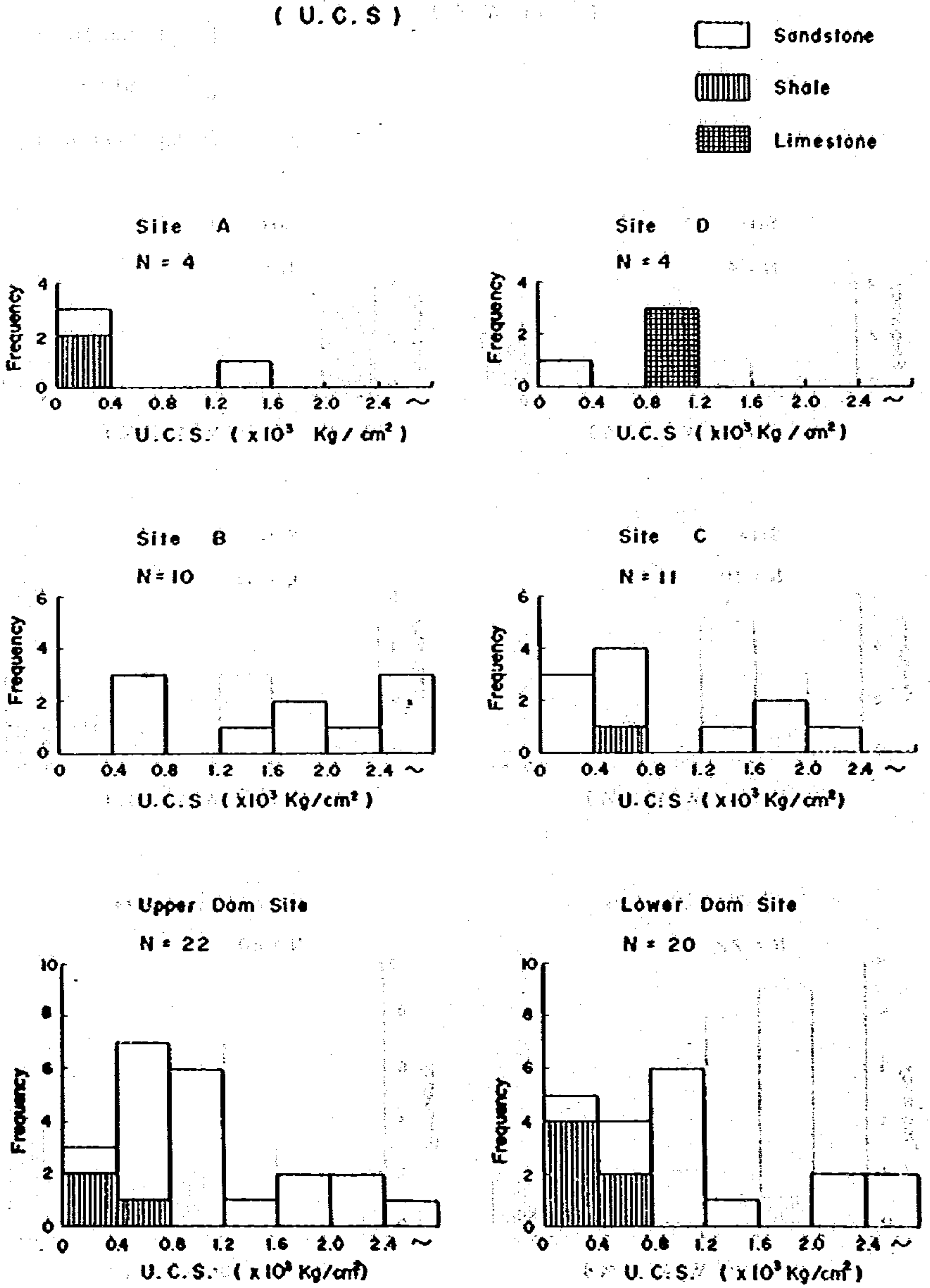


Fig.7.4 Modulus of Elasticity Dynamic

(E)

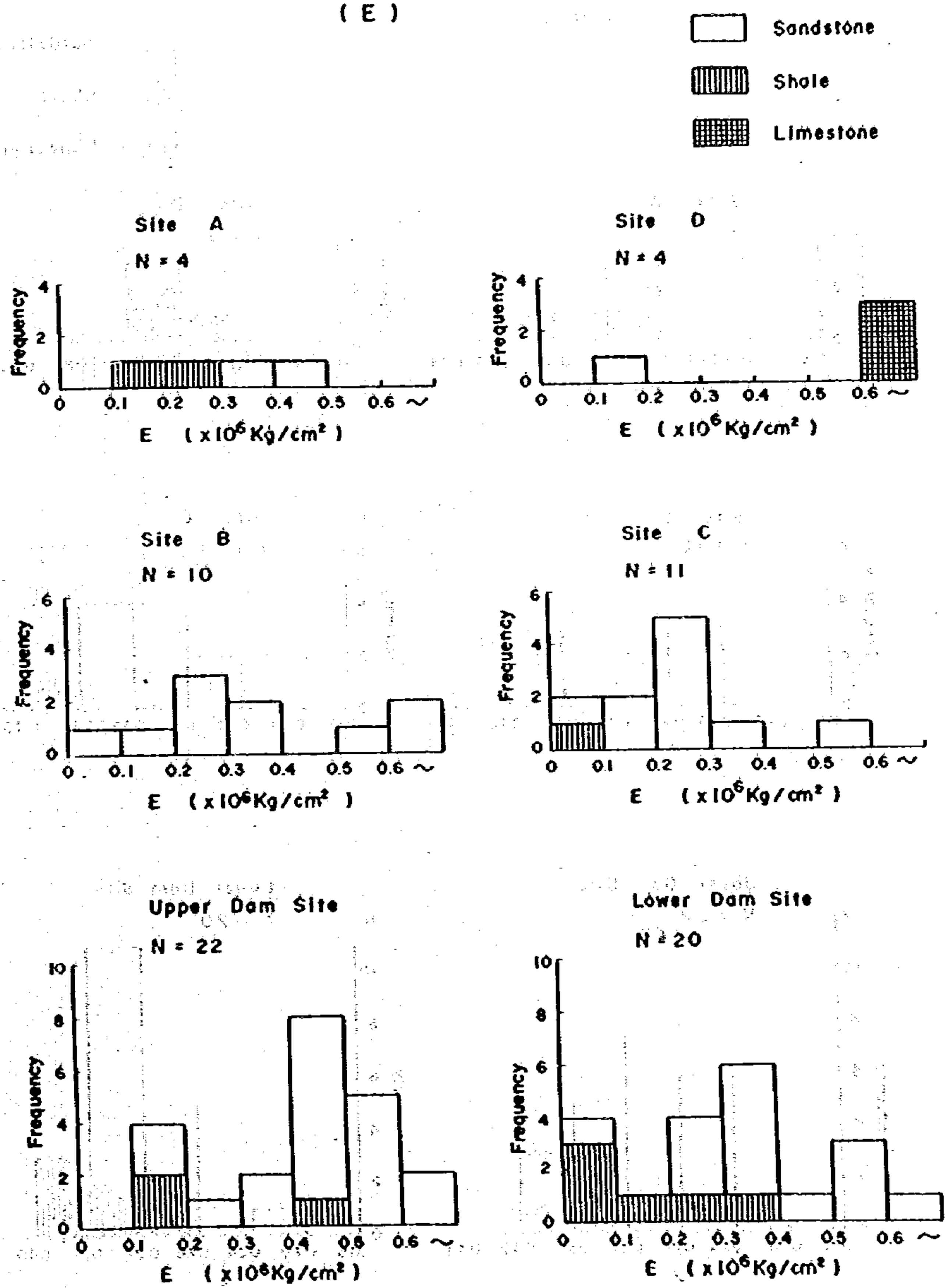


Fig.7.5 Poisson Ratio Dynamic

(ν)

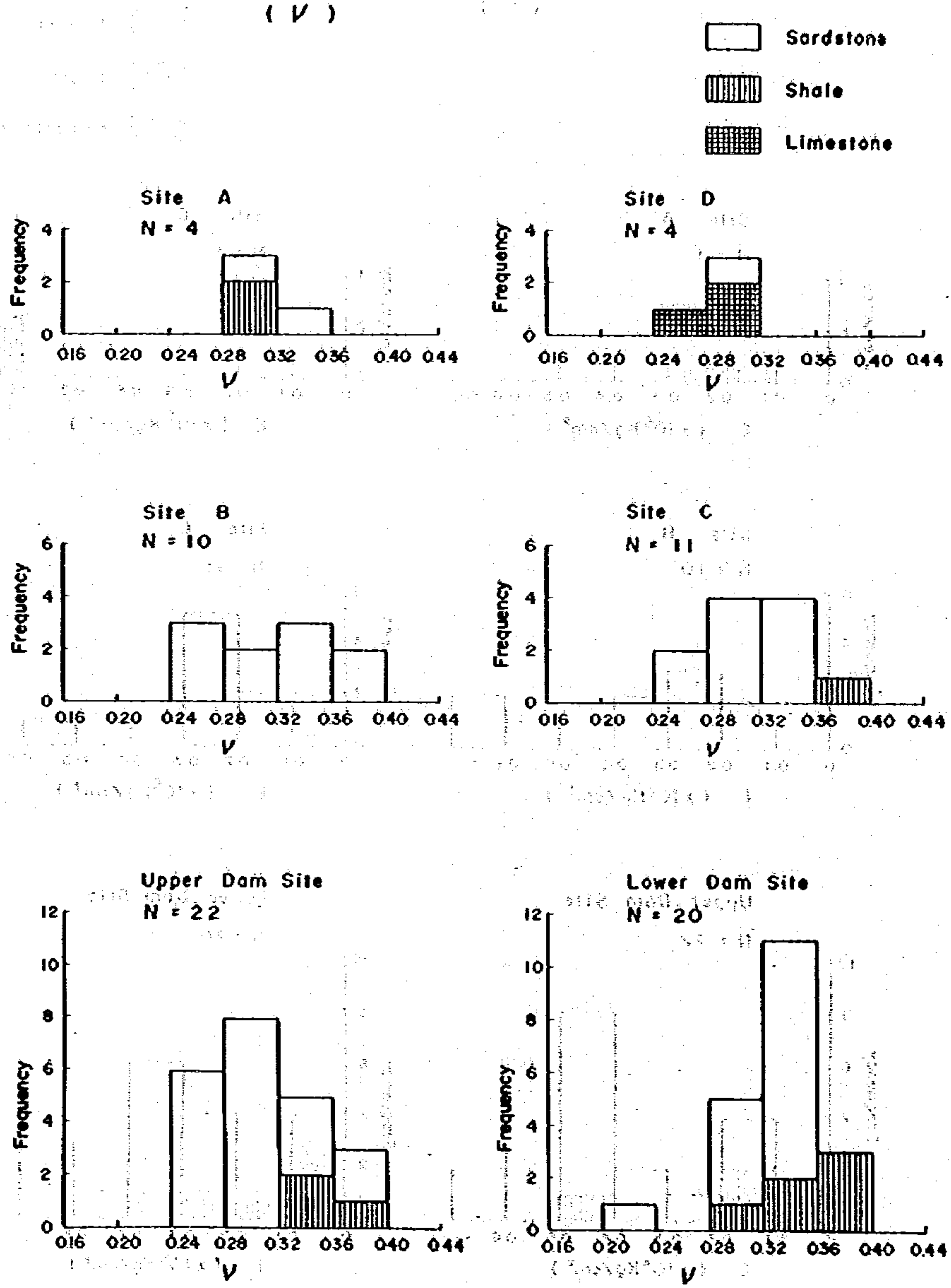


Fig.7.6 Ultrasonic Wave Velocity P wave (Vp)

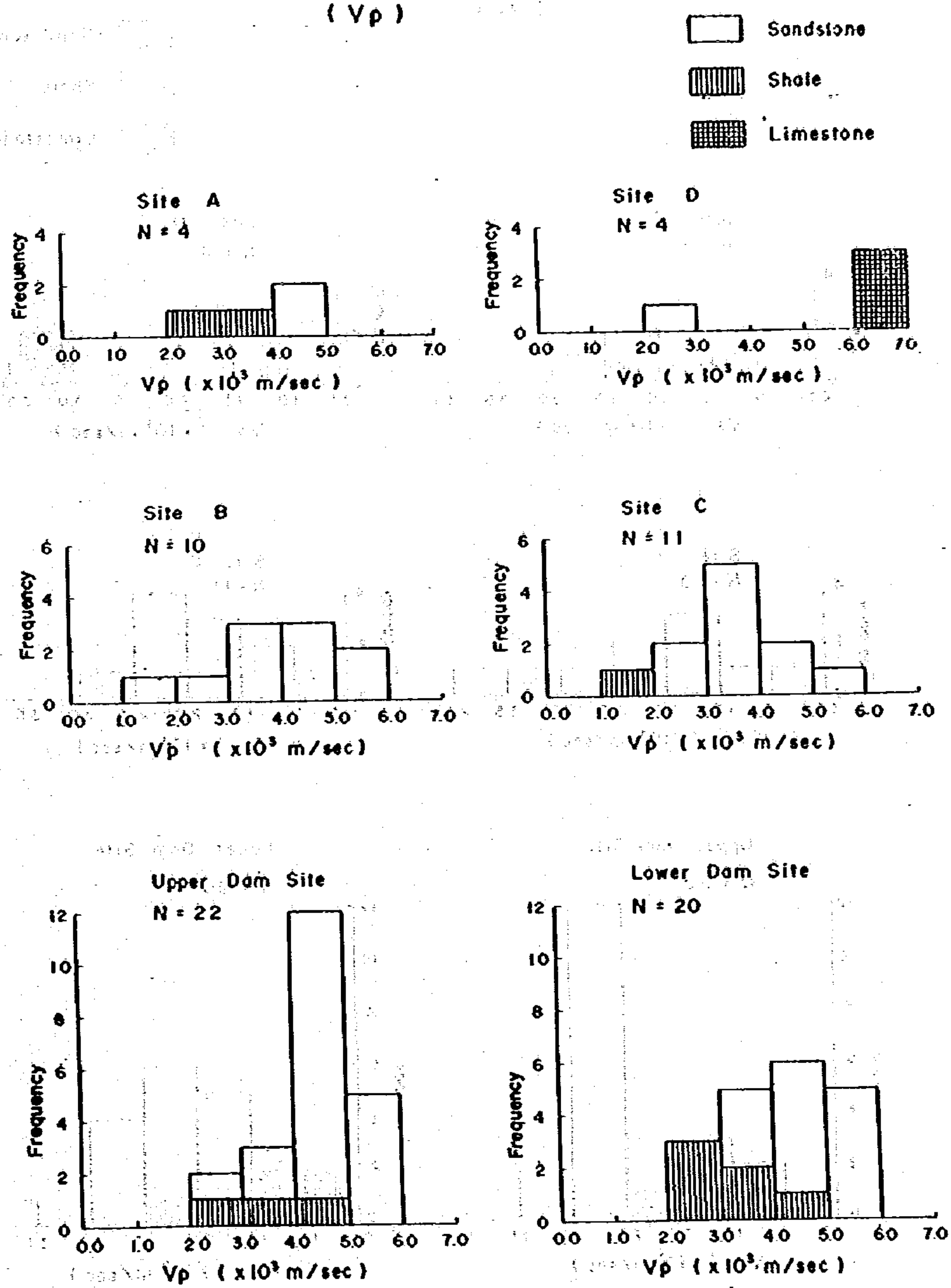


Fig.7.7 Ultrasonic Wave Velocity S wave

(Vs)

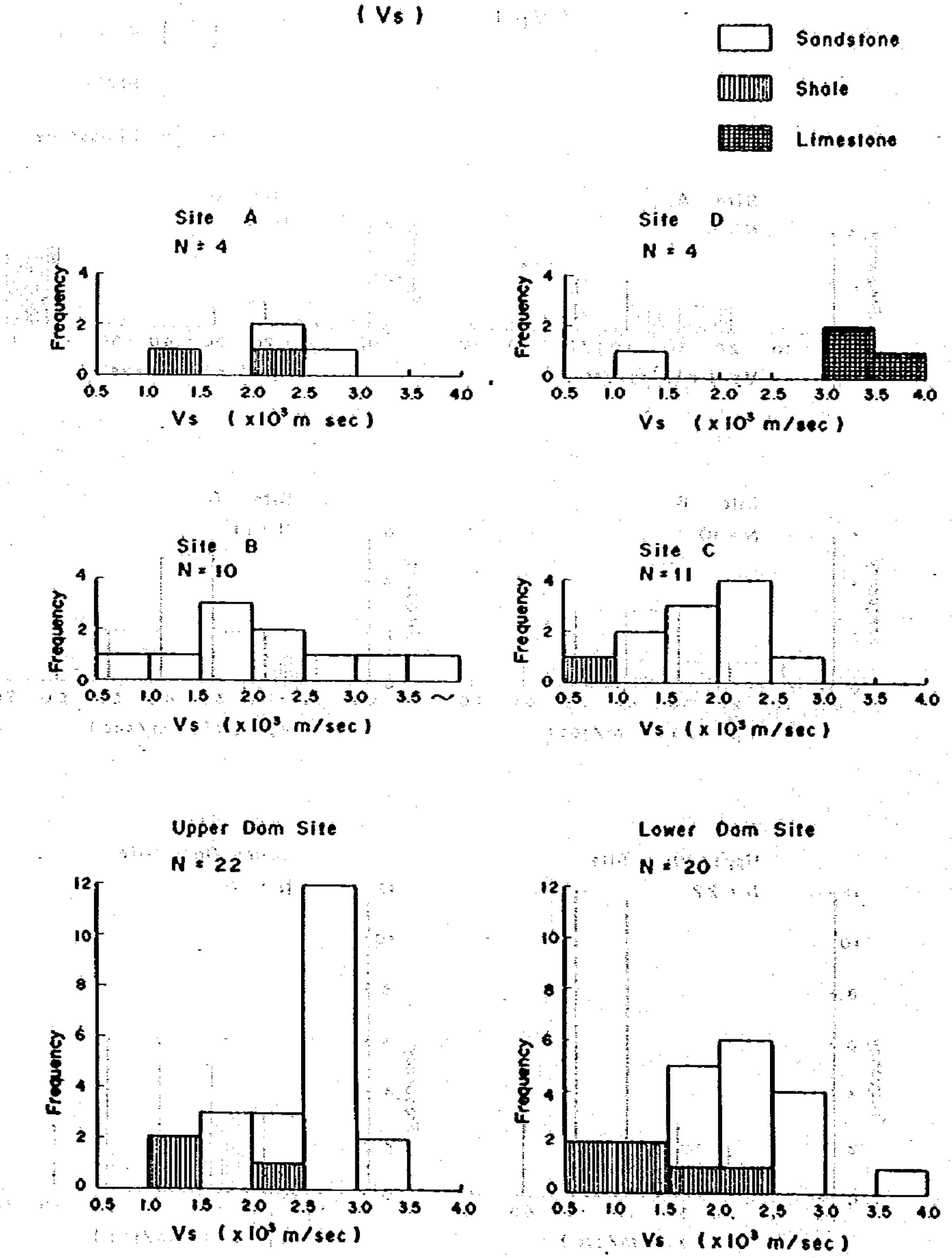


Fig.7.8 Specific Gravity (Dry Specimen)

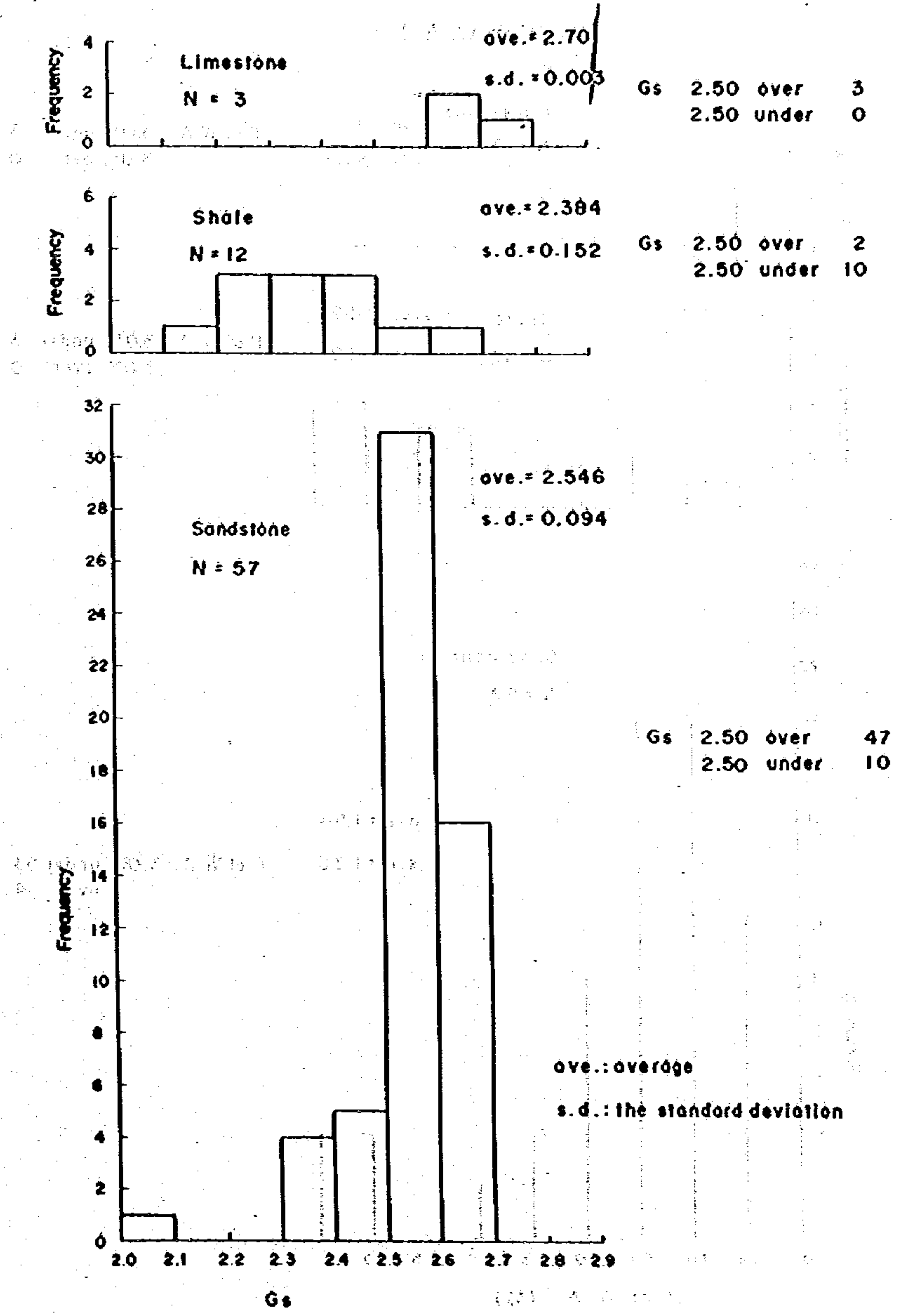


Fig.7.9 Percentage of Water Absorption

(P. of W. A.)

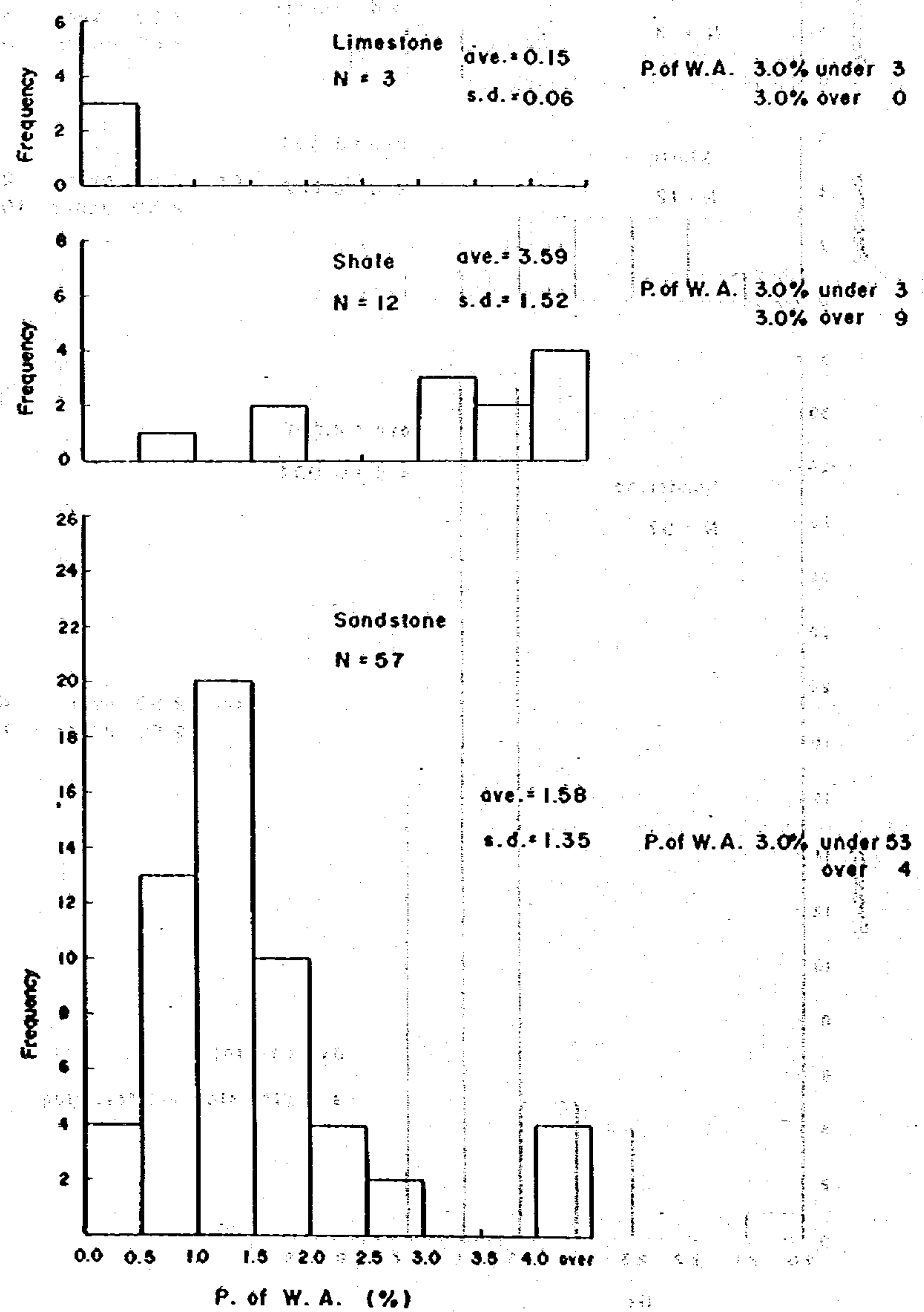


Fig.7.10 Unconfined Compression Strength (U.C.S)

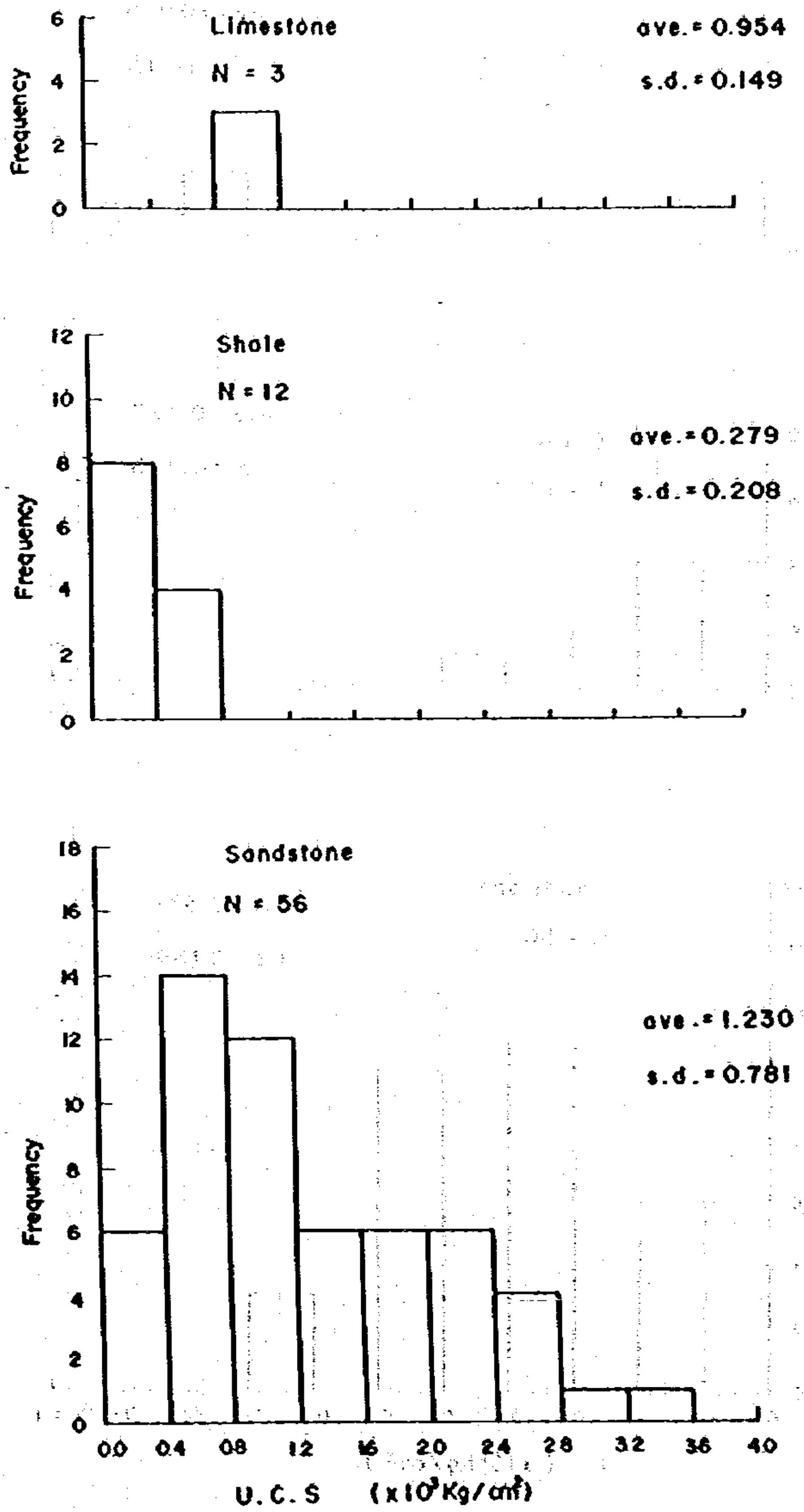


Fig. 7.11 Modulus of Elasticity Dynamic

(E)

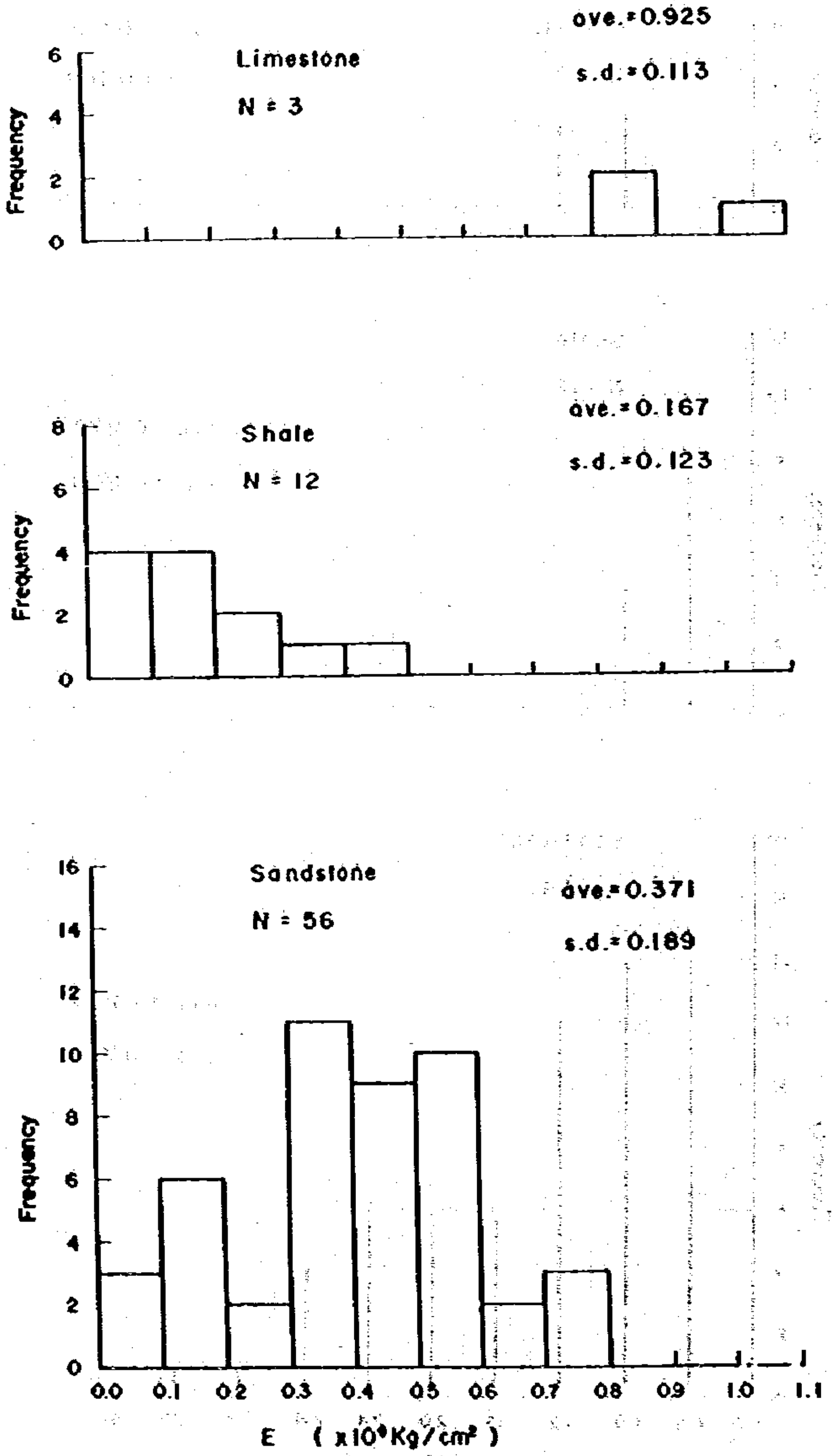


Fig.7.12 Poisson Ratio Dynamic

(ν)

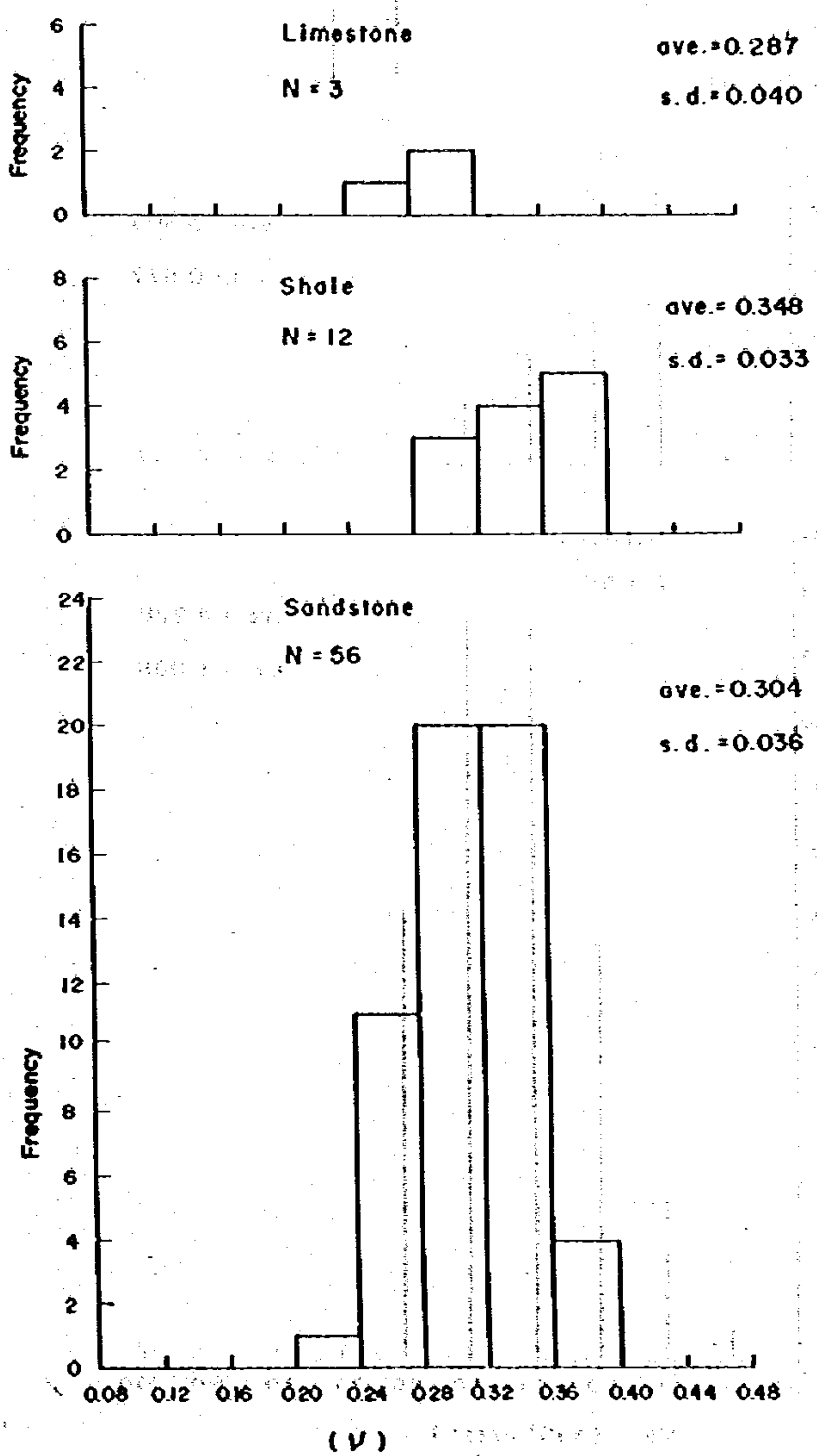


Fig.7.13 Ultrasonic Wave Velocity P wave

(Vp)

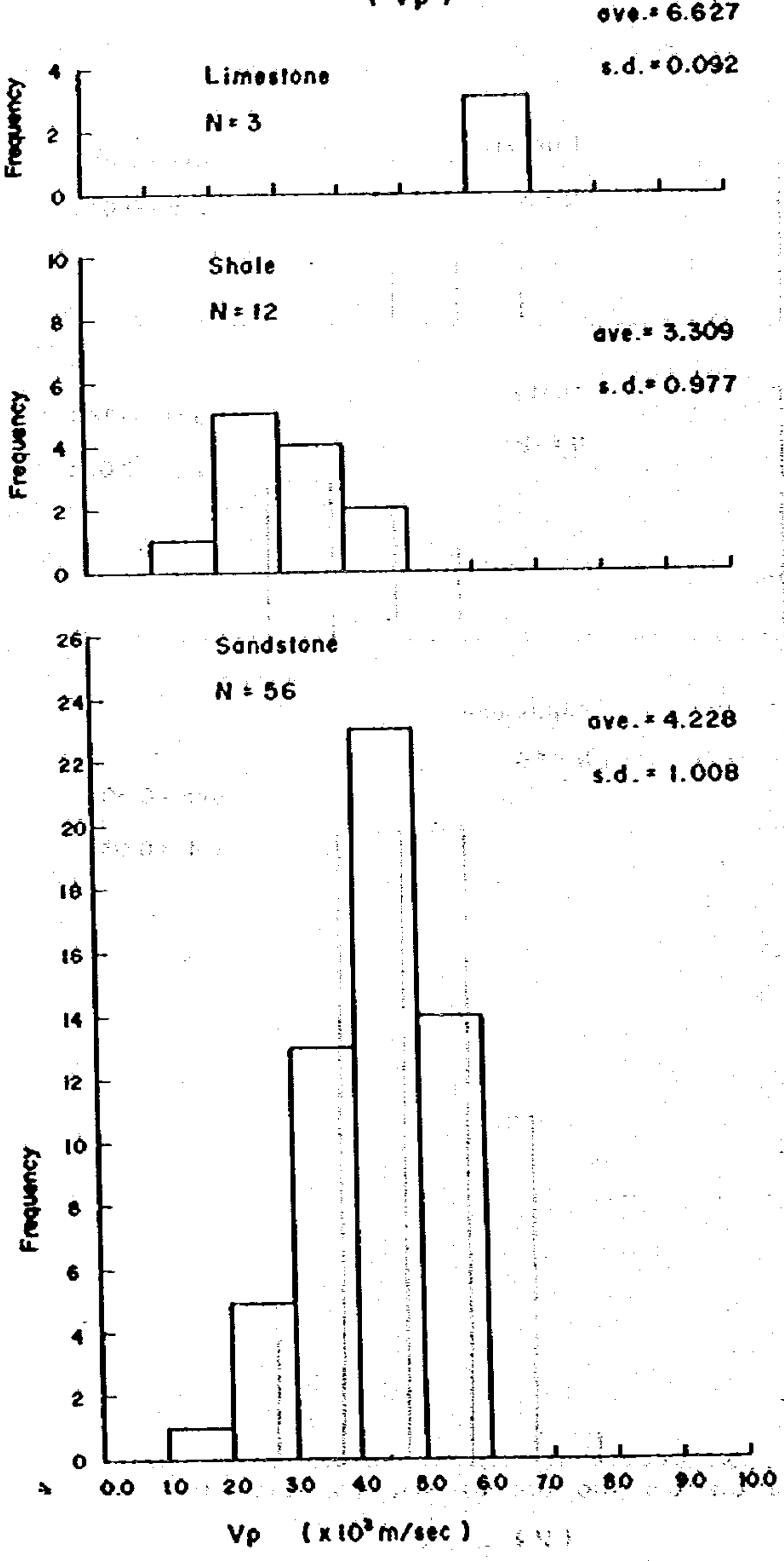


Fig.7.14 Ultrasonic Wave Velocity Pwave

(Vs)

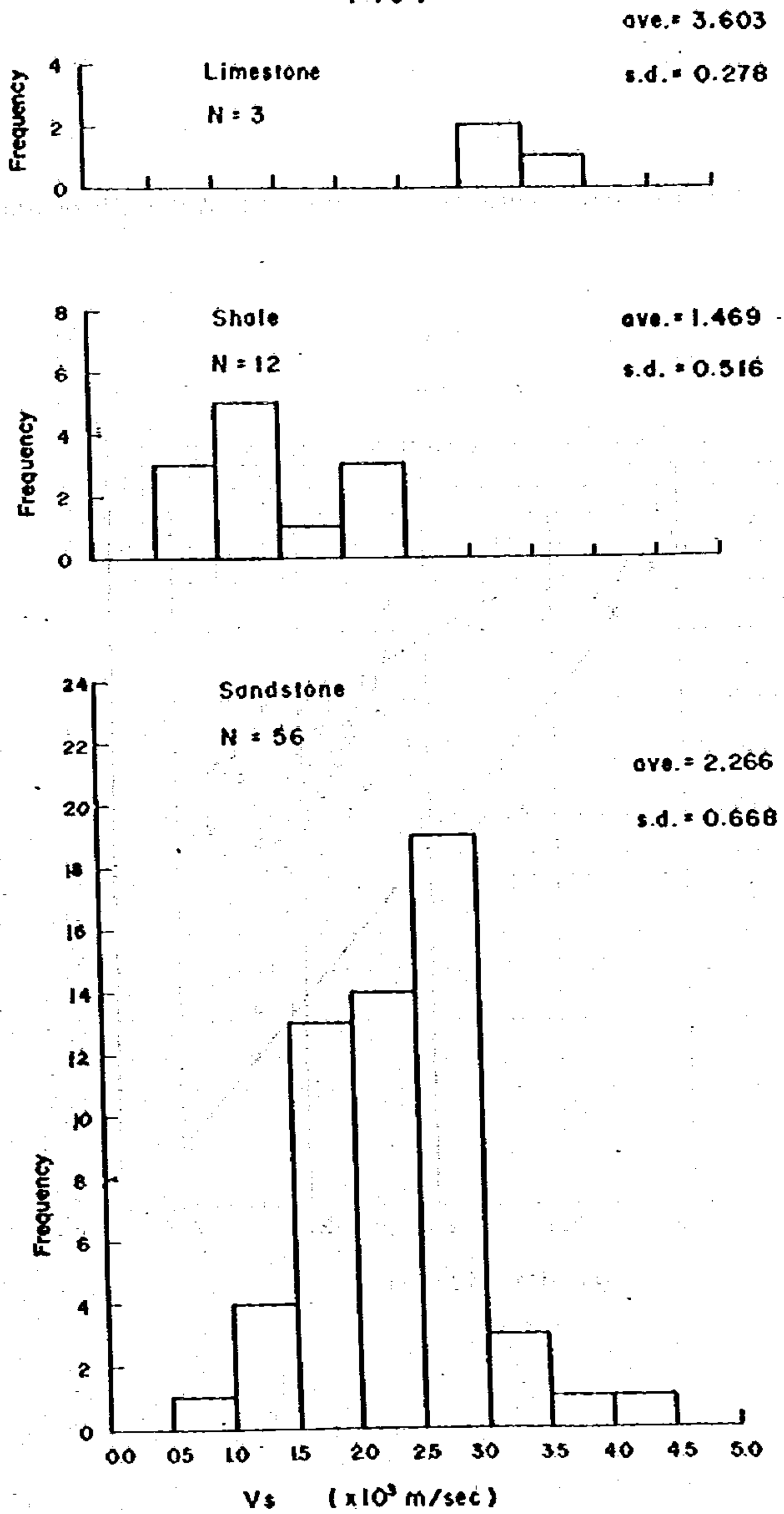


Fig. 7.15. Relationship between Density and Water Absorption

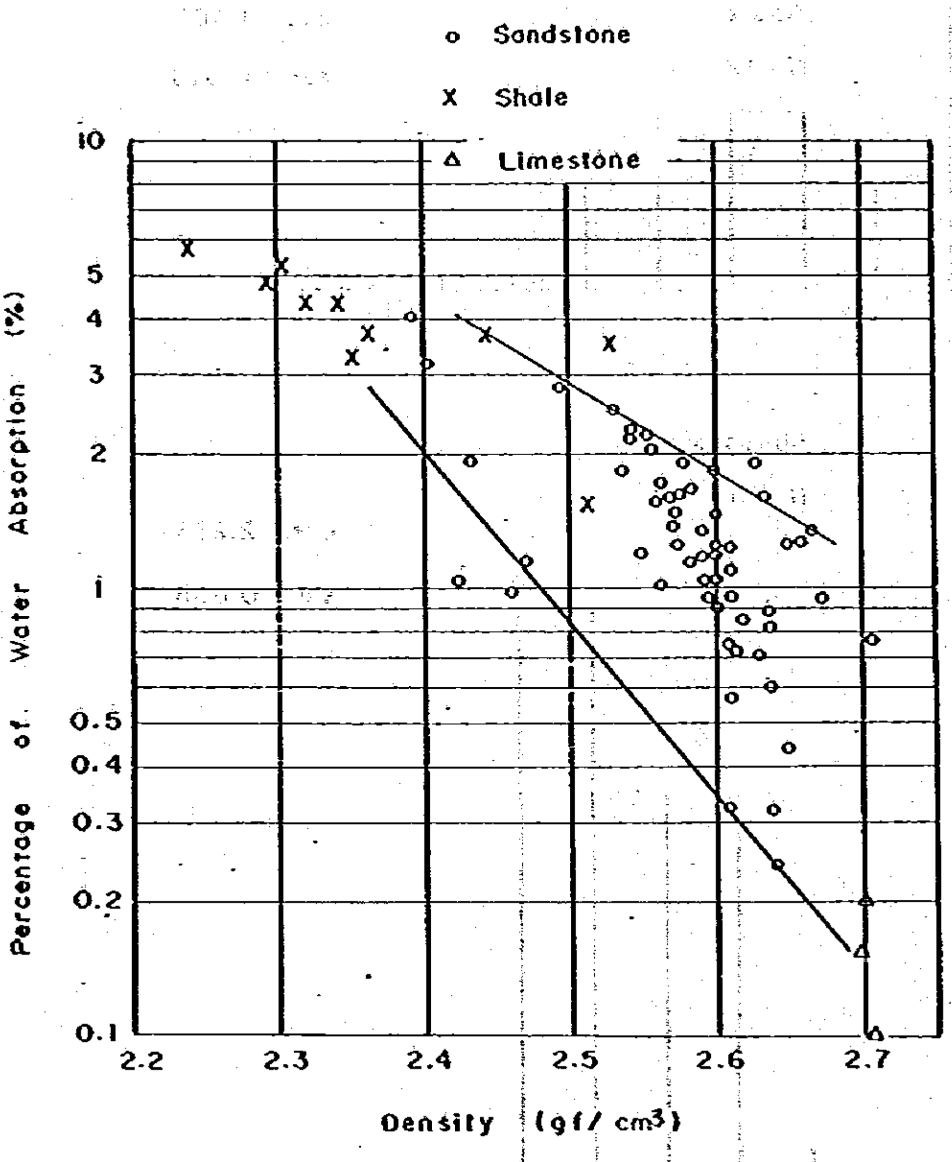


Fig. 7.16 Gs, P. of W.A. and Weathered Relation (Sandstone)
(P. of W.A.: Percentage of Water Absorption)

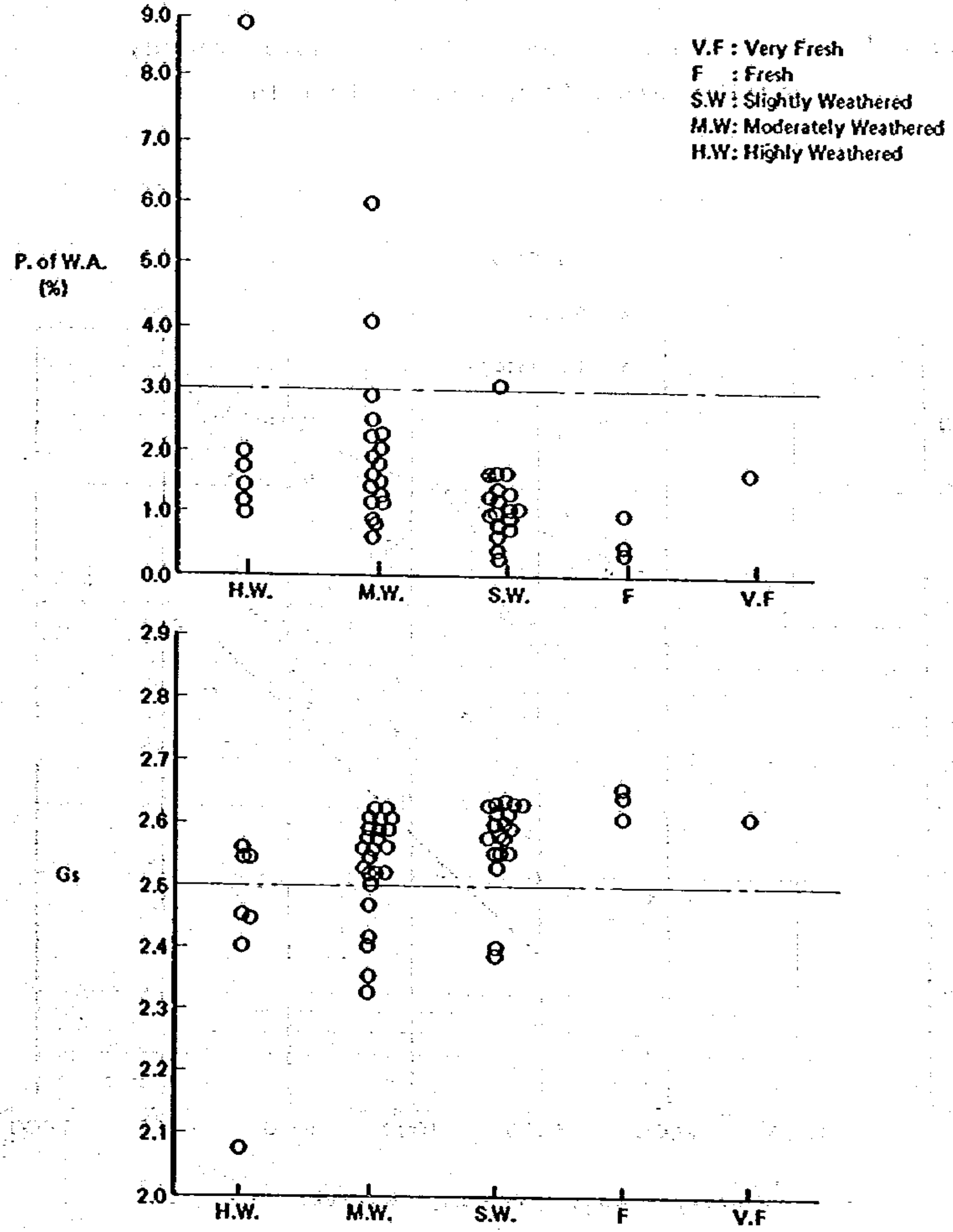
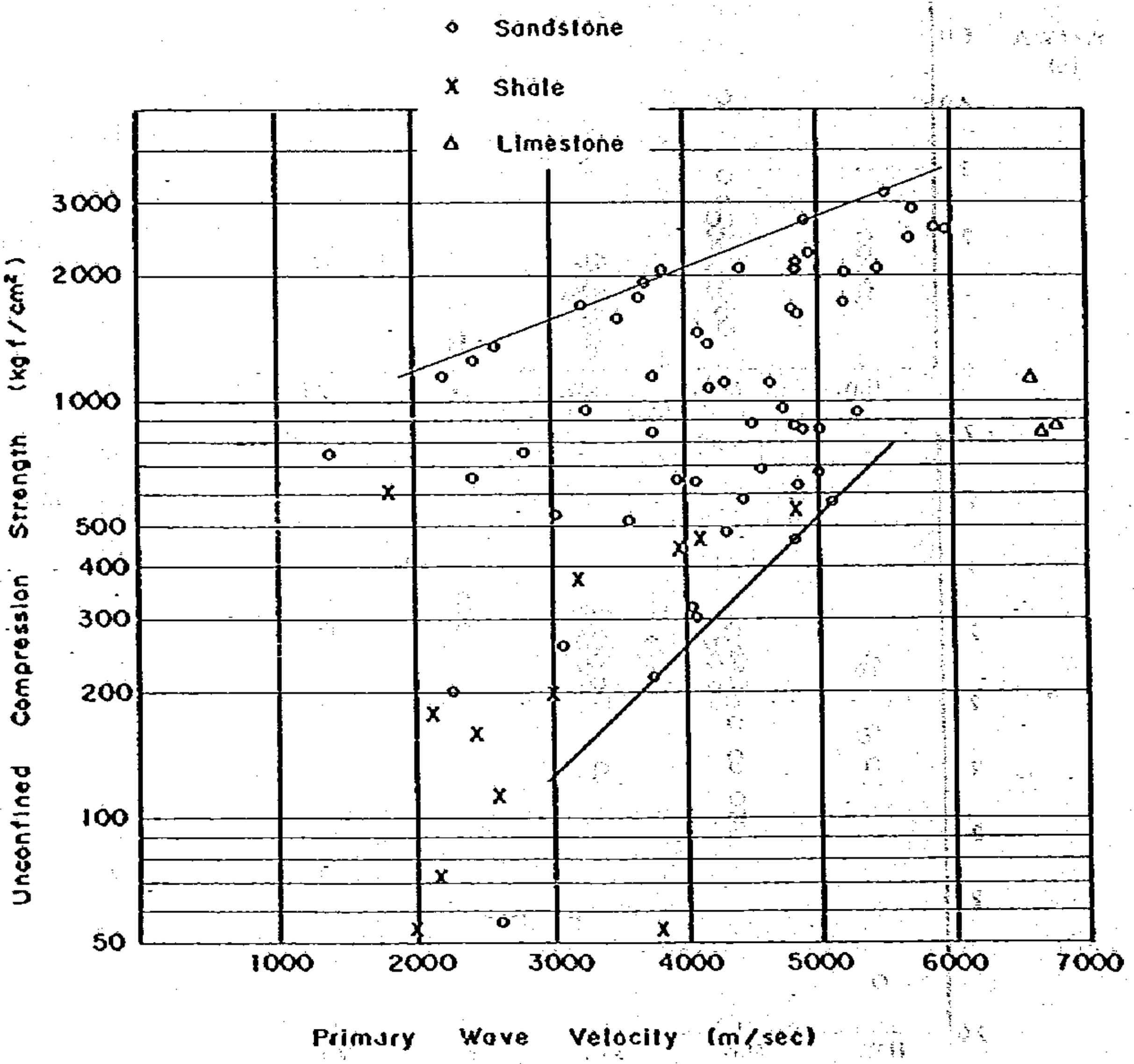


Fig. 7.17. Relationship between Primary Wave Velocity and Unconfined Compression Strength



7.5 GENERAL COMMENTS

In general, rock and aggregate materials must fulfil the following conditions.

(1) It must be strong and hard, having few cracks.

(2) It must be durable against weathering.

(3) It must not contain an injurious amount of organic impurities.

Physical tests measuring absorption and specific gravity are used in forming a judgment on these conditions. In essence, this means that if rock has high specific gravity and low absorption, it can be considered to have few voids and to be compact in texture, strong and hard, and durable.

According to the Japanese industrial standard for concrete aggregate (JIS A 5005-65), crushed rock coarse aggregate must have an absorption rate of 3% or less and a specific gravity (dried condition) of 2.50 or more.

When judged on the basis of the JIS the sandstone and limestone at the Tekai Site is found to fulfil these conditions (Figs. 7.8, 7.9). The shale which is distributed widely at Sites A and C, however, has an average absorption rate of at least 3.5% and, in large part, a specific gravity of less than 2.50; thus, it falls short of fulfilling the conditions required for aggregate.

The limestone distributed at Site D is judged to be good material for aggregate, having a specific gravity of 2.7 and an absorption rate of 0.15%.

Although sandstone fared well in the testing, some specimens were found to have an absorption rate above 3% and a specific gravity below 2.5. A check on these findings, based on weathering, is given in Fig. 7.16. This figure shows that, roughly, specific gravity decreases in proportion to weathering, making it advisable to avoid using highly weathered sandstone for aggregate. It follows that only those of the above source rocks in a moderately weathered condition or better should be taken for use as aggregate.

There exists no clear relationship between absorption and weathering; all specimens had an absorption rate of 3.0% or less regardless of their condition.

If the same standards used for judging aggregate are applied to rock material as well, it will be possible to assure the use of good quality rock material in the construction. It is therefore advisable to use sandstone in a condition of moderate weathering or better. The exception is in places in the lower part of the dam which will always be under water, where the progress of weathering is slow. Here, highly weathered sandstone, or shale, can be used without problem.

To summarize:

Aggregate:

To use sandstone and limestone in moderately weathered or better condition.

Rock material (above L.W.L.):

To use same as above.

Rock material (below L.W.L.)

To use sandstone, shale and limestone in C_L or better condition.

7.6 CONCLUSION

The results of rock tests carried out using drilling cores indicate that sandstone and/or limestone from each Tekai Site, in a moderately weathered or better condition, can be used successfully as aggregate. With respect to rock material, we estimated that the same types of rock found suitable for aggregate should be used for structures that will be above low water level, while all types of sandstone, shale, and limestone can be used in places which will fall below low water level.

The results of investigations suggest that the most economical source for rock material and concrete aggregate of the upper and lower dams is Quarry Site B.

8. ENGINEERING GEOLOGY ASSESSMENT

8. GEOLOGICAL ENGINEERING ASSESSMENT

8.1 UPPER DAM SITE

As evident from Fig.* 4.6, the upper dam site is considered to be a good site for a dam since it is composed of sandstone and shale, with its valley relatively narrow and endowed with a shallow layer of rock suited for the foundation of a fill dam. Although the strata have undergone folding, in the dam site area, it generally dip upstream.

The center core type rock fill dam is considered suitable for this site since construction materials (core, rock, etc.) are available in its vicinity.

On the right bank of the dam site, there is inferred to exist a fractured zone of relatively large scale (estimated at 20 to 40 meters), which intersects with the dam axis at a low angle. Careful attention is therefore required in foundation treatment and the design of structures.

8.1.1 Dam

(1) Study of Excavation Line

Fig.* 8.1 shows a geological profile of the dam axis.

River bed deposits are assumed to exist in a layer 5 to 7 meters thick. There are almost no top soil and talus deposits in the left bank, but in the right bank they are distributed in a layer with the maximum thickness of 5 meters.

As a result of the drilling and seismic prospecting, it appears that rock classification is as illustrated in Fig.* 8.1; C_L Class rock is found in somewhat greater thickness in the right bank, but in both banks, the higher the layer, the thicker the weathered rock.

Table 8.1 shows the average depth of rock from the ground surface, according to rock classification:

Table 8.1 List of Depth of Rock Classification of Foundation Rock

(Unit: m)

Location / Rock Classification	Left Bank			Center of River bed	Right Bank		
	Near Crest	Near Midpoint	Near Bottom		Near Bottom	Near Midpoint	Near Crest
C_L	7	4	2	-	6	4	3
C_H	18	6	8	6	10	13	13
Over C_H	37	28	27	17	20	31	34

The evaluation of rock, according to rock classification, to be used for rock fill dam with some 100 m dam height, is given as follows.

Due to weathering, the D Class rock (completely weathered rock) no longer has its original rock texture, but is found in a sand-clay. This is not a desirable condition as the foundation for a center core type rock fill dam. Also, as it may cause deformation and piping among other problems, it is necessary to thoroughly remove the D Class rock in the core foundation.

In the rock foundation, on the other hand, it is necessary to embank rock material after removing the loose (not solid), near-surface portion of the D Class rock.

The C_L Class rock, which mainly comprises highly weathered rock, is relatively fragile and capable of being broken by a hammer. Cracks are developed in this type of rock, so it is necessary to remove it from the core foundation because it threatens to cause piping in the core. It appears to present no particular problems for rock foundation.

The C_H Class rock, which is relatively hard and moderately weathered, is suitable for a core foundation, but it requires foundation treatment.

The B and C_H Class rock, which is hard, fresh, and slightly weathered, should present no problem if used for a dam foundation.

As mentioned above, the excavation line of a core foundation must be based on the C_M Class rock, after removing other rocks up to C_L Class rock. In the foundation for rock material, it is necessary only to remove the loose, clayey (sand~clay) portion of the D Class rock.

The drilling core observation sets the boundary line of rock classification between C_L and C_M Class (see Fig. 8.1) within the range where the C_M Class rock accounts for more than 80 percent over the length of 5 meters, as illustrated in Fig. 8.2.

The permeability of the rock foundation, on the other hand, is shown in the Lugeon Map in Fig. 8.3. In the left bank, near surface of the slope, there is highly permeable zone above 50 Lu, which is distributed continuously down to the river bed. In this figuer the left bank also shows a deep, low permeability zone of below 2 Lu, On the contrary, a relatively shallow low permeablitiy zone (below 2 Lu) is distributed in the right bank down to the river bed. In the right bank, however, there is inferred to exist a fractured zone with a width of 20 to 40 m. Since full leakage of drilling water was seen in UD-16 borehole, this fractured zone is considered to form a high permeability zone.

From the viewpoint of permeability, it seems necessary to set up a dam excavation line by which a high permeability zone above 50 Lu can be completely removed.

An excavation line is shown in Fig. 8.1 and 8.3 in consideration of rock classification and permeability as mentioned above. The depth of the excavation line from the ground surface is also shown in Table 8.2.

Table 8.2 Proposed Excavation Depth of Upper Dam (Rock Fill Dam)

(Unit: m)

	Left Bank			Center of River Bed	Right Bank		
	Near Crest	Near Midpoint	Near Bottom		Near Bottom	Near Midpoint	Near Crest
Depth(m)	18	16	12	12	14	15	20

For reference, the depth from the ground surface of excavation line used in constructing a concrete gravity dam of similar scale has also been given in Table 8.3.

Table 8.3 Excavation Depth of Upper Dam (Concrete Gravity Dam)

(Unit: m)

	Left Bank			Center of River Bed	Right Bank		
	Near Crest	Near Midpoint	Near Bottom		Near Bottom	Near Midpoint	Near Crest
Depth (m)	22	28	25	20	20	32	30

Great bearing capacity is needed for the foundation rock of a concrete gravity dam with some 100 m dam height. This requires a foundation made of the C_H Class rock, which has adequate bearing capacity, but it also requires an enormous volume of excavation. It is considerated that a rock fill dam presents an advantage for geological reasons.

(2) Study of Foundation Treatment

As is obvious from the Lugeon Map in Fig. 8.3, a relatively shallow low permeability layer below 2 Lu is distributed in the right bank and river bed of the upper dam, whereas a deep one is distributed in the left bank.

Under these conditions, the foundation rock must be improved by curtain grouting. With the rock improvement target being set at 2 Lu, it is advisable to perform foundation treatment so that the rock will be safe enough to be proof against leakage or piping. When foundation treatment is to be improved up to 2 Lu, the range of improvement is as illustrated in Fig. 8.3.

The distribution of layers below 2 Lu is shallow in the river bed area, so it is advisable to ensure water stop by providing curtain depths based on the following criteria:

- Range of curtain grouting up to the third order borehole (3-meter pitch)

$$L \doteqdot \frac{H}{2} \doteqdot 50 \text{ m}$$

- Range of curtain grouting up to the second order borehole (6-meter pitch)

$$L \fallingdotseq \frac{2 \cdot H}{3} \fallingdotseq 70 \text{ m}$$

Note: L = grout depth (meters)

H = dam height (meters)

As for the high permeability zones above 50 Lu, which remain below overflow water level in the left bank, it is necessary to accomplish water stop by performing grouting up to the fourth borehole (1.5-meter pitch), with grout tunnels being constructed in both banks. Particularly for the fractured zone in the right bank, it is advisable to provide complete water stop by performing curtain grouting up to the fourth borehole (1.5-meter pitch) to the level of the river bed (corresponding to hydrostatic pressure). This is because the high permeability zone might continue deep within the strata.

8.1.2 Penstock and Power Station

It is desirable to build a power station in a relatively flat place on the right bank of the Tekai River at the toe of slope downstream from the dam. The penstock shall therefore go through the right bank of the Tekai River.

Fig. 8.4* shows a geological profile of the passage from penstock to power station.

The geological distribution of this path comprises sandstone in general, but the shale is distributed over a distance of about 130 m from the intake and in parts thereafter.

The path of the penstock is deeply covered, it presumably consists of favorable rock above C_H Class for the most part. Also, a continuous fractured zone (about 20 to 40 meters wide) is inferred to extend from the right bank of the dam site throughout the penstock path.

Excavation of this fractured zone may cause water to spring. Careful attention and safe engineering methods are thus required for excavation in and around the fractured zone.

Most of the penstock path is thought to go through a hard sandstone. For this reason, the NATM Method is possible here, where effective excavation can be accomplished by providing adequate support when encountering a geologically inferior section.

The site of the power station is geologically made up of sandstone and shale, overlain by top soil and talus deposits. But according to the drilling survey conducted in the vicinity (UD-9, UD-17), top soil and talus deposits is distributed only between 5 and 10 meters, suggesting that there exists a hard rock above C_M Class. This rock presents no problems for the foundation of a power station.

8.1.3 Spillway

Spillway is located at the left bank of the Tekai River. *Fig. 8.5 shows a geological profile of the spillway. Sandstone and shale are distributed in the foundation of the spillway in nearly the same quantities. In the neighborhood of the energy dissipator located downstream, there is thought to lie a fractured zone about 5 to 20 m wide under the terrace deposits about 5 to 10 m thick.

The rock foundation of the spillway shall principally consist of moderately weathered rock of C_M Class. As shown in *Fig. 8.5, the depth of the C_M Class rock generally ranges between 5 and 20 m, estimated by the drilling and seismic prospecting.

As an alternative for spillway, a scheme proposing an access to the dam body is now under consideration. *Fig. 8.6 shows a geological profile in line with the alternative plan.

The alternative spillway foundation is mainly composed of sandstone. The depth of the C_M Class rock which forms the foundation of the spillway is assumed to range between 10 and 20 m. The alternative plan provides for greater depth; in that case, a foundation partially composed of C_L Class rock may be practicable.

At their present accuracy, the geological surveys conducted so far have not confirmed any clear comparative merits or demerits between the two types of spillways.

8.1.4 Diversion Tunnel

In relation to geological and topographical conditions as well as to the layout of structures, the location of a diversion tunnel shall be set at the left bank of the Tekai River. Fig. 8.7 shows a geological profile of the diversion tunnel.

The path of this tunnel consists mostly of sandstone, but is thought to be composed of shale in the vicinity of the dam axis. It also presents a complicated geological structure because there is one anticline axis and two syncline axes, where strata are not uniformly inclined. It appears that a fractured zone (20 to 40 m wide at the intake and 5 to 10 m wide at the tailrace) goes through the layers in the neighborhood of both intake and tailrace. Adequate consideration is therefore necessary in order to secure a stable slope at the pit mouth.

Because there is sufficient depth from ground surface, the diversion tunnel is expected to pass through favorable rock above C_H Class for the most part. For this reason, the NATM Method is considered practicable for the purpose of excavation.

8.1.5 Cofferdam

In relation to the layout of structures, both upper and lower cofferdams shall be located at the spot indicated in Fig. 4.6. Fig. 8.8 shows a geological profile of the cofferdam.

At the site of the lower cofferdam, an outcrop of hard sandstone is seen on the river bed, where deposits are expected to be of lesser thickness. Also, the presence of a series of favorable cores has been confirmed by the drilling in the vicinity (UD-18). It is believed, therefore that sandstone suited for the foundation of a cofferdam will be available by removing the top soil and river bed deposits.

At the site of the upper cofferdam, an outcrop of hard sandstone is also seen on the river bed, where deposits are expected to be of lesser thickness. The terrace deposits are distributed on the left bank. It appears that somewhat deeper excavation will be required in order to remove those deposits.

8.2 CONSTRUCTION MATERIALS FOR THE UPPER DAM

The following quantities of construction materials are needed for the upper dam site, including the upper dam and incidental structures:

- Soil material (core material) ...... 580,000 m^3
- Rock material (shell material, filter material, and concrete aggregate) .... 2,700,000 m^3

For the purpose of procuring these construction materials, an investigation was conducted in the upper site after selecting two borrow sites (Sites A and B) and one quarry site (Site B).

8.2.1 Borrow Site (Sites A and B)

(1) Site A

Site A features a widespread distribution of shale, where a distribution of thick weathered rock had been anticipated. However, as evident from the result of drilling shown in Table 4.7, the weathered rock (suitable for core material) here, except UB-1, has proved to be rather thin in general, i.e. about 3 m thick at most. Fig. 8.9* is a geological profile based on the result of drilling and seismic prospecting. Fig. 8.10* is an isopach map of weathered zones. Weathered zones with seismic velocity below 1,000 m/sec. are generally thin (less than 5 m), except on the upstream side where they are a little thicker (about 10 m). The layers of top soil and organic soil are about 1 m thick as shown in Table 4.18, and the average thickness of weathered zones (suitable for core material) is estimated at approximately 2 m.

The following data are the result of soil testing which confirmed the fulfillment of various conditions required for core material.

- Specific gravity: 2.50 to 2.75 (average 2.62)
- Moisture content: 10 to 30% (average 18.2%)
- Grading : (average grading distribution)

gravel	7.7%
sand	26.1%
silt/clay	66.2%

- Maximum dry density: 1.57 to 1.92 g/cm^3
(average 1.79 g/cm^3)
- Natural moisture content/optimum moisture content:
$Wopt = Wn \pm 4$ (%)
- Permeability coefficient: 1.15×10^{-7} to 8.14×10^{-7} cm/sec.
(average 4.6×10^{-7} cm/sec.)

Qualitatively, Site A is a place suited for procuring core material, but quantitatively, it will be by no means easy to procure a large volume of core material effectively in this site.

(2) Site B

Site B is located in an area featuring a widespread distribution of shale. No drilling has been conducted in Site B, Fig.* 8.11 is a geological profile based on the results of seismic prospecting and Fig.* 8.12 is an isopach map of weathered zones. Weathered zones with seismic velocity below 1,000 m/sec. are about 1 to 10 meters thick in this site. Top soil and organic soil are about 1 meter thick as shown in Table 4.19. The thickness of weathered zones (suitable for core material) is thus estimated at approximately 5 m.

The following data are the result of soil testing which confirmed the fulfillment of various conditions required for core material.

- Specific gravity: 2.50 to 2.75 (average 2.62)
- Moisture content: 12 to 26% (average 18.9%)
- Grading: (average grading distribution)

gravel	6.9%
sand	27.9%
silt/clay	65.2%

- Maximum dry density: 1.58 to 1.87 g/cm^3 (average 1.74 g/cm^3)
- Natural moisture content/ optimum moisture content:
$Wopt = Wn \pm 3$ (%)
- Permeability coefficient: 1.10×10^{-7} to 2.66×10^{-7} cm/sec.
(average 2.0×10^{-7} cm/sec.)

Qualitatively, the soil in Site B is suited for core material. Site B is a more promising place than Site A.

8.2.2 Quarry Site (Site B)

The upper site is primarily composed of sandstone and shale. As shown in Figs. 7.1 to 7.14 and Table 8.4, sandstone is superior in quality as rock material.

Table 8.4 Results of Rock Testing

(Mean value)

	Specific gravity (D.S.)	P. of W.A. (%)	U.C.S (kgf/ cm^2)	M. of E. dynamic ($\times 10^3$ kgf/cm^2)	Poisson ratio dynamic	Velocity P wave ($\times 10^3$ m/sec)	Velocity S wave ($\times 10^3$ m/sec)
Sandstone	2.571	1.10	1267	410	0.30	4.395	2.384
Shale	2.435	2.79	449	249	0.32	3.613	1.833

For this reason, sandstone shall be used as rock material, but shale is also considered usable, as explained in 7. "Rock Testing", if it is used below the water surface of low water level at the dam bottom.

Compared with rock material, concrete aggregate requires better, compact, and harder rocks. Sandstone shall be used as concrete aggregate for this reason.

Site B features a wide distribution of sandstone. Fig. 8.11 shows a geological profile of Site B based on the result of drilling and seismic prospecting.

The drilling have revealed that, as shown in Table 4.9, the depth of base rock (suitable for rock material) ranges between 5 to 20 m, where there is relatively thick. Here, the average thickness of weathered zones (including top soil and organic soil) with seismic velocity below 1,000 m/sec. is estimated at approximately 8 m.

Seismic velocity of 4,000 to 5,000 m/sec. was observed in the fresh rock section. Drilling were also rewarded with favorable results. These outcomes indicate that Site B will make a good quarry.

8.2.3 Working Program for Construction Materials

(1) Core Material

580,000 m^3 of core material are necessary. About 1,000,000 m^3 of site volume will become necessary when the yield rate and bulking factor are taken into consideration.

Both Site A and B are qualitatively suited for core material, but effective gathering of core material would be difficult in Site A. Therefore, Site B is the first potential, and Site A shall be supplementary site in case of the shortage of material from Site B.
Since weathered zones are relatively thick in the quarry site (Site B), it is advisable to perform effective gathering by appropriating these zones for core material.

(2) Rock Material

2,700,000 m^3 of rock material are necessary. About 2,400,000 m^3 of site volume will become necessary when the yield rate and bulking factor are taken into consideration.

Rock material is sufficiently available in the upper quarry site (Site B). Excavated material may be reused for rock material since both diversion tunnel and penstock are assumed to go through a hard sandstone.

8.3 LOWER DAM SITE

The lower dam site is composed of sandstone and shale as illustrated in Fig. 5.5. This site is geologically inclined upstream. Weathered zones in both right and left banks are a little deeper than those in the upper dam site.

A fill dam and concrete gravity dam are among the dam types possible here. But the latter brings greater advantages because of lower construction costs. As for the former no promising place has been found secuing construction materials, especially core material.

8.3.1 Dam

(1) Study of Excavation Line

Figs.* 8.13 and 8.14 show geological profiles of the dam axis and overflow section.

River bed deposits are distributed about 5 m thick. Top soil and talus deposits also form a thin layer on both the left and right banks.

Fig.* 8.13 shows an inferred example of rock classification based on the results of drilling and seismic prospecting. This figure illustrates that C_L Class rock is distributed in a relatively thick in the left bank. Here, the higher the layer, the thicker the weathered zone. In the right bank, on the other hand, the C_L Class rock is quite thick in the vicinity of the river bed, but in other places it joins the C_M Class rock within a distance of 10 m.

The presence of a large fractured zone has been confirmed by the drilling (LD-4) conducted at the levels between 72 to 80 m deep in the river bed of the right bank. Since this fractured zone lies deep underground, it is presenting problems regarding the bearing capacity of foundation rock for a dam.

The average depth of foundation rock from ground surface, according to rock classification, is shown in Table 8.5.

Table 8.5 List of Depth of Rock Classification of Foundation Rock

(Unit: m)

Location / Rock Classification	Left Bank			Center of River Bed	Right Bank		
	Near Crest	Near Midpoint	Near Bottom		Near Bottom	Near Midpoint	Near Crest
C_L	5	7	-	3	-	-	3
C_M	20	16	5	5	5	-	7
Over C_H	33	38	25	23	15	12	28

According to rock classification, the assessment of foundation rock to be used for some 40-meter concrete gravity dams is given as follows.

Due to the progress of weathering, the D Class rock (completely weathered rock) no longer has its source rock texture, but is found in sand ∿ clay. Conditions like these are not desirable for the foundation rock of a dam.

The C_L Class rock, which mainly comprises highly weathered rocks, is relatively fragile and capable of being broken by hammer. If adequate foundation treatment is provided, this class of rock can be used as the foundation for the upper part of a dam where no great bearing capacity is required.

The C_M Class rock is a comparatively hard and moderately weathered rock, in which the development of cracks is observed. It is considered to have satisfactory shear strength (about 10 kg/cm^2) meeting the conditions required for the foundation of some 40-meter concrete gravity dam. The C_H Class rock is made up of hard, fresh and slightly, weathered rocks, and presents no problem when used for dam foundations.

As mentioned above, the dam excavation line must be based on the C_M Class rock, after removing C_L Class rock. If adequate foundation treatment is provided, however, the C_L Class rock is applicable to the upper part of a dam where no great bearing capacity is required.

Drilling core observation set the boundary line of rock classification between C_L and C_M Class (Fig. 8.13*) within the range where the C_M Class rock accounts for more than 80 percent of rocks over the length of 5 m, as illustrated in Fig. 8.15*.

The permeability of foundation rock, on the other hand, has been confirmed by the Lugeon Map shown in Fig. 8.16*. A high permeability zone (above 50 Lu) is distributed at the near surface right and left banks and river bed, forming a layer about 5 to 10 m thick.

From the aspect of permeability, it is necessary to study a dam excavation line by which high permeability zones above 50 Lu can be completely removed.

The excavation line studied in view of rock classification and permeability, as mentioned above, is shown in Figs. 8.13* and 8.16. Table 8.6 below shows the depth of excavation lines from ground surface.

Table 8. 6 Proposed Excavation Depth of Lower Dam

(Unit: m)

	Left Bank			Center of River Bed	Right Bank		
	Near Crest	Near Midpoint	Near Bottom		Near Bottom	Near Midpoint	Near Crest
Depth(m)	12	18	13	10	12	21	12

(2) Study of Foundation Treatment

As evident from the Lugeon Map in Fig. 8.16*, the Lugeon value does not decline even deep into the lower part, indicating that a high permeability zone about 10 Lu extends continuously into the depths. Permeable zones above 20 Lu are also distributed in the fractured zone, which has been confirmed in LD-4.

Generally, curtain grouting can be improved within the range of $L = \frac{H}{3} + \alpha$ ($\doteqdot \frac{H}{2}$, L: grout depth, H: dam height, α: allowance length). However, since a high permeability zone extends deep into the lower dam site, it is advisable to secure complete water stop by performing curtain grouting up to the third borehole (3-meter pitch), with the improvement target being set at 10 Lu.

Since the high permeability zones are present in this dam site, it is necessary to undertake a program of deeper drilling of pilot holes, thereby confirming that there is no high permeability zone in depths below the curtain grouting.

Consolidation grouting in the dam foundation shall be performed not only to increase the bearing capacity of foundation rock but also to ensure the water stop.in the vicinity of rock joints. The depth of consolidation grouting shall be set at approximately 7 m, and it is advisable to perform adequate rock improvement.

8.3.2 Power Station

It is desirable to build a power station in a relatively flat place on the right bank of the Tekai River.

As shown in Fig. 8.17,* the site of the power station is geologically composed of sandstone and shale overlain by top soil and talus deposits. The thickness of river bed deposits is estimated at 3 to 5 m. The borehole (LD-7, LD-8) has revealed that there is a layer of C_L Class rock (highly weathered) about 10 m thick. It is necessary, therefore, to establish a foundation of C_M Class rock, after removing C_L Class rock.

8.4 CONSTRUCTION MATERIALS FOR THE LOWER DAM

About 91,000 m^3 of concrete aggregate are necessary for the lower dam site, including the lower dam and incidental structures.

In searching for a place from which concrete aggregate is obtainable, we made a study of the lower quarry site (Site C) and alternative quarry site (Site D), which had been selected by a geological survey.

8.4.1 Lower Quarry Site (Site C)

Site C is an area geologically predominated by sandstone. As shown in Figs. 7.1 to 7.14 and Table 8.7, the results of rock testing on sandstone have made it clear that concrete aggregate of good quality is available at Site C, but it is estimated that the thickness of weathered layer for core materiale is not so thick.

Table 8.7 Results of Rock Testing

(Mean value)

	Specific gravity (D.S.)	P. of W.A. (%)	U.C.S. (kgf/cm^2)	M. of E. dynamic ($\times 10^3$ kgf/cm^2)	Poisson ratio dynamic	Velocity P wave ($\times 10^3$ m/sec)	Velocity S wave ($\times 10^3$ m/sec)
Sandstone	2.520	1.99	1221	330	0.31	3.608	1.943

A geological profile based on the results of drilling and seismic prospecting, as well as an isopach map of top soil and weathered zones, are shown in Figs. 8.18 and 8.19 respectively.

The foundation of Site C features low seismic velocity, i.e. 2,500 to 2,800 m/sec. at most. Also, weathered rock of C_L and C_M Classes was found by the drilling. It is considerated that the fresh rock above C_H Class which is similar to the rock testing samples, can rarely find.

Under these conditions, it is by no means easy to obtain concrete aggregate effectively here, leading to the conclusion that Site C is not suited for a quarry site.

8.4.2 Alternative Quarry Site (Site D)

As illustrated in Fig. 1.2, Site D is located near Kg. Lubuk Payong on the left bank of the Tembeling River, about 7 km downstream from the junction of the Tembeling and Tekai Rivers. The geological conditions of this area are already known as reference can be made on

"Geological Investigation for Proposed Quarry Site, near Kg. Lubuk Payong, Jerantut, Pahang (Geological Report)" conducted in 1974 by the Geological Survey of Malaysia at the request of N.E.B. Figs. 8.20 to 8.24 show the geological maps and profiles given by this report.

Site D is composed of sandstone and limestone, strata (limestone) having a thickness of 250 to 340 feet (75 to 102 meters), striking north-northwest and south-southeast and dipping east at an angle of about 40°.

The rock testing was rewarded with favorable results, the details of which are shown in Figs. 7.1 to 7.14 and Table 8.8.

Table 8.8 Results of Rock Testing

(Mean value)

	Specific gravity (D.S.)	P. of W.A. (%)	U.C.S. (kgf/ cm^2)	M. of E. dynamic ($\times 10^3$ kgf/cm^2)	Poisson ratio dynamic	Velocity P wave ($\times 10^3$ m/sec)	Velocity S wave ($\times 10^3$ m/sec)
Limestone	2.701	0.15	954	925	0.287	6.627	3.603

Site D is a good site for a quarry, as the top soil here is thin and there is assumed to be a generous quantity of concrete aggregate available for the lower dam.

8.4.3 Working Program for Construction Materials

91,000 m^3 of concrete aggregate are necessary. About 100,000 m^3 of site volume will become necessary when the yield rate and bulking factor are taken into consideration.

Good concrete aggregate is not available at Site C, where excavated materials mainly consist of weathered rocks, incapable of being appropriated or re-used. These conditions lead us to the conclusion that Site D is suited for a quarry; otherwise, the quarry must be selected on the basis of a comparison with the upper quarry site (Site B).

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