NO. 53

THE KINGDOM OF THAILAND

The Feasibility Study on

THE RAMA VI BRIDGE CONSTRUCTION PROJECT

FINAL REPORT
IN
PHASE II (PRELIMINARY DESIGN)

MARCH 1982

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

In response to the request of the Government of the Kingdom of Thailand, the Government of Japan decided to conduct a study on the RAMA VI Bridge Construction Project in Bangkok which consists of two phases: Phase I (Feasibility Study) and Phase II (Preliminary Design), and entrusted it to the Japan International Cooperation Agency (JICA).

JICA sent to Thailand a Survey Team headed by Mr. Terukazu Endo from June 7 to December 6, 1981. The team exchanged views with the officials concerned of the Government of Thailand over the Project and conducted a field survey in Bangkok, and completed the Phase I Study in September 1981.

Subsequently further studies were made and the present Phase II 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 Kingdom of Thailand for their close cooperation extended to the team.

March, 1982

Keisuke Arita

KEISUKE ARITA
President
Japan International Cooperation Agency

ABBREVIATIONS

GOVERNMENT OFFICES:

BMA	Bangkok Metropolitan Administration
DOH	Department of Highway, Ministry of Communications
EGAT	Electricity Generating Aughority of Thailand
JICA	Japan International Cooperation Agency
OECF	The Overseas Economic Cooperation Fund (Japan)
PWD	Department of Public Works, Ministry of Interior
RID	Royal Irrigation Department, Ministry of Agriculture and Cooperatives
RTASD	Royal Thai Armed Forces Survey Department, Ministry of Defence
SRT	The State Railway of Thailand

ECONOMICS:

IRR	Internal Rate of Return
NPV	Net Present Value

ENGINEERING:

AASHTO	American Association of State Highway and Transportation Officials
BS	British Standard
DWT	Dead Weight Tonnage
GL	Ground Level
MSL	Mean Sea Level
HWL	High Water Level

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SUMMARY

1. Background

In July, 1980, the Government of Thailand made a request to the Government of Japan to conduct a feasibility study on the RAMA VI Bridge Construction Project. The proposed bridge is situated across the Chao Phraya River being as a vital link of the Middle Ring Road in Bangkok.

In response to the above request, the Government of Japan decided to conduct the Feasibility Study for the project and entrusted JICA with dispatching a field survey team to Thailand.

The study started in June 1981 and was carried out until December 1981 for 6 months. The study was made up of two phases: Phase I as a Feasibility Study and Phase II as a Preliminary Design of the bridge, and the Final Report of Feasibility Study in Phase I had been prepared and submitted to PWD of Thai Government in September 1981.

The present study of Phase II consists of the Preliminary Design for bridges and roads including an economic evaluation of the project. The Draft Final Report for Phase II Study had been compiled and submitted to PWD in December 1981 and, in return, PWD had issued a comment for the report to JICA at the end of January 1982.

This report comprises the Final Report of Preliminary Design in Phase II of the RAMA VI Bridge Construction Project.

2. Objectives of the Study

The objectives of the Preliminary Design are to carry out a refined and more accurate design for the project on the basis of the precedent Feasibility Study: the selected optimum Route Alternative III and the adopted bridge type.

Almost all of design conditions adopted in the outline design of the Feasibility Study have been reconfirmed as viable and appropriate for the project in the process of the Preliminary Design in Phase II Study.

A notable alteration has been made on the Thonburi side intersection planning: the intersections at grade have been changed to a small scale trumpet-type interchange in order to attain more smooth traffic flows in connecting with the existing roads in the area.

Also as for the railway viaduct on the Thonburi side, the prestressed concrete bridge has been changed into a steel railway bridge in order to satisfy technical requirements caused by the double tracking plan by SRT.

Except for the above two major changes, the conclusions obtained from surveys and analyses in the Preliminary Design including a topographic survey, a soil survey, stress analyses of structures and traffic capacity checks at intersections, all have verified the previously established design factors in the Feasibility Study.

3. Bridges

In the outline design of the Feasibility Study, the main bridge design has been determined to adopt a 3-span (85 m + 120 m + 85 m) arrangements and a prestressed concrete box girder structure with cantilevering method.

In this Preliminary Design study, a topographic survey has been conducted and a more elaborate map in scale of 1/1,000 has been produced.

Recheckings have been made as to the bridge spans and arrangements of various structures.

It was affirmed that the design factors in the outline design of the Feasibility Study could be applied appropriately in the Preliminary Design of Phase II Study without any technical problem.

Foundation

The driven length into earth of both driven piles on shore and those of reverse circulation drilled piles have been studied using geological soil survey data and the assumed values in the Feasibility Study of MSL-20 meters for the former and MSL-45 meters for the latter and the both have been reconfirmed as appropriate.

Box girder structures

The box girder structure has been adopted in the Feasibility Study. In this Preliminary Design of Phase II Study, a further check has been made as to the best choice of the alternatives: 1) Continuous girders having better trafficability for travelling vehicles, or 2) Continuous T-shaped frame connected by sliding hinges in central span, which is a more economical and easier to construct.

As the result, the latter type with central hinges has been selected mainly because, for the proposed bridge design, the design speed of travelling vehicles will not exceed 60 km/h causing no trafficability inconvenience.

As the width of the main bridge varies from the minimum of 29.3 m to the maximum of 38.46 m in a scope of one continuous girder, this has presented a very important problem as to the configuration of girder cross sections, both in stress analyses and in construction procedures.

Hence the following design factors have been taken into considerations.

1) To design a constant cantilevered length for slabs in order to attain good aesthetical appearances.

2) To avoid abrupt changes in the girder cross section in a scope of one main girder length to attain more smooth flow of stresses in the structures.

3) To limit the maximum clear span length of carriageway slabs as not to exceed 7 meters.

As the results, a 1-cell in 1-box cross section configuration has been adopted for Thonburi side T-frames, and a width-changeable 2-cell in 1-box section has been used for Bangkok side where the roadway width varies longitudinally.

On the basis of above mentioned structural types, an analysis has been conducted in order to determine dimensions for each girder depth and each structural member.

After establishing dimensions for each structure, the span arrangement of the Feasibility Study has been reaffirmed as the most preferable configuration.

Bridge appearance

As the comment from PWD had referred to appearance of the proposed bridge, special considerations have been applied to the design of piers in the Preliminary Design.

In the proposed main bridge, the planned elevation of the bridge is rather low compared with the thick girder depths at the piers. The distance between the top of pile caps and the bottom of the main girder is very small and a stubby look would be attributed in case of adoptation of an ordinary upside-down T-shaped pier-footing.

A special design technique has been applied in order to eliminate these unfavorable effects in appearances.

Viaduct

The span arrangements of approach viaducts have been slightly altered in response to changes of highway design in the Preliminary Design.

Although, in the Feasibility Study, voided slabs have been used for viaduct cross sections, this has been changed for uses of unvoided slabs, because the latter has been found as more economical, even for larger dead loads, after comparing form work costs and labor costs for both types.

Railway Viaduct

The future plan of the SRT for double track of the existing RAMA VI Bridge railway line has become materialized during the Preliminary Design, and SRT requested that a special consideration should be taken as to the design of the railway viaduct on the Thonburi side.

The proposed viaduct should be the type of structure which enables the double track improvement could be done without changing the assumed distance of the double track on the existing RAMA VI Bridge.

The prestressed concrete railway bridge will not be used because the bridge width becomes much wider and the distance between the 2 tracks will be 2 meters longer than that of the RAMA VI Bridge tracks.

Accordingly, the railway viaduct has been redesigned as a steel railway viaduct in the Preliminary Design.

4. Highways and Alignment

In designing highway alignment, the optimum Route Alternative III has been rechecked using 1/1,000 scale topographic maps.

Thonburi Intersections

The intersections at grade designed in the outline design of the Feasibility Study has had a problem in dealing with incoming traffic from the bridge due to its short distance between the bridge end and the approach viaduct.

PWD has issued a comment that the traffic flows should be more smooth in Thonburi intersections in view of increased traffic in the near future. In response to the request, a small scale trumpet interchange has been designed as a solution.

Horizontal Alignment

Based on control points established on both sides of the river, curved alignments have been adopted: Bangkok side R=1,500 m, Thonburi side R=270 m. A=140 m. On the main bridge, a straight line has been adopted.

Longitudinal Alignment

The proposed main bridge comes upon a curved section of the Chao Phraya River. The longitudinal grade has been established as 4%, after checking necessary width of navigational and height under the main bridge.

On the Bangkok side, the distance between the end of on-and-off rampways and the Pibul-Wongsawang intersection has been so short, decelaration and accelaration lanes have to be installed besides the main roadway on the main bridge section.

The final longitudinal alignment attained for rampways has resulted in the combination carve of a sag and a crest for the Bangkok side section.

On the Thonburi side, the roadway has to under-cross a railway viaduct. The elevation of roadway has been controlled by the vertical clearance under the railway and also by HWL of small khlongs in the vicinity.

Cross section

The design standard of main roadway cross sections have been based on both DOH and BMA specifications, in both of which 30-meter wide right-of-way and 6-lane roadway are specified.

In reference with both the DOH and BMA standards, the width of main roadway for the main bridge and highway has been established as 3.25 meters.

The width of roadway in frontage road has been set as 3.00 meters, that of bus-stops as 2.70 meters.

Superelevation of 4% has been adopted for the interchange section on the Thonburi side, where traffic speed of 80 km/h will be anticipated.

Intersections

At the Bangkok side intersection, signalization will have to be installed at surface level.

After detailed capacity checks on each direction of the intersection, numbers of lane have been determined. They vary from the maximum of 5 lanes for the RAMA VI Bridge traffic to 3 or 4 lanes for other traffic lanes.

On the Thonburi side, the specially designed trumpet-type interchange will accommodate smooth flows of traffic at least in the foreseeable future.

5. Conclusions

Using more accurate quantities data obtained from the Preliminary Design, construction costs have been recalculated including land acquisition and compensation costs. The revised total construction cost has been estimated to 782 million Baht, only 2% below the cost of 800 million Baht calculated in the Feasibility Study.

The total cost has been broken down as follows:

(Unit; Million Baht 1981 Prices)

	Foreign Portion	Local Portion	Sub Total
Construction Cost	252.8	253.0	505.8
Main Bridge	(137.2)	(116.8)	(254.0)
Highway Viaduct	(68.4)	(87.0)	(155.4)
Road Work	(40.8)	(44.2)	(85.0)
Railway Viaduct	(6.4)	(5.0)	(11.4)
Land Acquisition	-	52.0	52.0
Compensation	35.7	71.3	107.0
Contingency	28.2	38.3	66.5
Administrative Cost	25.4	25.2	50.6
Total	342.1	439.8	781.9

The construction costs have increased by 6 million Baht due to design changes on bridge structures and highway intersections. The total cost, however, has decreased by 17 million Baht due to decreases on land acquisition and compensations.

To determine viability of the project, the NPV and IRR have been recalculated using the same benefit value computed in the Feasibility Study. Both of the resulting indices have exceeded the precedent values in the Feasibility Study thus indicating the viability of the study project.

NPV	660 million Baht
IRR	20.6%

Percentages against the total cost for the local currency portion and the foreign currency portion are, respectively, 56% and 44% although the percentages against the construction cost only are devided in half, namely 50% and 50% respectively.

In case the present trend of inflation in Thailand would be prevalent for the next 5 years, the total cost for an assumed construction schedule of 30 months starting from 1983 autumn, would amount to 1,224 million Baht in the case of 15% inflation and 1,331 million Baht in 18% inflation.

It is recommended that the New RAMA VI Bridge Construction Project should be commenced at an earlier possible time.

CHAPTER 1

BACKGROUND TO THE DESIGN STUDY

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BACKGROUND TO THE DESIGN STUDY

In order to cope with the rapidly increasing Middle Ring Road traffic around the Bangkok Metropolitan Area, the Government of Thailand made a request in July 1980 to the Government of Japan to conduct a feasibility study on the RAMA VI Bridge construction project including:

1) A feasibility study on either rehabilitation or new bridge construction in the vicinity of the existing RAMA VI Bridge.

2) A further study on either rehabilitation or new bridge construction for the section of the Chao Phraya River between Nonthaburi and Samuthprakan.

In response to the above request, the Government of Japan decided to cooperate with the Government of Thailand in the carrying out of the said study, and JICA, the official agency responsible for implementing the technical cooperation programs of the Japanese Government, dispatched a fact-finding mission headed by Dr.H. Tada, to Thailand in March 1981 to make a preliminary survey and to formulate the scope of work for the study.

An agreement was set forth in accordance with the results of the survey and a feasibility study for the construction of a new bridge including its appropriate approaches and connecting roads was to be conducted in Bangkok during the eleven months period starting May 1981 by the study team organized by JICA.

The study of the RAMA VI Bridge consists of the two phases: Phase I has been conducted as the Feasibility Study during the first three months period of the study and the Interim Report (Draft Final Report of the Feasibility Study Report for Phase I) has been submitted to PWD on September 4, 1981 and Phase II as the Preliminary Design of the new proposed bridge has been carried out for the subsequent three months and the Draft Final Report of the Preliminary Design for Phase I has been submitted on December 3, 1981 to PWD and a comment on the report was issued by PWD at the end of January 1982.

This report comprises the Final Report of the Preliminary Design of the RAMA VI Bridge which is required to be submitted to PWD in the seven months period after the commencement of the study.

CHAPTER 2
THE DESIGN STUDY

CHAPTER 2
THE DESIGN STUDY

2-1 OBJECTIVES

The main objective of the engineering studies at this stage, is to recommend the most appropriate design and construction methods to be adopted and add more accuracy to the outline design of the bridge and roads carried out in the Feasibility Study of Phase I.

The preliminary structural design is mainly concerned with further refinement of the structural analysis. The work has embodied confirmation of the span arrangements and determination of the main dimension of superstructures and substructures.

The calculations of the analysis have mostly regarded the basic stresses in the structures both during construction and on completion.

The road design has embodied confirmation of more detailed alignment including necessary clearances at control points and also more detailed design for earth work, pavement, drainage as well as traffic control schemes and layouts of the intersections and the interchanges.

The comments received from PWD after the submission of the Draft Final Report of the Feasibility Study have been throughly studied and several alterations and improvements for the design have been carried out.

After designing the most desirable configurations for the bridge and roads, a refined construction cost has been recalculated and a revised economic evaluation based on that construction cost has been produced.

In accordance with the clause of the agreement between the Thai Government and the JICA mission, additional researches have been conducted as regards the durabilities and repairing feasibilities of the existing Krung Thon and Krung Thep Bridges, and the report has been prepared to be submitted in an attached volume to this report.

2-2 SCOPE OF WORK AND METHODOLOGY

2-2-1 Scope of the study in Phase II

The scope of the study in the Preliminary Design stage comprises surveying and engineering design and also includes a refinement of the economic studies conducted in Phase I. Additional budgetal and financial study and implementation program have been added to the present study in Phase II.

(1) Surveys

Surveying includes a control points survey, a hydrographic survey and a soil survey.

1) Control points survey

Control points survey has been conducted based on the aerial photographs provided by PWD. The photos have been controlled by the ground survey and prepared for further uses in alignment design and right-of-way plans. The ground survey has been carried out in order to increase the accuracy of the 1/1500 scale mosaic maps.

2) Hydrographic survey

An echo sounding survey has been executed in order to identify the possible deep scours or obstructions which will affect the design of the bridge.

3) Soil survey

A survey of surface and sub-surface soil conditions which is essential for the design of foundation of the new bridge and approach viaducts, embankment and subbase has been performed. A total of four borings, two in the river and two on-shore has been executed and results of laboratory tests have been obtained.

(2) Engineering Studies

In the engineering design at this stage, a more detailed refinement as regards structural stresses and logical analysis from an engineering point of view have been described for designing of road sections such as viaducts, intersections and interchanges.

Engineering studies comprise a description of design standard and road alignment design and a structural analysis for the main bridge and viaducts, a railway bridge and minor bridges on klongs.

A revised cost estimate of the whole construction has been prepared for construction of the bridge and roads as well as land acqusitions and compensations.

(3) Rehabilitation studies of the Krung Thon and Krung Thep bridges

General survey concerning degrees of deterioration of the two bridges and structural analysis has been conducted and technical recommendations have been submitted in a separate volume attached to this report.

2-2-2 Methodology

The procedures of the Phase II study have been described in schematic flow chart in Fig. 2-1.

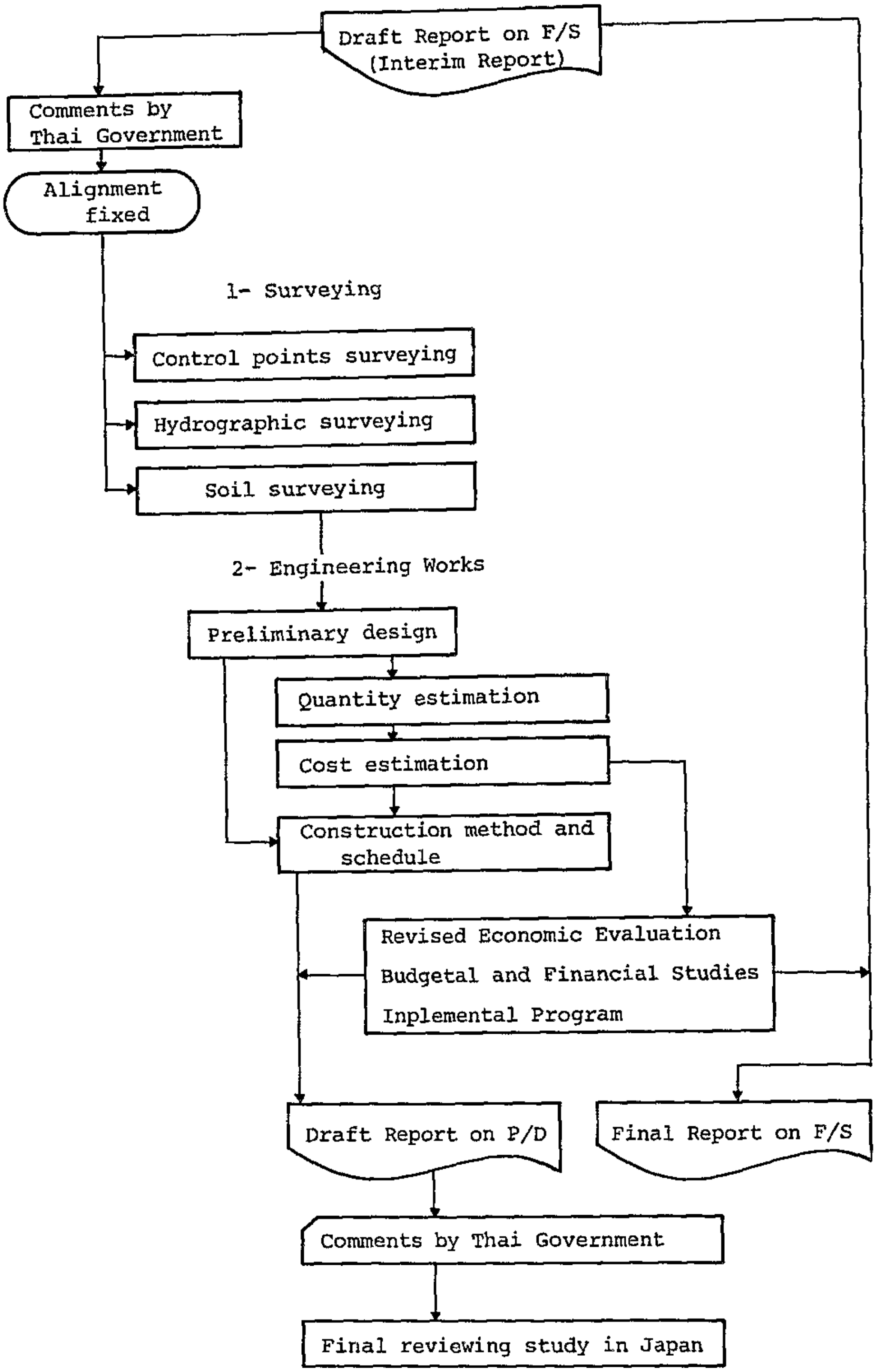

FIG. 2–1 WORK FLOW FOR PHASE II

CHAPTER 3

SURVEYING AND INVESTIGATION

CHAPTER 3
SURVEYING AND INVESTIGATION

3–1 CONTROL POINTS SURVEYING

3-1-1 Control points surveying

In this Preliminary Design stage of Phase II, the study has been scheduled to use 1/1000 scale mosaic maps produced from 1/6600 scale aerial photographs.

In order to produce the mosaic maps, a traverse survey has been carried out by a local surveyor establishing several control points at the site. Field checking for obstruction structures has also been conducted to provide necessary data for route selection.

(1) Control Points

For each control point an iron pipe of approximately 1 meter long has been hammered into the earth and a steel rivet with concrete top has been buried in the earth. Both locations of R4 and R5, R14 and R15 have been so selected as to be visible to each other in view of their repeated use.

(2) Traverse Surveying

The base coordinate has been determined after consulting with PWD. The point R1 has been set on Thailand Royal Surveyor's 1/50,000 maps and coordinates E = 665,450,000 and N = 1,529,050,000 have been obtained.

The azimuth has been determined at R1 using a compass. Computations have been made upon a closed traverse connecting all of 15 control points.

The lower limit for accuracy in calculation of the coordinates has been set not less than 1/5000 ratio of error of closure.

(3) Field checking

The provided aerial maps have been photographed in

March 1980. Since that time, various buildings have been developed in the survey area and in order to furnish accurate data for route selection, an on-the-site survey of obstructing buildings and steel towers etc, has also been carried out.

(4) Others

Cross levelings have been made for each of five different stations in the width of 50 meters each for both right and left sides of the center line.

Using approximately 1/1,620 scale photographs, field checking has been made in order to locate newly built structures not shown on the photos.

3-1-2 Comments

For the aerial photographs of scale 1/6600 which have been provided by PWD for this study, no detailed data of photographing conditions such as flight height, focal distance and bubble tube have been provided. It is recommended that more detailed and accurate maps should be produced for the detail design of this project.

Two different methods can be conceived : one is photographing survey including deviation adjustments and the other is ground survey using plan tables or transit and staffs.

Although a plan table survey takes more time, the most accurate maps can be produced.

Since the project area is located amidst a building-congested urban area, it is recommeded that highly accurate maps made by the plane survey method should be utilized in the detail design stage of this study.

3-2 HYDROGRAPHIC SURVEYING

In order to obtain accurate data and information for the Preliminary Design, the following hydrographic surveys and investigations were performed.

- River bed survey in Chao Phraya River
- Investigations for water levels and velocity of stream current of Chao Phraya River

3-2-1 River bed survey in Chao Phraya River

To identify the positions of any possible deep scour holes or possible obstructions which are to be avoided, three-line river bed surveys were undertaken across the Chao Phraya river with echo sounding equipment.

As a result of the survey, the following facts have been confirmed :

1) No obstructions or deep scour hole have been discovered in the river bed in the vicinity of the adopted horizontal alignment (route).

2) The river bed survey data show that effect of the curved section of the river has not affected the river bed.

3) At the deepest point, the elevation of river bed is 14.5 meters below MSL.

4) At the center line of the river having 200 meter width, a depth of more than 10 meter below MSL has been observed. The results of the survey are shown in Fig. 3-1 to Fig. 3-4.

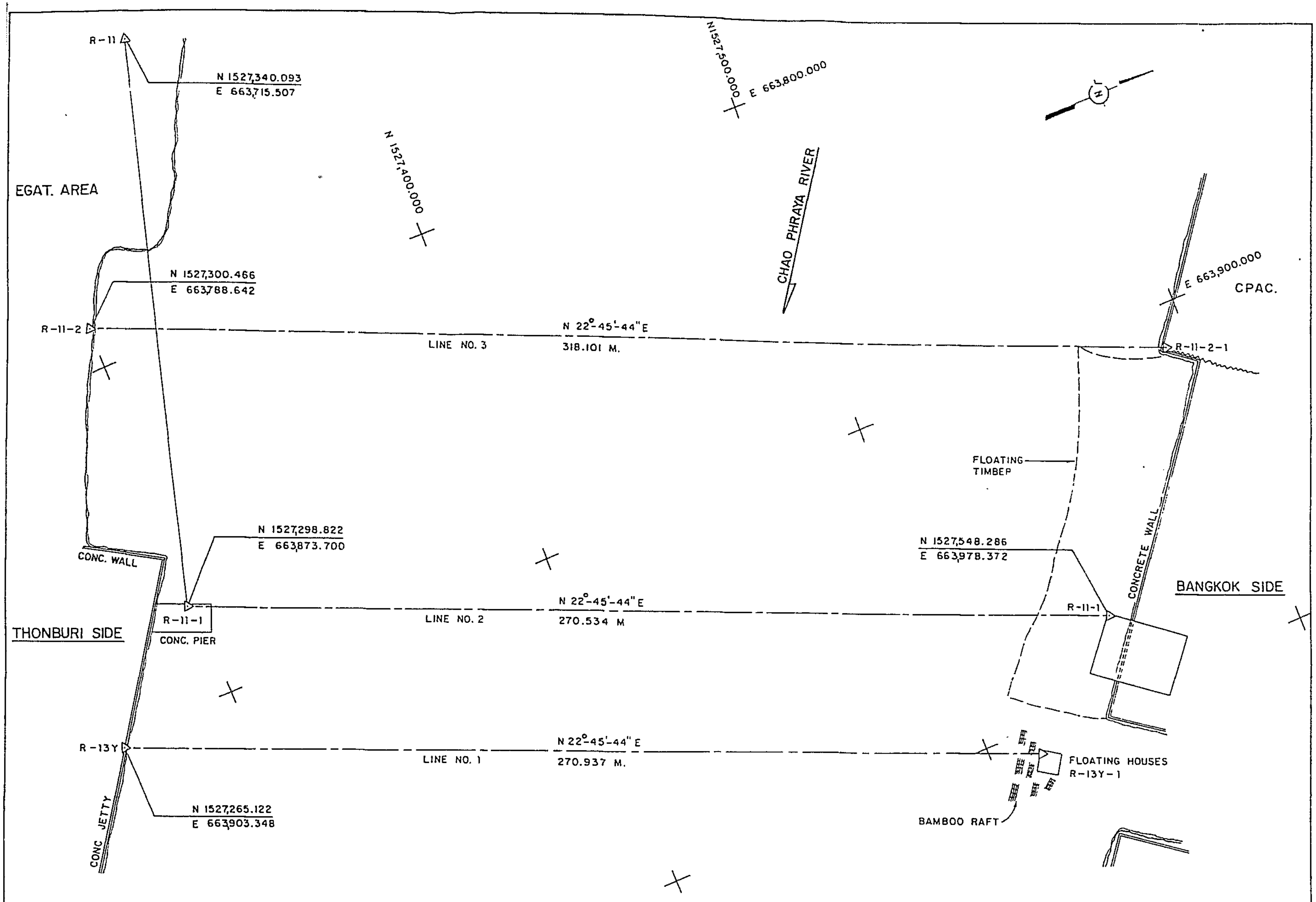

FIG. 3-1 LOCATION OF RIVER BED SURVEY

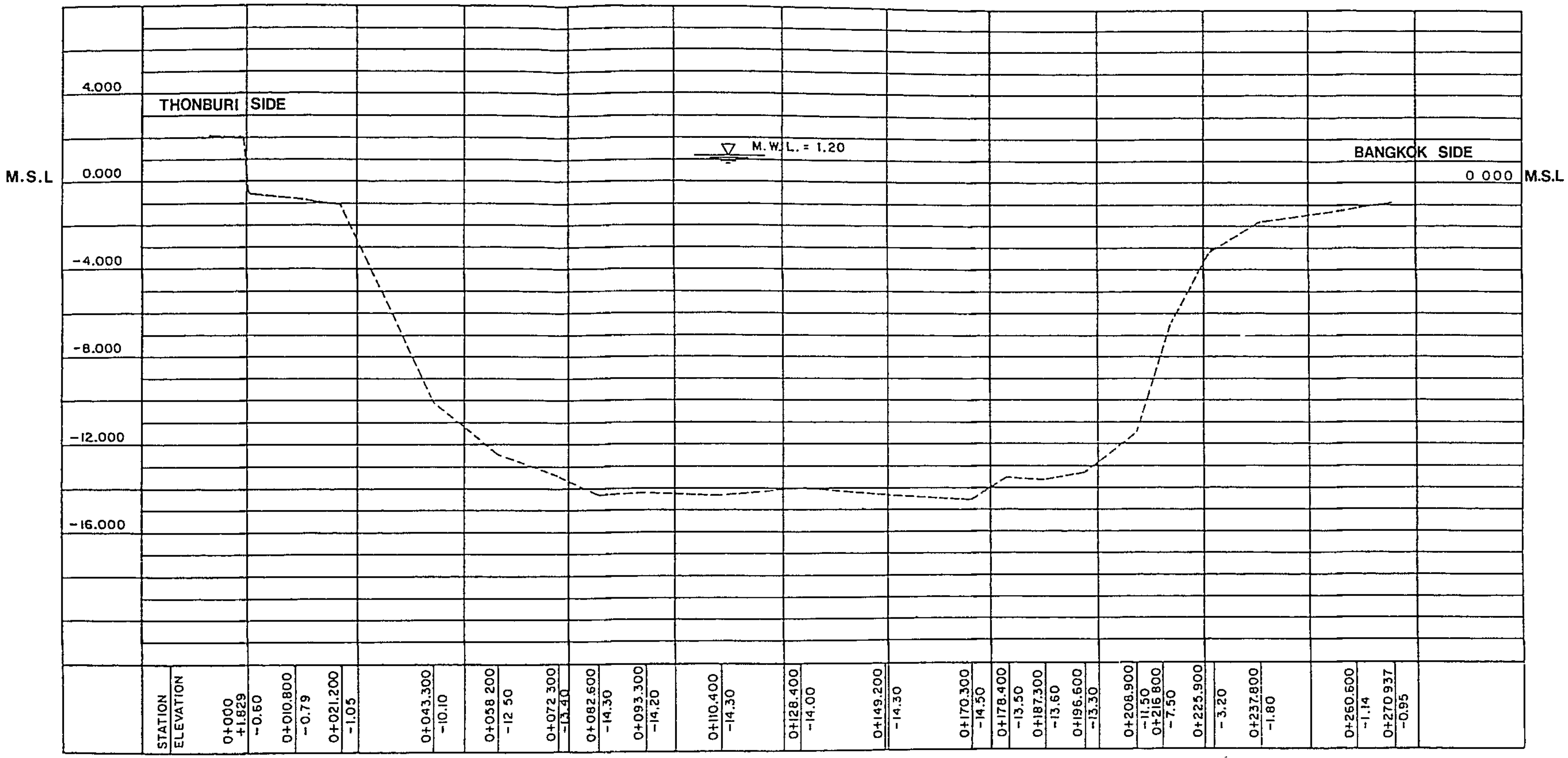

FIG. 3–2 LINE NO. 1

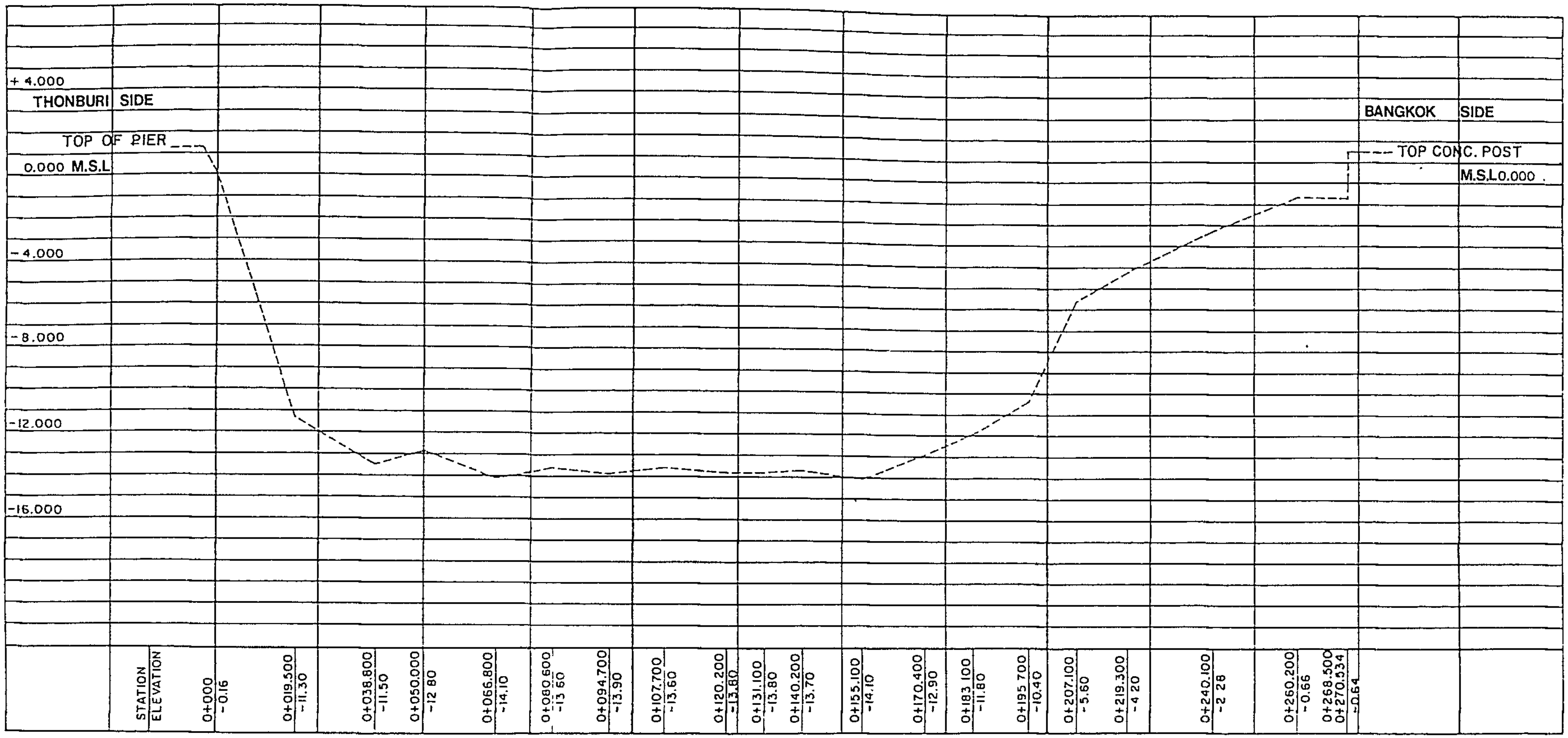

FIG. 3–3 LINE NO. 2

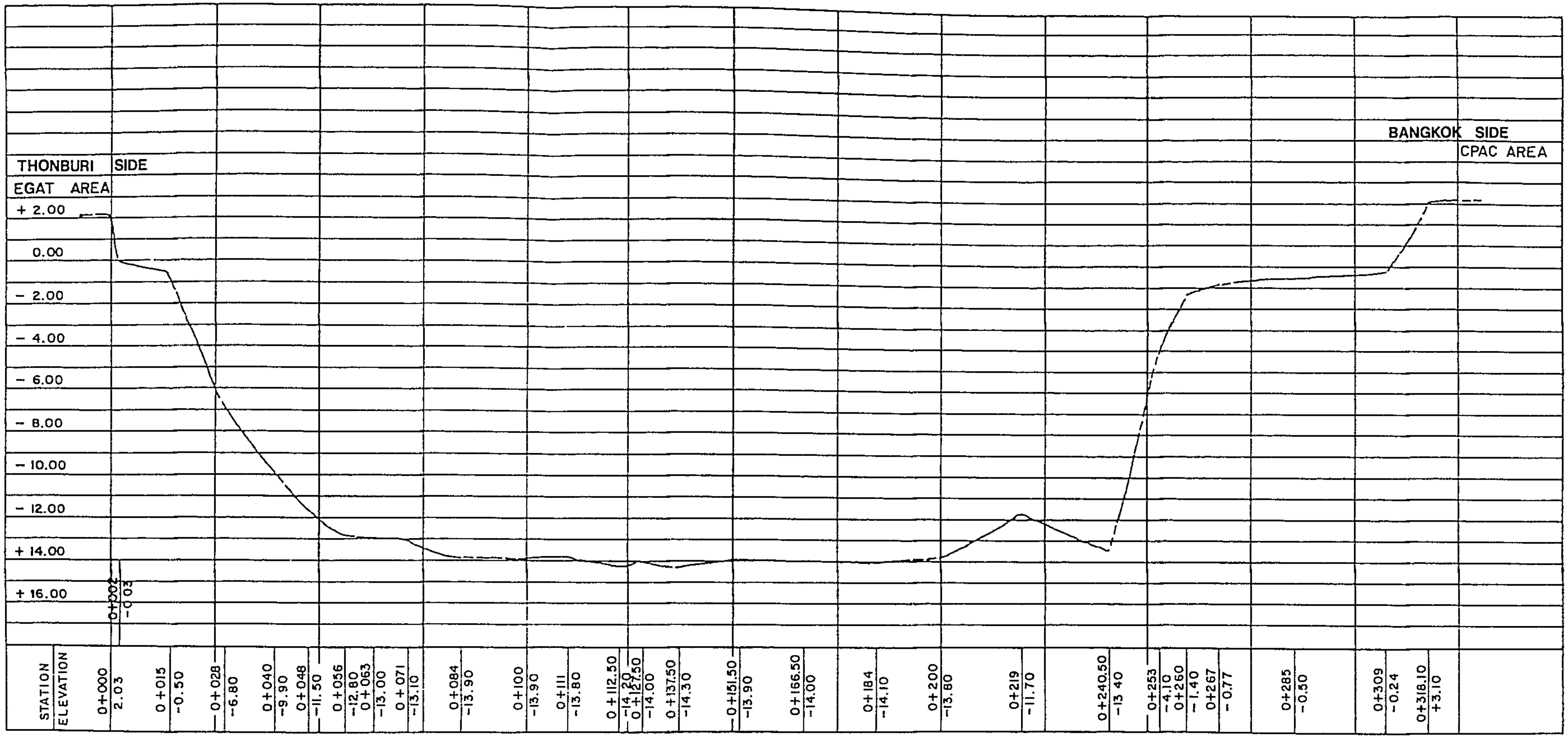

FIG. 3–4 LINE NO. 3

3-2-2 Investigations for water levels and velocity of stream current in Chao Phraya River

For the Preliminary Design certain hydrological and hydraulic data were collected. These include an assessment of :

- high water level
- maximum velocity of stream current

In these matters, information and data can be obtained at the following agencies ;

- Public Works Department, Ministry of Interior (PWD)
- Royal Irrigation Department (RID)
- Harbours Department, Ministry of Communications
- Hydrographic Department, Royal Thai Navy
- Metrological Department, Ministry of Communications
- Asian Institute of Technology (AIT)
- Electricity Generating Authority of Thailand (EGAT)
- Metropolitan Water Works Authority (MWWA)
- Port Authority of Thailand

As the result of the investigations, following criteria were established for the Preliminary Design of the New RAMA VI Bridge.

1) High water level

The data of the existing RAMA VI Bridge showed 1.829 meters above MSL as HWL. But for 20 years, the width of the river was decreased by construction of riverbank structures and embankment. According to this effect, it is considered that high water level at new alignment will rise to approximately 2.00 meters above MSL. Also we can find mention of equivalent values in many design reports, as follows:

- Detailed design documents of Charansanitwong Road (DOH).

- Preliminary Design Report of the Nonthaburi and Pathumthani Bridges Construction Project. (Fig. 5-18).

So in the Preliminary Design, the value of 2.00 meters above MSL is adopted as high water level.

2) Maximum velocity of stream current

In the last 10 years, the detailed design of 4 bridges crossing the Chao Phraya River has been carried out.

In these reports, the value of 3.00 meters/sec was adopted as the maximum velocity of stream current for bridge design. The location of the New RAMA VI Bridge is between the New Nonthaburi Bridge and the Sathorn Bridge.

Considering those data, the value of 3.00 meters/sec is adopted as the maximum velocity of stream current for the foundation design of the main bridge. The location of gauging stations on Chao Phraya River and the longitudinal section of maximum flood level between the Memorial Bridge and Bang Sai are shown in Fig. 3-5 and Fig. 3-6.

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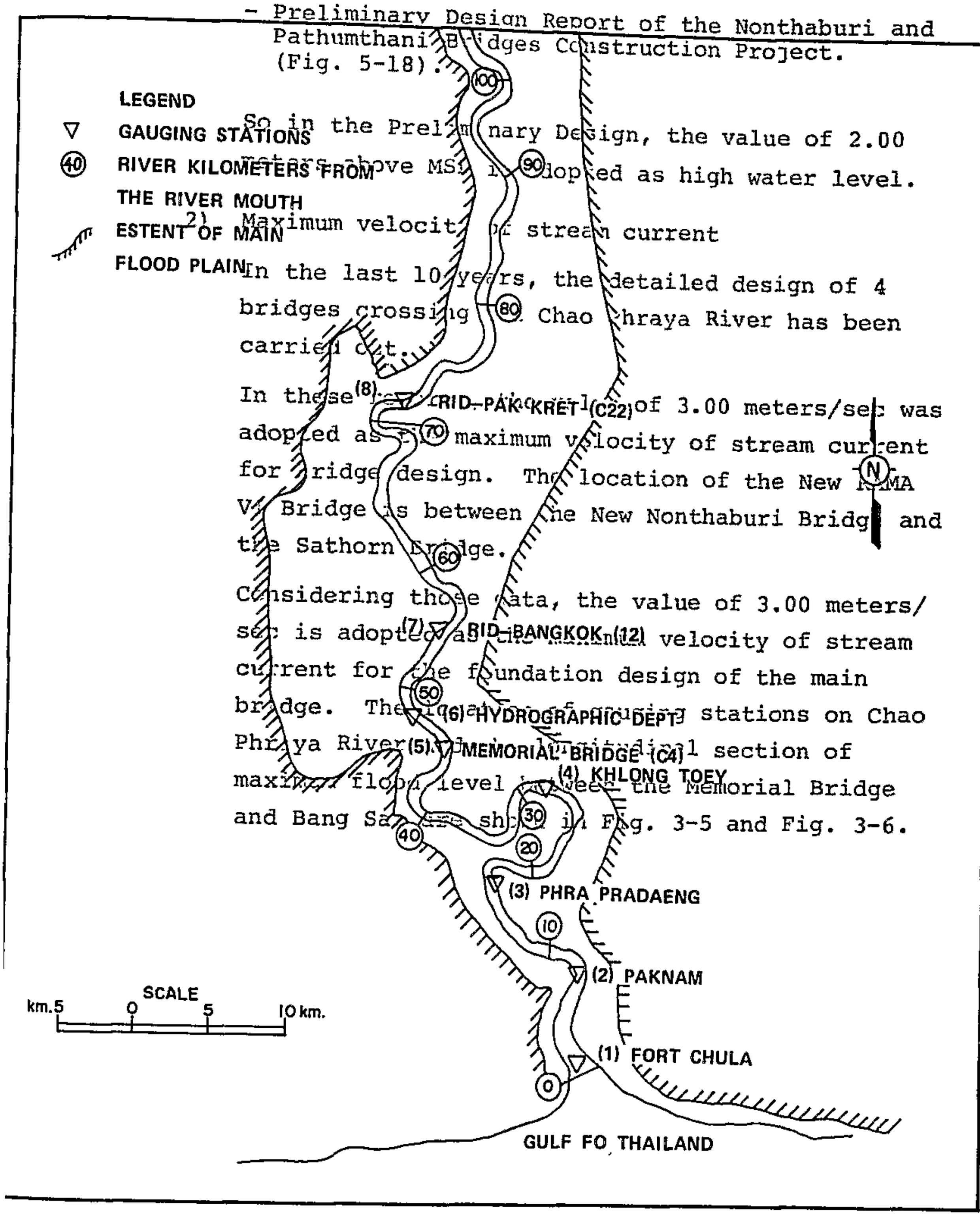

FIG. 3–5 LOCATION OF GAUGING STATIONS ON CHAO PHRAYA RIVER

3-2-2 Investigations for water levels and velocity of stream current in Chao Phraya River

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- maximum velocity of stream current

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- Harbours Department, Ministry of Communications
- Hydrographic Department, Royal Thai Navy
- Meteorological Department, Ministry of Communications
- Asian Institute of Technology (AIT)
- Electricity Generating Authority of Thailand (EGAT)
- Metropolitan Water Works Authority (MWWA)
- Port Authority of Thailand

As the result of the investigations, following criteria were established for the Preliminary Design of the New RAMA VI Bridge.

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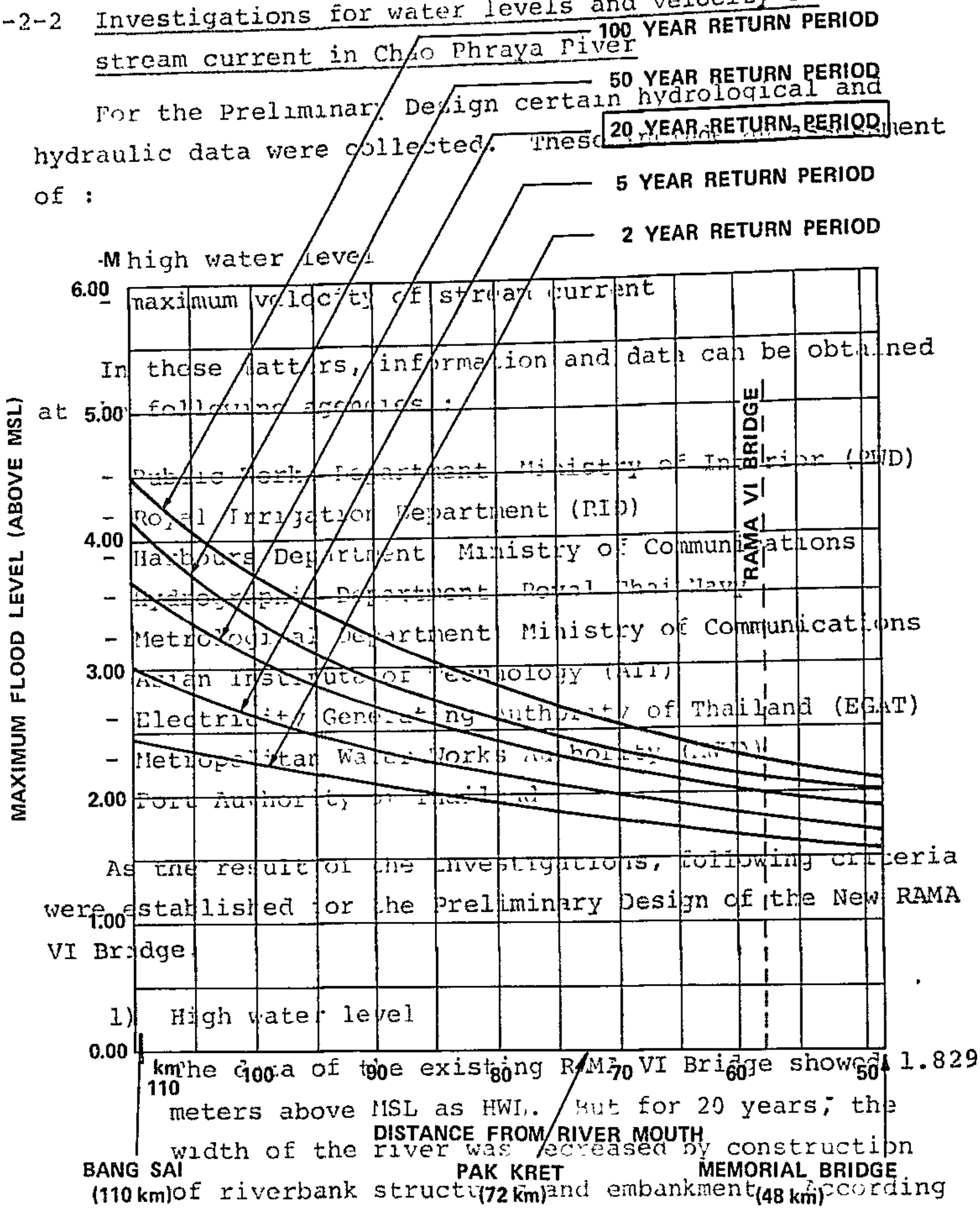


FIG. 3-6 LONGITUDINAL SECTION OF MAXIMUM FLOOD LEVELS BETWEEN MEMORIAL BRIDGE AND BANG SAI

of equivalent values in many design reports, as follows:

- Detailed design documents of Charansanitwong Road (DOH).

3-3 SOIL SURVEYING

3-3-1 Background

A subsurface investigation was carried out to obtain data necessary for the Preliminary Design of the new RAMA VI Bridge. The investigation which consisted of borings, hand-auger borings, test pits, standard penetration tests and laboratory soil tests was carried out at onshore and offshore locations along the proposed alignment. It was found that stratification below the proposed bridge consisted of Weathered Bangkok Clay (Fill), Soft Bangkok Clay (Ac), Stiff Bangkok Clay (Dc) and Dense Sand (Dsu, Dsl) in descending order. The locations of soil survey are shown in Fig. 3-7.

The results of the subsurface investigation are described in subsection 3-3-2. An engineering study of various foundation is described in subsection 3-3-3. The quantity of field exploration and laboratory tests are tabulated in Table 3-2.

3-3-2 Results of subsurface investigation

The results of borings, standard penetration tests and laboratory soil tests are shown in Fig. 3-8, which provides a summary of subsoil characteristics and classifications. Detailed drilling logs are shown in Figs. 3-14 (1) to 3-14 (4). A soil profile of the Chao Phraya River is shown in Fig. 3-15.

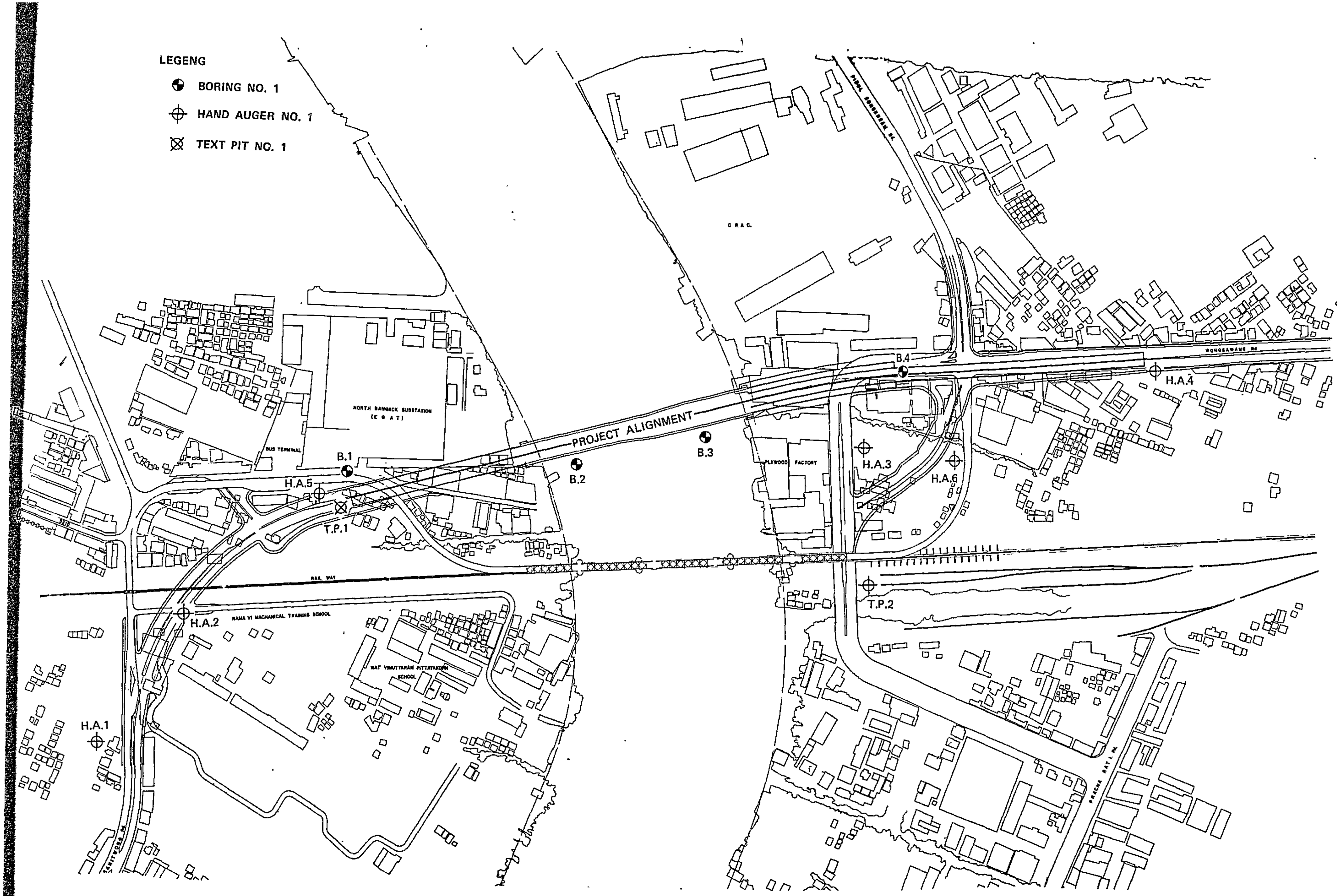


FIG. 3–7 LOCATION MAP OF SOIL SURVEY

Table 3-1 Quantity of Field Exploration, Field and Laboratory Tests

	Type of Investigation	Deep Boring				Soil Investigation Hand - Auger						Test Pits		Total
	Boring No.	B1	B2	B3	B4	HA1	HA2	HA3	HA4	HA5	HA6	TP1	TP2	-
	Location	Tbri	Off-Shore	Off-Shore	BKK	Tbri	Tbri	BKK	BKK	Tbri	BKK	Tbri	BKK	-
	Elevation (M.S.L.) (m)	+2.33	-12.77	-10.59	+2.02	+1.39	+0.93	+1.23	+1.49	+1.86	+1.56	-		-
	Ground Water Table (EL m)	+1.33	+0.73	+0.51	+1.22	+1.19	+0.93	+1.13	+1.19	+1.46	+0.86	-		-
	Depth, m	47	45	45	44	3	3	2.5	3	3	3	2	1.5	202
Sampling & Firld Tests	Undisturbed Sampling	11	4	5	13	-	-	-	-	-	-	-	-	33
	Standard Penetration Test	36	45	44	35	-	-	-	-	-	-	-	-	160
Laboratory Tests	Natural Water Content	11	-	2	13	-	-	-	-	-	-	2	2	30
	Liquid & Plastic Limits	11	-	2	13	-	-	-	-	-	-	2	2	30
	Sieve Analysis	11	-	2	13	-	-	-	-	-	-	2	2	30
	Hydrometer Analysis	9	-	2	13	-	-	-	-	-	-	2	2	28
	Specific Gravity	9	-	2	13	-	-	-	-	-	-	2	2	28
	Unit Weight	9	-	-	13	-	-	-	-	-	-	-	-	22
	Unconfined Compression	9	-	-	12	-	-	-	-	-	-	-	-	21
	Consolidation	7	-	-	5	-	-	-	-	-	-	-	-	12
	Compaction	-	-	-	-	-	-	-	-	-	-	2	2	4
	C B R	-	-	-	-	-	-	-	-	-	-	2	2	4

(NOTE) Tbri = Thonburi, BKK = Bangkok

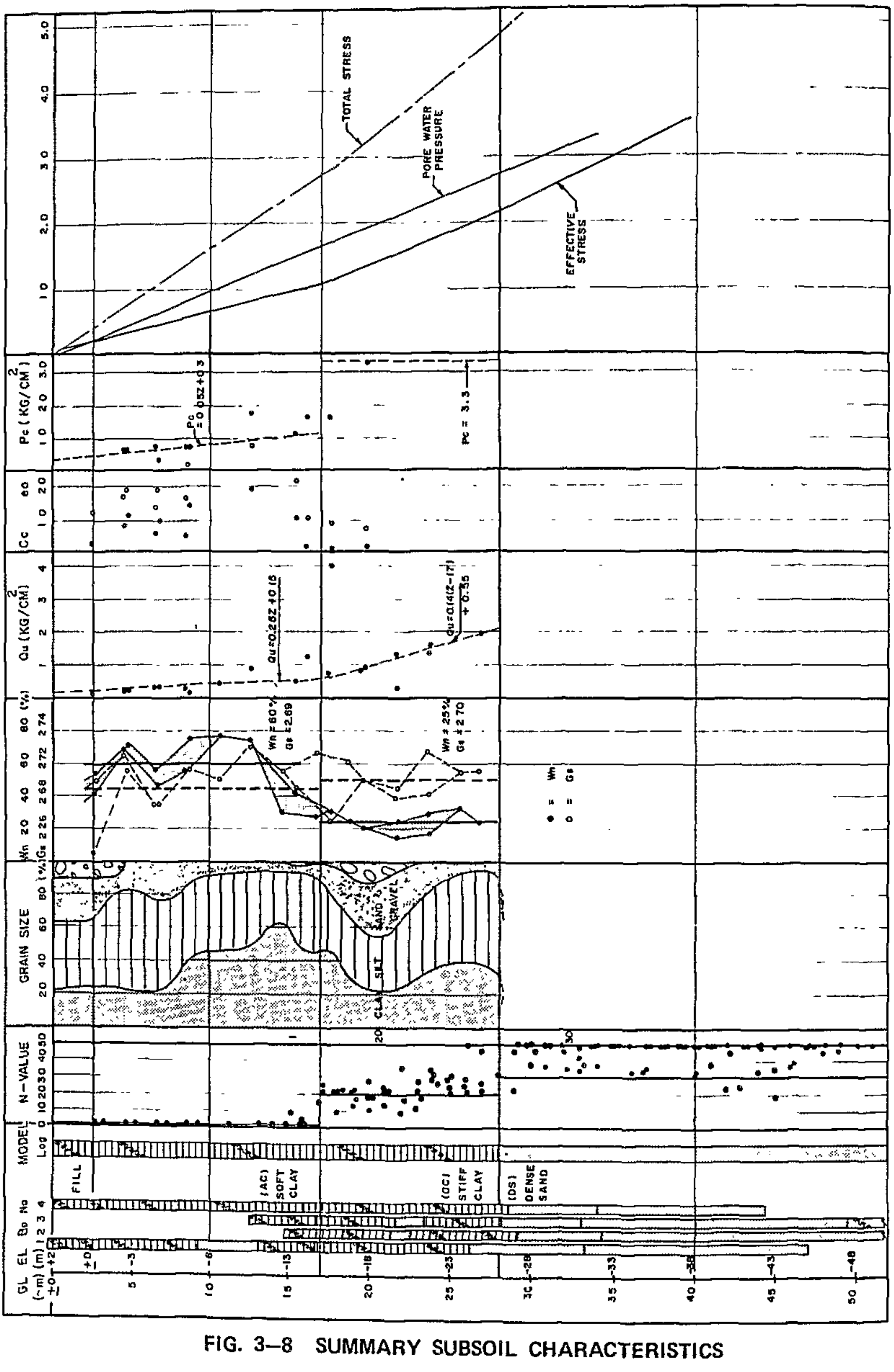


FIG. 3–8 SUMMARY SUBSOIL CHARACTERISTICS

3-3-3 Engineering Study

The foundation engineering study for the proposed RAMA VI Bridge addressed questions relating to :

- Pile Foundations
- Embankment Settlement
- Embankment and Concrete Materials

(1) Pile Fundations

Pile Foundations proposed for the bridge are as follows.

Structure	Pile Type
1) Pier on River	Bored Piles
2) Pier and abutment on land ...	Concrete Driven Piles

Engineering of pile foundation is described in regard to :

a) Vertical allowable bearing loads of pile foundations

b) Elastic settlement of pile foundation

c) Consolidation settlement of stiff clay below the driven concrete pile tip

The above engineering studies were made in reference to guidelines given in "Specifications for Highway Bridges and Substructures" May 1980, Japan Road Association.

a) Vertical Allowable Bearing Loads of Pile Foundations

Vertical allowable bearing loads for river-sited bored piles of 1500 mm diameter and for bank-sited driven concrete piles are plotted in Fig. 3-9 and Fig. 3-10 respectively.

In the latter figure, bearing loads for three different sizes of piles are given, namely

35 cm x 35 cm, 40 cm x 40 cm, and 52.5 cm x 52.5 cm. It is anticipated that these concrete piles will not be able to penetrate more than 2 to 3 meters into the dense sand layer due to the very dense layer below EL-26 meters.

b) Elasticity Settlement of Pile Foundations

Calculations regarding the elasticity settlement of the above mentioned concrete piles are presented below on the basis of 75% of the bearing load transmitted to the pile tip.

Table 3-2 Elasticity Settlement by Pile Size

Pile	Ra (t)	75% Ra (t) V	Layer of Pile Tip	N Value
35 x 35 cm	50	37.5	Dc	20
40 x 40 cm	80	60	Ds	30
52.5 x 52.5 cm	100	75	Ds	30

The results of elasticity settlement calculations are shown in Table 3-4.

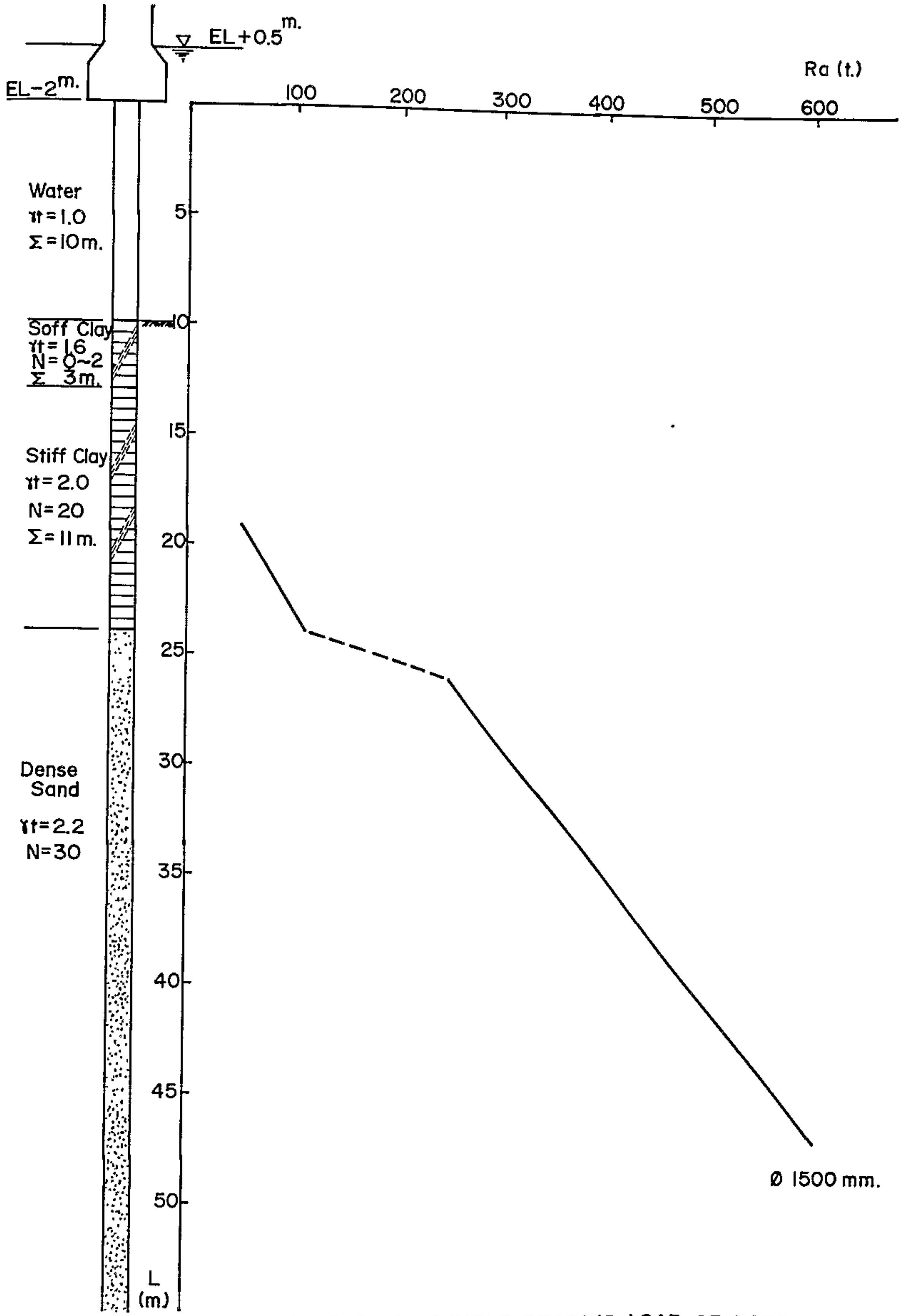


FIG. 3–9 ALLOWABLE BEARING LOAD OF BORED PILE

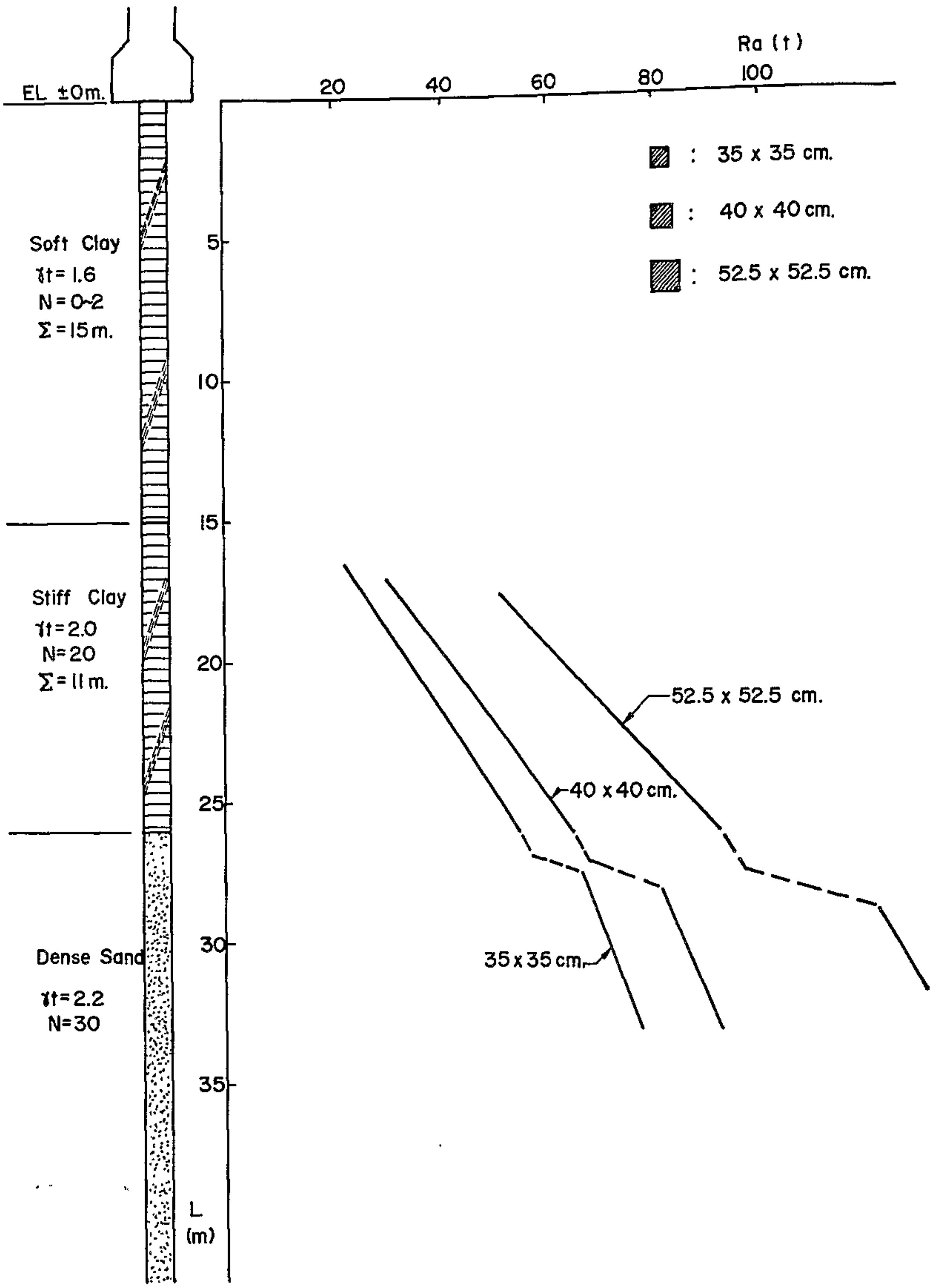


FIG. 3–10 ALLOWABLE BEARING LOAD OF CONCRETE PILES

Table 3-3 Elasticity Settlement

	$\phi=35^{cm} \times 35^{cm}$	$\phi=40^{cm} \times 40^{cm}$	$\phi=52.5^{cm} \times 52.5^{cm}$
A or AV (cm^2)	1225	1600	2756.25
$BV = \sqrt{AV}$	35	40	52.5
$Eo = 28N$ (kg/m^2)	560	840	840
$Kvo = \frac{1}{30} \cdot Eo$ (kg/cm^3)	18.7	28	28
$Kv = 2.8Kvo \cdot BV^{(-3/4)}$ (kg/cm^3)	16.6	22.5	18.4
$\frac{1}{Kv}$ (cm^3/kg)	0.060	0.044	0.054
V (kg)	37,500	60,000	75,000
$\frac{V}{A}$ (kg/cm^2)	30.6	37.5	27.2
$\delta_v = \frac{1}{Kv} \cdot \frac{V}{A}$ (cm)	1.84	1.67	1.48

c) Consolidation Settlement of Stiff Clay Below Driven Concrete Pile Tip

Projected depth of the driven concrete pile tips is the middle of the still clay layer between EL-15 and 26 meters for consolidation settlement. The type, length and load of piles is as follows;

35 x 35 cm,	L = 20 m,	Ra = 22.5 t
40 x 40 cm,	L = 21 m,	Ra = 44.8 t
52.5 x 52.5 cm,	L = 22 m,	Ra = 72.4 t

The pressures of piles on the layer (=P) are calculated below.

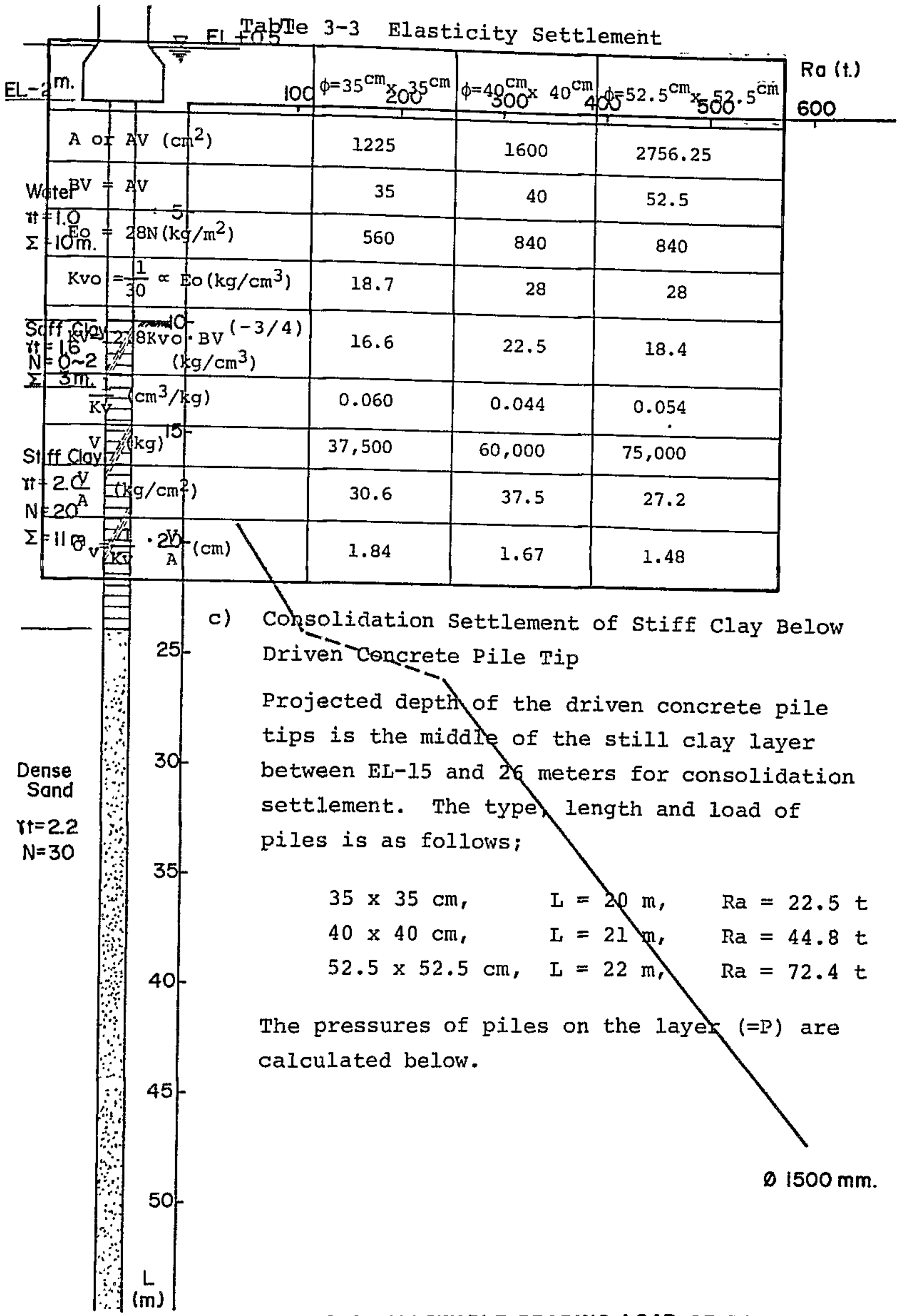


FIG. 3–9 ALLOWABLE BEARING LOAD OF BORED PILE

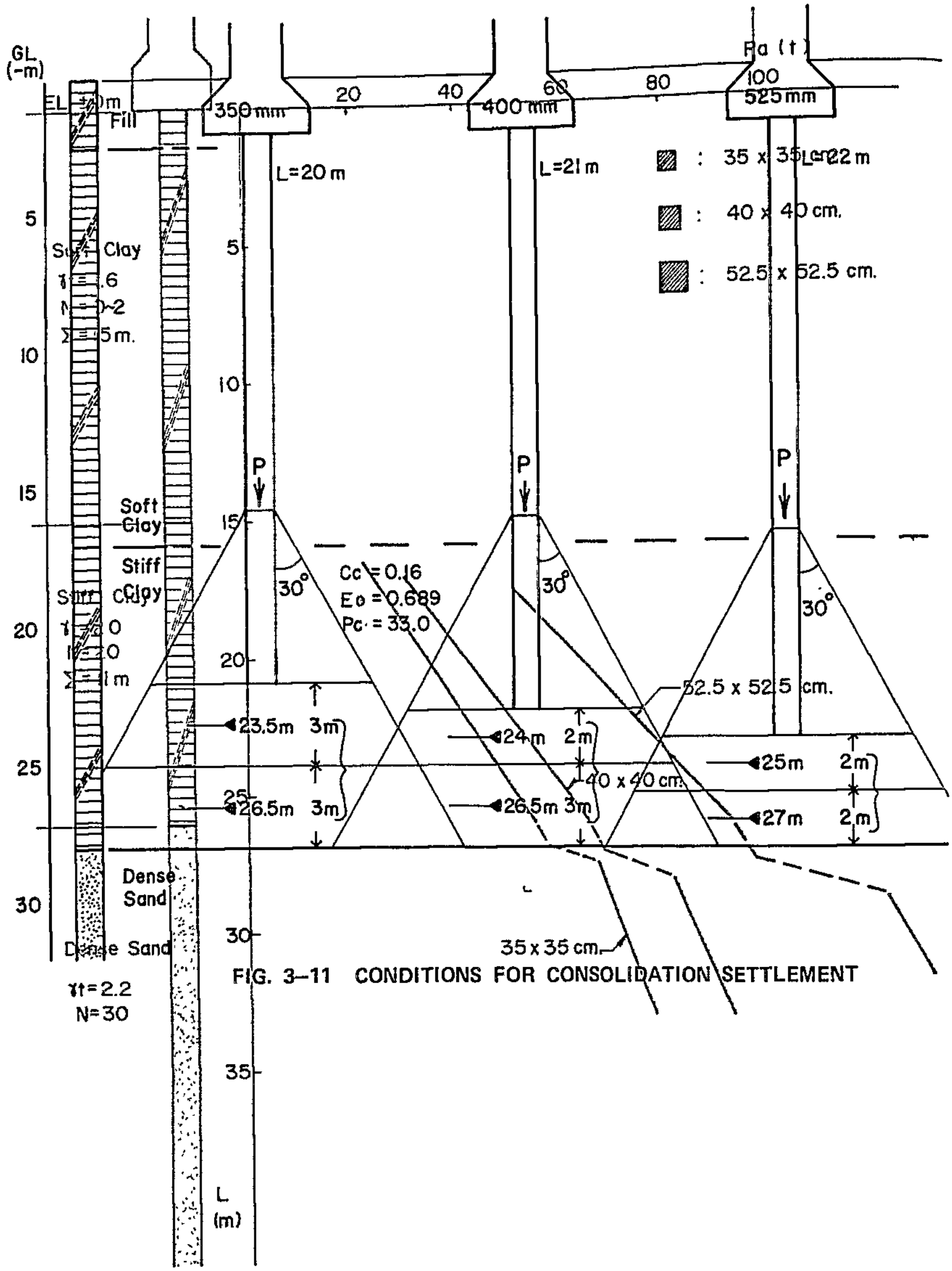


FIG. 3–11 CONDITIONS FOR CONSOLIDATION SETTLEMENT

FIG. 3–10 ALLOWABLE BEARING LOAD OF CONCRETE PILES

Table 3-4 Load for Consolidation Settlement

	75% Ra (t)	NF (t)	W (t)	P (t)	p (t/m^2)	P (kg/cm^2)
350x350 mm L=20m	25.1	31.5	3.68	60.28	492.1	49.21
400x400 mm L=21m	33.6	36.0	5.04	74.64	466.5	46.65
525x525 mm L=22m	54.3	47.25	9.11	110.66	401.5	40.15

Conditions for Consolidation settlement are shown in Fig. 3-11.

Settlement value is calculated in Table 3-6.

Table 3-5 Consolidation Settlement

		ΔH (cm)	Cc	e_o	$\frac{Cc}{1+e}$	σoz (t/m^2)	$\sigma_1 z+\Delta\sigma$ (t/m^2)	A*	$\frac{Cc\Delta H}{1+e^o}$	s (cm)	Σs (cm)
35x35 cm	1	300	0.16	0.689	0.095	33.0	39.2	0.075	28.5	2.1	
L=20m	2	300	0.16	0.689	0.095	33.0	32.0	-	-	0	2.1
40x40 cm	1	200	0.16	0.689	0.095	33.0	39.6	0.079	19.0	1.5	
L=21m	2	300	0.16	0.689	0.095	33.0	32.9	-	-	0	1.5
52.5x 52.5 cm	1	200	0.16	0.689	0.095	33.0	32.6	-	-	0	
L=22m	2	200	0.16	0.689	0.095	33.0	30.1	-	-	0	0

* $\log_{10}\frac{\sigma_1 z+\Delta\sigma z}{\sigma_0 z}$

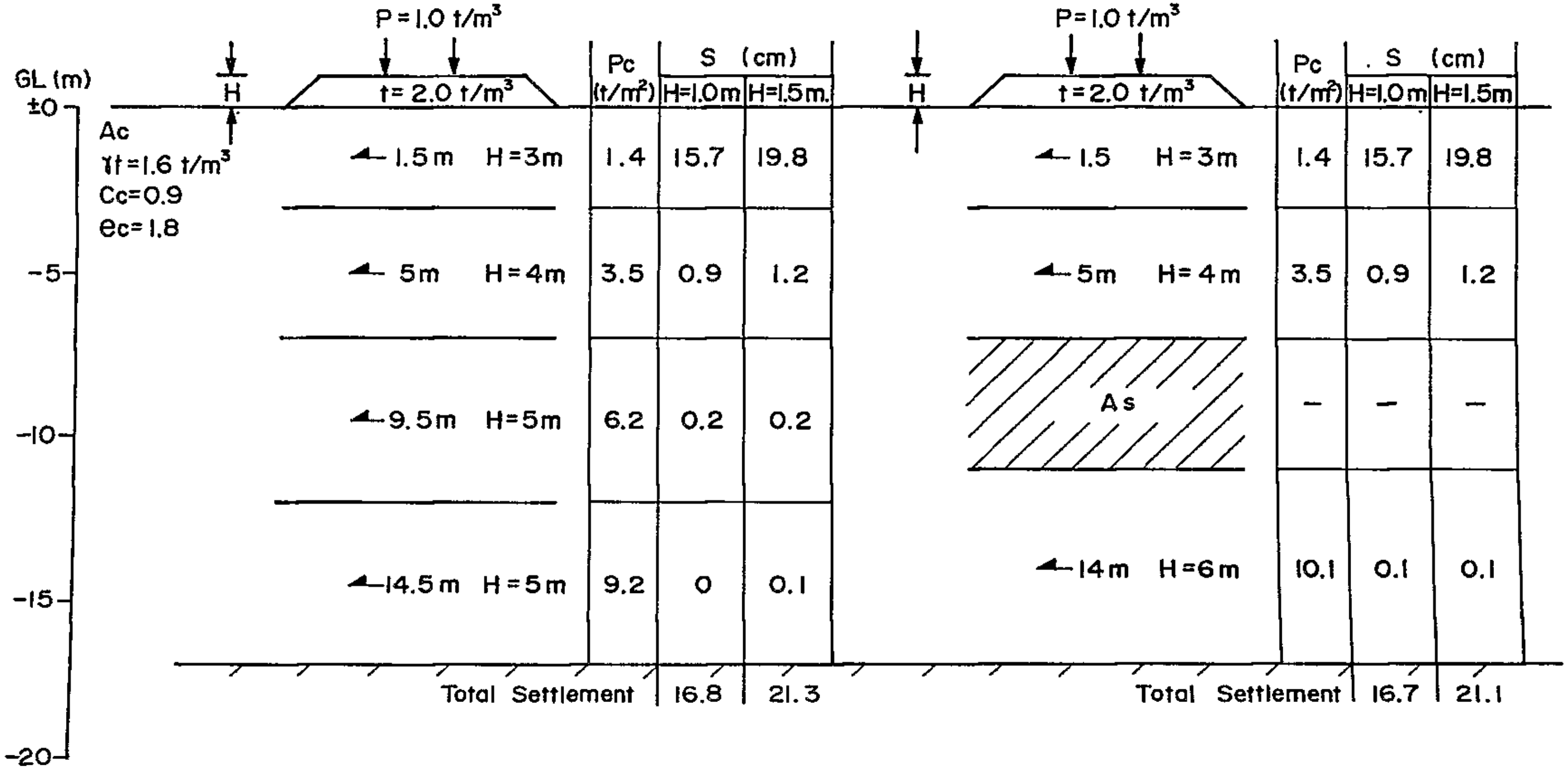


FIG. 3-12 RESULTS OF CONSOLIDATION SETTLEMENT FOR EMBANKMENT

(2) Embankment Settlement

Calculations for embankment settlement were made for 2 cases of 1 m and 1.5 m embankment height. Preconsolidation pressure, (Pc value) of the soft clay is slightly over consolidated pressure according to the results of consolidation tests. The results of consolidation settlement are shown in Fig. 3-12. This value represents only embankment settlement.

(3) Embankment and Concrete Materials

The city of Bangkok is located on an alluvial plain (low land) called the Chao Phraya Delta, which is a product of the Chao Phraya River and its estuaries. As a result, there are neither base/subbase course materials for use in construction nor fine to coarse aggregate for concrete at the project site. Quarry sites exist in the hinterland. Several commercial plants are in operation in Bangkok. The locations of material sources are shown in Fig. 3-13(1) to (3).

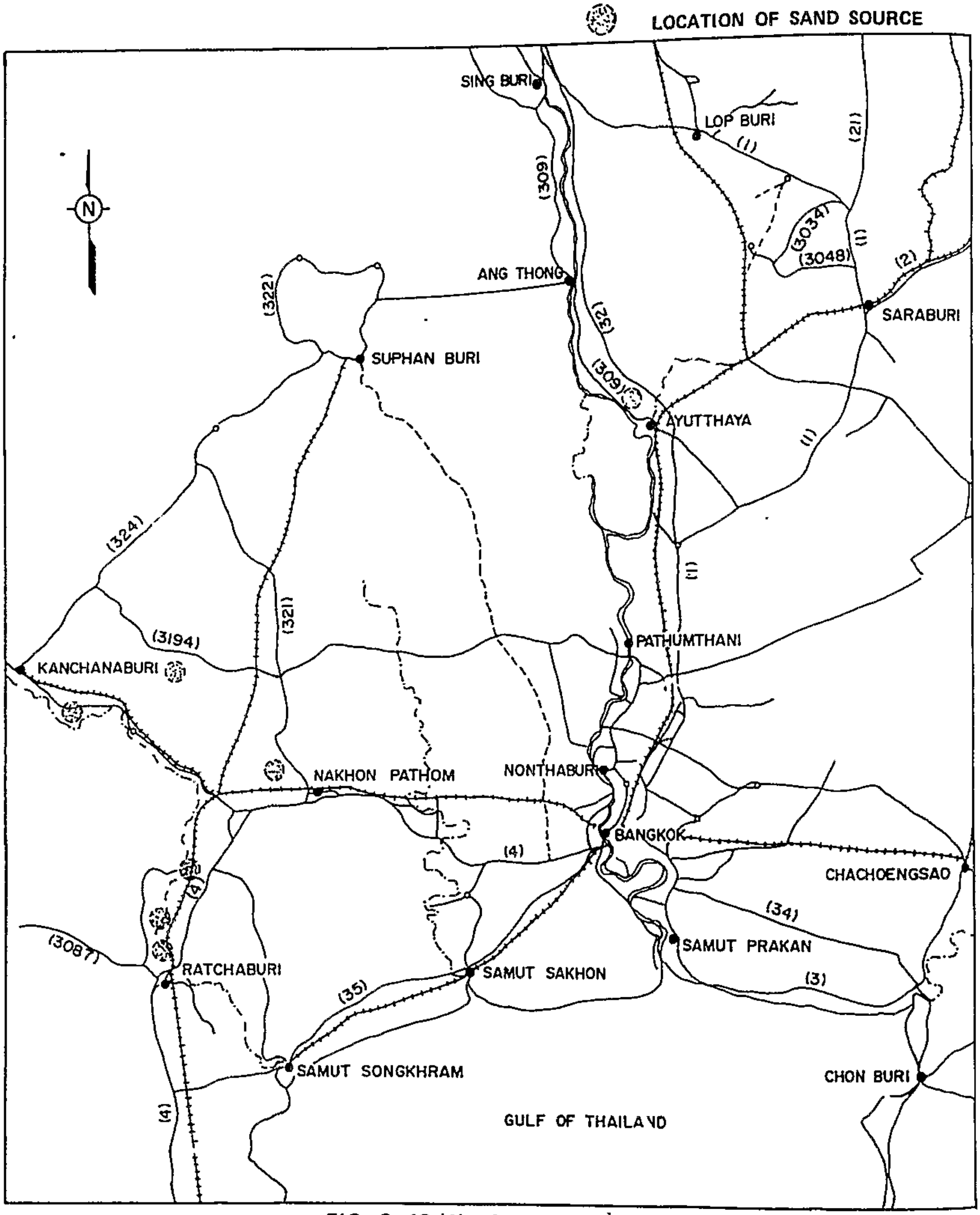


FIG. 3–13 (1) SAND SOURCES

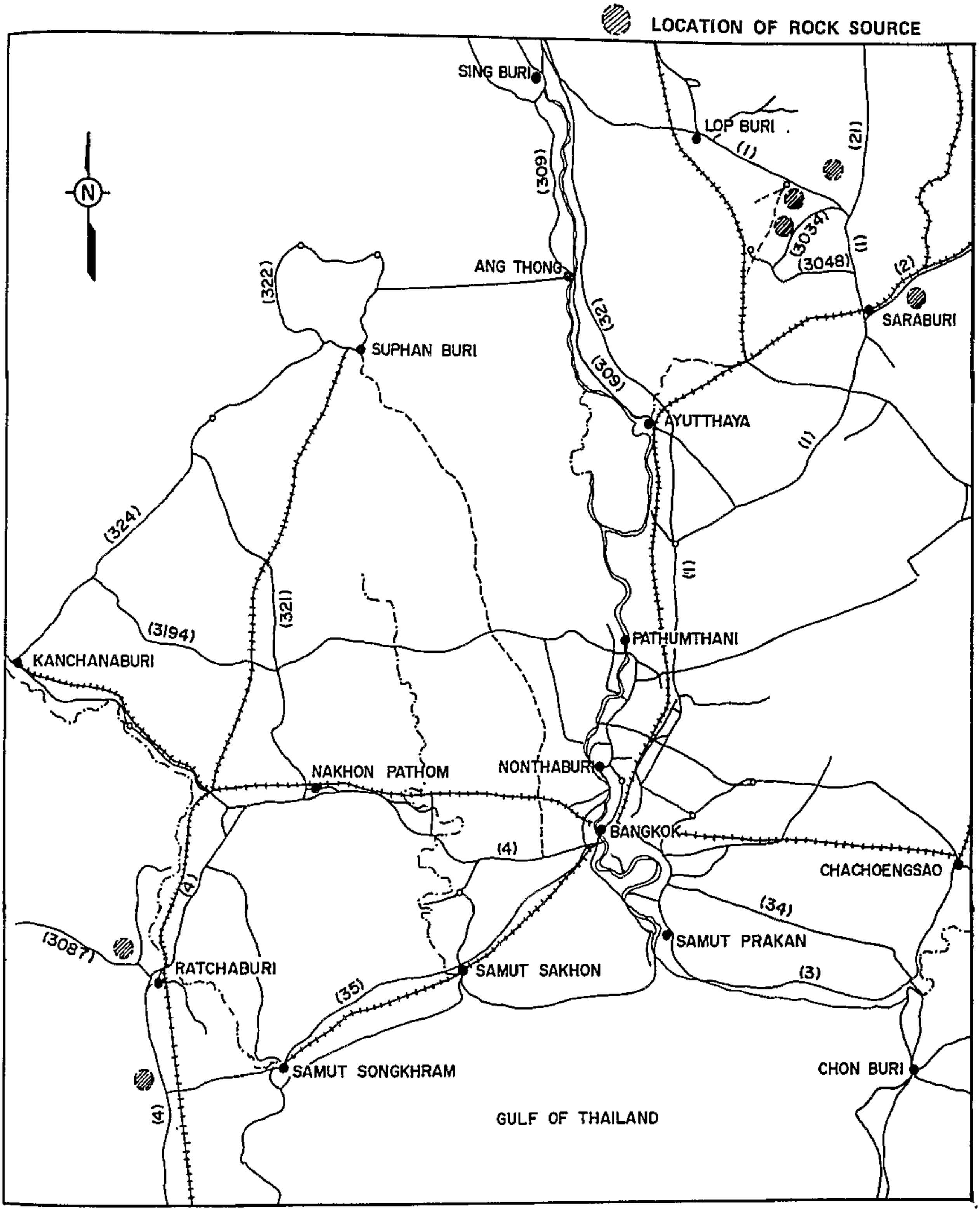


FIG. 3–13 (2) ROCK SOURCES

LOCATION OF LATERITE SOURCE

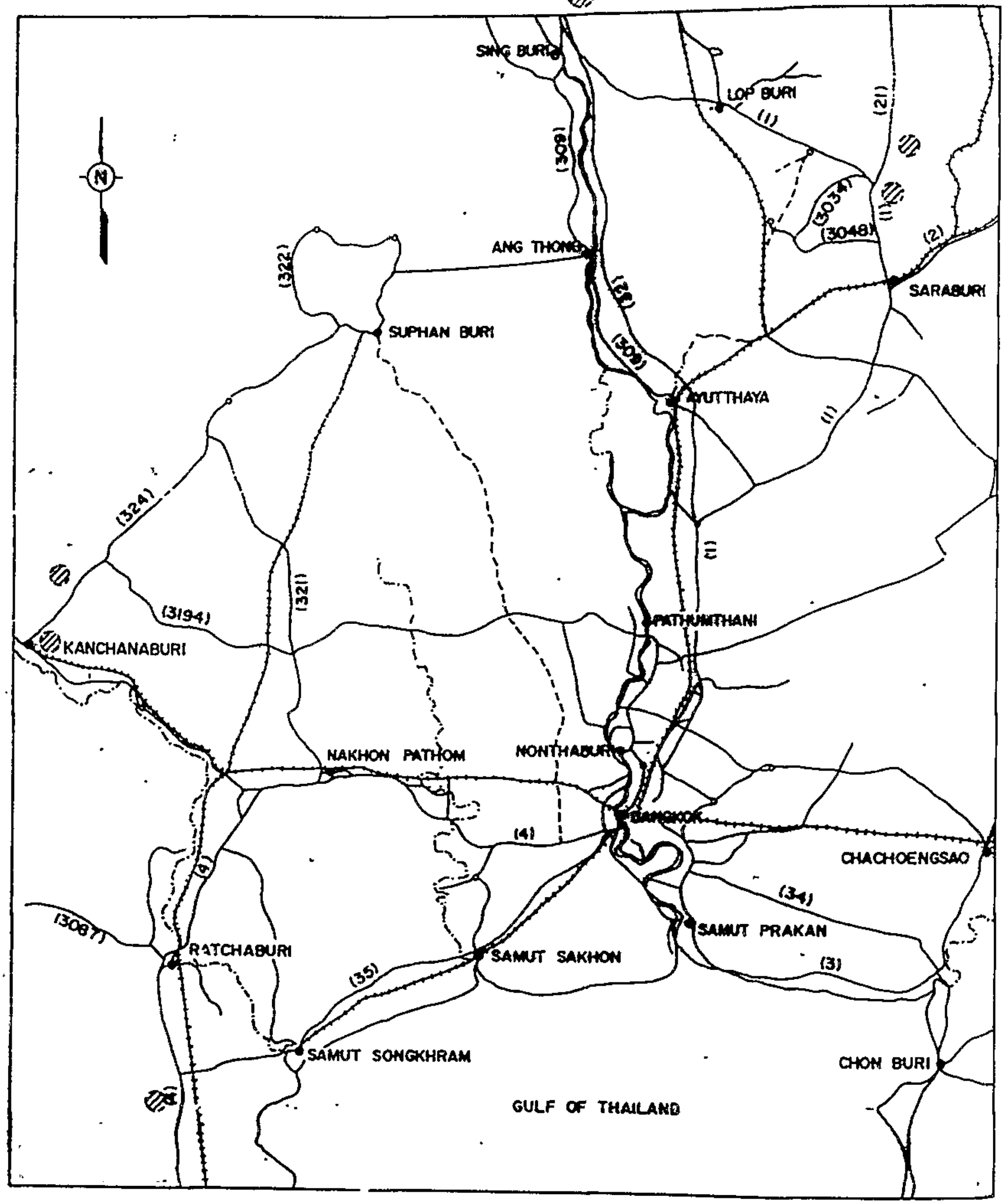


FIG. 3–13 (3) LATERITE SOURCES

DRILLING LOG

Project THE RAMA VI BRIDGE CONSTRUCTION
Hole Number B-1 Elevation EL +2.33
Water Table EL +1.33 m MSL
Type of Drilling WASH
Date 25–28 Aug. 1981
Driller TONGCHAT (KAMIJO)

Casing depth is 15 m from ground

Scale in m	Elevation in m	Depth in m	Thickness in m	Type of Soil	Colour	Relative Density or Consistency	General Remarks
	2.33						
1	1.83	0.50	0.50	fill		medium	Clayey sand with gravel
2	0.33	2.00	1.50	weathered clay	brown grey	medium	with organic matter
3	0.47	2.80	0.80	silty clay		soft	with slicken side trace of organic matter
4–5	3.17	5.50	2.70	silty clay	grey	soft	with slicken side
6–9	7.17	9.50	4.00	silty clay	grey	soft	parting of fine sand
10–12	10.17	12.50	3.00	silty fine sand	grey	loose	with mica fragments and layer of silty clay
13	11.17	13.50	1.00	silty fine sand	grey	medium compact	with mica fragment
14–17	14.67	17.00	3.50	silty clay	grey	soft	layer and parting of fine sand
18–19	16.67	19.00	2.00	silty clay	olive grey	very stiff	with slicken side
	17.17	19.50	0.50	silty clay		very stiff	trace of concretion
20–21, 25/8	19.27	21.60	2.10	silty clay		stiff to very stiff	lateritic soil; trace of fine sand mottled with grey
1.23 m, 23–25	22.47	24.80	3.20	silty clay		very stiff	with slicken side with grey mottled; 24.0 – 24.8 m trace of laterite
	23.27	25.60	0.80	silty clay	grey	very stiff	mottled with brown
26	24.17	26.50	0.90	sandy silt	grey	medium compact	with layer of silty clay
27, 26/8, 2.8 m						medium compact to compact	mottled with yellow
29–31				fine to medium sand	light brown	medium compact to very compact	28.5 – 29.5 m seam of silty fine sand; 30.5 – 31.5 m trace of coarse sand
32–33	31.17	33.50	2.00	fine to coarse sand	light brown	compact	31.5 – 32.5 m fine to coarse sand; 32.5 – 33.5 m medium to coarse sand
34–38	36.17	38.50	5.00	fine to medium sand	grey; light brown; grey	very compact	34.5 – 35.5 m trace of coarse; 35.3 – 37.0 m fine sand
39–40	37.67	40.00	1.50	fine to coarse sand	grey	very compact	trace of gravel
41	39.17	41.50	1.50	fine to medium sand	light brown	compact	trace of fine gravel
42–45	42.67	45.00	3.50	fine to coarse sand	grey	medium compact; compact	layer of silty lay
46	44.17	46.50	1.50	fine to medium sand	grey	very compact	trace of fine travel
28/8, 1.0 m	45.17	47.45	0.55	fine to coarse sand	light brown	very compact	little fine gravel layer of clay

Standard Penetration Test or Core Recovery

Sampling Depth in m	Sample No.	N Value (30 cm)	Blows per each 15 cm (15 cm)	(15 cm)	(15 cm)
2.00					
3.00	S1-1	PP-0.50 kg/cm²			
2.50	P1-1	2	0	1	1
4.00					
4.50	S1-2	PP-0.25 kg/cm²			
4.95	P1-2	1	0	0	1
6.00					
6.50	S1-3	PP-0.25 kg/cm²			
6.95	P1-3	1	0	0	1
8.00					
8.50	S1-4	PP-0.25 kg/cm²			
8.95	P1-4	1	0	0	1
10.00					
10.50	D1-1				
10.95	P1-5	6	3	3	3
12.00					
12.50	D1-2				
12.95	P1-6	14	3	6	6
14.00					
14.45	P1-7	2	0	0	2
15.50					
16.00	S1-5	PP-0.50 kg/cm²			
16.45	P1-8	2	2	1	1
17.50					
18.00	S1-6	PP-2.75 kg/cm²			
18.45	P1-9	21	4	9	12
19.00					
19.45	P1-10	22	7	11	11
20.00	S1-7	PP-1.75 kg/cm²			
20.45	P1-11	10	2	4	6
21.00					
21.45	P1-12	21	5	9	12
22.00	S1-8	PP-3.25 kg/cm²			
22.45	P1-13	36	7	16	20
23.00					
23.45	P1-14	27	6	11	16
23.70					
24.00	S1-				
24.45	P1-15	30	6	11	16
25.00					
25.45	P1-16	22	8	10	12
26.00					
26.45	P1-17	28	5	8	20
27.00					
27.45	P1-18	27	5	10	17
28.00					
28.45	P1-19	32	6	10	22
29.00					
29.45	P1-20	24	9	10	14
30.00					
30.45	P1-21	33	13	18	15
31.00					
31.45	P1-22	56	16	25	31
32.00					
32.45	P1-23	46	16	22	24
33.00					
33.45	P1-24	35	16	15	20
34.00					
34.45	P1-25	38	18	21	17
35.00					
35.45	P1-26	48	12	18	10
36.00					
36.45	P1-27	56	13	17	39
37.00					
37.45	P1-28	36	12	16	20
38.00					
38.45	P1-29	63	13	30	33
39.00					
39.45	P1-30	50	20	21	29
40.00					
40.45	P1-31	32	15	15	17
41.00					
41.45	P1-32	39	16	20	19
42.50					
42.95	P1-33	25	5	8	17
44.00					
44.45	P1-34	49	4	17	32
45.50					
45.95	P1-35	87	27	63	44
47.00					
47.45	P1-36	59	15	30	29

N–Value: 10 20 30 40 50
Core Recovery: 20 40 60 80 100%

FIG. 3–14 (1) DRILLING LOG

DRILLING LOG

River bed is RWL–13.5 m
Casing depth is 18.0 m
from RWL ±0 m

Project: THE RAMA VI BRIDGE CONSTRUCTION
Type of Drilling: WASH
Hole Number: B–2
Elevation: EL–12.77 m MSL
Date: 3–7 Sep. 1981
Water Table: EL +0.73 m MSL
Driller: TONGCHAI (KAMIJO)

Scale in m	Elevation in m	Depth in m	Thickness in m	Type of Soil	Colour	Relative Density or Consistency	General Remarks
	–12.77						
	15.67	2.30	2.30	silty clay	dark grey	soft	little fine sand and some of organic matter
	16.77	2.90	0.60	silty clay	grey	medium	
3/9	19.37	6.60	3.70	silty clay	grey	stiff / very stiff	with organic matter and mica fragment; 4.5 – 5.5 m mottled with light grey; 5.5 – 6.6 m some of fine sand
	20.57	7.80	1.20	fine sand		very compact	
	25.77	13.00	5.20	silty clay	brown / grey red / grey	very stiff / Hard	with slicken side; 10.5 – 11.5 m trace of laterite; 12.5 – 13.0 m mottled with brown
	26.52	13.75	0.75	silty clay		very stiff	with layer of clay
	27.27	14.50	0.75	silty clay		stiff	with layer of fine sand
	30.27	17.50	3.00	fine to medium sand	grey / brown	very compact	trace of coarse sand; 14.5 – 15.5 m seam to clay; 16.5 – 17.5 m with fine gravel
	31.27	18.50	1.00	fine to coarse sand	brown	very compact	trace of fine gravel
	34.27	21.50	3.00	fine to medium sand	light brown / grey	very compact	18.5 – 19.5 m trace of coarse sand; 20.5 – 21.5 m trace of coarse sand
	35.27	22.50	1.00	coarse sand	grey	very compact	trace of fine gravel
	36.27	23.50	1.00	fine to medium sand	light brown	very compact	trace of gravel
	38.27	25.50	2.00	fine to coarse sand	light brown	very compact	trace of fine gravel
	39.27	26.50	1.00	fine to coarse sand	light brown	very compact	
	41.27	28.50	2.00	fine to coarse sand	light brown	very compact	
	43.27	30.50	2.00	fine sand		very compact	
	44.27	31.50	1.00	fine to medium sand	light brown	medium compact	trace of coarse sand
	47.27	34.50	3.00	fine to coarse sand	light brown / grey	compact / very compact	trace of fine gravel; 31.5 – 32.5 m fine to coarse sand; 32.5 – 33.5 m medium to coarse sand; 33.5 – 34.5 m fine to coarse sand seam to silty clay
	51.27	38.50	4.00	fine sand	grey	very compact	34.5 – 35.5 m layer of silty-clay; 35.5 – 37.5 m trace of medium sand; 37.5 – 38.5 m trace of fine gravel
7/9	8 22	45.45	6.9	medium to coarse sand	grey	very compact	trace of fine gravel

Sampling Depth in m	Sample No	N Value (30 cm)	Blows per each 15 cm (15 cm)	15 cm	15 cm
1.00					
1.45	P2–1	1	0	0	1
2.00					
2.45	P2–2	1	0	0	1
3.00					
3.45	P2–3	9	3	4	5
4.00	S2–1				
4.45	P2–4	13	3	6	7
5.00					
5.45	P2–5	28	12	14	14
6.00	S2–2	PP–3 kg/cm²			
6.45	P2–6	23	7	9	14
7.00					
7.45	P2–7	68	9	23	45
8.00					
8.45	P2–8	21	5	9	12
9.00					
9.45	P2–9	31	6	13	18
10.00					
10.45	P2–10	24	6	11	13
11.00	S2–3	PP–3.0 kg/cm²			
11.45	P2–11	21	8	10	11
12.00					
12.45	P2–12	47	14	21	26
13.00	S2–4	PP–4.25 kg/cm²			
13.45	P2–13	27	9	11	16
14.00					
14.45	P2–14	18	8	8	10
15.00					
15.45	P2–15	71	15	29	42
16.00					
16.45	P2–16	61	22	31	30
17.00					
17.45	P2–17	82	22	33	49
18.00					
18.45	P2–18	49	22	21	28
19.00					
19.45	P2–19	63	25	29	34
20.00					
20.45	P2–20	77	18	33	44
21.00					
21.45	P2–21	106	37	49	57
22.00					
22.45	P2–22	66	22	31	35
23.00					
23.45	P2–23	105	41	55	50
24.00					
24.45	P2–24	62	12	28	34
25.00					
25.45	P2–25	33	11	10	23
26.00					
26.45	P2–26	83	29	38	45
27.00					
27.45	P2–27	55	20	25	30
28.00					
28.45	P2-28	69	18	31	38
29.00					
29.45	P2–29	63	26	29	34
30.00					
30.45	P2–30	71	20	35	36
31.00					
31.45	P2–31	19	3	7	12
32.00					
32.45	P2–32	39	11	17	22
33.00					
33.45	P2–33	56	17	22	34
34.00					
34.45	P2–34	53	18	20	33
35.00					
35.45	P2–35	81	18	35	46
36.00					
36.45	P1–36	116	35	54	62
37.00					
37.45	P2–37	76	30	38	38
38.00					
38.45	P2–38	58	14	23	35
39.00					
39.45	P2–35	55	18	27	28
40.00					
40.45	P2–40	65	26	32	33
41.00					
41.45	P2–41	64	20	31	33
42.00					
42.45	P2–42	87	34	40	47
43.00					
43.45	P2–43	76	28	36	40
44.00					
44.45	P2–44	81	29	39	42
45.00					
45.45	P2–45	84	30	41	43

Standard Penetration Test or Core Recovery

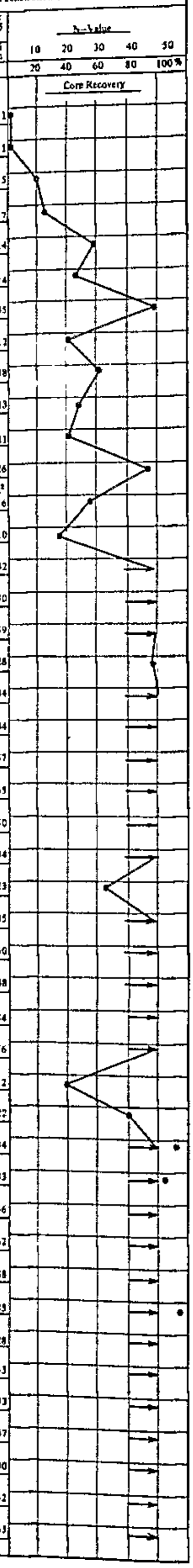


FIG. 3–14 (2) DRILLING LOG

DRILLING LOG

Project: THE RAMA VI BRIDGE CONSTRUCTION
Type of Drilling: WASH
Hole Number: B-3
Elevation: EL-10.59 m MSL
Date: 9-12 Sep 1981
Water Table: EL +0.51 m MSL
Driller: TONGCHAI (KAMJJO)

River bed is RWL-11.1 m
casing depth is 18.0 m
from RWL±0 m

Elevation in m	Depth in m	Thickness in m	Type of Soil	Colour	Relative Density or Consistency	General Remarks
-14.89	4.30	4.30	silty clay	brown / grey brown	very soft	with mica fragment trace of fine sand
-18.59	8.00	3.70	silty clay		very stiff	5.0 – 7.0 m with lateritic matter and trace of fine sand; 7.0 – 8.0 m trace of calcareous matter and mottled with brick
-19.59	9.00	1.00	clayey silt		compact	little calcareous matter mottled with
-20.59	10.00	1.00	silt with fine sand		loose	mottled with grey
-21.39	10.80	0.80	silty fine sand		medium compact	trace of calcareous matter
-22.59	12.00	1.20	silty clay		hard	with slicken side
-26.19	15.60	3.60	silty clay	grey	very stiff	12.0 – 14.0 m with slicken side; 14.0 – 15.0 m trace of lateritic matter
-31.09	20.50	4.90	fine to coarse sand	light brown	very compact	trace of fine gravel
-33.09	22.50	2.00	fine to medium sand	grey	very compact	with little coarse sand
-34.09	23.50	1.00	fine sand	grey	very compact	trace of medium to coarse sand
-35.09	24.50	1.00	fine sand		very compact	layer of silty clay
-36.09	25.50	1.00	fine sand	grey	very compact	with mica fragment
-39.09	28.50	3.00	fine sand	brown	very compact	28.0 – 28.5 m trace of medium to coarse sand
-40.09	29.50	1.00	fine to medium sand	brown	medium compact	trace of coarse sand and fine gravel
-47.39	36.80	7.30	medium to coarse sand	grey	compact	trace of fine gravel
-49.09	38.50	1.70	sandy clay	dark grey	medium compact	sand is fine to coarse with some gravel and organic matter and mica
-51.59	41.00	2.50	sandy clay	grey	medium compact	trace of gravel with mica fragment sand is coarse
-56.04	45	4.45	fine to coarse sand	light brown / grey	very compact	with silt pocket and mica fragment

Sampling Depth in m	Sample No	N Value (30 cm)	Blows per each 15 cm (1st 15 cm)	(2nd 15 cm)	(3rd 15 cm)
2.00					
2.45	P3-1	1	0	0	1
3.00					
3.45	P3-2	1	0	0	1
4.00					
4.45	P3-3	15	2	6	9
5.45	P3-4	21	4	8	13
6.00					
6.45	P3-5	21	4	9	12
7.45	P3-6	19	3	6	13
8.00					
8.45	P3-7	13	3	5	8
9.00	S3-1				
9.45	P3-8	8	2	3	5
10.00					
10.45	P3-9	12	2	4	8
11.00					
11.45	P3-10	34	8	14	20
12.45	P3-11	29	5	11	18
13.00					
13.45	P3-12	25	7	10	15
14.00	S3-3				
14.45	P3-13	23	5	9	14
15.00					
15.45	P3-14	21	7	9	12
16.00					
16.45	P3-15	47	16	21	26
17.00					
17.45	P3-16	52	18	24	28
18.00					
18.45	P3-17	37	17	18	19
19.00					
19.45	P3-18	50	17	22	28
20.00					
20.45	P3-19	43	18	19	24
21.00					
21.45	P3-20	52	19	23	29
22.00					
22.45	P3-21	66	22	31	35
23.00					
23.45	P3-22	57	23	24	33
24.00					
24.45	P3-23	34	7	14	20
25.00					
25.45	P3-24	83	17	39	44
26.00					
26.45	P3-25	52	21	24	28
27.00					
27.45	P3-26	57	17	24	33
28.00					
28.45	P3-27	55	20	35	20
29.00					
29.45	P3-28	24	18	13	11
30.00					
30.45	P3-29	24	11	10	14
31.00					
31.45	P3-30	34	18	17	17
32.00					
32.45	P3-31	36	12	18	18
33.00					
33.45	P3-32	38	11	19	19
34.00					
34.45	P3-33	57	16	26	31
35.00					
35.45	P3-34	46	14	18	28
36.00					
36.45	P3-35	47	25	25	22
37.00					
37.45	P3-36	18	9	9	9
38.00					
38.45	P3-37	25	9	12	13
39.00					
39.45	P3-38	21	6	9	12
40.00					
40.45	P3-39	19	6	8	11
41.00					
41.45	P3-40	61	19	30	31
42.00					
42.45	P3-41	61	21	32	28
43.00					
43.45	P3-42	66	22	36	30
44.00					
44.45	P3-43	82	16	30	52
45.00					
45.45	P3-44	124	24	42	81

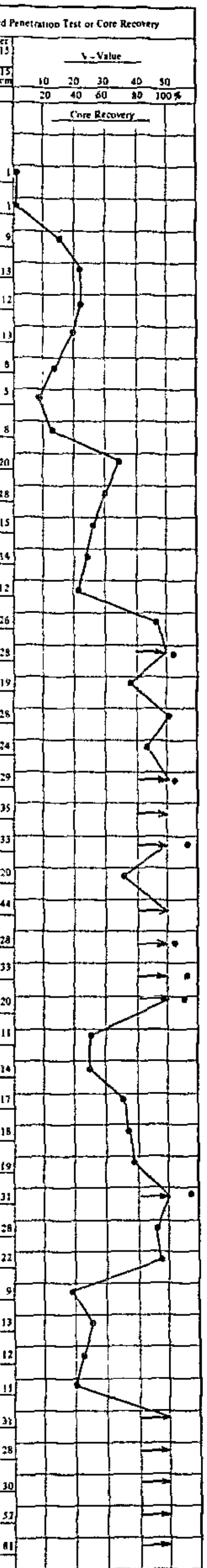


FIG. 3-14 (3) DRILLING LOG

DRILLING LOG

Project THE RAMA VI BRIDGE CONSTRUCTION

Hole Number B-4 Elevation EL +2.03

Water Table EL +) 33 m MSL

Type of Drilling WASH

Date 18-23 Aug 1981

Driller TONGCHAI (KAMIJO)

Casing depth is 27 m from ground

Scale in m	Elevation in m	Depth in m	Thickness in m	Type of Soil	Colour	Relative Density or Consistency	General Remarks
	2.03						
1	1.23	0.80	0.80	fill	brown	medium	
2	0.53	1.50	0.70	fill		medium compact	fine to medium sand
3	1.47	3.50	2.00			medium soft	clay with organic matter
4–12; 18/8 0.8m	0.47	12.50	9.00	silty clay	grey	very soft; soft	with mica fragments 3.5 – 6.0 m very soft clay trace of shell fragments; 6.0 – 8.0 m soft clay with slicken side trace of shell fragments layer of silty clay; 8.0 – 12.5 m soft clay
13–16	3.97	16.00	3.50	silty clay	dark grey; grey	medium	with mica fragments; with slicken side trace of laterite with mica fragments
17–23; 19/8 5.5m	21.47	23.60	7.50	silty clay		stiff	with mica fragments 16.0 – 19.0 m trace of lateritic matter; 19.0 – 20.5 m little fine sand; 20.5 – 21.5 m some fine sand; 21.5 – 23.5 m some of fine sand and little lateritic soil
24–25	22.77	24.80	1.30	silty clay		very stiff	
26	23.97	26.00	1.70	silty clay		very stiff	lateritic soil with slicken side
27	25.47	27.50	1.50	silty clay		Hard	trace of lateritic with slicken side brown spots
28; 20/8 18.60	26.57	28.60	1.10	silty clay		stiff	some fine sand trace of laterite
30–41	39.87	41.00	12.40	fine to medium sand	grey/ grey	very compact	28.6 – 29.5 m silty fine sand layer of clay; 31 m red spot 29.5 – 33.5 m fine to medium sand; 33.5 – 36.5 m fine sand; 36.5 – 39.5 m silty fine sand; 39.5 – 41.0 m fine to medium sand
22/8 10.5m; 43; 23/8 2.0m	42.47	44.45	3.45	fine to coarse sand	light brown; grey; light brown	very compact	41.0 – 42.5 m fine to coarse sand; 42 m fine gravel; 42.5 – 44.25 m fine sand

Sampling / Standard Penetration Test or Core Recovery

Depth in m	Sample No	N Value (30 cm)	Blows per each 15 cm: 15 cm	15 cm	15 cm
2.50					
3.00	S4-1				
3.45	P4-2	1	0	0	1
4.50					
5.00	S4-2				
5.45	P4-2	1	0	0	1
6.50					
7.00	S4-3				
7.45	P4-3	1	0	0	1
8.50					
9.00	S4-4				
9.45	P4-4	1	0	0	1
10.50					
11.00	S4-5				
11.45	P4-5	1	0	0	1
12.50					
13.00	S4-6	PP-0.75 kg/cm²			
13.45	P4-6	2	0	1	1
14.50					
15.00	S4-7				
15.45	P4-7	9	2	4	5
16.50					
17.00	S4-8	PP-1.25 kg/cm²			
17.45	P4-8	24	6	11	13
18.00					
18.45	P4-9	21	6	9	12
19.00	S4-9	PP-3.0 kg/cm²			
19.45	P4-10	17	5	8	9
20.00					
20.45	P4-11	19	4	9	10
21.00					
21.45	P4-12	22	6	9	13
22.00	S4-10	PP-1.25 kg/cm²			
22.45	P4-13	17	4	7	10
23.00					
23.45	P4-14	19	5	8	11
24.00	P4-11				
24.45	P4-15	27	8	13	14
25.00					
25.45	P4-16	30	7	13	17
25.00					
26.00	P4-17				
26.45	P4-17	52	13	24	28
27.00					
27.45	P4-18	35	10	16	15
27.00					
28.00	S4-13	PP-3.0 kg/cm²			
28.45	P4-19	15	4	7	8
29.00					
29.45	P4-20	62	9	25	37
30.00					
30.45	P4-21	48	15	27	21
31.00					
31.45	P4-22	62	21	26	36
32.00					
32.45	P4-23	36	14	18	18
33.00					
33.49	P4-24	39	10	15	24
34.00					
34.45	P4-25	64	25	33	31
35.00					
35.45	P4-26	73	33	48	25
36.00					
36.45	P4-27	34	33	23	11
37.00					
37.45	P4-26	111	21	41	70
38.00					
38.45	P4-29	80/6"	47	80/6"	–
39.00					
39.45	P4-30	77/7"	44	77/7"	–
40.00					
40.45	P4-31	84	16	44	40
41.00					
41.45	P4-32	63	20	24	39
42.00					
42.45	P4-33	46	14	19	27
43.00					
43.45	P4-34	66	71	30	36
44.00					
44.45	P4-35	114	36	56	58

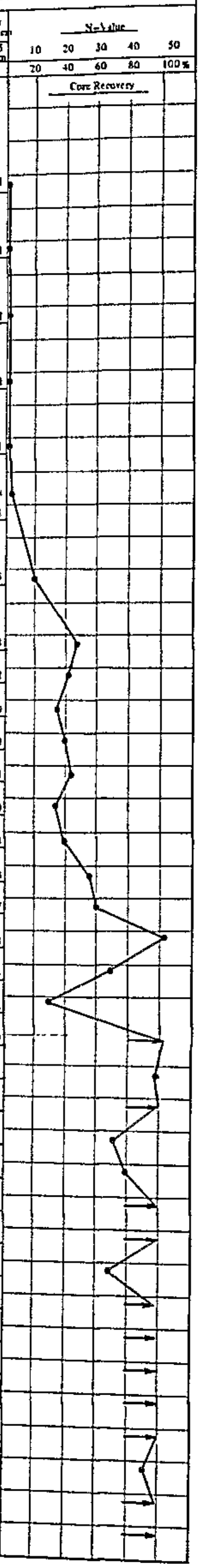
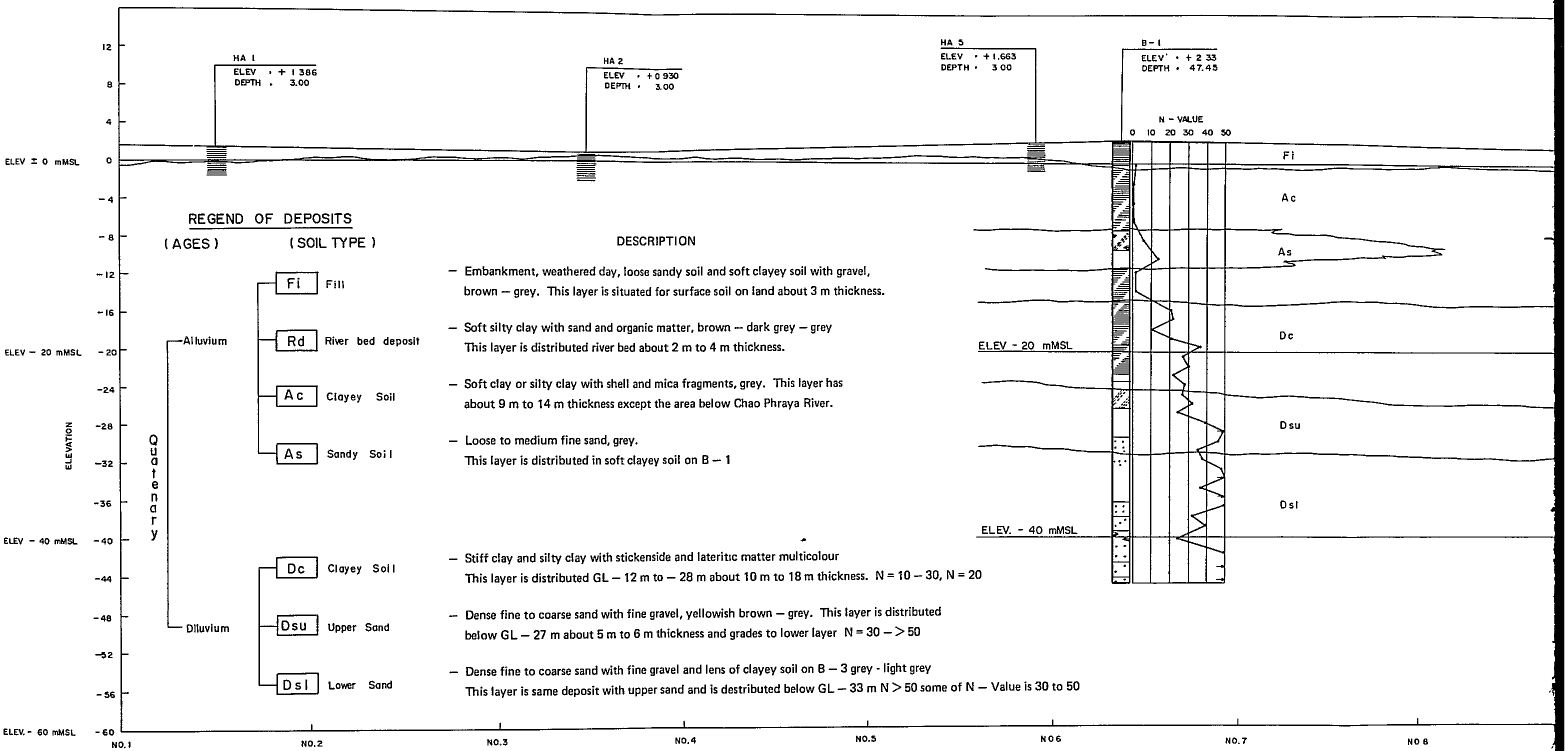
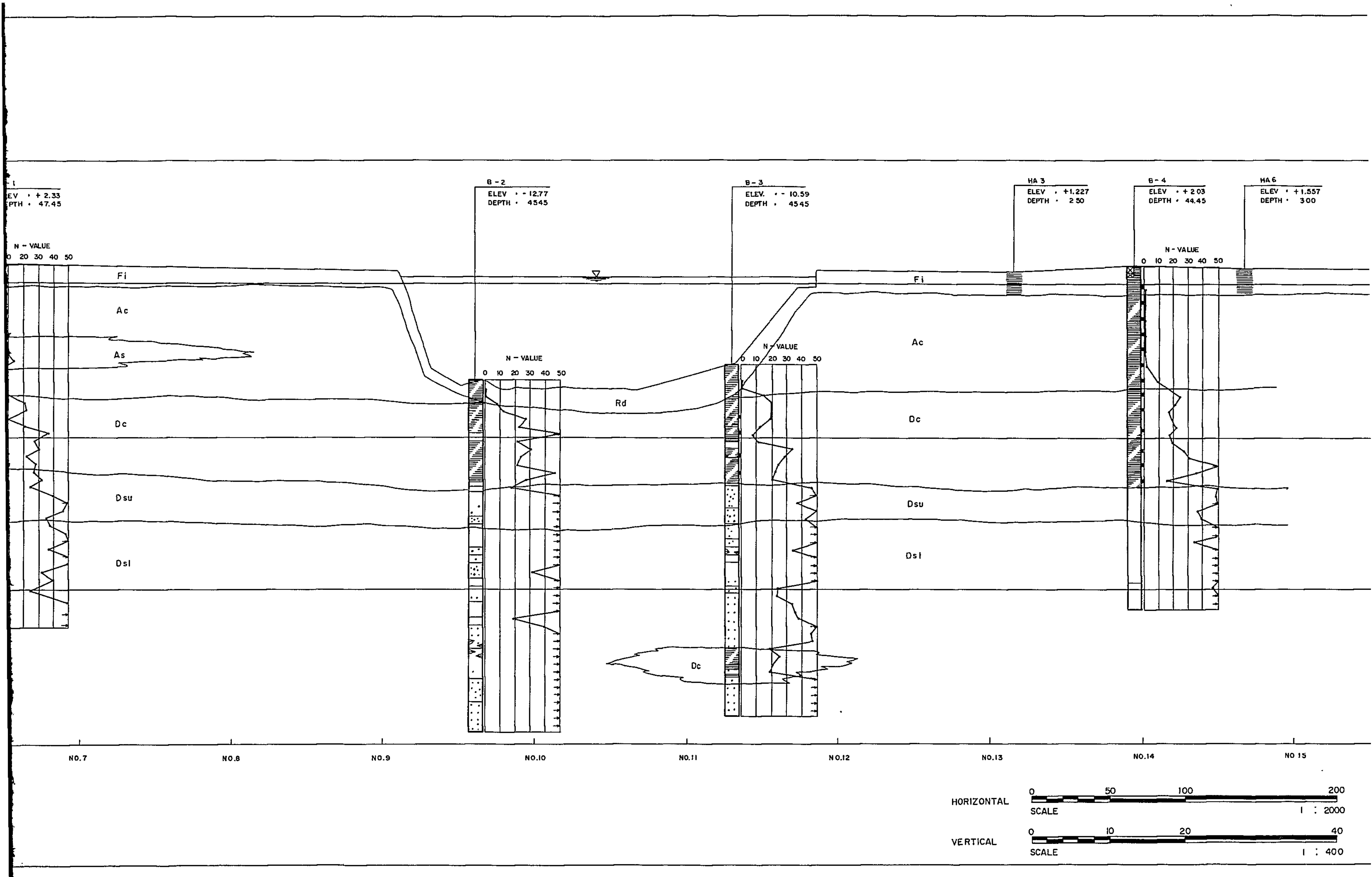


FIG. 3–14 (4) DRILLING LOG


HA 1
ELEV • + 1.386
DEPTH • 3.00
HA 2
ELEV • + 0.930
DEPTH • 3.00
HA 5
ELEV • + 1.663
DEPTH • 3.00
B - 1
ELEV • + 2.33
DEPTH • 47.45
N - VALUE
0 10 20 30 40 50
ELEV ± 0 mMSL
ELEV - 20 mMSL
ELEV - 40 mMSL
ELEV. - 60 mMSL
ELEVATION
12
8
4
0
-4
-8
-12
-16
-20
-24
-28
-32
-36
-40
-44
-48
-52
-56
-60
NO.1
NO.2
NO.3
NO.4
NO.5
NO.6
NO.7
NO.8
Fi
Ac
As
Dc
Dsu
Dsl
REGEND OF DEPOSITS
(AGES)
(SOIL TYPE)
DESCRIPTION
Quaternary
Alluvium
Diluvium
Fi Fill
— Embankment, weathered day, loose sandy soil and soft clayey soil with gravel, brown — grey. This layer is situated for surface soil on land about 3 m thickness.
Rd River bed deposit
— Soft silty clay with sand and organic matter, brown — dark grey — grey This layer is distributed river bed about 2 m to 4 m thickness.
Ac Clayey Soil
— Soft clay or silty clay with shell and mica fragments, grey. This layer has about 9 m to 14 m thickness except the area below Chao Phraya River.
As Sandy Soil
— Loose to medium fine sand, grey. This layer is distributed in soft clayey soil on B — 1
Dc Clayey Soil
— Stiff clay and silty clay with stickenside and lateritic matter multicolour This layer is distributed GL — 12 m to — 28 m about 10 m to 18 m thickness. N = 10 — 30, N = 20
Dsu Upper Sand
— Dense fine to coarse sand with fine gravel, yellowish brown — grey. This layer is distributed below GL — 27 m about 5 m to 6 m thickness and grades to lower layer N = 30 — > 50
Dsl Lower Sand
— Dense fine to coarse sand with fine gravel and lens of clayey soil on B — 3 grey - light grey This layer is same deposit with upper sand and is distributed below GL — 33 m N > 50 some of N — Value is 30 to 50



ELEV • + 2.33
DEPTH • 47.45
N - VALUE
0 20 30 40 50
B - 2
ELEV • - 12.77
DEPTH • 4545
B - 3
ELEV. • - 10.59
DEPTH • 4545
HA 3
ELEV • +1.227
DEPTH • 2 50
B - 4
ELEV • + 2 03
DEPTH • 44.45
HA 6
ELEV • +1.557
DEPTH • 300
N - VALUE
0 10 20 30 40 50
Fi
Ac
As
Dc
Dsu
Dsl
Rd
NO.7
NO.8
NO.9
NO.10
NO.11
NO.12
NO.13
NO.14
NO 15
HORIZONTAL
0 50 100 200
SCALE
1 : 2000
VERTICAL
0 10 20 40
SCALE
1 : 400

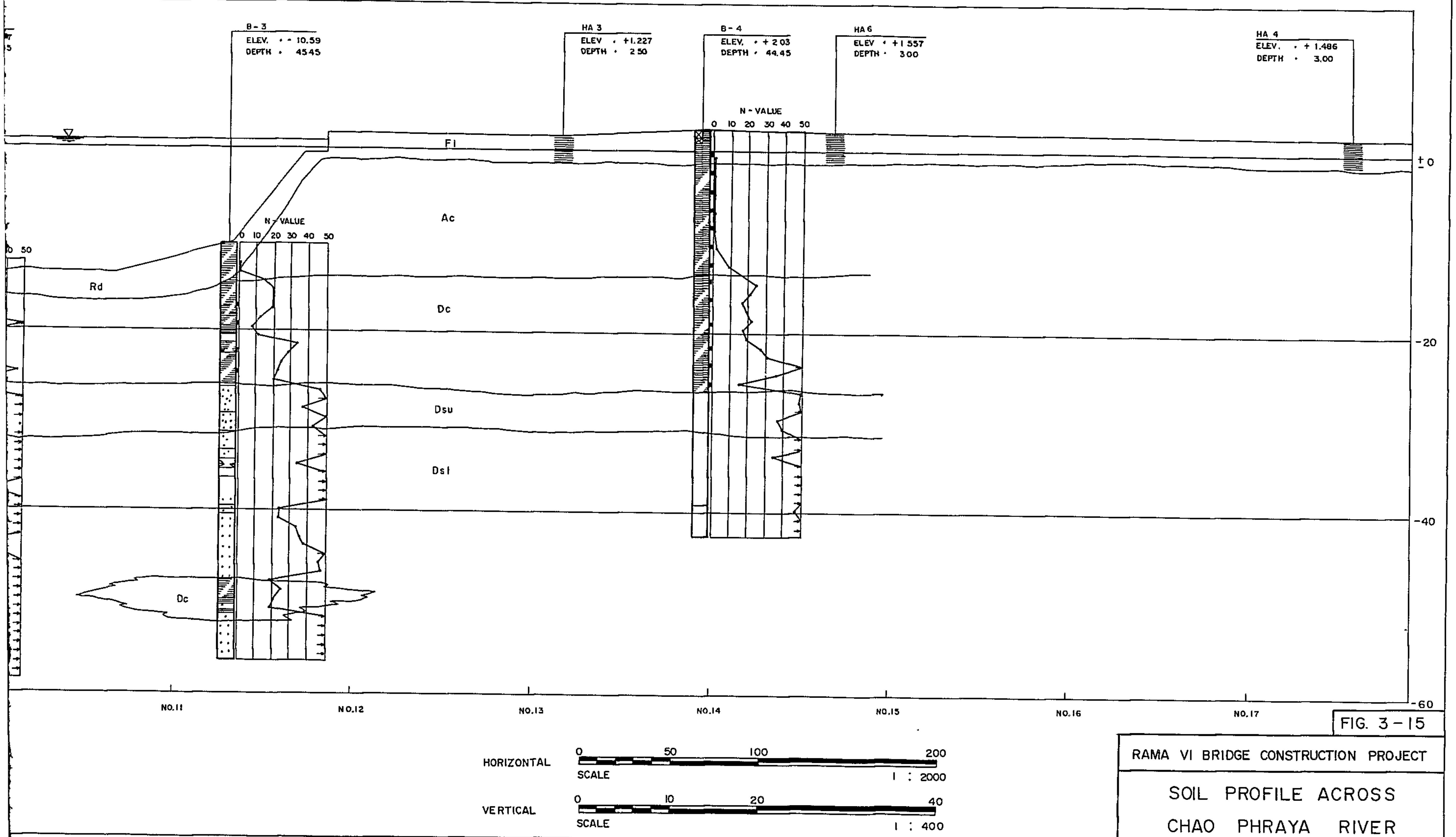


FIG. 3-15

RAMA VI BRIDGE CONSTRUCTION PROJECT

SOIL PROFILE ACROSS CHAO PHRAYA RIVER

CHAPTER 4
DESIGN STANDARD

CHAPTER 4
DESIGN STANDARD

4-1 GEOMETRIC DESIGN STANDARDS

The geometric design standards for the project have been selected in accordance with :

A Policy on Geometric Design of Urban Highways -AASHTO

The main points of the adopted standards are summarized in Table 4-1 and are described below.

Table 4-1 Geometric Design Criteria

Item	Unit	Design Criteria
Design speed	Km/H	60
Minimum Radius of Horizontal Curve R	m	175
Clothoid Parameter, A	m	R/3 = A = R
Critical Radius for Using Clothoid	m	1500
Maximum Grades	%	4
Length of Vertical Curve (Parabolic)		
crest vertical curves	m	200
sag vertical curves	m	80
Maximum Superelevation	%	4
Stopping Sight Distance	m	75
Minimum Passing Sight Distance	m	350
Carriageway Cross Slope	%	2

4-1-1 Design Speed

The design speed is the maximum safe driving speed which can be maintained with comfort when weather and traffic conditions are favourable. The design speed directly affects the minimum standards of such features as sight distance, radius of horizontal curve, and superelevation all of which assist the safe operation of vehicles.

The design speed of 60 Km/H has been adopted since the project area is located in urban area and also due to various designing restrictions such as the necessity of installing a flyover and rampways on the Bangkok side and the existence of a small scale interchange on the Thonburi side and many at-grade intersections for the Charan Sanitwong Road.

4-1-2 Minimum Radius of Horizontal Curve

It is desirable to use the maximum curve radius which is consistent with the topographical features and control points on the alignment. It is, however, necessary to specify a minimum radius to ensure standards of safety and comfort for driving. The value must also be realistic for use in all parts of the alignment. In order to counteract the centrifugal forces on a vehicle which is travelling along a curve, it is usual to provide superelevation. The amount of superelevation to be provided has to satisfy the following formula :

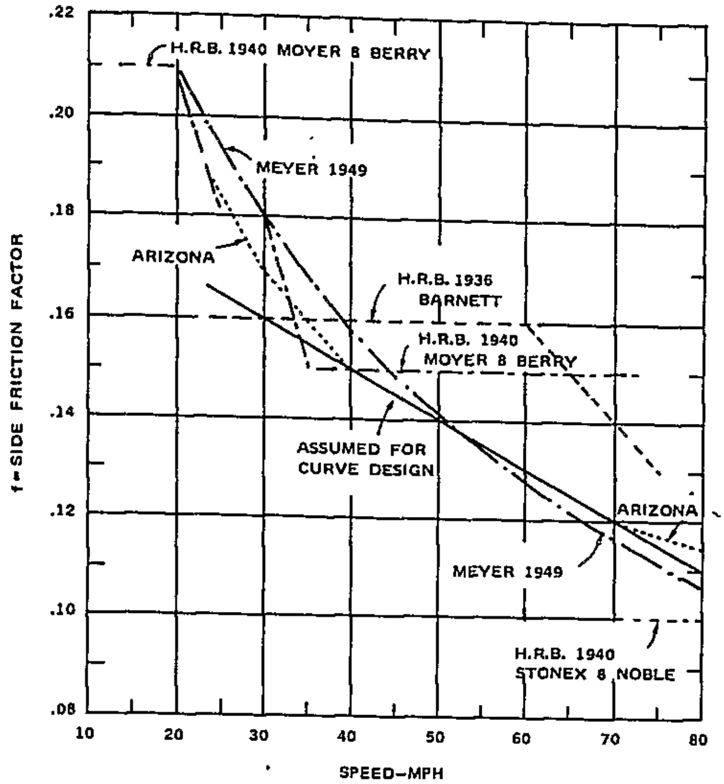
$$R = \frac{v^2}{15\ (e+f)} \qquad (v^2 = 127R\ (e+f))$$

where v = speed of vehicle in Km/H
R = radius of the curve in m
e = superelevation (ratio)
f = coefficient of side friction of the road (ratio)

Because the project road passes through the urban area and crosses several minor roads, an appropriate upper limit of superelevation has been considered to be 4%.

From the standpoints of speed, road conditions and comfort, an appropriate coefficient of side friction of the road is 0.15.

Table 4-2 Maximum safe side friction factors



from GDRH of AASHTO

The minimum radius of horizontal curve at the travelling speed of 80 Km/H has been calculated as 280 meters because corresponding side friction has been set as 0.14.

This value should be recommended minimum radius of horizontal curves, wherever it is possible that larger radius of curves will be provided.

4-1-3 Lane Width

The minimum traffic lane width has been established in AASHTO standards by observations on the behaviour of drivers of heavy vehicles when they pass vehicles travelling in the opposite direction on tangent or on Gradual curve. The desirable traffic lane width as determined from these observations is 3.6_m (12 feet).

However, a lane width of 3.25_m recommended is the same as the existing lane width in DOH & BMA.

4-1-4 Shoulder Width

Shoulder has been generally defined as a side space provided for protection of main structures of roadway and also sufficient width for a stopped vehicle due to mechanical failure.

Shoulders give safety and preferable trafficability for travelling vehicles and prevent accidents and resulting traffic congestion.

A shoulder width of 0.3_m for roadway sections and of 0.5_m for structures have been recommended in compliance with the project located in urban area and the shoulder widths of the existing roads and the project road.

4-1-5 Median Width

A median width of 3.4_m at Thonburi side and 4.0_m at Bangkok side has been recommended for roadway sections except structures. This allows the space for guardrails, lighting columns, traffic signs, a suitable green belt, etc. A median width of 1.0_m has been recommended as an absolute minimum for the main bridge.

4-2 STRUCTURAL DESIGN STANDARDS

"The Standard Specifications of Highway Bridges of AASHTO" is adopted as the design standard of the proposed bridge. However, in compliance with the actual traffic involving heavy trucks in the GBA, "the British Standards of Steel, Concrete and Composite Bridges (BS 5400)" is specifically adopted for live loads instead of AASHTO. The number of units of HB loading is 45.

Recent trends in structural design concepts indicate that load factor design has become more and more common in use. Although in AASHTO, either allowable stress design or load factor design are allowed to be used in designing for reinforced concrete members, the allowable stress method has been adopted in the present Preliminary Design in order to enable an easy comparison with the design of the similar bridges recently built or designed on the Chao Phraya River.

4-2-1 Loads and Forces

The structures have been so designed as to satisfy the loading conditions described in clauses (1) to (8) below.

(1) Dead load

(2) Highway bridge live load

(3) Construction loading

(4) Wind load

(5) Temperature effect

(6) Force of stream current

(7) Impact from river traffic

(8) Earth pressure

Of these loading condition, (2) has been based on BS, and (4), (5), (7) and (8) have been adopted in compliance with the recently designed value for the bridges spanning the Chao Phraya River, paying due attention to the local characteristics of the loading factors. The other loads have been computed based on AASHTO standards.

(1) Dead load. No explanation is needed.

(2) Highway bridge live load.

Fig. 4-1 gives a diagrammatic description of the highway carriageway and lanes. The carriageway is the part of the running surface which includes all of traffic lanes, hard shoulders, hard strips and marker strips. The carriageway width is the width between raised curbs. In the absence of raised curbs it is the width between safety fences, less the amount of set-back required for these fences, being not less than 0.6m nor more than 1.0m from the traffic face of each fence. The traffic lanes are the lanes that are marked on the running surface and are normally used by traffic. The notional lanes are the parts of the carriageway used solely for the purpose of applying the specified live loads.

Requirements for deriving the width and number of notional lanes for the design purposes are specified as follows:

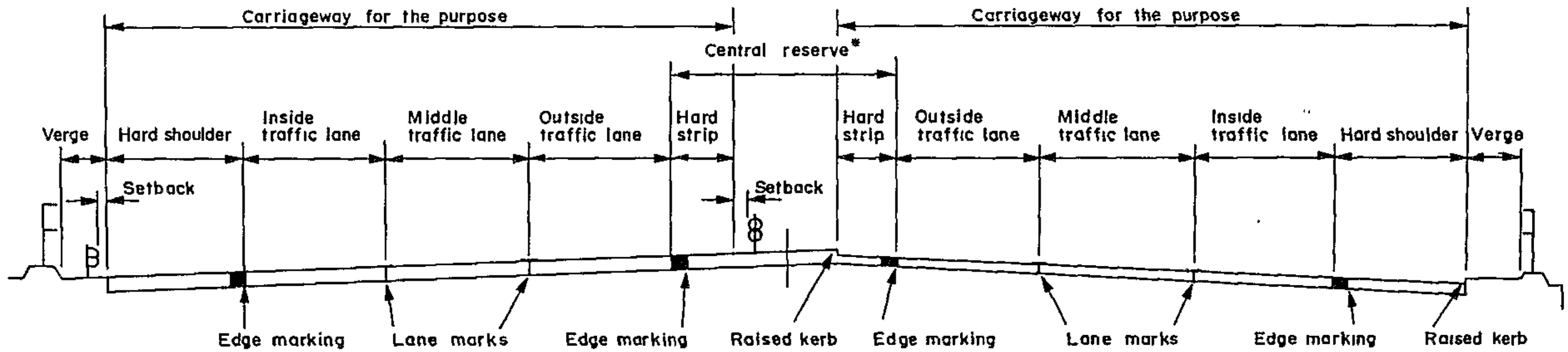
When the carriageway widths are 4.6 m or more, notional lanes shall be taken to be not less than 2.3 m nor more than 3.8 m wide. The carriageway shall be divided into the least possible integral number of notional lanes having equal widths as follows:

Carriageway width m	Number of notional lanes
4.6 up to and including 7.6	2
above 7.6 up to and including 11.4	3
above 11.4 up to and including 15.2	4
above 15.2 up to and including 19.0	5
above 19.0 up to and including 22.8	6

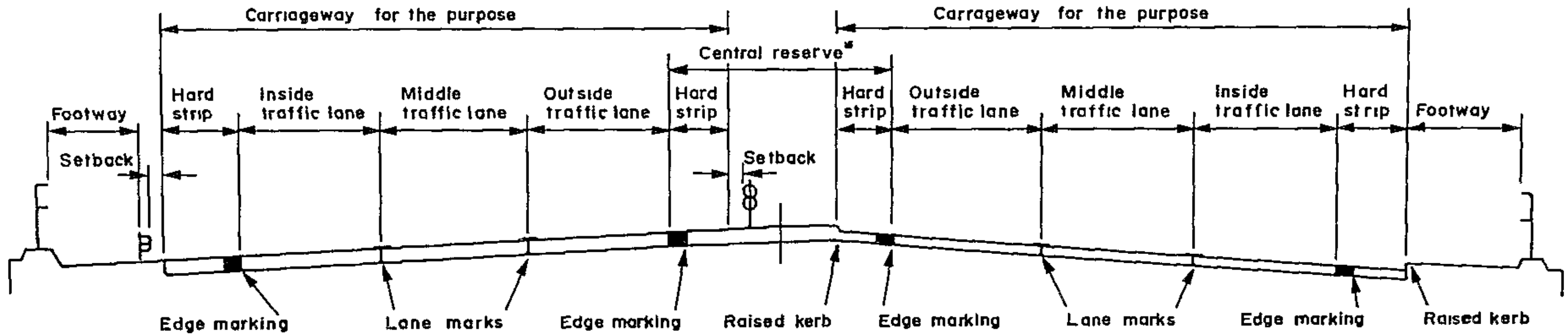
When the carriageway widths are less than 4.6 m, the carriageway shall be taken to have the number of notional lanes

$$= \frac{\text{width of carriageway (in meters)}}{3.0}$$

When the number of lanes is not an integer, the loading on the fractional part of a lane shall be taken pro rata from the loading for one lane.



(i) Motorway

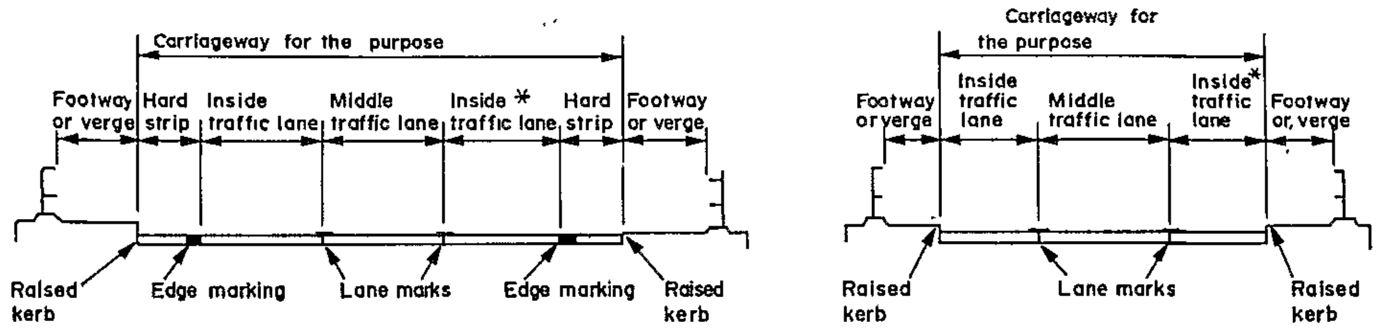


(ii) All-purpose road

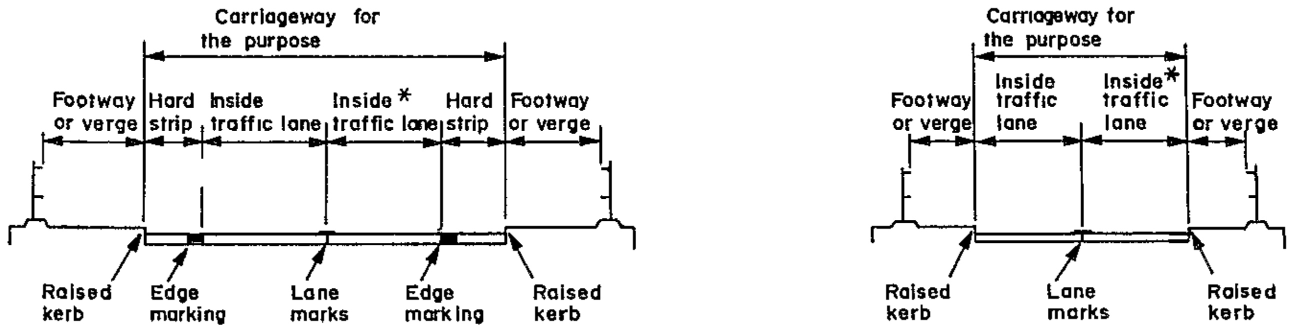
(a) Superstructures: dual carriageway

*Central reserve will be split on separate superstructures

FIG. 4–1 (1) HIGHWAY CARRIAGEWAY AND TRAFFIC LANES



(i) Single 3-lane carriageway



(ii) Single 2-lane carriageway

(b) Bridge carrying a single carriageway

*Where the carriageway carries unidiretional traffic only, this lane becomes the outside traffic lane.
NOTE 1. The same definitions of inside, middle and outside have been used for notional lanes.
NOTE 2 Where a safety fence replaces a raised kerb the limits of the footway or verge and the hard strip shall be as shown in figure I (a).

(concluded)

FIG. 4–1 (2) HIGHWAY CARRIAGEWAY AND TRAFFIC LANES

1) Type HA loading

Type HA loading consists of a uniformly distributed load and a knife edge load combined, or of a single wheel load as follows:

The nominal uniformly distributed load (UDL) shall be taken as 30 kN per linear meter of notional lane for loaded lengths up to 30 m, and for loaded lengths in excess of 30 m it shall be derived from the equation:

$$W = 15 \left(\frac{1}{L}\right)^{0.475} \quad \text{but not less than 9}$$

Where L = loaded length in m

W = load per meter of lane in kN

The nominal knife edge load (KEL) per notional lane shall be taken as 120kN. No allowance for the dispersal of the UDL and KEL shall be made.

Single nominal wheel load alternative to UDL and KEL:

One 100 kN wheel, placed on the carriageway and uniformly distributed over a circular contact area assuming an effective pressure of 1.1N/mm^2 (i.e. 340 mm diameter) shall be considered.

Alternatively, a square contact area may be assumed, using the same effective pressure (i.e. 300 mm side).

Dispersal of the single nominal wheel load at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

2) Type HB loading

The number of units of type HB loading that shall be 45. Fig. 4-2 shows the plan and axle arrangement for one unit of nominal HB loading. One unit shall be taken as equal to 10 kN per axle (i.e. 2.5 kN per wheel).
Nominal HB wheel loads shall be assumed to be uniformly distributed over a circular contact area, assuming an effective pressure of 1.1 N/mm^2. Alternatively, a square contact area may be assumed using the same effective pressure.
Dispersal of HB wheel loads at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.
Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

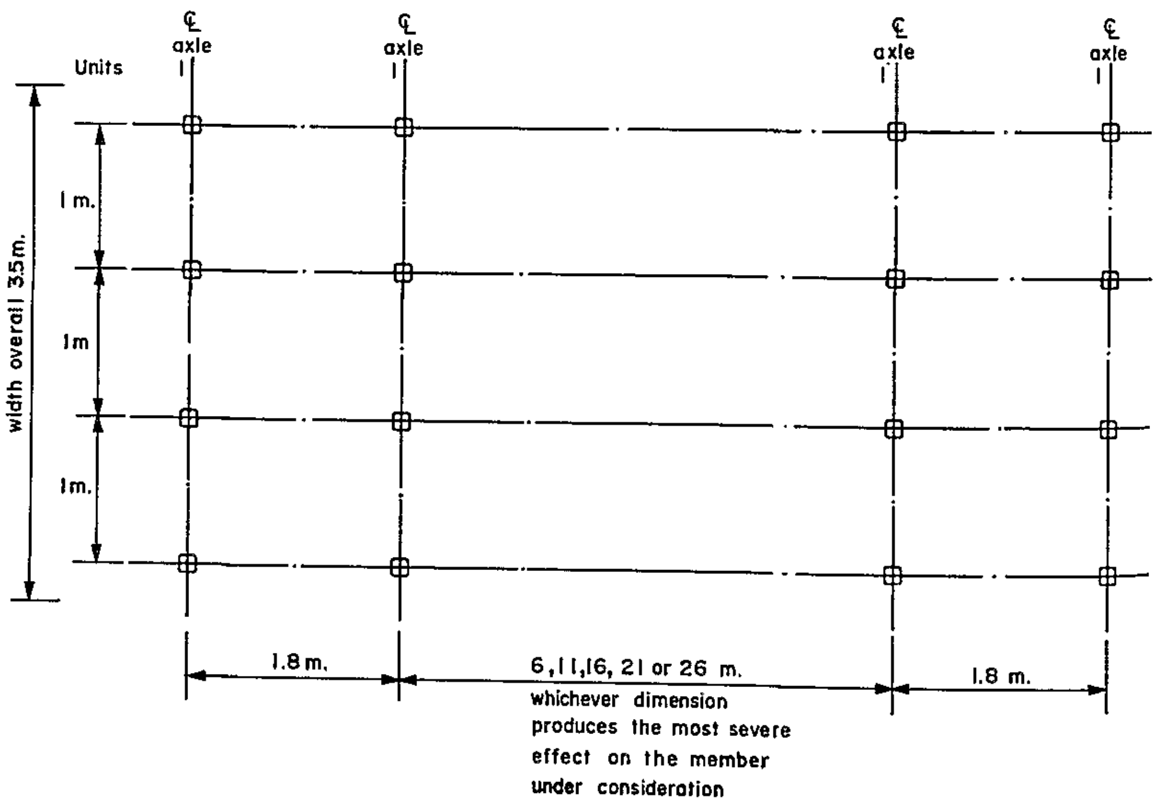


FIG. 4-2 DIMENSION OF HB VEHICLE

3) Application of types HA and HB loading

a) Type HA loading

The UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane.
Type HA UDL and KEL loads shall be applied to two notional lanes in the appropriate parts of the influence line for the element or the member under consideration and one-third type HA, UDL and KEL loads shall be similarly applied to all other notional lanes except where otherwise specified by the appropriate authority. The KEL shall be applied at one point only in the loaded length of each notional lane.

Where the most severe effect is caused by locating portions of loaded length, on one side of the superstructure over one portion of its length and on the other side of the superstructure in a longitudinally adjacent portion of its length, this shall be taken into consideration, using the loading appropriate to the combined length of the loaded portions.

The KEL shall be assumed to be acting as follows.

On plates, right slabs and skew slabs spanning or cantilevering longitudinally or spanning transversely : in a direction parallel to the supporting members or at right angles to the unsupported edges, whichever has the most severe effect. Where the element spans transversely, the KEL shall be considered as acting in a single line made up of portions having the same length as the width of the nominal lanes and having the intensities set out.

On longitudinal members and stringers : in a direction parallel to the supports.
On piers, abutments and other members supporting the superstructure : in a direction in lane with the bearings.
On cross members, including transverse cantilever brackets : in a direction in line with the span of the member.
In these cases, single wheel load is applied when the HA wheel load is applied to members supporting small areas of roadway, where the proportion of UDL and KEL that would otherwise be allocated to it is small.

b) Types HB and HA loading combined

Types HB and HA loading shall be combined and applied as shown in Fig. 4-3, which illustrates type HB loading in combination with type HA loading.

Type HB loading shall be applied to the bridge slabs specified as follows :

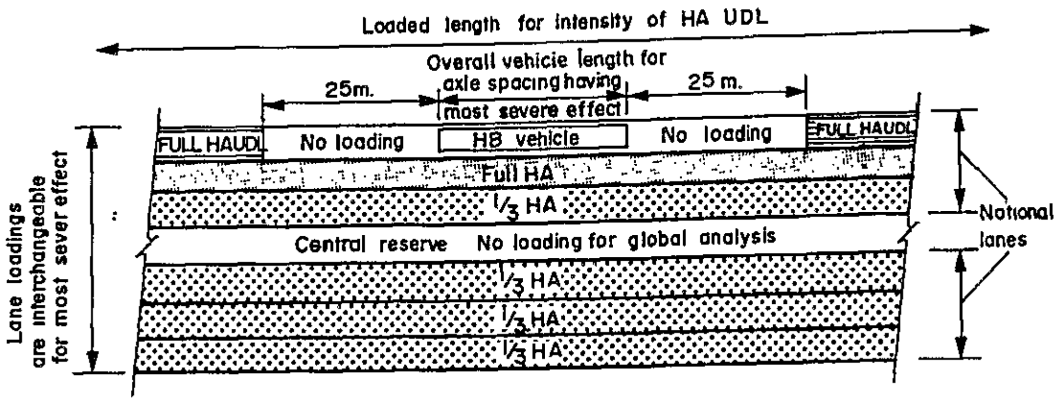
(a) Transverse cantilever slabs and slabs supported on all four sides. Transverse cantilever slabs shall be so proportioned as to resist the effects of the appropriate number of units of type HB loading placed in one notional lane in combination with 25 units of HB loading placed in one other notional lane. Proper consideration shall be given to transverse joints of transverse cantilever slabs and to the edges of these slabs because of the limitations of distribution.

This does not apply to members supporting transverse cantilever slabs.

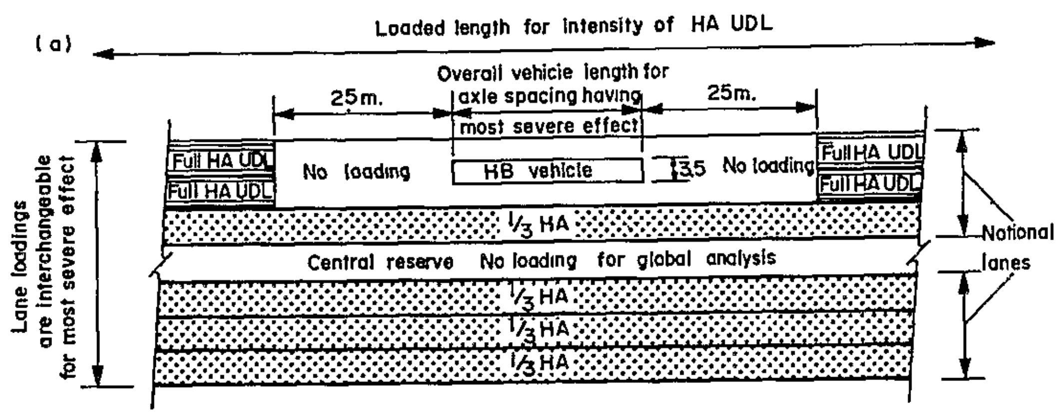
(b) Central reserves. On dual carriageways, the portion of the central reserve isolated from the rest of the carriageway either by a raised curb or by safety fences, is not required to be loaded with live load in considering the overall design of the structure, but it shall be capable of supporting 25 units of HB loading.

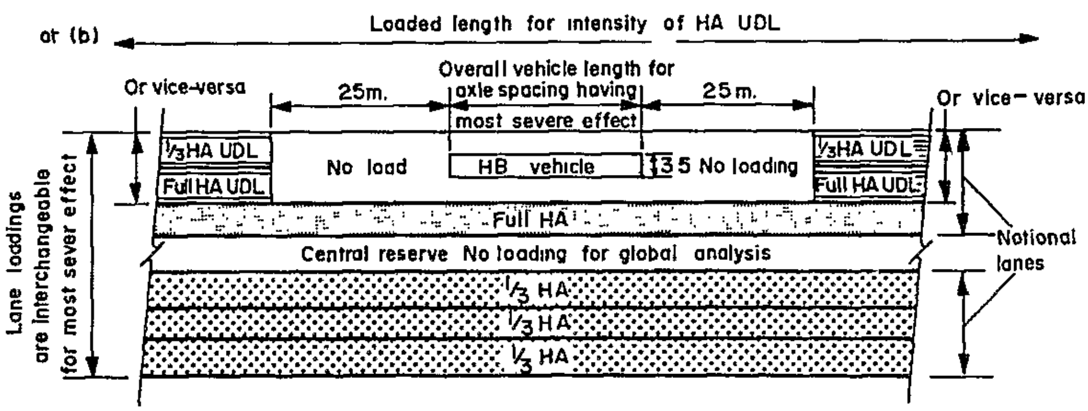
(c) Outer verges. Outer verges are not required to be loaded with live load in considering the overall design of the structure, but shall be capable of supporting any four wheels of 25 units of HB loading.

(1) HB vehicle within one notional lane



(2) HB vehicle straddling two notional lanes





NOTE 1. The overall length and width of the HB vehicle shall be as shown in FIG. 4 – 2

NOTE 2. Unless otherwise stated, type HA loading includes both uniformly distributed loading(UDL) and knife edge loading (KEL).

FIG. 4–3 TYPE HA AND HIGHWAY LOADING IN COMBINATION

4) Longitudinal load

The longitudinal load resulting from traction or braking of vehicles shall be taken as the more severe of following 2 cases, applied at the road surface and parallel to it in one notional lane only.

Nominal load for type HA:

The nominal load for HA shall be 8 kN/m of loaded length plus 20 t, subject to a maximum of 70 t, applied to an area one notional lane width x the loaded length.

Nominal load for type HB:

The nominal load for HB shall be 25% of the total nominal HB load adopted, applied as equally distributed between the eight wheels of two axles of the vehicle 1.8 m part.

(3) Construction loading

During construction, the structure must be able to withstand all forces resulting from dead loads; equipment loads and reasonable live loads.
Precautions must be taken to ensure that these assumed loadings are never exceeded.

(4) Wind load

The following wind load forces per square meter of exposed area are to be applied to all structures. The exposed area considered shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given below are for a wind velocity of 130 Km/H.

1) Superstructure design

A uniformly distributed wind load of the intensity of 160 kg/m^2 is to be applied horizontally at right angles to the longitudinal axis of the structure.

The total force shall not be less than 293 kg/m in the plane of the loaded spans.

The above forces are to be used for Group II loading (The various load combinations are detailed in Section 4-2-2). For Group III loading an additional load of 98 kg/m^2 must be applied at right angles to the longitudinal axis of the structure and 1.83 m above the deck as a wind load on moving live loads.

2) Substructure design

Forces transmitted to the substructure by superstructure and forces applied directly to the substructure by wind loads are to be taken into account.

3) Overturning forces

The effect of forces tending to overturn the structures must be calculated for Group II and Group III loading and an added upward force must also be applied at the windward quarter point of the transverse superstructure width. This force is 65 kg/m^2 of deck and sidewalk plan area for the Group II combination and 19 kg/m^2 for the Group III combination. The wind direction is assumed to be at right angles to the longitudinal axis of the structure.

(5) Temperature effect

Allowance must be made for the strains and forces resulting from:

- A temperature variation of 25 $\pm$ 20°C in the whole superstructure.
- A maximum difference in temperature of 10°C in the

top and bottom of the superstructure.

(6) Force of stream current

All piers and other portions of the structures which are subject to the force of flowing water are to be designed to resist the maximum stresses induced by stream current.

The effect of flowing water on piers may be calculated by the formula:

$$P = 52.5\ KV^2$$

P = pressure in kg/m^2

V = velocity of water in m/sec.

K = a constant, being 1.38 for square ends, 0.50 for angle ends where the angle is 30 degrees or less, and 0.67 for circular piers.

(7) Impact from river traffic

Impact from river traffic of 500t shall be considered.

(8) Earth pressure

The structures which retain fills must withstand pressure as given by Rankine's formula; provided, however, that no structure is designed for less than an equivalent fluid pressure of 480 kg/m^2.

All designs shall provide for the through drainage of the back-filling material by means of weep holes and crushed rock, pipe drains or gravel drains, or by perforated drains.

4-2-2 Loading Combinations

The following Service Load Groups represent various combinations of loads and forces to which the structures may be subjected. Each part of the structure, or the foundation on which it rests, shall be proportioned for all combinations of these forces as applicable to the particular sites or structures, at the percentage of the basic unit stress specified for the various groups except that no increase in allowable unit stresses is permitted for members or connections carrying wind loads only.

The maximum section required for any loading case shall be used.

			Percentage of Basic Unit Stress
Group I	=	$D+L+I+\beta_E E+B+SF+CF$	100%
Group II	=	D+E+B+SF+W	125%
Group III	=	Group I +LF+0.3W+WL	125%
Group IV	=	Group I +R+S+T	140%
Group V	=	Group II +R+S+T	140%

For the load factor design applied for the prestressed concrete structure, load factors and coefficients specified in article 1.2.22 Standard Specifications for Highway Bridges of 1977 AASHTO are to be added to the above loading combinations respectively.

where

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy
W = Wind Load on Structure
WL = Wind Load on Live Load
LF = Longitudinal Force from Live Load
CF = Centrifugal Force
R = Rib Shortening
S = Shrinkage
T = Temperature
SF = Stream Flow Pressure
β_E = Coefficient equal to 1.0 or 0.50 for lateral loads on a rigid frame (use whatever value governs).

4-2-3 Reinforced Concrete Design

The design is to be based on the Service Load Design Method with design and details in accordance with the AASHTO 1977 Standard Specifications.

The following design stresses are to be used:

(1) Concrete

1) Ultimate Compressive Strength

The ultimate compressive cylinder strengths at 28 days (f'_c) shall not be less than those tabulated below:

Type of Structure	f'_c kg/cm^2
Rigid Piers of Main Bridge	350
Reinforced Concrete Girder	260
Piers and Footings	260
In-situ Bored Piles	260
Abutment Structures and Sheet Piles	260

2) Allowable stress

For service load design, the stress in concrete shall not exceed the following:

a) Flexure

Extreme fiber stress in compression,

f_c .... $0.40\ f'_c$

Extreme fiber stress in tension for plain concrete,

f_t .... $0.21\ f_r$

Modulus of rupture, to be taken from tests, or in the absence of test data; For normal weight concrete

f_r .... $1.99\ \sqrt{f'_c}$

b) Shear

- Beams, one-way slabs and footings shear carried by concrete,

  v_c .... $0.25 \sqrt{f'_c}$

  Maximum shear carried by concrete plus shear reinforcement,

  v .... $v_c + 1.06\sqrt{f'_c}$

- Two-way slabs and footings shear carried by concrete,

  v_c .... $0.48 \sqrt{f'_c}$

  Maximum shear carried by concrete plus shear reinforcement,

  v .... $0.80 \sqrt{f'_c}$

c) Compression (or Combined Compression and Flexure)

The allowable axial load (or combined axial and flexural load) capacity of compression members is taken as 35% of that computed in accordance with the Provisions of Article 1.5.33 of the AASHTO 1977 Standard Specifications with the following modifications:

P_u = 2.5 x Design axial load

ϕ = strength reduction factor = 1

(2) Reinforcing Steel

Import of reinforcing steel is prohibited by the law, unless otherwise noted. All reinforcing steel of 10 mm diameter or greater is to be deformed bar conforming with the Thai Industrial Standard 24-2516 Grade SD 30 with minimum yields strength of 3,000 kg/cm^2.

The following allowable stresses have been determined in accordance with the relations between allowable stresses of reinforcing steel and material qualities which have been specified in AASHTO.

Tension in flexural member	=	1,400 kg/cm^2
Tension in web reinforcement	=	1,400 kg/cm^2
Compression in columns	=	1,125 kg/cm^2
Compression in beams	=	1,400 kg/cm^2

All reinforcing bars smaller than 10 mm diameter are to be plain bars conforming to the Thai Industrial Standard 20-2520 Grade SR 24 with minimum yield strength of 2,400 kg/cm^2.

4-2-4 Prestressed Concrete Design

Design shall be based on strength (load factor design) and on behavior at service conditions (allowable stress design) at all load stages that may be critical during the life of the structure from the time prestressing is first applied.

(1) Allowable stress

1) Concrete

a) Ultimate Compressive Strength

The ultimate compressive cylinder strength at 28 days shall not be less than 350 kg/cm^2.

b) Flexural Stress

Temporary stress before losses due to creep and shrinkage.

Compression : Pretensioned members 0.60 f_{ci}
Post-tensioned members 0.55 f_{ci}

Tension : In compressive zone 1.99 $\sqrt{f'_{ci}}$

where f_{ci} = compressive strength of concrete at the time of initial prestress.

Stress at service load after losses have occurred

Compression : 0.40 f_c

Tension : In precompressed tensile zone (Bonded) $1.59 \sqrt{f'_c}$

2) Prestressing Steel

Temporary stress before loss due to creep and shrinkage $.70 \ f'_s$

Stress at service load after losses $.80 \ f_{yp}$

where f_{yp} = yield point stress of prestressing steel

f'_s = ultimate strength of prestressing steel

(2) Load factor design

The computed strength capacity shall not be less than the largest value from the load factor design in article 1.2.22 Highway Bridge Specification of 1977 AASHTO for the loading combinations shown in article 4-2-2 of this report. The following strength capacity reduction factors shall be used:

For factory produced precast prestressed concrete members .... $\phi = 1.00$

For posttensioned cast in place concrete members .... $\phi = 0.95$

For shear .... $\phi = 0.90$

The design of shear reinforcement shall be in accordance with article 1.6.13 Highway Bridge Specification of 1977 AASHTO which is based on Load Factor Design Method.