REPUBLIC OF INDONESIA

REPORT
ON
DETAILED DESIGN
FOR
NEW RAILWAY LINE FOR CENGKARENG AIRPORT

AUGUST 1984

JAPAN INTERNATIONAL COOPERATION AGENCY

(JICA)

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PREFACE

In response to the request of the Government of the Republic of Indonesia, the Government of Japan decided to conduct a Detailed Design of the Project to construct a New Railway Line linking the city of Jakarta with the Cengkareng Airport to be opened in April 1985, and entrusted the work to the Japan International Cooperation Agency (JICA). The JICA sent to Indonesia a study team headed by Mr. Masanao Koyama, Director of the Japan Transportation Consultants in July 1983 under the guidance of the Supervisory Committee chaired by Mr. Hitoshi Takashima, Director of Administration Division, Department of Research and Data Processing, Transport Policy Bureau, Ministry of Transport.

The team held discussions with the officials concerned of the Government of Indonesia on the Project and conducted a field survey over a period of three months in Indonesia. Subsequently, further studies were made in Japan and the present report has been prepared.

I hope that this report will serve for the development of the Project and contribute to the promotion of friendly relations between our two countries.

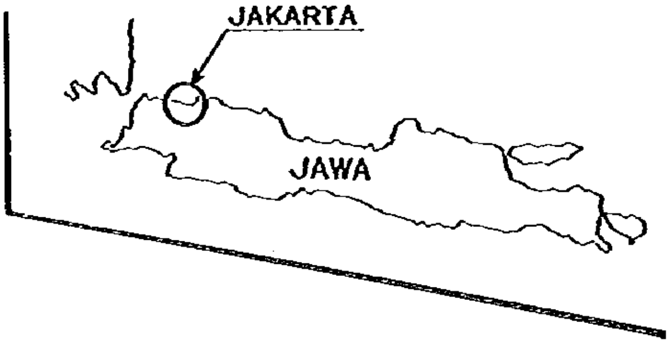
I wish to express my deep appreciation to the officials concerned of the Government of the Republic of Indonesia for their close cooperation extended to the team.

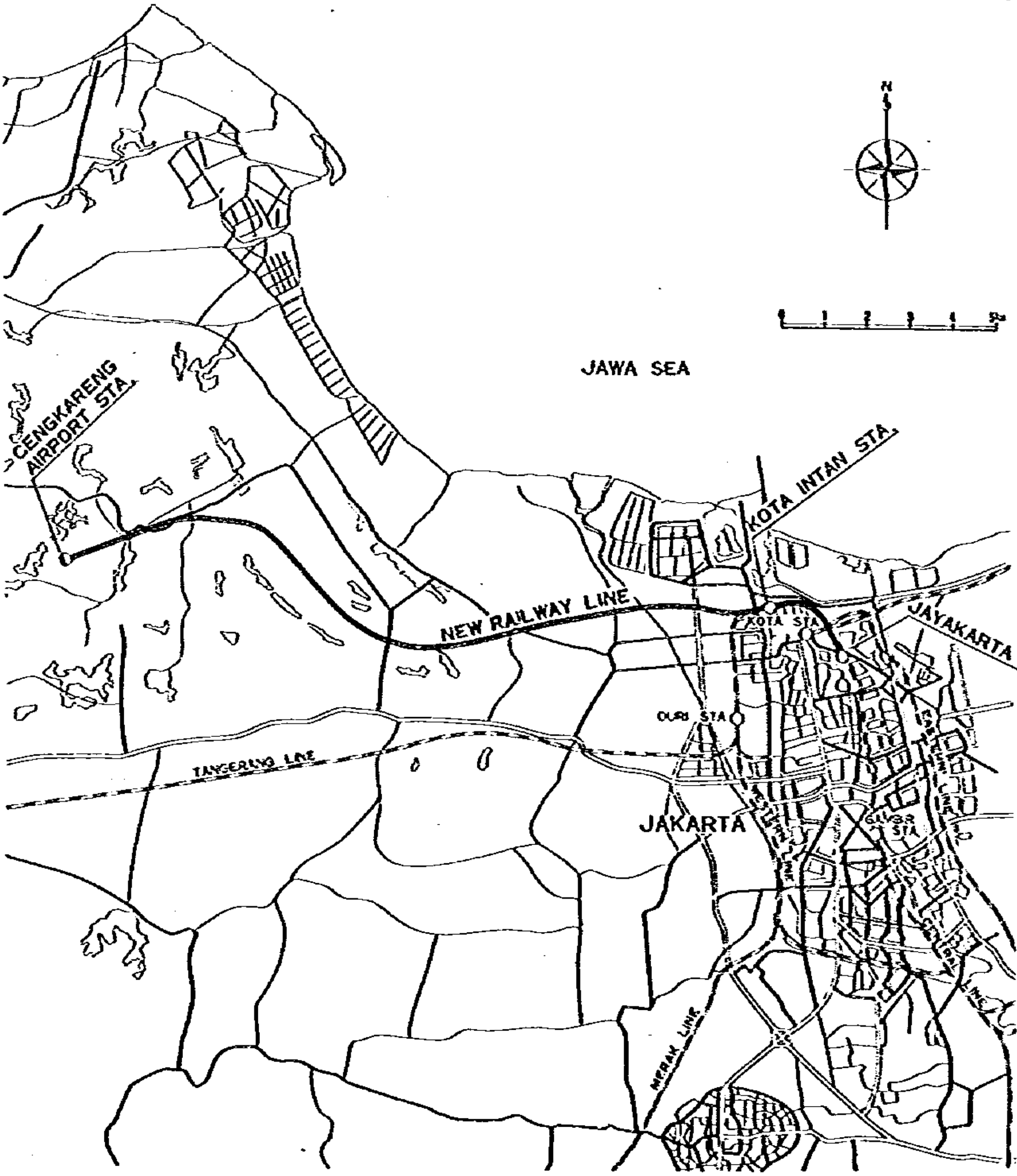
August 1984

Keisuke Arita

Keisuke Arita
President
Japan International Cooperation Agency

PROJECT LOCATION MAP





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1 INTRODUCTION

1. INTRODUCTION

A new international airport is being constructed at a location apart approximately 15 km from the Jakarta City towards north-western district and the first phase construction is to be completed by December 1984 to cope with the ever increasing air traffic demand.

It was planned by the Government of Indonesia to construct a new railway line to connect the new airport and the hub of Jakarta as a most appropriate means to serve not only air passengers but also well-wishers or personnel in their activities at airport area.

By request of the Government of Indonesia, the Japanese Government conducted feasibility study by JICA Study Team which completed their work and submitted the final report in the month of July 1983.

In consequence to the completion of the feasibility study, detailed design team was dispatched by the Japanese Government and investigation was commenced in July 1983.

During three months of the works in Indonesia, the detailed design team carried out various investigations at site and collected necessary data and information with the cooperation extended by the Government of Indonesia. The findings of the study in Indonesia were incorporated in the Progress Report. Explanation of the Progress Report and discussions were held on the later part of October, 1983.

After return to Japan, the team developed their study further in accordance with the principles and comments as specified in the Record of Discussion. Detailed design was followed by preparation of drawings, documents and cost estimate and the Interim Report was prepared.

Through the period of February 28 and 29, 1984, explanation for the Interim Report and discussions were taken place and the result was recorded on the Record of Discussion for which the Government of Indonesia and Japanese Supervisory Committee mutually agreed upon and signed.

The team completed Final Report by making final adjustment to the contents as well as incorporating various comments on the Record of Discussion.

The scope of works under this study comprised the following:

(1) Technical Basic Investigations (Surveying, Geological Survey and Hydrological Survey)

(2) Detailed Design and Preparation of Drawings

(3) Preparation of Technical Specifications and Bill of Quantities

(4) Preparation of Bid Documents

(5) Construction Cost Estimate

This study was carried out by the following organizations.

Indonesian Steering Committee Members

1.	Ir. Giri S Hadihardjono MSE:	Directorate General of Land Transport and Inland Waterways
2.	Gatot Soedjantoko:	Directorate General of Land Transport and Inland Waterways
3.	Ir. Wuryanto:	Directorate General of Air Transportation
4.	Hajadi:	Jakarta International Airport Cengkareng
5.	Sampoerna Rafioedin:	Indonesian State Railways
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7.	Nurdin Manurung:	Directorate General of Bina Marga
8.	Agung Alfian:	Birro I Department of Transportation
9.	Ir. Pantiarso:	Jabotabek Railway Project
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Japanese Supervisory Committee Members

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3.	Mr. Koichi, AOKI (Soil Structure)	Railway Construction Public Corporation
4.	Mr. Kenichi, NOGAMI (Track)	Ministry of Transport
5.	Mr. Kaoru, UMEMOTO (Station)	Japanese National Railway
6.	Mr. Nobuyuki, KUMAGAI (Electrification)	Japanese National Railway
7.	Mr. Satoru, ONOYAMA (Signal, Telecom)	Ministry of Transport
8.	Mr. Nobuo, HAZEYAMA (Cost Estimate)	Overseas Economic Cooperation Fund

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5.	Mr. Tatsujiro, KUBO	Surveying
6.	Mr. Masaaki, SHIMOJIMA	Surveying
7.	Mr. Harumi, NANMA	Surveying
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9.	Mr. Makoto, MASUDA	Surveying
10.	Mr. Tsutomu, KURATA	Surveying
11.	Mr. Tadayuki, YAMADA	Surveying
12.	Mr. Fumio, NISHIYASU	Surveying
13.	Mr. Katsuya, YONEYAMA	Surveying
14.	Mr. Toshiyuki, TERAKI	Geological Survey
15.	Mr. Hiroshi, AKIMOTO	Geological Survey
16.	Mr. Takatoshi, YAMADA	Roadbed
17.	Mr. Shigeto, KURIMARU	Roadbed
18.	Mr. Kairyu, AGATA	Bridge & Viaduct
19.	Mr. Kenji, SATA	Bridge & Viaduct
20.	Mr. Masataka, YABUKI	Bridge & Viaduct
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23.	Mr, Shinji, SHIBATA	Bridge & Viaduct
24.	Mr. Isamu, MIZUNO	Track Layout
25.	Mr. Toshiaki, NAKAMURA	Track Layout
26.	Mr. Makoto, KAGAMI	Track
27.	Mr. Shuichi, ONUMA	Track
28.	Mr. Kimihiko, MITSUMOTO	Track
29.	Mr. Masayuki, KAMAE	Track
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31.	Mr. Koichi, ISUMI	Air Terminal Station
32.	Mr. Kiyomasa, USHIROGATA	Telecommunication

33.	Mr. Noriyuki, TERAUCHI	Transformer
34.	Mr. Shoji, HASEGAWA	Electrification
35.	Mr. Norikazu, YAMAUCHI	Power Supply
36.	Mr. Tsuneharu, AJIMI	Signalling
37.	Mr. Tamotsu, NAKAJIMA	Tender Documents, Technical Specifications
38.	Mr. Yukichi, USHIISHI	Cost Estimates
39.	Mr. Kazuhiko, DENDA	Cost Estimates

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34.	Ir. James Tampubolon	Site Surveyor	JAYA CONSULTANT
35.	Ir. Suharto	Site Surveyor	JABOTABEK

2 BASIC PRINCIPLES OF THE STUDY

2. BASIC PRINCIPLES OF THE STUDY

The detailed design of New Railway Line for Cengkareng Airport, the main objective of this study, was conducted following basic principles:

(1) Role of the Proposed New Railway Line

The proposed railway line will link the new Cengkareng Airport with the existing JABOTABEK railway network to provide passengers with rapid, safe and punctual transportation between the new Airport and downtown Jakarta.

Keeping in mind this role of the proposed New Railway Line and on the basis of the Report on Feasibility Study of New Railway Line for Cengkareng Airport (July 1983, JICA), the optimum design for the above mentioned purpose for the route selected by the Government of Indonesia has been prepared.

(2) Outline of the Proposed Railway Line

The major items on the proposed railway line are as follows:

Route	: Route connecting Cengkareng Airport Station and the proposed Jayakarta Station on the Central Line.
Gauge	: 1,067 mm
No. of Tracks	: Single track (Double track in the future)
No. of Stations	: Two; Airport Station and Kota Intan Station
No. of Signal Stations	: Two
Power System	: Electrified, C.C. 1,500 V
Rolling Stock	: Railcar (The performance characteristics will be identical to those now being used by JABOTABEK)
No. of Cars per Train	: Four (Eight in the future)
Maximum Speed	: 100 km/h
Minimum Headway	: 20 minutes (10 minutes in the future)
Block System	: Automatic block system

(3) Compatibility with Related Projects in the JABOTABEK Area

This project has been planned as a part of the JABOTABEK Project and it must, therefore, be implemented to be compatible with other parts of the JABOTABEK Project. In particular, as this project is closely related to electrification planning of the Western Line, for which the detailed design has already been completed, and the plan to elevate the Central Line, for which the feasibility study has already been completed, the design work must be conducted in consideration of these plans to avoid unnecessary investment.

(4) Connection with the Central Line

Connection of the proposed railway line with the Central Line, which is planned to be elevated, will be an area adjacent to Jayakarta. However, taking account of the possibility of the elevating project being delayed, the detailed design in this study will be for the connection of the proposed railway line with the existing Central Line at the ground level. A tentative design is also made for connecting the proposed line with the future elevated Central Line.

(5) Aesthetic Considerations and Protection of Environment

The proposed railway line links the modern new airport terminal with downtown Jakarta, transporting not only local people but also a number of foreign passengers, and it would be out of the most important transportation means to represent the Republic of Indonesia. Great care must, therefore, be paid in designing the airport station building and its facilities to keep them in architectural harmony with other airport facilities. The same care must be paid to the design of Kota Intan Station and other facilities.

Since the proposed railway line passes through the residential, commercial and industrial areas in the center of the city and its surrounding areas, consideration should be given to minimize vibration and noise generated by moving trains and care should be taken in the design of various structures so that they do not mar the appearance of the urban area as well.

(6) Maintenance of Safety, High-speed and Punctuality of Train Operation

The proposed railway line is planned to serve the passengers between the airport and city with safe, high-speed and punctual transportation. In order to meet these conditions, automatic block system, relay interlocking system, automatic train stop system, automatic barriers at level crossings and other necessary systems will have to be incorporated. At the same time, the same care will have to be taken in the design of alignment, track layout and track structures.

(7) Relationship between Single Track and Future Double Tracking

As described in Paragraph (2), the proposed railway line has been designed for a single track in the initial stage, but double tracking is scheduled in the future to cope with expected increase in the traffic demand. Due consideration will be paid to this future possibility when making the initial design.

For instance, it would be desirable to acquire railway right-of-way for double tracking in the initial stage. Also, it would be expendient to design the piers and abutments for the double track at the time they are designed for the single track.

(8) Cost Saving

The basic task of design is to construct facilities which will function adequately and efficiently at the lowest possible cost. This must be regarded when designing the proposed railway line. Although it is important to keep in mind the need for double tracking in the future due to increase in traffic demand, care must be paid to keep down the construction costs in the initial stage.

3 TECHNICAL BASIC INVESTIGATIONS

3. TECHNICAL BASIC INVESTIGATIONS

3–1 Surveying

3–1–1 3rd and 4th Grade Control Point Surveying

A 3rd and 4th grade control point survey was conducted at the DKI control points established near the centerline of railways. The DKI numbers of the control points used in this survey are as follows: DKI–394, 397, 438, 439, 445, 446, 538 and 539.

(1) Specific Features of DKI Results

The ring closure method was applied to the DKI surveying. Its network diagram is given below.

(2) Work Method used in the Project

Since these are the DKI Results as stated above, there is no problem about the data on each block. But a torsion phenomenon, which is apt to take place in the ring closure method, occurred at the adjacent connecting point between blocks. When surveying nearly 20 kilometers of the route in this project, a similar phenomenon occurred at the point where two blocks adjoin each other.

To avoid such problems, no connection was made at the DKI point established halfway between the route, but a substitute connection was made at a point located near the terminal.

(3) Comparison between DKI Results and Remeasured Results

Comparison between the DKI results and remeasured results is as shown in Table 3–1–1.

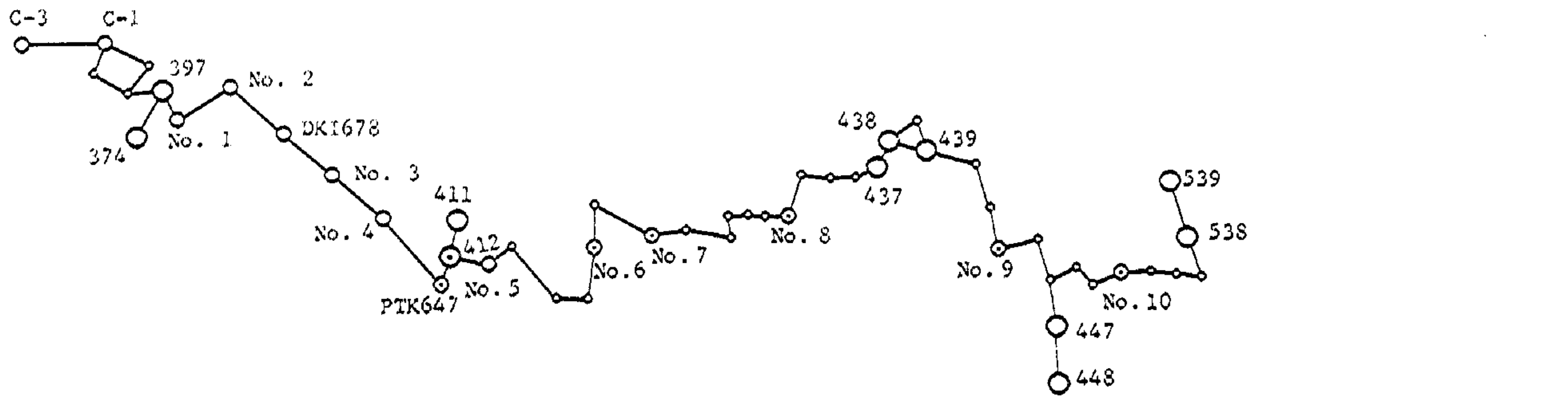


Fig. 3-1-1 3rd Grade Control Points Network Diagram

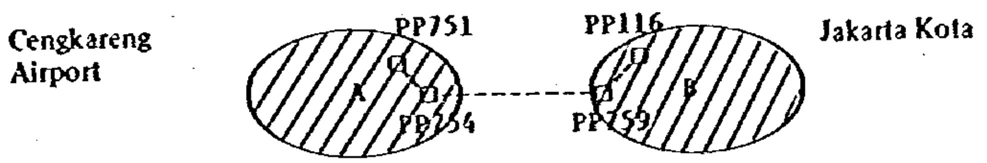
Table 3-1-1 Comparison between DKI Results and Re-measured Results

Upper: DKI Results

Lower: Re-measured Results

Control Points	X	Y	Vector ΔS
DKI 412	+ 11595.976	– 7497.070	
	+ 11596.156	– 7497.050	6°20'25"
Difference	– 0.180	– 0.020	0.181
DKI 438	+ 12855.529	– 491.908	
	+ 12855.243	– 492.037	204°16'40"
Difference	+ 0.286	+ 0.129	0.314
DKI 439	+ 12805.836	+ 201.216	
	+ 12805.709	+ 201.117	217°56'15"
Difference	+ 0.127	+ 0.099	0.161
DKI Control Points used as Additional Points			
DKI 445	+ 10828.968	+ 1907.207	
DKI 397	+ 14060.619	– 12040.287	
DKI 539	+ 11054.615	+ 3910.710	

3-1-2 Bench Mark Surveying



A 0.217 m level difference was observed in Areas A and B during the bench mark survey at points PP751 to PP759 (accident points for the construction work in this survey), which was conducted in April 1983. The level difference was again inspected during the route surveying work and confirmed that the previous 3rd grade bench mark survey was still within the prescribed limit.

Consequently, the results from DKI bench marks located in Area B were applied to the points of bench mark necessary for route surveying. Also, 0.217 meters was added to the height data on DKI bench marks located in Area A.

Results of the measurement conducted at points PP751 through PP754 to PP116 are as follows:

Previous measured result		Re-measured result	
I (Going)	– 0.136	– 1.492	
II (Coming)	+ 0.134	+ 1.487	
			PP751 ∿ PP116
Average	– 0.135	– 1.490	– 1.625

As a result using DKI results:	PP 751	2.772
	PP 116	1.311
	Difference	– 1.461

ϵ = (Difference - Average) = + 0.164

Result of 3rd grade bench mark surveying based on B area:

PP 751	2.989 (result as added with + 0.217)
PP 116	1.351
Difference	–1.638

$\epsilon = -0.013$

Table 3-1-2 Comparison with DKI Bench Mark Results

DKI	PP No.	I DKI Results	II Results Based on B Area	II – I = ϵ
(A)	PP 705	4.509	4.726	+ 0.217
	PP 707	4.043	4.260	+ 0.217
	PP 711	4.100	4.317	+ 0.217
	PP 712	3.512	3.729	+ 0.217
	PP 716	5.196	5.413	+ 0.217
	PP 721	4.526	4.743	+ 0.217
	PP 730	5.914	6.064	Accident Point + 0.150
	PP 728	6.199	6.416	+ 0.217
	PP 751	2.772	2.989	+ 0.217
	PP 754	2.732	2.850	+ 0.118
	PP 757	3.172	3.733	Accident Point + 0.561
	PP 718	6.096	6.313	+ 0.217
(B)	PP 116	1.311	1.351	Accident Point + 0.040
	PP 125	3.509	3.509	0
	PP 130	2.811	2.811	0
	PP 131	2.906	2.906	0
	PP 137	1.603	1.603	0
	PP 780	1.769	1.661	Accident Point – 0.108
	PP 128	1.251	1.092	Accident Point – 0.159
	PP 778	2.452	2.452	0
	PP 759	1.991	1.991	0

The altitude of bench mark of which $\epsilon = 0$ at B Area was used for the bench mark surveying under this study.

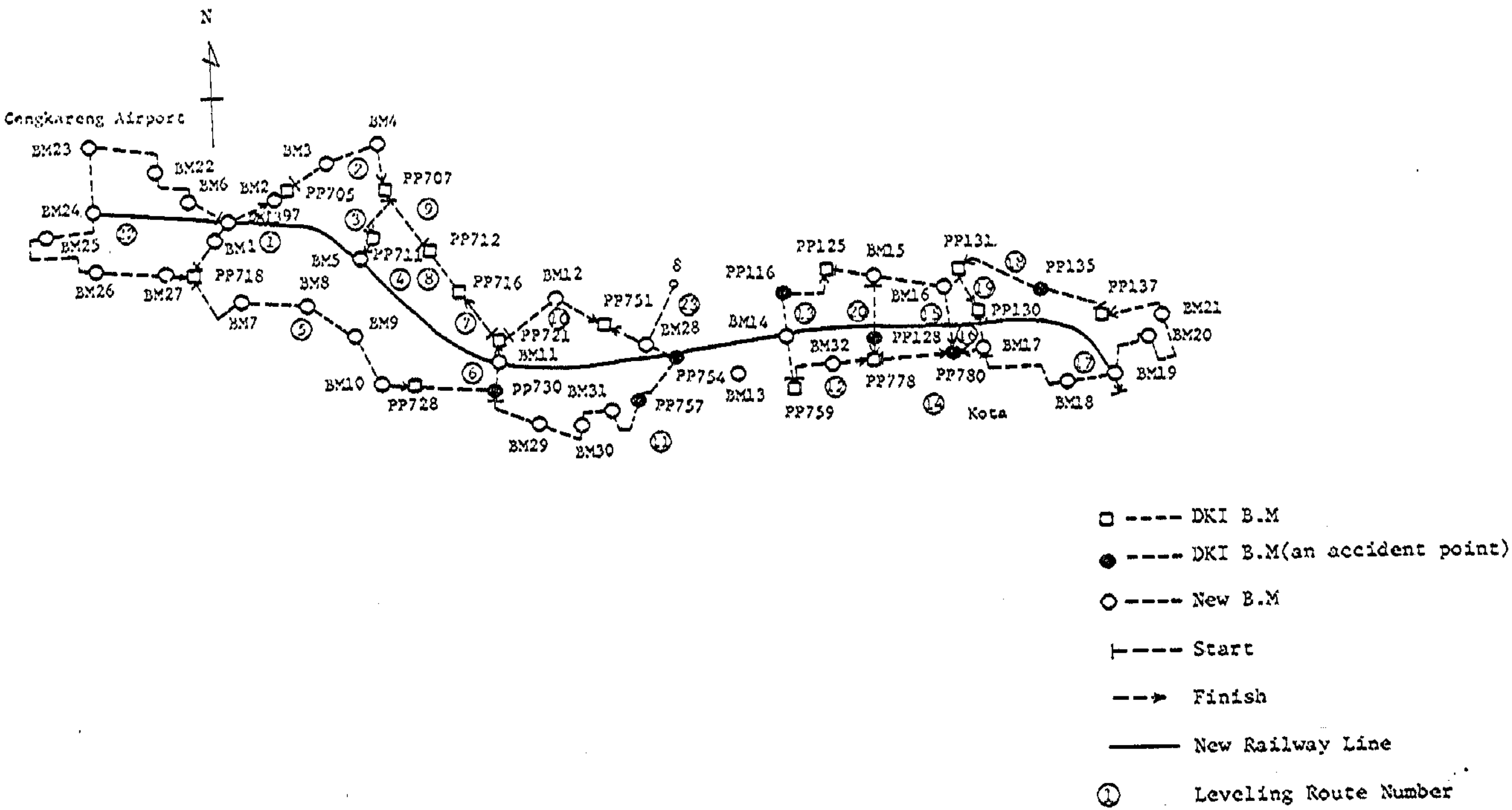


Fig. 3-1-2 Leveling Route

3–1–3 Relation to the Bench Marks in Cengkareng Airport

As already stated in the chapter on bench mark survey, all DKI bench mark data in Area A were modified on the bench marks in Area B, considering the 0.217m difference in level between Areas A and B. The results of this modification are given below:

Measuring Point	I Results of Airport	II DKI Results Kota (B Area)	(I – II) Difference
C1	6.129	6.094	+ 0.035
R 500	5.546	5.502	+ 0.044
R 501	6.188	6.149	+ 0.039
R 502	6.690	6.645	+ 0.045
C3	7.562	7.528	+ 0.034

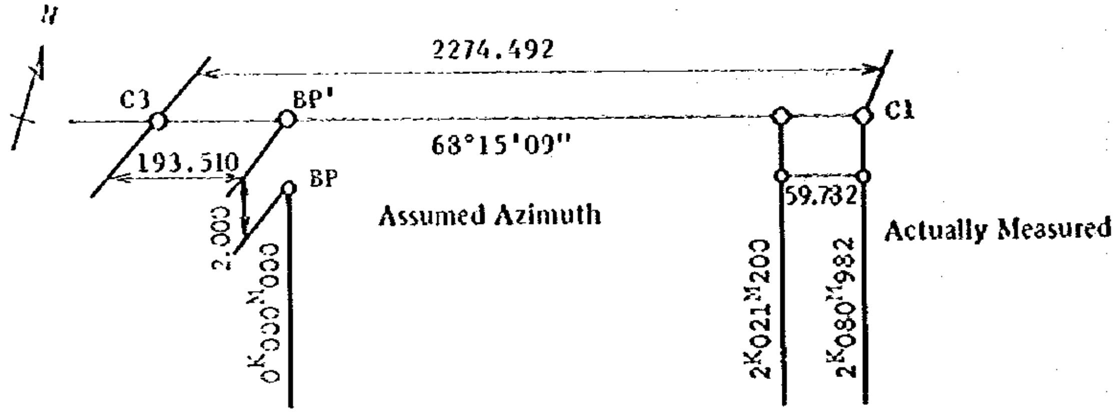
In view of the data above, it would be safe to say that all these coincide with the bench marks on the Kota side (Area B), granted a margin of error of + 0.04m.

3–1–4 Starting Point of Railway in Cengkareng Airport

The distance between C1 and C3 (measuring points for construction work) established in Cengkareng Airport was measured. The result of this measurement are as follows:

Result of airport authorities (C3∿C1):	2,274.492m
Result of remeasurement (C3∿C1):	2,274.537m
Difference:	+ 0.045m

In view of the above figures, it seems there is no question about the distance between C1 and C3. At the present time, there is a bridge under construction located near the 2km040m point. Some gap was observed in the distance between this bridge and the airport borderline which is established near the 2km097m point. Then, by means of a 1/500 plan, a computation was made on the basis of the 193.510m distance between C3 and BP' (railway starting point). As a result, the point 2.000m at right angles south of BP' was determined as the starting point of railways. The distance from the central point of the bridge was calculated on the basis of the actually measured figure, whereas the airport borderline was established at the point of C1. The DKI results were applied to the calculation of azimuth angles. The data of the airport authorities were applied to the distance between C1 and C3, which was then converted into the DKI results.



BP Coordinate U + 13069.183
(DKI Result) T – 14854.400

3–1–5 Centerline Surveying

(1) Installation of Center Stakes

By applying the coordinate system to the 4th grade control points, the center stakes were installed at intervals of 50m as well as at change points and main points on the curve. Bamboo sticks, 2 to 3m long, were used in places such as damp ground and paddy field, where center stakes could not be installed. Raking piles were installed in both lateral and longitudinal directions at the points including a housing area, where center stakes could not be installed.

(2) Profile Levelling

Profile levelling was carried out in areas where centerline surveying was performed.

(3) Cross Levelling

Cross levelling, extending over the width of 60m, was conducted at intervals of 50m as well as at main and altering points. In the proposed station areas, cross levelling was performed within the prescribed limits.

(4) Raking Pile Installation Surveying

Raking piles were installed at main points on the curve, i.e. BTC, BCC, IP, SP, ECC and ETC. In view of site conditions and work accuracy, triangular raking piles were used instead of X and T type ones, both of which were considered unsuitable.

The decision to use triangular raking piles was based on factors such as maintainability and durability of piles as well as substitution for traverse points. The points for a triangular raking pile were selected and established so that two or more triangles could be formed per main point.

(5) Accuracy and Color Identification for Center Stakes and Raking Piles

The stakes for centerline surveying were installed conforming to the specifications. Observation of raking piles was made in conformity with the specifications of 4th grade control points. Color identification was ensured by red paint for the centerline, blue paint for the main point, and white paint for the raking pile.

3-2 Geological Survey

3-2-1 Topography

Although the proposed route for the railway line for Cengkareng Airport is mainly located at coastal plain, its topographical features are classified into the patterns by the following three zones:

Zone adjacent to airport (Western area)	0 – 4.0 km
Zone between airport and suburb of Jakarta (Central area)	4.0 – 13.5 km
Urban zone in Jakarta (Eastern area)	13.5 – 19.5 km

(1) Zone adjacent to Airport

This zone has been formed on the coastal plain with a number of natural levees which has been formed by flood sediments of the river Cisadane, the main river in the region and micro ridges such as point bars which had been developed by a low gradient of river (1/1000).

Natural levees consist of sand gravel that is a relatively coarse andesite and basalt chain among flood deposits and surface layer is loose while deeper layer is well compacted.

In general, upper part of levee consist of silt and fine sand and lower part consist of coarse sand.

Area between micro ridges and low lands consist of swampy area where the sediments of fine grain materials flooded over the natural levees at the time of flood, plants will likely to grow easily because of defective draining, and form a soft layer that includes organic materials.

The difference in height between micro ridges and low lands is a few metres.

(2) Zone between Airport and Suburb of Jakarta

This zone is formed with a beach ridge plain consisting of a number of beach ridges formed in parallel with coastal line. The beach ridge can be seen in the advancing beach and has been formed by a combination of ridges and furrows in between.

Components (materials) are sand, gravel including shells and muds drifted by wave action, tides or current.

Sediments seen in surf zone are moved by the littoral current.

The difference in height between micro ridge and furrow is approximately 2 m, and micro ridges have been utilized for housings and farmes while furrows consists of paddy fields and swamps.

(3) Urban Zone in Jakarta

This zone is located on the delta formed by the river Ciliwung. Since the delta is formed by the deposits such as sand, silt and clay transported in the stream from the hilly area and stuck at the mouth of river because of decrease in the velocity of river flow, deposit consist of coarse sand at the mouth of the delta.

In general, soft clay layer becomes more thicker toward the shore line. Area adjacent to Jalan Juanda of Jakarta there was once beach ridge and it is judged from topographical conditions that the direction of flow was diverted there.

Accordingly, micro ridges had ever located at northern district of Jakarta City, however, site preparation is in progress because of urbanization, the altitude is approximately 2 to 3 m at present and topographic features is nearly plain.

3-2-2 Geology

Geological constitution at the project location is as shown on Table 3-2-1 and its distribution pattern is as shown in Fig. 3-2-1.

Table 3-2-1 Constitution of Geology

Geological Period		Formation	Description
Quaternary	Holocene	Alluvium	Unconcrete sediments such as clayey and sandy soil covering diluvium
	Pleistocene	Diluvium	Volcanic ash covering Genteng Formation
Neogene	Pliocene	Genteng Formation	Fine-grained, tuffaceous sandstone and muddy-rocks

Alluvium is the sediments consisting of unconcrete sandy soil, clayey soil and detrital materials eroded which foms coastal plain, delta, beach ridge, swampy areas and natural levees.

Diluvium is the sediments of volcanic ash soil and mainly has formed a hilly area at southern district and laterized by weathering.

Laterization is those conditions of weathering that lead to removal of silica and alkalies, resulting in a soil or rock with high concentrations of iron and aluminium oxides and containing reddish soil developed from weathering.

Genteng formation consists of fine-grained tuffaceous sandstone, intercalated with coarse-grained layers, also tuff clay, used as fill materials, having a fair bearing capacity.

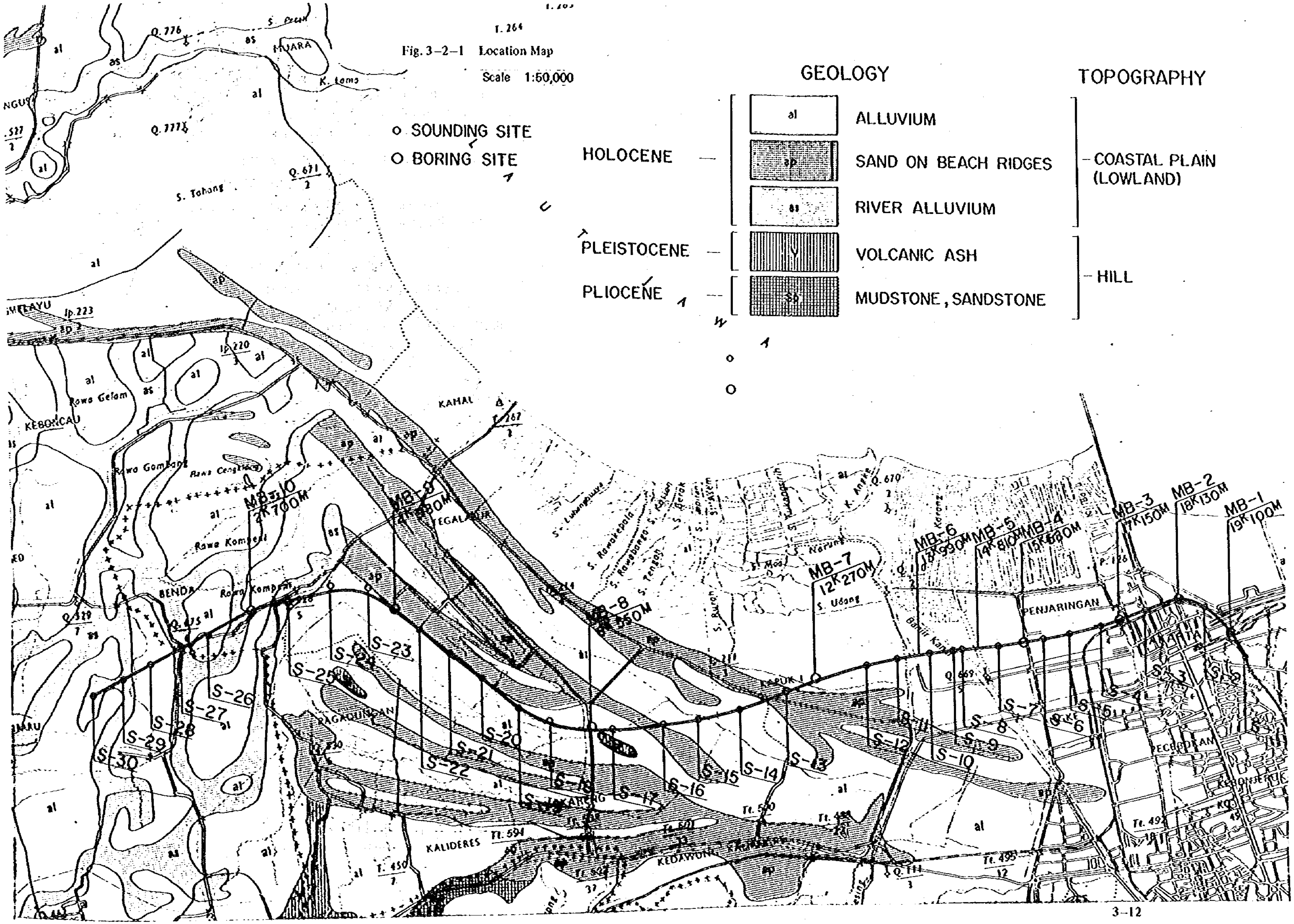


Fig. 3–2–1 Location Map

Scale 1:60,000

3-2-3 Result of Geological Survey

Boring and sounding were carried out along the proposed project location for the planned areas where the prime railway structures to be constructed and soft ground areas for identification of soil bed conditions, bearing capacity and dynamic characteristics of the soil.

Local consultant under supervision of JICA Study Team carried out the geological survey and outline of the work is summarized as follows:

Purpose of the Work : Data collection for design of prime railway structure foundation, i.e., bridge and viaduct and analysis of bearing layer of embankment.

Period of Survey : From July '83 to October '83.

Detail of the Work : Rotary Drilling – up to 20 m depth at 4 areas
– up to 30 m depth at 6 areas
Total: 10 areas and 260 m depth
Standard Penetration Test for every 2 m depth
Total: 130 times
Sounding (Dutch Cone)
Total: 30 areas and 438.2 m depth
For detail, refer to Table 3-2-2.

Location of Survey : Refer to Fig. 3-2-1.

Table 3-2-2 Breakdown of Survey Work

Location No.	Kilometerage (km)	Depth (m)	Standard Penetration Test (time)	Location No.	Kilometerage (km)	Depth (m)	Standard Penetration Test (time)
S – 1	19.47	16.40		S – 14	10.96	11.20	
MB – 1	19.10	30	15	S – 15	10.23	14.80	
S – 2	18.68	23.20		S – 16	9.80	14.60	
MB – 2	18.13	30	15	S – 17	8.96	25.00	
S – 3	17.59	22.80		MB – 8	8.55	20	10
MB – 3	17.15	30	15	S – 18	7.89	9.80	
S – 4	16.81	18.80		S – 19	7.40	14.40	
S – 5	16.37	19.40		S – 20	6.68	25.00	
S – 6	16.03	18.80		S – 21	6.05	11.60	
MB – 4	15.58	30	15	S – 22	5.44	14.20	
S – 7	15.12	16.60		MB – 9	4.98	20	10
MB – 5	14.81	30	15	S – 23	4.50	8.80	
S – 8	14.42	6.00		S – 24	4.03	6.60	
S – 9	14.29	4.00		S – 25	3.31	12.60	
S – 10	14.02	11.80		MB – 10	2.70	20	10
MB – 6	13.99	30	15	S – 26	1.99	8.00	
S – 11	13.40	16.80		S – 27	1.49	18.20	
S – 12	12.99	13.40		S – 28	0.99	22.20	
MB – 7	12.27	20	10	S – 29	0.50	12.20	
S – 13	11.68	8.40		S – 30	0.00	12.60	
Total	Depth {S = 438.20 m, MB = 260 m} Standard Penetration Test (130 times)						

Based on the results of the soil survey, bed conditions and soil characteristics at the above mentioned three zones are described as follows:

(1) Zone adjacent to Airport ($0^{km}000^{m} \sim 4^{km}000^{m}$)

This zone consists of a river-flooded plain, including a number of sandy soil bed forming a complicated bed conditions in the whole. Silt and sand are filled over the natural levees and point bars forming a relatively favorable ground conditions, however, fine grained soil, such as silt and clay are filled over the swampy areas forming a soft ground.

At soft soil bed that is subject to be consolidated at the embankment area, a thickness of bed is relatively shallower in general, namely 1 to 4 m (N value < 5, qc < 10 kg/cm²), however, at location of $2^{km}800^{m}$ a wastefulled valley exists and soft clay sands are filled by approx. 14 m.

This wastefulled valley is assumed to be part of the ancient river and a bed (sandy soil mixed with gravel) that is assumed to be the ancient river bed slightly sedimented in the bottom of alluvium.

The valley similar to this is often observed in the river-flooded plain zone and giving a great variation to the foundation depth in many instances. Also, at the surveyed location, upper part depth of the bed (N > 30, qc > 200 kg/cm²) that can be the bearing layer suitable for the middle and heavy weight structures have been greatly varied from 12 m to 25 m.

(2) Zone between Airport and Suburb of Jakarta ($4^{km}000^{m} \sim 13^{km}500^{m}$)

This zone has been formed with beach ridges and low swampy lands and the proposed route travels on the low swampy lands along the beach ridges from $4^{km}000^{m}$ to $8^{km}000^{m}$ and the beach ridges from $8^{km}000^{m}$ to $13^{km}500^{m}$.

The beach ridges is to be composing mainly sandy soil and relatively favorable ground for the embankment.

However, the extent of the area on which the proposed route traverse is not lengthy and the most of the route traverses the low swampy lands.

The soil characteristics of the upper part of the low swampy lands is composing mainly clayey soil and soft, and a thick sandy soil layer is not intercalated.

The thickness of the bed subject to consolidation over the embankment section ($0^{km}000^{m} \sim 12^{km}765^{m}$) is approximately 3 to 8 m and the sectors where the consolidation settlement could be the problem will be $5^{km}500^{m} \sim 7^{km}000^{m}$, $8^{km}000^{m}$, $11^{km}000^{m} \sim 12^{km}765^{m}$.

The thickness of the upper part of the bed which could be the bearing ground of the elevated track is approximately 14 m.

(3) Urban Zone in Jakarta ($13^{km}500^{m} \sim 19^{km}500^{m}$)

This area is the delta formed by the deposits transported by the flow of the river Ciliwung and filled by the fine-grained soil such as clay and silt.

The alluvium consists of river sediments containing organic materials (detrital material eroded) and shells, and formed by blue-greyish marine deposit and the depth of the layer is approximately 10 to 20 m.

Most of the alluvium is the bed subject to consolidation composing very poor clayey soil and (N = 0) can be seen at MB–1 and MB–2 points.

The elevated track is to be constructed at this sector and type of foundation for the structure will be pile foundation, therefore, consolidation settlement problem due to the construction of railway will not be likely, however, as a result of qc value, strength increase towards the bottom of the alluvium cannot be seen and it is assumed that this would be the bed of which natural consolidation is unfinished; and caused by lowering of water table due to the pumping of industrial water and water for the daily life by the exclusive wells of factories or individual office buildings, therefore, a thorough study will be required to the negative friction acting on the piles.

Also, careful study to the horizontal displacement of the piles will be required as this bed is poor ground and lateral ground reaction cannot be anticipated.

Upper part depth of the bed that could be the bearing ground for the pile foundation is 12 to 23 m and the area between $17^{km}000^{m}$ and $19^{km}000^{m}$ consists of a deep valley.

Fig. 3–2–2 Geological Profile

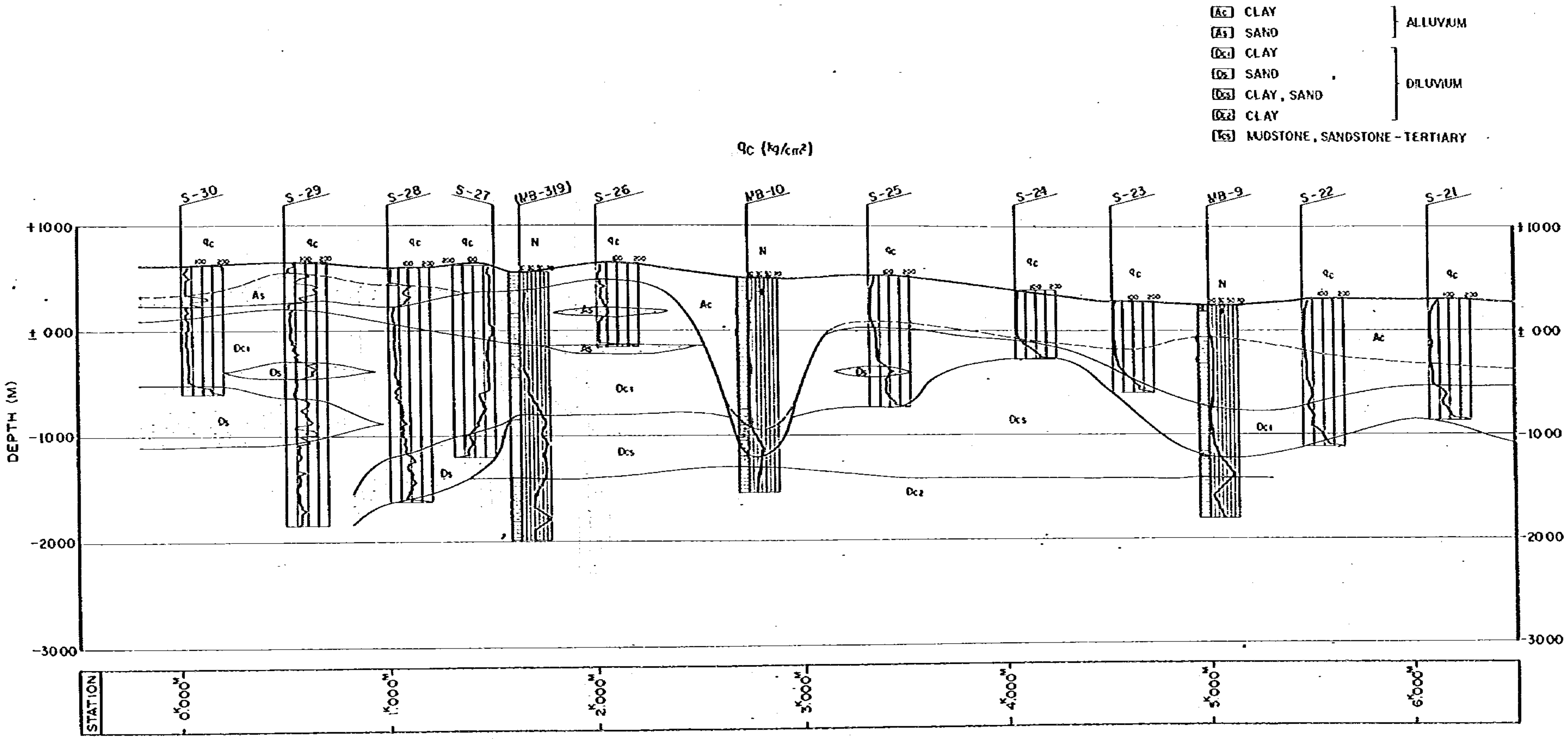


Fig. 3-2-3 Geological Profile

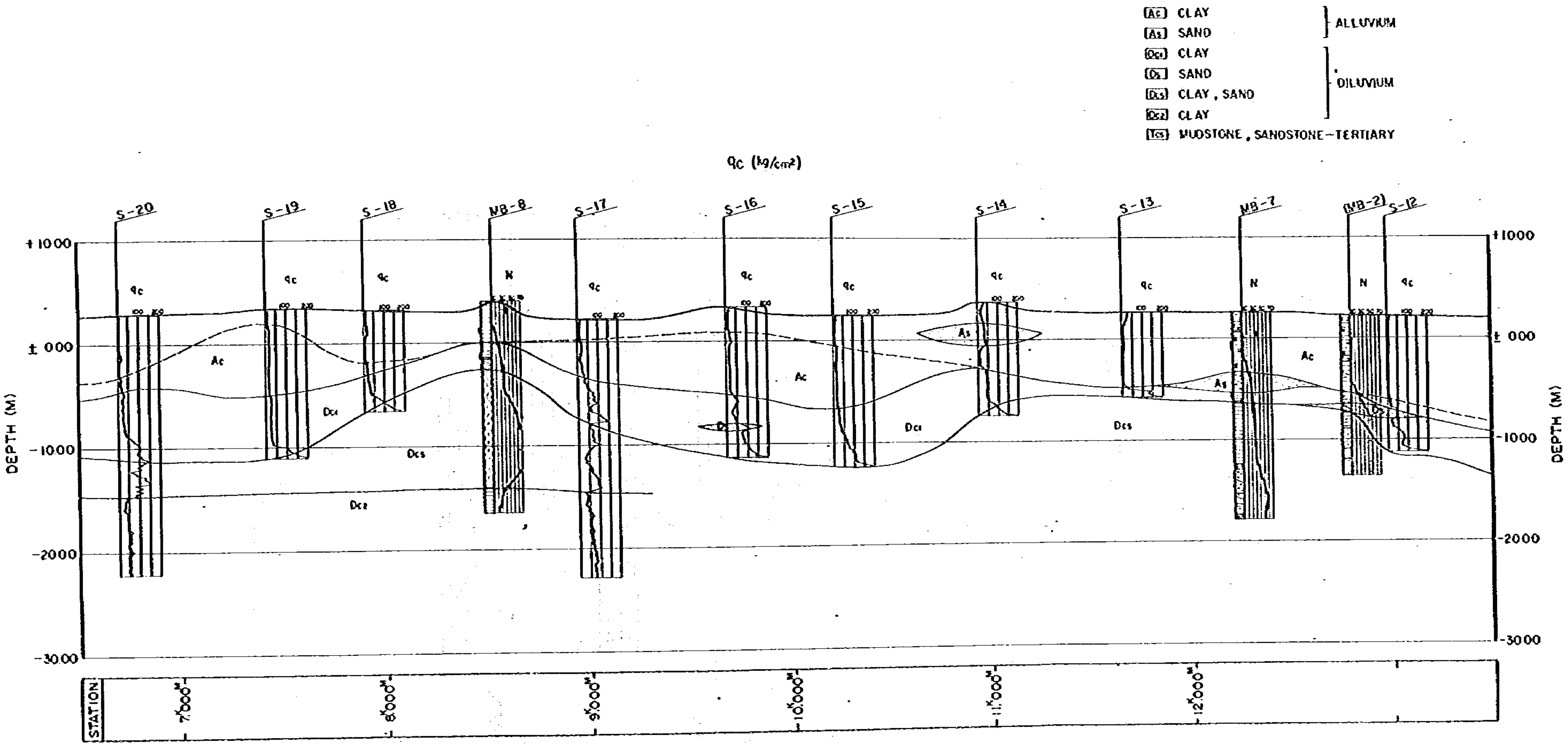
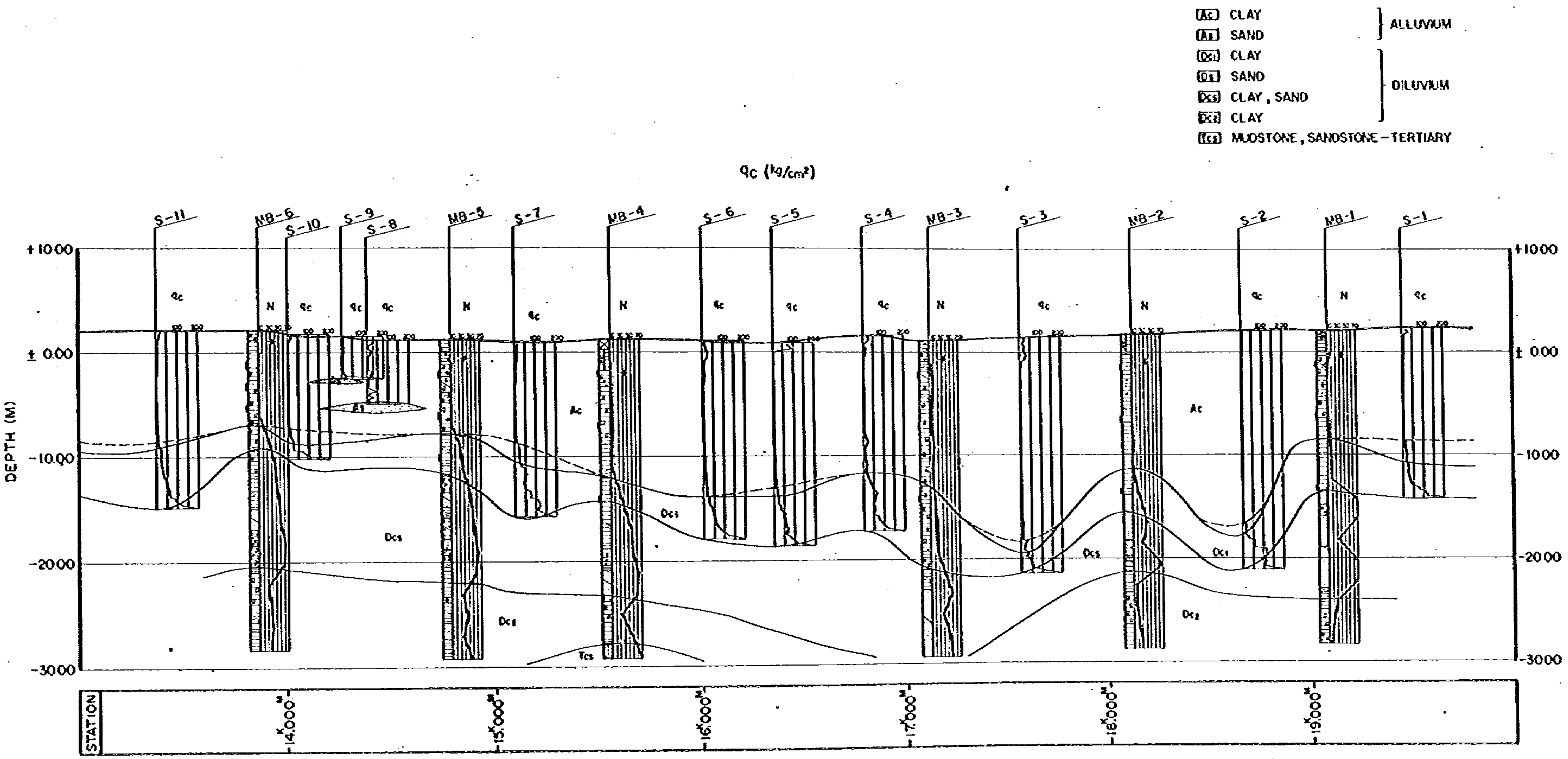
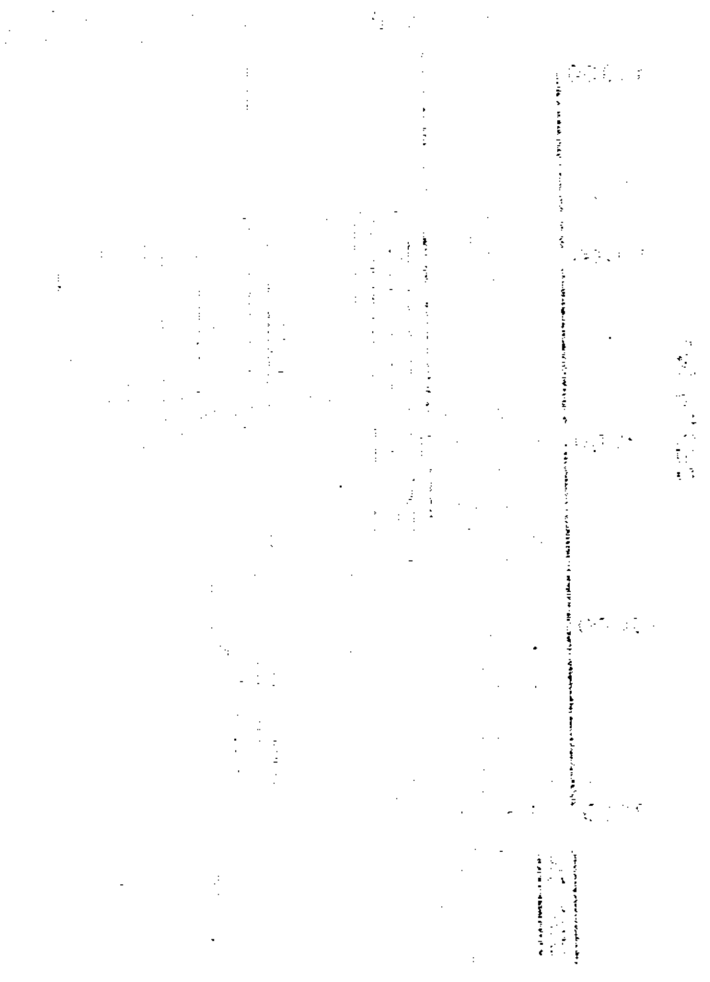


Fig. 3-2-4 Geological Profile





3–2–4 Laboratory Test Result

Undisturbed soil specimens per each bore hole were sampled at three locations where viaduct will be constructed and two locations of embankment sections, and the laboratory tests were made to the following items:

- Specific Gravity
- Water Content
- Grading
- Liquid Limit
- Plastic Limit
- Unit Weight
- Unconfined Compression Test
- Triaxial Compression Test
- Consolidation Test

In addition, triaxial compression test was carried out under the condition of unconsolidated-undrained test for the elevated sector while the test was carried under the condition of consolidated-undrained test for the embankment sector in order to obtain the strength increase ratio conforming to the consolidation in the case of slow banking in addition to the above test condition.

Results of laboratory test and diagram of soil properties are as shown on Table 3–2–3 and Fig. 3–2–5 through 3–2–14.

Table 3–2–3 Results of Laboratory Test (1)

	Sample Number	BM. 1			BM. 2			BM. 3			BM. 4			BM. 5		
	Depth (m)	3.00 ~ 3.50	8.00 ~ 8.50	15.00 ~ 15.50	3.00 ~ 3.50	8.00 ~ 8.50	15.00 ~ 15.50	3.00 ~ 3.50	8.00 ~ 8.50	15.00 ~ 15.50	3.00 ~ 3.60	8.00 ~ 8.55	15.00 ~ 15.55	3.00 ~ 3.60	7.00 ~ 7.60	11.00 ~ 11.60
Grain Size Properties	Gravel (over 2000μ)(%)	0.0	0.0	0.0	0.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Sand (74–2000μ)(%)	71.8	13.8	1.8	5.7	5.1	0.4	84.95	3.8	1.4	10.7	7.0	4.3	3.1	10.5	1.2
	Silt (5μ–74μ) (%)	19.2	55.2	67.7	58.8	58.3	63.6	15.05	64.2	67.6	54.3	67.0	68.7	50.9	51.5	51.8
	Clay (less than 5μ) (%)	9.0	31.0	30.5	35.5	36.0	36.0	0.0	32.0	31.0	35.0	26.0	27.0	46.0	38.0	47.0
	Maxim Grain Size (mm)	4.76	4.76	4.76	2.00	9.52	4.76	9.52	4.76	4.76	4.76					
	Coefficient of Uniformity Uc	28.6	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	Coefficient of Curvature U'c	4.6	–	–	–	–	–	–	–	–	–	–	–	–	–	–
Consistency Properties	Liquid Limit wL (%)	46.80	92.60	91.00	80.00	112.50	85.40	NP	100.80	128.00	102.30	113.00	75.95	94.20	105.10	73.25
	Plastic Limit wp (%)	28.30	47.81	54.51	37.36	33.57	34.72	NP	56.73	65.76	68.75	54.81	48.165	31.285	45.30	48.275
	Plasticity Index Ip	18.50	44.79	36.49	42.64	78.93	50.68	–	44.07	62.24	33.55	58.19	27.785	62.915	59.80	24.975
	Consistency Index Ic	0.336	–0.056	1.016	0.535	0.297	0.649	–	0.403	0.334	0.291	0.223	0.201	0.244	0.092	0.354
Classification of Soil																
Specific Gravity Gs		2.657	2.661	2.658	2.650	2.653	2.662	2.667	2.661	2.664	2.667	2.663	2.664	2.657	2.666	2.658
	Water Content w (%)	40.58	95.13	53.93	57.18	89.08	52.49	26.56	83.05	107.23	92.54	100.0	64.26	78.88	92.62	64.41
	Unit Weight γ_t (g/cm³)	1.815	1.464	1.674	1.670	1.527	1.496	1.932	1.530	1.380	1.450	1.453	1.615	1.541	1.527	1.611
	Void Ratio e	1.058	2.548	1.443	1.495	2.283	1.714	0.747	2.183	3.000	2.542	2.663	1.710	2.086	2.485	1.712
	Saturation Sr (%)	100	99.35	99.34	100	100	81.52	94.83	100	95.22	97.09	100	100	100	100	100
Strength and Consolidation Properties — Unconfined Compression Test	Compressive Strength qu (kg/cm²)	0.081	0.070	1.203	0.135	0.113	0.505	NP	0.089	0.420	VS	VS	0.387	0.090	0.090	0.350
	Modulus of Deformation E_{50} (kg/cm²)	1.76	3.50	60.15	1.930	1.378	31.56	NP	3.34	11.66			16.83	3.00	6.428	20.59
	Sensitivity Ratio Si	1.361	1.045	1.174	1.483	1.948	1.097		1.854	2.10			2.465	1.097	1.022	1.296
Triaxial Compression Test		UU	UU	UU	UU	UU	UU		UU	UU			UU	UU	VS	UU
	Cohesion C (kg/cm²)	0.16	0.08	0.42	0.10	0.085	0.40		0.02	0.17			0.13	0.04		0.30
	Angle of Shearing Resistance φ°	4.8	4.1	9.1	3.0	4.1	23.5		3.6	7.0			10.1	4.0		12.4
Consolidation	Yield Stress Py (kg/cm²)	0.72	0.90	0.97	0.68	1.00	0.87	0.67	0.85	0.63	0.75	0.92	0.70	0.90	0.85	0.80
	Compression Index Cc	0.70	1.49	0.39	0.71	0.25	0.54	0.08	1.50	1.13	0.67	1.45	0.19	0.71	1.54	0.19

Remarks: Unconsolidated-Undrained Shear Test : UU
Consolidated-Undrained Shear Test : CU
Consolidated-Drained Shear Test : CD

Table 3–2–3 Results of Laboratory Test (2)

		Sample Number	BM. 6			BM. 7		BM. 8		BM. 9		BM. 10	
		Depth (m)	3.00 ~ 3.60	5.00 ~ 5.60	7.00 ~ 7.60	3.00 ~ 3.60	8.00 ~ 8.55	2.50 ~ 3.00	5.00 ~ 5.50	3.00 ~ 3.50	5.00 ~ 5.50	3.00 ~ 3.50	8.00 ~ 8.50
Grain Size Properties		Gravel (over 2000μ) (%)	0.0	0.0	0.0	0.0	0.0	0.0	44.2	0.0	9.2	0.50	0.0
		Sand (74–2000μ) (%)	22.6	12.3	5.9	26.7	16.1	1.4	33.8	8.0	30.8	13.3	0.7
		Silt (5μ–74μ) (%)	38.4	50.7	62.1	41.3	44.4	58.6	16.0	53.0	37.0	46.2	61.3
		Clay (less than 5μ) (%)	39.0	37.0	32.0	32.0	39.5	40.0	6.0	39.0	23.0	40.0	38.0
		Maxim Grain Size (mm)											
		Coefficient of Uniformity Uc	–	–	–	–	–	–		–	–	–	–
		Coefficient of Curvature U'c	–	–	–	–	–	–		–	–	–	–
Consistency Properties		Liquid Limit wL (%)	69.20	90.40	111.40	70.20	85.20	104.80	NP	63.80	56.85	47.60	80.10
		Plastic Limit wp (%)	46.55	33.345	30.475	31.235	32.33	30.57	NP	30.635	36.16	21.15	42.755
		Plasticity Index Ip	22.65	30.475	80.925	38.965	52.87	74.23	–	33.165	20.69	26.45	37.345
		Consistency Index Ic	–0.079	–0.112	0.131	–0.030	0.235	0.740	–	0.039	0.298	0.588	0.646
Classification of Soil													
		Specific Gravity Gs	2.664	2.663	2.652	2.656	2.658	2.661	2.664	2.661	2.659	2.661	2.655
		Water Content w (%)	72.89	94.12	100.78	71.38	72.78	49.88	35.76	62.50	50.68	32.06	55.96
		Unit Weight γt (g/cm³)	1.578	1.484	1.451	1.605	1.527	1.737	1.932	1.592	1.590	1.756	1.673
		Void Ratio e	1.918	2.486	2.673	1.835	2.007	1.296	0.872	1.715	1.520	1.001	1.474
		Saturation Sr (%)	100	100	99.99	100	96.50	100	100	97.0	88.66	85.22	100
Strength and Consolidation Properties	Unconfined Compression Test	Compressive Strength qu (kg/cm²)	VS	VS	VS	0.127	0.295	0.688	NP	0.325	0.315	0.151	0.890
		Modulus of Deformation E_{50} (kg/cm²)				4.536	24.58	34.4		21.66	35.0	18.87	24.72
		Sensitivity Ratio Si				1.512	0.797	1.764		2.77	2.25	0.909	1.435
	Triaxial Compression Test		VS	VS	VS	CU	CU	CU		CU	CU	CU	CU
		Cohesion C (kg/cm²)				0.26	0.30	0.25		0.50	1.18	1.00	0.05
		Angle of Shearing Resistance φ°				7.69	25.13	11.00		11.59	4.00	5.43	18.26
	Consolidation	Yield Stress Py (kg/cm²)	0.84	0.93	0.87	0.90	0.90	0.95	0.85	0.080	0.85	0.76	0.80
		Compression Index Cc	1.87	0.93	1.53	1.11	0.32	0.38	0.37	0.39	0.49	0.17	0.35

Remarks: Unconsolidated-Undrained Shear Test : UU
Consolidated-Undrained Shear Test : CU
Consolidated-Drained Shear Test : CD

Fig. 3-2-5 Diagram of Soil Properties

KILOMETERAGE BM. 1

ELEVATION + 1.367 m GROUND WATER LEVEL - 2.35 m

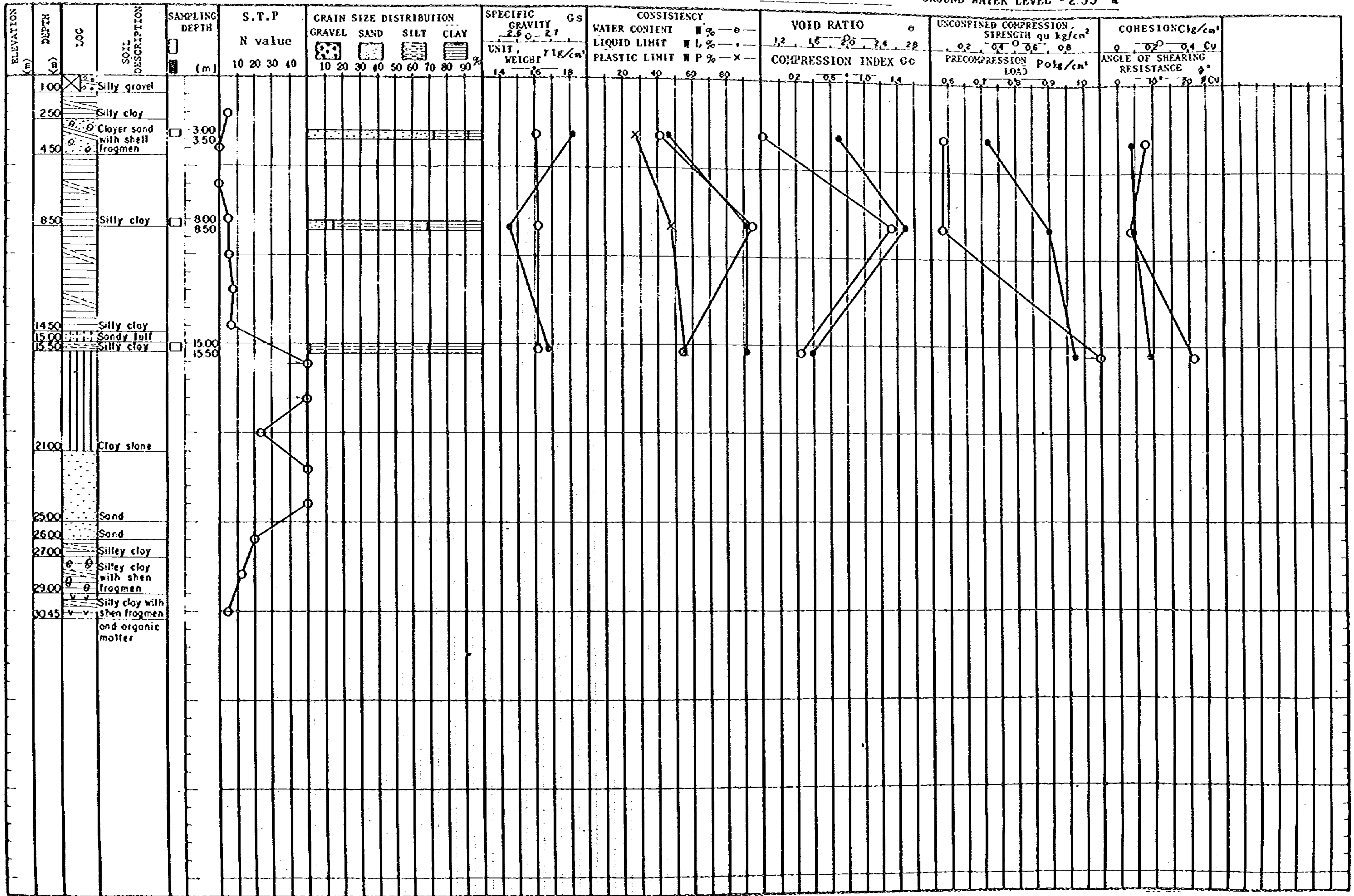


Fig. 3-2-6 Diagram of Soil Properties

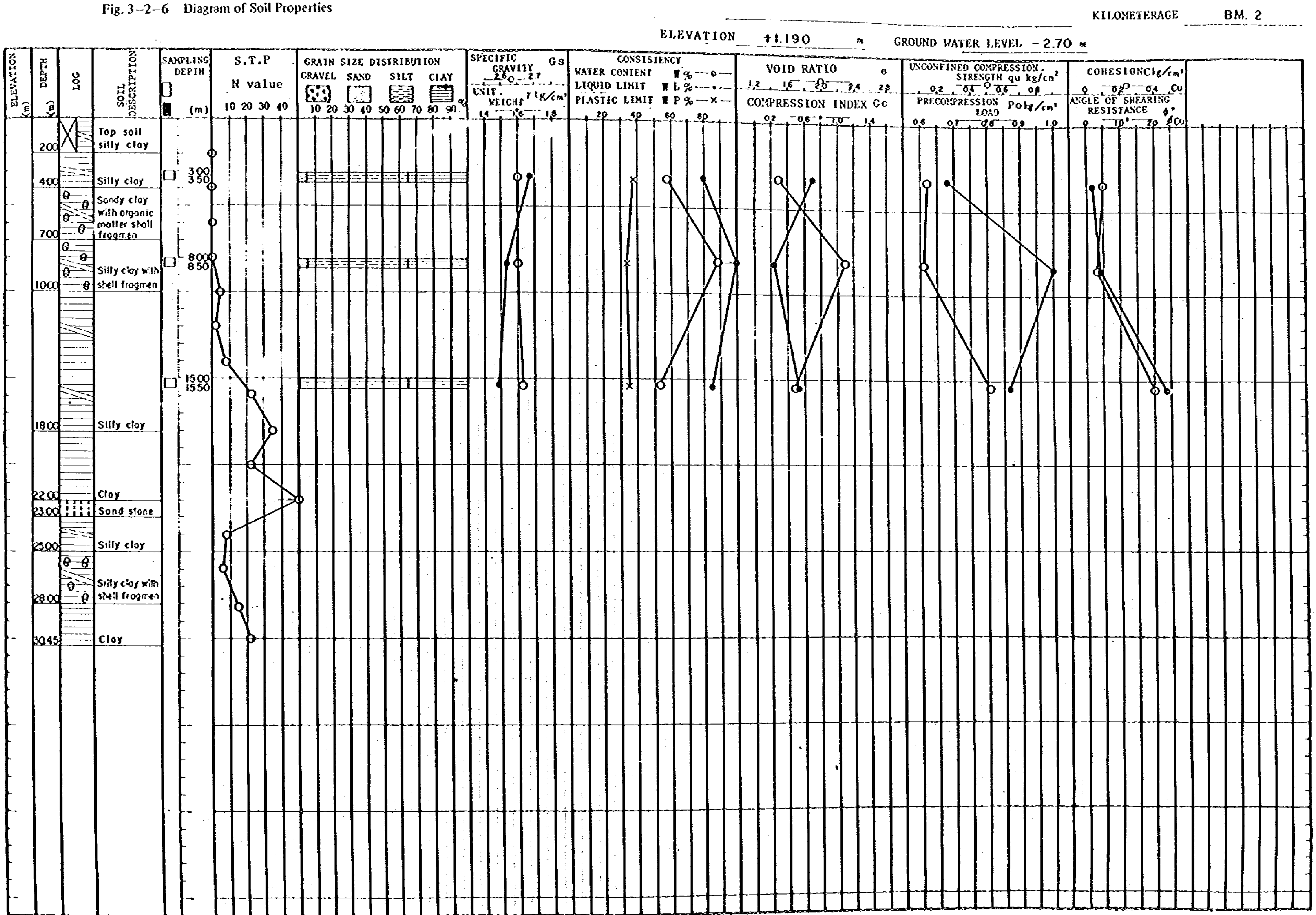


Fig. 3-2-7 Diagram of Soil Properties

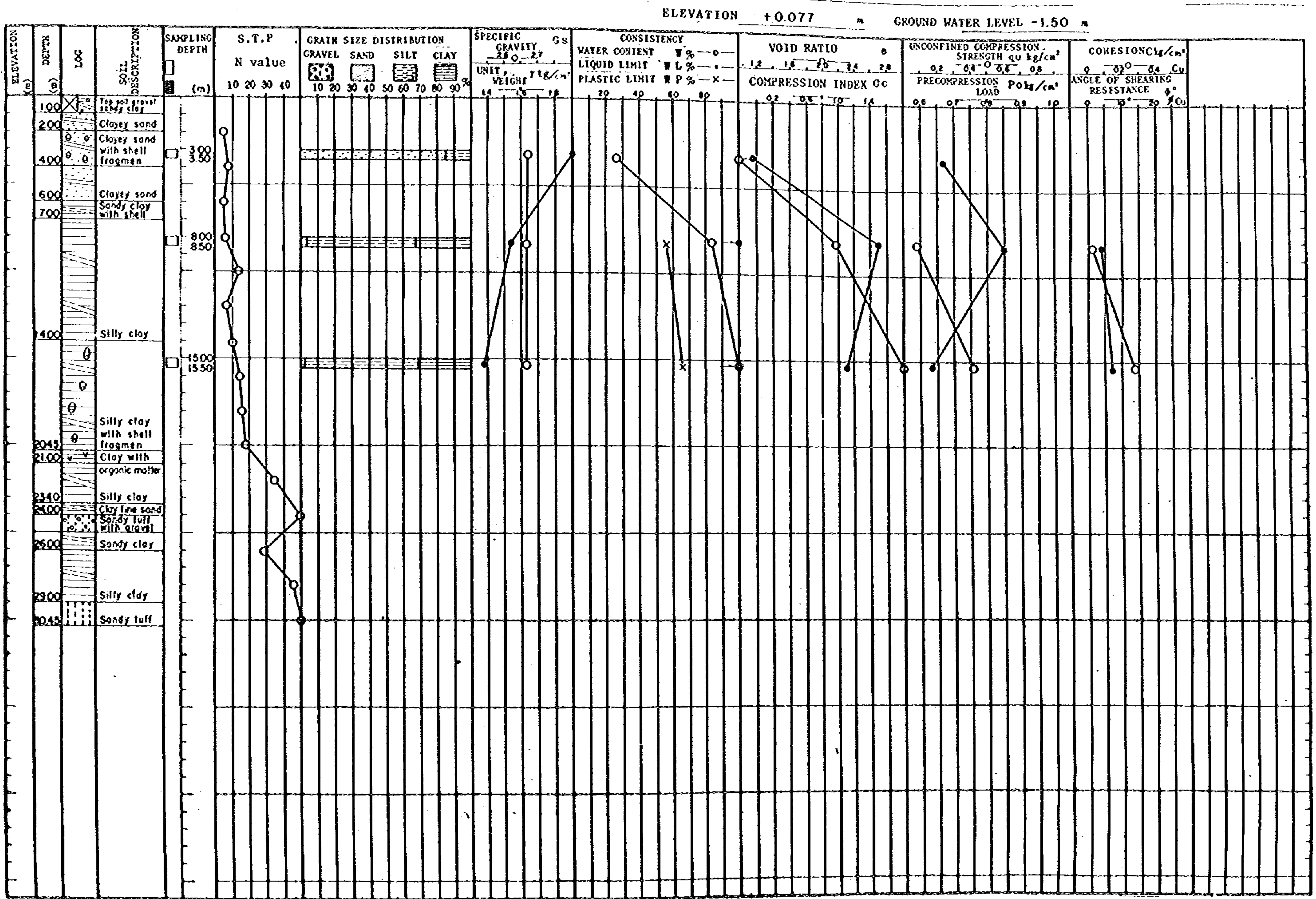


Fig. 3-2-8 Diagram of Soil Properties

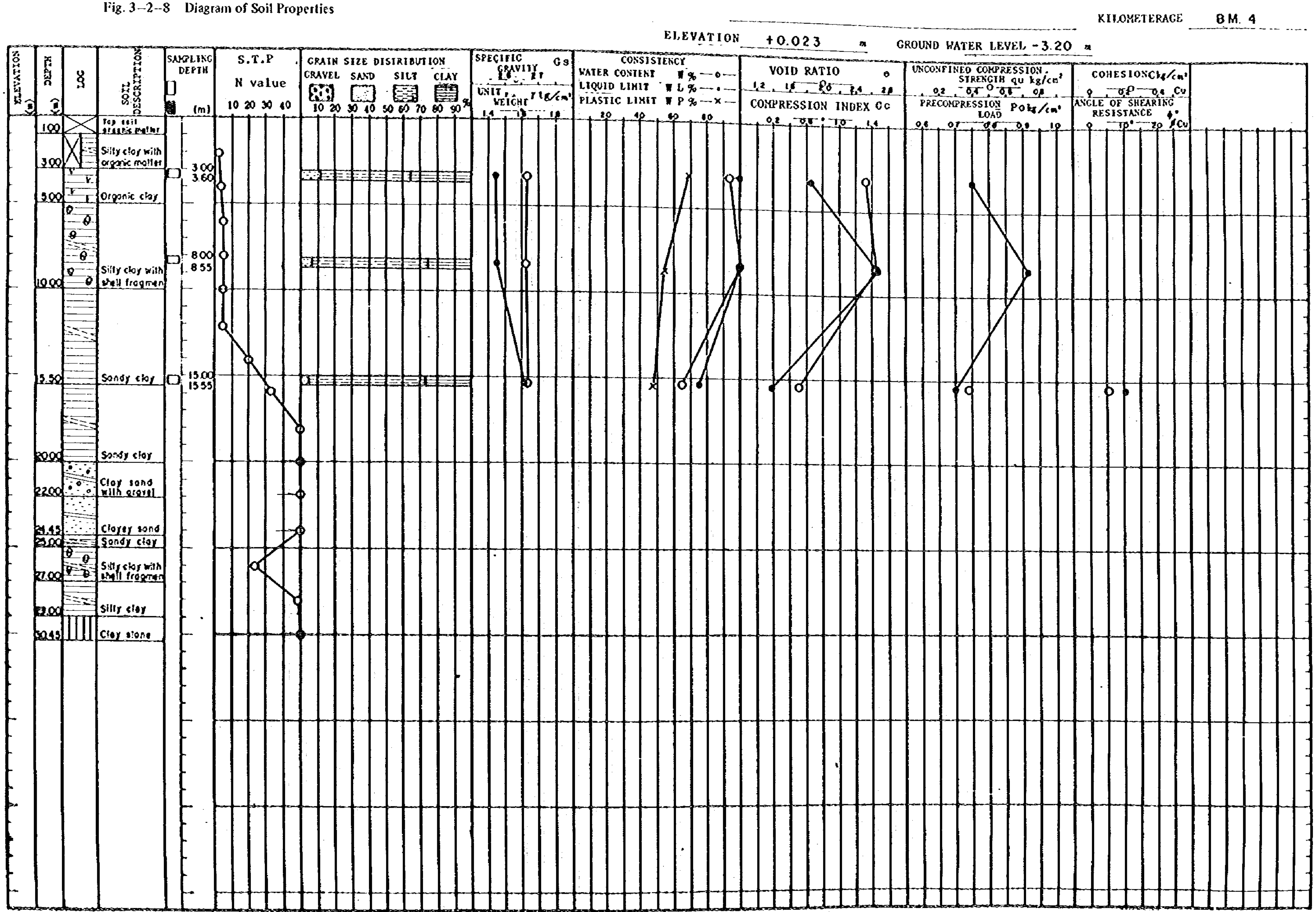


Fig. 3-2-9 Diagram of Soil Properties

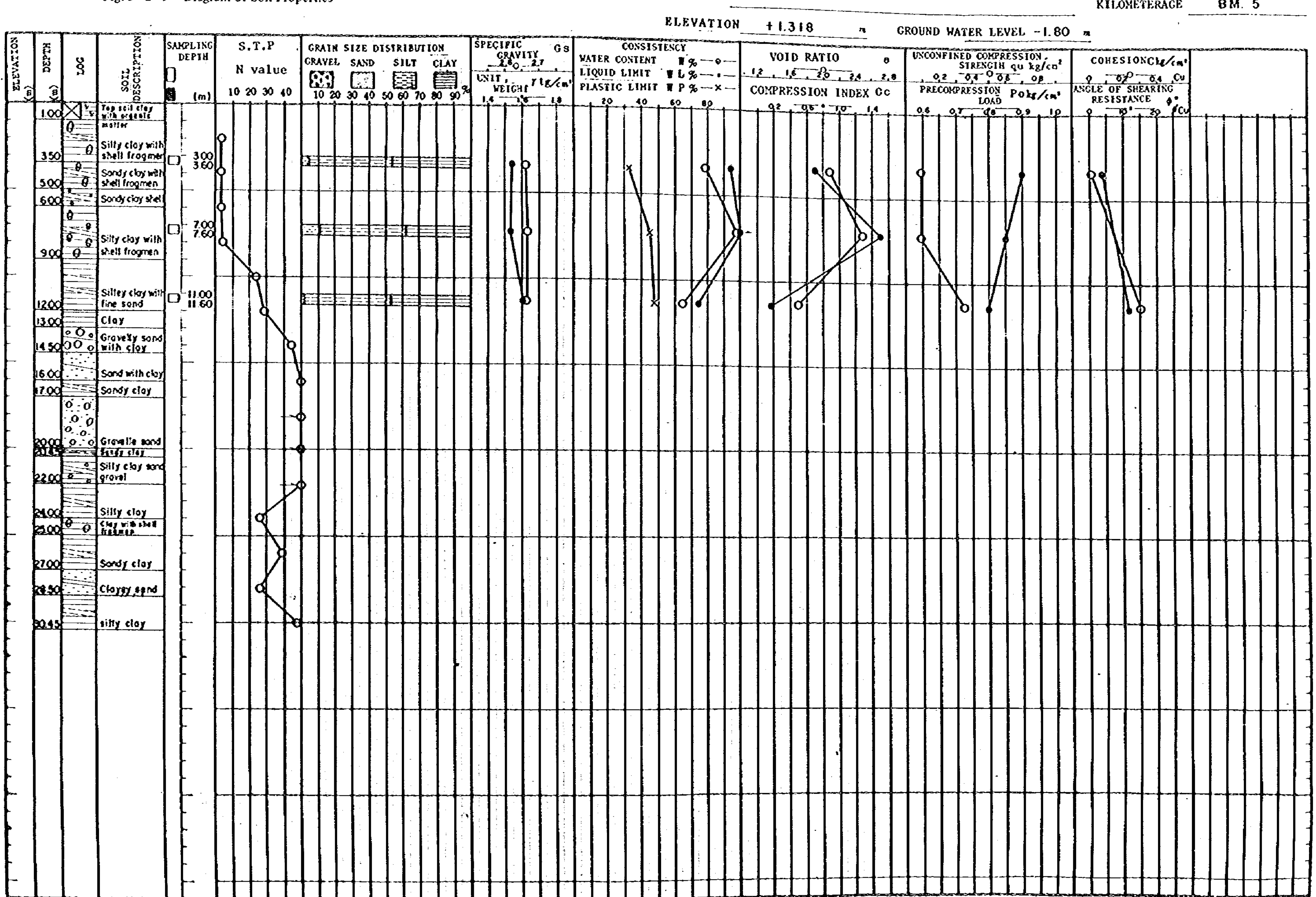


Fig. 3–2–10 Diagram of Soil Properties

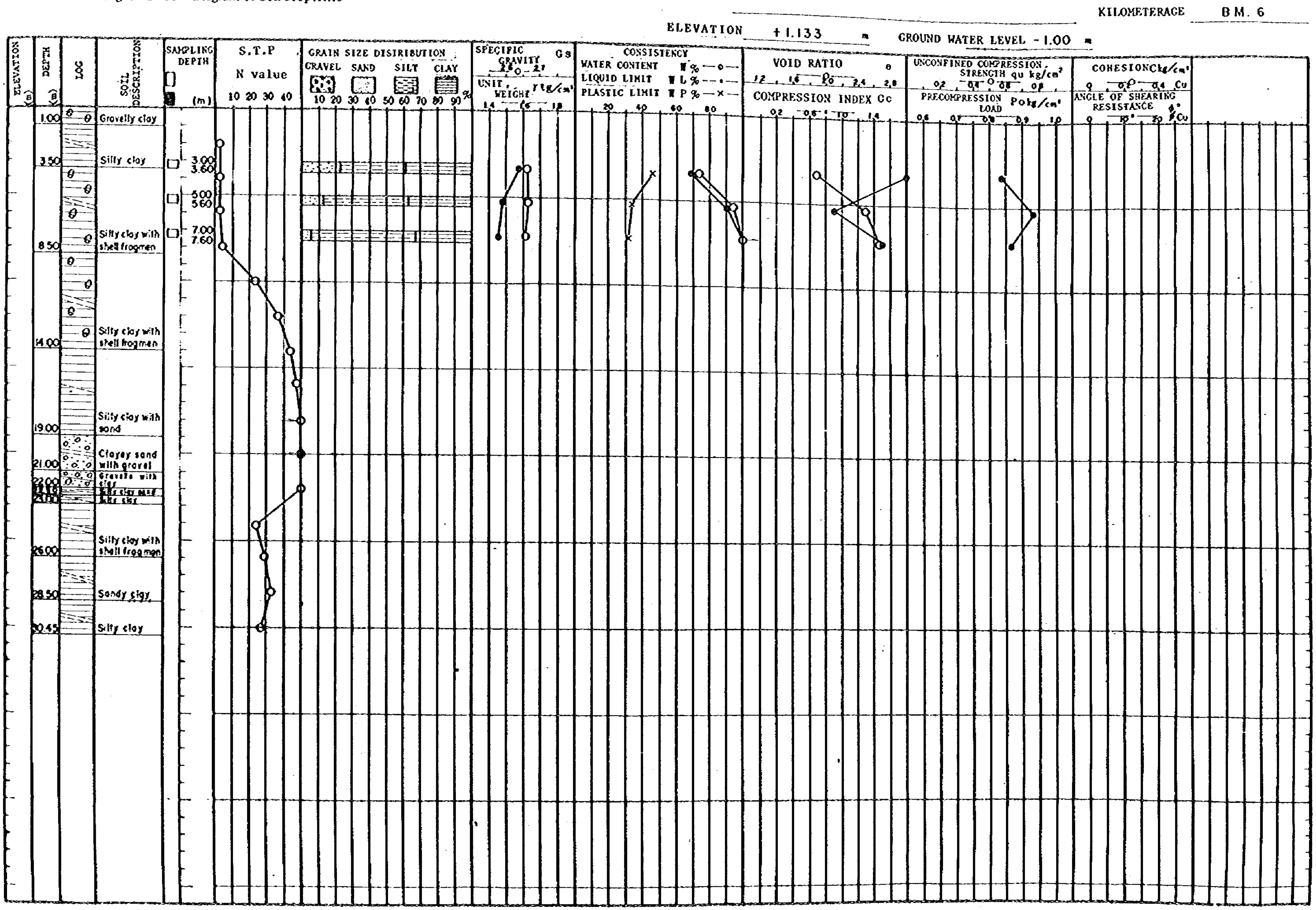


Fig. 3–2–11 Diagram of Soil Properties

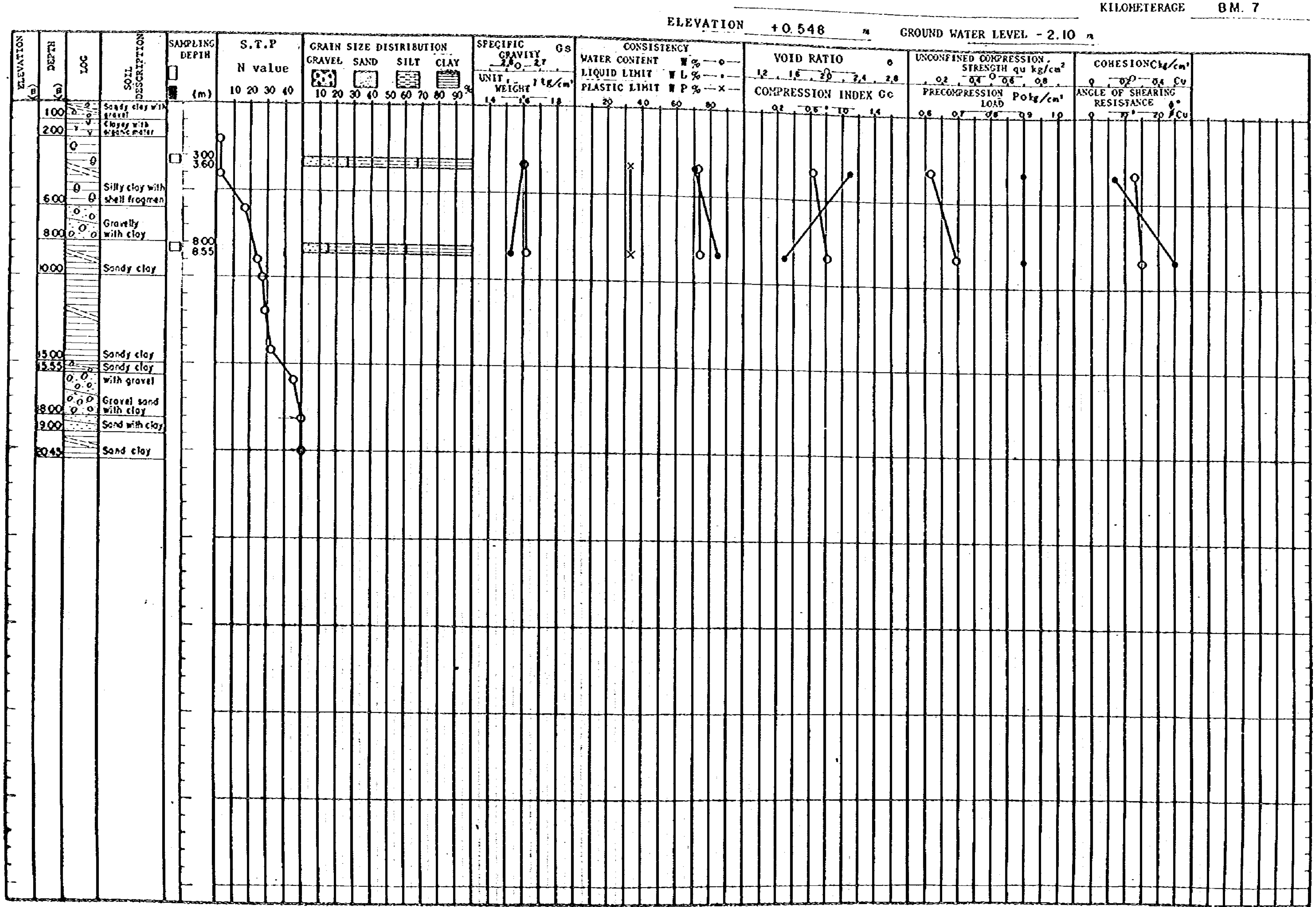


Fig. 3-2-12 Diagram of Soil Properties

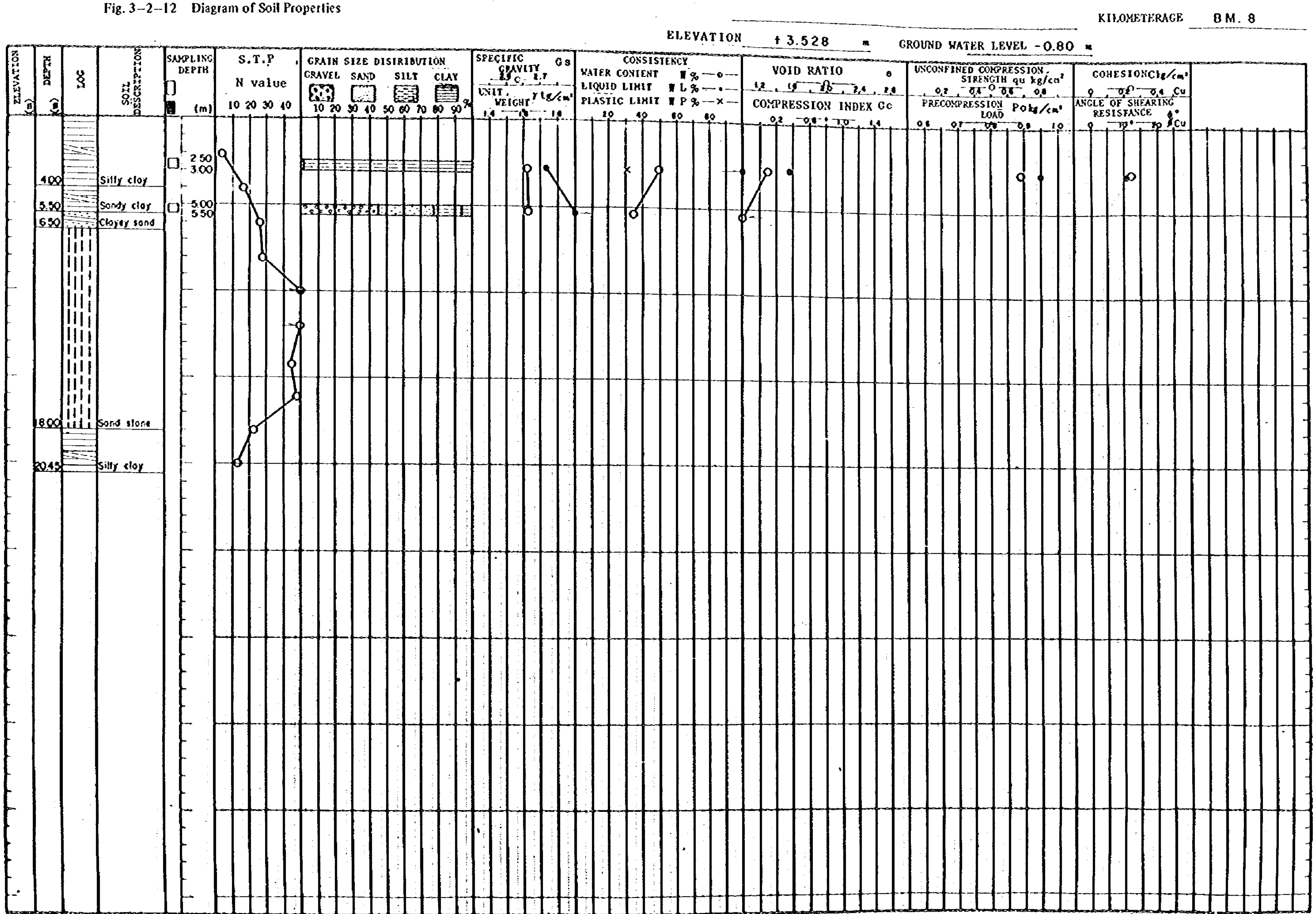


Fig. 3–2–13 Diagram of Soil Properties

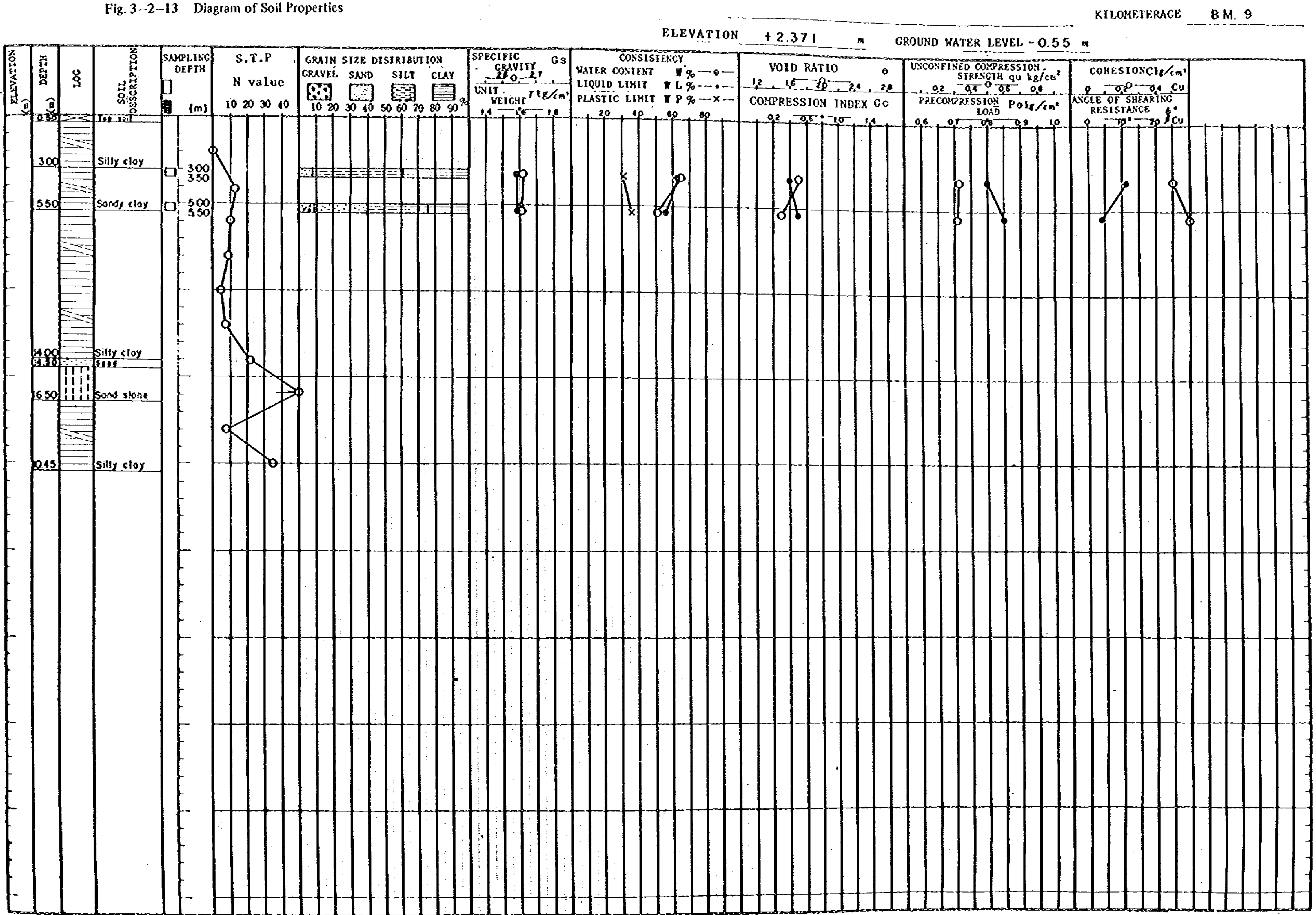
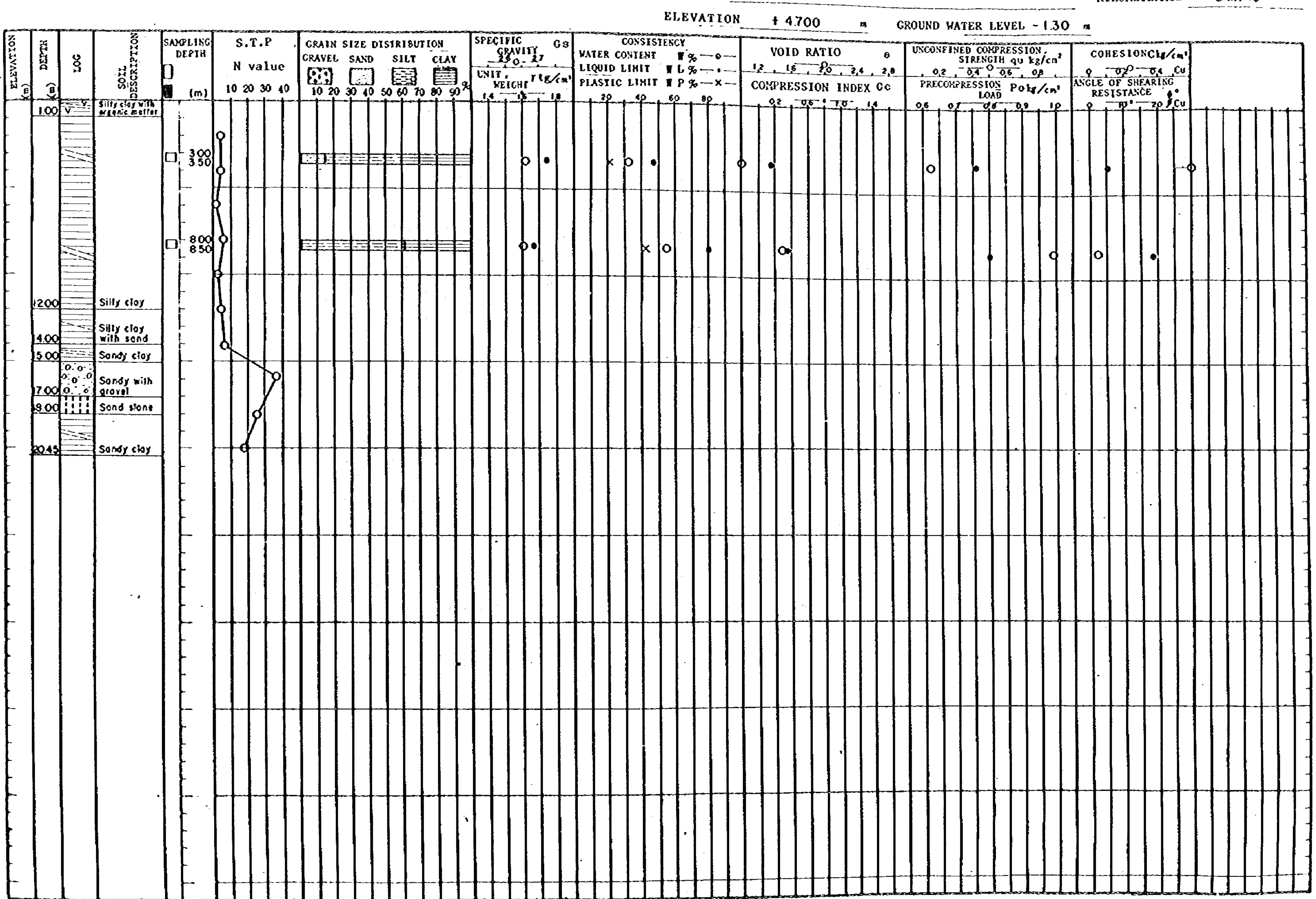


Fig. 3–2–14 Diagram of Soil Properties



3–2–5 Evaluation of the Soil Test Results

Samples used for the soil test can be divided into four categories, i.e. alluvial sand, alluvial clay, diluvial sand and diluvial clay. Soil properties of each formation are as explained below.

(1) Basic Characters of Soil

1) Grain Size Analysis

The results of grain size analysis of each soil formation are given in Table 3–2–4 below.

Table 3–2–4 Grain Size Analysis of Each Formation

Soil Formation		Gravel (%)		Sand (%)		Silt (%)		Clay (%)	
		Range	Mean Value	Range	Mean Value	Range	Mean Value	Range	Mean Value
Alluvial Sand	(As)	0	0	72–85	78.5	10–19	14.5	5–9	7.0
Alluvial Clay	(Ac)	1–9	3.0	1–31	10.0	37–67	53.0	23–46	34.0
Diluvial Sand	(Ds)	44.2	–	33.8	–	16.0	–	6.0	–
Diluvial Clay	(Dc)	0	0	1–16	4.2	44–69	60.6	27–47	35.2

As evident from the above table, the grain size distribution of both alluvial and diluvial clay formations varies widely (the degree of their grading varies) depending on the point where the analysis was made. There is relatively little difference, pertaining to the point of analysis, in the grain size distribution of the sand formation because fewer samples were available for the analysis.

2) Consistency Properties of Soil

The consistency properties of both alluvial and diluvial clay formations were examined. The results of this examination, based on the relations with water content and liquid limit, are shown in Fig. 3–2–15.

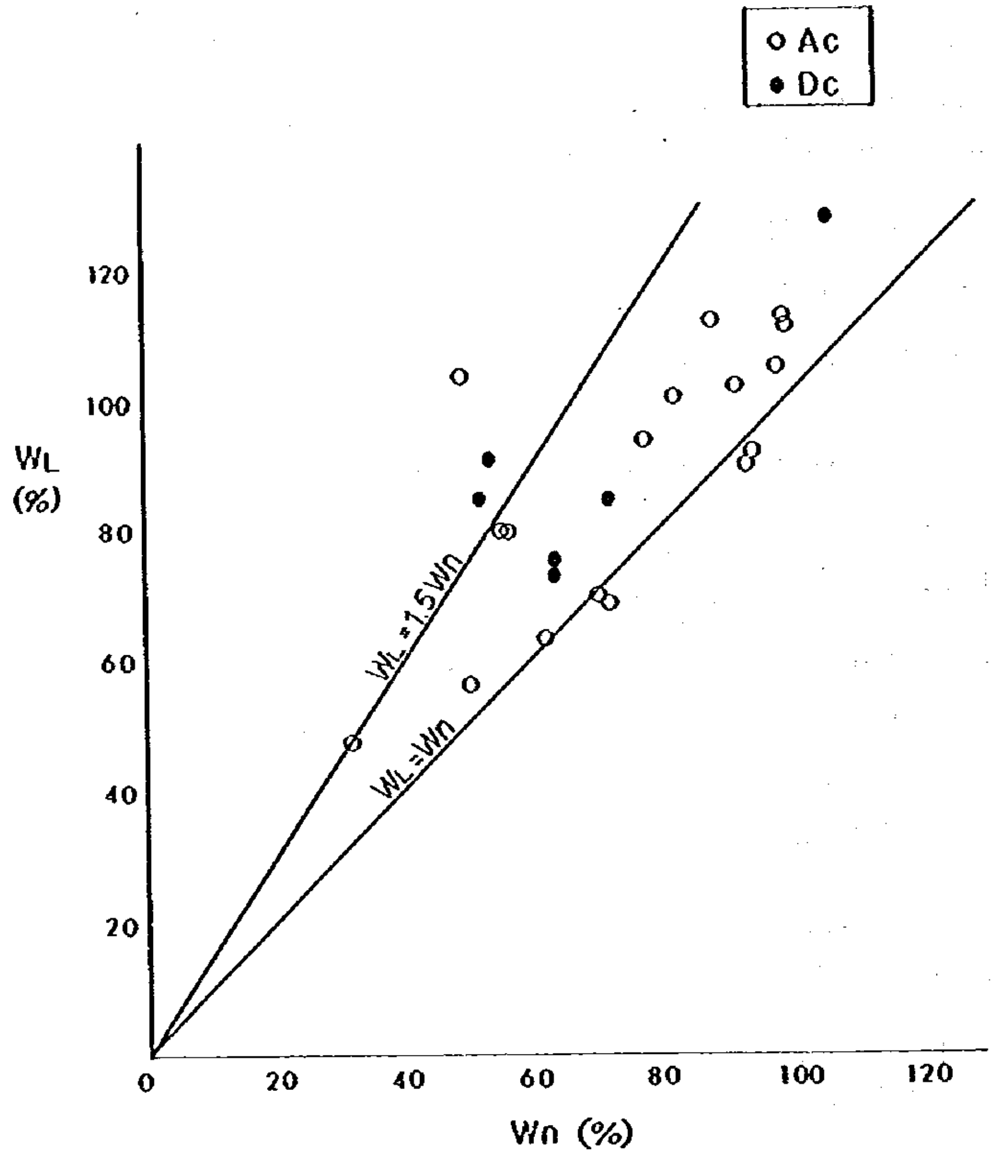


Fig. 3–2–15 Relation between Water Content and Liquid Limit

As obvious from Fig. 3–2–15, the soil properties at this point of examination are most often in the state of Wn <LL, with the exception of a few samples belonging to the Ac formation. The soil properties in the state of Wn > LL are found in the clay formation near the ground surface of BM 6, BM 7 and BM 9, all of which are embankment blocks.

The consistency index (Ic) ranges from –0.112 to 1.016 and averages 0.31, indicating that properties lack in stability. Under classification by a plasticity chart, this formation comes within the category of "MH" or "CH".

Note: MH includes inorganic silts, micaceous or diatomaceous fine sandy or silty soils, and elastic silts; CH includes inorganic clays of high plasticity and fat clays.

3) Specific Gravity, Unit Weight and Void Ratio

a) Specific Gravity

As shown in Fig. 3-2-16, the specific gravity of both Ac and Dc formations is 2.65 ($G_s \doteqdot 2.65$).

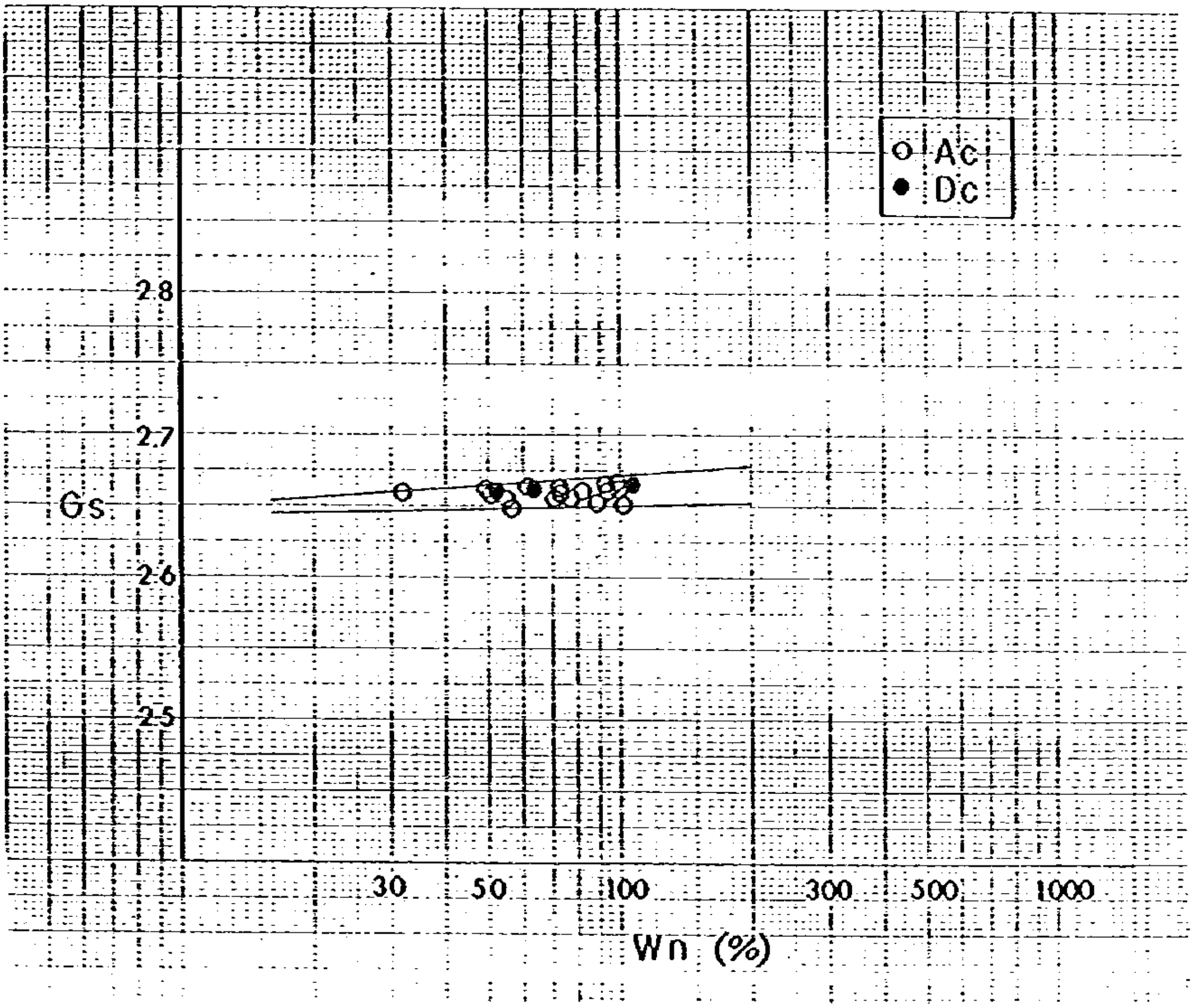


Fig. 3-2-16 Relation between Water Content and Specific Gravity

b) Unit Weight

The unit weight ranges widely in both formations, i.e. from 1.45 to 1.75 g/cm³ in Ac and from 1.38 to 1.67 g/cm³ in Dc. In both Ac and Dc formations, average unit weight ($\bar{x}$) is 1.565 g/cm³ and standard deviation value (σ) is 0.1. The unit weight of alluvial sand ranges between 1.82 and 1.93 g/cm³, nearly equal to the mean value of 1.85 g/cm³. The unit weight of diluvial sand, on the other hand, is 1.93 g/cm³.

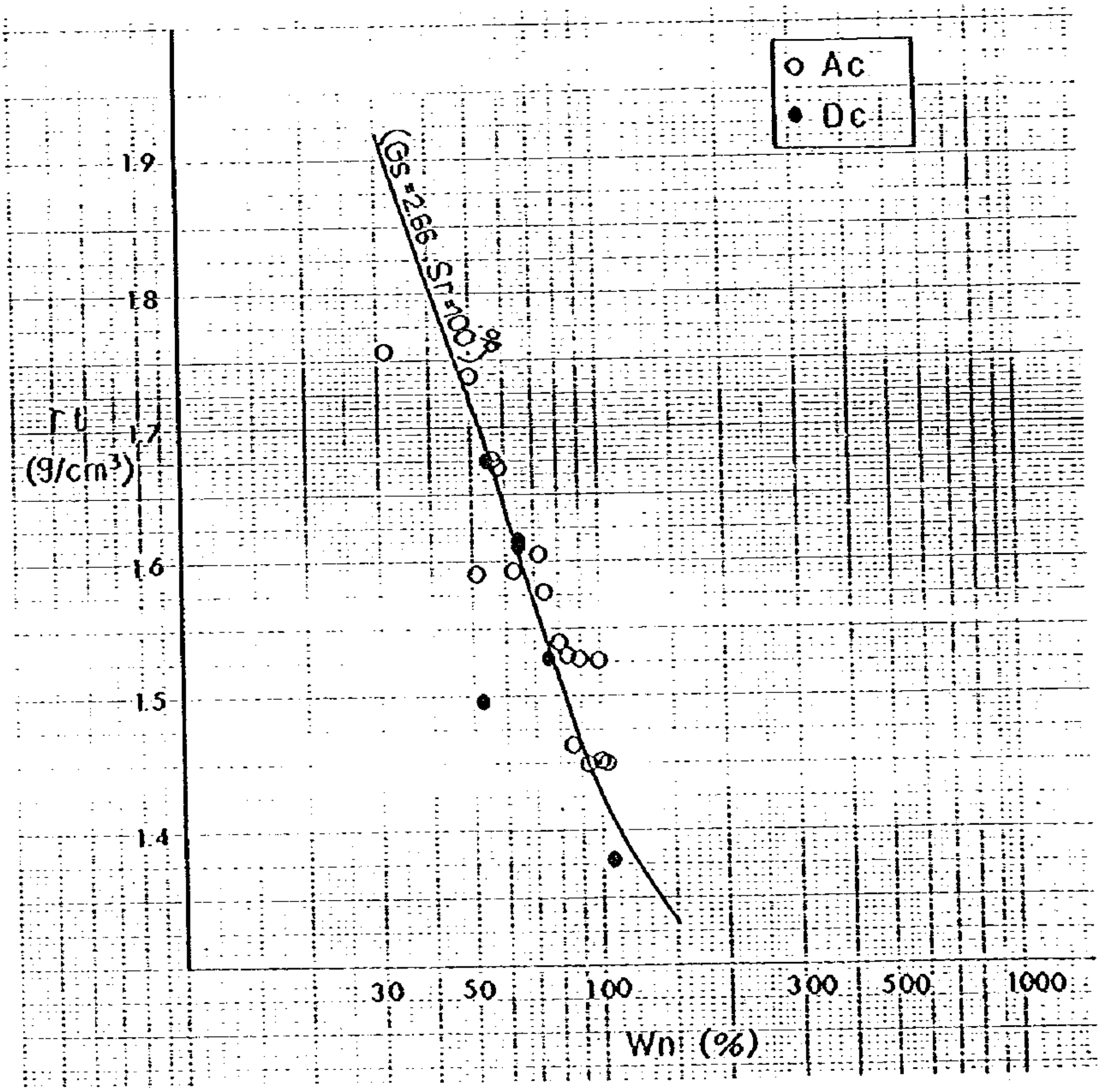


Fig. 3-2-17 Relation between Unit Weight and Water Content

c) Void Ratio

The void ratio has been illustrated in Fig. 3-2-18, reviewed with regard to water content. As evident from this figure, the void ratio (e) of clays ranges from 1.3 to 3.0 and generally tends to increase as the water content increases.

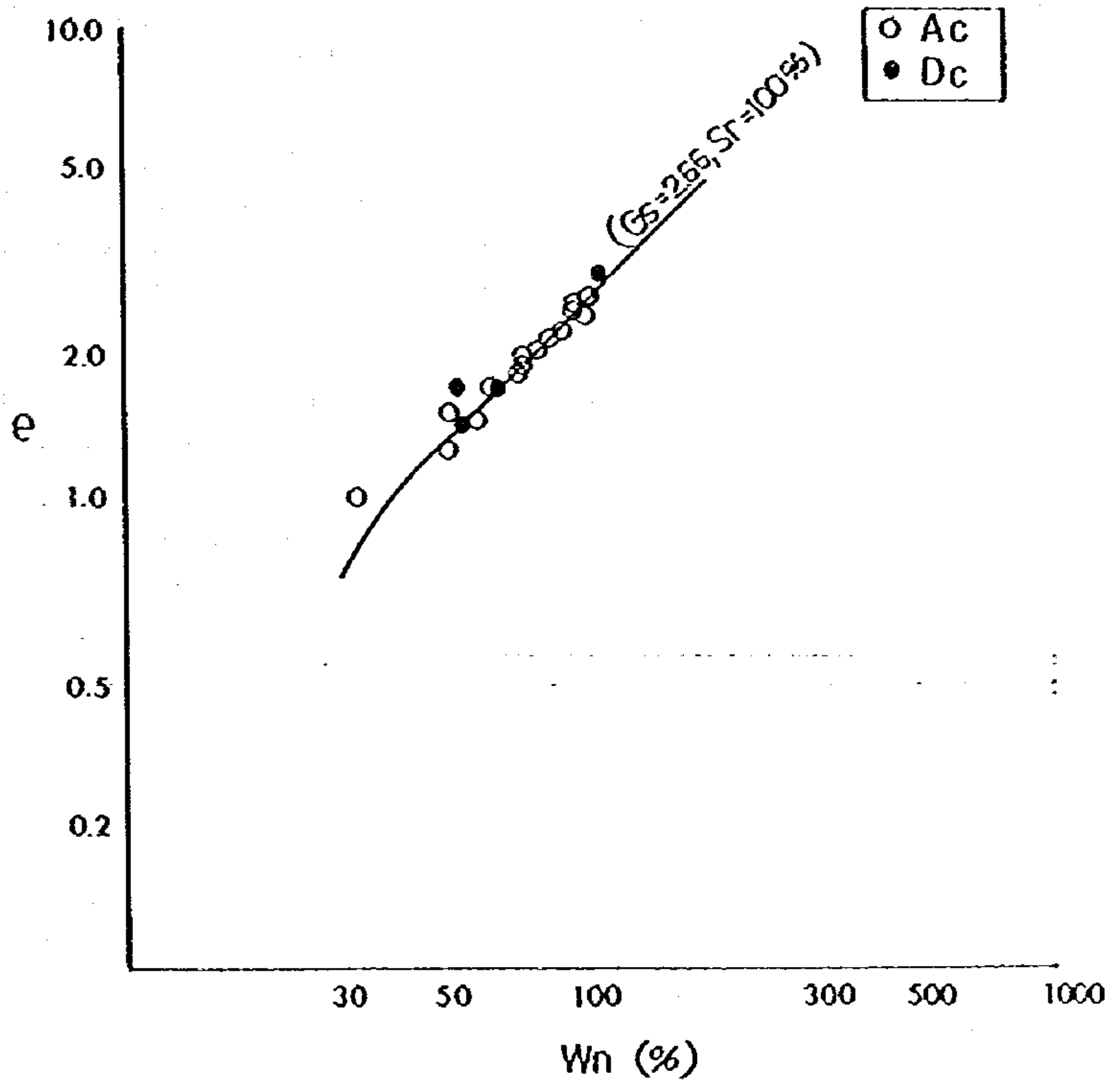


Fig. 3-2-18 Relation between Void Ratio and Water Content

4) Evaluation of the Basic Characters of Soil

- Both alluvial and diluvial clays are saturated soils.
- Soils are classified into elastic silts (MH) and fat clays (CH), which belong to inorganic silts or clays.
- The consistency index averages 0.31, but features most often an unstable state (Wn > LL) in the points from BM 6 to 9, which are embankment blocks.
- The findings of grain size analysis indicate that grain size distribution varies widely in both alluvial and diluvial clays, featuring a lower degree of grading.

(2) Mechanical Characters of Soil

1) Results from an Unconfined Compression Test of Soil

The unconfined compression strength (qu) of soil is illustrated with regard to depth in Fig. 3–2–19, with regard to deformation in Fig. 3–2–20, and with regard to E50 in Fig. 3–2–21. These figures indicate that enlargement of deformation and decline of strength are caused by the disturbance of soil samples.

In general, in the case of alluvial clays it is assumed that deformation obtained by unconfined compression strength shows less degree of disturbance in the case of less than 5% thereof and more disturbance in the case of more than 5% thereof. As shown in Fig. 3–2–20, about 60% of the qu value obtained from this test seems to represent the strength involving the effect of disturbance.

Based on the undisturbed qu value (deformation: below 5%), the soil strength of this region is expected to be at least qu = 0.3 kg/cm^2 in alluvial clays and 0.4 kg/cm^2 in diluvial clays.

For reference, the relation between qu and E50 is expressed by the following equation, based on Fig. 3–2–21.

$$E50 = 43.8\ qu + 2.6\ (kg/cm^2) \qquad qu > 0.05\ kg/cm^2$$

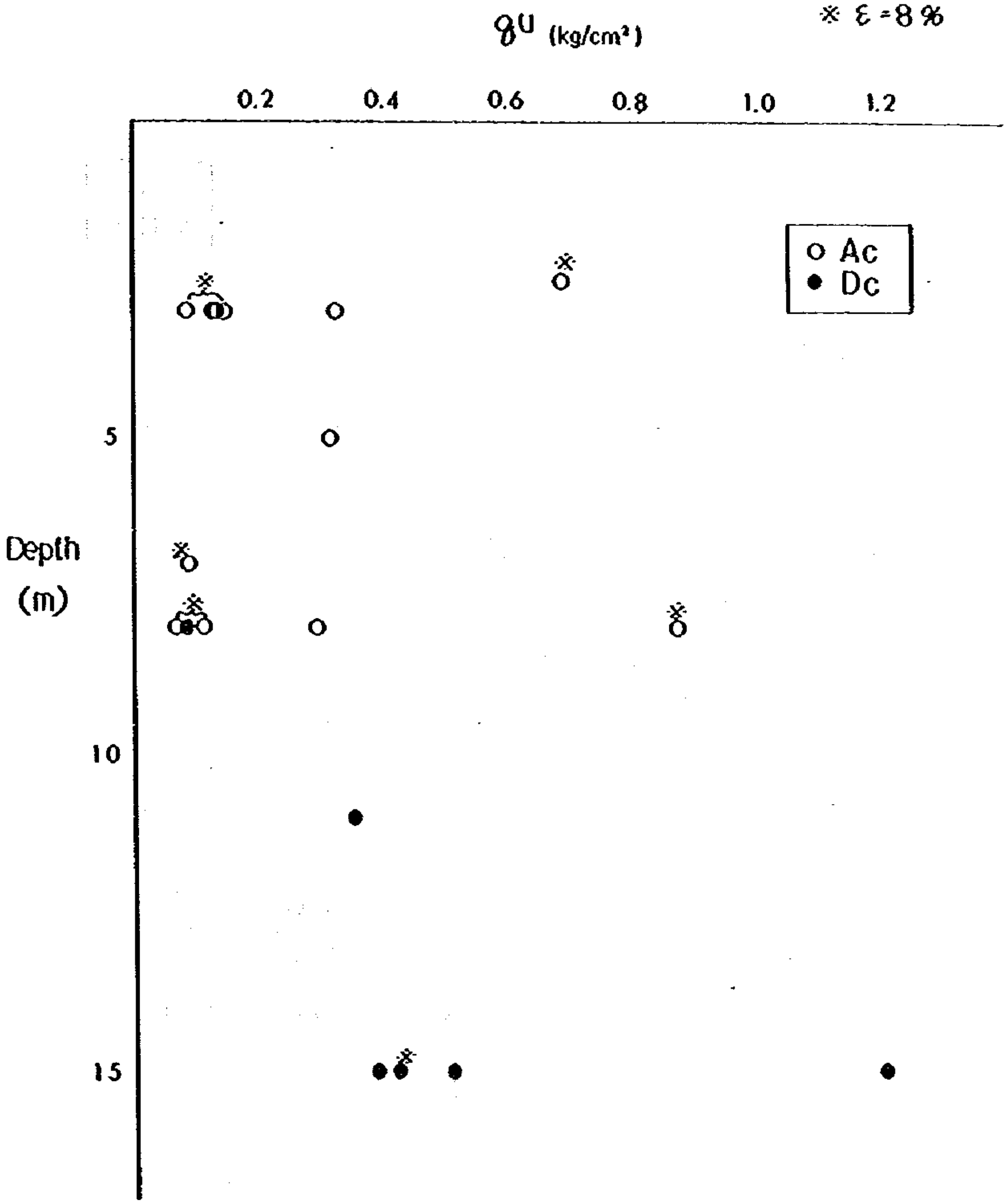


Fig. 3-2-19 Relation between Unconfined Compression Strength and Depth

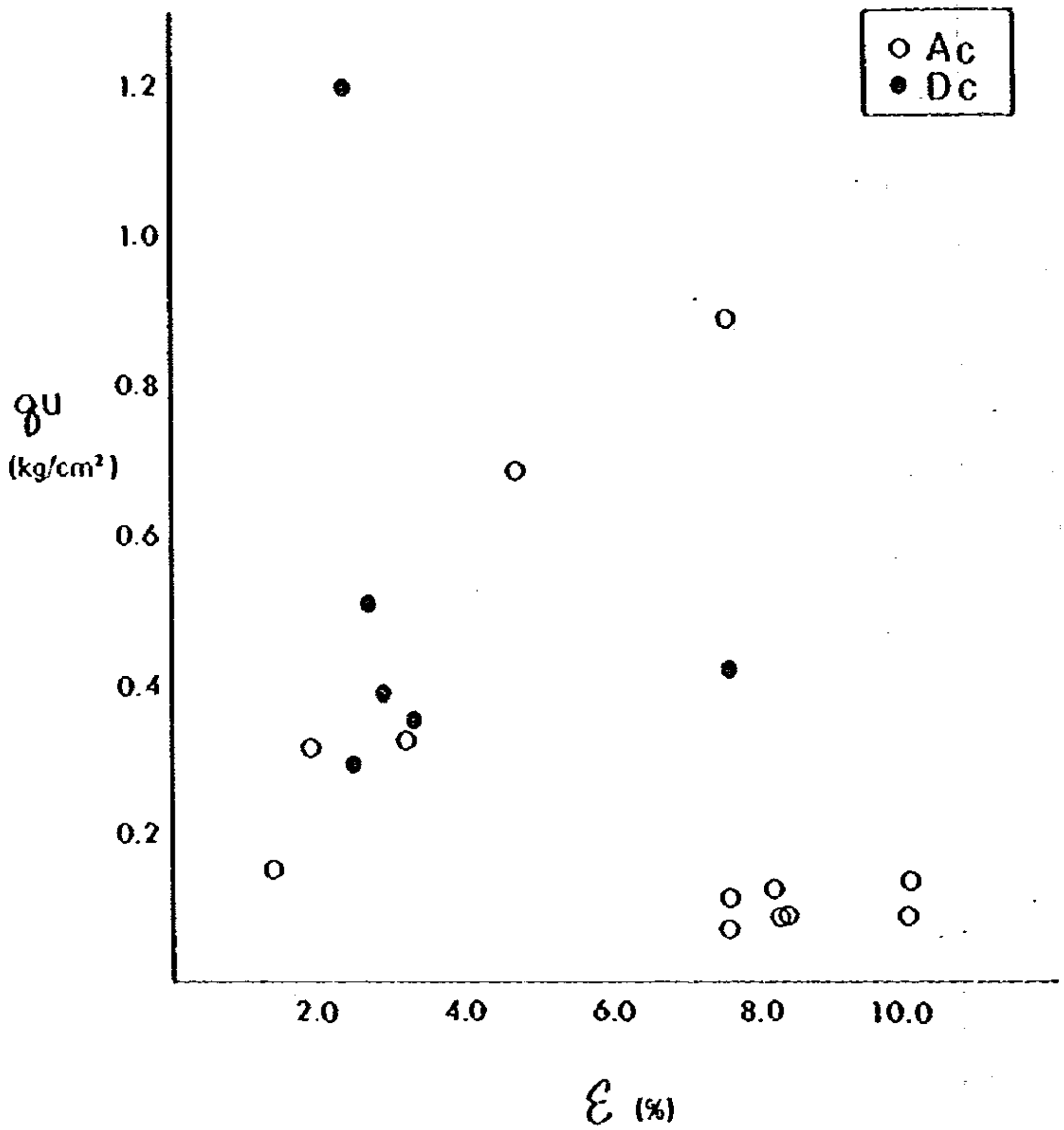


Fig. 3-2-20 Relation between Unconfined Compression Strength and Deformation

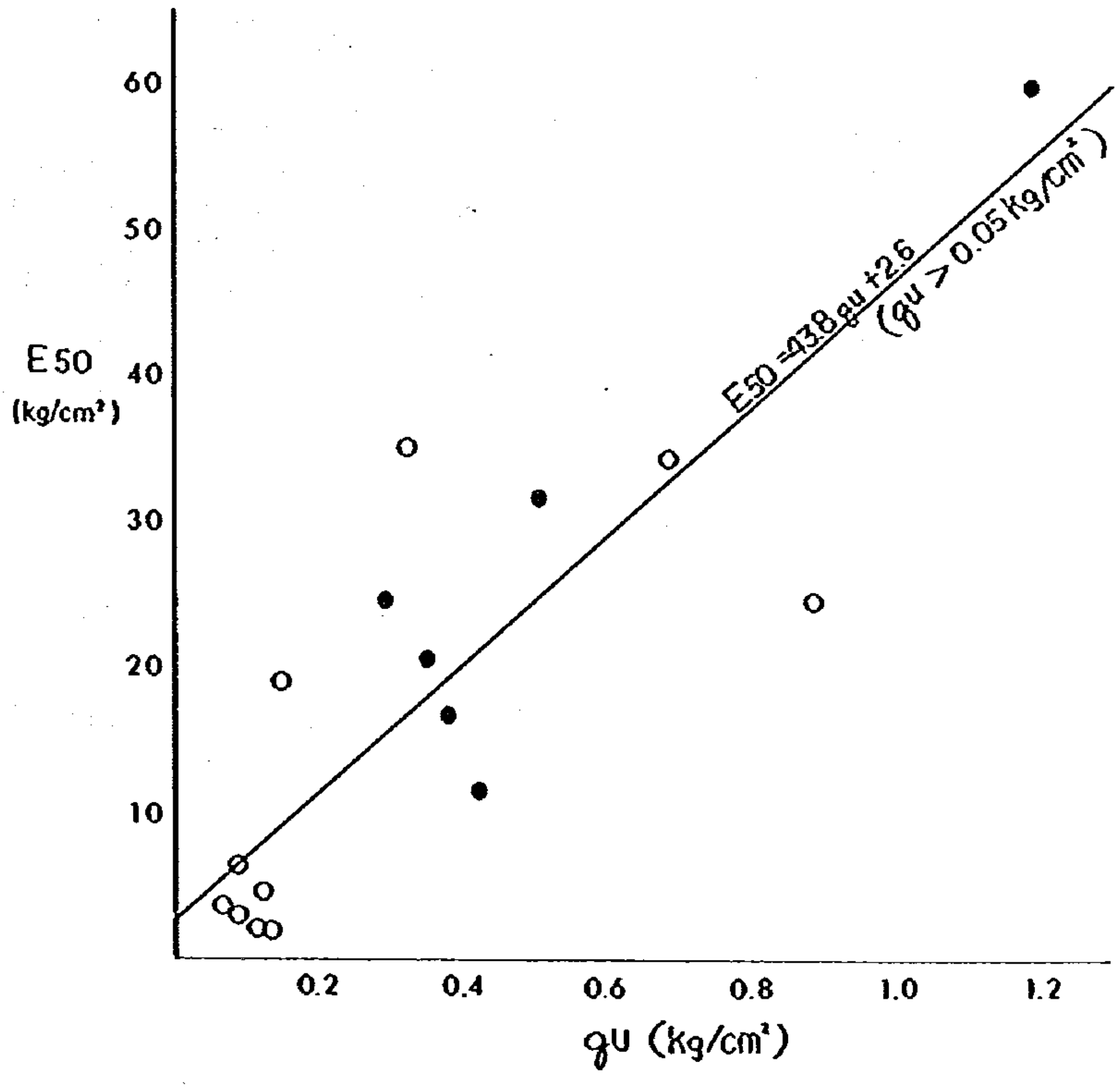


Fig. 3–2–21 Relation between Unconfined Compression Strength and Modulus of Deformation

2) Results of Triaxial Compression Test

The triaxial compression test was performed under unconsolidated–undrained conditions at the points from BM 1 to 6 and under consolidated – undrained conditions at the points from BM 7 to 10. The results of a UU test have revealed that the value of C ranges from 0.02 to 0.16 kg/cm^2 and averages 0.08 kg/cm^2 in alluvial clays. The shear resistance angle (ϕ), on the other hand, ranges between 3° and 4°, a value common to the soft clay layer.

In diluvial clays, the value of C ranges from 0.13 to 0.42 kg/cm^2 and averages 0.28 kg/cm^2. The shear resistance angle (ϕ) ranges between 7.0° and 23.5°, suggestive of a strength constant where the value around ϕ = 10° is predominant.

Based on the results of a CU test, the value of strength constants Ccu and ϕcu is available concerning the total stress. But it is impossible to make a computation through $\tau f = Ccu + \sigma \tan \phi cu$ by substituting these constants for the Coulomb's destruction criterion. Attention is needed because ϕcu, different by nature from the shear resistance angle under UU condition is a value showing the increase of strength produced after consolidation is finished.

The value of ϕcu, obtained from this test, ranges between 4° and 18.26° in alluvial clays and is 25.13° in diluvial clays.

3) Initial Consolidation Pressure

Fig. 3–2–22 shows the relations between depth and consolidation yield stress as examined in two different areas consisting of points BM 1 only and BM 7 to 10, all of which are embankment blocks.

The consolidation yield stress (Pc) in these areas ranges between 0.76 and 0.95 kg/cm^2, all indicating a state of over-consolidation.

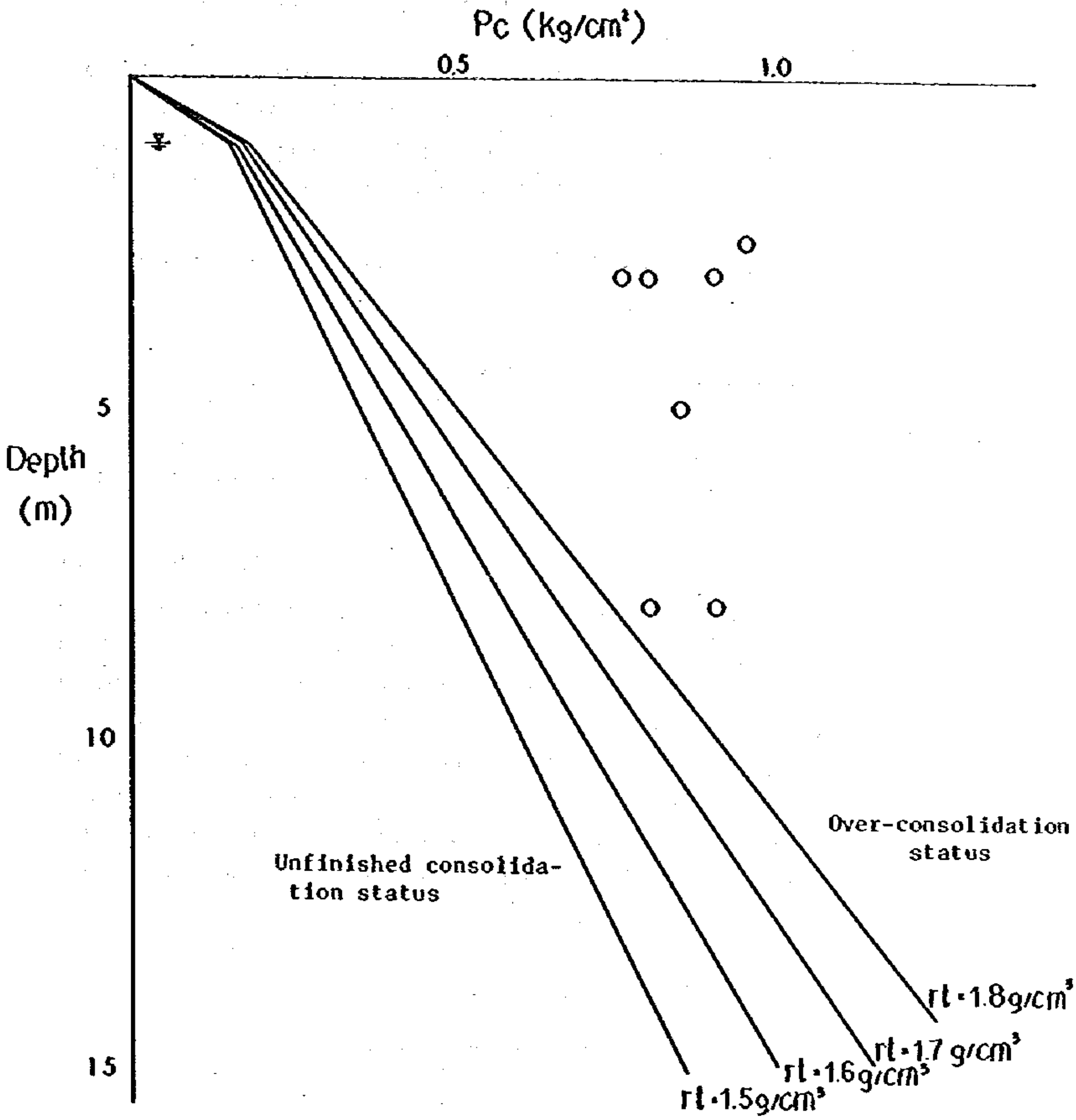


Fig. 3-2-22 Relation between Consolidation Yield Stress and Depth

4) Evaluation of the Mechanical Characters of Soil

- The unconfined compression strength (qu) is expected to be at least 0.3 kg/cm^2 in alluvial clays and 0.4 kg/cm^2 in diluvial clays.
- The intensity of adhesion (C) achieved in a triaxial test is approximately 0.1 kg/cm^2 in alluvial clays and 0.3 kg/cm^2 in diluvial clays. The shear resistance angle (ϕ) is expected to be from 3° to 4° in alluvial clays and 10° in diluvial clays. The value of ϕcu which indicates the increase of strength is about 10° in most cases.
- In the embankment sections, the consolidation yield stress is in the state of over-consolidation at every point of depth.

(3) Soil Conditions for Designing the Foundation Work

1) Soil Factors

In this paragraph we will study the soil factors required for designing the foundation work.

The factors pertaining to soil strength, whose value has been found through tests and analyses, are subject to the effect of soil disturbance. As mentioned previously, therefore, the constant of strength shown is lower than the actual constant. Differences in soil strength result from differences in sampling method. It is said that there is a nearly 30 % difference in strength between the sampling by tube (fixedpiston type and Denison type) and block sampling. Generally speaking, soil disturbance caused by the tube type of sampling is an inevitable problem depending on the mechanism of sampling equipment, sealing and conveying of samples, etc. In view of these factors, therefore, the actual strength of the soft clays located at the present site is considered to be 30% higher than the value gained from the tests.

a) Adhesion and Shear Resistance Angle of Soil

The adhesion of soil can be evaluated by the following method.

Value C found from a unconfined compression test:

The value of qu obtained from the sample of little soil turbulence is 0.3 kg/cm^2 in alluvial clays and 0.4 kg/cm^2 in diluvial clays. This leads to the following evaluations.

The unconfined strength of alluvial clays: $qu = 0.3 \times 1.3 = 0.39 \doteqdot 0.4$ kg/cm^2
The unconfined strength of diluvial clays: $qu = 0.4 \times 1.3 = 0.52 \doteqdot 0.5$ kg/cm^2

From the equation $C = qu/2$ ($\phi = 0°$), the value of C is approximately 0.2 kg/cm^2 in alluvial clays and 0.25 kg/cm^2 in diluvial clays.

Value C found from a triaxial compression test:

Since the samples used for the triaxial compression test are greatly affected by soil disturbance, it is difficult to apply the test data to the evaluation.

Value C based on value N:

The value of soil adhesion can be expressed by the equations $C = 0.63 \times N$ for alluvial clays and $C = N$ for diluvial clays. In this study, value N averages 3.7 in alluvial clays and 15.8 in diluvial clays.

Therefore, value C based on value N is estimated at:

$C \doteqdot 2.3\ t/m^2 \doteqdot 0.23\ kg/cm^2$ (alluvial clay)
$C \doteqdot 15.8\ t/m^2 \doteqdot 1.6\ kg/cm^2$ (diluvial clay)

Value C based on value qc (Dutch Cone):

In the blocks from S–1 to S–13 and the rest, the value of qc differs even if the soil formation belongs to the same alluvial period. In the blocks before S–13, the value of qc in the Ac layer averages $3.6\ kg/cm^2$, based on 450 pieces of data. In the blocks after S–13, the value of qc in the Ac layer averages $11.6\ kg/cm^2$, based on 363 pieces of data (in layers deeper than 5 m from ground surface). The value of qc in diluvial clays is expected to be $20\ kg/cm^2$ at least. Value C can be estimated on the basis of these values.

– Alluvial clay

In the case of $\phi = 3°$:
$C = qc/15 = 3.6/15 = 0.24 \doteqdot 0.20\ kg/cm^2$ (in the blocks before S–13)
$C = qc/15 = 11.6/15 = 0.77 \doteqdot 0.75\ kg/cm^2$ (in the blocks after S–13)
In the case of $\phi = 10°$:
$C = 3.6/20 = 0.18 \doteqdot 0.15\ kg/cm^2$ (before S–13)
$C = 11.6/20 = 0.58 \doteqdot 0.55\ kg/cm^2$ (after S–13)

In depths less than 5 m, however, the blocks after S–13 seem to have a value similar to blocks before S–13.

– Diluvial clay

$\phi = 3°$: $C = 20/15 = 1.33 \doteqdot 1.3$ kg/cm²

$\phi = 10°$: $C = 20/20 = 1.00 \doteqdot 1.0$ kg/cm²

Based on the above findings it is estimated that soil adhesion ranges from 0.20 to 0.55 kg/cm² and shear resistance angle from 3° to 10° in the soft clayey layer, when the turbulence of soil samples is taken into consideration. In view of the grain size distribution and specific local features of soil, however, the shear resistance angle was established at 0°, whereas the initial adhesion is as shown in the table below.

Boring No.	Area of Application	Initial Adhesion (t/m²)
1	$19^{km}000^{m}$ – $19^{km}600^{m}$	1.1
7	$11^{km}500^{m}$ – $13^{km}600^{m}$	0.8
8	$0^{km}000^{m}$ – $1^{km}750^{m}$ $8^{km}530^{m}$ – $11^{km}500^{m}$	3.4
9	$5^{km}000^{m}$ – $7^{km}500^{m}$	1.6
10	$1^{km}750^{m}$ – $5^{km}000^{m}$ $7^{km}500^{m}$ – $8^{km}530^{m}$	1.1

b) Unit Weight of Soil

As discussed in paragraph (1)–3), the unit weight of soil (γt) is estimated at 1.6 g/cm³ in both alluvial and diluvial clays, 1.85 g/cm³ in the alluvial sand, and 1.9 g/cm³ in the diluvial sand.

c) Horizontal Bearing Coefficient (Kh)

Horizontal bearing coefficient can be found by applying the modulus of deformation (E50) available from the unconfined compression test. The horizontal resistance of a pile is affected by the bearing strength ranging from the ground surface to the point of $1/\beta$. The value of E50 averages 2.2 kg/cm² in the distance (depth) from ground surface to $1/\beta$ extending over the blocks where pile foundations are to be established. However, the actual value of E50 applicable to computation is 2.0 kg/cm². The following equation (1) can be used to find the value of Kh.

$$Kh = 0.4\alpha\alpha' EoBh^{-\frac{3}{4}} \qquad \text{.......(1)}$$

The following formulas (Chang's equations) are available by applying $Bh = \sqrt{\ell y \cdot D}$, with the condition of pile head rigid connection being considered.

$$\ell y = \frac{3}{4} \cdot \frac{\pi}{\beta}, \quad \beta = \sqrt[4]{\frac{Kh \cdot D}{4EI}}, \quad Kh = \frac{4EI}{D} \cdot \beta^4$$

Depending on the conditions of the pile head, equation (1) leads to (2) or (3) as shown below.

Pile head fixed : $Kh = 0.27\,(\alpha\,Eo)^{1.103} \cdot D^{-0.310} \cdot (EI)^{-0.103}$ ....(2)

Pile head pinned : $Kh = 0.32\,(\alpha\,Eo)^{1.103} \cdot D^{-0.310} \cdot (EI)^{-0.103}$ ....(3)

Eo : Modulus of soil deformation (kg/cm^2)
α : Coefficient of correction for Eo computation and load condition (ordinary 4 and temporary 8 in the case of a unconfined compression test)
α' : Coefficient for the pile shape (1.2 in case of a circle)
D : Pile diameter (cm)
E : Coefficient of pile elasticity (kg/cm^2)
I : Secondary moment of the pile cross section (cm^4)

With the coefficient of pile elasticity being set at 3.5×10^5 kg/cm^2, the value of Kh is as given in Table 3–2–5 below.

Table 3–2–5 Value of Kh

Pile Diameter (cm)	Load Condition	Pile Head Fixed (kg/cm^3)	Pile Head Pinned (kg/cm^3)
35	Ordinary	0.076	0.091
t = 6.5 cm	Temporary	0.164	0.195
40	Ordinary	0.070	0.082
t = 7.5 cm	Temporary	0.149	0.177

Based on the values of Kh above, the displacement of the pile head when 1–ton horizontal force is applied will be as shown in Table 3–2–6 below.

Table 3–2–6 Displacement of Pile Head

Pile Diameter (cm)	Load Condition	Pile Head Fixed (cm)	pile Head Pinned (cm)
35	Ordinary	0.89	1.54
	Temporary	0.50	0.87
40	Ordinary	0.75	1.31
	Temporary	0.42	0.74

As explained above, horizontal bearing coefficient is smaller in the alluvial clayey soil extending under the project site of an elevated bridge, suggesting that horizontal resistance is not enough there. In order to increase the horizontal resistance, therefore, it is necessary to devise some countermeasures such as installing an inclined shaft in the foundation of the elevated bridge.

2) Selection of a Supporting Layer

The conditions of a supporting layer are subject to the magnitude of load working on the foundation. Generally speaking, however, it would be safe to say that the sand or gravel layer whose N value exceeds 30 and the clayey layer whose N value exceeds 20 are a good supporting layer. Nonetheless, even a good supporting layer in respect of N value still has a problem. If it is too thin, or if there is another relatively weak or consolidated layer beneath it, then it becomes important to consider the effect on the lower part.

The elevated bridge to be constructed in the project site extends from approximately 12 km to 19 km. In view of the N value, the Dcs layer would constitute a supporting layer in this site.

The Dcs layer features soil fluctuations, and there is beneath it a heap of clays whose N value is slightly lower than that of the Dcs layer. The supporting layer, therefore, plays the part of an intermediate layer here. There is some dispersion of N value observed in the Dcs layer, which comprises an intricate accumulation of clayey and sandy soils.

In consideration of these factors, the pile supporting layer here shall be established at the point with the N value of over 30. The penetoration length of a supporting layer depends on how to install a pile. In case of the driving method, it is necessary to penetrate a pile into the supporting layer more than the length of both 50cm and pile diameter.

The value of N intended to design the pile end ground is subject to the penetration length of a pile as well as to the form of a supporting layer. In this study, even a sufficient penetration length of a pile is not enough since the supporting layer takes the form of an intermediate layer here. For safety, it is advisable to find the value of N based on the following equation.

$$\bar{N} = \frac{N1 + N2}{2}$$

$\bar{N}$: N value for designing

N1 : N value at the pile end

N2 : Mean N value for the distance 4D above the pile end

Note: The upper limit of N value is 50.

Boring No.	Supporting Depth (GL–m)	N1	N2	$\bar{N}$	Remarks
BM – 1	16.5	50	28	39	Pile Diameter 400 m/m
BM – 2	18.5	30	28	29	
BM – 3	24.0	34	30	32	
BM – 4	17.5	40	35	38	
BM – 5	13.5	40	35	38	
BM – 6	13.5	40	38	39	

(4) Soil Conditions for Embankment Blocks

The blocks to undergo banking work extend from approximately 0 km to 12 km and around 19 km.

1) Thickness of a Poor Layer

The thickness of a poor layer shall be determined by the following method.

According to Terzanghi and Peck, the expression of soil strength can be divided into two categories, i.e. consistency and relative density. The soil which is regarded as weak falls under the "soft to very soft" category of consistency.

From the standpoint of strength, a poor layer features the unconfined compression strength of below 0.5 kg/cm², cone index of below 0.4 kg/cm², and N value of below 4.

In conformity with these criteria, the lower limit applying to a poor layer is shown by a dashed line in the geological profile.

2) Consolidation Properties of a Poor Layer

a) The e–log p Curve for Designing

Fig. 3–2–23 shows the data on the e–log p curve, which applies to the poor layers of an embankment block. As evident from this diagram, the e–log p curve features a similar form of curving, with the exception of 8 m in BM1 and 3 m in BM7.

This suggests that a poor layer shows a similar phenomenon of consolidation, if the consolidation load of embankment is the same. But this does not apply to the upper part (GL– 6.0 m) of point BM7 and lower layer (GL–4.5 to 8.5 m) of point BM1.

At the designing stage, therefore, these data will be used for the lower layer of BM1 and upper part of BM7. As for other points, the mean e–log p curve pertaining to BM 8 to 10 will apply (see Fig. 3–2–24).

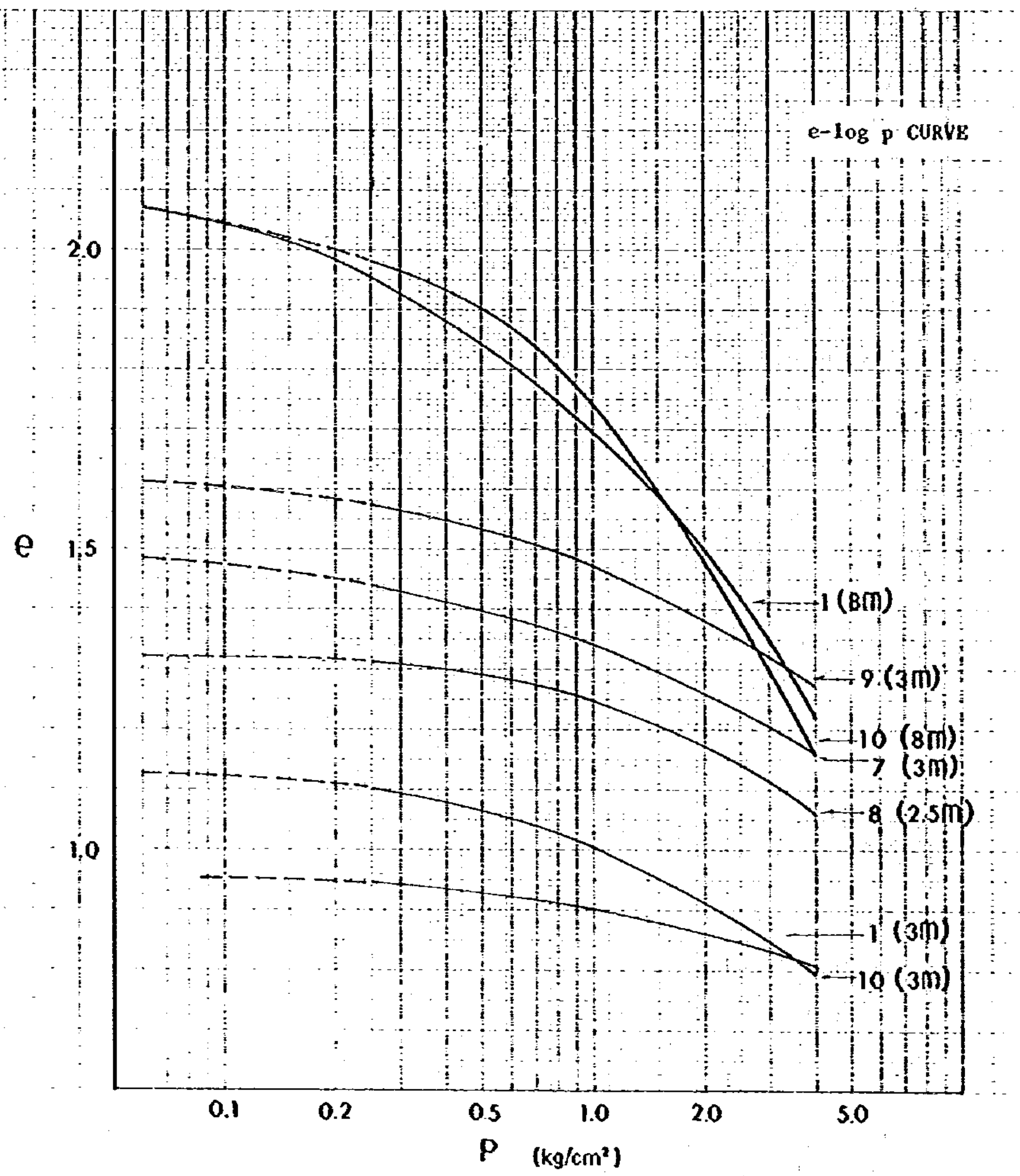


Fig. 3–2–23 e–log p Curve of Poor Layer

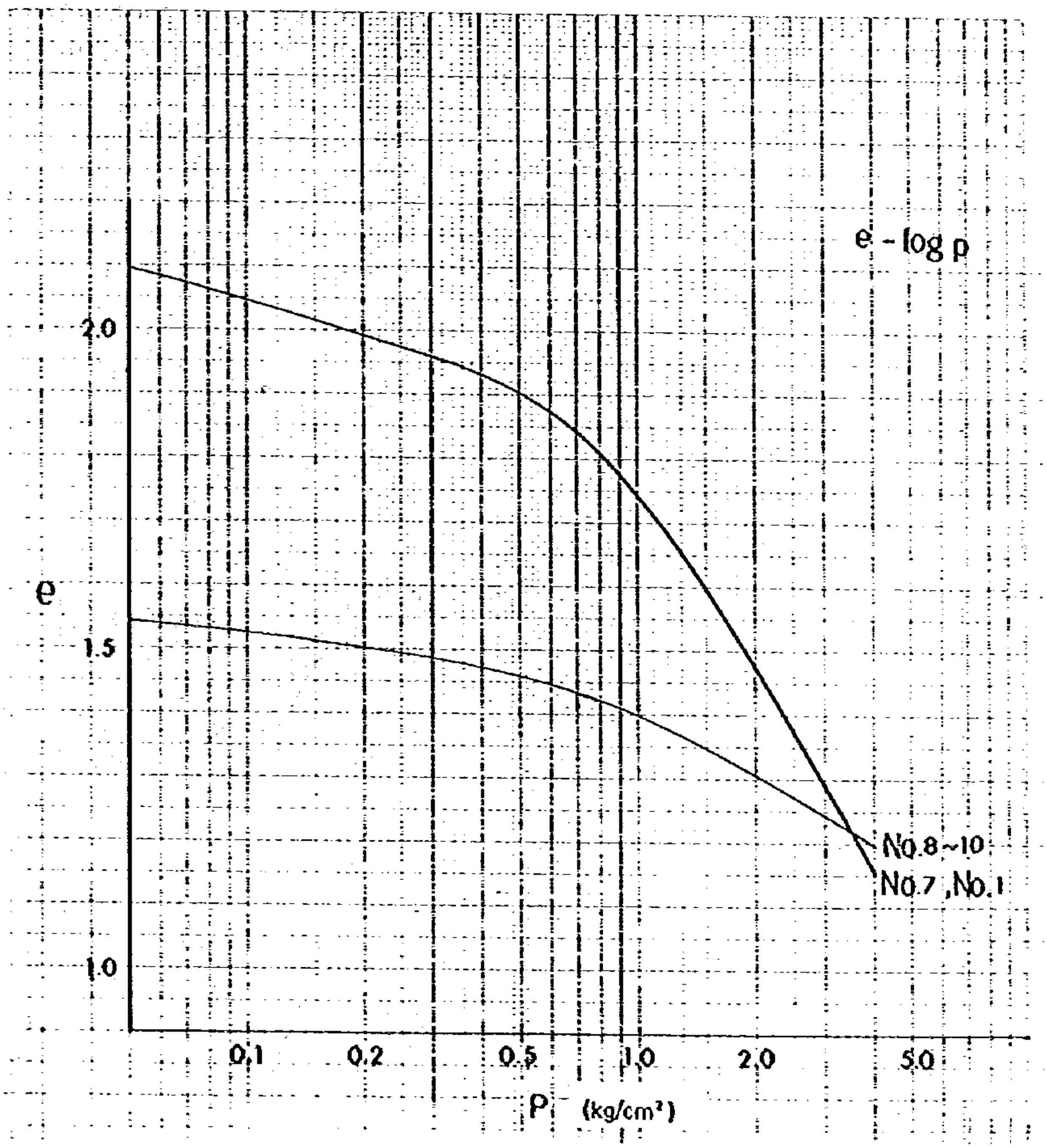


Fig. 3-2-24 e-log p Curve for Design

b) CV Curve for Designing

Fig. 3-2-25 shows the CV curve which has been prepared in the same way as the e-log p curve. As in the case of the e-log p curve, it is obvious from this diagram that the CV curve presents a similar coefficient of consolidation, except for the lower layer of BM1 and upper layer of BM7. Determination of the CV values for designing shall therefore be treated in the same way as that of the e-log p curve (see Fig. 3-2-26).

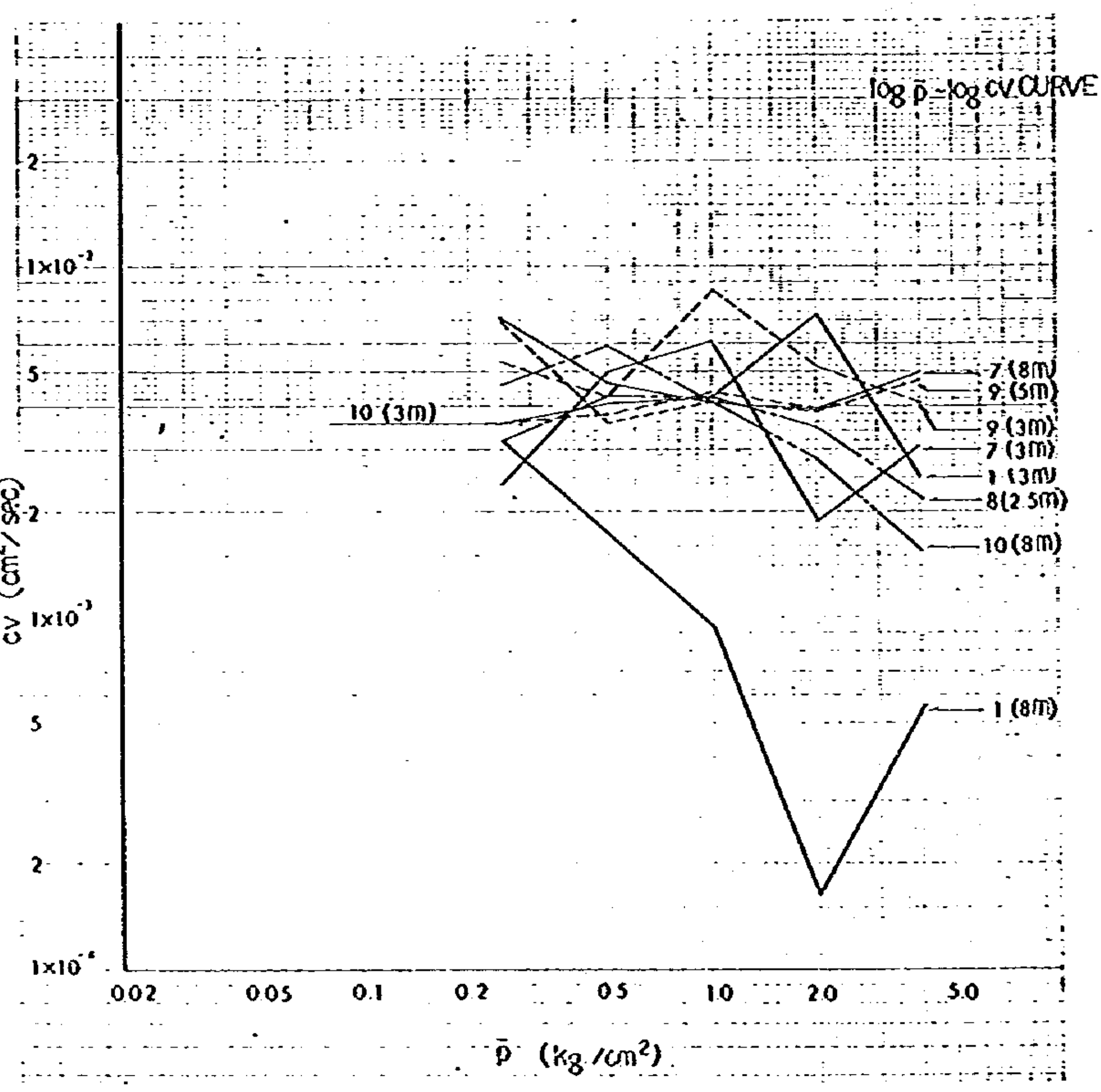


Fig. 3-2-25 Cv-p̄ Curve

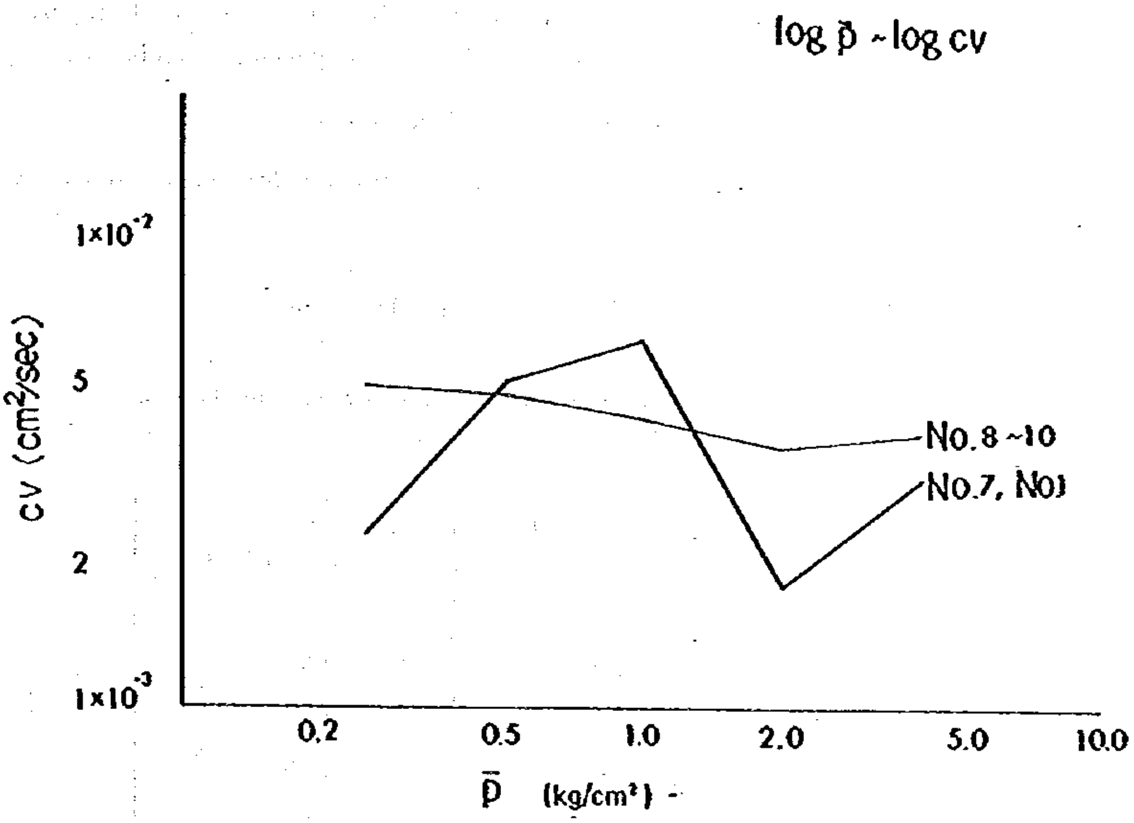


Fig. 3-2-26 Cv–p̄ Curve for Design

c) Strength Increase Rate of Poor Layer due to Consolidation Settlement

As consolidation progresses, soil increases in both strength and density. Generally speaking, the normal consolidation of clays is subject to the physical properties of soil and can be evaluated by the Skempton's equation, which is given below.

$$Cu/p = 0.11 + 0.0037\ Ip \quad (W < WL < 100\%)$$

The data on a triaxial compression test (Cu) are also available to determine the value of normal consolidation.

$$Cu/p = \frac{2\ \sin\ \phi cu}{1-\sin\ \phi cu} \quad \text{(in the case of total stress analysis)}$$

The values of Cu/p pertaining to each point, obtained from the two equations above, are as shown in the table below.

Boring No.	From Skempton's Equation	From Triaxial Compression Test
BM 1	0.18	–
BM 7	–	0.31
BM 8	–	0.47
BM 9	0.23	0.50
BM 10	0.21	0.21

From the results of the triaxial compression test in BM 8 and 9, it seems that the value of Cu/p is too high when grain size distribution is considered. The table below shows the strength increase rate, pertaining to each boring site, to be used for designing the embankment blocks.

Boring No.	Area of Application	Strength Increase Rate
1	$19^{km}000^{m}$ – $19^{km}600^{m}$	0.18
7	$11^{km}500^{m}$ – $13^{km}600^{m}$	0.31
8	$0^{km}000^{m}$ – $1^{km}750^{m}$ $8^{km}530^{m}$ – $11^{km}500^{m}$	0.30
9	$5^{km}000^{m}$ – $7^{km}500^{m}$	0.23
10	$1^{km}750^{m}$ – $5^{km}000^{m}$ $7^{km}500^{m}$ – $8^{km}530^{m}$	0.21

3–2–6 Embankment Materials

(1) Borrow Site

For determination of applicability of the embankment materials laterized clayey soil were collected at the three different sites as shown in Fig. 3–2–27.

a) Pondok Petir (TP–1, TP–2)

At the present time, excavation and loading are being conducted by means of heavy equipment at site, which is located about 25 km southwest of Jakarta. The borrow pit area covers a space of about 20,000 m^2. As the layer of borrow pit has a thickness of 6 m, the quantity of borrow material reaches approximately 120,000 m^3. This site has the disadvantage that the borrow material has to be transported through the urban district of Jakarta to the railway construction site.

b) Lengkong Gudang (TP–3, TP–4)

At the present time, excavation and loading are being conducted by means of heavy equipment at this site, which is located about 11 km south of Tangerang. The borrow pit area covers a space of about 20,000 m^2. As the layer of borrow pit has a thickness of 5 m, the quantity of borrow material reaches approximately 100,000 m^3. This site appears to be one of the major borrow pit as a number of trucks transporting borrow material observed.

c) Curug (TP–5)

At the present time, excavation and loading are being conducted by means of heavy equipment at this site, which is located about 12 km southwest of Tangerang. The borrow pit area covers a space of about 10,000 m^2. As the layer of borrow pit has a thickness of 4 m, the quantity of borrow material reaches approximately 40,000 m^3.

The soil material available from the above three borrow sites appears adequate to meet the requirement (about 220,000 m^3) for the construction of embankment of the proposed railway. In recent years, however, construction of new roads and reclamation of lowlands are being actively carried out, so that the borrow material available from the above three sites only may be inadequate to meet the needs of the railway construction project.

However, additional quantities of borrow material is obtainable, even if borrow pit is not available in the vicinity of a construction site. Through consultation with landowners, it is relatively easy to excavate nearby hilly areas and divert the excavated soil to housing districts. Also, the soil test of embankment materials has proved that there is little difference, in properties of soil among the sample soils taken from the above three sites, although these sites are located quite far apart from each other. These conditions suggest that there would be no problem in supplying a required quantity of embankment material for railway construction.

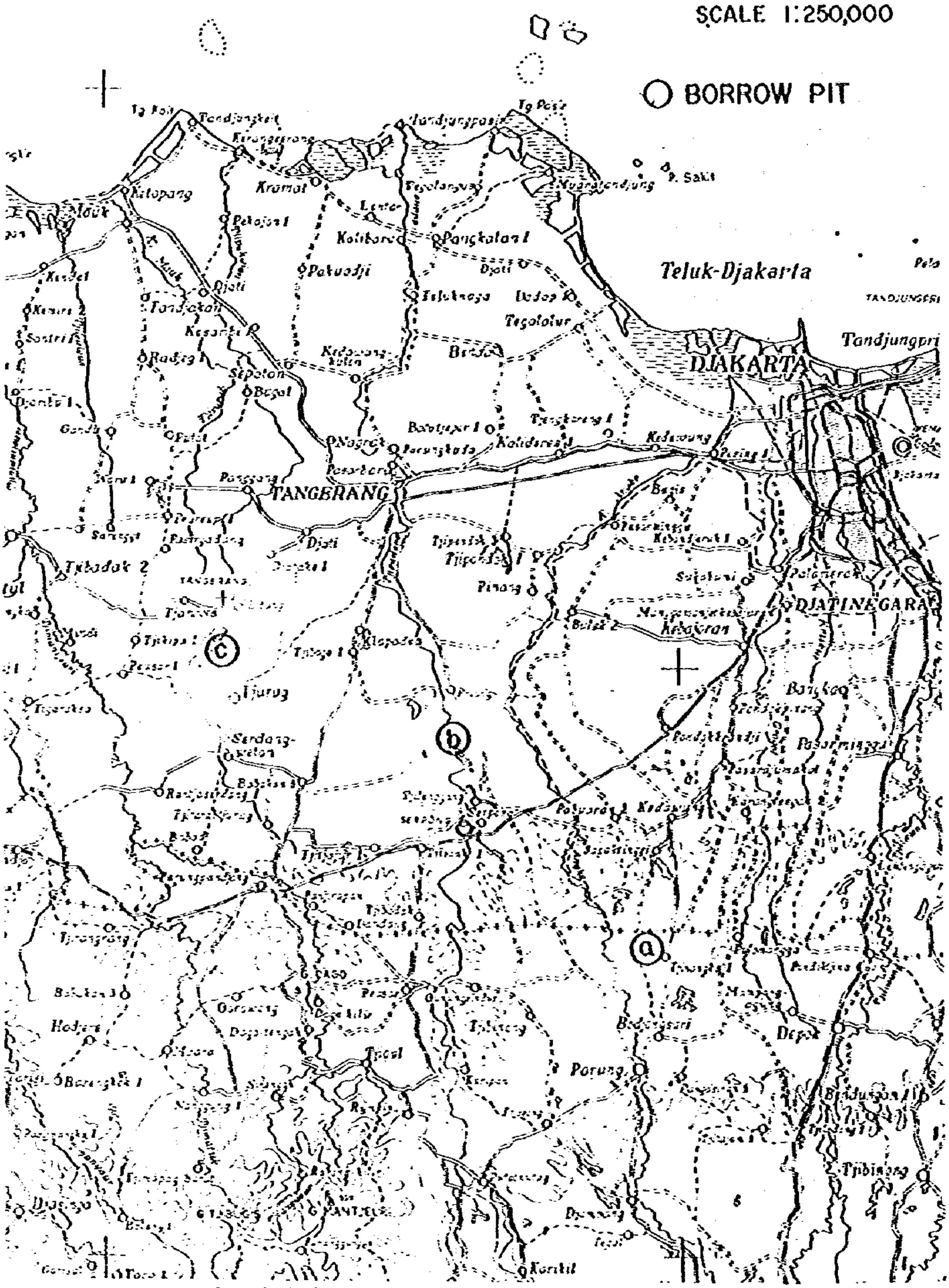


Fig. 3–2–27 Location of Construction Materials

Table 3–2–7 Sample Collecting Depth

Site	Quantity (pieces) of Collected Sample	Sample Collecting Depth (GL–m)
Pondok Petir	2	1.0, 2.7
Lengkong Gudang	2	1.0, 2.5
Curug	1	2.0

(2) Results of Soil Tests

In order to verify the suitability of materials for embankment, soil tests of collected samples were made to the following:

- Specific gravity
- Grading
- Liquid limit
- Plastic limit
- Compaction test by means of a rammer
- Test on the modificatory California Bearing Ratio of Soils

Table 3–2–8 shows the results of a laboratory test. The collected samples comprise laterized clayey soil, which is a diluvial deposit containing a lot of clay and very little sand. The percentage of natural moisture content is close to the plastic limit, indicating that these samples are quite stable. According to the results of a compaction test, the optimum moisture content ranges from 34 to 40% and the maximum dry density from 1.2 to 1.4 t/m^3, a quite high value for clayey soil. The modified C.B.R., which is intended to examine the strength properties of roadbed, reached 3.4 to 9.1%.

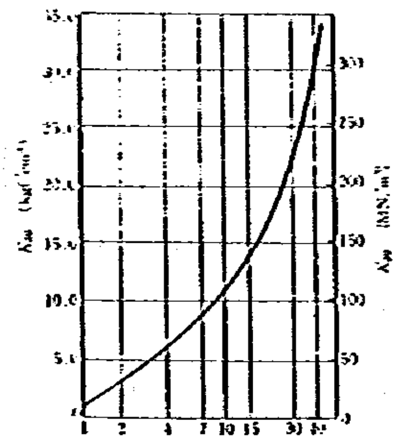
As reference, on-site C.B.R. values in Japan categorized by soil indicate that, the value is less than 3 in the loam which is composed of volcanic ash and features high water content, as well as in the soil which contains a lot of clay and silt and features high water content. The value ranges between 3 and 5 in the soil which is composed of volcanic ash and features lower water content, as well as in the soil which contains a lot of clay and silt but features low content. It mostly ranges from 3 to 7 in the loamy soil which is not composed of volcanic ash. It is about 7 to 10 in the sandy loam with low water content.

When compared with the data, the C.B.R. ranging between 3.4 and 9.1 represents a favorable value for the soil composed of volcanic ash. But this is not so favorable if compared with the sandy soil of which C.B.R. ranges from 7 to 15.

Table 3–2–8 Results of Laboratory Test

		TP–1	TP–2	TP–3	TP–4	TP–5
Sample Number		TP–1	TP–2	TP–3	TP–4	TP–5
Depth (m)		1.00	2.00	1.00	2.50	1.00
Grain Size Properties	Gravel (over 2000μ) (%)	0.0	0.0	0.3	0.0	0.0
	Sand (74 – 2000μ) (%)	1.7	3.0	3.2	2.3	4.7
	Silt (5μ – 74μ) (%)	43.3	40.0	29.5	44.7	30.3
	Clay (less than 5μ) (%)	55.0	57.0	67.0	53.0	65.0
	Maxim Grain Size (mm)	4.76	4.76	9.52	4.76	4.76
	Coefficient of Uniformity Uc	–	–	–	–	–
	Coefficient of Curvature U'c	–	–	–	–	–
Consistency Properties	Liquid Limit W_L (%)	110.80	87.02	92.50	91.80	91.50
	Plastic Limit w_p (%)	35.69	36.21	52.53	36.42	38.21
	Plasticity Index I_p	75.11	50.81	33.97	55.38	53.29
	Consistency Index Ic					
Classification of Soil						
Specific Gravity Gs		2.656	2.656	2.657	2.659	2.658
	Water Content w (%)	44.80	46.31	53.62	50.29	43.13
	Unit Weight γt (g/cm³)					
	Dry Density rd (g/cm³)					
	Void Ratio e					
	Saturation Sr (%)					
Compaction Properties	Method of Test					
	Optimum Moisture Content wopt (%)	35.0	37.5	39.5	39.5	34.32
	Maximum Dry Density rd max (t/m³)	1.321	1.302	1.233	1.224	1.359
C. B. R. Properties	Water Content w (%)					
	Dry Density rd (t/m³)					
	C.B.R. (%)					
	Water Content W (%)					
	Dry Density rd (t/m³)					
	Modificatory C.B.R. (%)	6.9	3.4	4.8	3.9	9.1

The modified C.B.R. and the coefficient of foundation (K_{30}) have a correlation as illustrated in the figure below. According to this figure, the value of K_{30} ranges from 5 to 8 kg/cm³, suggesting that extra attention must be paid to the control of water content during compaction.



CBR (%)
(Undisturbed Soil Immersed for 4 days)

Relationship between CBR and K_{30} Value

3–3 Hydrological Survey

3–3–1 Current Conditions

Topographic features of the area adjacent to Jakarta consists of coastal plain at northern district and hilly areas at southern district. Elevation of the coastal plain is P.P. 1 to 3 m, and distributed in the linear zone of width of approximately 3 to 5 km. Elevation of the hilly area at southern district is P.P. 5 to 25 m and widely eroded by the rivers Angke or Ciliwung and its slopes are low graded.

Since the beach ridges with elevation of P.P. 3 to 5 m are located between the coastal plain and hilly area, configuration of the river system is varied in the surrounding area.

The river with a large scale and voluminous water flow at the hilly area tends to erode laterally and meander remarkably because of the low river grade.

These rivers flow towards the north to the beach ridges, however, they were intercepted by the beach ridges, diverted its flow to the direction of east-west and fall down to the shore line at part of the lower elevation between the beach ridges.

At the coastal plain side, river grade becomes lower not having sufficient flow velocity to cause lateral erosion and the rivers have formed linear configuration.

Water table of the rivers are varied by approximately 0.2 to 1 m in rainy season and dry season. River system of the area adjacent to the project location is as shown on Fig. 3–3–1.

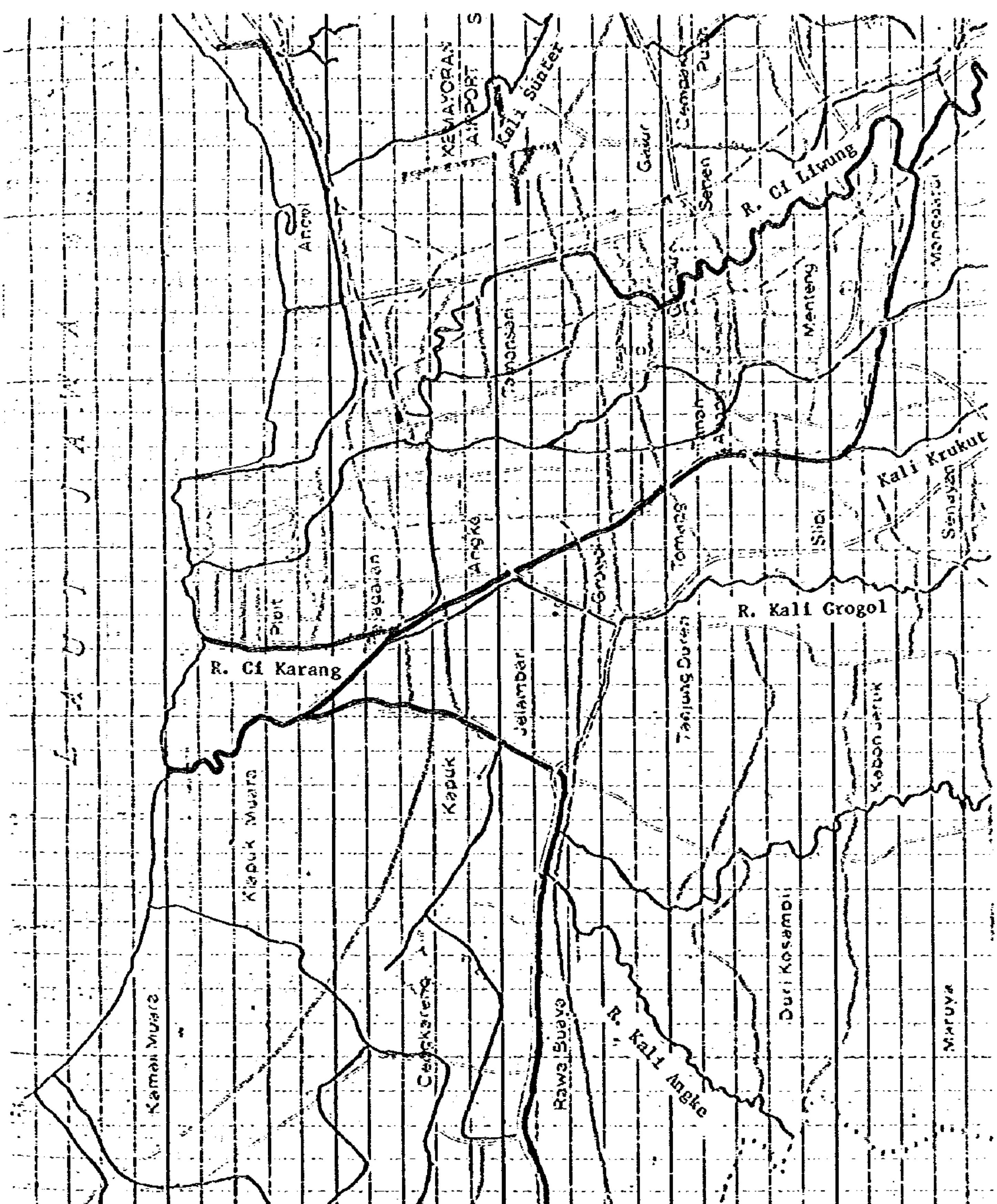


Fig. 3–3–1 Rivers in "JABOTABEK Area"

Scale 1 : 75,000

3–3–2 Water Table and Water Quality

Due to lack of data, investigation was made by hearing on the water table and water quality of wells located along the proposed route in the dry season.

Result of the investigation is as shown on Table 3–3–1.

Water table at the project location is 2 m to 3 m below the G.L. in the dry season, however, it becomes 0.5 m below the G.L. in the rainy season, increasing the table by about 1 to 2 m. Water quality comprises the freshwater and saltywater, and not much ground water is connected to the water table of the river assuming that the river water is freshwater.

In particular, over the beach ridge area, only a few wells containing the freshwater discovered and it is assumed that the freshwater channel is scattered in the pocket like configuration.

As a matter of civil engineering, countermeasures will be required against sulfate attack by salty water where steel piles that are readily corroded by chlorinated substances are adopted as the foundation materials for construction.

Table 3–3–1 Water Table and Water Quality Survey

Location (km)	Water Table (Dry Season) (G.L. – m)	Water Table (Rainy Season) (G.L. – m)	Water Quality	Remarks
19.0	2.0 ~ 3.5	0.4 ~ 1.5	Salty Water	Flooded in heavy rain
18.6	1.0	0	Salty Water	Chlorinated due to reclamation of adjacent canal
17.7	1.9 ~ 3.0	0.4 ~ 1.5	Fresh Water	
17.0	2.2	1.2	Salty Water	Water table of adjacent river is G.L. – 0.5 m
16.0	2.0 ~ 2.3	0 ~ 0.2	Salty Water	
15	1.8	1.2	Salty Water	
14	1.8	1.8	Salty Water	
11	3.5 ~ 4.0	0.8 ~ 1.3	Fresh Water	
10	3	0.8	Salty Water	
8.5	1	0.1	Salty Water	
7.5	1.9 ~ 2.2	0.7 ~ 1.3	Fresh Water	North side of Proposed Route
6.7	1.9	1.2	Salty Water	
6.7	1.7	0.4	Fresh Water	
6.1	2.4	0	Salty Water	North side of Proposed Route
6.1	2.2	0.4	Fresh Water	
5.5	2.0 ~ 2.8	0.6 ~ 2.0	Fresh Water	
4.0	1.6 ~ 1.9	0.6	Fresh Water	North side of Proposed Route
4.0	2.1	1.1	Salty Water	
3.3	1.3 ~ 1.8	0.9 ~ 1.4	Fresh Water	
2.7	1.6 ~ 3.0	1.0	Salty Water	
2.7	2.1	0.5	Fresh Water	
2.5	2.4	0.5	Fresh Water	

3-3-3 Meteorological Data

(1) Rainfall

Maximum rainfall as picked up by meteorological data per hour, per day and per month is indicated as follows:

(mm)

Year	Per hour	Per day	Per month
1973	61.8	90.2	367.6
1974	61.1	101.8	573.7
1975	50.8	68.4	287.9
1976	89.8	110.0	305.3
1977	78.0	203.9	660.3
1978	54.9	72.8	274.1
1979	95.7	198.0	734.0
1980	–	92.2	526.0
1981	–	125.2	576.0
1982	–	65.3	354.0

(2) Wind Velocity

Maximum wind velocity as picked up by meteorological data is indicated as follows:

(m/sec.)

Year	Month	Direction	Max. wind velocity
1973	7	E	11.5
1974	11	NE	12.5
1975	11	S	13.0
1976	6	SW	12.0
1977	6	SE	10.0
1978	3	W	10.0
1979	1	N	10.5
1980	2	N	9.5
1981	3	W	10.0
1982	1	NW	6.5

(3) Floods and Its Countermeasure

In view of geography, the sector of the proposed route is located along the coast and adjacent to inlets of large and small rivers and concentrated heavy rain particular to tropical areas is seen in the rainy season.

Effective drainage system however has not been provided in this area and caused frequent floods because the area is flat in topography.

Data of disaster due to the floods was not made available to the study team despite the request made to the office concerned. Flood problem of Jakarta city is the great concern of not only municipality but also the central government in view of the fact that the Jakarta city is the capital of the Republic of Indonesia.

The Government of Indonesia therefore, completed a master plan with assistance of NEDECO of Netherland in December, 1973 for Drainage and Flood Control of Jakarta.

The plan has recommended to construct West Banjir Canal and East Banjir Canal such that the city is surrounded by the canals from the south side of the city towards both east and west sides of the city.

Both canals are being constructed at present, however, diversion of the route for the West Banjir Canal would be inevitable due to the difficulty in acquisition of land, according to the news source.

4 TRAIN OPERATION PLAN

4. TRAIN OPERATION PLAN

4–1 Basic Concept of Train Operation Plan

As mentioned in the Report on Feasibility Study of New Railway Line for Cengkareng Airport, Japan International Cooperation Agency, 1983 (hereinafter referred to as "F/S Report"), the basic concept of train operation plan of the New Cengkareng Airport Railway Line includes the following major factors.

4–1–1 Transport Demand Estimate

Fig. 4–1–1 shows the estimated daily number of railway passengers, based on the F/S Report, going to and from the Cengkareng Airport using the Cengkareng Railway.

4–1–2 Fundamental Conditions of Train Operation Plan

(1) In view of the number of passengers estimated in Paragraph 4–1–1 above, the train service covering the proposed railway line shall extend from the Airport Station through Cengkareng and the Central Line to the Jatinegara Station.

(2) At the initial stage, a train will be made up of 4 cars (2 driving cars and 2 trailers), with a minimum headway of 20 minutes, in compliance with the predicted demand for transportation in 1990.

(3) The performance characteristics of the trains to be operated on the proposed railway line shall be the same as those of the trains now being used in the JABOTABEK district. This is because it is desirable that those trains have the same performance characteristics as the existing ones since they will run direct through the Central Line, as indicated in Paragraph (1).

The performance curve and the acceleration curve of the proposed train are shown on Fig. 4–1–2 and Fig. 4–1–3 respectively. The proposed train can be operated at balancing speed of 60 km/h on the gradient of 25 o/oo, and it can start to move even on such a gradient.

(4) To ensure the safety for the high-speed and high-density train operation, the proposed railway line will be provided with an automatic signaling system with automatic train stop devices.

(5) Service hours of the proposed railway line will be from 04:30 till 22:30.

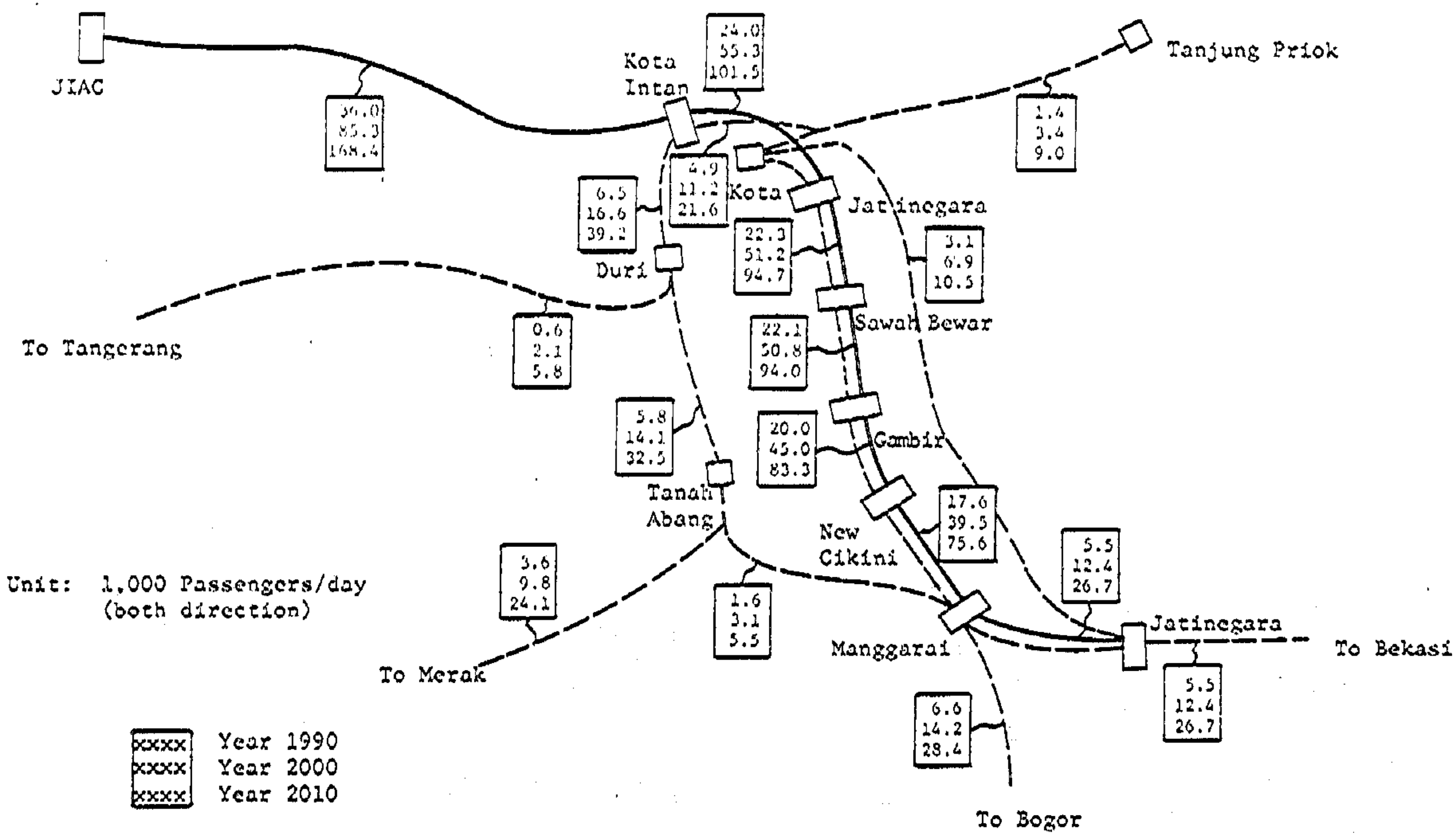


Fig. 4–1–1 Estimated Number of Railway Passengers
(reproduced from the Feasibility Study Report)

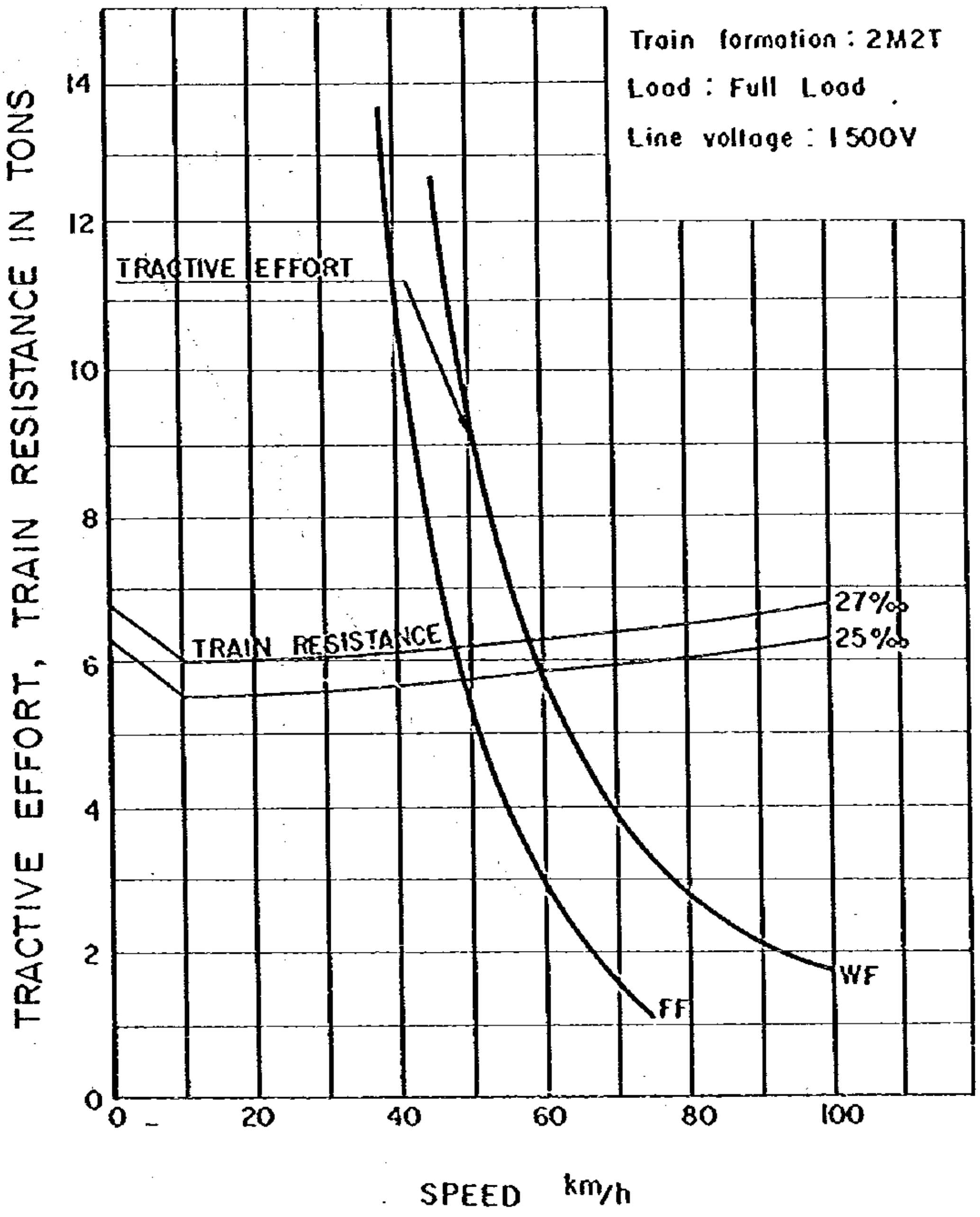


Fig. 4-1-2 Characteristic Performance Curve of Proposed Electric Railcar Train

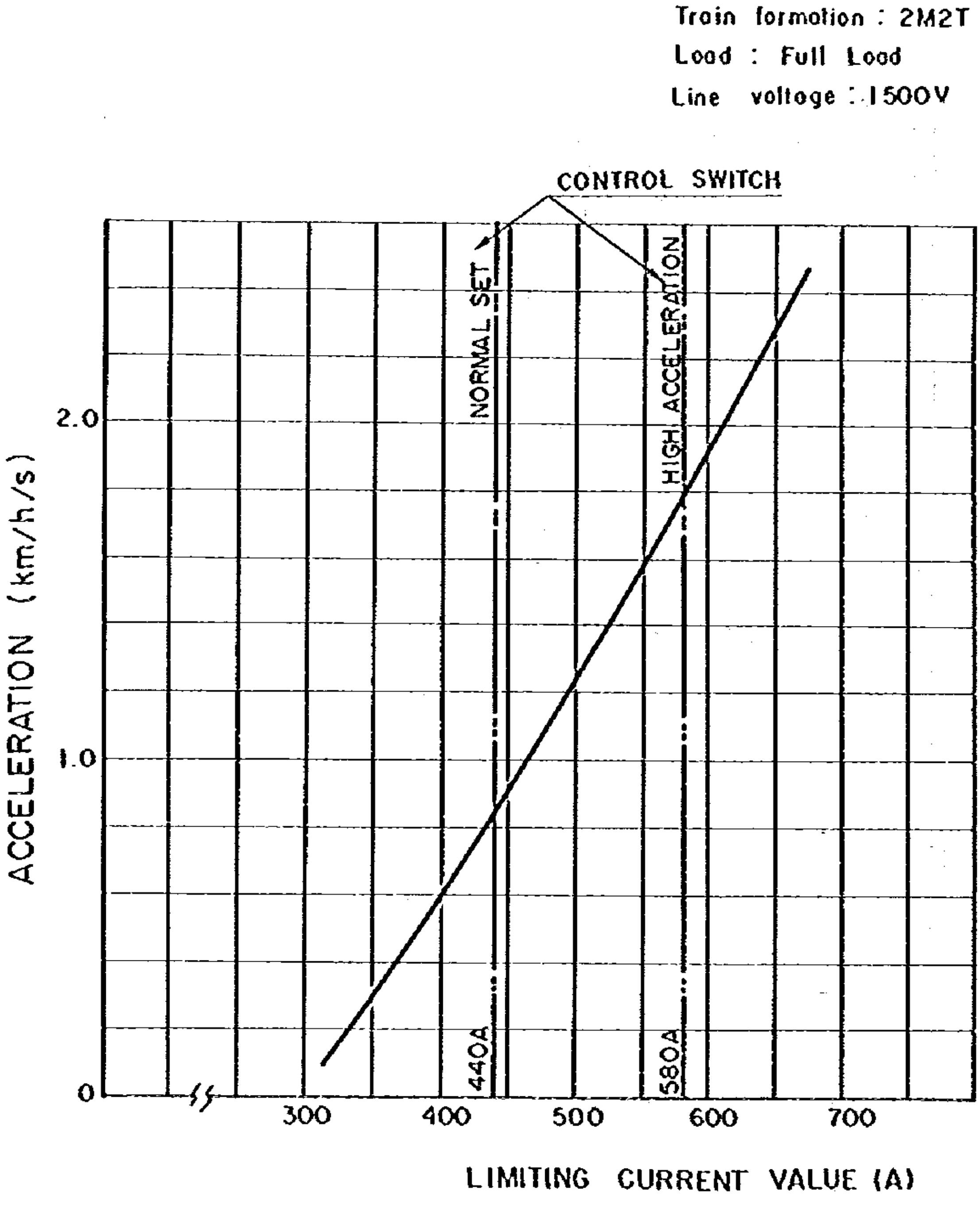


Fig. 4-1-3 Acceleration Curve of Proposed Electric Railcar Train at Starting on Gradient of 27 o/oo

4–2 Train Run Curve and Train Diagram

4–2–1 Train Run Curve

The train run curve has been based on the following conditions. In the vicinity of Jayakarta, the proposed railway line shall be connected to the Central Line, which shall remain on the ground level as it is now. Furthermore, a study was made assuming the Central Line was elevated.

(1) The highest speed of trains on the proposed railway line will be set at 100 km/h. The highest speed on the Central Line will be fixed at 60 km/h for the ground level line and 100 km/h for the elevated line.

(2) Trains on the proposed railway line will run within the speed of 45 km/h when passing on the turnout side of turnouts at each station. At the Manggarai Station of the Central Line, trains will run within the speed of 25 km/h on the ground level line and 35 km/h on the elevated line.

(3) The horizontal and vertical alignment of the Central Line will remain unchanged on the ground level line. In the case when the line is elevated, it will be formed as indicated in the Report on Feasibility Study on Track Elevation of Central Line, JICA, 1982.

(4) The line voltage will be 1,350 V (10% lower than 1,500 V), with the load factor being set at 100%.

(5) At the Airport Station, two platforms will be used alternately for the departure and arrival of trains. Here, the train run curve is drawn on conditions that trains run on the turnout side of turnout.

Fig. 4–2–1 and 4–2–2 show the train run curves prepared on the basis of the conditions described above.

4–2–2 Travel Time

Table 4–2–1 shows the travel time between stations calculated on the basis of the train run curve as explained in Paragraph 4–2–1. The travel time between Airport and Jatinegara Stations, including the one minute stop time at each intermediate station, is also shown in this table.

4–2–3 Train Diagram

Fig. 4–2–3 and 4–2–4 show the pattern of train diagram, with trains running at 20 minutes intervals based on the travel time mentioned in Paragraph 4–2–2 above.

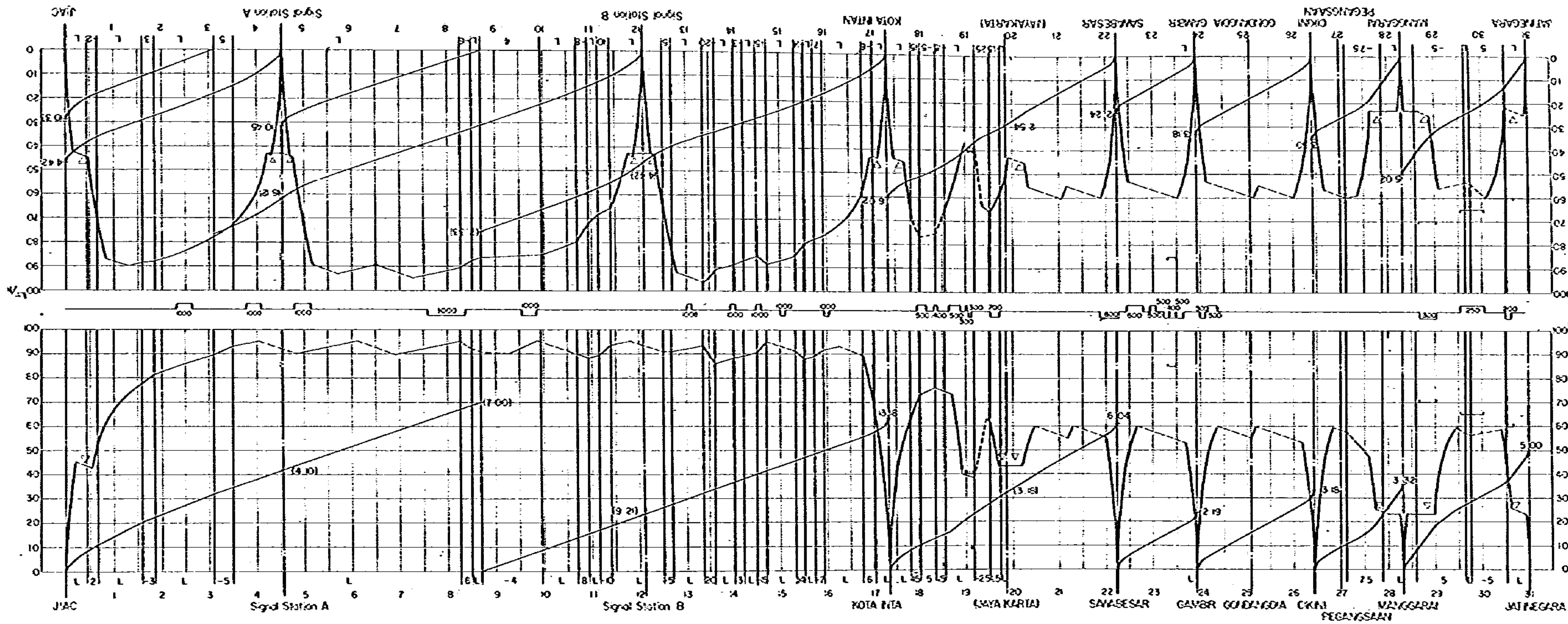


Fig. 4–2–1 Train Run Curve (Central Line: Ground Level)

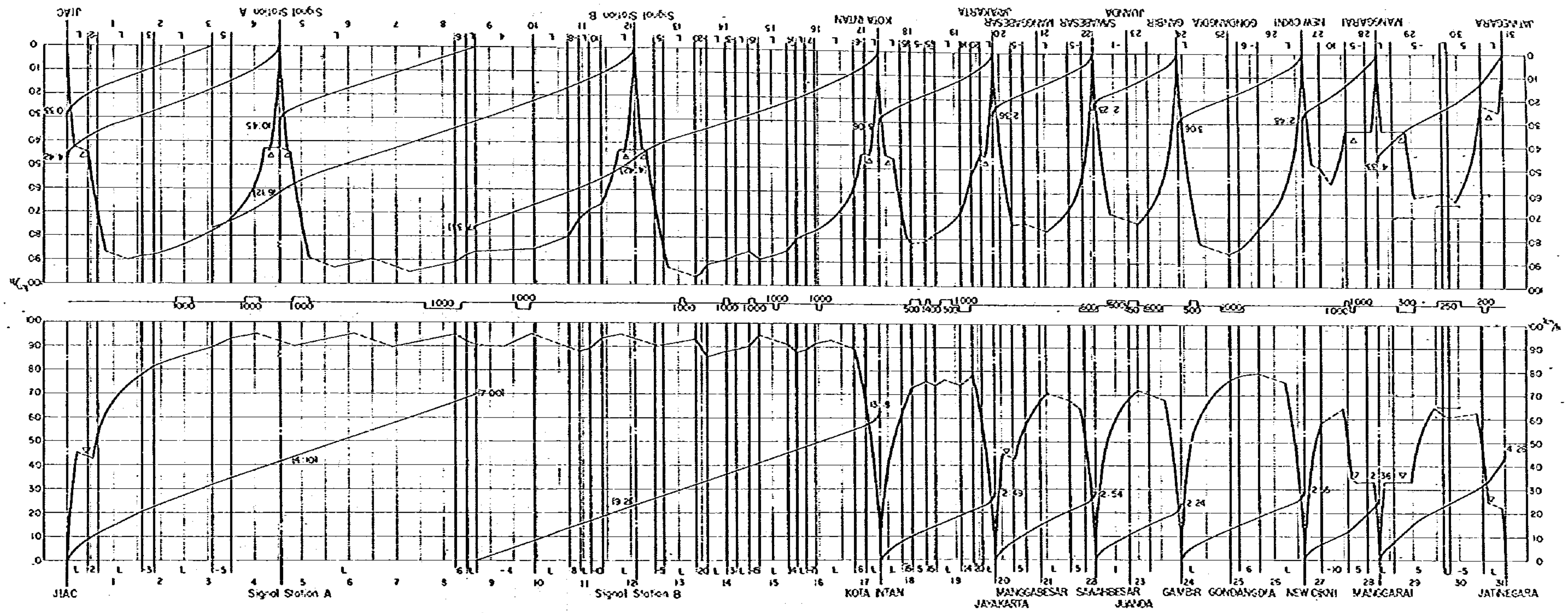


Fig. 4-2-2 Train Run Curve (Central Line: Elevated)

Table 4-2-1 Travel Time

(Ground Level Central Line) (min. : sec.)

	Kilometer	Station Distance	For Jatinegara Calc.	For Jatinegara Adjusted	For JIAC Calc.	For JIAC Adjusted
JIAC	0					
		(km) 4.540	4:10	4:30	4:42	5:00
Signal Station A	4.540					
		7.560	5:11	5:00	6:03	6:00
Signal Station B	12.100					
		5.265	3:57	4:00	4:42	4:30
Kota Intan	17.365					
		2.499	3:10	3:30	3:08	3:30
Jayakarta	(19.864)					
		2.190	2:54	3:00	2:54	3:00
Sawahbesar	22.054					
		1.860	2:19	2:30	2:24	2:30
Gambir	23.914					
		2.483	3:18	3:30	3:18	3:30
Cikini	26.397					
		1.857	3:32	3:30	3:30	3:30
Manggarai	28.254					
		2.746	5:00	5:30	5:02	5:30
Jatinegara	31.000					
Total		31.000	33:31	35:00	35:43	37:00
Travel time (inc. Station Stop)				40:00		43:00

(Elevated Central Line)

	Kilometer	Station Distance	For Jatinegara Calc.	For Jatinegara Adjusted	For JIAC Calc.	For JIAC Adjusted
JIAC	0					
		(km) 4.540	4:10	4:30	4:42	5:00
Signal Station A	4.540					
		7.560	5:11	5:00	6:03	6:00
Signal Station B	12.100					
		5.265	3:57	4:00	4:42	4:30
Kota Intan	17.365					
		2.499	2:49	3:00	3:06	3:00
Jayakarta	19.864					
		2.190	2:54	3:00	2:36	3:00
Sawahbesar	22.054					
		1.860	2:24	2:30	2:23	2:30
Gambir	23.914					
		2.750	2:55	3:00	3:06	3:00
New Cikini	26.664					
		1.590	2:36	2:30	2:48	3:00
Manggarai	28.254					
		2.746	4:26	5:00	4:33	5:00
Jatinegara	31.000					
Total		31.000	31:20	32:30	33:59	35:00
Travel time (inc. Station stop)				38:30		42:00

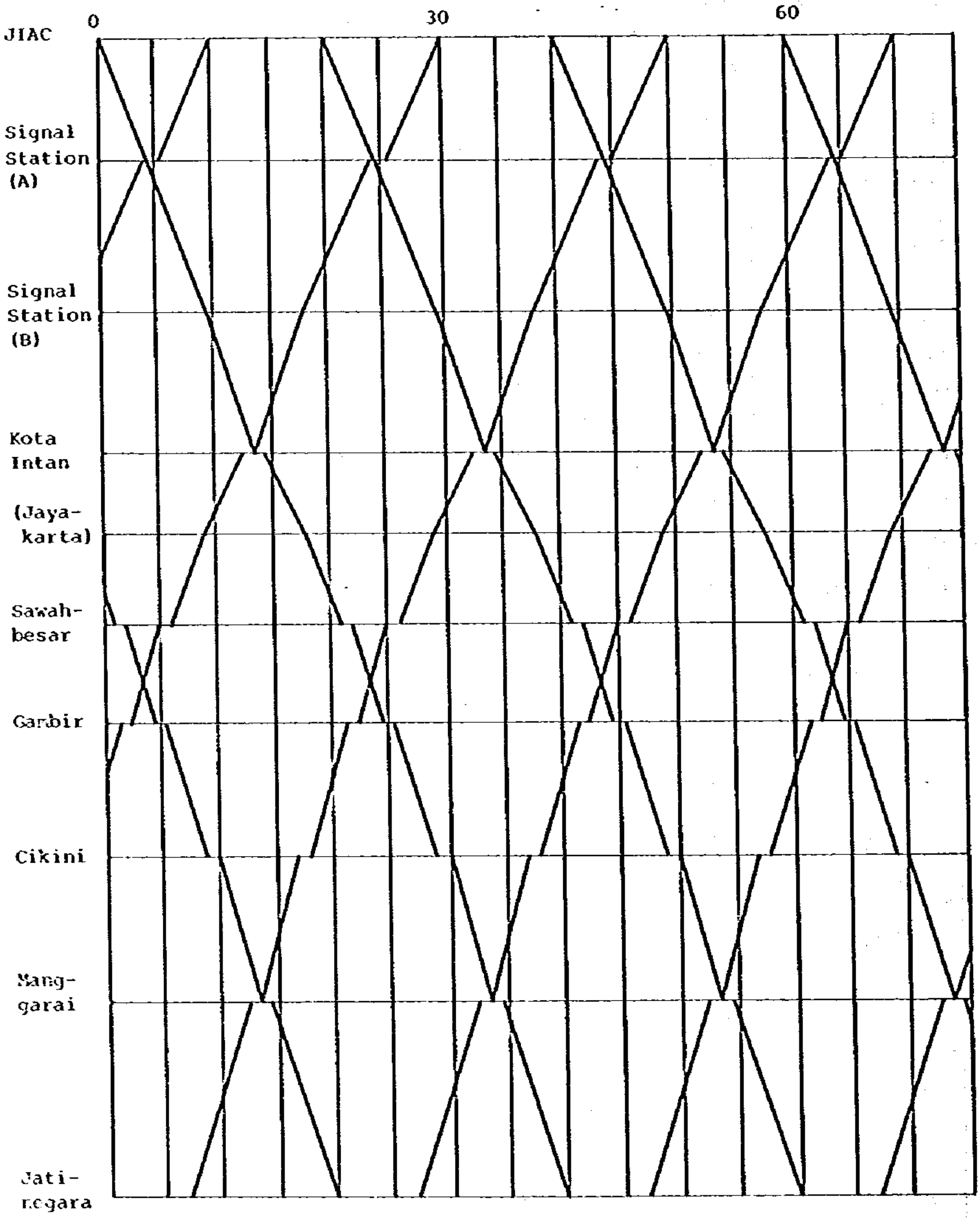


Fig. 4-2-3 Train Operation Diagram
(Central Line: Ground Level)

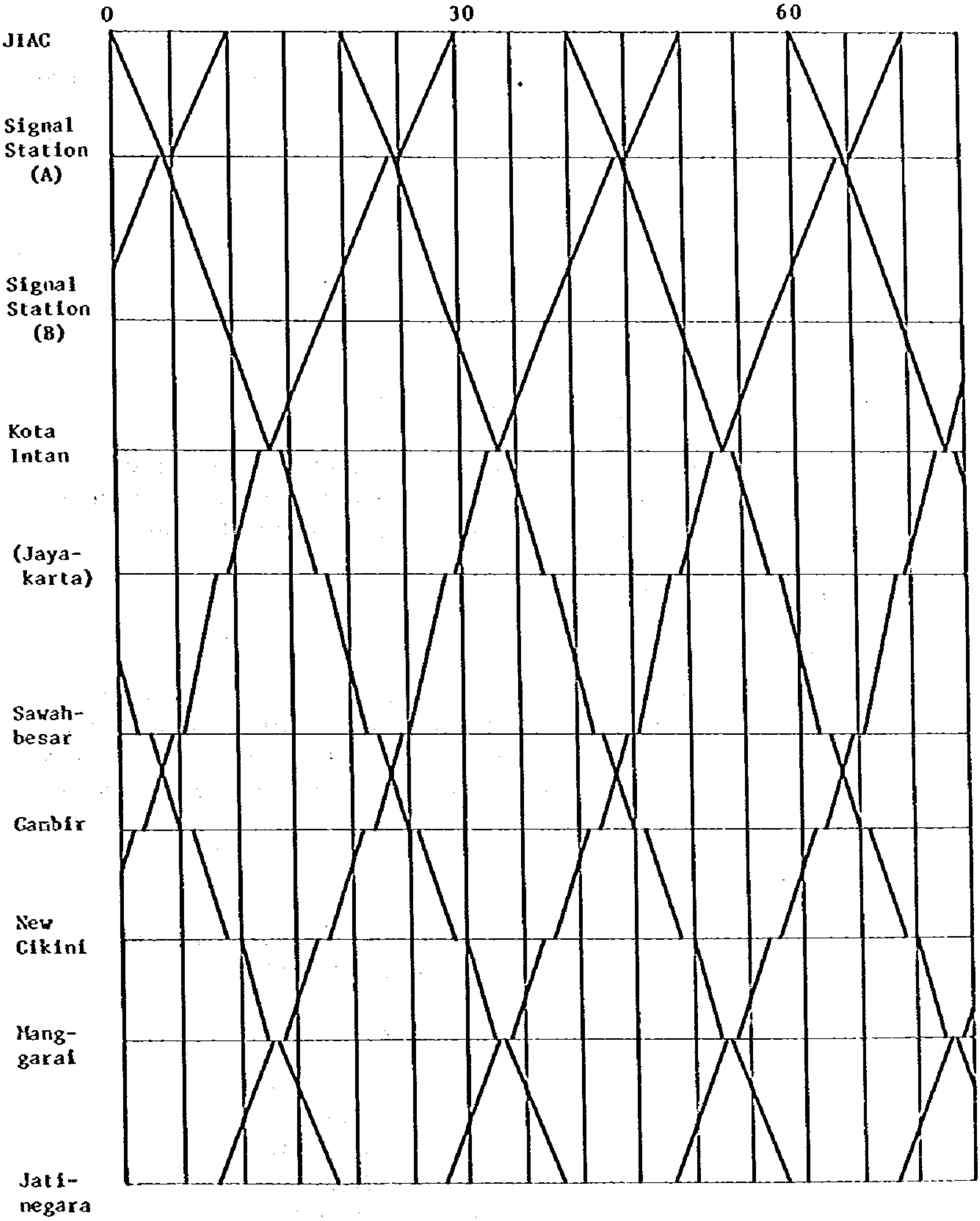


Fig. 4-2-4 Train Operation Diagram
(Central Line: Elevated)

4–3 Railcar Plan

4–3–1 Number of Railcars Required

Calculation of the time involved in a round trip between Airport and Jatinegara Stations runs as follows:

The ground level Central Line:

(40 + 43)* + (10 + 7)** = 100 (minutes)

The elevated Central Line:

(38.5 + 42)* + (10 + 9.5)** = 100 (minutes)

Note: * Travel time
** Return time at Airport and Jatinegara Stations

The above time schedule requires 5 sets of trains being operated at an interval of 20 minutes. With another set in reserve being added, the required set of trains totals 6. Since a train is made up of 4 cars at the initial stage, the required number of cars totals 24 (4 x 6).

4–3–2 Car Operation and Inspection

(1) Car Operation

From the standpoint of administrative efficiency, car operations will be conducted from the Bukit Duri Car Depot, located close to the Manggarai Station, where trains on the proposed railway line stop. Also, for the sake of operational efficiency, 3 sets of service trains (out of 5 sets) will be stabled at Bukit Duri Car Depot and the remaining 2 sets at the Airport Station at night.

(2) Car Inspection

Car inspection will be conducted periodically as shown below, based on the Report on Urban/Suburban Railway Transportation in Jabotabek Area, JICA, 1981.

Type of Inspection	Cycle
Daily	48 hrs. / 3,000 km
Monthly	60 days / 30,000 km
Half-yearly (for bogie)	1 year / 150,000 km
Principal equipment	2 years / 300,000 km
General	4 years / 600,000 km

The daily, monthly, and half-yearly (for bogie) inspections will be conducted at car depots, whereas the principal equipment and general inspections will be conducted in a workshop. Trains to be used for the proposed railway line will undergo daily inspection at the Bukit Duri Car Depot, monthly and half-yearly inspections at the Depok Car Depot (which will be built according to the master plan), and principal equipment and general inspections at the Manggarai Workshop.

5 DESIGN CRITERIA

5. DESIGN CRITERIA

5–1 Basic Concept

The design criteria for the detailed design were determined after a thorough study and various discussions with pertinent agencies taking into account the following aspects:

(1) The proposed railway line is intended exclusively for passenger traffic for which train operations with high speed, safety, punctuality and riding comfort would be inevitable.

(2) Various facilities should be sufficient to provide maximum service at minimum investment.

(3) Since the proposed railway line project is in close connection with "JABOTABEK metropolitan railway transportation plan", consistency in the design criteria with those of JABOTABEK is imperative.

(4) The design criteria should be based on those mentioned in the Feasibility Study Report as previously submitted by JICA, except for partial modification at a later stage that would be deemed necessary through further coordinations.

(5) The design criteria should comply, where possible, with various standards and regulations of the Indonesian State Railways.

5–2 Determination of Design Criteria

The design criteria as proposed and determined for the airport railway line are listed in Table 5–2–1 below. Some exceptional criteria applicable only to this line were agreed upon considering the special condition of the line. Of these items listed in this Table, however, major items mentioned in paragraph 5–1. (4) are described below:

(1) Gradient

It is envisaged in the plan to use the electric railcar of which performance characteristics is equivalent to those as currently in use in JABOTABEK area. The maximum gradient on which electric railcars are assumed to be operable is 25 o/oo. There was an intent to restrict the maximum gradient to 17 o/oo in consideration of the existing maximum gradient of 15 o/oo as provided and possibility of freight traffic operations in the future, however, it would create no operational problem even on the gradient of 25 o/oo provided that electric locomotives be introduced for the operation because the hauling tonnage of the freight train would be limited to 530 t in view of the effective track length within the JABOTABEK area. Concurrence was therefore obtained to adopt the maximum gradient of 25 o/oo for the airport railway line.

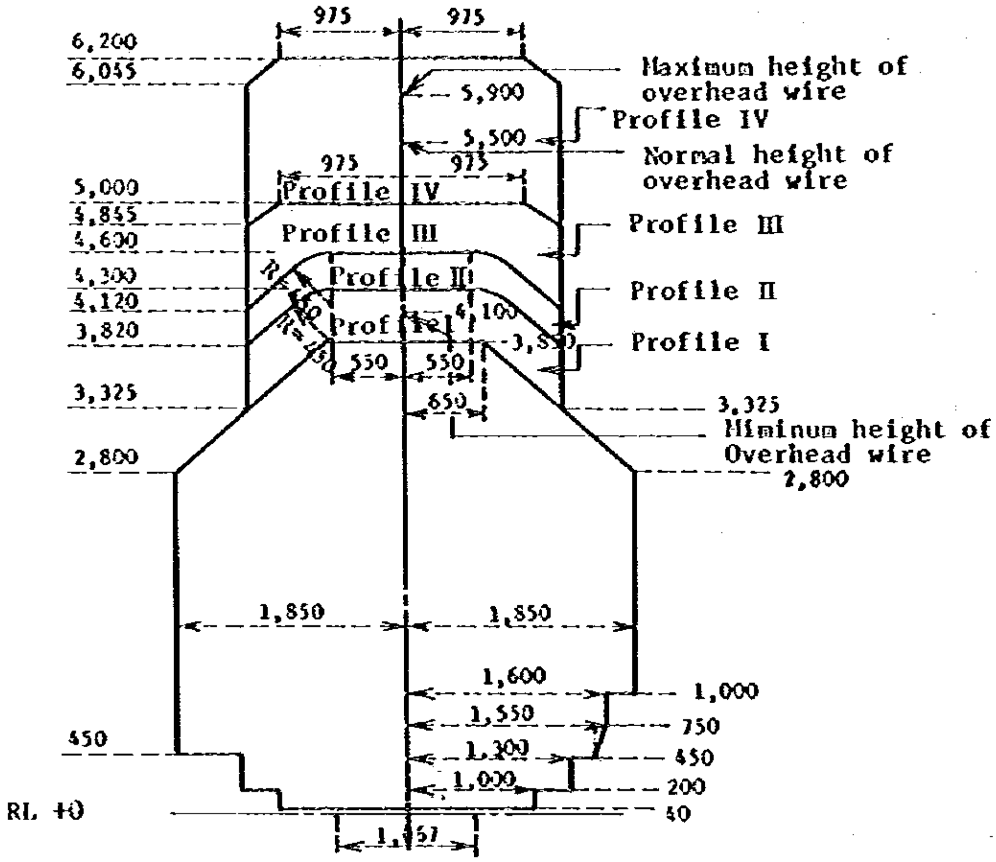
(2) Train Load

Load scheme equivalent to KS–12 will be adequate for this line. It was considered to alter the load scheme equivalent to KS–18 because of possible operation of freight train in the future on this line, however, the load scheme of "RM 75% 1921" equivalent to KS–16 currently used in the PJKA was considered generally adequate. Train load for the design, the load scheme equivalent to KS–16 was therefore adopted.

Table 5-2-1 Design Criteria

Item		Standard
Min. radius of curvature	Main track	600m (300)
	Turnout curve behind frog	320m (160)
	Section along platform	600m (500)
	Side track	160m (turnout curve behind frog)
Max. gradient	Main track	25 o/oo
	Main track in station	1.5 o/oo
Track-center distance	Outside of station	4.0m (3.8m), 5.5m for viaduct
	Inside of station	4.0m (3.8m)
	Gauge	1.067m
	Weight of rail	R54
Track	Sleeper	Prestressed concrete
	Ballast thickness of track	300mm
	Turnout	#12, #10 for side track
Width of formation level (from track center; respectively)		2.70m 2.95m for viaduct
Bridge bearing capacity (design train load)		kS-16
Others	Maximum design speed	100km/h
	Maximum cant	110mm
	Transition curve	Cubic parabola L = 60m in the case of R = 500m L = 60m in the case of R =1000m L = 25m in the case of R = 1400m L = transition curve length R = radius of curve
	Vertical curve	3000m in the case where radius of horizontal curve R ≧ 800m 4000m in other cases
	Construction gauge	As shown in Fig. 5-2-1

Note 1: Indication in parenthesis is applicable to unavoidable cases.



Profile I : Minimum profile for Bridge with speed restriction 60 km/hour

Profile II : Minimum profile for Tunnel and Viaduct with speed restriction 60 km/hour and for Bridge, no restriction

Profile III : Minimum profile for New Viaducts and New Construction, except tunnels and bridges

Profile IV : Normal profile for Electric Car

Fig. 5-2-1 Construction Gauge