Chapter 3

NATURAL CONDITION SURVEYS

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3.1 Natural Conditions of Can Tho

3.1.1 General Condition

Can Tho is situated in the central part of the Mekong Delta. This is an extremely flat area, only 2 to 3 meters above Mean Sea Level (MSL). Soil in this area is alluvial in nature and mostly washed and deposited from the Hau River during the flooding season. The total Can Tho provincial area is about 2,970 sq. km, of which 83% is agricultural land. Forest areas are only 1% of the total surface area.

3.1.2 Climate

The climate of the Mekong Delta, including Can Tho is typical of the monsoon weather pattern. Humid seasonal winds from the southwest prevail from May to October, and dry winds from the northeast from November to March. These two periods show a very distinct rainy season, which is associated with heavy tropical downpours. Data acquired from the Can Tho Observation Center has been analyzed, and the general weather patterns are summarized below:

(1) Temperature and Humidity

The annual average temperature of the area is 26.7°C, with a maximum of 36.5°C and a minimum of 17.7°C. The humidity averages 87% in the wet season and 77% during the dry season. (see Table 3.1, 3.2 and 3.3)

Table 3.1 Maximum Ambient Temperature

(℃)

Return period (Year)	10000	100	20	10	5	2	-	-	-	-	-
Frequency (P%)	0.1	1	5	10	20	50	70	75	80	90	95
Maximum	42.2	40.5	39.1	38.5	37.7	36.3	35.6	35.4	35.3	34.8	34.4

Table 3.2 Minimum Ambient Temperature

(℃)

Return period (Year)	10000	100	20	10	5	2	-	-	-	-	-
Frequency (P%)	0.1	1	5	10	20	50	70	75	80	90	95
Maximum	12.1	13.8	15.2	15.9	16.6	17.8	18.5	18.7	18.9	19.4	20

Table 3.3 Humidity

(%)

Month	1	2	3	4	5	6	7	8	9	10	11	12
Relative Average	82	79	77	78	83	86	85	86	87	86	85	83
Relative Minimum	32	37	30	31	27	39	49	42	46	40	31	43

(2) Rainfall

The weather pattern of Can Tho can be distinguished into two seasons; wet and dry. The wet season which prevails from May to November is characterized by having 90% of the annual precipitation with 15 - 20 rainy days per month. (see Table 3.4)

Table 3.4 Rainfall

Month Factor	1	2	3	4	5	6	7	8	9	10	11	12	Yearly
Monthly-Average (mm)	10	1	8	39	204	231	213	254	265	325	146	30	1,727
Max.-Daily (mm)	34	11	60	69	76	125	103	137	118	129	116	96	137
Monthly-Average Rainy days (day)	1.5	0.3	1.2	4.8	15.3	19.0	20.3	21.8	21.9	20.4	13.1	5.4	144.0

(3) Wind

The characteristics of wind data in Can Tho is tabulated in Table 3.5. In general, Can Tho has relatively gentle breezes of about 2.5 to 3.5m/sec. However, typhoon gusts can take place during the southwest monsoon period from June to September.

Table 3.5 Wind

Month	1	2	3	4	5	6	7	8	9	10	11	12	Yearly Max
Factor													
Vx (m/s)	14	14	14	24	21	30	31	31	28	20	18	13	31
Directions	E, NNE	SE	NE, SE	S	SE	SW, W	SW	SW	SW	W	E	ENE E	SW
Year Wise	1979 1981	1981	1979 1981	1980	1980	1970 1981	1979 1982	1979	1983	1983	1982	1977 1978	1979 1982

V_x - Maximum wind velocity in a month, and its directions

3.2 Topographic Survey

A topographic survey was conducted covering the area along the final centerline for the detailed design (shifted 220m to the downstream side from the centerline of F/S) and also covering the interchange, and service, resettlement areas.

The basic datum necessary for the topographic survey were applied according to the recent Vietnamese Standards:

(1) Horizontal datum: Ha Noi 1972

a) Ellipsoid Krasowski 1940

- Semi major axis : 6378245
- Inverse flattening : 1/298.3

b) Projection: Gauss-Kruger Grid system

- Latitude of origin : 0°00'00"
- Central meridian : 105°00'00"
- False Easting : 500,000m
- False Northing : 0
- Scale factor at central meridian : 1

(2) Vertical datum: Mean Sea Level (MSL) at Hon Dau Island (National Bench Mark system). This vertical datum used for topographic survey and also for hydrological and hydraulic analysis.

(3) Scope of survey work:

a) Primary control survey : 12 GPS points
b) Secondary control survey : 177 points of traversing network
c) Detailed survey
- Bridge and Right of Way (R.O.W.): 15.35 km x 100m = 154ha map in total at a scale of 1:1000
- Main structure (Bridge and Service Area) : 78.8 hr (1:500)
- Interchange : 106.3 hr (1:1000)
- Resettlement Area : 29.7 hr (1:500)
- 785 cross sections with 200m width and 15.35km longitudinal profile leveling
- Centerline survey for Demarcation and ROW of Land acquisition: 634 pegs

3.3 Collection of Meteorological Data

(1) The meteorological data was collected mainly of Can Tho station for five items:

- Wind velocity and directions
- Precipitation
- Air temperature
- Humidity
- Evaporation

(2) The wind data also collected of Soc Trang station (Downstream of Hau river from Can Tho city about 60km). The data was recorded from 1978 to 1998 and from 1949 to 1998 respectively. The design wind velocity was statistically treated based on this data. The basic wind velocity was 40m/sec, and other calculated outputs were as follows:

Table 3.6 Statistical Analysis Result of Wind Data

Statistical Method	Can Tho station (1978-98) (m/sec)	SocTrang station (1949-98) (m/sec)
a. Iwai's method	35.6	30.2
b. Gringorten's method	35.2	24.5
c. Gumbel's method	37.5	29.8

- Basic Wind Velocity : U = 40 m/sec
- Classification of Ground surface : II
  (Japanese Standards for Aerodynamic Stability, 1992)
- Altitude of a Target (Main Girder) : 42.8m (40m<Z<45m)
- Factor by altitude E : 1.26
- Design Wind Velocity : U = U x E = 50.4m/sec

3.4 Seismic Intensive Data

For the Detailed Design the Can Tho Bridge - a major bridge in Viet Nam - with the Design Frequency P = 1%, a report on the Seismic hazard assessment of the Bridge area has been compiled by the Department of Seismology, Institute of Geophysics of Viet Nam National Centre for Natural Science and Technology in July 1999.

The report consist of the following:

(1) Tectonic features of Can Tho region and adjacent areas to serve a basis for seismic hazard assessment.

(2) Seismic activity in this region and its relation with the tectonic

structures to serve a basis for delineation of seismic source zones and estimation of their seismic characteristics.

(3) Seismic hazard assessment consisting of:

- Compilation of seismic earthquake map and map of seismigenic faults
- Estimation of maximum seismic intensity I_{max} in MSK or MM scales and maximum acceleration (PGA) a_{max} and calculation of seismic intensities and acceleration with different return period T = 72, 145, 475, 950, 4750, 9500 ... years

3.5 Soil and Material Surveys

3.5.1 Regional Geology

(1) Geomorphology

Project area is located in the central part of Mekong Delta which is a typical alluvial fan plain.

In the short term the geomorphology can be described as a vast relic of Quaternary Lagoon. That is to say, very wide low land and numerous streams have appeared presently as a result of Quaternary Sea progression/ regression.

Also, in this region, floods have occurred periodically and many swamps are formed along the riverside.

(2) Stratigraphy

The base rock layers are from Jura-Creta to Neogene igneous and sedimentary rocks (granite, andesite, and sandstone, etc.) which are found around Cambodian border and northern region of Ho Chi Minh City (HCMC).

According to the geological map published by geological survey of Viet Nam, base rock is not found in the Project area down to elevation minus 250m.

As a consequence, major subsoil type of the region is marine and/or river origin sediments of quaternary, which is divided into five strata.

Upper stratum consists of river and/or marine fine soil of Holocene in geological time. On the other hand, lower stratum consists of marine sand of Pleistocene in geo-time which is named Long My and Moc Hoa Formation in the geological map mentioned above.

(3) Tectonics

Referring to general geological section published by geological survey of Viet Nam, the strata form basin structure in the Project area. The grade of basin curve is very gentle, however, it becomes deeper toward the Can Tho city area. In other words, alluvial (Holocene) sediments become deeper near the Project area.

Two fault system are assumed, one is northeast direction fault and the other is southeast direction one which just corresponds to the Hau River course in Can Tho City. Geological time of the faults is assumed Neogene Teriary (over 10^6 years ago).

3.5.2 Geotechnical Investigation

(1) Investigation Work

Boring with Standard Penetration Test (SPT). Cone Penetration Test (CPT), Pressure Meter Test in the field and soil test in the laboratory have been carried out.

Detailed investigation items are as follows:

a) Boring with SPT

- Hole No. D1 to D29 29 holes 2373m

b) Cone Penetration Test (CPT)

- No. CPT1 to CPT 55 points 550m

c) Pressure Meter Test 6 nos

Hole No	Depth	Hole No	Depth	Hole No	Depth
D12	50m	D13	50m	D14	50m
D12	70m	D13	70m	D14	70m

d) Laboratory Soil Test

- Undisturbed Sample from Bore Hole
- Disturbed Sample from S.P.T

Test Item and Specification are shown in Table 3.7

Table 3.7 Test Item and Specification

Test Item	Specification
Grain size analysis	ASTM D422
Specific gravity	ASTM D854
Unit weight	ASTM D854
Water content	ASTM D854
Atterberg limit	ASTM D4318
Unconfined compressive strength	ASTM D2166
Triaxial compression UU	ASTM D2850
Triaxial compression CU	ASTM D4767
Consolidation	ASTM D2435
Permeability	ASTM D2435

(2) Result of Boring and Cone Penetration Test (CPT)

As a result of boring and CPT composition of the layer of the Project site, the subsoil profile was prepared. It consists of the following 9 layers from up to downward.

a) Layer Rd

It is the recent river bed sediment which consists of sand and clay. Especially on the surface of riverbed a thin floating sand layer, 1m to 2m thick is found which is now dredged as the construction material.

b) Layer C1

On the land surface, gray color pure clay layer is distributed at the level of 10m to 30m deep. The clay sedimented during the period of Holocence marine progression. It is very soft, high plasticity and N-value is counted less than 2 (nearly self-down).

c) Layer C2

Continuous to the layer C1, in elevation -10m to -40m, brownish gray color clay lies about 30m deep. From grain size viewpoint it consists of silty clay and partly bears a little quantity of sand. Its consistency is soft to medium stiff in which N- value is counted 3 to 42 (average 16).

d) S/St

This layer lies dominantly with lens like distribution in the eastern side (HCM side) of the bridge site at the level of -10m to -50m. Fine sand and silt are mixing irregularly, therefore we assume that its sedimentary environment was old river course or flood area near the river (back marsh). The layer is not uniform and the feature changes in accordance with the distribution of sand. From the viewpoint of consistency, it is relatively hard and dense and N-value changes between 4 and 40 (average 23).

e) Layer St/C-1

Reddish light brown silty clay with a little sand lies in elevation -40m to -50m, transiting from clay layer C2. Residual soil (weathered soil in land) is included in large quantity, which suggest that sedimentary environment was near the land in the past time during sea regression. This layer has inhomogeneous face. As a result, N-value varies from 13 to 60 in proportion to sand content.

f) Layer S1

This layer lies in elevation -70m to -95m, and consists of in majority brownish color fine sand. Besides, silt and/or clay are sporadically interlayed. It is assumed that sedimentary environment was shallow sea or border between land and sea. N-value is 60 or more.

g) Layer St/C-2

This layer 15m to 30m thick interlies between sand layer (S1 and S3) in elevation -95m to -110m and thickness increases toward the western side (Can Tho side), and consists of silty clay with light yellow, reddish brown color. Hard soil is the feature in which semi-rock hardness is shown in some parts. N-value is from 42 to over 60.

h) Layer S2

This sand layer about 10m thick lens which exists in elevation -100m to -110m near the east river bank (HCMC side). It is interlaid between silty clay layer (St/C-2) according to borehole No.12. Yellowish and light brown color fine sand is main soil type and N-value is over 50.

i) Layer S3

This is distributed widely in elevation -110m below, and consists of homogeneous fines sand of marine origin with yellowish colour and N-value exceeds 60 (very dense) through the whole layer.

SPT result is summarized in each layer, as shown in Table 3.8.

Table 3.8 N-value by SPT

Layer	N-Value	
	Range	Average
Rd	2 ~ 8	4
C1	0 ~ 12	2
C2	3 ~ 42	16
(S/St)	14 ~ 40	23
St/C-1	13 ~ 60+	35
S1	60+	60+
St/C-2	42 ~ 60+	55
S2	60+	60+
S3	60+	60+

Note: 60+: Over 60

(3) Result of Pressure Meter Test

In order to examine the deformation character of the layer, pressure meter test was carried out. The result is summarized as shown in Table 3.9.

Table 3.9 Result of Pressure Meter Test

Test Location Hole No.	Depth (m)	EL (m)	Layer	Ep (Kgf/cm^2)	N
D12	50	-49	S/St (C2)	130	30
D12	70	-69	St/c -1	95	23
D13	50	-52	St/c -1	110	29
D14	50	-67	St/c -1	150	40
D13	70	-72	S1	275	>60
D14	70	-87	S1	280	>60

In general, there is an empirical formula E = 7N in the relation between Ep and N. The present test result shows a little small values of Ep compared with 7N.

(4) Result of Soil Test

The 90 undisturbed samples were collected and laboratory test was carried out. Individual test data are summarized in Figure 3.1 and Figure 3.2. Principal indexes are shown in accordance with depth. Those indexes picked up are qu, c, ϕ, E_{50}, Pc (Preconsolidation Pressure) and Cc (Compression Index). Soil type is classified in accordance with unified soil classification system, ASTM D2487.

3.5.3 Construction Material Investigation

(1) Road Embankment Material

a) Material and Production Locality

River sand is found available for road embankment at six locations around the Project site (See Table 3.10 and Figure 3.3).

Table 3.10 Locations of River Sand Production

Location	Distance to Project Site	Type of Sand
Hau Giang No.1	1Km downstream	Fine Sand
Hau Giang No.2	5Km downstream	Fine Sand
Dai Ngai	50 Km downstream	Fine Sand
Tra Ech	20 Km downstream	Silty Sand
Long Xuyen	60 Km upstream	Medium to Coarse Sand
Tan Chau	200 Km upstream	Coarse Sand

Among then, fine sand from Hau Giang No.1 to Tra Ech are used for subgrade material and Long Xuyen, Tan Chau sand are available for sand mat material.

b) Soil Properties for Embankment

Laboratory test was carried out in four types of river sand, i.e., Hau Giang No.1, Hau Giang No.2, Dai Ngai and Tra Ech. In addition, considering side borrow use, clay of the river bank was also examined.

Test items and result are summarized in Table 3.11. Main characteristics are shown as follows:

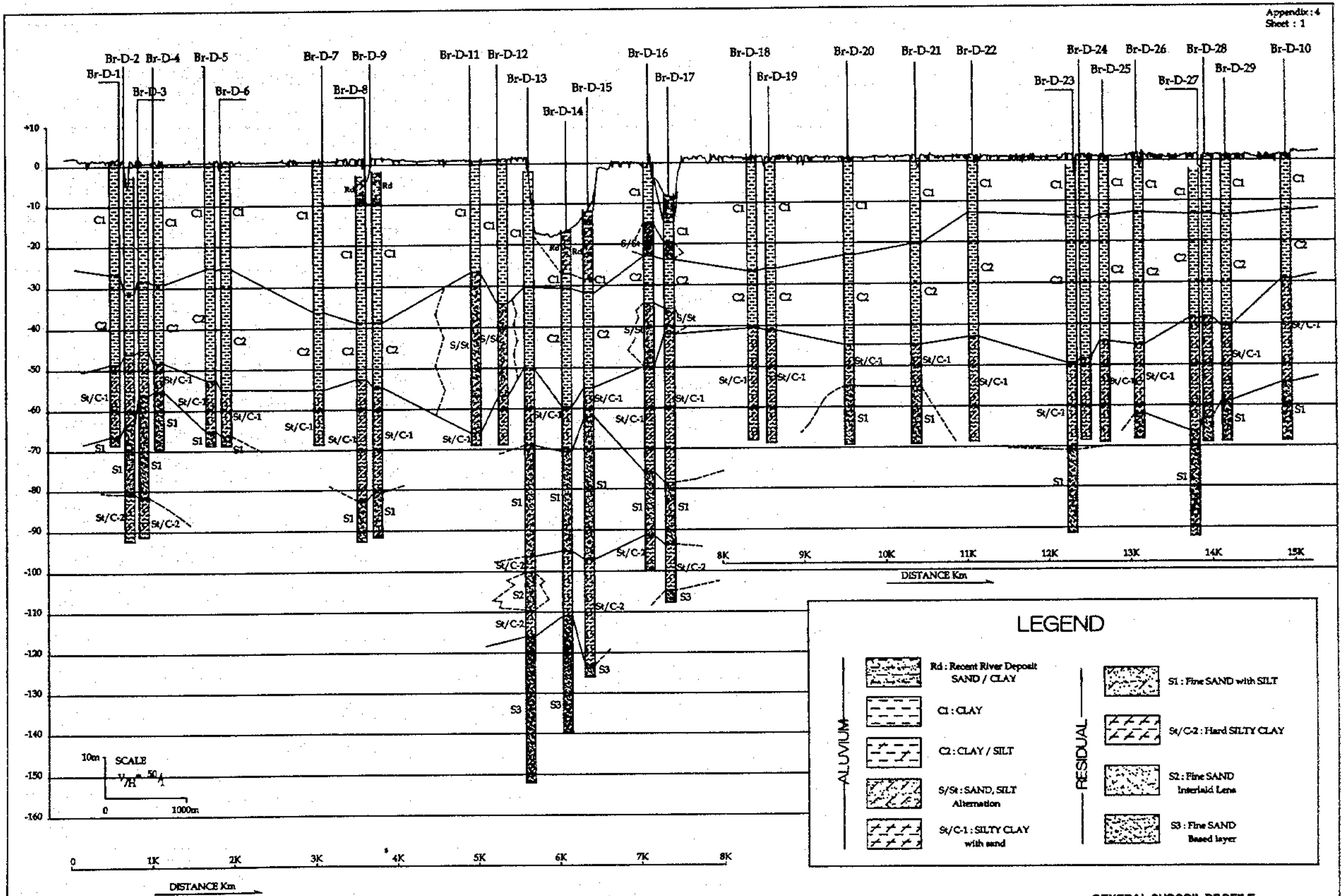
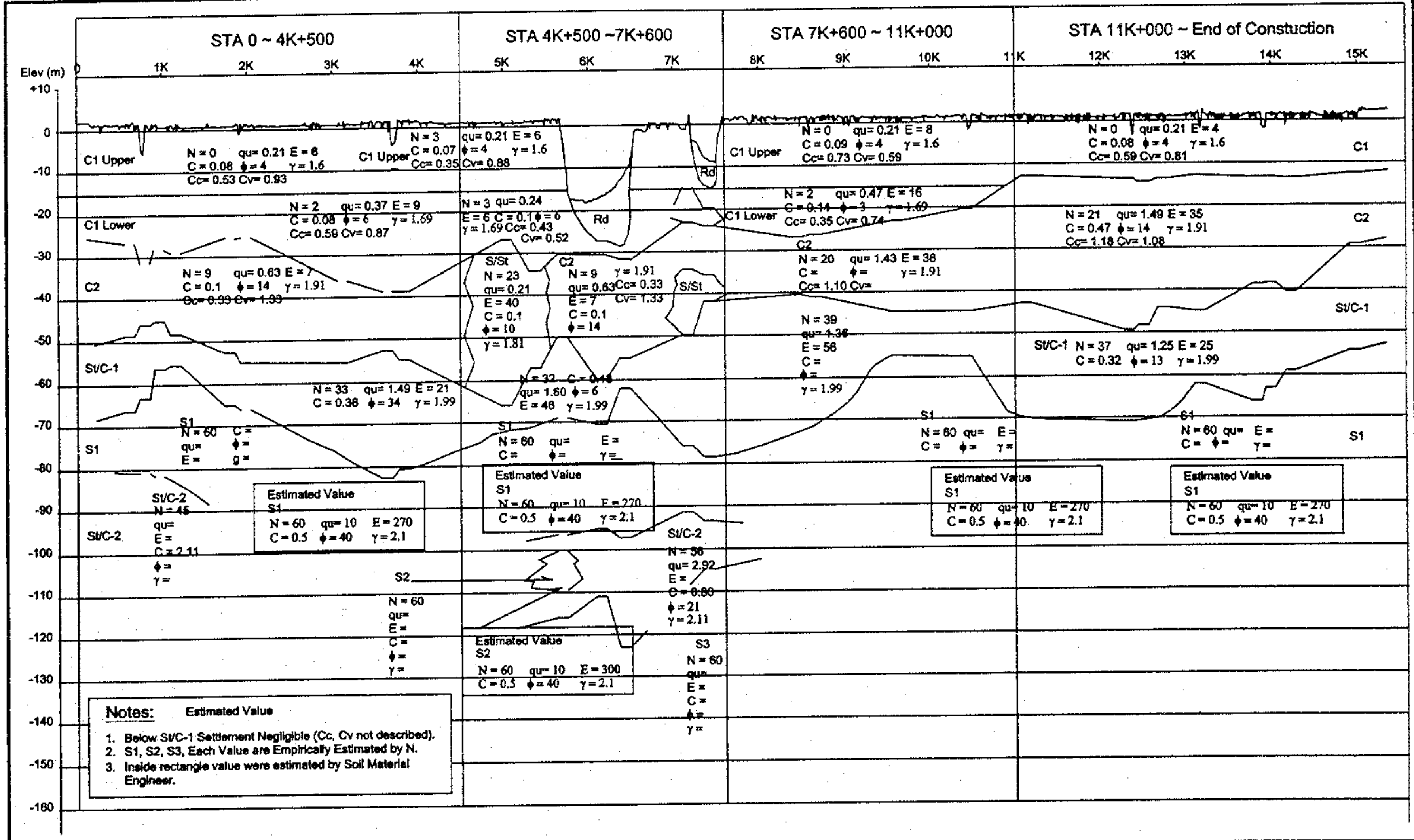


Figure 3.1 General Subsoil Profile



THE DETAILED DESIGN OF
THE CAN THO BRIDGE CONSTRUCTION
IN SOCIALIST REPUBLIC OF VIET NAM

Figure 3.2 Engineering Properties in the Soil Layer

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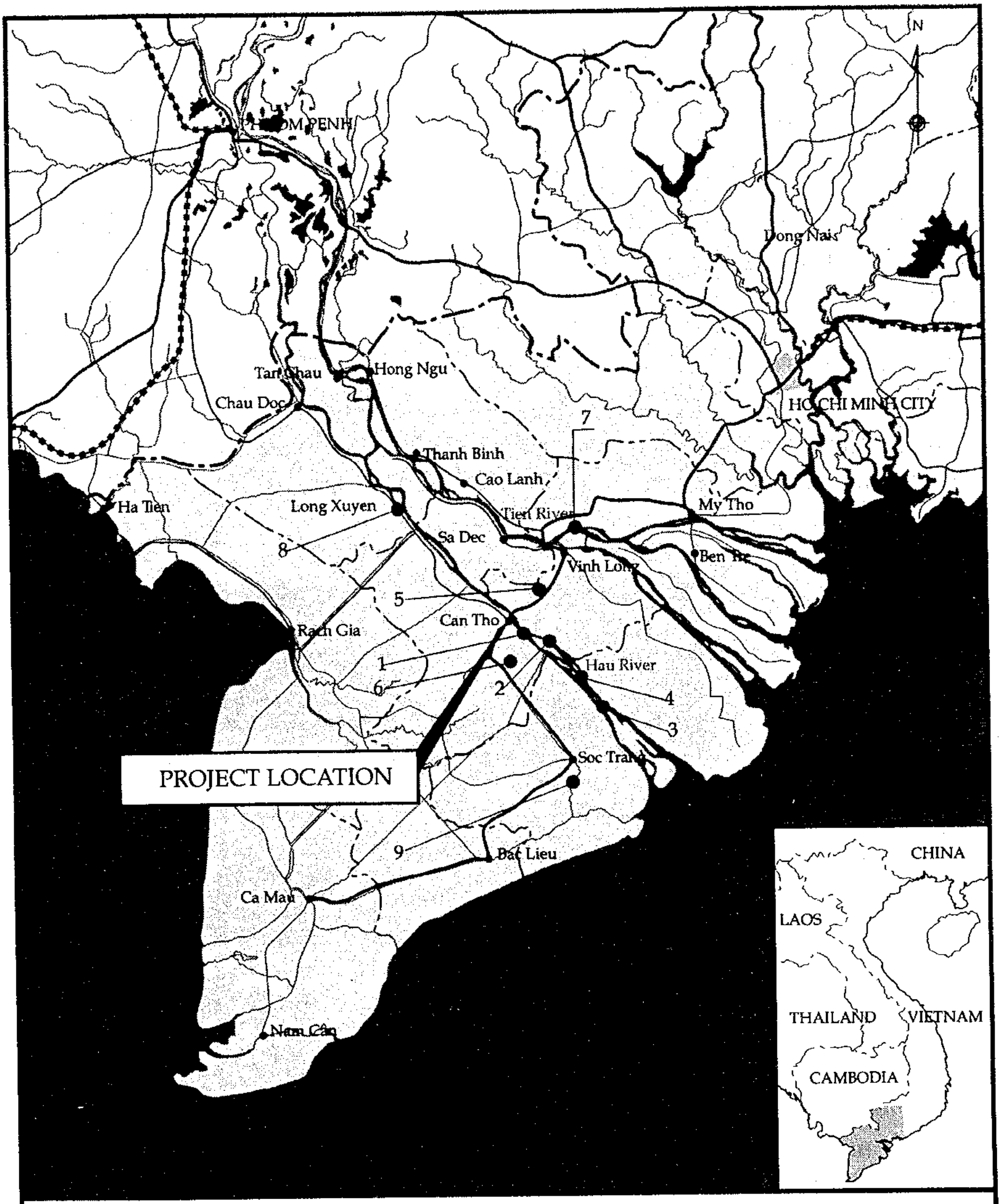


1. Hau Giang River Sand 1
2. Hau Giang River Sand 2
3. Dai Ngai River Sand
4. Tra Ech River Sand
5. Side Filling Soil
6. Side Filling Soil
7. Vinh Long River Sand
8. Tra Vinh Ridge Sand
9. Soc Trang Ridge Sand

THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.3 Location of Embankment Materials
	JAPAN INTERNATIONAL COOPERATION AGENCY

Table 3.11 Summary of Soil Properties (Embankment Material)

Location No.	Soil Type	Grading (%)				Atterber Limits			Compaction			Soaked CBR			Triaxial Test (UU)		
		4.75 to 2 mm	2 to 0.425 mm	0.425 to 0.075 mm	< 0.075 mm	LL %	PL %	PI %	MDD g/cm^3	WD g/cm^3	OMC %	95% left side	MDD	95% right side	95% MDD	φ Degree	C kg/cm^2
1	River Sand	-	1	96.8	2.2	-	-	-	1.56	1.86	19.7	9.6	19.7	23.5	1.48	32° 25	0.2
2	River Sand	-	0.5	96.5	3	-	-	-	1.54	1.83	18.7	12.6	7.8	23.7	1.46	32°27	0.14
3	River Sand	-	8	80	12	-	-	-	1.59	1.86	17	10.6	13.2	9.4	1.51	32° 9'	0.34
4	River Sand	-	6	62	32	21.3	17.7	3.6	1.88	2.13	13.5	6.1	13.8	12.4	1.79	29° 39'	0.75
5	Clay	4	6	8.6	81.4	57.2	32.4	24.8	1.46	1.85	27	0.8	2.1	1.84	1.39	-	-
6	Clay	-	1	13	86	40.4	24.6	15.8	1.64	1.96	19.5	0.94	3.7	1.9	1.36	-	-

Notes:

1) PL Plastic Limit
2) UU Uncosolidation Undrained
3) WD Wet Density
4) MDD Max Dry Density
5) OMC Optimum Moisture Content
6) LL Liquid Limit

- Physical Elements

  Fine grain sand is major and grading is poor, especially in the Tra Ech sand, it contains 32% silt particle, and its nature will be similar to clay soil.

- Effects of Compaction

  Optimum Moisture Content (OMC) is about 19% in Hau Giang and Dai Ngai sand, while the value of Tra Ech sand is 14%. Regarding density Maximum dry density (MDD) and Maximum wet density (MWD) are, respectively, 1.60 and 1.90 in the Hau Giang and Dai Ngai sand. In case of Tra Ech sand, MDD and MWD are, respectively, 1.88 and 2.13.

c) CBR

Soaked CBR values which were measured on the condition of 95% compaction are: Hau Giang Sand 7 to 9, Dai Ngai Sand 9, and Tra Each Sand 8 to 12.

d) Strength

Cohesion (C) Internal friction angle (ϕ) were examined by means of triaxial compressive test. For this purpose, samples are prepared on 95% compaction. The value of C, ϕ are shown as follows:

$C = 0.1 \sim 0.3$ (kgf/cm^2), $\phi = 32°$, in Hau Giang and Dai Ngai Sand

$C = 0.75$ (kgf/cm^2) ϕ - 29°, in Tra Ech Sand

e) Geotechnical Considerations for Design

- Grain Size of River sand

  Grain sizes concentrate to under 0.425mm, therefore for practical use, it is desirable to mix coarser material. Especially in case of Tra Ech Sand, it is constitutionally near clay material, so it needs to be combined with another coarse material.

- CBR for Design

  CBR value obtained form laboratory test was 7 to 12 (average 8) on the condition of 95% of OMC. However, in the actual construction, control of water content is supposed to be difficult, therefore as for design value, the values of the laboratory test must be considered conservatively.

- Strength

  The value of Cohesion (C) is 0.2kgf/cm^2 (Hau Giang and Dai Ngai Sand) and 0.75 kgf/cm^2 (Tra Ech Silty Sand) on the condition of 95% OMC. Considering the rainy condition in the construction site and period, it is difficult to control water content to the 95% OMC. That is, decrease of C is required for design purpose.

- Sand Mat Material

  Coarse sand is collected near Long Xuyen City and Tan Chau District. Finest Modulas of them are 2.3 to 2.8 and those are available for sand mat material.

- Side Borrow

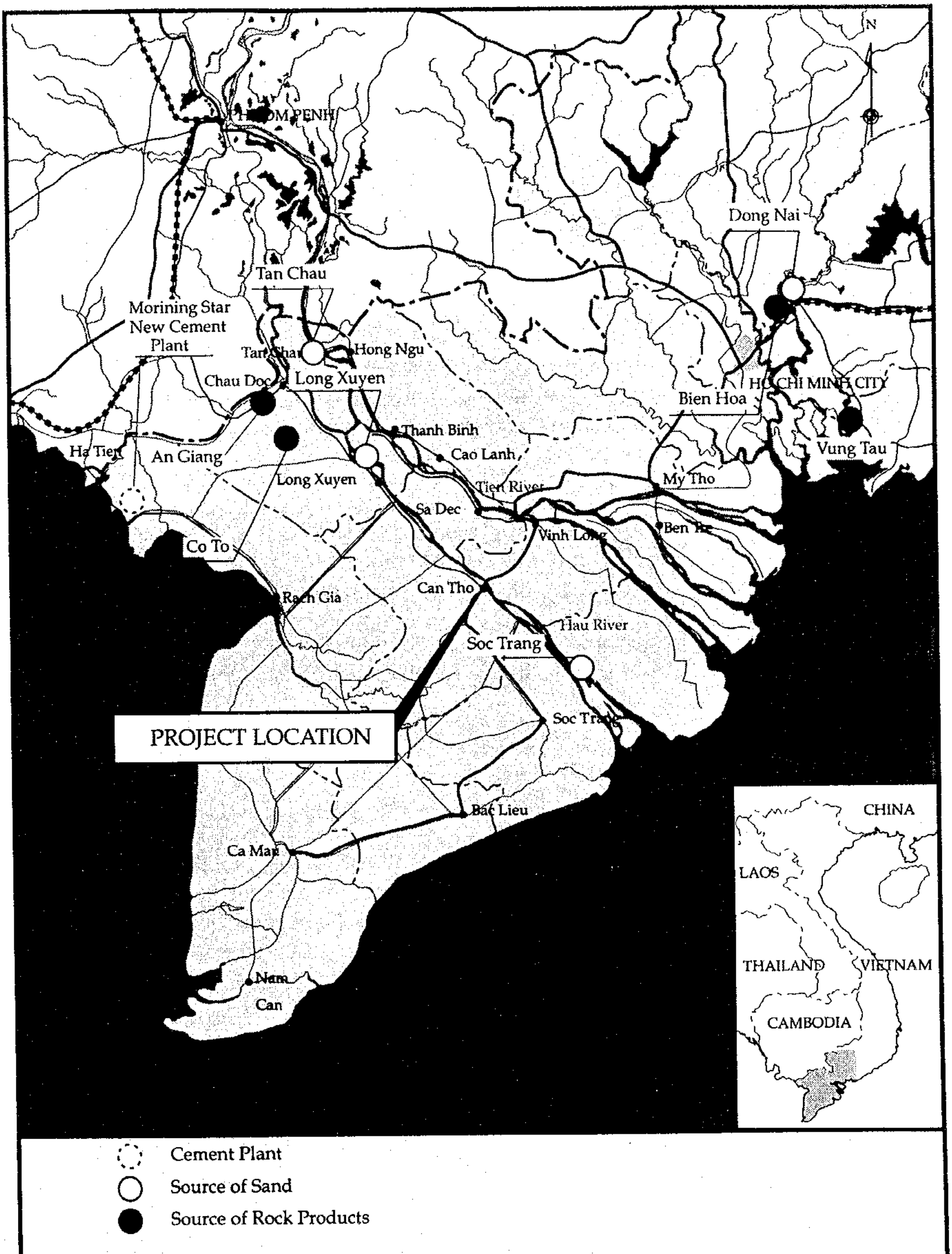
  Since the soil of side borrow is very high water content, and soft clay it is not suitable for embankment.

(2) Pavement Material

a) Subbase and Base Course Material

- Material Type and Locality of product

  Currently around the Project area, the following materials are used for subbase and base course. Locations of quarry and sand borrow are shown in Table 3.12 and Figure 3.4.



THE DETAILED DESIGN OF THE CAN T1GE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.4 Location of Aggregates Sources JAPAN INTERNATIONAL COOPERATION AGENCY

Table 3.12 Location of Subbase and Base Course Materials

Name	Material Type	Distance to the project site (by ship)
1. Bien Hoa Quarry	Crushed rock Sandstone	300km
2. Vung Tau Quarry A	Crushed rock Granite	200km
3. Vung Tau Quarry B	Crushed rock Andesite	200km
4. Coto Quarry	Crushed rock Granite	200km
5. Dong Ngai Sand Borrow	River Sand	300Km
6. Long Xuyen Sand Borrow	River Sand	60km
7. Tan Chau Borrow	River Sand	200km

The rocks and sands are now widely used for road construction in Viet Nam and quality is regarded to be good, although granite is apt to be cracked into flat rectangular shape by latent joints.

b) Aggregates for Asphalt Surface Course

Crushed rocks and river sand mentioned above are also used for asphalt surface course. Since those are commonly used in the local road construction, the quality is regarded to be good.

Appropriate preparation of grain size is key point in the actual construction. That is, suitable mixture of grain size is required in the construction. For that purpose, normal arrangement of crushing system should be established.

(3) Concrete Material

a) Cement

Three local cements are available for the construction. They are Chin Fong Cement (Produced in Hai Phong City), the other is Morning Star Cement (Produced in Kien Giang Province), and Nghi Son Cement to be available in 2000 in Northern Viet Nam.

b) Aggregates

- Type and Locality for Concrete

The following type and locality for concrete aggregate in Table 3.13 are available.

Table 3.13 Aggregates for Concrete

Name	Material Type	Distance to Project Site (by River)
<Coarse Aggregates>		
Coto Quarry	Crushed Rock Granite	200km
Vung Tau Quarry	Crushed Rock Granite	250km
Hoa An Quarry (Bien Hoa)	Crushed Rock, Andesite Sandstone	300km
<Sand>		
Tan Chau	River Sand	200km
Dong Nai	River Sand	300km

- Engineering Properties

Principal properties for concrete aggregates were examined in laboratory test and especially in case of granite, alkali reaction index was tested. With regards to Bien Hoa quarry rock and Dong Nai River sand, test was omitted, because those are very popular aggregates in this area and the quality is assured.

Test result is shown in the Table 3.14 and Table 3.15, and the characteristics of representative localities are summarized as follows:

i) Coto Granite

Although abrasion loss is rather big, as a whole, quality is fairly good for concrete aggregate and regarding alkali reaction factor, it is judged to be harmless.

As a matter to be checked, poor balance of grain size distribution is pointed out. Coarser size of 20mm and over is dominant in its distribution. The reason is that a smaller size crusher is not installed in the quarry. Therefore in order to produce specified grading aggregate, a normal crushing system (secondary crushers) needs to be established.

ii) Tan Chau River Sand

It is well graded in grain size. Coarse particle is included moderately (FM = 2.8 to 3.0), therefore it is regarded as a good sand for concrete.

Table 3.14 Engineering Properties of Coarse Aggregates

	Grain Size (% Passing)							Fineness	Water	Bulk	Specific	Water	Compressive	Soundness	Abrasion	Solute	Remarks
	38.1	25.4	19.05	12.7	9.52	4.76	2.38	Modulus	Content	Density	Gravity	Absorption	Strength	(2 cycles)	Los Angeles	SiO_2(%)	
1* COARSE AGGREGATE																	
VUNG TAU	100	100	88.7	34.8	6.5	0.5	0.4	7.69	0.49	2.627	2.697	0.98	936.8	0.054	35.0	24-	
COTO	100	100	84.6	36.6	8.8	0.3	0	7.70	0.37	2.653	2.711	0.82	1245.4	0.034	33.4	N.A.	
HOA AN	100	100	80.5	17.2	2.5	0.6	0.6	7.99	0.49	2.822	2.912	1.06	1647.8	0.03	15.0	12-	

Table 3.15 Engineering Properties of Fine Aggregates

	Grain Size (% Passing)							Fineness	Water	Bulk	Specific	Water	Compressive	Soundness	Abrasion	Solute	Remarks
	9.52	4.76	2.38	1.19	0.63	0.315	0.15	Modulus	Content	Density	Gravity	Absorption	Strength	(2 cycles)	Los Angeles	SiO_2(%)	
2* SAND (FINE AGGREGATE)																	
DONG NAI	100	98.8	94.6	78.6	34.4	4.3	0.6	2.85	0.60	-	2.642	1.456	N.A.	0.291	N.A.	7.5 -	
TAN CHAU	99.6	96.7	88.2	68.6	37.3	13.4	2.7	3.093	0.44	-	2.643	1.407	N.A.	0.165	N.A.	4.0 -	

3.5.4 Trial Mix

In this Project, high strength concrete (target qu = 600 kgf/cm^2) is required for the P.C girder. For its purpose trial mix of concrete was carried out. Both Chin Fong and Morning Star cement with combination of 3 kinds of coarse aggregate and Tan Chau, Dong Nai sand.

(1) Trial Mixing

The objective of the test to expect suitable cement quantity and mixing proportion. The test was carried out in variation of cement quantity from 450kg/cm^2 to 520kg/cm^2 and W/C ratio about 30%. As a result, it was found out that qu = 600 (kgf/cm^2) is possible with the cement quantity about 500 kg/m^3.

(2) Mix Proportions

Table 3.16 Mix Proportion of Concrete Using Chin Fong Cement

Case of mixing	Max. Size of coarse aggregate (mm)	Water cement ratio W/C (%)	Sand aggregate ratio S/A (%)	Unit content (Kg/m^3) Water (W)	Cement (C)	Fine aggregate (S)	Coarse aggregate (G)	Water reducing agent (g)	Slump (cm)	Air content (%)	Temperature
A	20	28.0	40.0	150	535	695	1,065	6,688	1.5	2.0	29
B	20	30.0	40.4	150	500	714	1,082	6,250	2.5	2.0	29
C	20	32.0	39.6	150	469	711	1,112	5,863	7.0	2.0	30

Table 3.17 Mix Proportion of Concrete Using Morning Star Cement

Case of mixing	Mas. Size of coarse aggregate (mm)	Water cement ratio W/C (%)	Sand aggregate ratio S/A (%)	Unit content (Kg/m^3) Water (W)	Cement (C)	Fine aggregate (S)	Coarse aggregate (G)	Water reducing agent (g)	Slump (cm)	Air content (%)	Temperature (C)
A	20	28.0	40.0	150	535	695	1,068	6,688	9	2.0	30
B	20	30.0	40.4	150	500	711	1,076	6,250	13	2.0	30
C	20	32.0	39.6	150	469	708	1,106	5,863	18	2.0	29

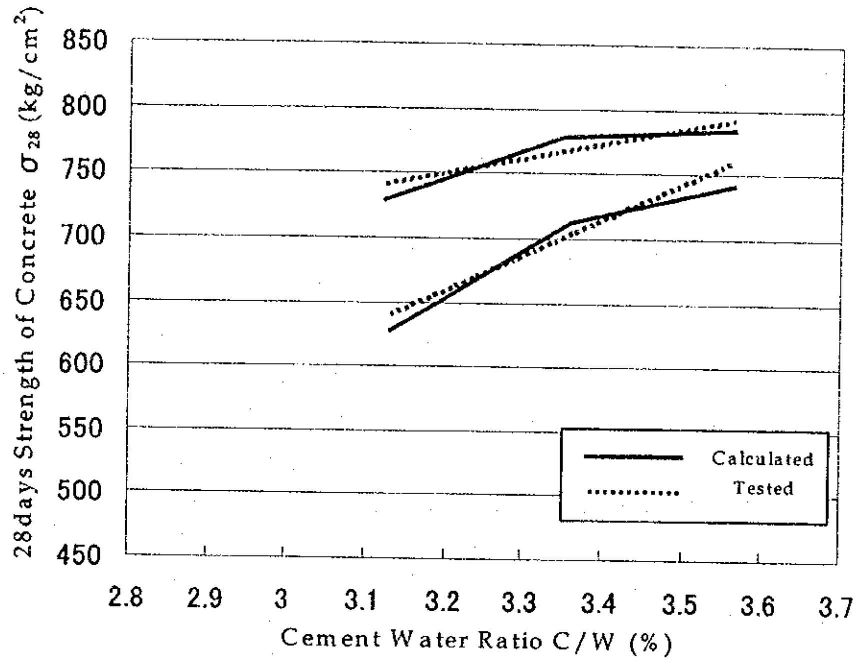


Figure 3.5 Comparison between Calculation and Test

(3) Conditions for Mix Proportion

- Target strength of trial mixing concrete is 600 kg/cm^2 in relation to the required design strength of 500kg/cm^2.

- The cement used for trial mixing was Chin Fong and Morning Star cements

- Fine aggregate and coarse aggregate for trial mixing were from Coto and Tan Chau, respectively.

- Trial mixings were three cases in accordance with the mix proportion for each type of cement.

(4) Results of Compressive Strength Test

Table 3.18 Compressive Strength Test (3 days)

Mixing Case Number	Type of Cement	3 days				
		Strength of Test Piece (kgf/cm^2)				Average (kgf/cm^2)
A	Chinfong Cement	652	566	625	578	605
	Morning Star Cement	519	507	520	503	512
B	Chinfong Cement	633	623	628	636	630
	Morning Star Cement	606	596	610	591	601
C	Chinfong Cement	606	583	603	523	597
	Morning Star Cement	566	567	554	573	565

Table 3.19 Compressive Strength Test (7 days)

Mixing Case Number	Type of Cement	7 days				
		Strength of Test Piece (kgf/cm²)				Average (kgf/cm²)
A	Chinfong Cement	655	690	691	685	680
	Morning Star Cement	628	608	609	611	614
B	Chinfong Cement	664	657	693	681	674
	Morning Star Cement	535	526	521	543	531
C	Chinfong Cement	666	668	661	656	663
	Morning Star Cement	491	508	516	505	505

Table 3.20 Compressive Strength Test (28 days)

Mixing Case Number	Type of Cement	28 days				
		Strength of Test Piece (kgf/cm²)				Average (kgf/cm²)
A	Chinfong Cement	757	786	776	786	776
	Morning Star Cement	736	745	746	739	742
B	Chinfong Cement	764	756	778	756	764
	Morning Star Cement	702	696	697	704	700
C	Chinfong Cement	712	734	730	737	728
	Morning Star Cement	626	627	616	623	623

(5) Conclusions

Based on this compressive strength of concrete and through the laboratory testing, the following items are concluded.

- The case of mixing (C-mix) proportion (with cement content of 469kg/m^3) can reach the target strength of 600km/cm^2 for each use of Chin Fong and Morning Star cement.

- The compressive strength in case of Chin Fong cement was higher than that of Morning Star cement, however, its slump was 7cm on average for the case - C proportion. It is suggested to increase air - entraining and/or water - reducing agent to obtain the greater slump of 10cm to 15cm.

- To obtain more accurate results, it is suggested to equip much more suitable facilities for testing the compressive strength of concrete for the future performance.

3.6 Plan of Bearing Capacity Test

3.6.1 Bearing Capacity Tests for Foundations

The direct confirmation of bearing capacity for the foundations is vitally important. The various test methods have advantages and disadvantages from the technical and economical viewpoints. The methods that can be applied for the driving and in-situ piles, and caisson foundation will be as follows:

(1) Wave Equation Method

(2) Dynamic Formula Method (ENR formula)

(3) Rapid Loading Test (Berming Hammer Method)

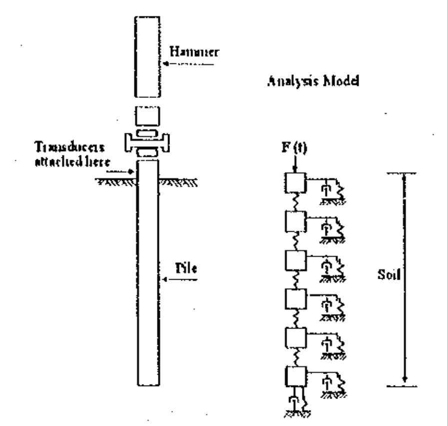
(4) ASTM Standard (D1143) with Weighted Box or Platform

(5) In-situ Loading Test using Self-boring Instrument

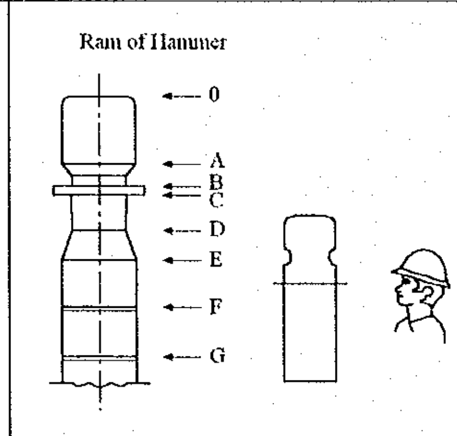
(6) Osterberg Cell Method

For the pile foundations of the bridge which will be constructed in the approach road sections, the methods from (1) to (4) can be used subject to the type of pile foundations. As for the foundation of the main bridge. The combination of (5) and (6) methods can be used for the foundations of the main bridge. The measured results from (5) In-situ Loading Test using Self-boring Instrument will be referred for final judgement comparing with the results from the method (6).

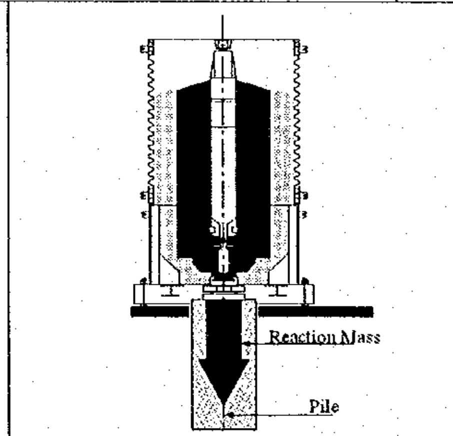
Loading Tests	Way of Test	Features of Test	Application
(1) Wave Equation Method	- Monitor force and impact velocity for hammer through strain transducer installed on pile. The records of force and velocity are processed using a Pile Driving Analyzer (PDA).	- Can be run on a personal computer - Useful construction control - Estimate pile capacity with minimized empirical judgement	- Driving pile - Comparatively shallow (10 ~ 30 m)
(2) Dynamic Formula Method (ENR formula)	- The idea come from the fact that a known energy input from the pile hammer must be related to the soil resistance multiplied the pile penetration.	- Allowable pile load: $Q_A = \frac{2W_H H}{S+C}$ - Simple instrument and easier observation - A greater amount of experience	- Driving pile - Comparatively shallow (10 ~ 30 m)
(3) Rapid Loading Test (Berming Hammer Method)	- The dynamic force produced by reaction mass (20G acceleration) is acted on the pile. - Stress under this loading is the same condition as static loading.	- Rapid and effective loading - Cost minimal without reaction piles - High availability above the water without the special preparation - Easier construction control	- Driving and in-situ piles - Comparatively shallow (10 ~ 30 m)
(4) ASTM Standard (D1143) with Weighted Box or Platform	- A known test weights or loading material (such as concrete blocks) placed on the box or platform be applied directly on the pile or pile cap.	- Simple and direct measurement of the pile capacity - Weights preparation is large and heavy	- Driving and in-situ piles - 15 ~ 45 m deep
(5) In-situ Loading Test using Self-boring Instrument	- Place the testing plate (ex. 90 mm) at the bottom of drilling hole and drills into the fresh soil. - Load through the load-cell.	- Small testing plate size (ø9 ~ 30 cm) gives conservative volume against the large size foundation (caisson foundation). - Size effect of the testing plate should be carefully treated.	- Bearing capacity in the deep soil layer - 15 ~ 100 m deep
(6) Osterberg Cell Method	- Install the load-cell at the bottom of foundation - By pressure fluid (mainly water), the piston in the pressure chamber moves down. - Displacement of the piston measured through dial gauges.	- Simple and smaller test equipment including load-cell - Can be used for both driven pile and in-situ pile.	- In-situ foundations - 15 ~ 100 m deep



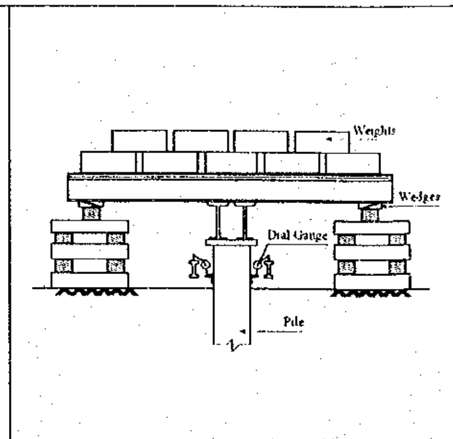
(1) Wave Equation Method



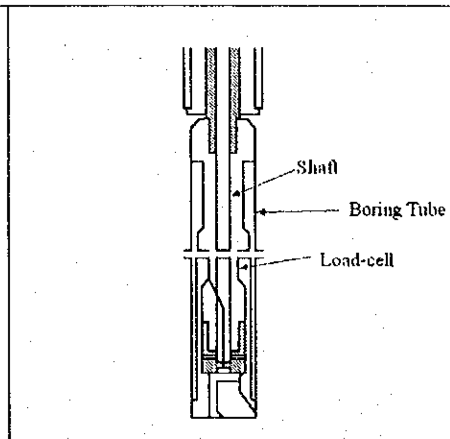
(2) Dynamic Formula Method (ENR formula)



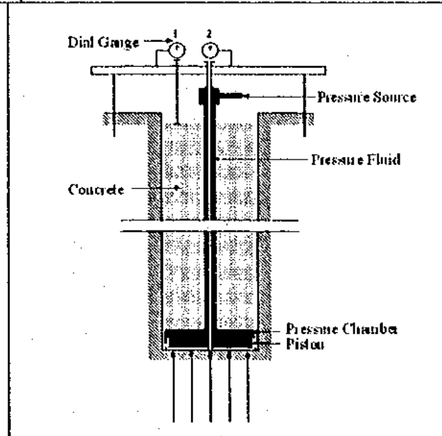
(3) Rapid Loading Test (Berming Hammer Method)



(4) ASTM Standard (D1143) with Weighted Box or Platform



(5) In-situ Loading Test using Self-boring Instrument



(6) Osterberg Cell Method

Figure 3.6 Bearing Capacity Tests for Foundation

3.7 Flood Surveys and Analysis

3.7.1 Flood Surveys

(1) Flood Phenomenon

There is a high variation in the water flow in the rivers of the Mekong Delta due to well defined wet and dry periods. During the wet season, the river water increases and large scale inundation occurs in the north. Generally, flooding in this area starts in July or August, and extends up to southern part of the delta, and in the areas where the drainage facilities are poor and water logged for as long as five months. The maximum discharge of Can Tho and My Thuan station are shown in Table 3.21.

Table 3.21 Historical max. Discharge of Can Tho and My Thuan Station

Year	Can Tho (m/s)	My Thuan (m/s)
1977	19,800	21,000
1978	27,200	22,600
1979	20,700	17,100
1980	23,400	19,773
1981	27,300	17,600
1982	21,600	19,100
1983	22,669	19,773
1984	24,100	24,400 (Max)
1985	25,400	20,800
1986	23,700	23,100
1987	19,900	21,000
1988	18,600	15,800 (Min)
1989	18,200	16,700
1990	24,500	19,300
1991	27,900 (Max)	24,300
1992	22,600	17,100
1993	7,800 (Min)	16,700

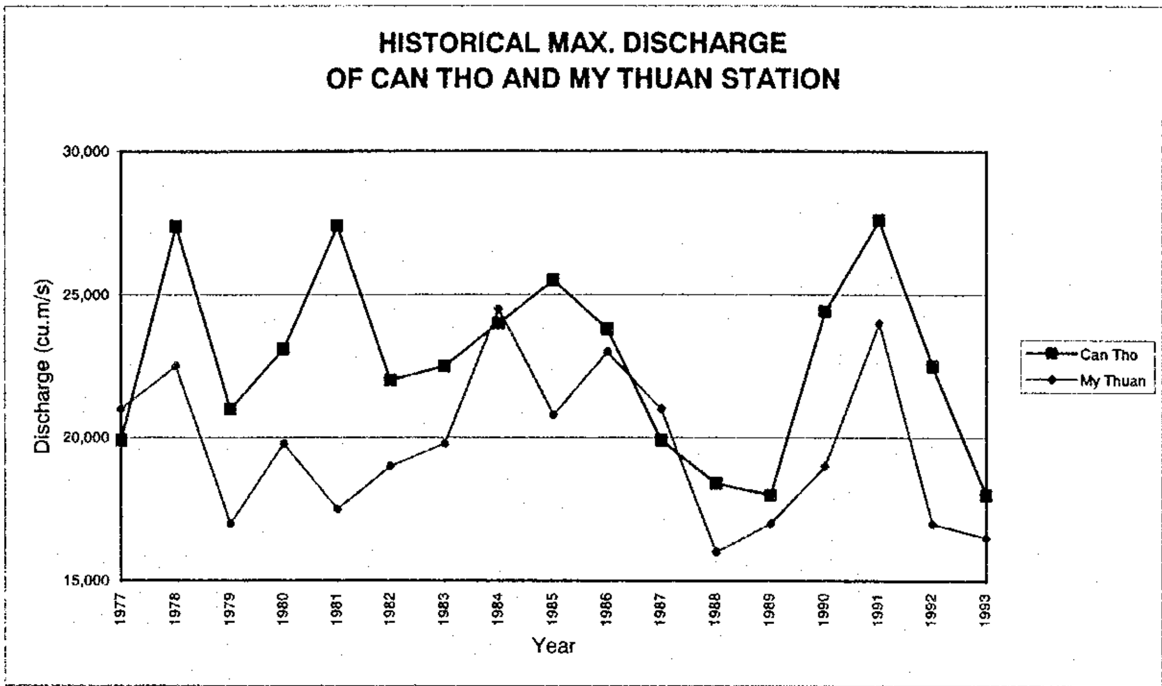


Figure 3.7 Max. Discharge of Can Tho and My Thuan Station

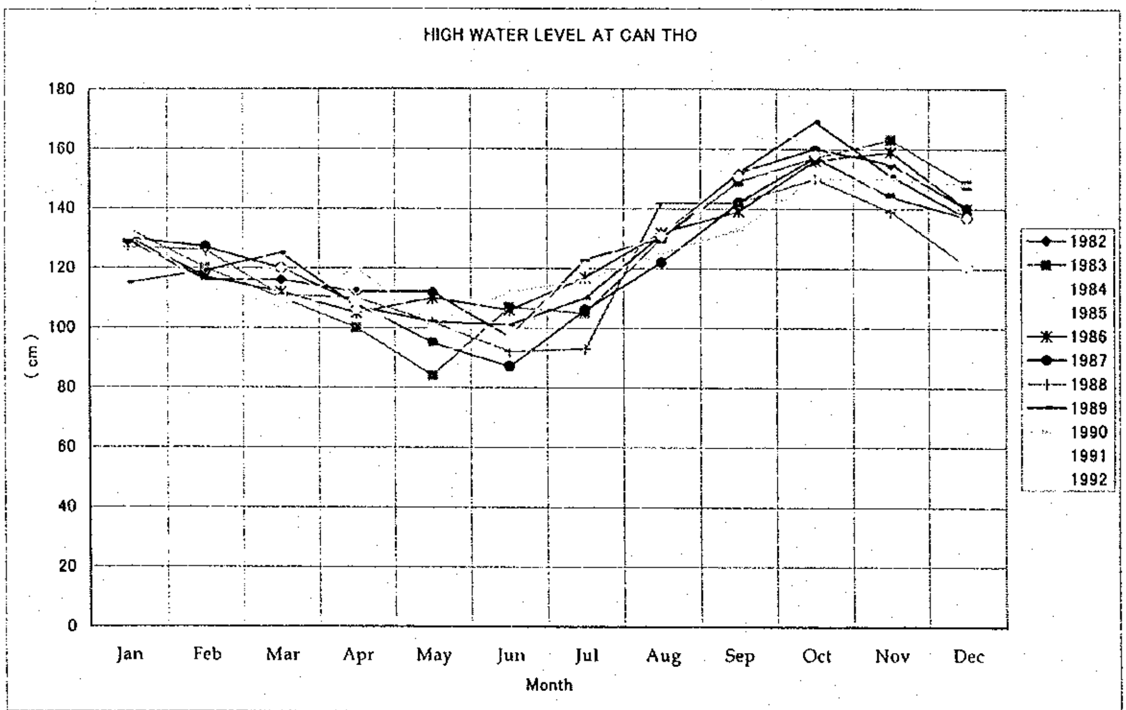


Figure 3.8 High Water Level of the Hau River at Can Tho Station

(2) Flood Condition

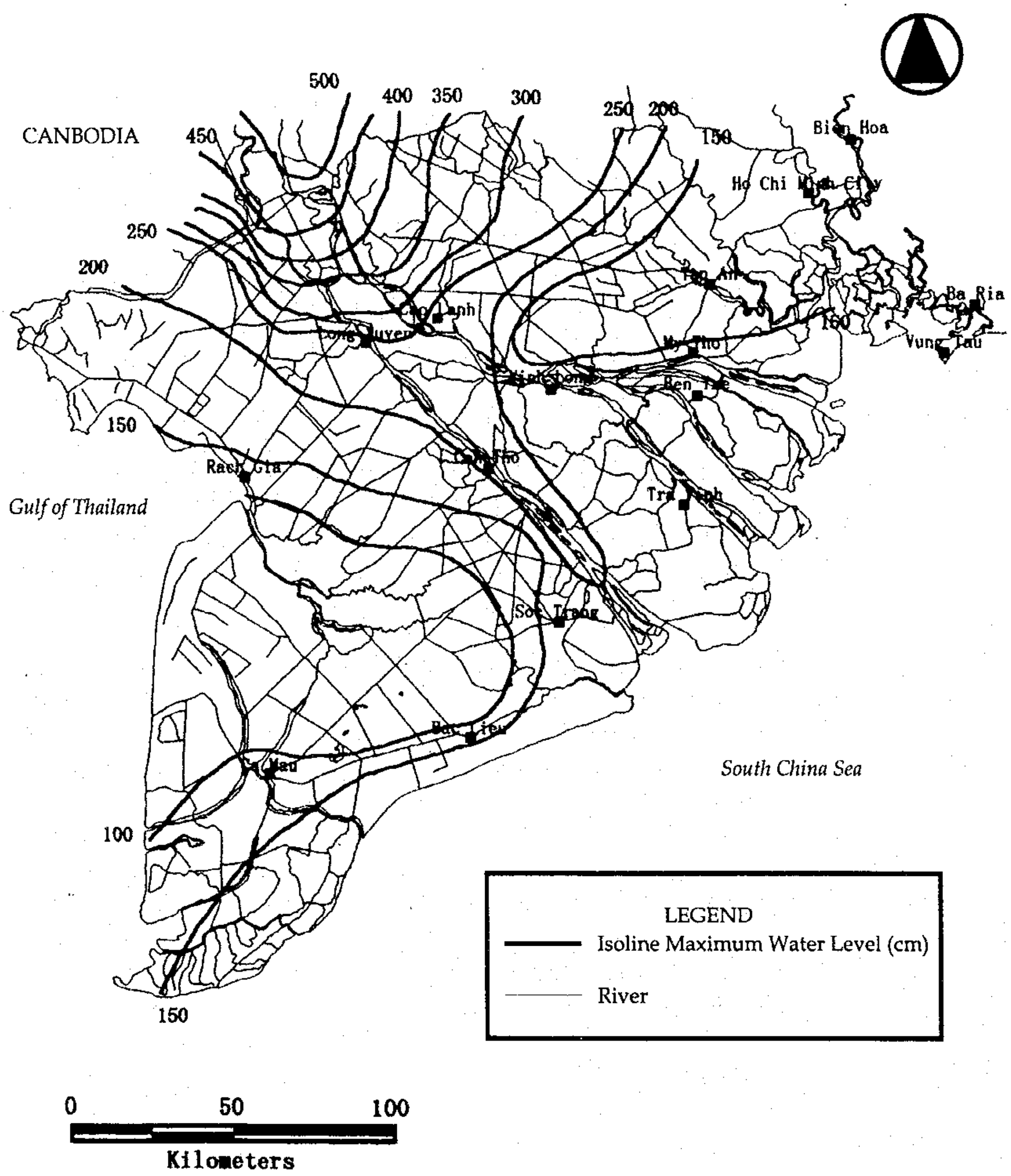
According to the F/S, the flood inundation area in 1984, 1992, and 1993 is described below:

- In the 1984 flood, the maximum water level was 2.0m and its duration was 3 to 5 months at Can Tho.

- During the past three years the whole area of An Giang and Dong Thap provinces were inundated. In 1984 and 1992 Kien Giang, Can Tho, Vinh Long, and Tien Giang provinces were almost flooded, and 50% of the Long An, Tra Vinh, and Soc Trang provinces were flooded. The inundation area of Ben Tre province was relatively small. Also, in 1994 about 50% of Kien Giang, Can Tho, Vinh Long, Tien Giang and Long An provinces were flooded, with flooding in the rest of the areas minimal.

- When comparing 1984 and 1992 inundated areas, paddy fields of Tra Vinh were classified as flooded areas in 1992. The rest of the areas were similar to the 1984 classification. This was probably due to the similarity in the water level pattern, and it was observed that the monthly average maximum water level change showed a similar pattern according to the measurements at the Can Tho measuring station.

- From the water depth comparison for the years 1984 and 1994 where data was available, it was found that the water depth of the inundated area was above 1 meter for locations up to 120 km away from the estuary. This area was in the vicinity of Long Xuyen, and northwest of the upper catchment. However, a water depth of more than 0.5 meters in 1984 covered the whole area of Hau river, and was only 15 km away from the Can Tho bridge site. Further, it was observed that the depth of the inundation area was higher than at the estuary in the upper section for the river. This is because the Hau River watershed is almost flat, but the difference in the depth of the inundated areas could be due to the better drainage in the vicinity of the estuary than at the upper river sections.



THE FEASIBILITY STUDY ON THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.9 Maximum Water Level Contour of the 1984 Flood JAPAN INTERNATIONAL COOPERATION AGENCY

3.7.2 Flood Analysis

A frequency analysis was conducted by using the existing 21-year series data of flow discharge (from 1977 to 1998, except 1996) and water level (from 1978 to 1998). The design flood discharge and higher water surface level at the bridge site of the Hau River (being necessary for the bridge scale- both the horizontal and the vertical clearance, position of piers and bridge side space) was then estimated.

The design flood discharge and higher water surface level were calculated by several methods which use the log-normal distribution method (Iwai's method, Slade's method, Hazen's method, Shen's method) and Hazen plot method. The curve fitting was done by least square method, i.e., by mathematical method instead of graphical one. The volume of the design flood was then determined in conjunction with the F/S analysis which was based on the relevant series date before 1993 (see the following table).

The results in the table show that the higher water surface level and flood discharge calculated by both Scade's and the Shen's methods are nearly the same. These analyses can be summarized as below:

- The result of the Hazen Plot Method is value. It may be reasonable only in the middle of the distribution.

- The log-normal distribution contains three interdependent parameters. When hydrologic data are plotted on a log-normal probability paper, straight-line trend is possible only for one value of Cs (the coefficient of skewness) when Cv (the coefficient of variation) is given. For other value of Cs, the plots are curves even for the same Cv. Since the value of Cs computed from ordinary hydrologic data is not so reliable, it has been suggested that if the plot shows curvature, the value of Cs should be modified so that a straight-line plot obtained. Otherwise, a special probability paper may be constructed if a straight-line plot for the original value of Cs is desired. However, there is no such modification in Slade's Method.

- Among the other Log-normal Distribution Methods in which relative modification is conducted, the Hazen's Method is an empirical one, and the Shen's Method is based on the Hazen's one but a theoretical modification was make on the method. The Iwai's Method was derived theoretically in Japan. One of the most important features in the Iwai's method is that there is a lower limit value 'b' in the distribution. With different 'b Values', the curve of the distribution in a probability graph paper can vary. It makes the method have better applicability in comparison with the other methods.

- The flood discharge with 20-year recurrence interval calculated by Iwai's method, Slade's method, and Shen's method are almost the same. However, the higher water level calculated by Iwai's method is higher than those by both the Slade's and the Shen's method.

Table 3.22 Design Flood at Can Tho Gauging Station of the Hau River

Recurrence Interval (year) P%	100	50	20	10	5	2	Max - Actual Record (year)
Factor	1	2	5	10	20	50	
Higher Water Level (cm)							184
This analysis							(1977)
Log-normal Distribution							
Iwai's Method (recommended for this D/D)	184.97	181.91	177.59	173.99	169.91	162.90	
Slade's Method	182.33	179.99	176.55	173.54	169.97	163.34	
Hazen's Method	187.73		178.52		169.68	162.47	
Shen's Method	182.31		176.54		169.99	163.30	
Hazen plot	181.41	179.20	175.99	173.57	169.66	163.34	
F/S (Slade's method)	172.40	170.87	168.60	166.60	164.20	159.69	
(value in the F/S report)	(212.40)	(210.87)	(208.60)	(206.600)	(204.40)	(199.69)	
Pre F/S	171.55	170.28	168.34	164.36	164.36	159.95	
Discharge (m/s)							27,900
This analysis							(1991)
Log-normal Distribution							
Iwai's Method (recommended for this D/D)	30,529	29,436	27,855	26,504	24,931	22,109	
Slade's Method	30,696	29,528	27,858	26,453	24,847	22,041	
Hazen's Method	30,098		27,694		24,954	22,186	
Shen's Method	30,273	29,175	27,862		24,836	22,022	
Hazen plot			27,621	26,258	24,724		
F/S (Slade's method)	30,999	29,849	28,204	26,819	25,232	22,453	
Pre F/S	30,345	29,414	28,030	26,813	25,356	22,622	

Remark: The State Datum was employed in this analysis.

The result of higher water level in F/S was referred to a different datum that is 40 cm lower than the State Datum.

The data series for the analysis of flood discharge are from 1977 to 1998, but there is no record in 1996.
The data series for the analysis of higher water level are from 1978 to 1998.

- There is a larger difference between the higher water level in F/S and that in this analysis. In F/S, the employed data series was shorter and the recorded max-actual water level was 169cm occurred in 1989. However, this analysis used a longer series of data and the max-actual water level was 184cm recorded in 1997.

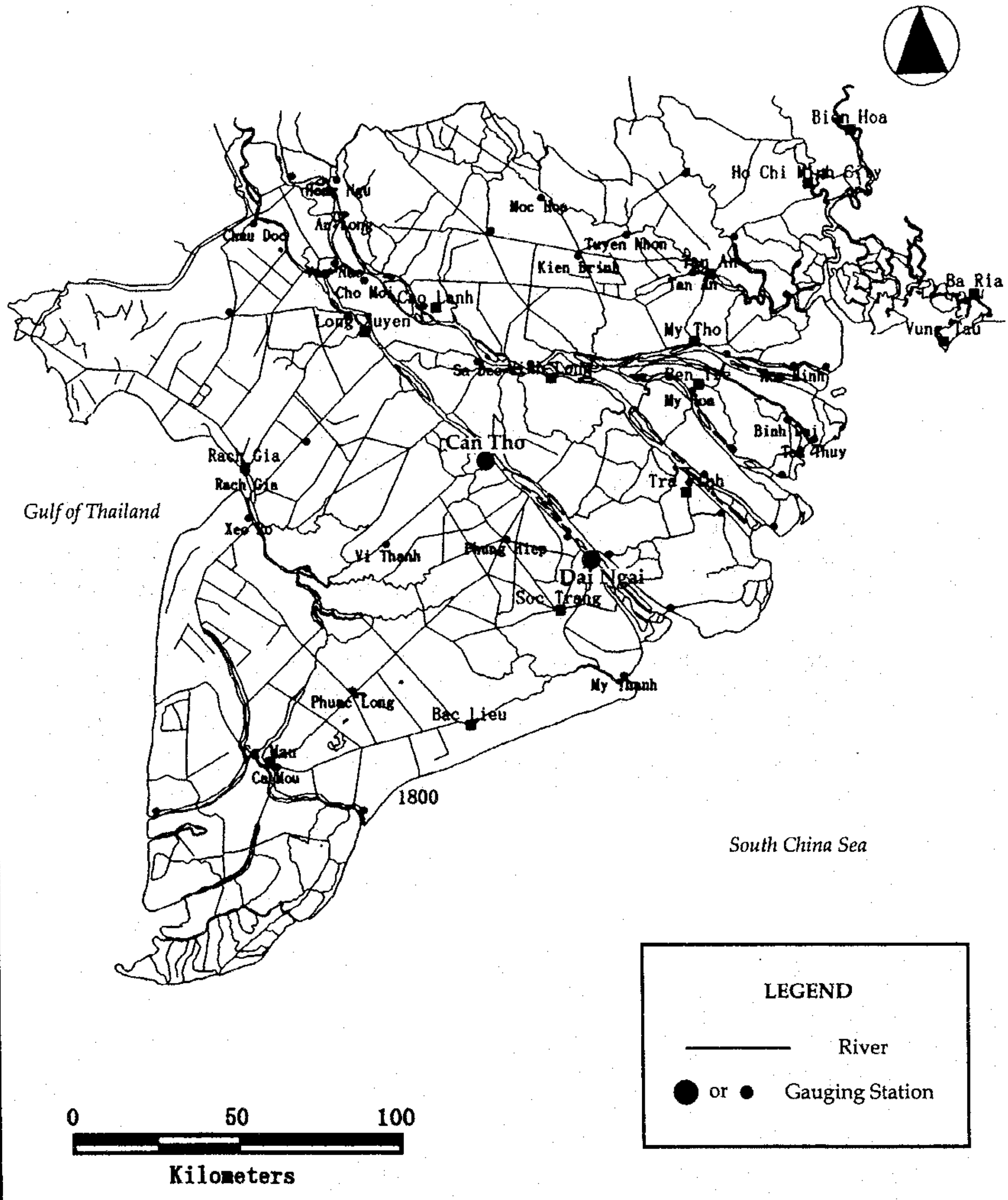
- The flood discharge in this analysis is a little bit smaller than that in F/S. That is because the data series was extended to include those in

1998, with smaller flood discharges in 1997 and 1998 were not so large.

For these reasons, the Log-normal Distribution Iwai's Method should be used to estimate the design flood discharge and the higher water level.

There are two gauging stations in the downstream reach of the Hau River from the Can Tho. They are Can Tho and Dai Ngai gauging stations, as shown in Figure 3.10. The Cabn Tho station is located about 45km upstream from the Dai Ngai station. The hourly records of water surface level in 1998 at both the Can Tho and the Dai Ngai gauging stations were considered for the design flood water level. The figures in the hourly records show that the higher water surface levels at Can Tho are obviously lower than those at Dai Ngai during dry season although the Dai Ngai is located about 45km downstream from the Can Tho. During flood season, they are almost the same as or a little bit lower than the corresponding higher water level at the Dai Ngai. However, there is a certain difference of time when the corresponding higher water level occurred at the Can Tho and the Dai Ngai.

It means that the corresponding higher water level that occurred at the Dai Ngai may also occur at the Can Tho during flood season although the water surface slope which corresponds to the discharge does exits in the Can Tho reach. In a tidal river, water surface slope cannot be used to deduce required water surface level from that at another location but can be used as a reference. The Can Tho bridge site is located between the Can Tho gauging station and the Dai Ngai, and is about 3km downstream away from the Can Tho station. Hence, the above analyzed higher water level can be directly applied to the Can Tho bridge site.



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.10 GAUGING STATION MAP JAPAN INTERNATIONAL COOPERATION AGENCY

Table 3.23 Design Flood at Can Tho Bridge Site of the Hau River

Recurrence Interval (year) P% / Factor Value	100 1	50 2	20 5	10 10	5 20	2 50	Max - Actual Record at the Can Tho Station (year)
Higher Water Level (cm) This Analysis	184.97 155.46	181.91 153.93	177.59 151.66	173.99 149.66	169.91 147.26	162.90 142.75	184 (1997)
F/S (C Route) (Value in the F/S report)	(195.46)	(193.93)	(191.66)	(189.66)	(187.26)	(182.75)	
Discharge (m^3/s) This Analysis F/S	30,529 30,999	29,436 29,849	27,855 28,204	26,504 26,819	24,931 25,232	22,109 22,453	27,900 (1991)

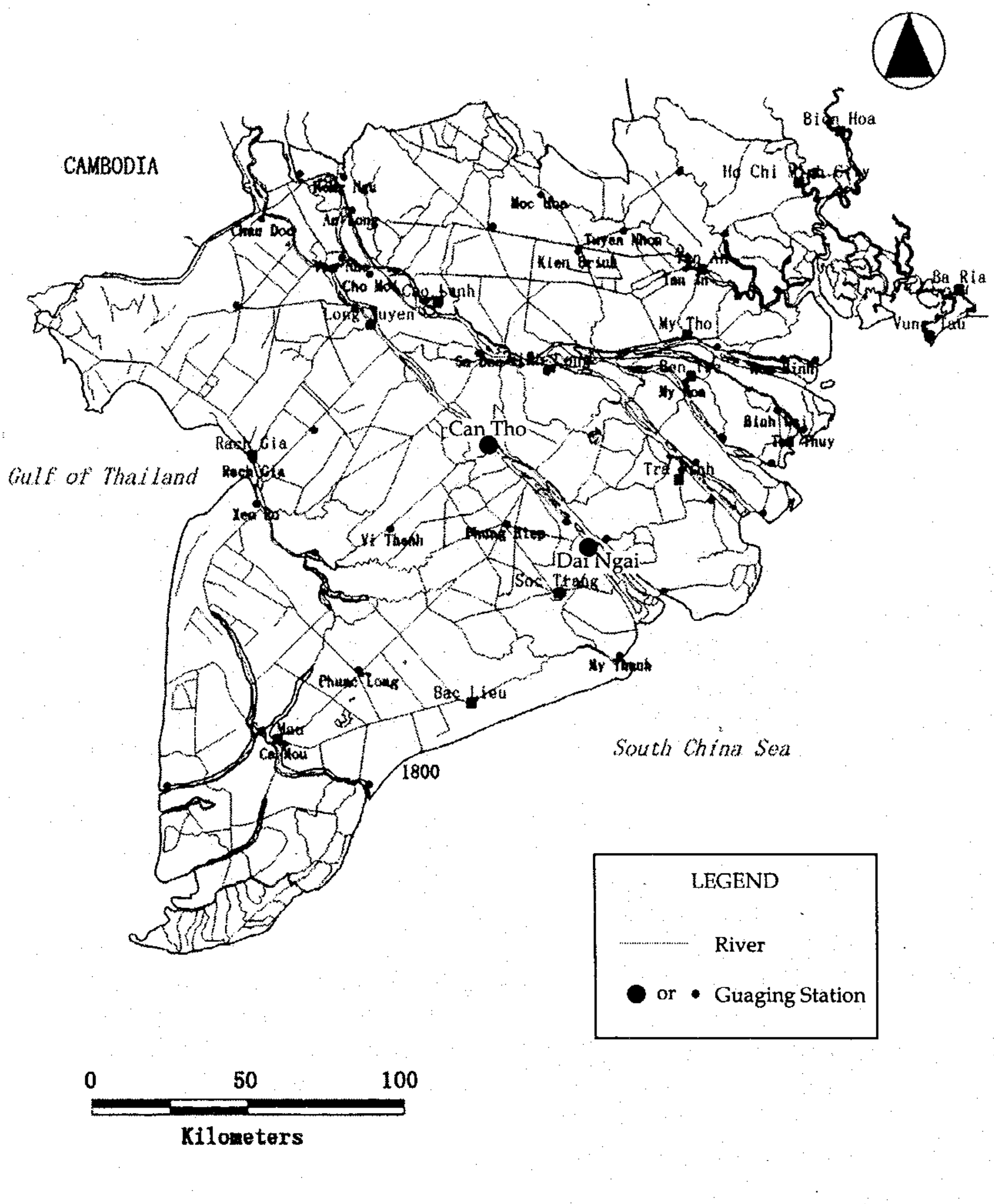
Remark: The State Datum was employed in this analysis.

The result of higher water level in F/S was referred to a different datum that is 40 cm lower than the State Datum. The water surface slope in the Can Tho reach of the Hau River was assumed to be equal to 0.000055 in the F/S. In the tidal river, however, water surface slope cannot be used to deduce required water surface level from another location.

3.8 Hydrological and Hydraulic Survey and Analyses at the Bridge Site

3.8.1 General Conditions at the Bridge Site

The proposed Can Tho bridge in Socialist Republic of Viet Nam across over the Hau River is situated at 1889km of the National Highway No.1, in Can Tho City of the central portion of the Mekong Delta. The Mekong River, having catchment area of 795,000km^2 that is about 5.5 times bigger than that of the Red River, is one of the largest rivers in the world, ranking seventh in term of length. The river, from upstream to the mouth, is 4,200km long and flows through six countries, namely China, Myanmar, Laos, Thailand, Cambodia and Viet Nam. Every year in downstream Mekong River, from September to November, coarser particles of silt carried by the flood flow are deposited along the riverbank when the river waters overflow the bank during flood. Spreading from the bank, there is a suddenly reduction in velocity, hence a higher ridge of land along the river is resulted in. Behind these ridges are low-lying areas inundated by floodwaters that contain very fine clayey particles. Therefore, with Phnom Penh as its head, the sea as the bottom line and Thailand gulf as the west line, and Vam Co river as the east line, the Mekong Delta which consists of a vast alluvium built up by the river is formed. It covers an area of 49,520km^2. The lower Mekong Delta in Viet Nam is shown in Figure 3.11 with the location of gauging stations. The delta has many interlacing rivers and canals, distributed in a flat and low terrain which is from +0.5 to +1.0m (MSL - Mean Sea Level, National Datum of Viet Nam) along the coastal land and about +1.5~+2.0m (MSL) near the northern border, and the average gradient is at about 1/100,000.



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.11 The Mekong Delta in Viet Nam and Gauging Stations
	JAPAN INTERNATIONAL COOPERATION AGENCY

The Hau River is one of the two main branches of lower Mekong River basin. At Phnom Penh located about 70km away from the Viet Nam - Cambodia border, the Mekong River is divided into two channels flowing into Viet Nam, namely the Tien and the Hau River upon entering Viet Nam, respectively. Near their mouths, both rivers again subdivide. The Hau River is interconnected with the Tien River by Vam Nao by-pass channel and many other natural and man-made channels.

Mekong River network consists of upstream rivers with 100 to 300m wide section and downstream rivers with section up to 2km wide. The widest section of the Hau River is about 18km wide. Besides the main river channels, there are plentiful river channels and canals with 30 to 100m wide and 2~4m deep, and small channels with the section less than 30m wide and 1.5~2m deep.

Water level measurements in the Mekong Delta were being taken as early as 50 years ago at some gauging stations and are very valuable for hydrologic analysis. Precipitation data has been recorded at quite a number of stations since the turn of the century. However, prior to 1958 there had been very few discharge measurements in both the Mekong mainstream and most of its tributaries. Since 1968, phenomenal progress has been made in hydrological and meteorological measurements and in hydrographic surveys of many portions of the river.

The Mekong Delta is situated at low latitude and affected by southeast monsoon. High temperature remains constant around the year with temperature variation from 3~5°C between the hottest and the coldest month. Every year, rainfall occurs principally from April or May to October or November resulting in distinct wet and dry seasons of the area. The rainy season is concurrent with southwest monsoon. It accounts for 90% of the yearly rainfall with the average from 200 to 400mm in a month of which number of rainy days accounts for 2/3 of total days and is current-like rains. On contrary, in dry season, average rainfall in a month is only about 10 to 15mm. Total rainfall in dry season accounts for about 10% of the yearly. Rainy season is also the season of rather high humidity (over 85%) with cloudy sky and rarely sunny days. In dry season, humidity reduces considerably (to about 75%) with plentiful sunshine days which is equivalent to 150~200 sunny hours. This is a major factor influencing waterway depths. The Mekong River has an annual flow cycle, which, although varying somewhat from year to year, is quite uniform in general pattern. The river begins to rise slowly near the start of the rainy season in April or May, and reaches its peak in September or October when the accumulated flooding waters spread over a large part of the delta. Then it drops rapidly with the end of the rainy season in November and recedes gradually during the dry winter season to reach its lowest level before the onset of the monsoon again in April. Because of the relatively flat topography, the inundation caused by local rainfall in the delta area is reinforced by flood of the Mekong during flood season.

Since these rivers are close to the China Sea, considerable tidal effects and opposing water currents influence the flow. The South China Sea tide is semi-diurnal and the tidal range is from 3.8 to 4.0 meters. Inside the gulf of Thailand, the tide is diurnal and the tidal range at Ha Tien is only about 1.2 meters. Because the water surface slope is very gentle in the downstream Mekong River, the tidal effect can reach to the portion of the river in Cambodia. During flood season, velocities over 2m/s have been measured at Can Tho in the Hau River and some elsewhere. During low-water periods, the flows of major rivers reverse with the tide and an upstream-pointed flow with the velocity over 0.6m/s has occurred at Can Tho in flood tide. This reversal in direction rarely occurs during the flood season in these streams. At Long Xuyen, which is about 60km upstream from Can Tho, even during the river low-water season, the flood tidal current is very weak.

3.8.2 Hydrological and Hydraulic Conditions

As stated above, the Mekong Delta is very susceptible to flooding during the rainy season each year. Due to increase of water level and velocity in the river, bank erosion and riverbed scouring occur.

(1) General Conditions

The width of the Hau River extends from 1.2 to 2 km from Can Tho city to the estuary. Annual maximum discharge of the Hau River is 17,800 ~ 27,900m^3/s (according to the data measured before 1993) in the wet season. Erosion is predominant and scour is complicated and quite deep.

(2) Flood Phenomenon

During the wet season, the river water level rises and large-scale inundation occurs in the northern area. Generally, flooding in this area starts in July or August, and lasts up to November or December. The inundation also occurs in the southern part of the delta, and in the areas where the drainage facilities are poor and the water is logged for as long as five months.

(3) Tidal Range

The daily tidal change in the vicinity of the estuary is considerably large, and the range of high tide varies considerably. The effects of tides are less pronounced in the upper reaches of the river. It is also observed that the tidal changes are greater in the dry season than that in the flood season.

(4) Stream Flow

The velocity of the river flow increases in the flood season, and its maximum is about 2.5m/s as recorded at the Can Tho gauging station. In the dry season, seawater entering into the river and reverse flow with velocity over 0.6m/s at Can Tho was observed. During the wet season, the reverse flow is rarely observed although the water level is still affected by the tide.

In vicinity of the Can Tho ferry port, a whirlpool has been created due to the high changes in the local water flow. Riverbed and bank erosion increases in this area due to this phenomenon.

(5) Erosion and Sedimentation

The Mekong Delta was formed from the sediment carried by rivers and deposited over the period of time. This phenomenon is still continuing, and the increase of water level during the wet season, together with the increased water velocity further accelerates the erosion and sedimentation in the region. This phenomenon creates changes in river channels, which are still in their natural form, and applies to the Hau River, which are susceptible to erosion and sedimentation.

(6) Scour

According to the historical water level data and river surface change information, scour has been occurring in the riverbed due to water flow in the vicinity of the Can Tho bridge site in the Hau River. Especially, during the flooding season, increasing of water velocity causes more scouring. Therefore, extra precautions need to be taken to maintain the durability and safety of the piers and abutments of the bridge during the planning stage.

(7) Velocity Survey

From September 18 to 25, 1997, for the feasibility study, the hydrological survey team conducted the current velocity measurements along the centerlines of the three alternative routes of the proposed Can Tho bridge. The survey was implemented during the flood season of the Hau River. The results showed that the velocity in Can Tho reach of the Hau River during the flood season could be over 2m/s.

(8) The Past Floods

An interview investigation was carried out along the three routes - A, B, and C during the feasibility study. Inhabitants of the area were interviewed to comment on the flooding status of the area. The information collected on the flood inundation of the Can Tho bridge site can be enumerated as follows:

- A flood is experienced once or twice a year between September to November
- The flood duration is relatively short, and generally is about 2 hours in one day (it was told, however, that the flood duration continued for 5 days in some years)
- Inundated depth is low, and it was told that no dwelling was affected by flooding
- It was told that the largest flood occurred in 1984 (1 person), 1988 (1 person), 1993 (1 person), 1996 (2 person), and 1997 (1 person).

Besides the interview investigation, the satellite remote sensing data analysis had shown that during the floods in 1984, 1992, and 1993, the whole area of An Giang and Dong Thap provinces were inundated. In 1984 and 1992 Kien Giang, Can Tho, Vinh Long, and Tien Giang provinces were almost flooded, and 50% of the Long An, Tra Vinh, and Soc Trang provinces were flooded. The inundation area of Ben Tre province was relatively small. Also, in 1994 about 50% of Kien Giang, Can Tho, Vinh Long, Tien Giang, and Long An provinces were flooded, with flooding in the rest of the areas minimal. When comparing 1984 and 1992 inundated areas, paddy fields of Tra Vinh were classified as flooded areas in 1992. The rest of the areas were similar to the 1984 classification. This was probably due to the similarity in the water level pattern, and it was observed that the monthly average maximum water level change showed a similar pattern according to the measurements at the Can Tho gauging station.

3.8.3 Historical Plan Form Change of the Hau River

Historical records and hearing investigations showed that the Study area is a flood prone area, and as a consequence of these floods plan form changes of the river had occurred. Therefore, the plan form changes that had occurred in the past need to be identified and established in establishing the appropriate bridge site. Three Landsat data sets, in 1972, 1973, and 1993, respectively, were acquired in recognizing the plan form changes between the 1972~1973 and the 1993 periods. The following conclusions were obtained:

- The Hau River in this area is almost linear.

- The sand bar - Cu Lao Linh Island is almost oval in shape, and extends from the northwest to southeast. The sand bar of the northwest upstream, which is facing the flow of the river, is being eroded, and on the other side, deposition is observed in the northeast of the downstream side. The rate of erosion is 4.5m/year, and the deposition is 1.5m/year.

- In the front of Cu Lao Cai Khe, the riverbank is facing the river flow, and erosion is occurring. The erosion rate for the period of 1972-73 and 1993 is 3m/year.

- The sand bar - Cu Lao Lat Island was flooded in 1973, but it has been integrated into a land area in 1993.

The locations of the islands are shown in the following Figure 3.12.

3.8.4 Hydrological and Hydraulic Investigation

(1) Flood Level and Discharge

Following the analysis in the feasibility study, further investigation was conducted to obtain additional data upon which to base the design parameters. New hydrological data from 1994 were collected from the Southern Region Hydro-meteorological Center to supplement the series of annual maximum flow discharge and annual highest water surface level. Interview with the local dwellers was further conducted to investigate the flood and bank erosion situations. It was told that the most serious flood inundation occurred in 1997 with about 30cm inundation depth near the river side which lasted about 2 hours. The hydrological records at Can Tho gauging station also showed that the recorded highest water level - 184cm (MSL - Mean Sea Level) occurred on November 2nd, 1997. However, the flood discharge was not the maximum. The recorded annual maximum flood discharge - 27,900m^3/s occurred in 1991 and the highest water level in 1991 was 158cm (MSL). It can be deduced that higher tidal level was the key factor in that flood inundation.

Based on the existing 21-year series hydrological data at Can Tho gauging station of the Hau River, a frequency analysis showed that the design flood discharge and higher water surface level at the bridge site with 20-year recurrence interval are 27,855m^3/s and 1.776m (MSL), respectively. The corresponding flood discharge and higher water surface level with 100-year recurrence interval are 30,529m^3/s and 1.85m (MSL), respectively. Although reverse flow occurs in Can Tho reach due to the influence of tide, the water is always fresh and seawater never reach there. Concentration of suspended sediment in the reach is not so high. Local people take the river water as their drinking water, even in the flood season.

(2) Hydrographic and Hydrological Surveys

The following site surveys and associated studies were carried out as part of the detailed design:

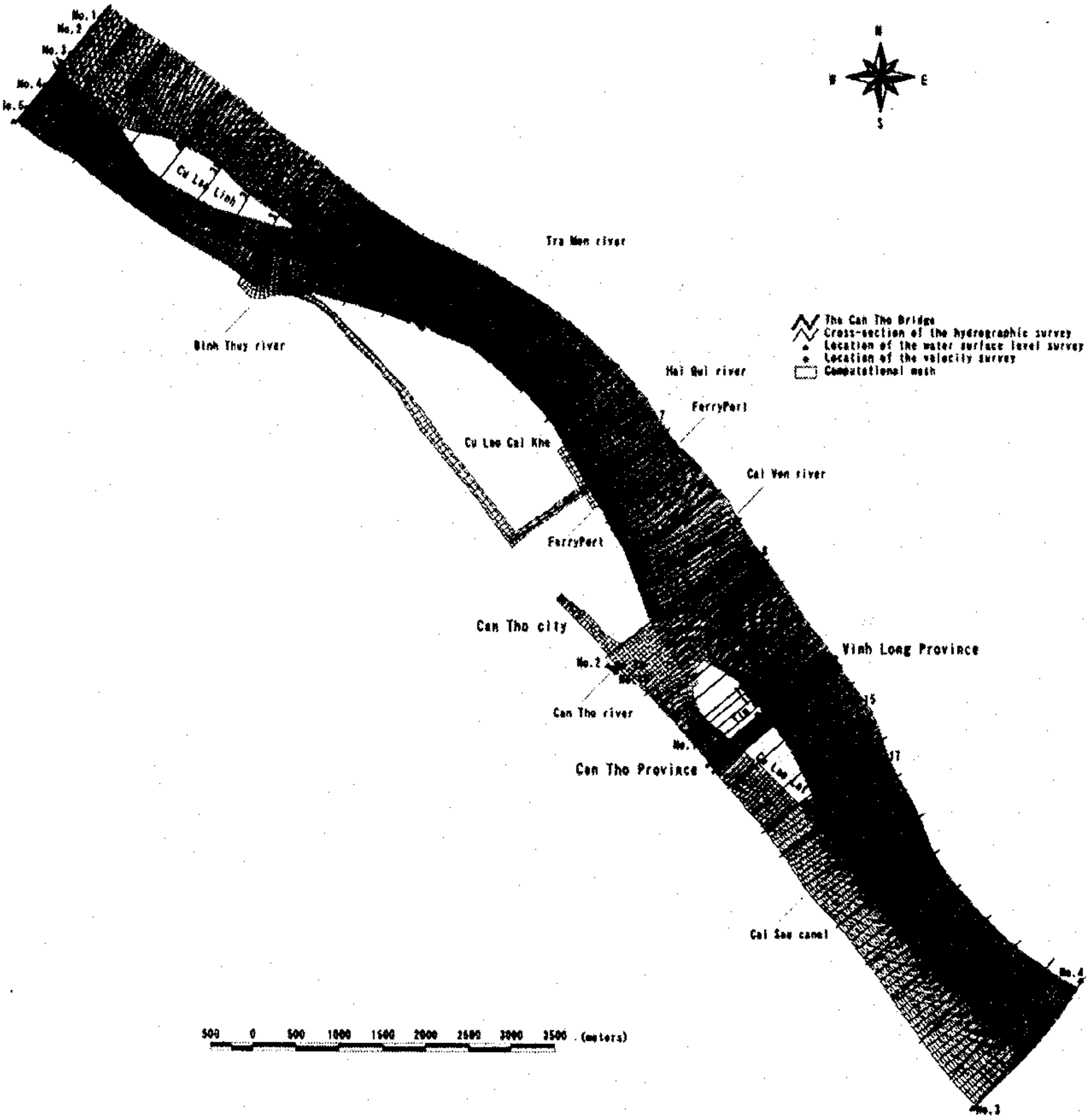
- Hydrographic and hydrological data collections
- Hydrographic and hydrological surveys
- Hydrological and morphological studies including numerical modeling of the Hau River around the bridge site
- Riverbed material sampling and analysis

A highly hydrographic detailed survey was undertaken in the area surrounding the Can Tho bridge. This area included the bridge site itself and detailed riverbed data on both sides of the bridge centerline within 100m from the centerline was collected. Then the cross-section survey was extended to the upstream side about 11km away from the bridge centerline and also to the downstream side about 5km away from the centerline, respectively. The extent of the hydrographic charting carried out is shown in Figure 3.12. Total cross-sections were 52.

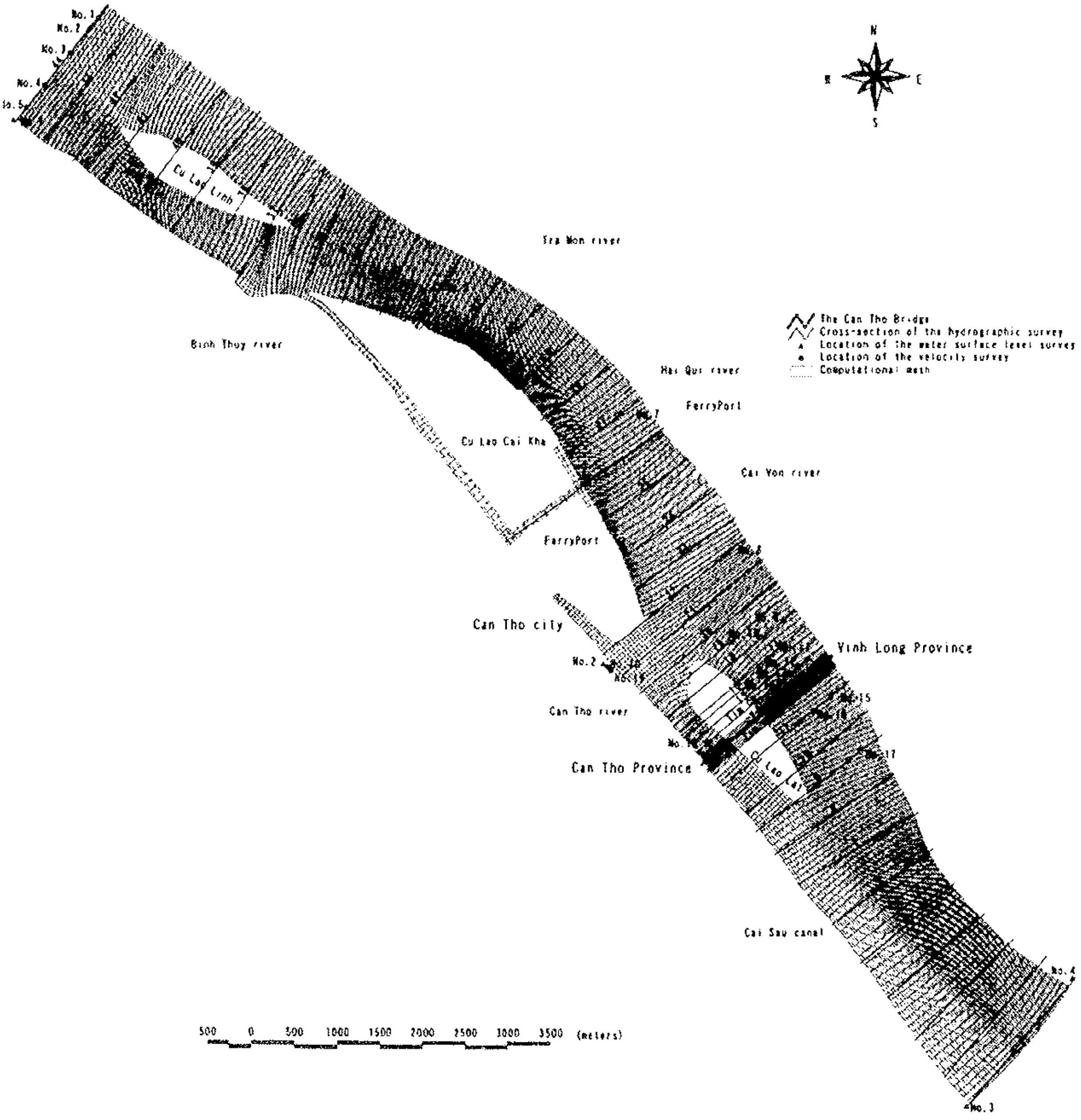
Data was collected by taking cross-sections of the river perpendicular to the direction of flow. The interval between the sections was 100~500m, except those around the bridge centerline which were at 25m interval. The datum of the survey was based on the National

Datum of Viet Nam. All the locations of hydrographic data were plotted with the local water depth at scale of 1:10,000. The hydrographic data around the bridge centerline were plotted in details at scale of 1:500 with 1m contours. The total 52 cross-sections were plotted with the scale of 1:2,000 horizontal and 1:100 vertical.

Twenty verticals of velocity measurement and four points for water surface level were arranged, as shown in Figure 3.12. The measurement of flow velocity was carried out from 31 May to 1 June 1999. The measurements at all the verticals and water level points were carried out simultaneously at the interval of one hour. Measurement of flow velocity was undertaken at 6 points/vertical which were at the following depth: Water Surface-0.5m, 0.2H, 0.4H, 0.6H, 0.8H and Bed+0.5m, respectively. H is the local water depth. Discharges at the cross-section upstream side 11km away from the bridge centerline and at the cross-section of the Can Tho River mouth were calculated by using the local measured results of the velocity and cross-section.



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.12 Scope and Location in the Survey, and the Computational Mesh
	JAPAN INTERNATIONAL COOPERATION AGENCY



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.12 Scope and Location in the Survey, and the Computational Mesh JAPAN INTERNATIONAL COOPERATION AGENCY

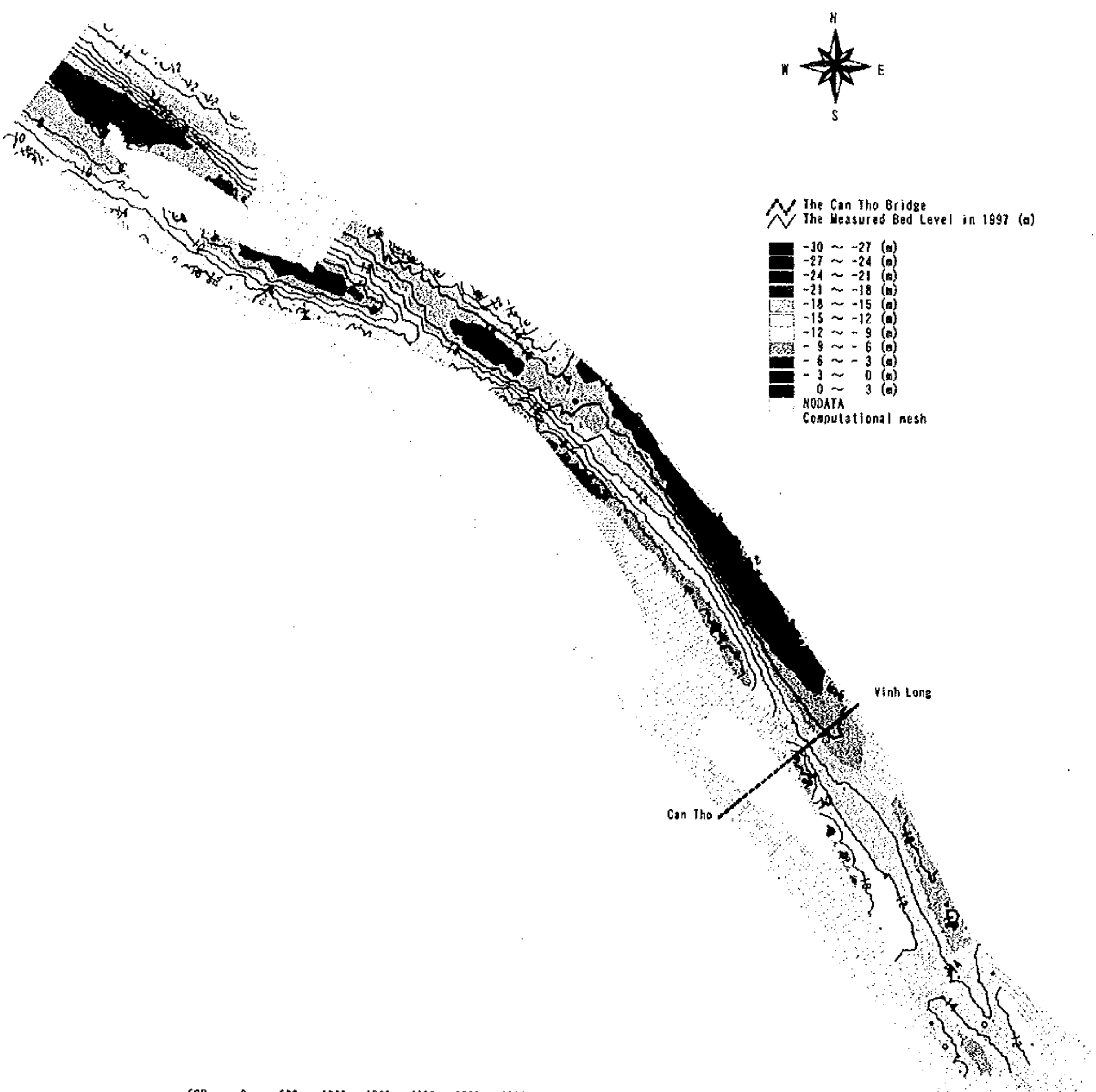
(3) Recent Change of Riverbed

Recent three years', in 1997, 1998, and 1999, respectively, bathymetrical chartings measured in Spring of each year by Viet Nam Maritime Safety Company were collected. Only the data around the centerline of the river are available because the measurements are mainly served for navigation. Figure 3.13 ~ Figure 3.15 showed the bed level contours - MSL in Can Tho reach of the Hau River in 1997, 1998 and 1999, respectively. The bed level in 1999, shown in Figure 3.15, was supplemented by the measured bed level data in this hydrographic survey (in May and June 1999).

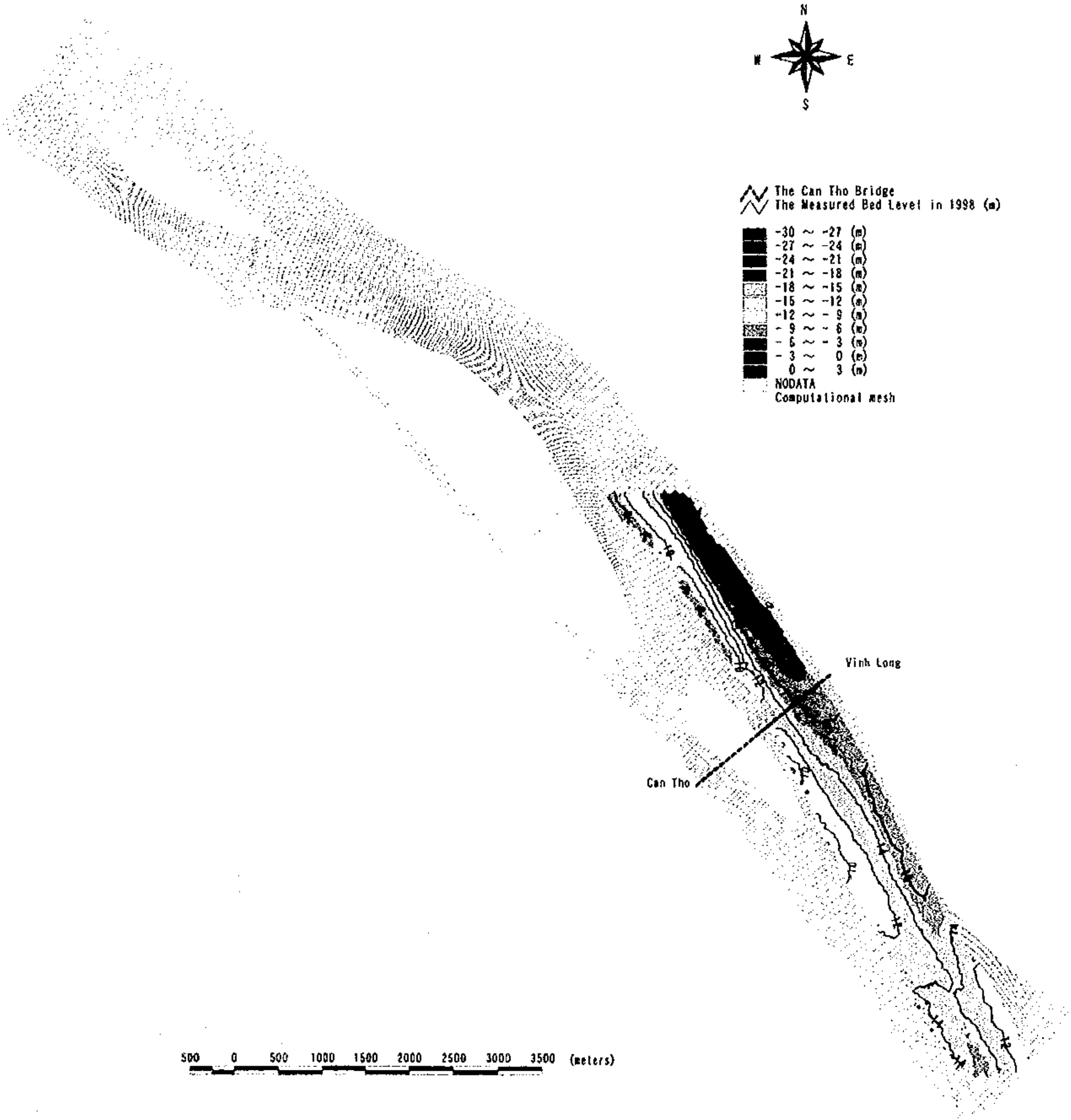
The results show that degradation of the riverbed in the whole Can Tho reach of the Hau River is now progressing. Erosion near the ferry port in Vinh Long side was considerable. In our site investigation, we were told that during the flood in 1998, the ferry abutment in Vinh Long side collapsed due to local bank erosion and one person died in that disaster. Erosion in the vicinity of the proposed Can Tho bridge was also not negligible. Contour of the -18m is approaching to the centerline of the proposed Can Tho bridge site. Contour of the -16m around the bridge site is uniting into one with that in downstream side which was at 1km downstream away from the bridge in 1997.

3.8.5 Flow and Bed Deformation Study by Numerical Method

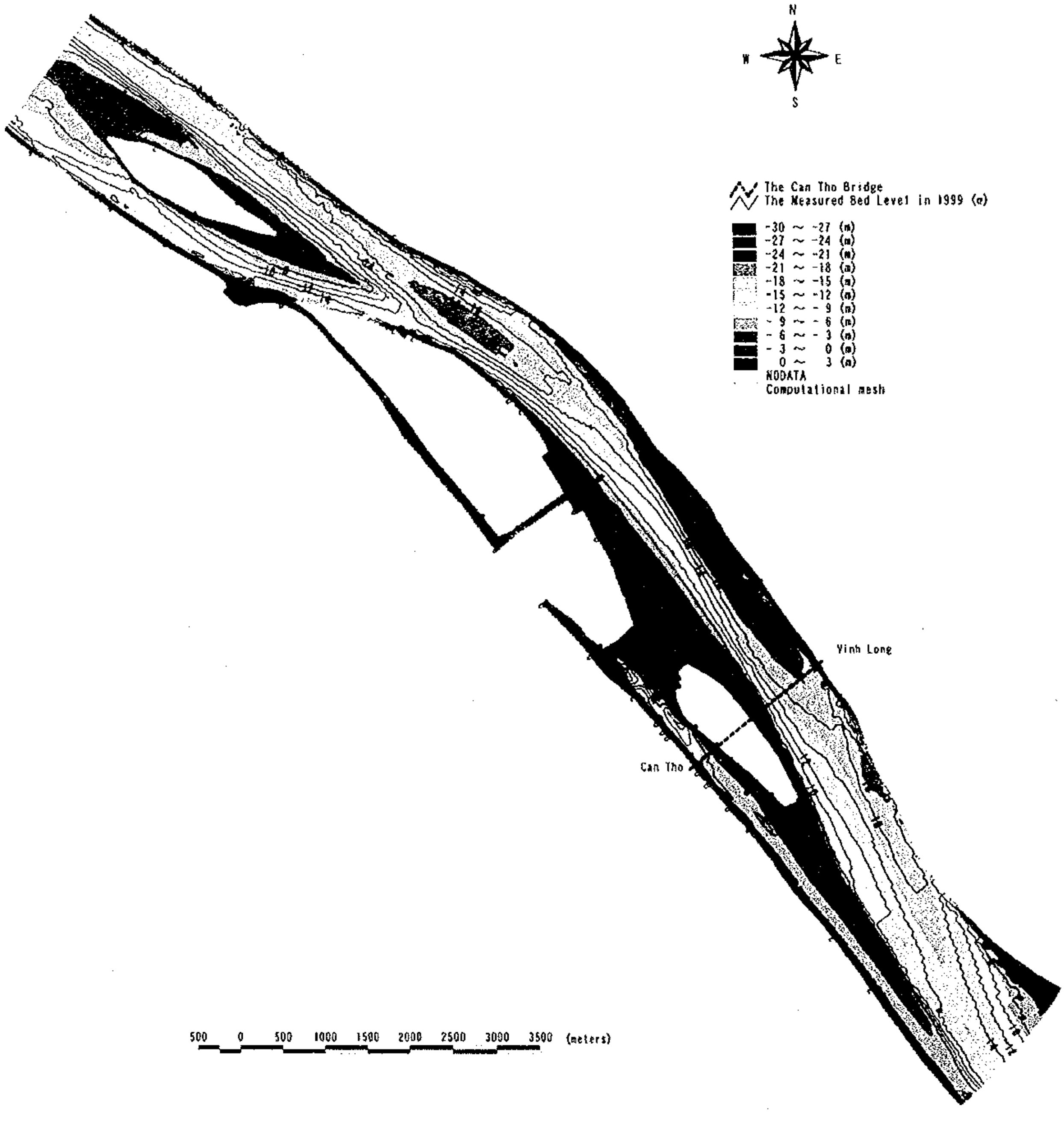
Failure of a bridge due to scour at their support is a common occurrence. It may be due to scour at a pier or at an abutment. The erodible bed deforms until it reaches an equilibrium scour configuration for which the rate of sediment supplied to the scour area is balanced by the rate of transport out of the area. Since the flow pattern of scour is very much complicated by the configuration of the obstruction, it is often necessary to utilize model studies in order to establish the scour depth as a function of the pertinent variables.



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.13 Bed Elevation in Can Tho Reach of the Hau River in 1997
	JAPAN INTERNATIONAL COOPERATION AGENCY



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.14 Bed Elevation in Can Tho Reach of the Hau River in 1998 JAPAN INTERNATIONAL COOPERATION AGENCY



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.15 Bed Elevation in Can Tho Reach of the Hau River in 1999
	JAPAN INTERNATIONAL COOPERATION AGENCY

A mathematical method can provide an advantage in evaluating extreme conditions with the same temporal and spatial scales. Herein numerical modeling was employed to study the flow condition and potential of bed deformation around the proposed bridge site. The model was first validated by the field data measured in the above mentioned hydrographic and hydrological surveys in May and June 1999. Then the model was employed to predict the features of flow and bed deformation under the design flood conditions. The predicted flow parameters were used to evaluate the local extreme scouring depth around the piers of the bridge by using empirical methods. Finally, in order to safeguard the bridge from the effects of such changes, the required protective measures will be discussed.

(1) Description of the Numerical Model

For assessing the river flow conditions and determining what change to the channel may occur in future in vicinity of the bridge site, a depth-integrated two-dimensional numerical model was developed. Because the river plan form which includes several islands is very complicated, the model was based on a boundary-fitted numerically generated orthogonal curvilinear coordinate system. The domain of interest, which is about 16km long with three islands, and the numerically generated boundary-fitted orthogonal curvilinear mesh were shown in Figure 3.12.

Governing equations referred by the model for water flow were depth-integrated Reynolds equations expressed in orthogonal curvilinear coordinates. Constant density and hydrostatic pressure variations were assumed and the Boussinesq's approximate for turbulent stresses were employed. Data on the sediment load of the river, which were extremely limited because only a few samples have been collected, indicated that the suspended sediment concentration is not as high as those in many other large streams. Therefore, bedload transport was only taken into account in the evaluation of bed deformation. The bed materials sampling analyses showed that the sediment in Can Tho reach of the Hau River is relatively uniform, hence uniform material was considered in the following simulations. Mass conservation equation for bedload transport was employed to compute the bed level variations. Bedload transport rate was evaluated by Ashida-Michiue's formula with the consideration of both flow direction and bed slope. Experiments showed that the repose angle in water of sediment with mean diameter 0.075mm is between $24^{\circ}\sim27^{\circ}$. The lower limit, 24°, was employed as the repose angle of the sediment in the following simulations.

As a boundary condition, observed tidal elevation at the downstream boundary was employed. Temporal flow-rate was specified at the upstream boundary and also at the mouth of Can Tho River. The impenetrable condition was applied to the bank boundaries, i.e., the velocity normal to the bank was equal to zero and the tangential component, conforming to the resistance law, was not zero. Equilibrium bedload transport was assumed at the open boundaries. The initial conditions were taken as the corresponding boundary values at beginning. The initial bed elevation, as shown in Figure 3.15, was generated from the measured bed level data in this hydrographic survey (in May and June 1999) and in 1999's bathymetrical chart (surveyed in Spring) provided by Viet Nam Maritime Safety. In the latter there were much more measurement points than this hydrographic survey, but they were limited only around the centerline of the river. In the former, the interval of the measurement cross-sections were large except those in the vicinity of the proposed Can Tho bridge. However, the shallow areas were also measured in this hydrographic survey.

A "staggered" grid arrangement was employed, i.e., components of vectors, e.g., velocity, were "staggered" with scalar variables, e.g., water level and bed elevation, at the center of the control volume. The governing equations for water flow were discretized by a special finite difference scheme and solved first by Tri-Diagonal Matrix Algorithm (TDMA). Then streamlines and velocities on the bottom were calculated. Finally bed variation was evaluated. Starting from certain initial conditions and under specified boundary conditions, the procedure was carried out step by step and the time varying velocity fields, water surface level and bed elevation could be obtained.

(2) Validation of the Numerical Model

According to the measurement results of the hydrographic and hydrological surveys in May and June 1999, the Manning's roughness was estimated to be 0.02. The bed material sampling analyses indicated that the mean diameter of the sediment at the bridge cross-section is 0.075mm and the mean d_{65} is 0.076mm. The sediment in the central area of the river, including the main pier site of the bridge, is coarser a little. In the following simulations, d_{65}=0.076mm was determined as the representative diameter of the bed materials in Can Tho reach of the Hau River.

The discharges at the upstream boundary and at the mouth of Can Tho River and water surface level at the downstream boundary obtained in this hydrological survey from May 31 to June 1 1999, as shown in Figure 3.16 ~ Figure 3.18, were specified as the boundary conditions for the simulation. The discharge in flood tide was expressed in negative value and that in ebb tide was in positive one. Datum for the water surface level was based on the National Datum of Viet Nam. The data showed that tidal amplitude at the bridge site during the hydrological survey from May 31 to June 1 1999 was larger and reached to about 2m. The effect of tide was considerable. The flow discharge amplitude during the survey was about 18,000m^3/s, in which the maximum discharge of flood tide was over -6,000$^3/s$ and the maximum ebb tide discharge reached to about 12,000m^3/s. The discharge amplitude at the mouth of Can Tho River, one of tributaries in Can Tho reach of the Hau River, was also larger and varied from about -2,000m^3/s in flood tide to about 2,000m^3/s in ebb tide. This fact indicated that the effect of Can Tho River on the flow in the Hau River was not negligible.

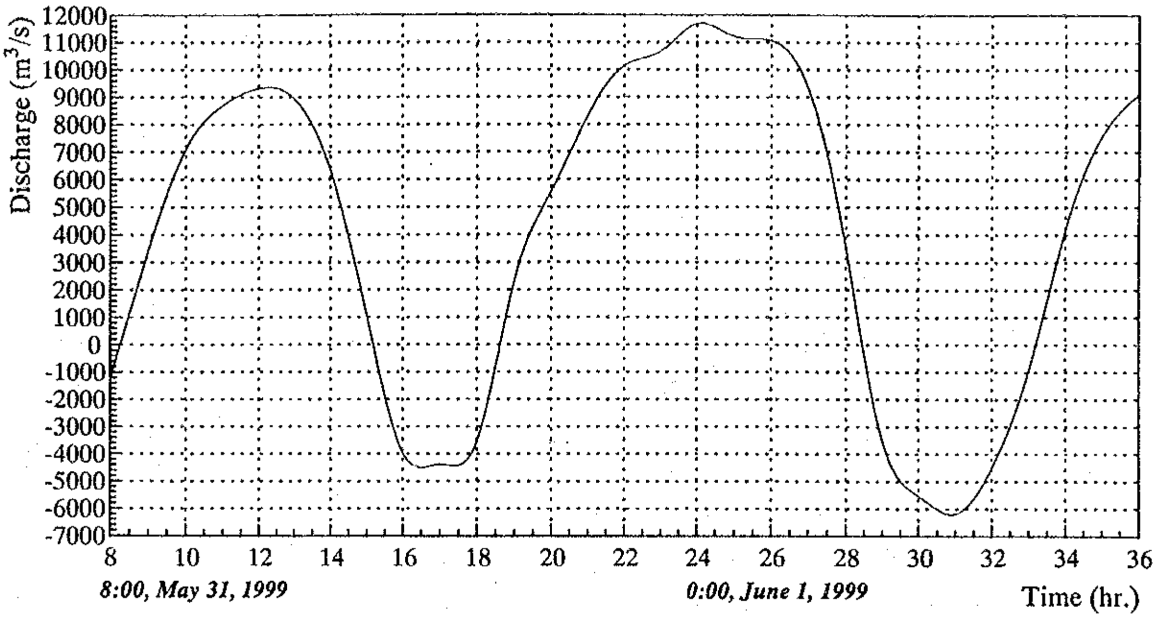


Figure 3.16 Hydrograph of Discharge Measured at the Upstream Boundary during May 31 ~ June 1, 1999

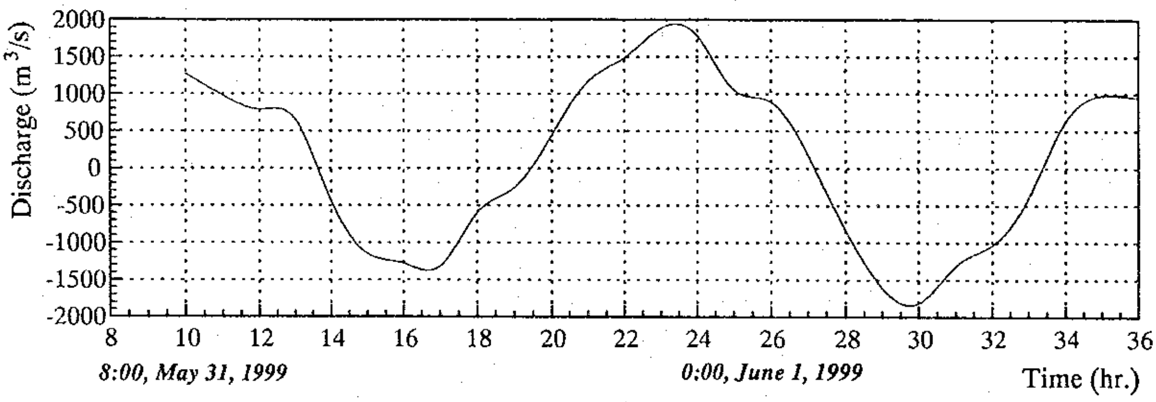


Figure 3.17 Hydrograph of Discharge Measured at the Mouth of Can Tho River during May 31 ~ June 1, 1999

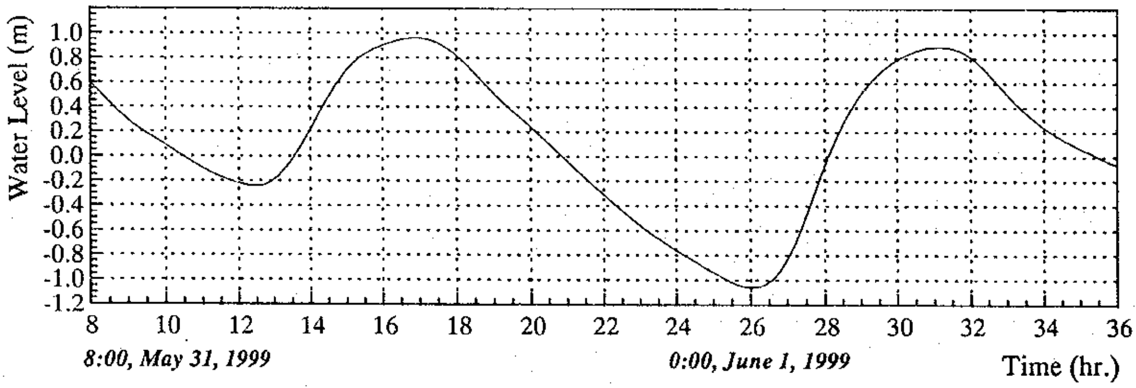
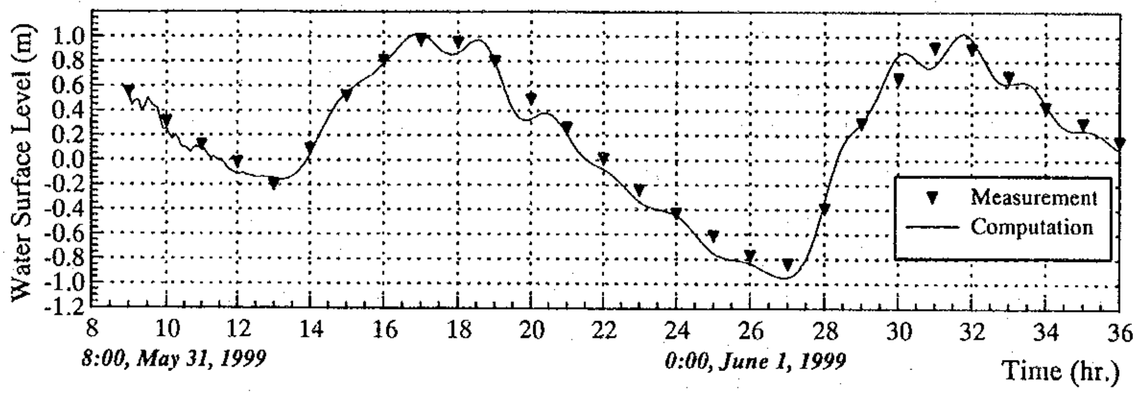


Figure 3.18 Hydrograph of Water Surface Level Measured at the Downstream Boundary during May 31 ~ June 1, 1999

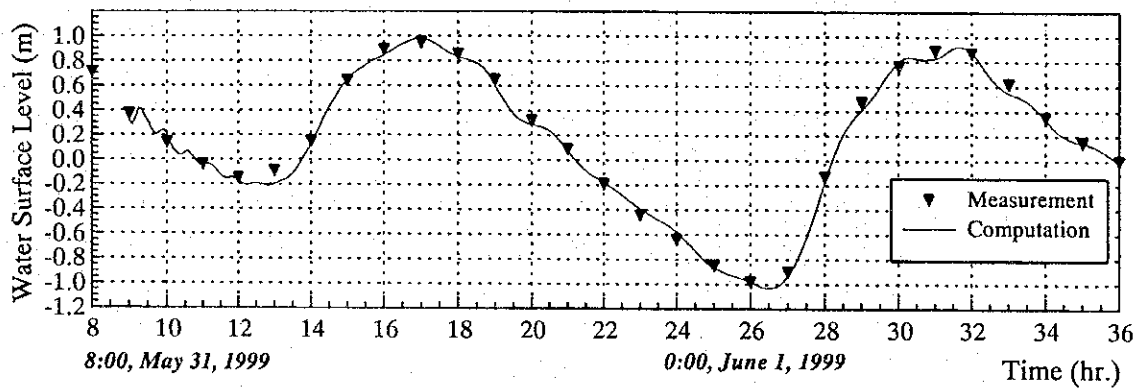
Figure 3.19 (a)~(b) showed the comparison of the computed water surface levels at the mouth of Can Tho River and at nearly upstream end with the measurements, respectively. The measurements of the water surface level at the downstream boundary had been used as the downstream boundary condition. The measurements of the velocity at the upstream boundary (the vertical line No.1~5) and at the mouth of Can Tho River (the vertical line No.19~20) had been employed to calculate the corresponding discharges and specified as the corresponding boundary conditions. The remained were employed to verify the model. The computed depth-integrated velocities and the measurements at the vertical lines were displayed in Figure 3.20 (a)~(f). During the hydrological survey from May 31 to June 1, 1999 the maximum discharge in ebb tide occurred at about 0:00 and the maximum discharge in flood tide occurred at about 7:00, June 1. The discharge and water level shown in the figures were those at the upstream end and at the downstream, respectively. The discharges at the mouth of Can Tho River were 1,783m^3/s at 0:00 and -1,312m^3/s at 7:00.

The comparisons showed that the computed water surface levels mimicked perfectly the measurements. From the overall viewpoints, the computed velocities, both their magnitudes and directions, coincided very well with the measurements at all the verticals. Small difference between the computation and the measurement did exist at several verticals. The causes were multiple and very complicated. First of all, the river network in Can Tho reach of the Hau River is complicated. Besides Can Tho river, Tra Mon river, Hai Qui river and Cai Von river in Vinh Long side and Binh Thuy river and Cai Sau canal in Can Tho side, which are the main tributaries with certain scale in the concerned reach, converge into the Hau River. As stated above, the discharge at the mouth of Can Tho river was larger in the hydrological survey from May 31 to June 1, 1999. Therefore, it could be expected that the effect of these tributaries on the flow in the Hau River was also not negligible. Unfortunately, there were no any data on flow discharge in these tributaries. In the simulation, these tributaries had to be ignored. On the other hand, the feature of the computed velocity hydrograph at vertical No.18, which was located at the right channel from the Cu Lao Lat Island, was similar to that of the discharge at the mouth of Can Tho River. Due to serious deposition of sediment, the front area of Cu Lao Lat Island, as shown in Figure 3.15, is very shallow. It hindered water in the main waterway from exchanging with that in the right channel of the island. The computed results showed that velocity in this area was very slow. Therefore, the flow features in the right channel of Cu Lao Lat island were closely influenced by the Can Tho River.

It could be concluded by these comparisons and analyses that the present model reproduced correctly the flow characteristics in the proposed Can Tho bridge site of the Hau River. The model can be employed to predict the flow in the concerned reach of the Hau River.

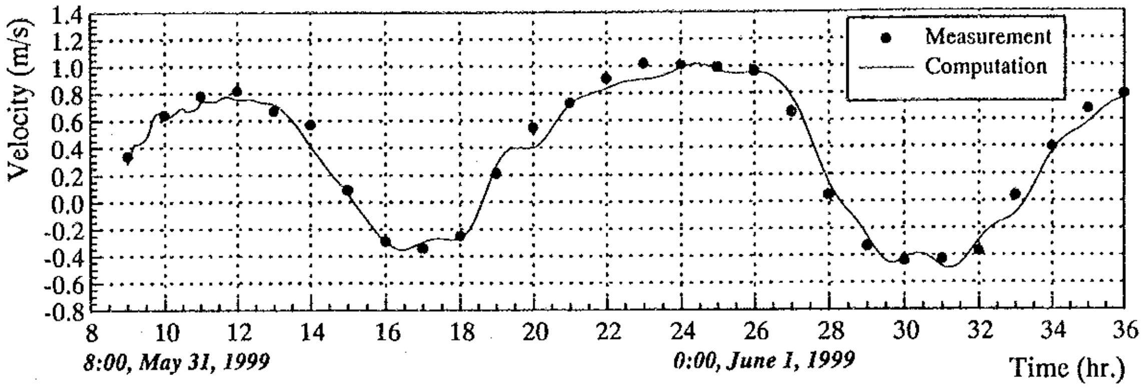


(a) At the Upstream Boundary

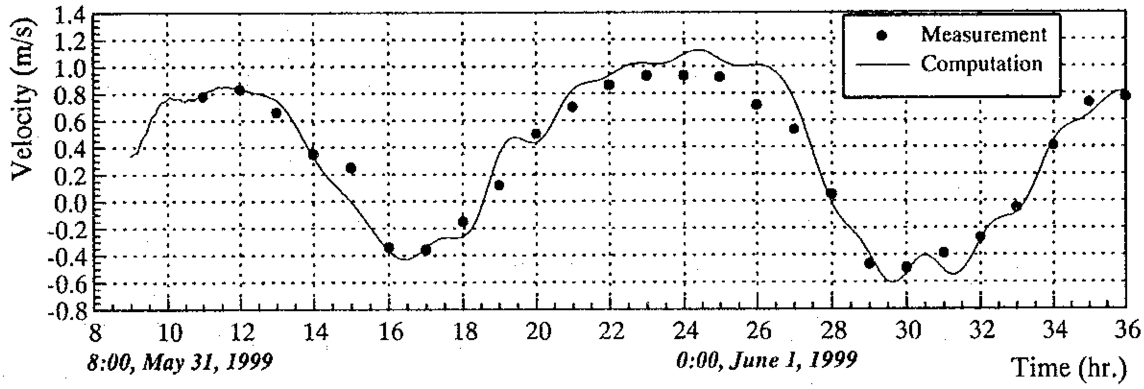


(b) At the Mouth of Can Tho River

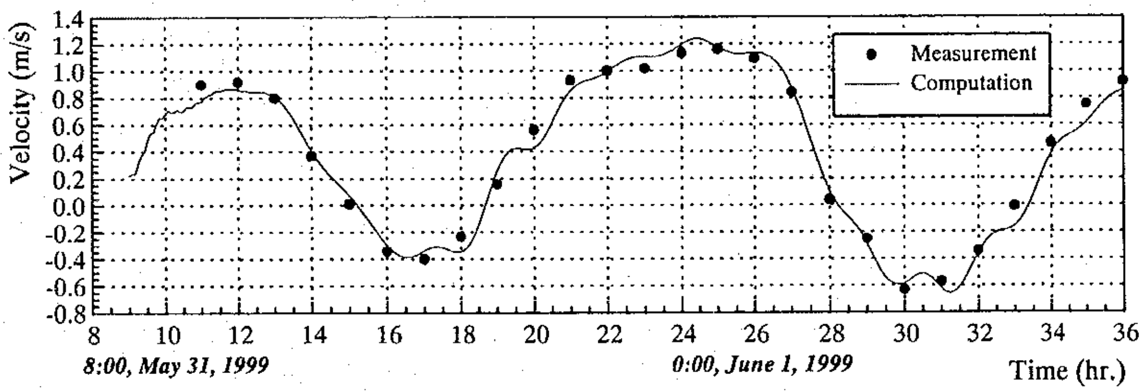
Figure 3.19 Hydrograph of Water Level



(a) At the Vertical No.6

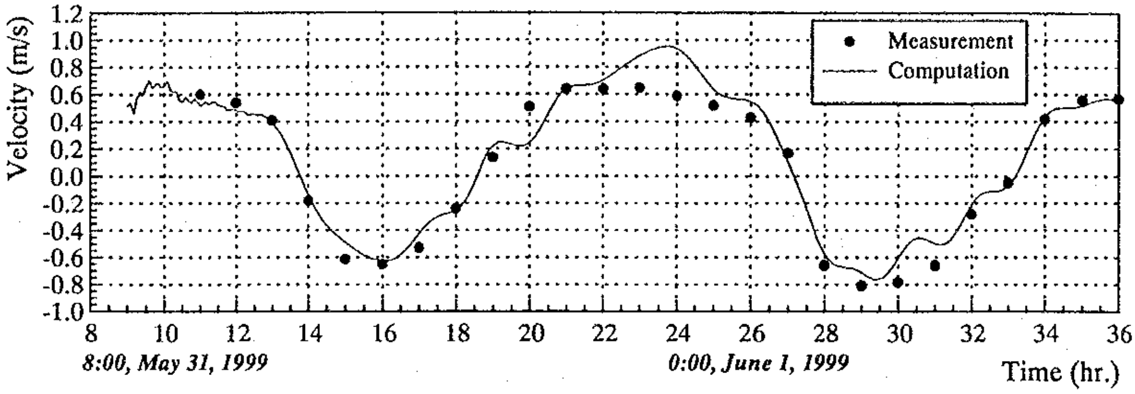


(b) At the Vertical No.10

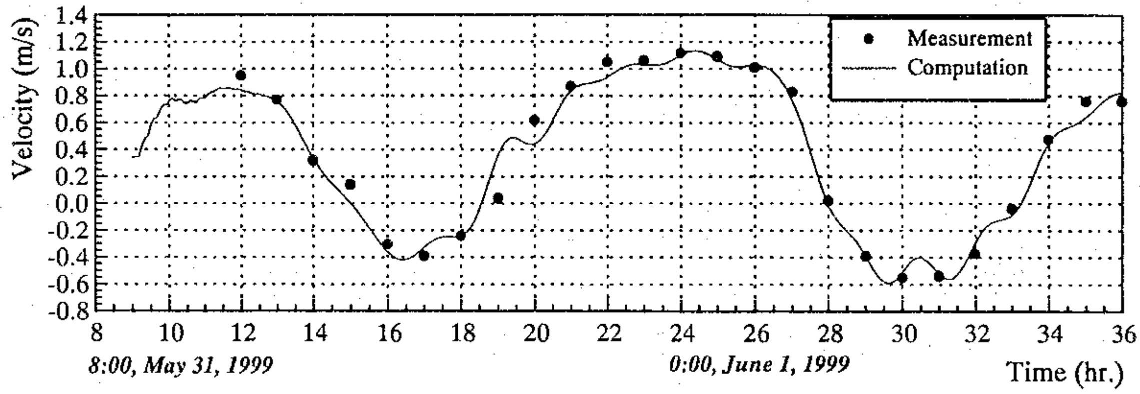


(c) At the Vertical No.11

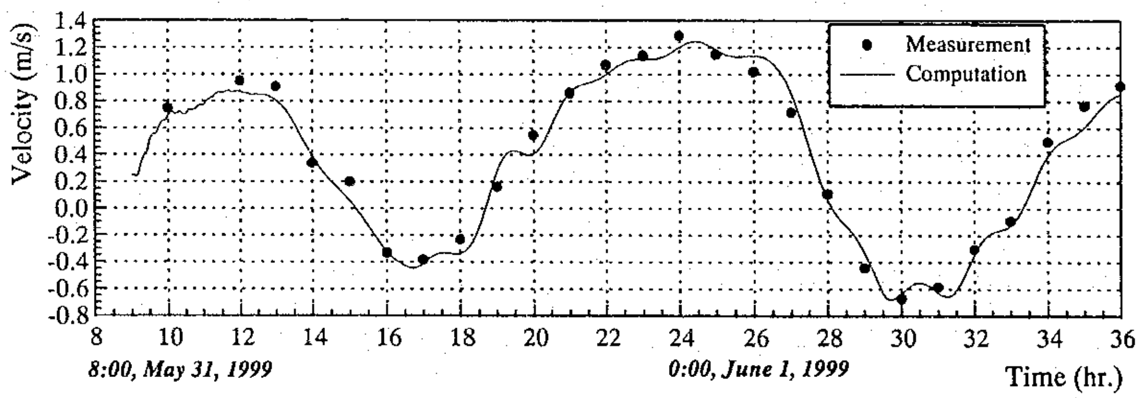
Figure 3.20 Hydrograph of Depth-Averaged Velocity (continued)



(d) At the Vertical No. 13



(e) At the Vertical No. 17



(f) At the Vertical No. 18

Figure 3.21 Hydrograph of Depth-Averaged Velocity

(3) Prediction of Flow and Bed Deformation

The flow pattern past a cylinder, protruding vertically from riverbed, is very much complicated in detail, and the complexity increases with the development of a scour hole in the surrounding of the cylinder. The pier of south tower consists of 6 circular cylinders with diameter 10m, in which the three are in upstream side of the bridge centerline and the others in the downstream side. Flow in vicinity of bridge pier is very complicated. The cylinders will affect each other. Therefore, it is anticipated that the features of the flow and bed deformation in the ambient of the south tower will be quite different from those around a single cylinder. For capturing the features reasonably, a completely three-dimensional model with the standard k-ε turbulence closure was applied to the surrounding of the bridge south tower. In order to sophisticatedly evaluate flow features and local bed deformation in vicinity of the bridge piers, computational meshes in the surrounding of the main pier (the south tower of the bridge) were refined. Besides the south tower, there are other 16 sub-piers which will be set in right side of the south tower. The effect of these sub-piers on the flow was taken into account as form resistance to the flow in the simulations.

Major scouring usually occurs during floods, that is, when the flow is unsteady and may even be angled to low flow direction. Therefore, design flood with the recurrence interval 100 years was set as prediction flow condition. Design flood discharge with the recurrence interval 100 years - 30,529m^3/s at Can Tho of the Hau River was used as the prediction flow condition. This design flood discharge was directly specified at the upstream boundary of the computational domain shown in Figure 3.12. Then a hydrograph of discharge at the upstream boundary was created according to the regularly hourly measurements of flood discharge at Can Tho and the results of this hydrological survey from May 31 to June 1, 1999. The recorded maximum flood discharge - 27,900m^3/s at Can Tho occurred in 1991, but there were no hourly records at that time. Recently maximum flood discharge in the hourly discharge measurements at Can Tho, which was 23,800m^3/s, was recorded on September 24, 1994 with the corresponding minimum discharge 9,590m^3/s in the flood tide. Therefore, the prediction discharge amplitude was determined to vary from 9,590m^3/s to 30,529m^3/s. The corresponding recorded water level at Can Tho varied from 0.2m to 1.26 in that period. This was employed to create the hydrograph of water level, incorporated with the design higher water level 1.85m at the recurrence interval 100 years. Water level difference in ebb tide between Can Tho gauging station and the downstream boundary could be estimated to be about 0.1m from the measurements of this hydrological survey. The measurements showed that the difference in flood tide was small. So the prediction water level at the downstream end was determined to vary from 0.69m to 1.85m.

The created prediction hydrograph of discharge at the upstream boundary, varying from 9,590m^3/s in flood tide to 30,529m^3/s in ebb tide, was shown in Figure 3.22. The prediction hydrograph of discharge at the mouth of Can Tho River, varying from 645m^3/s to 5,073m^3/s, was extended from the measurement hydrograph in this hydrological survey in proportion to that at the upstream boundary. It was displayed in Figure 3.23. The hydrograph of water level at the downstream boundary was created according to the measurement hydrograph in this hydrological survey with the amplitude from 0.69m to 1.85m and is shown in Figure 3.24.

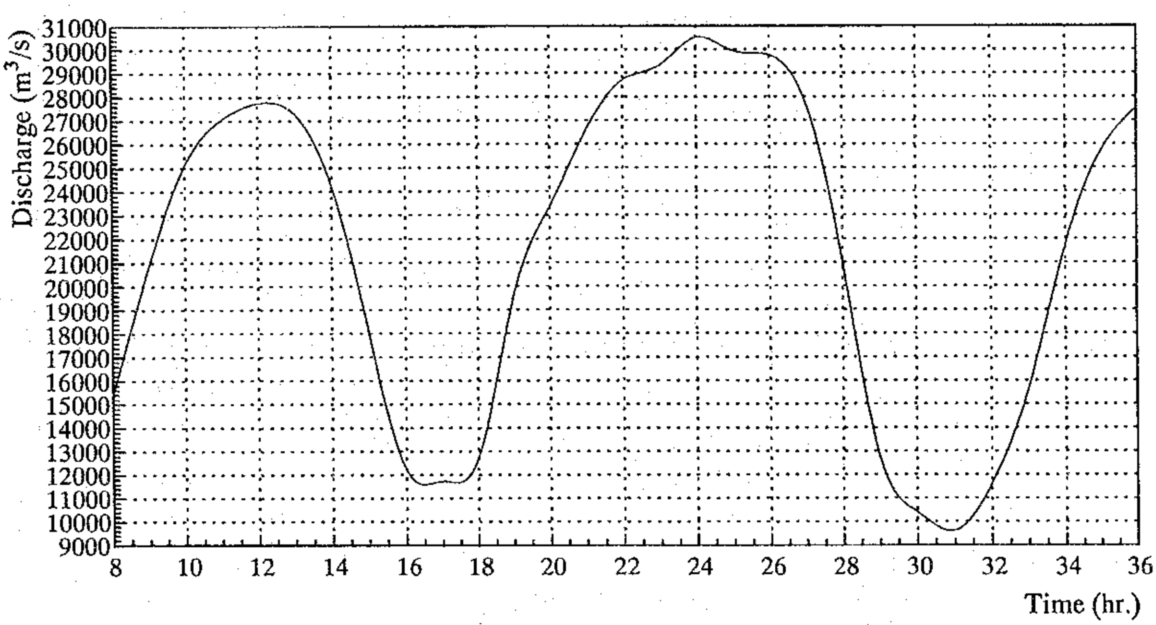


Figure 3.22 Hydrograph of Design Discharge at the Upstream Boundary with the Recurrence Interval 100 Years

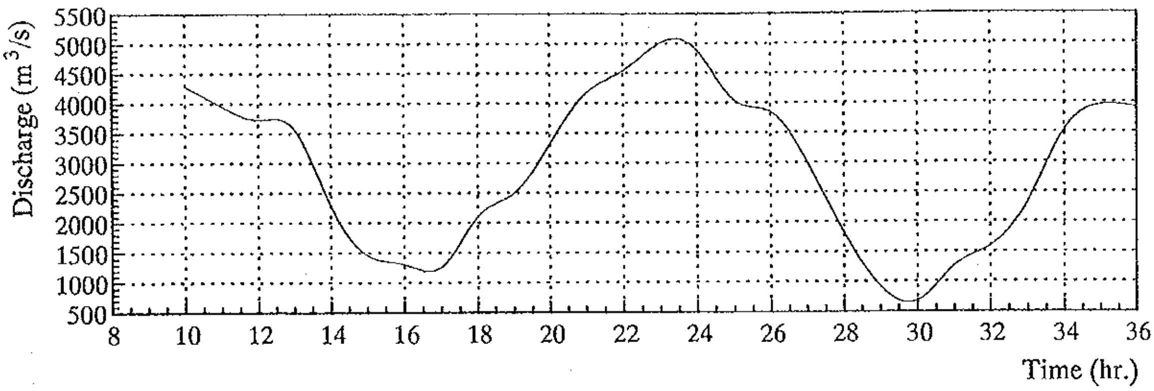


Figure 3.23 Hydrograph of the Corresponding Design Discharge of Can Tho River

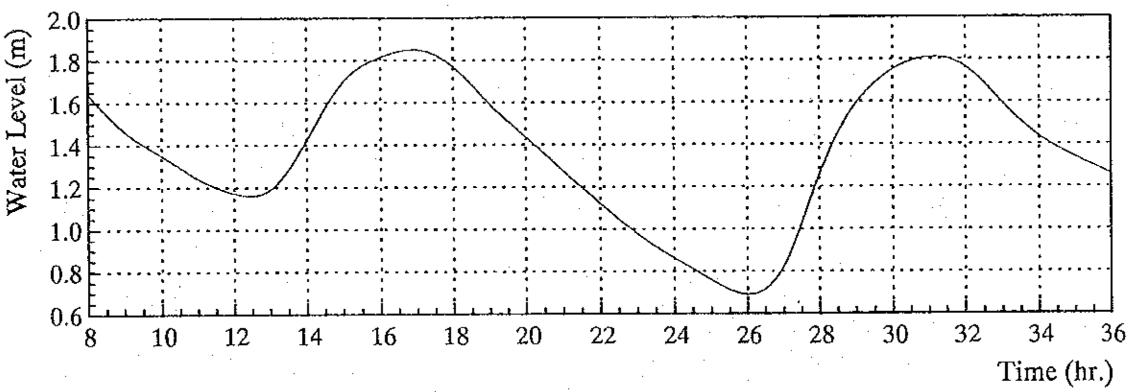
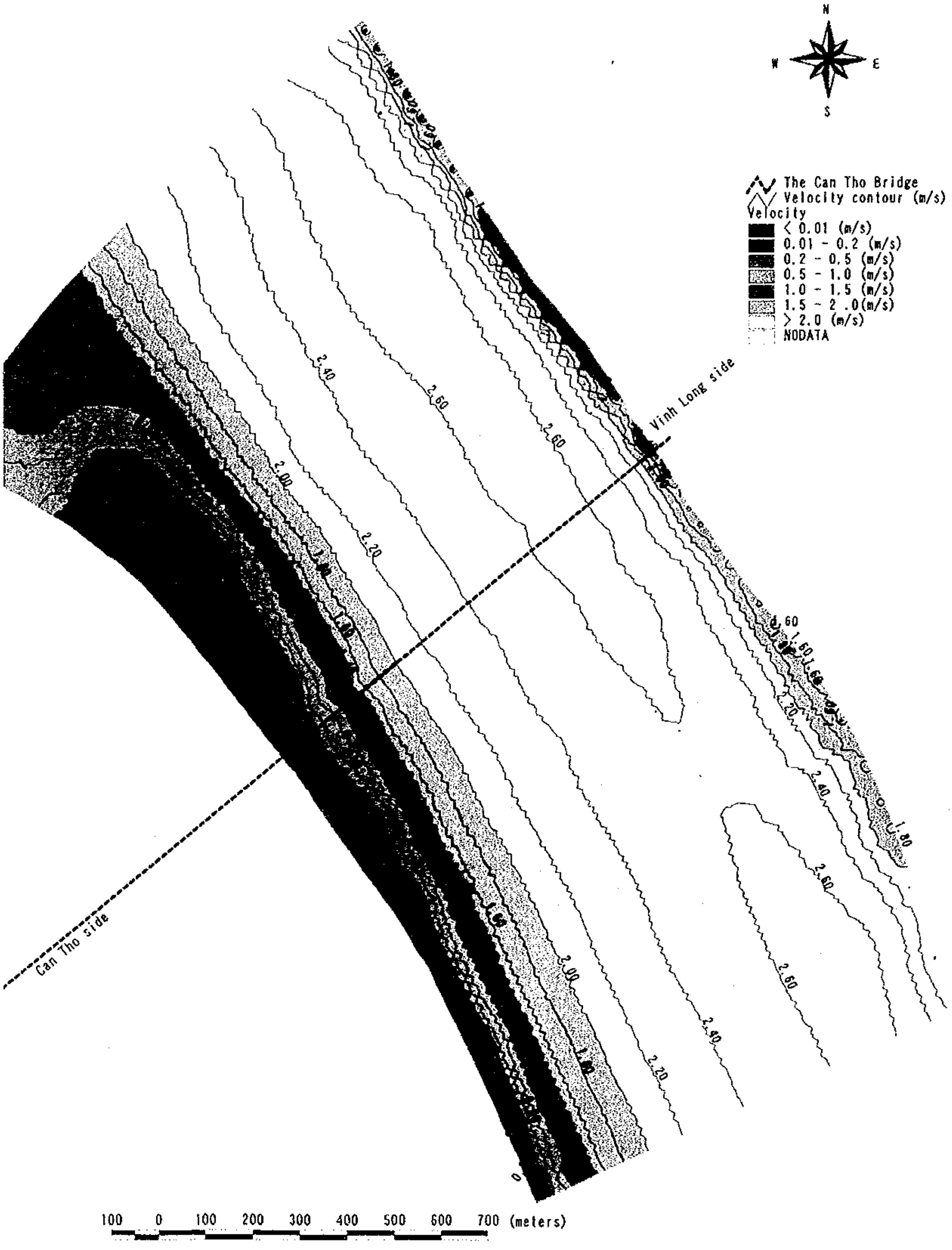


Figure 3.24 Hydrograph of the Corresponding Water Level at the Downstream Boundary

a) Effect on the flow of the bridge piers

Velocity magnitude and water level contours in the main channel from Cu Lao Lat Island at peak of the design flood were plotted in Figure 3.25 and Figure 3.26, respectively. As comparisons, the corresponding computed results without the bridge were plotted in Figure 3.27 and Figure 3.28, respectively. The discharge and water level shown in the figures were those at the upstream end and at the downstream, respectively. The corresponding discharge at the mouth of Can Tho River was 4,463m^3/s.

It showed that effect of the south tower on the flow was limited in the front area within 150m from the centerline of the bridge. However, the influence could last until 800m downstream area. The water level in the upstream side near the bridge may rise about 2cm due to the existence of the piers. The maximum velocity at peak of the flood in the bridge site will exceed 3.0m/s due to the effect of the bridge. The corresponding maximum velocity at the site is about 2.5m/s if the bridge doesn't exist.



THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.25 Velocity Contours at the Peak Design Flood (Discharge = 30,512m^3/s, Water Level = 0.86m)
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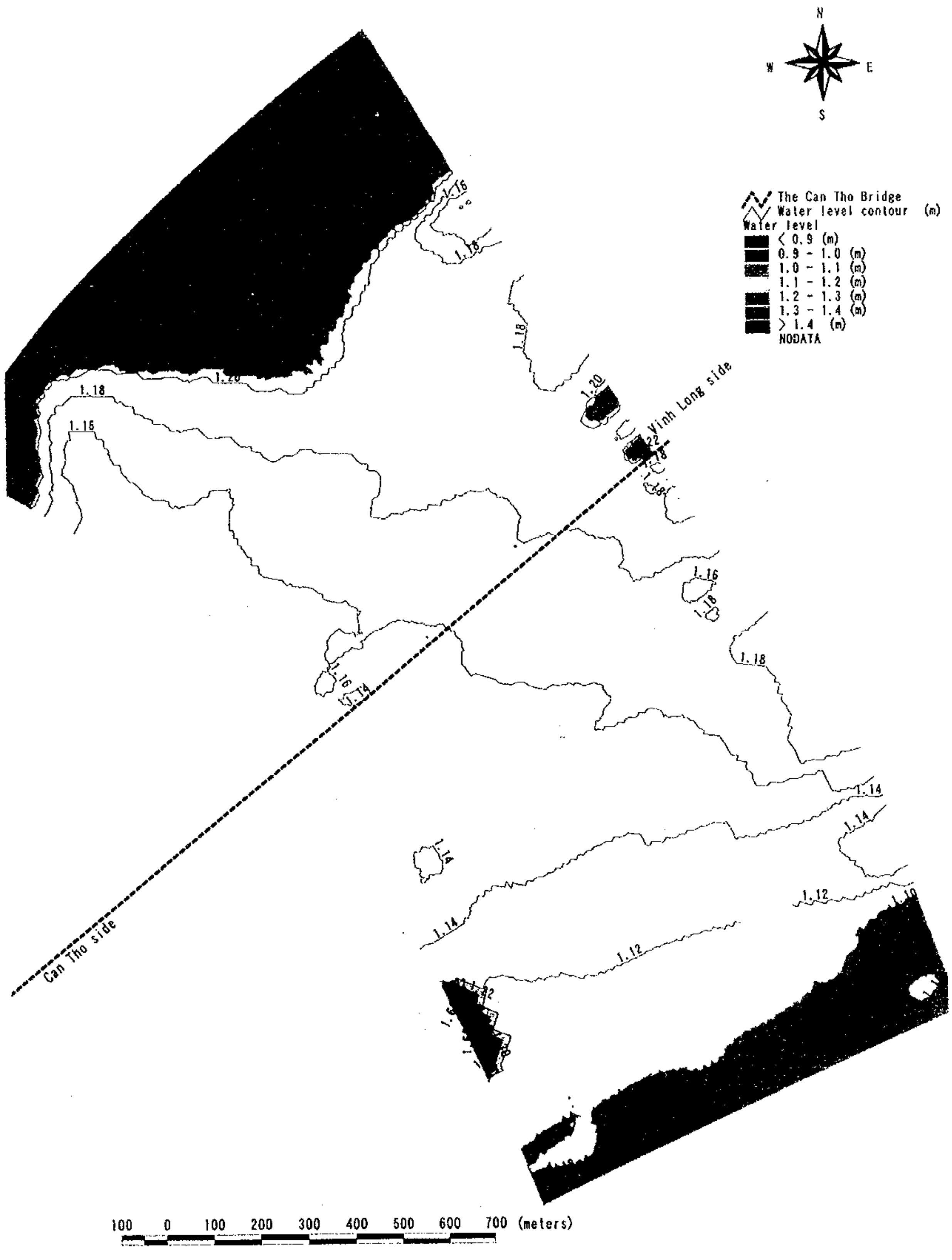
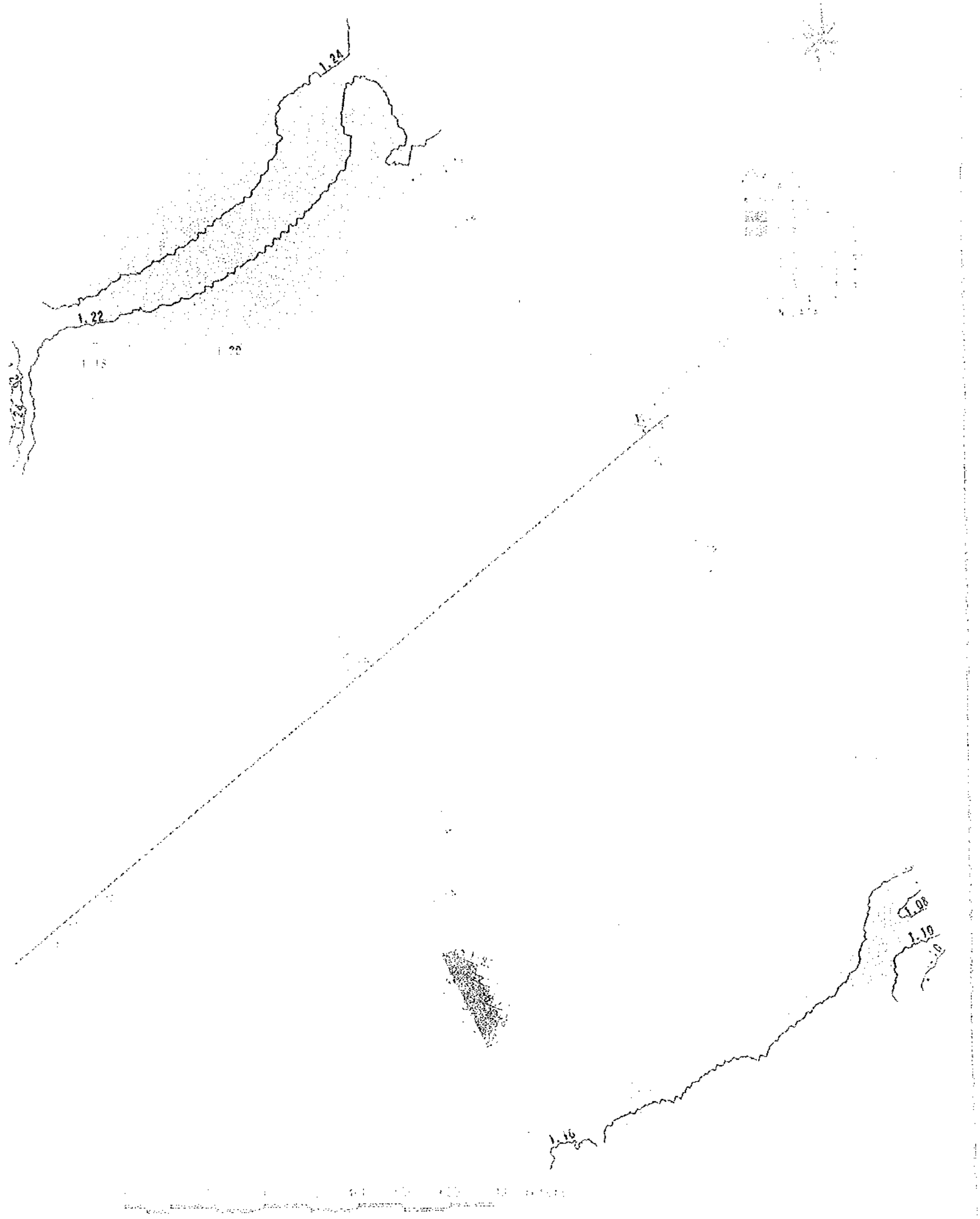


THE DETAILED DESIGN OF THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Figure 3.26 Water Level in the Main Channel at the Peak Flood (Discharge = 30,512m^3/s, Water Level = 0.86m)
	JAPAN INTERNATIONAL COOPERATION AGENCY



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