9.3 Appropriate Bridge Type

9.3.1 Appropriate Bridge Type and Reasons for Selection

(1) Main Bridge

A Hybrid Cable-stayed bridge for the main span is recommended for the following reasons:

- The Hybrid (steel and concrete) bridge types can maximize the using of construction materials locally procured, which in turn can economize the bridge construction cost compared with an all steel bridge type.

- The longer span of the Hybrid Cable-stayed bridge type can minimize the number of piers since the foundations have to penetrate into the bearing stratum 90 m to 95 m deep.

- The Hybrid Cable-stayed bridge type can be applicable to longer spans, that means, it is able to be free from the hydrological and hydraulic problems such as river bank erosion, local scouring around piers, required horizontal navigational clearance, and can minimize the girder depth for the higher vertical navigational clearance (39.0 m).

- The cable-stayed bridge with a partial concrete structure (hybrid system) can be advantageous in the case of aerodynamic stability.

- The towers and cables can provide symbolic and landmark views, which is excellent from the aesthetic aspect.

(2) Approach Span Bridges

Balanced-cantilever PC Box type and PC-Box Girder types are recommended for the approach span bridges.

The balanced-cantilever PC Box type is an appropriate and economic bridge type for the river branch, considering its required navigational clearance.

PC-Box Girder types are recommendable for the other portion of the approach span bridges as an economic, conventional, and locally familiar type of superstructure.

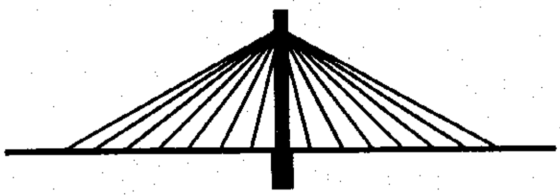
Technical assessment and optimization for these types of the approach span bridges were described in Chapter13, "PRELIMINARY DESIGN".

9.3.2 Design Considerations on the Superstructure Type

(1) Design of the Cable System Bridge

In the case of a cable-stayed bridge, the cable-stayed arrangement, shape of tower and supporting system of the main girder can be determined from the structural characteristic comparison as follows:

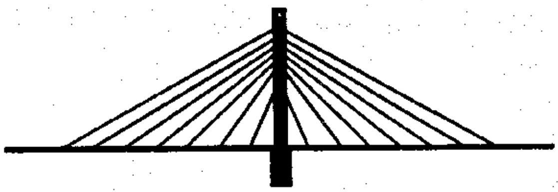
a) Cable-staying Arrangement



Radial layout

Radial layout

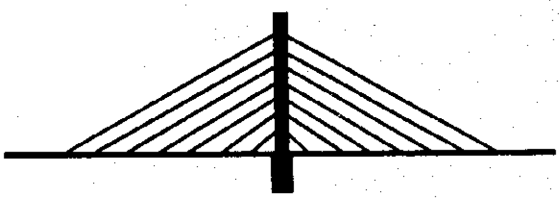
All the cables converge at the top of the tower, so the anchors concentrate on a restricted section of the tower.



Fan layout

Fan layout

The cables are regularly spread along the top part of the tower. It does not differ much from the radial layout but has the advantage of easier anchoring, since the cables are further apart.

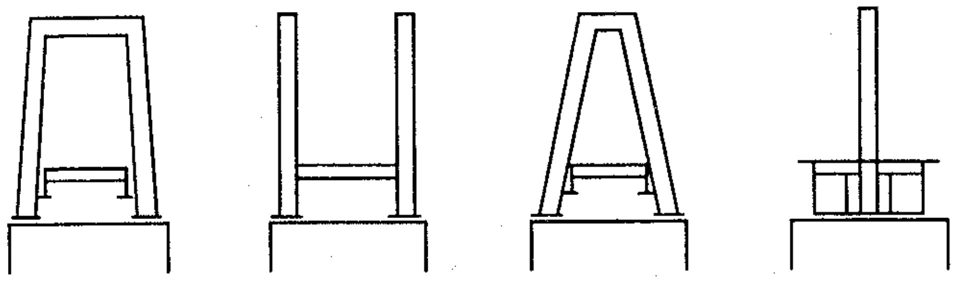


Harp layout

Harp layout

The cables are parallel to each other and reduces the risk of tower instability, as the anchorage points are spread along the whole height of the tower. This allows simpler construction procedures, and is suitable for concrete tower construction.

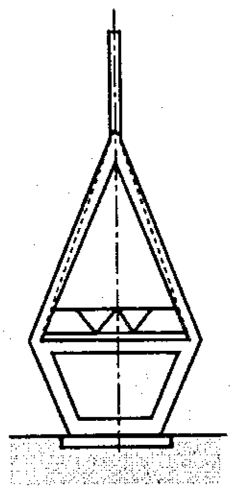
b) Shape of Tower



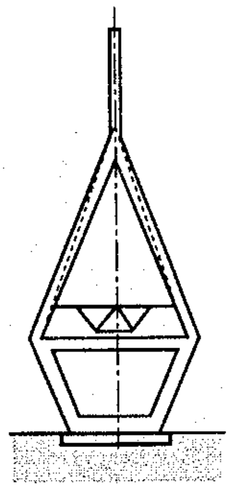
Portal Tower Twin Towers A-frame Tower Single Tower

A portal tower type was used in the design of early cable-stayed bridges, as in the case of suspension bridges where this type was commonly used to obtain stiffness against the wind load, which the force of the cable transfers to the towers. However, later investigations of cable-stayed bridges indicated that the horizontal forces of the cables were relatively small, so that freely standing tower legs could be used without any disadvantages. The inclined state cables give a stabilizing restraint force when the top of the tower is moved transversely. With single towers with no cross-members, the tower is subject to displacement at the top due to wind forces, the length of the cables is increased, and the resulting increase in tension provides a restoring force. Longitudinal moment of the tower is restricted by the restraining effect of the cables fixed at the saddle or tower anchorage.

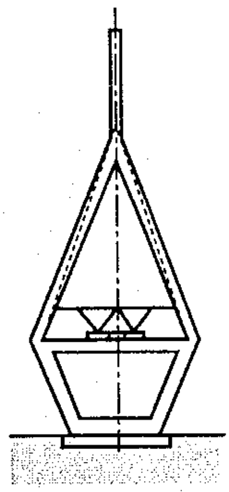
c) Supporting System of Main Girder



Rigid Frame System



Floating System



Bearing Shoe System

The simplest and the best way is to connect a girder and pier firmly with concrete (called a rigid frame structure). This method is frequently applied to a two-span cable-stayed bridge with one tower since the connection is stable and economical. This connection, however, may not be applied to a three-span cable-stayed bridge with two towers because it is usually difficult to prestress a girder if both connections are rigid structures except for the flexible high piers. Otherwise a floating system must be applied to a three-span cable-stayed bridge. Since the metal shoes are movable, the bearing shoe system can give a structural solution to the three-span continuous cable-stayed bridge.

(2) Hybrid Cable-Stayed Bridge

For cable-stayed bridges longer than 300 m span, it is effective to apply a hybrid structure. The bending moment of the girder due to dead load of a PC cable stayed bridge is bigger than a PC hybrid cable stayed type and the amount of cable stay to be applied for a hybrid type is less than a PC cable stay type due to the light weight of the steel girder. Therefore, a PC hybrid type is more economical.

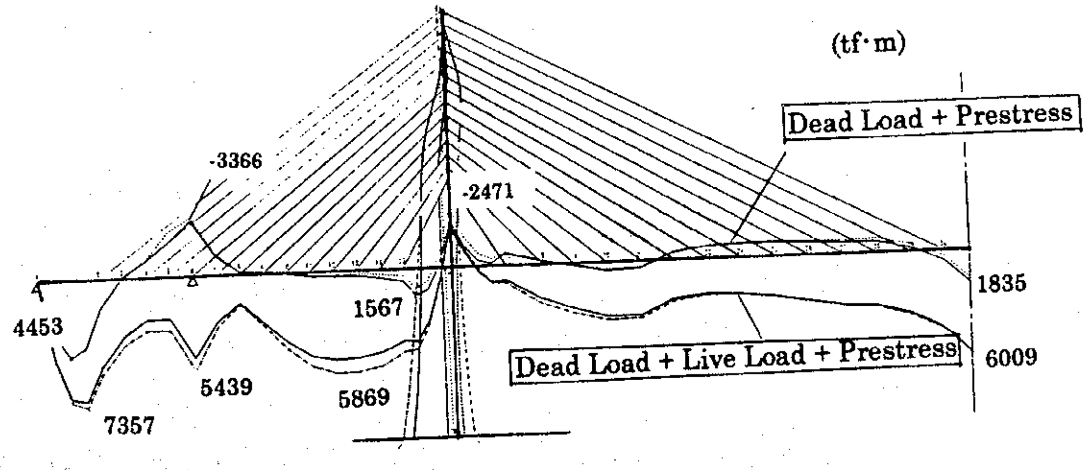


Fig. 9.5 Moment Diagram of the Hybrid Cable-Stayed Bridge

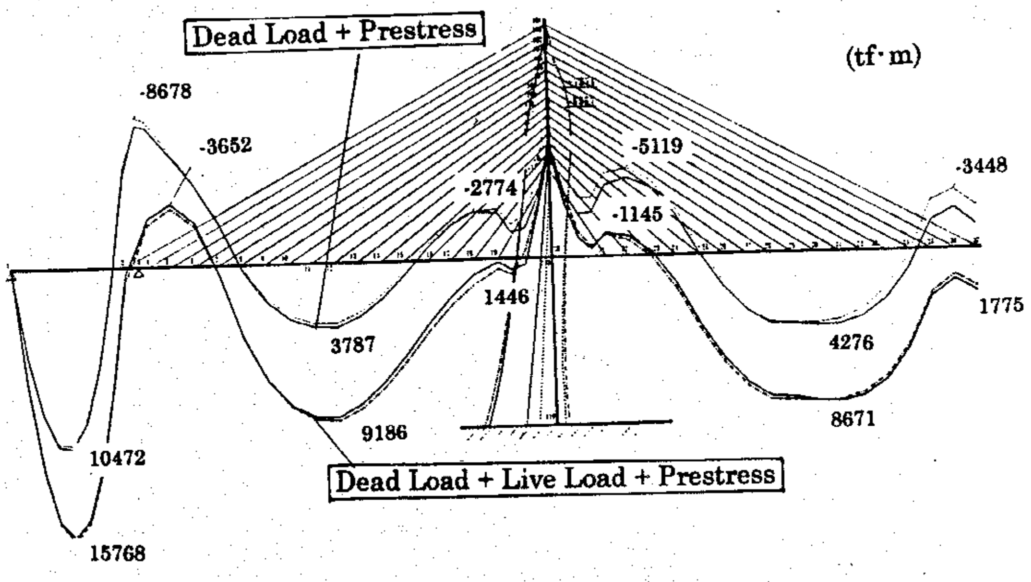


Fig. 9.6 Moment Diagram of the PC Cable-Stayed Bridge

(3) Connection of Hybrid Structure

It is not appropriate to connect directly a rigid PC and flexible steel girder. The rigidity for bending of a PC girder is extremely different from that of a steel girder and the behavior of bending under dynamic loads (live loads etc.) is also quite different.

Stress does not flow smoothly at the connection point between steel and concrete structures, and there is a reduction of allowable tensile stress due to fatigue. Therefore, it is necessary to provide a buffer zone considering the extreme variation of rigidity between a PC and steel girder.

The following connection method for a PC and steel girder is normally used. The connection point should be located near the pier position. The accumulated high compressive force due to the horizontal component of the prestressing stay force acts on the girder section at the connecting point and is thus much more stable for bending. It has been shown after considerable experimental study that the above-mentioned method to connect a hybrid structure is most effective.

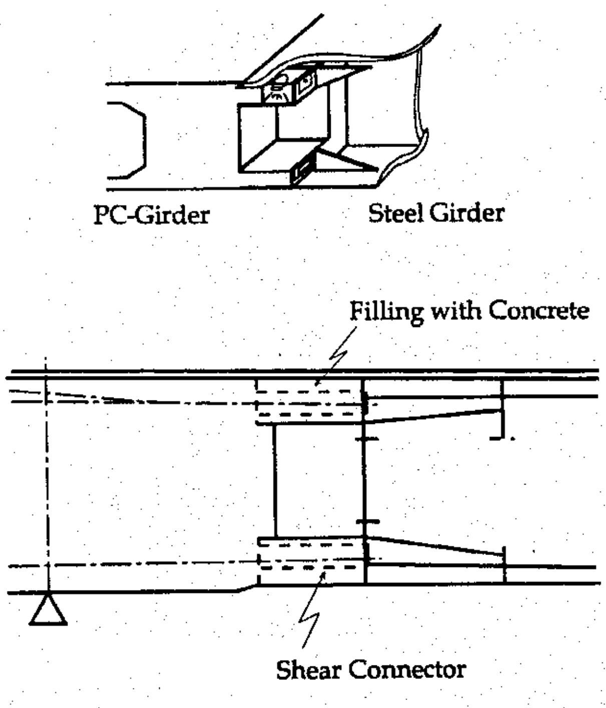


Fig. 9.7 Concept of Connection of the Hybrid Structure

9.3.3 Substructure Type

The substructure type to be designed closely relates with the structural system of the superstructures, especially in the cases of cable system bridges. Normally, the cable system bridge requires higher towers. Furthermore, it is crucial point for the tower of the Can Tho Bridge that the projected height from the soil (which is to be the bearing stratum of the bridge foundations), is significantly large since the projected height will be estimated as the integrated height of the vertical navigational clearance, river water depth and local scour depth around the bridge pier. Therefore, an ideal tower height should be as small as possible subject to the structural analysis of the bridge. The typical layouts of towers for each applicable type of the main bridge are shown in Fig. 9.8.

9.3.4 Foundation Type

The following five foundation types are applicable for the foundations of the main and approach bridges for the Can Tho Bridge from both technical and economic viewpoints (see Fig. 9.9).

1) Cast-in-place Concrete Pile
2) Steel Pipe Pile
3) Open Caisson
4) Pneumatic Caisson
5) Cast-in-situ Diaphragm Wall

The figure below shows the technical and economic features of each foundation type. From the subsoil conditions and easier methods of construction, cast in place piles (ϕ2,000 m/m), driven steel pipe pile (ϕ2,000 ~ ϕ2,500 m/m) and multi open caisson (ϕ10,000 m/m) using a jack force sinking down method are the most cost effective type.

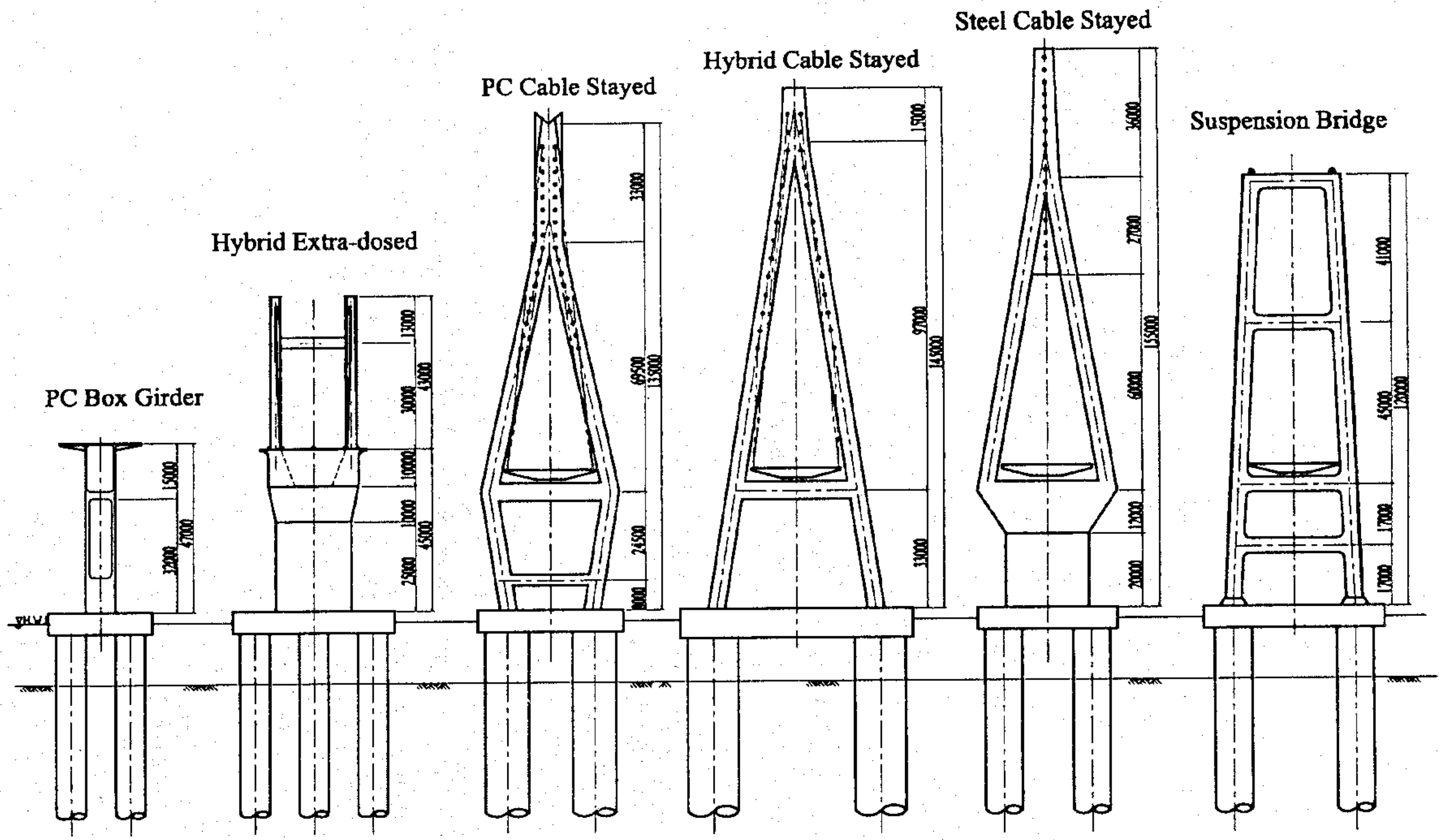


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Fig.9.8 Configuration of Substructure

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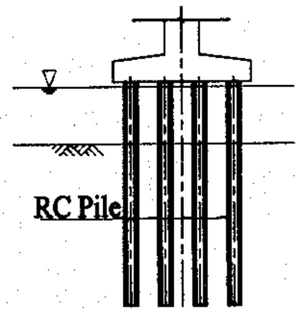
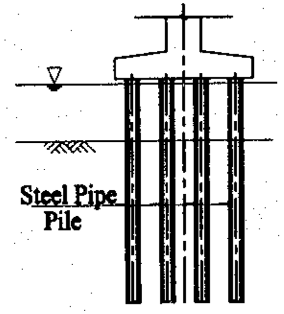
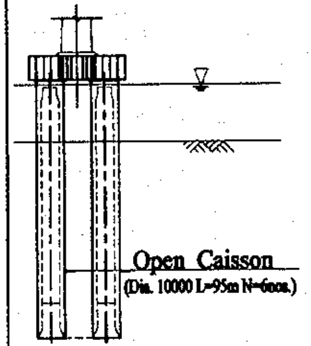
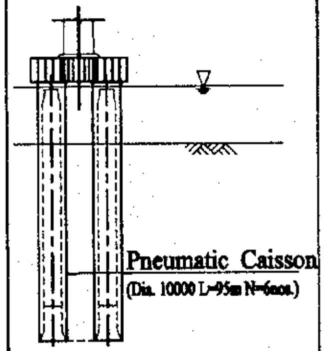
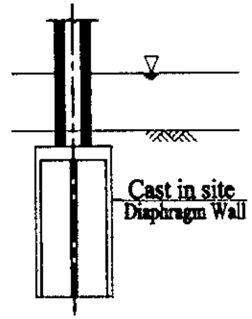
Foundation Type		① Cast-in-Place Concrete Pile	② Steel Pipe Pile	③ Open Caisson	④ Pneumatic Caisson	⑤ Cast-in-Site Diaphram Wall
Configuration of Foundation						
Construction Aspect	Construction Method	• Steel casing pipes are driven by vibrohammer. Bore holes are made using a reverse-circulation drilll. Pile cap is supported on the piles.	Placing of piles, placing method of pile installation of inner excavation method (reverse circuration drill method)in case of large resistance on middle layer	• Open caisson with under water excavation and jack down method.	• Completely automatic excavation system in pneumatic caisson method.	• Multi box type diaphragm wall • Trenching excavation in rotating type excavation
	Procurement of Materials	• Steel casing pipes are imported. • Platform and large capacity crane.	• Working platform. • Pile driver, floating crane(FC).	• Working platform. • Under water excavation system. • Temporary artificial islands.	• Working platform. • Completely automatic excavation system. • Temporary artificial islands.	• Temporary artificial islands. • Large capacity crane.
	Construction Period	8 months/pier	7 months/pier	10 months/pier	10 months/pier	9 months/pier
	Construction Cost(Ratio)	1.15	1.05	1.00	1.50	2.10
	Difficult Point Of Construction	• Construction accuracy and quality control are important because of long piles. • Scour on foundation peripherals.	• If thick middle clay layer, operate reverse-circulation drill method. • Large scour affect on foundation peripherals	• Because of large scale of temporary works platform, large blocking at the cross sectional area of river occur during construction period. • Temporary artificial islands are required.	• If high pore water pressure, will cause high air pressure in the tube, then there will be difficulty in construction. (for excavation system control and maintenance purpose)	• Temporaly artificial islands compulsory, distore, dismantle of soil also compulsory
Alternative A		○	○	○	△	△
Alternative B		○	○	○	△	△
Alternative C		○	○	○	△	△

○ ; Good   △ ; Fair   × ; Bad

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IN SOCIALIST REPUBLIC OF VIET NAM

Fig.9.9 Comparison of Foundation Types (Main Bridge)

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on The Can Tho Bridge Construction in
Socialist Republic of Viet Nam

CHAPTER 10

PRELIMINARY EVALUATION OF THE ALTERNATIVE ROUTES

CHAPTER 10 PRELIMINARY EVALUATION OF THE ALTERNATIVE ROUTES

10.1 Preliminary Benefit Estimate

10.1.1 General

The economic evaluation pursued in this chapter assesses the feasibility of alternatives in the context of the project entity rather than the economic development of the Mekong Delta. Therefore, the economic evaluation emphasizes quantifiable direct benefits/costs in monetary terms. Indirect benefits are not estimated in this chapter, though they are estimated in the following chapters on economic evaluation.

10.1.2 Direct Benefits

(1) Vehicle Operating Costs (VOC)

In this study two types of VOCs were estimated, they are time related VOCs and distance related VOCs. Time related VOCs are constituted of passenger time costs, crew costs, cargo time costs and standing costs.

Assuming the yearly vehicle utilization in terms of hours, the standing costs are interpreted as time related costs. Distance related costs are constituted of costs for vehicles, fuel, oil, tires, maintenance and related costs. Time related costs are included in the distance related costs in this study for the estimate of changes in VOCs accruing from the alteration of route due to the construction of the bridge.

All costs are based on constant 1997 prices and were calculated in terms of their financial and economic costs. Economic costs do not include taxes and fees.

(2) Ferry Operation and Improvement Cost

Those costs can be considered as savings in the ferry operations in the "with bridge" cases and interpreted as benefits of the bridge construction.

Also, the additional investment for ferries to cope with the overflow of future traffic can be diverted into benefit as a result of the bridge construction.

Since it is likely that ferry operation costs will increase as transport demand across the Hau River increases, the future ferry operational cost was estimated in proportion to the future transport demand crossing the Hau River.

The following unit costs of ferry improvement projects are taken into account in this study.

Purchasing new ferry boat (200t)	US$ 2.0 million / boat
Repairing ferry boat	US$ 0.7 million / boat
Administration and operation	US$ 0.27 million / boat-year

10.2 Preliminary Cost Estimate

10.2.1 General

The economic cost of the project is mainly composed of "the construction cost", and "the maintenance cost".

The economic construction costs were estimated by applying conversion factors to the direct cost as part of the total project cost.

A detailed estimate of economic construction costs is described in the next section, 10.2.2 Economic Construction Cost.

The economic maintenance cost was estimated by assuming a uniform percentage of the economic project cost by referring to OECD (Organization for Economic Cooperation Development) reports. Table 10.1 shows the composition of the economic and project costs.

Table 10.1 Composition of the Preliminary Economic Cost and Project Cost

Component	Economic Cost	Project Cost
1)Construction Cost (Direct / Indirect Cost) for: - Mobilization & Demobilization - Approach Roads, - Main & Approach Span Bridges	E(= 80% of P)	P
2) Engineering & Administration Cost	E(= 100% of P)	P
3) Land Acquisition	-	P
4) Compensation	-	P
5) Contingency	E*	P
6) Maintenance Cost	0.1% of 1) / year	-

E: Figures for Economic Cost

P: Figures for Project Cost

*: E = { 1) of P x 80% + 2) of P x 100% } x 5%

Contingency = { 1) + 2) } x 5%

10.2.2 Economic Construction Cost

The tax portion of the construction cost for each section of the work, i.e. approach roads, foundation, substructure, superstructure, and mobilization & demobilization, is estimated at about 20% of the project costs in general.

As a result, the economic construction cost for each section of the work was estimated at 80% of the project construction cost.

Engineering cost and administration cost is not usually regarded as construction cost, however, those costs were included in the economic construction costs in this section. This cost was then allocated to the period of construction based on the proposed implementation schedule.

(1) Mobilization and Demobilization

The cost of mobilization and demobilization was estimated as 5% of the total economic cost (Table 10.2).

Table 10.2 Cost of Mobilization and Demobilization

Unit: Thousand USD

Alternative route	Economic Construction Cost of Approach Roads	Economic Construction Cost of Main & Approach Spans	Economic Construction Cost Total	Economic Cost of Mobilization & Demobilization
A -1	17,126.7	180,247.1	197,373.8	9,868.7
A -2	13,519.3	180,247.1	193,766.4	9,688.3
B -1	15,967.4	200,258.5	216,225.9	10,811.3
B -2	9,527.7	200,258.5	209,786.2	10,489.3
C -1	18,375.2	154,707.7	173,082.9	8,654.2
C -2	20,641.1	154,707.7	175,348.8	8,767.4
C -3	43,203.0	154,707.7	197,910.7	9,895.5
C -2/3	40,256.6	154,707.7	194,964.3	9,748.2

(2) Approach Roads (Alternative routes and options for preliminary cost estimate)

There are three major alternative routes, and each major alternative route has 2 or 3 optional routes. Totally, there are 7 route options.

A general description of these alternative routes and options is shown in Table 10.3 and Fig. 10.1.

Table 10.3 Alternative routes and Options

Alternative route	Connection Point of Vinh Long Side	Connection Point of Can Tho Side	Total Alignment Length (m)
A-1	a-1 (N.H.No.1)	a-2 (N.H.No.1)	10,500
A-2	a-1 (N.H.No.1)	a-3 (N.H.No.91)	7,500
B-1	b-1 (N.H.No.1)	b-2 (N.H.No.1)	6,380
B-2	b-1 (N.H.No.1)	b-3 (N.H.No.91)	5,304
C-1	c-1 (N.H.No.1)	c-3 (N.H.No.1)	10,050
C-2	c-1 (N.H.No.1)	c-4 (N.H.No.1)	12,200
C-3	c-2 (N.H.No.1)	c-5 (N.H.No.1)	15,500
C-2/3	c-2 (N.H.No.1)	c-4 (N.H.No.1)	14,846

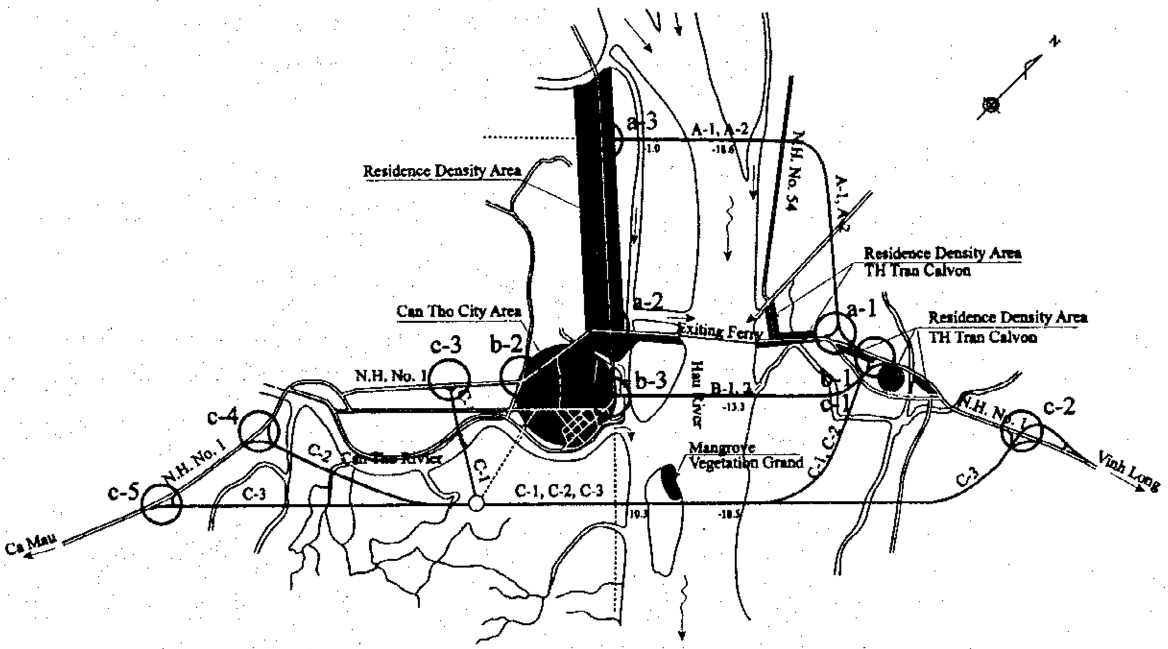

Fig. 10.1 General Alignment of Alternative routes and Options

Table 10.4 shows the estimated economic construction cost of each alternative route.

Table 10.4 Economic Construction cost of Approach Road
(includes structures)

Unit: thousand USD

Alternative route	Total Alignment Length (m)	Economic Construction Cost		
		Total	Embankment	Structures
A -1	10,500.0	17,126.7	9,287.6	7,839.1
A -2	7,500.0	13,519.3	5,697.0	7,822.3
B -1	6,380.0	15,967.4	3,585.8	12,381.7
B -2	4,900.0	9,527.7	2,125.6	7,402.1
C -1	10,050.0	18,375.2	8,928.5	9,446.7
C -2	12,200.0	20,641.1	12,255.8	8,385.4
C -3	15,500.0	43,203.0	15,583.0	27,620.0
C -2/3	14,846.0	40,256.6	13,908.3	26,348.3

(3) Main and Approach Bridge
(Selection of Most Appropriate Type of Main Bridge for Each Route)

Three alternatives of a combination of main and approach bridge types for each alternative route were described in the previous section. These main and approach bridge types were established considering the cost, hydraulic condition, meteorological condition, structural features, construction method, maintenance, aesthetic aspect, etc.

The total economic construction cost for each bridge alternative is shown in Table 10.5, and the construction cost of each construction item is shown in Table 10.6.

Table 10.5 Total Economic Construction Cost for the Main and Approach Spans

Unit: thousand USD

Alternative Route		Type of Main Span	Type of Side Span	Total Length of Bridges (m)	Total Economic Cost of Construction
A	1	Steel Cable Stayed	PC Extradosed	2,600.00	180,247.1
	2	Steel Cable Stayed	PC Extradosed	2,600.00	186,000.0
	3	Suspension	-	2,600.00	276,298.3
B	1	Steel Cable Stayed	-	2,904.00	200,258.5
	2	Steel Cable Stayed	-	2,904.00	208,365.2
	3	Suspension	-	2,904.00	271,723.7
C	1	Hybrid Cable Stayed	-	2,660.00	154,707.7
	2	Steel Cable Stayed	-	2,660.00	190,159.8
	3	Steel Cable Stayed	-	2,660.00	177,999.3

Table 10.6 Economic Construction Cost for the Individual Construction Work for the Main and Approach Spans

Unit: thousand USD

Alternative route		Total Economic Cost of Construction	Foundation	Substructure	Superstructure
A	1	180,247.1	50,311.2	21,561.9	108,374.0
	2	186,000.0	51,926.8	22,254.4	111,818.8
	3	276,298.3	77,286.0	33,122.6	165,889.7
B	1	200,258.5	55,912.9	23,962.7	120,382.9
	2	208,365.2	58,189.6	24,938.4	125,237.2
	3	271,723.7	75,983.1	32,564.2	163,176.4
C	1	154,707.7	43,135.3	18,486.5	93,085.9
	2	190,159.8	53,091.5	22,753.5	114,314.7
	3	177,999.3	49,676.4	21,289.9	107,033.0

The following types for each alternative route were selected as a favorable combination of main and approach bridge types, not only from cost, but also from technical points of view such as the hydrodynamics and span lengths required.

- Route A; Steel Cable-Stayed Bridge for the Main Bridge

  PC Extra-dosed Bridge for the Side Bridge

- Route B; Steel Cable-Stayed Bridge for the Main Bridge

  PC Extradosed Bridge for the Side Bridge

- Route C; Hybrid Cable-Stayed Bridge for the Main Bridge

  PC Extradosed Bridge for the Side Bridge

(4) Engineering and Administration Cost

The engineering and administration cost applied to the economic cost is shown in Table 10.7;

Table 10.7 Engineering and Administration Cost

Unit: thousand USD

Alternative route		Economic Construction Cost Total	Engineering and Administration Cost
A	-1	197,373.8	2,588.3
A	-2	193,766.4	2,541.4
B	-1	216,225.9	2,833.1
B	-2	209,786.2	2,749.5
C	-1	173,082.9	2,272.8
C	-2	175,348.8	2,302.2
C	-3	197,910.7	2,595.2
C	-2/3	194,964.3	2,557.0

10.2.3 Economic Maintenance Cost

0.1% of the project construction cost was estimated as the appropriate yearly maintenance cost.

This percentage was derived with reference to the guidelines set by OECD (Organization for Economic Cooperation and Development).

10.2.4 Tentative Implementation Schedule

A tentative implementation schedule is necessary to distribute economic cost on a yearly basis. Fig. 10.2 shows the general tentative implementation schedule applied for the economic evaluation.

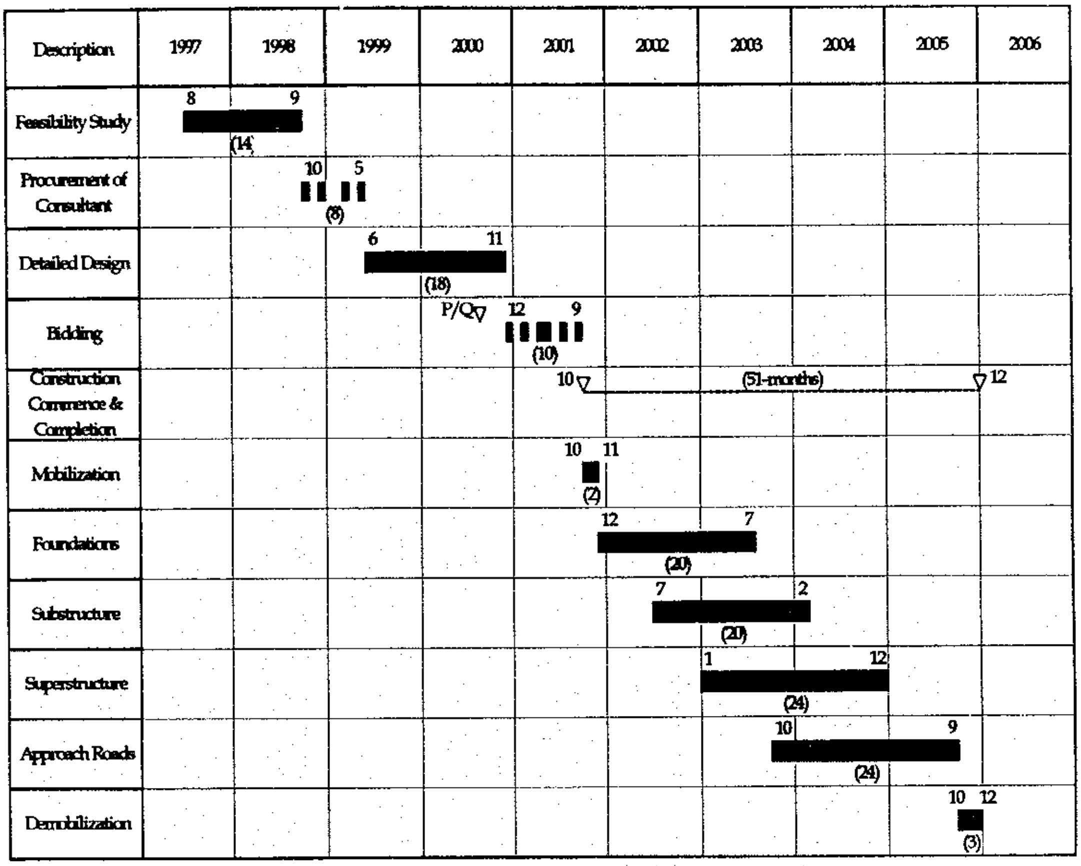

Fig. 10.2 General Tentative Implementation Schedule

10.2.5 Preliminary Economic Cost

A summary of the preliminary economic cost is shown in Table 10.8.

Unit: thousand USD

Route	A-1		A-2		B-1		B-2		C-1		C-2		C-3		C-2/3	
Type of Main Bridge	Steel Cable Stayed		Steel Cable Stayed		Steel Cable Stayed		Steel Cable Stayed		Hybrid Cable Stayed		Hybrid Cable Stayed		Hybrid Cable Stayed		Hybrid Cable Stayed	
Year	Cost		Cost		Cost		Cost		Cost		Cost		Cost		Cost	
	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance	Construction	Maintenance
1997																
1998																
1999	1,006.6		988.3		1,101.8		1,069.2		883.9		895.3		1,009.3		994.4	
2000	1,581.7		1,553.1		1,731.3		1,680.2		1,388.9		1,406.9		1,586.0		1,562.6	
2001	18,227.1		17,940.0		20,007.8		19,495.2		15,934.9		16,115.2		17,911.1		17,676.5	
2002	36,655.3		36,655.3		40,736.6		40,736.6		31,427.1		31,427.1		31,427.1		31,427.1	
2003	86,873.9		86,423.9		96,134.5		95,329.6		75,029.1		75,312.3		78,132.6		77,764.3	
2004	64,906.6		63,102.8		70,571.4		67,351.6		57,579.2		58,712.2		69,993.1		68,519.9	
2005	11,616.6		10,168.8		11,677.9		9,093.6		11,445.5		12,354.9		21,409.3		20,226.9	
2006		220.9		216.8		242.0		234.8		193.7		196.2		221.5		218.2
2007		220.9		216.8		242.0		234.8		193.7		196.2		221.5		218.2
-	~															
2054		220.9		216.8		242.0		234.8		193.7		196.2		221.5		218.2
2055		220.9		216.8		242.0		234.8		193.7		196.2		221.5		218.2
Total	220,867.7	11,043.5	216,831.3	10,841.5	241,961.3	12,098.0	234,755.9	11,738.0	193,688.6	9,684.5	196,223.9	9,811.0	221,468.4	11,073.5	218,171.7	10,908.5
	231,911.2		227,672.8		254,059.3		246,493.9		203,373.1		206,034.9		232,541.9		229,080.2	

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Table 10.8 Summary of the Preliminary Economic Cost

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10.3 Preliminary Economic Evaluation of the Alternative Routes

10.3.1 Conditions for Preliminary Economic Evaluation

Although the main objective of economic evaluation is to ascertain information on the viability of alternatives, economic evaluation in this chapter also has the role to provide information on the relative supremacy of alternatives from an economic viewpoint. The detailed economic evaluation to obtain information on an effective combination of investments or effective operations of a bridge are to be carried out in the later stages of this study.

The basic conditions for preliminary economic evaluation are as follows:

(1) Base Year

The start year of the Project, 1999, was set as the base year for the economic evaluation.

(2) Evaluation Period

Because of budgetary constraints it is not likely that in the near future many large bridge projects will be implemented in Viet Nam. Therefore, the project life in the economic evaluation should be considerable. A 50-year period after opening of the bridge was assumed, therefore, as the evaluation period.

(3) Evaluation Indicator

As evaluation indicators, the economic internal rates of return (EIRRs) were calculated for the evaluation period.

10.3.2 Evaluation Results

The economic internal rates of return (EIRRs) of the alternatives show values between 9.3-10.5%. The alternative C-1 shows the highest EIRR, followed by C-2, and A-2, C-3 and A-1. Comparing the EIRRs of the alternative routes, the alternative C-1 with a lower project cost has a relative advantage and shows a higher EIRR. Similarly alternatives with lower project costs at each river crossing point show higher EIRRs. Of these alternatives only C-1 and C-2 showed positive values of Net Present Value and the values over 1.2 with respect to Benefit Cost Ratio at the discount rate of 8%, and large value of NPV among alternatives (Table 10.9).

Table 10.9 Results of the Preliminary Economic Evaluation for the Alternative Routes

	A-1	A-2	B-1	B-2	C-1	C-2	C-3	C-2/3
EIRR	9.7%	9.8%	9.3%	9.4%	10.5%	10.4%	9.8%	9.9%

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CHAPTER 11

SELECTION OF ALTERNATIVE ROUTE

CHAPTER 11 SELECTION OF ALTERNATIVE ROUTE

11.1 Selection of Alternative Route

The most desirable alternative route for the Can Tho Bridge construction should be selected considering the engineering, economic, and environmental aspects of the Project. Table 11.1 summarizes the evaluation of the alternative routes including their options.

Route C is recommended as the most suitable route for the following reasons:

- Economically advantageous, i.e. the economic indicator (EIRR) is 9.8 ~ 10.5%.
- Less problems with the hydrological and hydraulic conditions of the river
- Less compensation for resettlement of houses and land acquisition
- Less influence on the stability of the ecosystems

The option C-1 is economically advantageous because the economic indicator (EIRR) is the highest (10.5%) among the group of options for Route C. However, the alignment is not smooth, compared with other options.

In terms of less compensation on land acquisition and resettlement of houses, and considering about traffic congestion and alignment, C-2/3 is recommended as a most suitable route with only a small difference of economic indicators compared with that of Route C-1.

Table 11.1 Evaluation by Alternative Route and Option

Description			A-1	A-2	B-1	B-2	C-1	C-2	C-3	C-2/3
			Alternative Routes and Option							
Outline of Each Route	Location		From N.H. No.1 connects to N.H. No.91	From N.H. No.1 joins N.H. No.91 and then connects to N.H.No.1	From N.H. No.1 to Can Tho city Street	From N.H. No.1 to Can Tho city street and then onto N.H. No.1	From N.H. No.1 to N.H. No.1 crossing Can Tho River with the shortest distance of Route C	From N.H. No.1 to N.H. No.1 after across the existing Can Tho River bridge	From N.H. No.1 to N.H. No.1 with longest distance of approach road of Route C	From N.H. No.1 to N.H. No.1 with same route with C-3 on the Vinh Long side, with C-2 on the Can Tho side
	Approach Road Length	Vinh Long	4,900	4,900	2,000	2,000	3,280	3,280	5,580	5,580
		Can Tho	3,000	2,500	1,970	894	4,110	6,260	7,260	6,260
	Bridge Length (m)		2,600	2,600	2,410	2,410	2,660	2,660	2,660	2,660
	Total Length (m)		10,500	7,500	6,380	5,304	10,050	12,200	15,500	14,500
	Bridge Type of Main span		- Hybrid Cable Stayed - Steel Cable Stayed - Suspension Bridge		- Hybrid Cable Stayed - Steel Cable Stayed - Suspension Bridge		- Hybrid Cable Stayed - Steel Cable Stayed - Suspension Bridge			
Evaluation	a) Hydrological Aspect (River Flow)		- Due to simuosity, shift to out side and deep left side - Erosion of river bank		- Comparatively stable but deep left side - Occurrence of whirlpool		- Comparatively shallow but high velocity - Possible inundation on the sand bar			
	b) Degree of Function of Route		- Advantage to the north direction	- Long distance diversion from / to N.H.. No.1	- Disadvantage to pass Can Tho city area	- Advantage to access to the Can Tho city area	- Advantage to access to the Can Tho city area	- Advantage without construction of bridge	- Advantage for connecting N.H. No.1 with better alignment	- Both advantage of C-2 and C-3
	c) Traffic Congestion of city Area		- Traffic congestion due to passing city area of Can Tho along N.H. No.91	- Traffic congestion to the central part of Can Tho city	- Very traffic congestion in Can Tho city	- Very traffic congestion in Can Tho city	- Comparatively traffic congestion in Can Tho city	- Less traffic congestion of the city area	- Less traffic congestion of the city area	- Less traffic congestion of the city area
	d) Consistency to Future Plans		- Fair	- Good	- Good	- Good	- Good	- Fair	- Fair	- Fair
	e) Impact on Ecology		- Serious	- Serious	- Comparatively less	- Comparatively less	- Comparatively less	- Comparatively less	- Serious	- Comparatively less
	f) Land Acquisition and Compensation		- Comparatively less	- Serious	- Serious	- Serious	- Comparatively less	- Comparatively less	- Serious	- Comparatively less
	g) Bridge Engineering Aspect		- Longer center span length for main bridge (more than 600m) due to river condition		- Longer center span length for main bridge (more than 600m) due to river condition		- Comparatively shorter span length for main bridge (approximately 500m)			
	h) Preliminary EIRR		9.7%	9.8%	9.3%	9.4%	10.5%	10.4%	9.8%	9.9%
	i) Overall Evaluation		-	-	-	-				To be recommended

EIRR: Economic Internal Rate of Return

Mark by Rating for Evaluation	
-	Excellent
○	Good
△	Fair
×	Bad

	A-1	A-2	B-1	B-2	C-1	C-2	C-3	C-2/3
a) Hydrological	×	×	×	×	○	○	○	○
b) Degree of function	△	×	×	○	○	○	○	○
c) Traffic congestion	×	△	×	×	△	○	○	○
d) Future plan	△	○	○	○	○	△	△	○
e) Impact to ecology	×	×	△	△	△	△	×	△
f) Land acquisition	△	×	×	×	△	△	×	△
g) Bridge engineering	△	△	△	△	○	○	○	○
h) Preliminary EIRR	△	△	△	△	○	○	○	○
Evaluation	△	×	×	△	○	○	○	

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CHAPTER 12

PLANNING CONDITIONS FOR THE BRIDGES OF ROUTE C

CHAPTER 12 PLANNING CONDITIONS FOR THE BRIDGES OF ROUTE C

12.1 Navigational Clearance

12.1.1 Review of the Existing Data and Previous Surveys

The review of the data and studies on the navigational clearance required for the Can Tho Bridge was based on the following existing data and previous surveys, especially, those which were studied during the period of the feasibility study and the detailed design of the My Thuan Bridge.

- International shipping survey
- Study for improvement of the access channel to the Hau (Bassac) River
- Hydraulic and morphological conditions at the Vam Nao Pass
- Existing utilities such as electricity lines and telephone cables
- An update of important subjects of the Mekong Navigation Strategy Review
- Study of the international aspects of the My Thuan Bridge
- Existing and planned ports along the Hau (Bassac) and Mekong rivers
- The decision on the navigational clearance of the My Thuan Bridge

12.1.2 Navigable Condition of the Hau (Bassac) River (refer to the Annexure 7.1)

Based on the review of the existing data, the previous surveys and the information obtained in this study, the findings in relation to the navigable conditions on the Hau (Bassac) River are summarized as follows (see Fig. 12.1 and 12.2):

- According to the Attribute Analysis of Shipping Transits to Phnom Penh based on the International Shipping Survey from 1/5/94 to 30/9/94, the Dead Weight tonnage (DWT) of the ships using the Hau (Bassac) is commonly recorded at less than 2,000 DWT. The maximum-recorded (DWT) is approx. 11,335 with an air draught of approx. 45 m (38 m by Fig. 12.2). However, the vessel attribute frequency analysis shows that the 95 percentile weight on travel to Phnom Penh is 3,246

DWT on the Hau (Bassac) River. The air draught for this percentile can be calculated at 25.5 m.

- The study under Belgian assistance that covers the dredging work at the mouth/estuary of the Hau (Bassac) River in relation to vessel sizes such as 5,000, 7,000 and 10,000 DWT is from the economic and technical viewpoints. It is assumed that due to the highly complex hydrodynamic processes and sedimentation behavior, the appropriate feasible vessel size to pass the entrance/estuary of the Hau (Bassac) River will be a maximum of 7,000 DWT.

- According to the hydrographic data from 1992 and the planform data for the period of 35 years (since 1958) for the Vam Nao Pass (collected and analyzed during the feasibility study stage of the My Thuan Bridge), the character of the Vam Nao Pass is that foreseeable future major changes will not occur and the river is in a state of dynamic stability and minor variations only are to be expected. The existing morphological condition in relation to the navigable condition of water depth and ship sizes are that the Cho Moi channel has reduced in size has become more shallow at the channel entrance and is therefore closed to large ships. For the Vam Nao Pass (the western side of Cu Lao Toy), there is a generally deep and navigable channel which allows for ship sizes of 3,000 to 5,000 DWT, however, at the northern entrance, shoaling occurs with a water depth (surveyed in 1994) of approx. 6 m below MSL (approx. 3,000 DWT draft). It is generally expected that 3,000 to 5,000 DWT vessels can pass without dredging of the channel.

- The electricity lines crossing the Hau (Bassac) River are in two locations upstream of the Can Tho ferry point. One with 40 m air clearance is immediately upstream of the ferry point and another with 30 m air clearance is situated at Long Xuen. This has resulted in limiting passing vessels to approx. 5,000 DWT in size, unless there are improvements made.

- The shipping survey analysis based on the data collected and records available for the period from 1990 to September 1994 (including the International Shipping Survey), indicate that for both the Tien (Mekong) and the Hau (Bassac) River ships are small with 65 - 80% less than 1,500 DWT, and others occasionally recorded up to approx. 7,500 DWT. The larger ships normally use the Hau (Bassac) River by reason of its superior navigable condition. About 20% of vessels have an air draft exceeding 25 m, 50% exceeding 20 m and no vessels exceeding 31 m were recorded. In the case of 5% probability of exceedance, the corresponding DWT and air draft were 3,246 tonnes and 25.5 m to 31 m, respectively.

Consequently, the larger ships/vessels using the Hau (Bassac) River are from less than 2,000 DWT to approximately 11,335 DWT with an air draft of approximately 38 m. From Fig. 12.2, the case of the 95% percentile is 3,246 DWT with an air draft of 25.5 m. If without any improvement in the dredging work, the vessel size may be restricted up to 7,000 DWT at the mouth/estuary of the Hau River. The MTC Nominal Ships are listed upto 10,000 DWT with an air clearance of 37.5 m maximum. Fig. 12.1 shows the navigable condition and port capacity of/along the Mekong streams based on the existing shipping data, dredging at the estuary of the Hau River and the record of arrival ships.

The maximum vessel capacities for the existing and planned ports along the Hau (Bassac) and Mekong Rivers are 6,000 DWT for Phnom Penh Port, 10,000 DWT for both EPZ port and the existing Can Tho Port, and 20,000 to 30,000 DWT for the future Can Tho Port (which will be constructed on the downstream side of the Can Tho Bridge).

In any case, the navigational clearance for the Can Tho Bridge must be able to accommodate the conditions of the freedom of navigation as described in the principles of cooperation of the Agreement of the Mekong River Basin, 5 April 1995, Mekong River Commission, and the technical and economical feasibility for future dredging works.

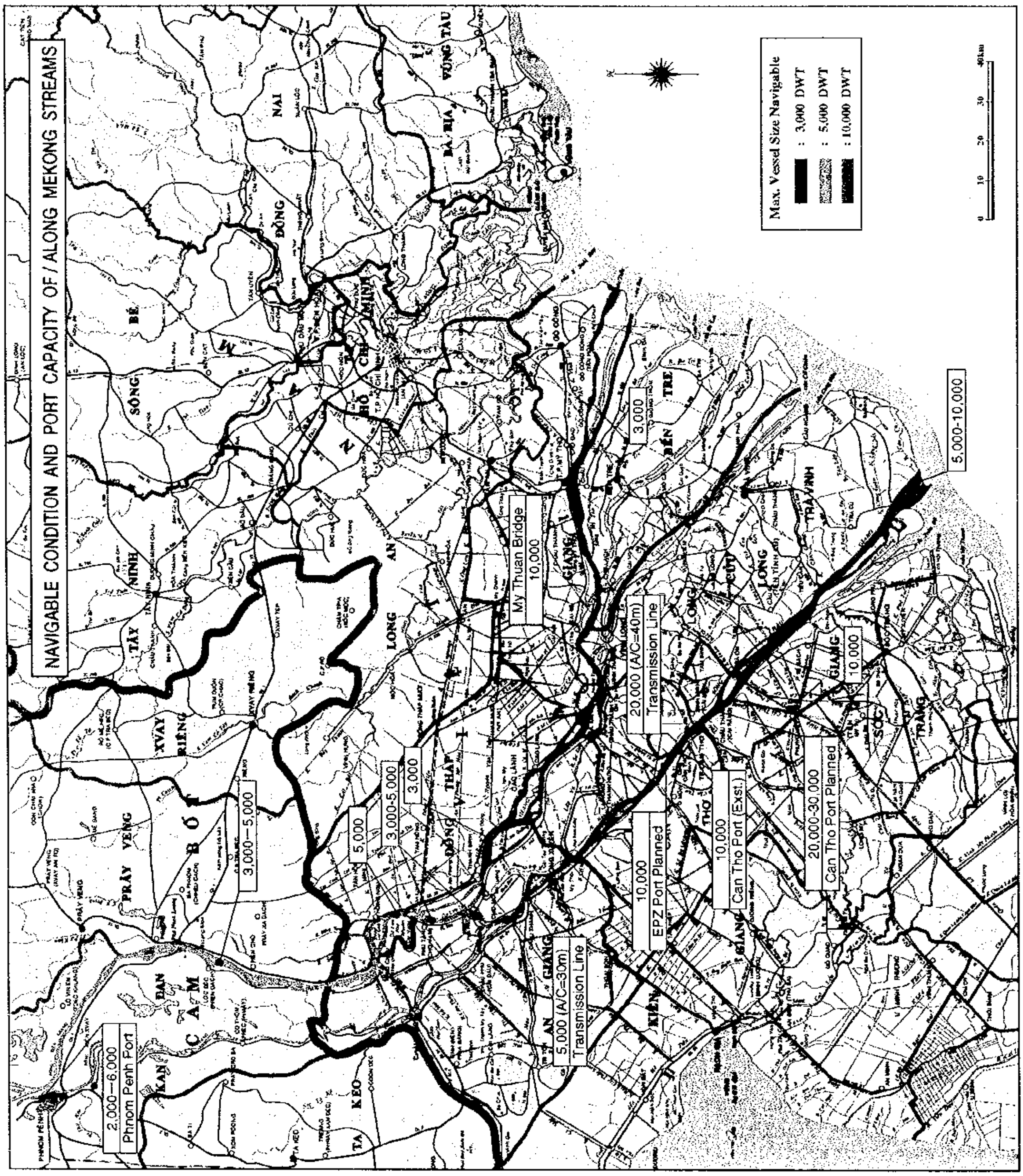

Fig.12.1 Navigable Condition and Port Capacity of /along Mekong Streams

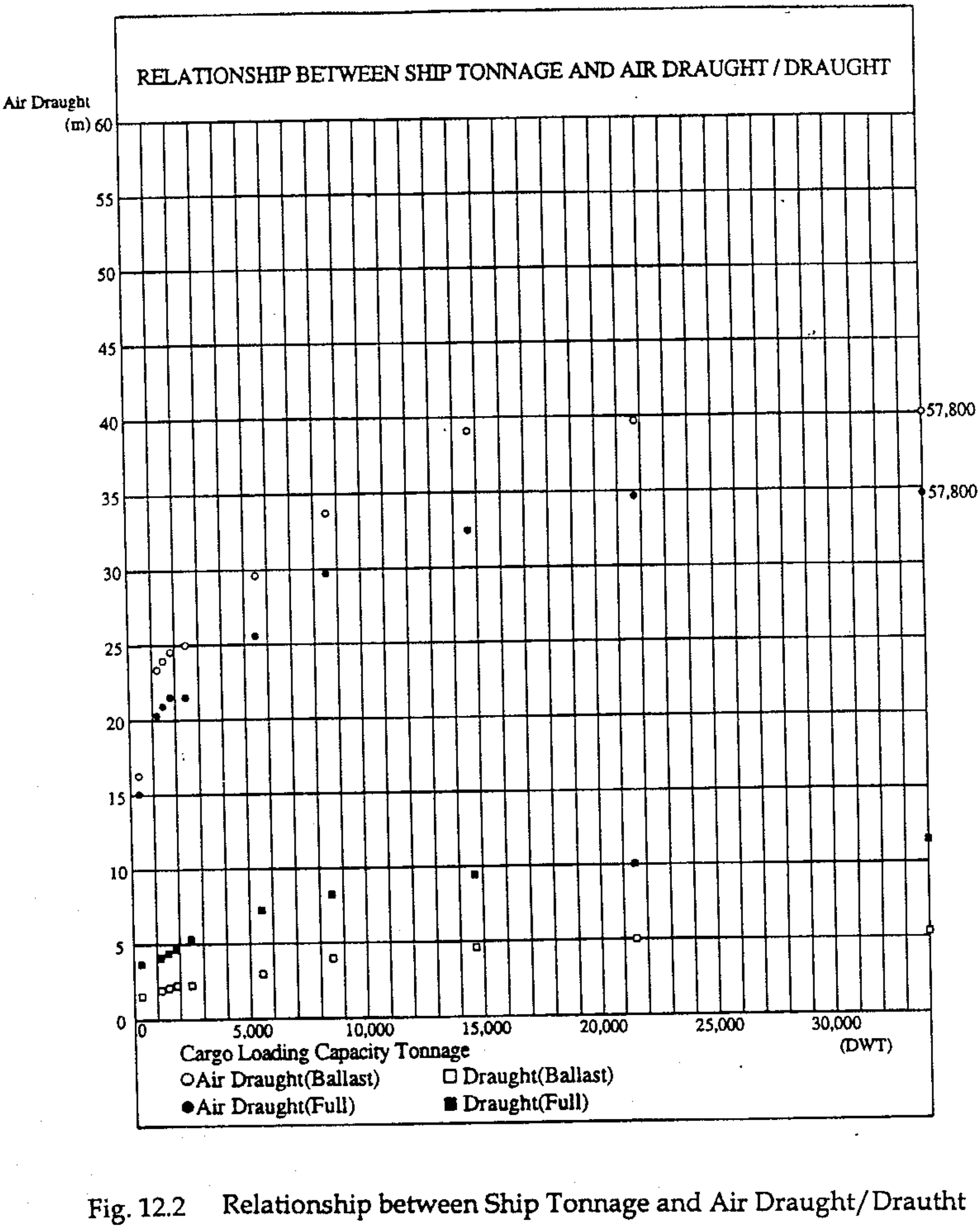

Fig. 12.2 Relationship between Ship Tonnage and Air Draught/Drautht

12.1.3 Navigational Conditions for the Bridge Construction

(1) Vertical Navigational Clearance

The level of the girder soffit can be determined from the following navigational and hydrological conditions.

River Navigational Clearance corresponding vessel size are:

Vessel Size (DWT)	Ballast Air Drought (in meter)
3,000	25
5,000	30
7,000	32
10,000	37 (37.5 m for My Thuan)
15,000	39
20,000	40

Source: Honshu-Shikoku Bridge Project, Japan based on International ship data

Finally, the vertical navigational clearance was required as 39m, and the technical assessment and the decision process are described in the next section 12.1.4.

(2) Horizontal Navigational Clearance

The total opening under the bridge shall be for two maneuvering lanes and not less than 168 m to 200 m. Furthermore, the width for the protection of the bridge piers shall be considered in addition to these figures. It was finally recommended for the total opening under the bridge that 200m plus two times 50 m for protecting the piers was considered and a total of 300 meters was considered reasonable.

(3) Navigational Clearance for Canals and Substream

The required navigation (vertical) clearances on the major canals and river branches (tributaries) greatly affect the proposed height (embankment height) of the approach roads. The requirements for these navigational clearances, therefore, are crucial. The ideas below are the figures for the preliminary design (subject to confirmation with the Inland Waterway Authorities), made by referring to the Technical Classification of Inland Waterways (TCVN-5664-1992).

Table 12.1 Navigational Clearance for Canals and Substream

Side	Substream	Type of Stream	Classification	Required Clearance Vert.(m)	Horiz.(m)
- Vinh Long	(a)	Canal	V	6	25
	(b)	Canal	VI	2.5	10
	(c)	River	V	6	40
- Can Tho	river branch*	River	II	9	60
	(d)	River	VI	2.5	15
	(e)	River	V	3.5	25

Note: All vertical clearance figures are considered above the water level of 5% frequency.

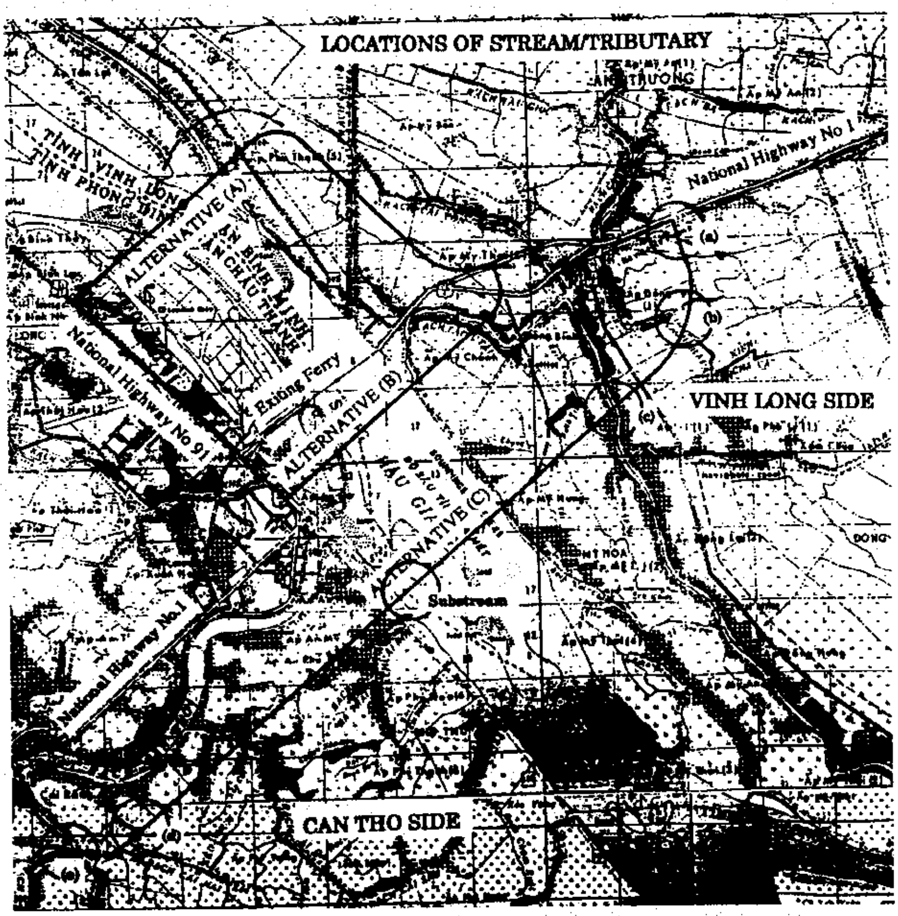

Fig. 12.3 Locations of Stream/Tributary

12.1.4 Technical Assessment and Decision Process for Navigational Clearance

Based on discussions during the meeting, held on 18th January 1998 in Hanoi (Steering Committee Meeting for the Interim Report), the assessment on the cases of vertical clearance of 37.5 m (for 10,000 DWT vessel) and 41.0 m (for 20,000 DWT vessel) for the mainstream of the Hau River as well as confirmation with the Inland Waterway Authority (IWA) on the required navigational clearances for the canals and the substream (tributary) crossing for Route C were requested.

Based on the detailed studies of the Navigational Conditions of the Hau River which are included in the Annexure, the assessment on both cases of the vertical clearance is summarized as follows:

- According to the Attributed Analysis, frequency analysis and declaration of ship arrivals to the existing Can Tho Port, the maximum vessel size reaching Can Tho Port is 20,330 DWT. Shipping is restricted by navigation conditions at the estuary of the Hau River to less than a 10,000 DWT vessel, as it requires a draft depth (full loaded) of 8.0 m. However, the water depth at the estuary is only 4.0 m below Low Level Water (LLW) and at high tide 7.0 m (+3 m LLW).

- The section of the Hau River from the estuary to Vam Nao Pass is able to allow the safe passage of a vessel size of approximately 10,000 DWT except at An Giang where a transmission line restricts the vertical clearance to only 30 m (equivalent to a 5,000 DWT vessel).

- At the Vam Nao Pass, the vessel size is limited to 3,000 DWT for safe passage due to a shallow channel portion, and dredging of approximately 1,680,000 cu. m of material would be required to allow the passage of 10,000 DWT vessels.

- The maximum vessel capacity for (a) the Phnom Penh port and (b) the existing and the future Can Tho Port (to be located on the downstream side of the Can Tho Bridge) is 6,000 DWT to 10,000 DWT and 20,000 to 30,000 DWT, respectively.

- Before vessels of 10,000 DWT and 20,000 DWT are allowed to travel under the Can Tho Bridge, the riverbed of the Hau River and the seabed at the estuary would have to be dredged. From the Dinh An

Estuary to the existing Can Tho Port, the dredging volumes based on the hydrographic atlas 1990 - 1992 and the Service of Maritime Safety 1994 chart, are calculated as below:

Table 12.2 Dredging Volume of Individual Case

Description	Dredging Volume (cu. m)	
	10,000 DWT	20,000 DWT
a) Can Tho ~ Dinh An Estuary (for LLW datum)	26,274,842	54,970,382
b) Can Tho ~ Dinh An Estuary (for +2 m LLW at the estuary)	14,634,140	38,721,902
c) Can Tho ~ Dinh An Estuary (for +3 m LLW at the estuary)	10,366,576	31,943,912

Note: For reference, actual dredged records at the estuary were 200,000 cu. per each year in 1990 and 1997.

Therefore, the dredged volume to maintain navigable conditions for 20,000 DWT vessels at the existing Can Tho Port is two or three times that for a 10,000 DWT vessel, and hence extremely costly.

As a result, the navigational (vertical) clearance to be considered for the Feasibility Study on the Can Tho Bridge Construction should be a minimum similar to the My Thuan Bridge of 37.5 m (corresponding to a vessel size of 10,000 DWT) and a maximum of 41.0 m (for 20,000 DWT vessels) for the following reasons:

a) The navigable condition at the estuary of the Hau River is limited to vessel sizes of 10,000 DWT at the high tide level and also subject to the loaded conditions of such vessels.

b) Up to the Vam Nao Pass from the Dinh An estuary, the navigable size of vessel is in general, 10,000 DWT except for the restriction by the overhead electricity line, where the limit is reduced to 5,000 DWT vessel capacity.

c) Sailing to Phnom Penh as an international destination, the channel at Vam Nao Pass (present condition only allows 3,000 DWT vessels) necessitates the dredging of 1,680,000 cu·m of material to accommodate 10,000 DWT vessels.

d) The desirable vessel capacity for the existing Can Tho Port is 10,000 DWT. However, this vessel size can only reach the port in a half-loaded condition when using the Dinh An estuary.

e) The future Can Tho Port, which will be able to accommodate 20,000 to 30,000 DWT vessels, will be located on the downstream side of the Can Tho Bridge and will not be required to pass under the Bridge.

f) The sailing of 20,000 DWT vessels from the Dinh An estuary to the existing Can Tho Port necessitates dredging of two to three times the volume required for the 10,000 DWT vessel case, and will be more costly.

g) The vessel capacity at the existing Phnom Penh Port is restricted to a maximum of 6,000 DWT and with this accepted capacity, the port was designed and construction completed in December 1996.

Referring to the navigational conditions of the Hau River, the conclusion of assessment, and as a result of the Steering Committee Meeting held on 27th March 1998, the following navigational and hydrological conditions were required for the Can Tho Bridge construction.

- The space for the existing and proposed navigation should not be disturbed by the new bridge structures. The space for navigation should be the combined area of 39.0 m (vertical) x 110 m (horizontal) and 30 m x 300 m to secure the navigable space.

- The navigational clearance should be confirmed with Cambodia and accepted by the Mekong River Commission of Vietnam and its Secretariat Offices.

- The basic water level for the vertical navigation clearance shall be high water level of 5% frequency, i.e. a 20 years return period.

- The required navigational clearance for the river branch located on the right river bank of the Hau River shall be followed in accordance with the Technical Classification of Inland Waterway (TCVN-5664-1992).

On 8th April 1998, the Mekong River Commission of Vietnam (MRCV) transmitted a document (No.172 CV/UBMC) about the navigational clearance of the Can Tho Bridge as the reply for the following two documents:

- No.4041/PMUMT of 31st December 1997 of the Ministry of Transport
- No.145/PMUMT of 9th February 1998 of PMU My Thuan

This transmitted document was prepared after discussion with the Secretariat of the Mekong River International Commission (Bangkok, Thailand). The MRCV commented that the navigational clearance of 37.5 m was found to be reasonable from the technical and economic point of view and suitable to the waterway transport requirements on the Hau river in the present and future stages. With accepting this comment, the Steering Committee regarded that 39m of vertical navigational clearance was satisfied.

The above navigational conditions were concluded as the final design condition for the Feasibility Study, in the Steering Committee Meeting held on 9th July 1998 for the Draft Final Report in Hanoi.

12.2 Possible Bearing Stratum (see Fig. 12.4)

The soil classification of the upper and middle parts are clay. The lower part is sand. The upper part is a very soft clay layer with an N-value of <5 (20 m to 40 m deep).

The middle part with an N-value of 5 to 35 (40 m to 70 m), is clayey soil and a comparatively good soil layer. The lower part with N-value of more than 50 (70 m to 100 m), is considered as the bearing stratum.

Consequently, the bearing stratum required to support the bridge structure adequately is to be the soil layer classified as S1, which exists at a depth of 70 m to 95 m.

Detail of analysis is described in Chapter 6.

12.3 Hydrological and Hydraulic Conditions

The hydrological and hydraulic conditions summarized below were considered for planning the bridge and especially for determining the bridge opening and location of the central span of the main bridge. Detail of analysis is described in Chapter 6.

(1) Design Discharge
(Referred to Flood Water Conditions at Can Tho Station)

Flood Water	100 year	20 year	Max Act-Record
High Water Level (cm)	212.40	208.60	209.00 (1989)
Design Discharge (m^3/sec)	30,999	28,204	27,900 (1991)

Note: 100 year for Design Flood Level
20 year for the critical water level for ship navigation

(2) Design Flood Water Level of Route C

Flood Water Level:

100 year (for design flood level)	195.46(cm)
20 year (for the critical water level for ship navigation)	191.66(cm)
Referred to Max Act-Record	192.06(cm)

For all figures, the basic level is Mean Sea Water Level (MSL).

(3) Hydrological and Hydraulic Conditions

The hydrological and hydraulic conditions of each route for bridge planning, especially for the main bridge are presented in following table.

Table 12.3 Hydrological and Hydraulic conditions for Route C

	Description
a) Riverbed (water depth)	- Comparatively shallow (15 m) - Wider riverbed with depth (15 m)
b) Water Flow Velocity	- Faster velocity (2.033 m/s) due to the main channel discharge
c) Planform Change	- Sandbar occurrence from 1973 to 1993, with possible inundation in a large flood of the sandbar
d) Special Observation	- Low level of sand bar, and 30% of the total sandbar area will be inundated in a large flood
e) Center Span Location of Main Bridge	- Comparatively left side and subject to further hydro-dynamic study
f) Bridge Opening of Main Bridge	- Not less than 500 m opening

12.4 Local Scouring Depth around Piers

The local scouring around bridge piers depends on discharge, riverbed slope, direction of stream, riverbed material, alignment of piers and their shape and size. Local scouring around piers is the result of vortex systems which develop as the river stream is deflected around the pier and forms a horseshoe-shaped hole. The scouring hole increases in size until an equilibrium depth is reached. The calculation results of the local scouring depth around the piers in case of the Can Tho Bridge are summarized for design purposes below:

	Formula Used	Scouring Depth
1)	Indian Road congress	31.95 m say 32 m
2)	Laursen's	21.9 m
3)	Japan Railway's	21.8 m

Note: Flood discharge: 31,000 m^3/sec
Water Depth: 15 m
Riverbed Material: Fine Sand
Breadth of Water Surface: 900 m
Breadth of Foundation: 15 m

A scouring depth of 32 m was considered for Route C because of the soft and fine size soil deposition of the riverbed. (refer to Annexure 7.3).

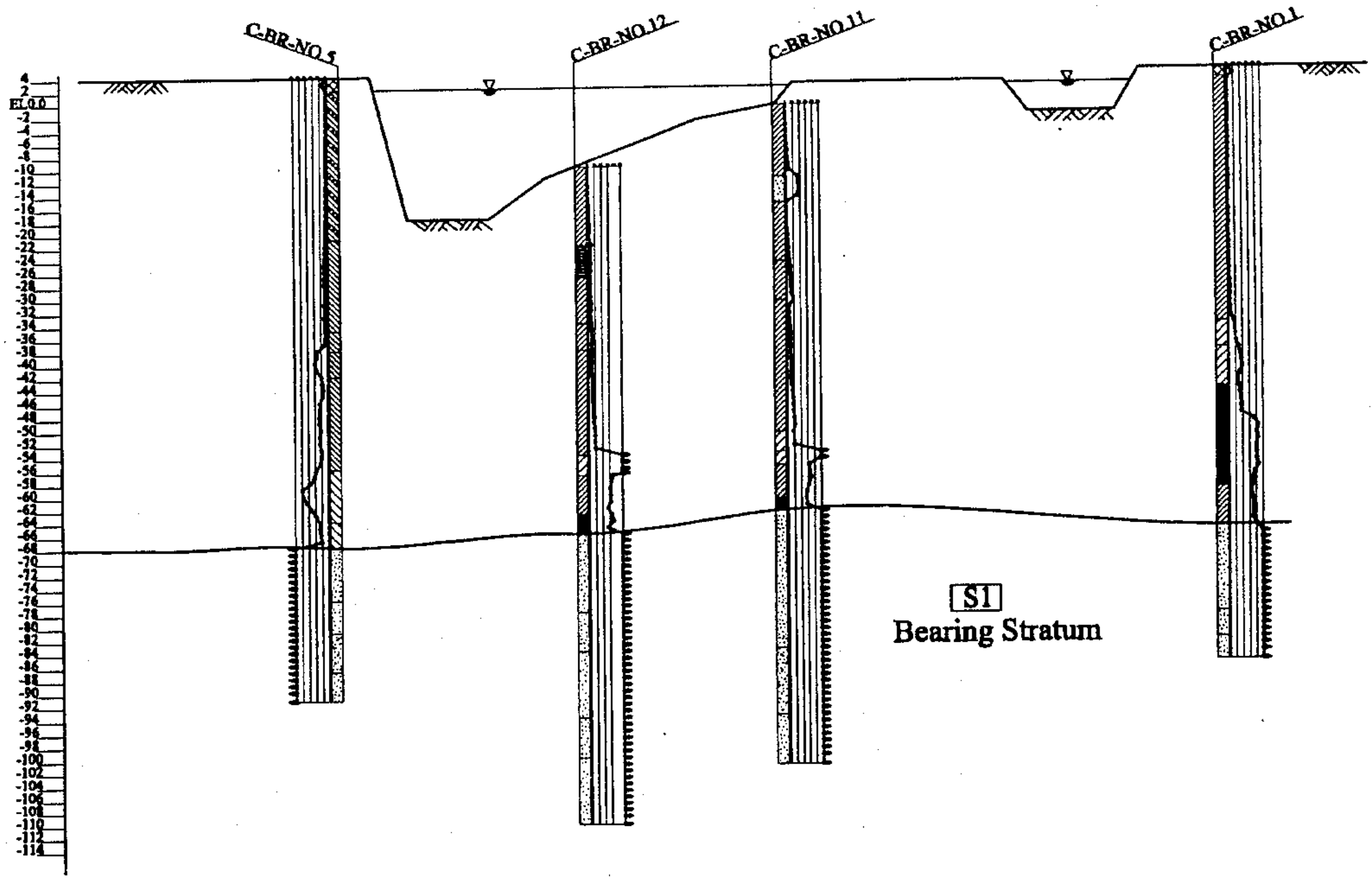

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Fig. 12.4 Bearing Stratum at C-Route

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CHAPTER 13

PRELIMINARY DESIGN

CHAPTER 13 PRELIMINARY DESIGN

13.1 Technical Assessment and Optimization for Design

13.1.1 Optimum Span Length and Bridge Type for the Main Bridge

The required span length of the main bridge can be determined by the required horizontal navigational clearance and the magnitude of structure required from assessing the conditions that the bridge structures (abutments and piers) are stable and safe from hydrodynamic issues such as local scouring of the riverbed around the foundations. Since the required span length is governed by hydrodynamic issues rather than the horizontal navigational clearance (300 m), the requirements from the hydrodynamic and economic viewpoints are discussed as below.

(1) Hydrodynamic Issues and Span Length

- The right riverbank 7 km upstream of Route C faces the river stream and erosion has occurred at a rate of 3 m/year from 1972 to 1993 (according to the satellite image analysis).
- The left riverbank 4.5 km upstream of Route C also faces the river stream and erosion has also occurred.
- Immediately upstream of Route C, the riverbed has deepened. This riverbed deepening may be related to the planform (riverbank) changes at the upstream river sections previously described.
- At the river section where Route B crosses, the left side (outside of the river flow) riverbed has increased in depth up to 28 m from 1961 to 1997.
- At the river section where Route C crosses, the riverbed has become deeper year by year. This was recognized from the changes in the maximum river water depths of 13 m, 17 m, and 19 m in the years 1963, 1991, and 1997, respectively.
- In addition, at the river section of Route C, the river current velocity was substantial (approx. 2.0 m/sec, Oct. 1997) and this velocity was observed from the left riverside to the middle of the

river. This velocity effect will also considerably affect the hydrodynamic issues.

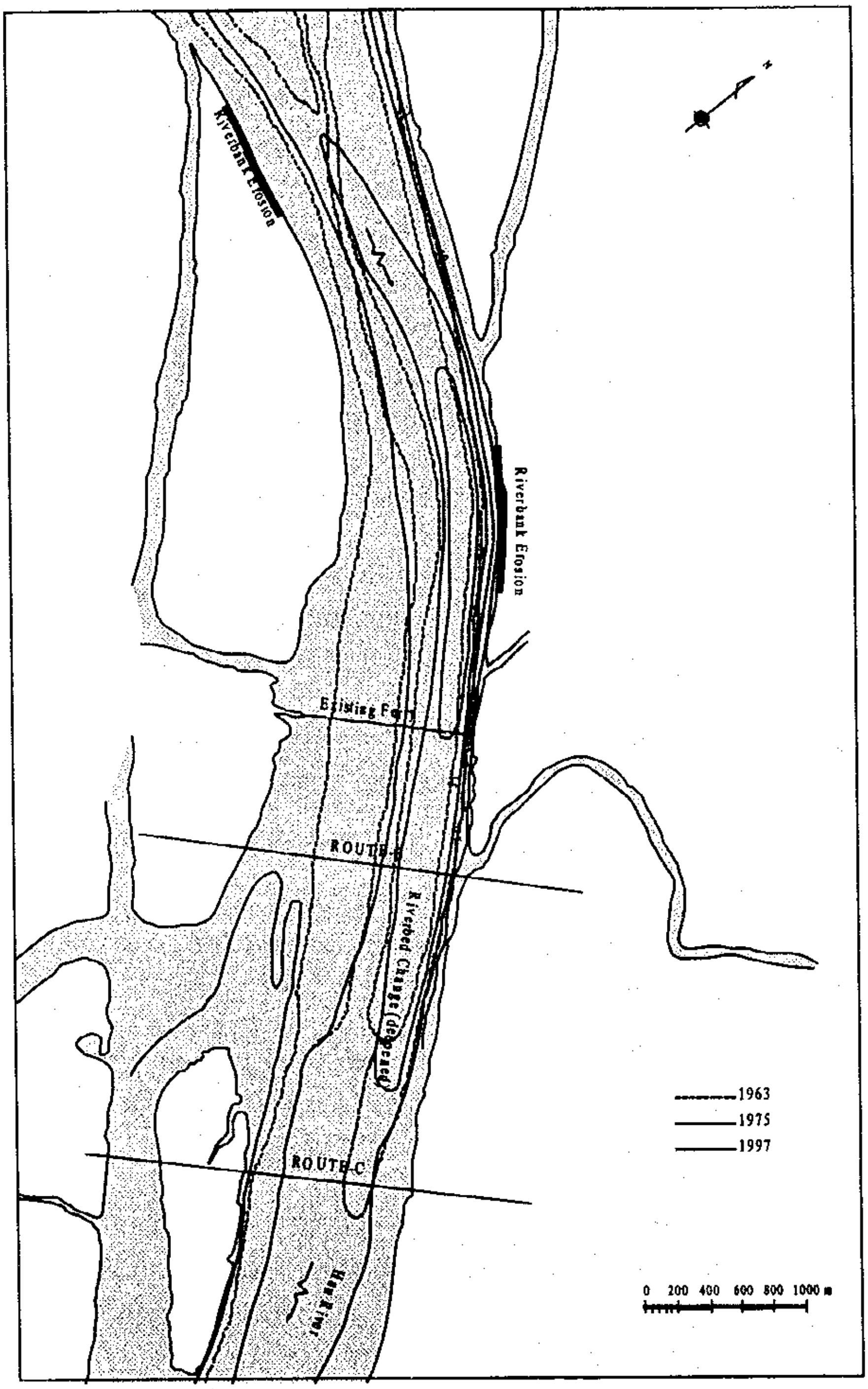


Fig. 13.1. Water Depth of the Hau River

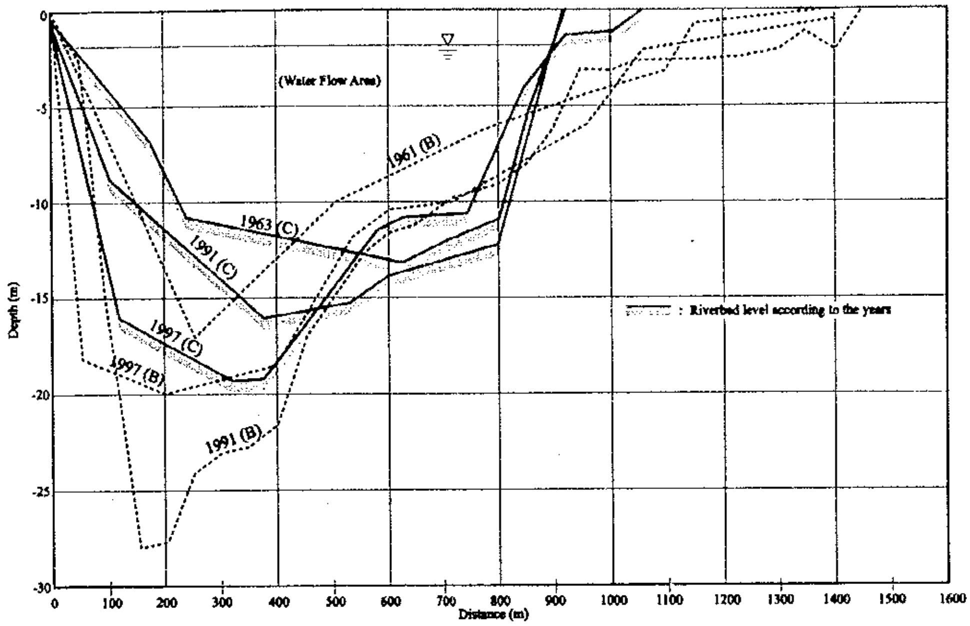


Fig. 13.2 Riverbed Changes at the Section Crossing Route-C

- Thus, the required span length for the mainstream should be greater than 500 m so that the main towers of the bridge can straddle the channel where the riverbed is changing in depth. Also the portion becoming deeper is developing downstream and may be affected by flood waters and the planform changes at the upstream sections. This riverbed change (hydrodynamic behavior) may cause the bridge to become unsafe, if the span length is less than 500 m.

(2) Optimum Span Length and Bridge Type

- The difference in cost between a Hybrid Cable-Stayed bridge of span length 500 m and a PC Cable-Stayed bridge is small (1% ~ 4%).

- The 350 m span PC Cable-Stayed bridge is not recommended as the towers of bridge will be situated in the deep water area (20 m ~ 25 m deep) and may be involved in the hydrodynamic issues and as a consequence, the bridge structure may itself become unsafe.

- Though a PC Cable-stayed bridge is economical with only a narrow cost difference compared with a Hybrid Cable-Stayed bridge, the 400 m central span length of the PC Cable-Stayed is a maximum from technical reasons, and not able to achieve the required span length 500 m.

The economic study results show small differences but a 500 m span length must be achieved for the bridge type to be adopted. Consequently the Hybrid Cable-stayed bridge type is recommended for the main span.

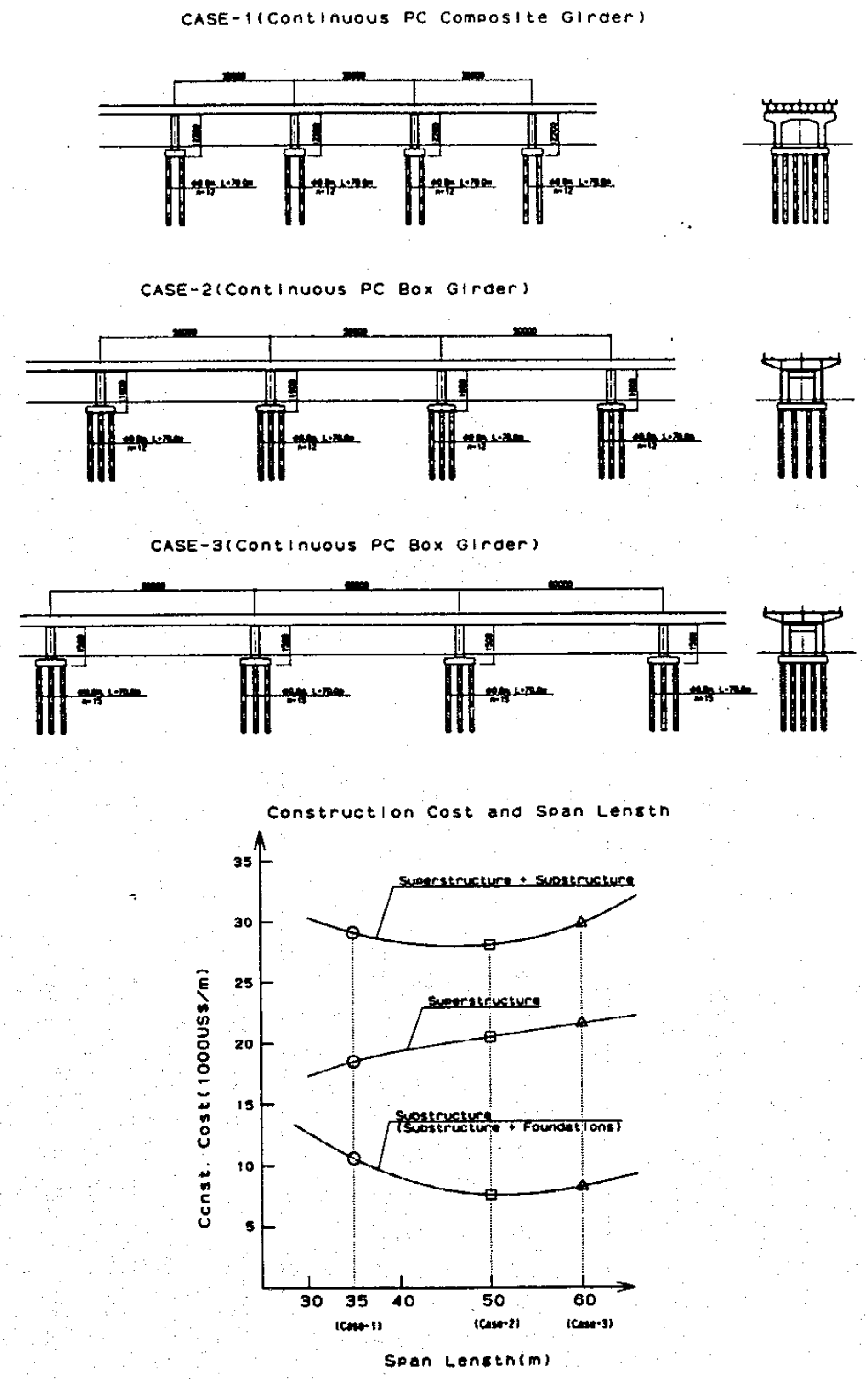
13.1.2 Optimum Bridge Type and Span Length for the Approach Bridge

To determine the appropriate bridge type and the optimum span length for the approach bridge span, the comparison of the three bridge types in conjunction with finding the most economical span length were studied (Fig. 13.3). For this study, a Continuous PC Composite Girder (35 m), a Continuous PC Box Girder (50 m) and a Continuous PC Box Girder (60 m) were compared with each other on the basis of the construction cost of superstructure and substructure including foundations. The Continuous PC Box Girder type is recommended as it is the most viable economic alternative.

13.1.3 Study on the Bridge Type for the River Branch

In order to economize the bridge cost, the types of bridge crossing the river branch of the Hau River on Route C were studied. The costs of the three applicable bridge types, a Hybrid Extra-dosed type with a central span length of 180.0 m, a 120.0 m span PC Extra-dose type and a 70.0 m span PC box girder type were compared. The 9 m vertical clearance and multiple 60 m horizontal clearances (referring to the Technical Classification of Inland Waterways (TCVN-5664-1992)), were considered for this comparison study as well as the deep soft subsoil conditions.

Compared with case-2 of the PC Extra-dosed type, the case-1 Hybrid type can achieve a longer central span using a steel structure but it is costly and the longer span length involves a more concentrated reaction transmitted from the superstructure to the soft ground, which creates an adverse situation. The case-3 PC Box Girder type requires many piers due to the shorter spans involved. However, if compared with the case-2 Extra-dosed bridge, the cost difference is negligible. The case-3 continuous PC Box Girder type has less technical problems compared with case 1 and 2 and is more economically viable (see Fig. 13.4).



THE FEASIBILITY STUDY ON THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Fig. 13.3 The Comparison of the Three Bridge Types to find the Most Economical Span Length
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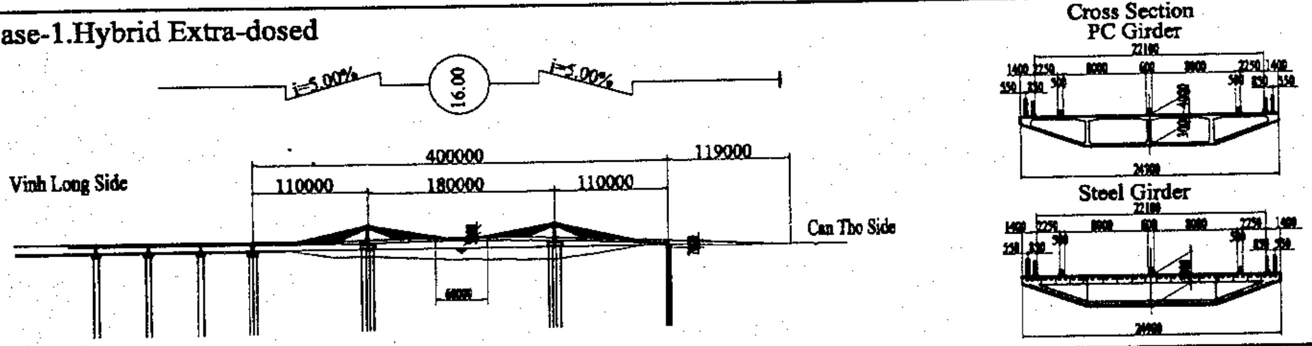
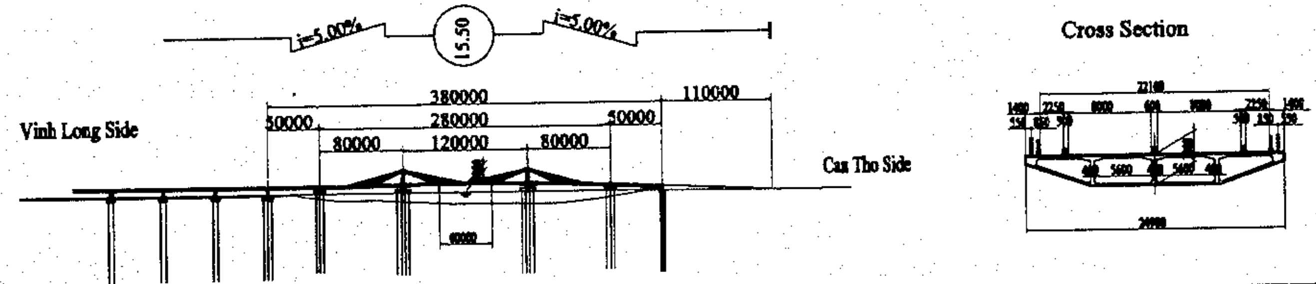
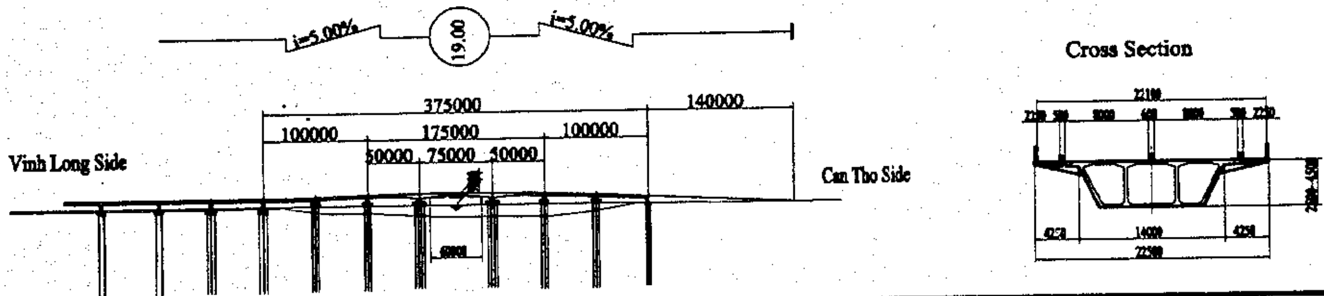
Configuration of Bridge and Gradient of Approach Portions	Cost ,$1000 (Ratio)	Remarks
case-1.Hybrid Extra-dosed 	30,981 (1.187)	
case-2. Extra-dosed 	26,109 (1.000)	
case-3. PC Box Girder 	26,186 (1.003)	to be recommended

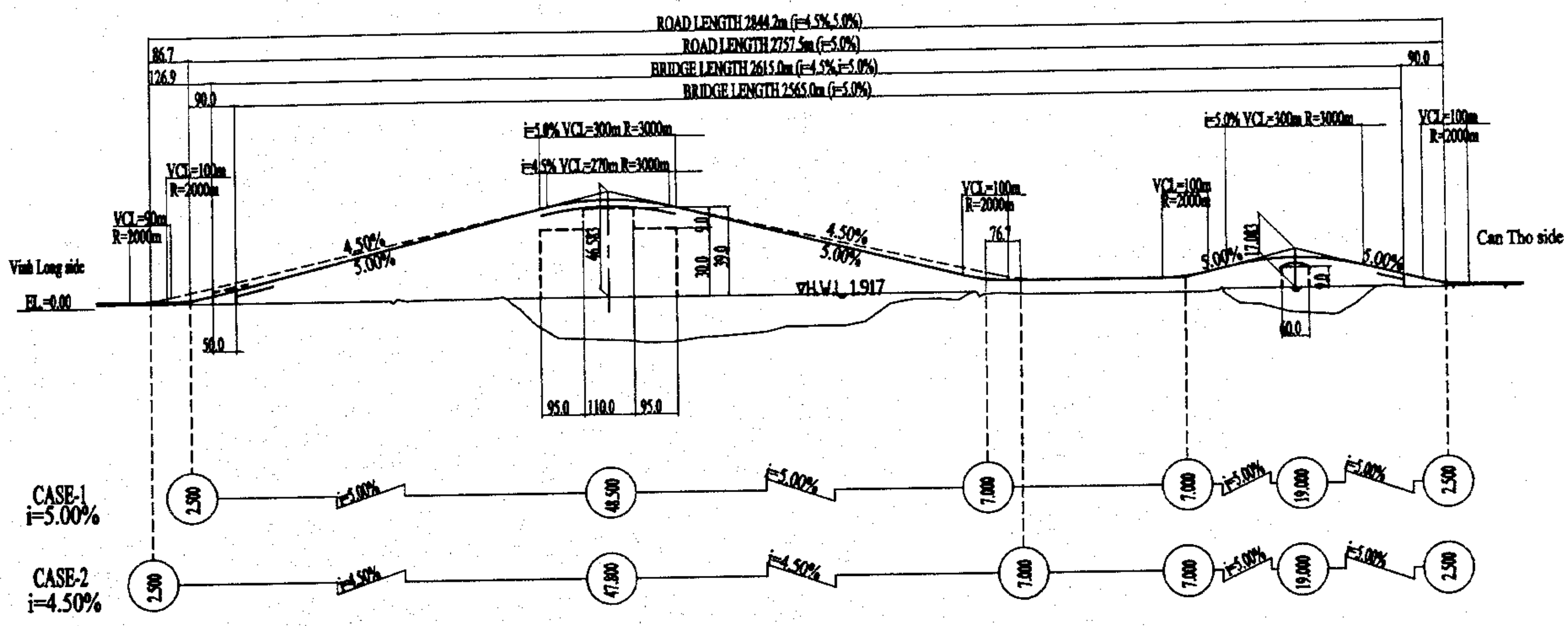
TEH FEASIBILITY STUDY ON THE CAN THO BRIDGE CONSTRUCTION IN SOCIALIST REPUBLIC OF VIET NAM	Fig. 13.4 Study on Bridge Type for the River Branch JAPAN INTERNATIONAL COOPERATION AGENCY

13.1.4 Gradient for Approach Portion

The gradient should be designed to encourage uniform operation throughout the bridge and approach roads. However, vehicle operating characteristics on gradients are different and depend on the type of vehicles. It is also ideal that vehicles can run at the design speed, however, from the economic viewpoint, a lowering of speed for such vehicles as trucks should be accepted to a certain degree. Therefore, it is necessary to consider the gradient in conjunction with the critical slope length of gradient. For the control of the design gradient of the bridge and approach roads, the gradient should not be greater than 6% (which is specified as the maximum gradient at the design speed of 80 km/hr in accordance with the Highway Design Standards (TCVN-4054-85), Viet Nam).

In comparing the gradient between 4.5% and 5%, the 4.5% case means a longer approach bridge by approx. 165.5 m, and thus a more costly bridge (see Fig. 13.5).

The maximum grade to be applied to the vertical alignment design should, however, be less than 5%, while the maximum permissible grade is 6% corresponding to the design speed (80 km/hr.). However, even though a 5% grade can reduce the length of the main bridge by 86.7 m (Vinh Long side) and 76.7 m (Can Tho side) compared with a 4.5% grade, the lesser 4.5% grade is favorable in terms of the critical length of grade that will reduce vehicle speeds to less than the minimum permissible speed (40 km/hr. compared to the design speed of 80 km/hr.), and the traffic capacity of the vehicle lane. In the case of the critical length of climbing slope for the main bridge portion, if a 5% grade was applied to the climbing slope, the traffic speed of vehicles would be reduced to less than 40 km/hr., while a 4.5% grade can maintain a speed of more than 40 km/hr. Furthermore, the traffic capacity of a 5% grade is reduced by 10% compared with that of 4.5%. As a consequence, a maximum grade of 5% referring to the design criteria was adopted throughout the alignment, and a 4.5% grade was adopted as a maximum for the main bridge portion.



Comparison of Const. Cost (million US$)

Study Case	Const. Cost.(Ratio)
CASE-1(i=5.00%)	202.9 (1.000)
CASE-2(i=4.50%)	205.0 (1.010)

Note: 1) Gradient comparison in case of C-Route

2) Difference of length between the cases 4.5% and 5% 86.7m.(Vinh Long side)

3) Difference of length between the cases 4.5% and 5% 76.7m.(Can Tho side)

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Fig. 13.5 Comparison of Gradient for the Approach Portions

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13.1.5 Alignment and Cross Section

At present, there are two standards available for the design of national highways in Viet Nam:

1) Geometric Design Standard, issued in October 1995

2) Highway Design Standard (TCVN 4054-85), translation, issued in July 1990

The typical transverse cross-section (bridge deck width and road width) can be determined based on the Vietnamese Standard after consideration of the traffic demand, traffic system, structural requirements, and cost. The Can Tho Bridge is situated on N.H. No. 1 and close to the My Thuan Bridge (under construction), and the bridge deck and road widths should be similar to that of the My Thuan Bridge. The following transverse cross-sections for the bridge and road (Fig. 13.6) are adopted for the Can Tho Bridge Construction.

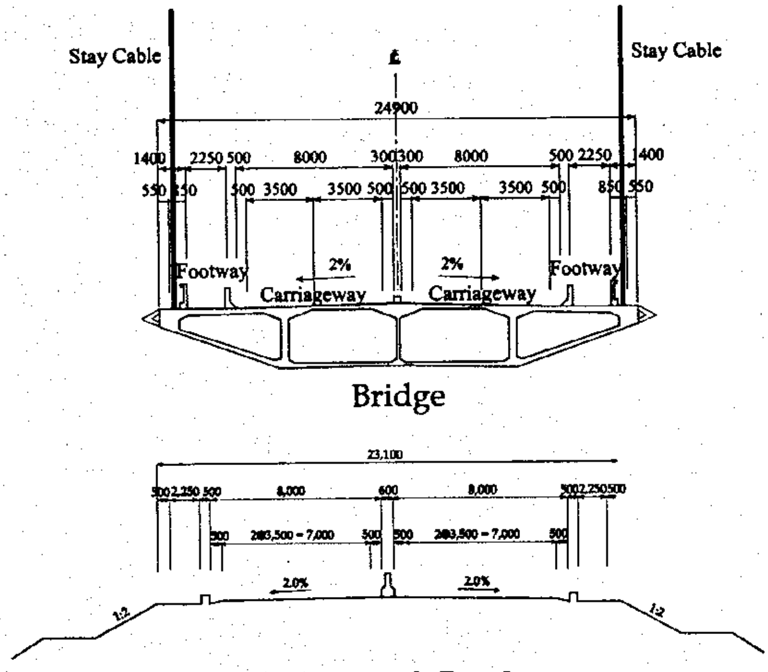


Fig. 13.6 Typical Cross-section

For the bridge cross-section, the possibility of widening from 4-lane to 6-lane in the future was also studied. As for the result of the traffic forecast, pcu/day will be 29,628 in 2010, and 75,262 in 2020. The capacity for a four-lane facility is approximately 60,000pcu/day as a general rule. This comparison study is indicated in Fig. 13.7. Case-B that is same as the cross-section indicated in Fig.13.6 is recommended for the following reasons:

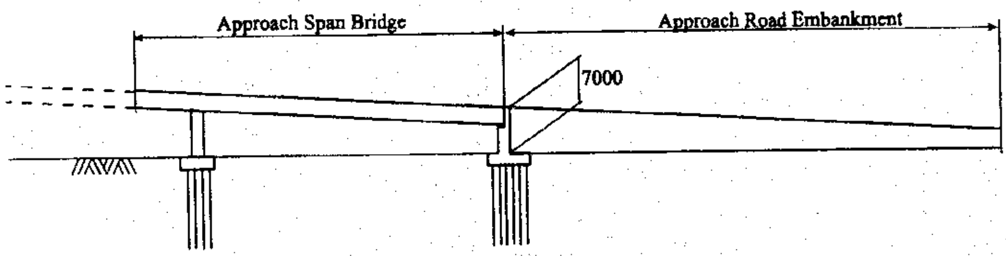
- Case-A is difficult for anchoring the cables in the closed section which is required to be consistent with the central span section against aerodynamic vibration.

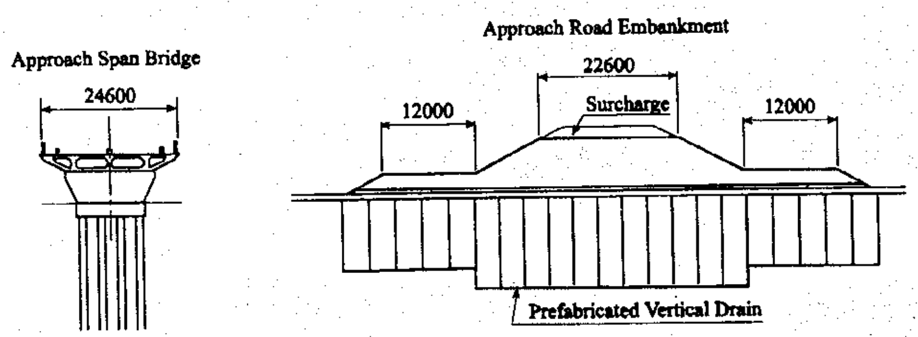
- Case-B can carry four lanes of vehicles and can be widened in future without any technical restrictions.

- Case-C is possible to be widened from 4-lanes to 6-lanes. However if widening is required, the bridge width of 4-lanes should be a wider width of 26.4 m in lieu of 25.1 m for the case-B alternative making the initial construction cost including additional widening of the footway, extremely high. There would also be many technical problems to be resolved for any future widening.

13.1.6 Embankment Height

The limitation of embankment height, i.e. location of the abutment (approach bridge end) or the beginning of the embankment (highest point of approach road embankment), is at the point where the height of approach road embankment is 7.0 m from the ground for economical optimization. The detailed study result is shown in Annexure 8.





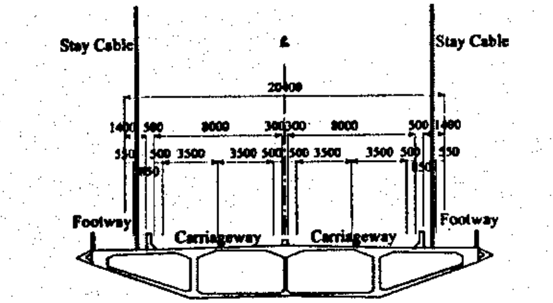
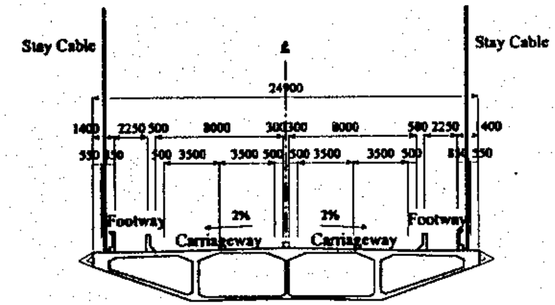
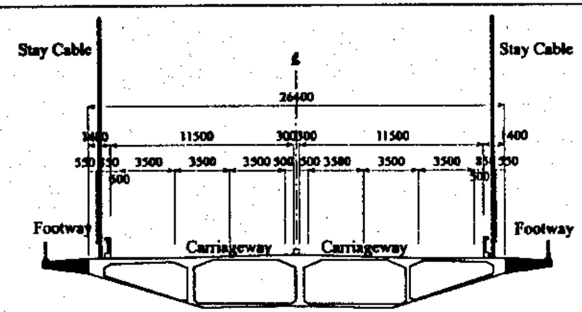
Case	Bridge Deck	Descriptions
A		- Stay cables are anchored on the inside of footways. - This section and arrangement of stay cable is normally applied 300 m ~ 350 m. (PC Cable-Stayed Bridge)
B		- Cable stays are anchored at the edge of the girder. - Easy construction for all required items at the same time. - Footways are provided on both sides, and are inside of the cables. - Capacity of stay cable is less than Case C because of less live load.
C		(In case of widening from 4-lane to 6-lane) - Footways have to be provided additionally; however, any addition will be structurally complicated being installed after completion of the bridge structure. - Capacity of stay cable must be higher than 4 lanes case for carrying bigger live load. It is necessary to provide bigger size stay cables, so their initial cost is higher. - Cross sectional frame stress due to T-load is bigger than Case B, then it is necessary to arrange more transverse prestressing tendons. Initial cost will be higher than Case B. - Width of bridge deck is wider than Case B by 1.3 m. - Additional costs for footways are roughly estimated 5 million USD.

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Table 13.7 Comparison of Bridge Width (4-lane and 6-lane) and Possibility of Widening

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13.1.7 Type of Intersection

To determine the type of intersection, the following technical and economical conditions have been considered (see Fig. 13.8).

(1) At Grade Intersections

An intersection at grade occurs where roads meet or intersect at the same level. The following are the basic types of grade intersections.

a) Unchannelized and Unflared Intersection

Unchannelized intersections are normally referred to as T or Y-shaped and multi-leg depending on the shape and number of legs which make up the intersection. They are normally adequate where minor roads meet and where a major road is joined by a minor road. In urban areas, many intersections remain unchannelized for economic reasons or can be controlled by signals and/or regulatory signs.

b) Flared Intersection

A flared intersection is a simple unchannelized intersection with additional through or auxiliary lanes, such as speed-change or passing lanes. Speed-change lanes allow left or right-turning vehicles to reduce or increase speed when leaving or entering the through road without adversely affecting the speed of through traffic. Passing lanes permit through vehicles to pass on the left side of another vehicle waiting to complete a right turn.

c) Channelized Intersection

A channelized intersection is one where paths of travel for various movements are separated and delineated. Raised traffic islands, raised markers, painted markings and safety bars can be used for channelization. A roundabout is a channelized intersection where traffic moves clockwise, adequately illuminated by street lighting or delineated by painted reflectors, signs etc.

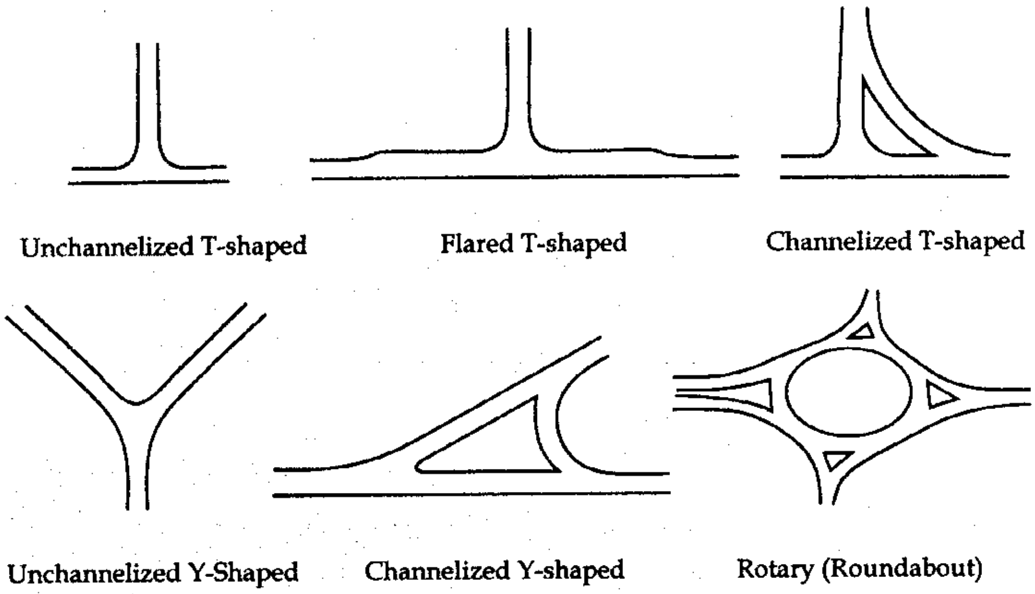

Fig. 13.8 Type of Intersections

(2) Necessity for Grade Separations and Interchanges

An interchange is a useful and an adaptable solution for many intersection problems, but because of the high initial cost, its use to eliminate existing traffic bottlenecks or to correct existing hazardous conditions is limited to those cases where the required expenditure can be justified. The following six items should be considered before reaching a rational decision to provide interchanges.

a) Design Designation

The decision to develop a highway with full control of access between selected terminals becomes the condition for providing highway grade separations or interchanges for all intersecting highways.

b) Elimination of Bottlenecks or Spot Congestion

Insufficient capacity at the intersection of heavily traveled routes results in untolerable congestion at one or all approaches. The inability to provide essential capacity with an at-grade facility provides a necessary situation for an interchange where development and available right-of-way permit.

c) Elimination of Hazard

Some at-grade intersections have a disproportionate rate of serious accidents. Lacking inexpensive methods of eliminating hazards, a highway grade separation or interchange may be warranted.

d) Site Topography

At some sites, grade-separation designs are the only type that can be constructed economically. The topography at the site may be such that any other types of intersection are physically impossible to develop or result in increased costs.

e) Road-user Benefit

The road-user costs due to delays at congested at-grade intersections must increase. Such items as fuel, tires, oil, repairs, time, and accidents that require speed changes, stops, and waiting, generate expenditure well in excess of those for intersections permitting uninterrupted or continuous operation.

f) Traffic Volume

The traffic volume warranted for interchange treatment would be the most tangible of any interchange decision. Although a specific volume of traffic at an intersection cannot be completely rationalized as the authorization for an interchange, it is an important guide, particularly when combined with the traffic distribution pattern and the effect of traffic behavior. Volumes in excess of the capacity of an at-grade intersection would certainly warrant consideration of interchanges treatment.

Thus, the following intersection types can be recommended considering the future traffic demand, site topography, and the road network arrangement in future master plans.

-	At grade intersection	:	At grade intersection (channelized Y-shaped) at the beginning point of Alternative Route C-3 (Vinh Long side)
-	Rotary Intersection (Roundabout)	:	At the intersection point between the approach road and the future road relating to the Can Tho Master Plan
-	At grade intersection	:	At grade intersection (channelized Y-shaped) at the end point of Alternative C (Can Tho side)

13.1.8 Study on the Rampways to the Island (Cu Lao Lat)

Based on the request by the Vietnamese side regarding additional rampways to the island (Cu Lao Lat) where Route C crosses, a study was conducted to examine their technical and economic feasibility. For the following reasons, additional rampways to the island are not recommended.

1) The acceleration lane including taper portion overlaps the side span of the main bridge (cable-stayed system), which may increase the cost and create technical difficulties in the anchor system of the cables.

2) If the rampways are built beside the carriageways of the main bridge, the vehicles merging, on and off into the traffic flow of the main carriageways, would cause frequent traffic accidents at the merging points due to the acceleration and deceleration of vehicles. This is a most adverse consequence which should be avoided.

3) Even if these problems were resolved, and vehicles came onto the island, the damage caused to the terrestrial and aquatic ecosystems would be considerable.

4) It is recommended to study the transport link between the island and the riverbank by river transport.

5) The stairs from the bridge deck to the island and, vice versa, should be considered with studies on the future development plan of the area.

6) With stairs a wider bridge deck would need to be provided for vehicles which stop and park to gain access to the stairs. This would increase the construction cost.

Thus, the rampways for the vehicles are not recommended especially in terms of car accidents and damage to the ecosystem of the island (Fig. 13.9). A study on the accessibility to the island by river transport should be considered. The stair plan option should also be considered in any study on the future regional development plan.

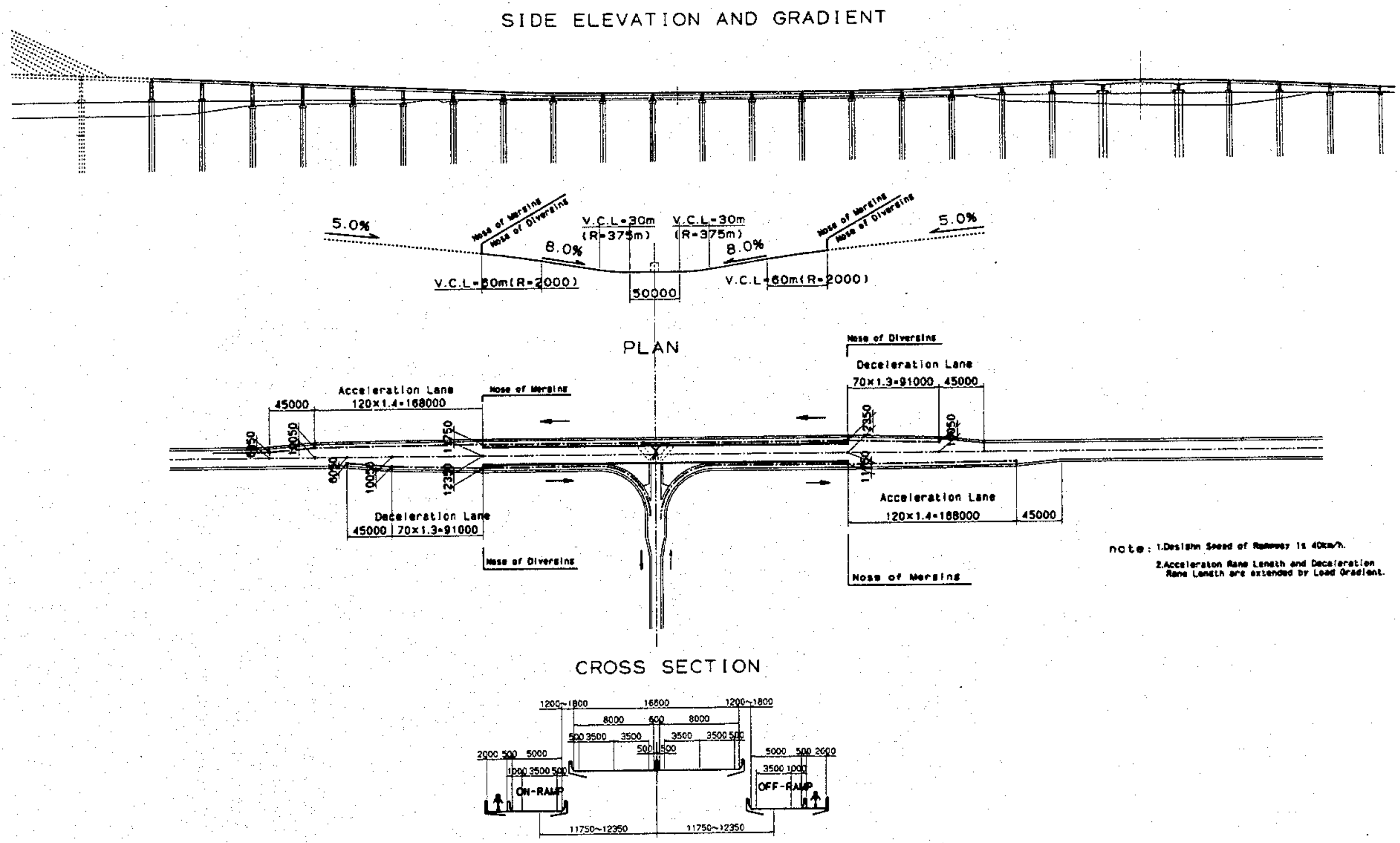

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Fig. 13.9 Study on Rampways to the Island (Cu Lao Lat)

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