6.2 Bridge Design Standards

6.2.1 Clearances

With respect to clearance the Study Team had discussions with the following agencies;

PMU Thang Long

Observatory for Hydrometeology and Environmental Control of the Red River Delta

Civil Aviation General Department of Vietnam (Tong cuc hang khong Vietnam)

Inland Waterway Bureau of Vietnam (Cuc duong song Vietnam)

Dyke Control Department of Ministry of Agriculture and Rural Development (Cuc quan ly de dieu Vietnam)

The Study Team has also reviewed the existing Ministry of Transport requirements as contained in document Vietnam Bridge Design Code 22 TCN 018- 79.

(1) River Navigation

At the commencement of the study the Inland Waterway Bureau of Vietnam confirmed that the navigational clearance for the bridge should be 10 metres above high water level (HWL) and this should be maintained over a length of 80 metres in addition the value of the HWL should be the predicted level of the river for a return period of 20 years. The Study Team has obtained from the Observatory for Hydrometeology and Environmental Control of the Red River Delta the levels of the river at Hanoi since 1945 and their predictions for a flood return period of 20 years is 12.5 metres. The river levels recorded and predicted are given in Appendix A6-2-1. The recorded levels show that the value of 12.5 metres has only been exceeded on three occurrences in the last fifty years by a maximum value of 1.47 metres. Since the 1980's a dam has been constructed at Hoa Binh, which is upstream of Hanoi, which is used to control approximately 60% of the Red River flow.

(2) Road

Highway clearance on the roads passing under the Ring Road shall be in accordance with the Table 6.2.1.

Vertical clearance on the roads passing under and over the Ring Road shall be with a height clearance of 5.0 metres.

Table 6.2.1 Clearance to Overbridges

Topography condition	Clear width for bridges comply with the class of highway							
	I	II	I	II				
	Case with the bicycle side (and primitive means of transport)		Case without bicycle side (in the difficult condition)		III	IV	V	VI
Normal topography	3+C1+9+C +9+CI+3	3+C1+9+C1+3	9+C+9	9	8	7	6	4 to 4.5

Source: MOT

<u>Notes:</u>

C1 - Separate width of bicycle side as 0.5 to 3 m

C - Width of centre separate line as 0.5 to 3 m

(3) Railway

The horizontal and vertical clearance for railways bridges shall be in accordance with the requirement of the Vietnamese Railways, extracted from the Vietnam Bridge Code, and are shown in Appendix A6-2-2.

(4) Flood

For rivers without navigation, flood clearance for bridges will be provided to the values given in Table 6.2.2

Table 6.2.2 Flood Clearances

Structure			Minimum clearance above Design flood level (m)	
			For Highway bridges	For Railway bridges
Girder	Water level rising by influence of piers	less than 1 m	0.50	0.50
		over 1 m	0.50	0.75
	Flooded material	Wooden logs and debris	1.00	1.50
		Roll stone	1.00	-
Bearing plate			0.25	0.25

Notes : From article 1.27 of Vietnam Bridge Design Code 22 TCN 018 - 79. Water level rising by piers is taken into account of flood level in the area, where there is water stagnating or reservoir, minimum clearance is 3/4 of wave depth.

(5) Aviation

The Study Team has obtained from the Vietnam Aviation Department their requirements for the height restrictions of Gia Lam Airport which are given in Table 6.2.3. However, they had informed that the airport is managed by the Combat Department, General Staff - Ministry of Defence and the Study Team should seek their agreement to the height restriction proposed by the Vietnam Aviation Department. It is necessary to confirm with Ministry of Defense during the detail design for the maximum tower heights given by the Vietnam Aviation Dept.

Table 6.2.3 Aviation Clearance Requirements (Vietnam Aviation Department)

Tower Location	Latitude (North)	Longitude (East)	Permitted Height
Hanoi Side	22.50	93.60	<83 m (91 m) **
Gia Lam Side	22.80	93.90	<91 m (99 m) **

Notes : ** above sea level

6.2.2 Design Loads

(1) Selection of Design Code

The project for the rehabilitation of National Highway No. 1 has adopted the AASHTO Specification for Highway Bridge for the design of bridge structures. The Study Team therefore decided to adopt AASHTO standard for the design of all structures on this project, as National Highway No. 1 connects directly onto Ring Road No. 3, in conjunction with Vietnamese Bridge Design Code Specification 22TCN 018-79. The AASHTO requirements have been modified, where appropriate to suit Vietnamese conditions.

(2) Live Loads

Live loads are defined as all moving vehicles and pedestrians that are permitted to travel on the highway. They will include pedestrian, bikes, motorcycles, vehicles (including cars and lorries) and non-ordinary vehicles. A non-ordinary vehicle is defined as a vehicle which is not permitted to travel along the highway without the permission of the police or highway authority. An example of non-ordinary vehicle would be a vehicle transporting an extremely large electrical generator to power station. For most highway structures the critical loading will either occur due to ordinary vehicles or non-ordinary vehicles.

The project for the rehabilitation of National Highway No. 1 has adopted the AASHTO Specification for Highway Bridges for the design of their bridge structures and in particular the design live load of HS20-44 * 125 % (Truck of Lane Loading) which ever produces the maximum stress. The Study Team adopted this loading standard for the design of all structures. Details of the AASHTO loading are given in Appendix A6-2-3.

(3) Comparison of AASHTO and Vietnamese Loading Standard

Following comments from PMU Thang Long, the Study Team undertook a comparison of the highway loading standards for both the AASHTO and Vietnamese Bridge Design Standard.

The H30 and XB80 loadings of Vietnamese code and (HS20-44) * 125%: loading of AASHTO code were chosen for comparison. When comparing the H30 train of vehicles to the (HS20-44) * 125%, observation of the Study Team is as follows:

(a) the maximum vehicle load for the H30 and HS20-44 * 125 is 30 and 40 tonnes respectively. Heaviest vehicles in Vietnam are either 40 or 44 tonnes and the proposed bridge loading standard should incorporate at least the requirement for 40 tonnes.

(b) the H30 arrangement consists of a train of vehicles at a spacing of approximately 15 metres. In the case of a small span bridge (20-30m) there is a high probability of achieving this arrangement. However, experience in Western countries has shown that following a traffic accident, during high traffic flows or where a bridge is situated at a road junction e.g. Cau Giay there is a possibility of stationary traffic and this situation could result in the spacing reducing to 7 meters which would result in a force greater than using present standard.

Standards not only have different loading arrangement but result in different permissible stresses. In the comparison the Study Team has limited ourselves to comparing only moments for a simple supported span. In Figure 6.2.1 the Study Team has compared the moments at mid span. The results show that for spans over 40 meters H30 is more critical primarily because the loading arrangement does not take account of the reduction in the load which would occur due to a traffic mix of heavy vehicle and cars.

With respect to the XB80 vehicle it is envisaged that this is an abnormal vehicle for transporting large equipment of electrical power station, generators or transformers. No such vehicle exists in the AASHTO standard. This is not to say that North American does not use non-ordinary vehicles. As a comparison the United Kingdom loading standard has non-ordinary vehicle designated the HB vehicle and has a gross weight of 180 tonnes. This vehicle is only an imaginary vehicle but when used in bridge design has certain limitation namely:

(i) only one vehicle is allowed per bridge
(ii) an overstress of 25% is permitted on the stress levels
(iii) no other vehicle is permitted within the traffic lane for 25 metres either side. This requirement is only relevant for medium to long span bridges.

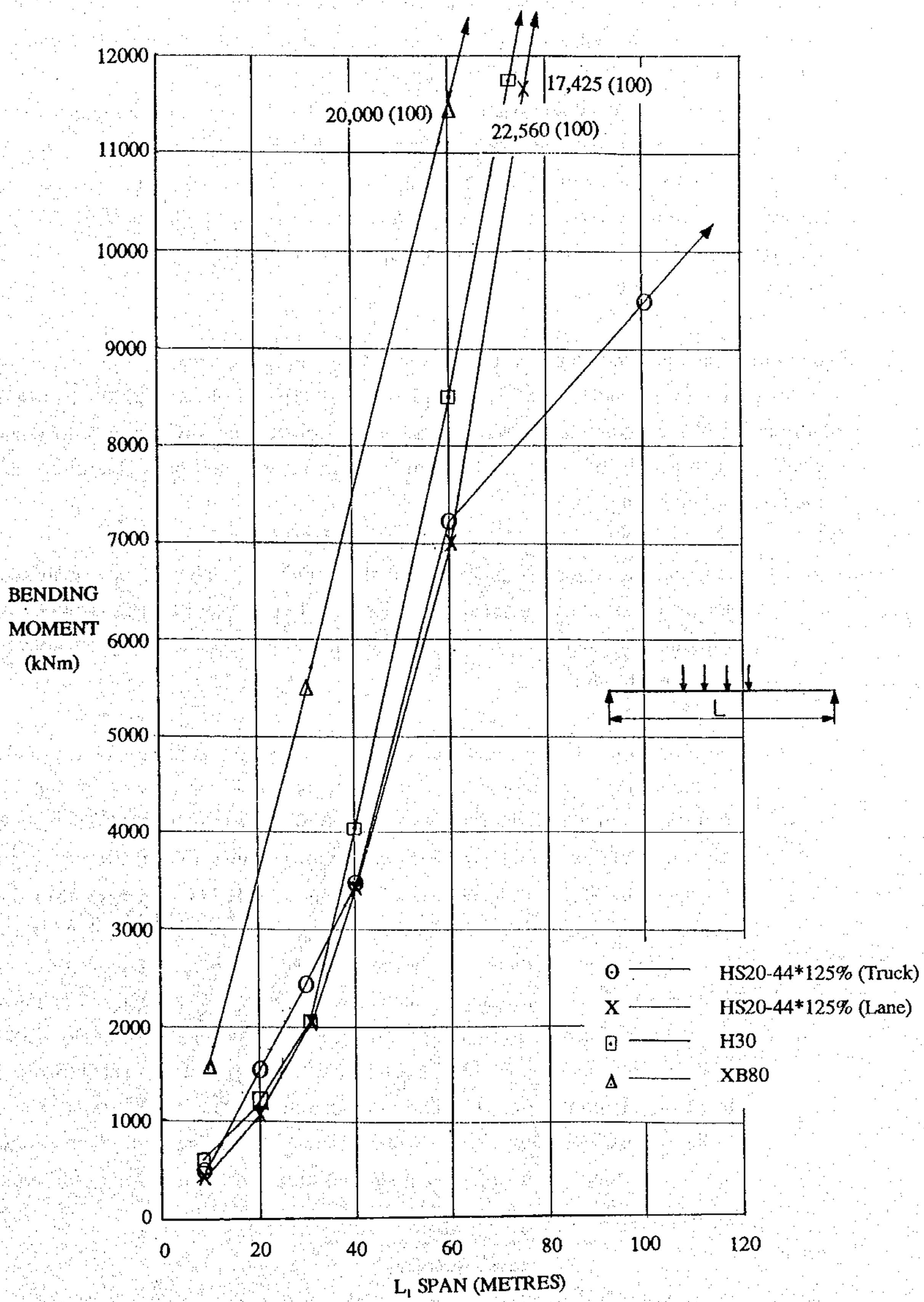

Figure 6.2.1 Comparison of Highway Loading Criteria

Clearly, overloading can still occur in any country whether it be USA, Japan or Vietnam. The experience in other South East Asian countries, which are not able to regulate axle loads using "Weigh Bridges" as in Western countries, gives rise to concerns of overstress during the design life especially at times of traffic congestion. In this respect, the Study Team would highlight the circumstances that exist in Thailand where the legal limit is 21 tonnes but virtually all heavy goods vehicles exceed this figure and actual weights of up to 100 tonnes for lorry/trailer combinations have been measured. To address this problem new structure over the Chao Phraya River in Bangkok (Rama XIII and the Industrial Ring Road Bridges) have been designed to a highway standard which gives long term load capacity.

The Study Team has undertaken a study to investigate the cost associated with increasing the design highway load from 125% of HS20-44 (1994) to 150% of HS20-44 (1994). Overloading of a structure can take two forms, local - due to an individual axle load and global - due to a train of vehicle. The Study Team investigated both conditions and concluded the following:

i) for concrete strengths in excess of 400 kg/cm^2 a slab will be capable of adequately supporting axle up to a design axle load of 150% HS20-44 (1994) without any increase in the normal slab thickness or the provision of additional reinforcement;

ii) the probability of a overload vehicles is low, noting that these would usually be interspersed with motor cars and light vans. However, over the design life of the structure the possibility of over load does exist and therefore to reduce the occurrence of overstress the Study Team has investigated the impact of increasing the highway loading to 150% HS20-44 (1994). The result of the study on both the approach and main river crossing show that for box girder type superstructures there will be an increase in the required prestress of between 6 and 9% depending on the span arrangement. The study has shown that for loaded length in excess of 40 metres the Vietnamese H30 and XB80 produce stresses in the structure greater that the proposed AASHTO loading. The Study Team therefore recommended that during the detailed design stage of the project a review of the loading criteria be undertaken. This study should review the highway loading and design standards of other recent built long span bridges in Vietnam.

(4) Wind Loads

For the rehabilitation of National Highway No. 1 structures have been designed to satisfy the requirements of AASHTO Article 3.15.

For girders and beams the design force shall be the greater of:

(i) a pressure of 245 kg/m^2 ; or
(ii) a total force not less than 447 kg/m

This is based on a design wind velocity of 160 km/hr.

The Study Team has obtained from the Observatory for Hydrometeorology and Environmental Control of the Red River Delta the maximum monthly wind velocities for the years 1945 - 96 and these are given in Appendix A6-2-4. The records show a maximum wind speed of 122 km/hr. The Study Team therefore considers the requirements of the AASHTO code are acceptable for use in the Hanoi area without any reduction or increase in the design wind velocity.

(5) Temperature

Daily and seasonal fluctuation in shade air temperature, solar radiation, re- radiation, cause the following:

(a) Changes in the effective temperature for bridge superstructures which, in turn, govern its movement.

Appendix A6-2-5 gives the yearly maximum and minimum temperatures for Hanoi between 1945 and 1996 by the Observatory for Hydrometeorology and Environmental Control of the Red River Delta. The maximum recorded temperature was 40.4°C and the minimum recorded temperature was 2.7°C.

The proposed range of effective bridge temperature is as follows:

	Temperature(max)	Temperature(min)
Concrete structures	50 °C	0 °C

(b) Differential temperature between the top surface and other levels in the superstructure. These are referred to as temperature differences and they result in loads and/or load effects within the superstructure.

The effects of temperature difference within the structure were derived for the data given in Appendix A6-2-6 and were based on information from the Hong Kong Design Manual for Highways and Railways.

Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top of the surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top of the bridge deck as a result of re-radiation and other effects.

Coefficient of thermal expansion shall be taken as $10 \times 10^{-6} / {}^{o}C$ for concrete.

(6) Seismic Forces

In the past, seismic forces have not been a significant design factor in Vietnam. However, geological conditions exist which requires consideration of seismic effects, especially in the northern part of Vietnam. The attached map (Figure 6.2.2) and Table 6.2.4, extracted from the Vietnamese Bridge Design Code Specification 22TCN 018-79, shows that Hanoi is classified as seismic intensity 8 and structures are required to resist an acceleration coefficient of 0.17.

Table 6.2.4 Acceleration Ratios

Seismic Intensity Scale shown in the map	6	7	8	9
Acceleration Coefficient	0.04	0.07	0.17	0.25
Seismic Performance Category	A	A	B	C

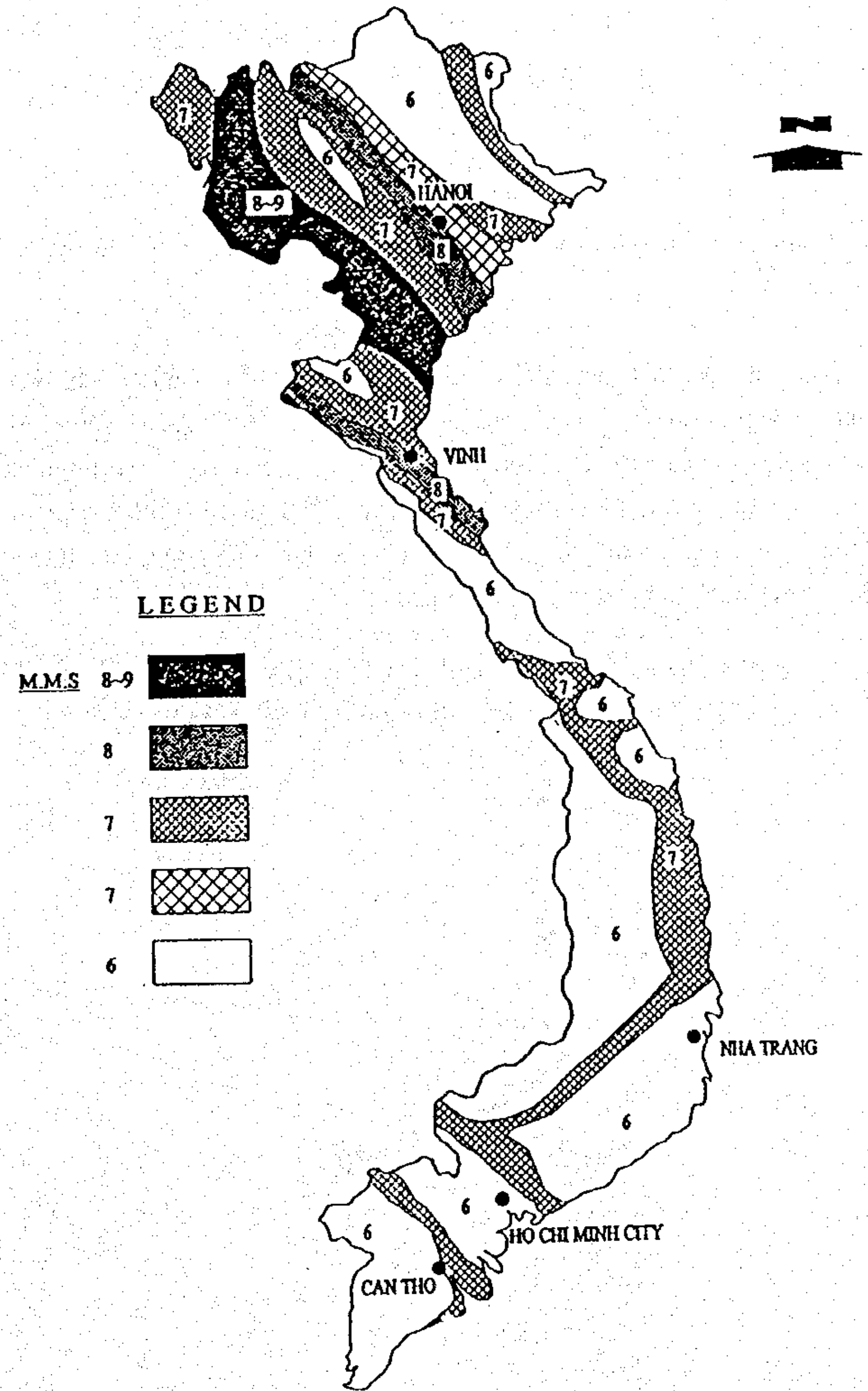

Figure 6.2.2 Vietnam Map of Seismic Intensity

(7) Ship/Barge Impact Forces

In the Steering Committee meeting held in January 1998, it was confirmed that the bridge would only need to be designed for ship/barge with a total weight of 1000 tonnes, ie an impact force on 125 tonnes in any direction.

(8) Load Combinations

The Study Team has proposed that the load combinations should be in accordance with AASHTO code.

The load combinations shall meet the requirement of part B - section 3 of AASHTO code. The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure or the foundation on which it rests shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for service load design and load factor design are given by:

Group (N) = $[\beta d \cdot D + \beta l (L+I) + \beta c\, CF + \beta e\, E + \beta \beta\, B + \beta\, coCoF + \beta\, wW + \beta\, sh\, Esh + \beta\, l\, BF + \beta\, r (R+S+T) + \beta\, eq\, EQ]$

Where:

N	=	Group number
γ	=	Load factor see Table 6.2.5
β	=	Coefficient, see Table 6.2.5
D	=	Dead load
L	=	Live load
I	=	Live load impact
E	=	Earth pressure
B	=	Buoyancy
CoF	=	Stream Flow Pressure
W	=	Wind load on structure
BF	=	Breaking force
CF	=	Centrifugal force
R	=	Rib shortening
S	=	Shrinkage
T	=	Temperature
Esh	=	Collision Force or Wind Load on Live Load, whichever is greater.

Table 6.2.5 Table of Coefficients γ and β

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
			β FACTORS														
GROUP		γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%	
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

---

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

$$** \text{ Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.00 for vertical and lateral loads on all other structures.

For culvert loading specifications, see Article 6.2.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

For Load Factor Design

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts.

β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20.

β_E = 1.0 for vertical earth pressure

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity ..... For Column Design

β_D = 1.0 when checking member for maximum axial load and minimum moment ....

β_D = 1.0 for flexural and tension members

β_E = 1.0 for Rigid Culverts

β_E = 1.5 for Flexible Culverts

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

6.2.3 Construction Materials

The proposed uses and properties of bridge construction materials are given below.

(1) Concrete

The usage of concrete and required strengths are shown in the Table 6.2.6 and 6.2.7 respectively.

Table 6.2.6 Concrete Classes and their Usage

Class of Concrete	Usage of Concrete
A-1	Prestressed concrete superstructure
A-2	Tower for cable stay bridge
B-1	Reinforced concrete slabs and cross beams of prestressed concrete bridge decks
B-2	Reinforced concrete for substructures including pier columns, cantilevered pier heads and abutments
B-3	Reinforced concrete piles
C	Gravity - type retaining wall
D	Ancillary items

Table 6.2.7 Strength of Concrete (kg/cm^2)

Class of concrete	Minimum compressive strength at 28 days by Cylinder test
A	400
B1 to B3	290
C	210
D	130

(2) Reinforcing Steel

Type, designation and strengths of reinforcing steel for concrete are specified in Table 6.2.8.

Table 6.2.8 Reinforcing steel strength (kg/cm^2)

Type	JISG 3112		ASTMA - 615	
	Designation	Yield strength	Designation	Yield strength
Rod bar	SD 295A	2400	Grade 40	2800
Deformed bar	SR 235	3000	Grade 60	4200

(3) Prestressing Steel

Yield and ultimate strength of steel used in prestressing and for cable stays are as shown in Table 6.2.9.

Table 6.2.9 Prestressing steel strength (kg/cm^2)

Type	Yield strength (kg/cm^2)	Ultimate strength ($kg.cm^2$)	Standard ASTM
Prestressing Bar	13500	15500	A421
Prestressing 7-Wire Strand	15000	17500	A416

(4) Steel Tube Piles

Class, designation and strengths of steel tube pile are given in Table 6.2.10.

Table 6.2.10 Steel Strength for Steel Tube Piles

Class	ASTMA 50		
	Designation	Yield point (kg/cm^2)	Tensile strength (kg/m^2)
2	Grade 40	2,900	4,100

6.3 Pavement Design Standard

6.3.1 General

The following points were taken into consideration for the study of pavement design standard:

- There are two types of design; flexible pavement of asphalt concrete and rigid pavement of Portland cement concrete;

- In the selection of pavement type, investment efficiency based on phased construction should be considered;

- Total cost during a project life span including construction cost, repair and maintenance cost, overlay cost, etc. should be considered in the selection of pavement type; and

- Construction aspects (availability of materials, construction period, etc.) and local site conditions (soil condition and climate) will influence the selection of pavement type.

6.3.2 Selection of Pavement Type

(1) Summary of Characteristics of Flexible and Rigid Pavement

Characteristics of flexible pavement and rigid pavement were compared together with local situations as summarized in Table 6.3.1.

Table 6.3.1 Flexible Pavement vs Rigid Pavement

Item	Flexible Pavement	Rigid Pavement
Stage construction	Two stages for 20 years	Single stage for 20 years
Maintenance	Periodic maintenance	Limited maintenance
Quality control	Sensitive to temperature during construction	Easy
Sensitivity to overloaded trucks	Sensitive	Not sensitive
Main material source	Imported asphalt	Locally produced cement
Adaptability to maintenance	Easy for repair and overlay	Careful traffic management is necessary

(2) Adaptability for Highway Construction

Both flexible and rigid pavement types are available for the construction of roads, however, the adoption of flexible pavement is recommended from the following view points:

i) Rigid pavement is very sensitive on soft ground flood plain area. The soil investigation results pointed out that the project area is mostly weak soil area and requires soil stabilization at high embankment stretches.

ii) The project highway is defined as urban toll road, therefore riding condition is one of important design factors.

(3) Comparison of Construction Cost

Preliminary cost estimate for both pavement types is shown below:

Pavement Type	Construction Price (VND/m^2)
Asphalt Concrete	466,500
Portland Cement Concrete	619,600

It is necessary to consider the cost of periodic overlay for asphalt concrete, however, from initial investment cost view point asphalt concrete pavement is superior.

(4) Selection of Pavement Type

As a result of the study, the asphalt concrete pavement is to be employed in consideration of:

- The fact that the asphalt concrete pavement can attain more smooth riding condition.
- The fact that flexible design will ensure expeditious construction.
- Lower initial cost.

However, we recommend to make more detailed study on selection of pavement type in the detailed design stage.

6.3.3 Design Standard of Flexible Pavement

The thickness design of the pavement were based on the "AASHTO Guide for Design of Pavement Structure, 1972 and 1986".

CHAPTER 7
FORMATION OF ALTERNATIVE PLANS

CHAPTER 7 FORMATION OF ALTERNATIVE PLANS

7.1 Route Alternative Study

7.1.1 General Concept

The southern section of Hanoi Third Ring Road (SHTRR) is located in Thanh Tri and Gia Lam districts, crossing the Red River 6.5 km downstream of Chuong Duong Bridge, and 3.5 km downstream of Pha Den (Hanoi) port.

The beginning point of SHTRR is located at Phap Van on National Highway No. 1 (NH-1) and the ending point is at Sai Dong on National Highway No. 5 (NH-5).

The cross-sectional configuration of SHTRR comprises access controlled through traveled way in the center and frontage roads on both sides. The access-controlled through traveled way is planned to become a toll road.

7.1.2 Definition of Terms

To determine alignment of SHTRR, three (3) steps of study were required, namely Corridor, Route and Alignment.

The term **"Corridor"** was perceived as the concept that it has 2 - 4 km wide strip and reveals effects or influences brought by the project road in the Study Area.

The corridor of SHTRR was fixed in between Phap Van on NH-1 and Sai Dong on NH-5. Accordingly, SHTRR will bring similar effects to both districts Thanh Tri and Gia Lam as well as the influence area even if a route location of SHTRR should vary within the corridor.

Cursory effects are summarized as follows:

i) to strengthen national highway network in metropolitan Hanoi;
ii) to mitigate traffic congestion on Chuong Duong Bridge;
iii) to divert through traffic in existing Central Business District (CBD) of Hanoi; and
iv) to stimulate urban development in Thanh Tri and Gia Lam districts as well as the southern part of metropolitan Hanoi.

The term "**Route**" was defined as having specific design controls such as type of road, design speed, road width together with number of lanes and location of interchanges/grade separation structures.

A route study could envisage location of road by averting physical constraints such as protected areas, historical monuments, large-scale public facilities and so forth. However, it still remained to identify exact lots and properties affected by the project road as the location of road was neither calculated nor marked up at site.

The term "**Alignment**" was defined as having coordinates vertically and horizontally based on outputs of detailed design such as finished grade, curvature, distance from centerline and so forth. Accordingly, affected lots and properties were identified topographically.

7.1.3 Methodology for Route Alternative Study

Taking into consideration the designated roles and functions of SHTRR based on the corridor study, a route study was conducted to select an optimum route and proceeded to the next step of alignment study.

Figure 7.1.1 shows the methodology of route study, which consists of the six (6) major work items:

(1) Design Controls

To realize the designated roles and functions of SHTRR, it is necessary to determine major design controls of which those elements have close relation horizontally and longitudinally to select a route.

(2) Toll Levying System

Additional design controls and facilities should be taken into account in case of toll road. The closed system may levy a toll from all users allowed to access and egress from a toll road over the entire stretch, while the open system may levy a toll not from short trip users but from medium and long trip users. Though both open and closed system will be applicable to apply to SHTRR, the closed system requires more toll gates and spaces.

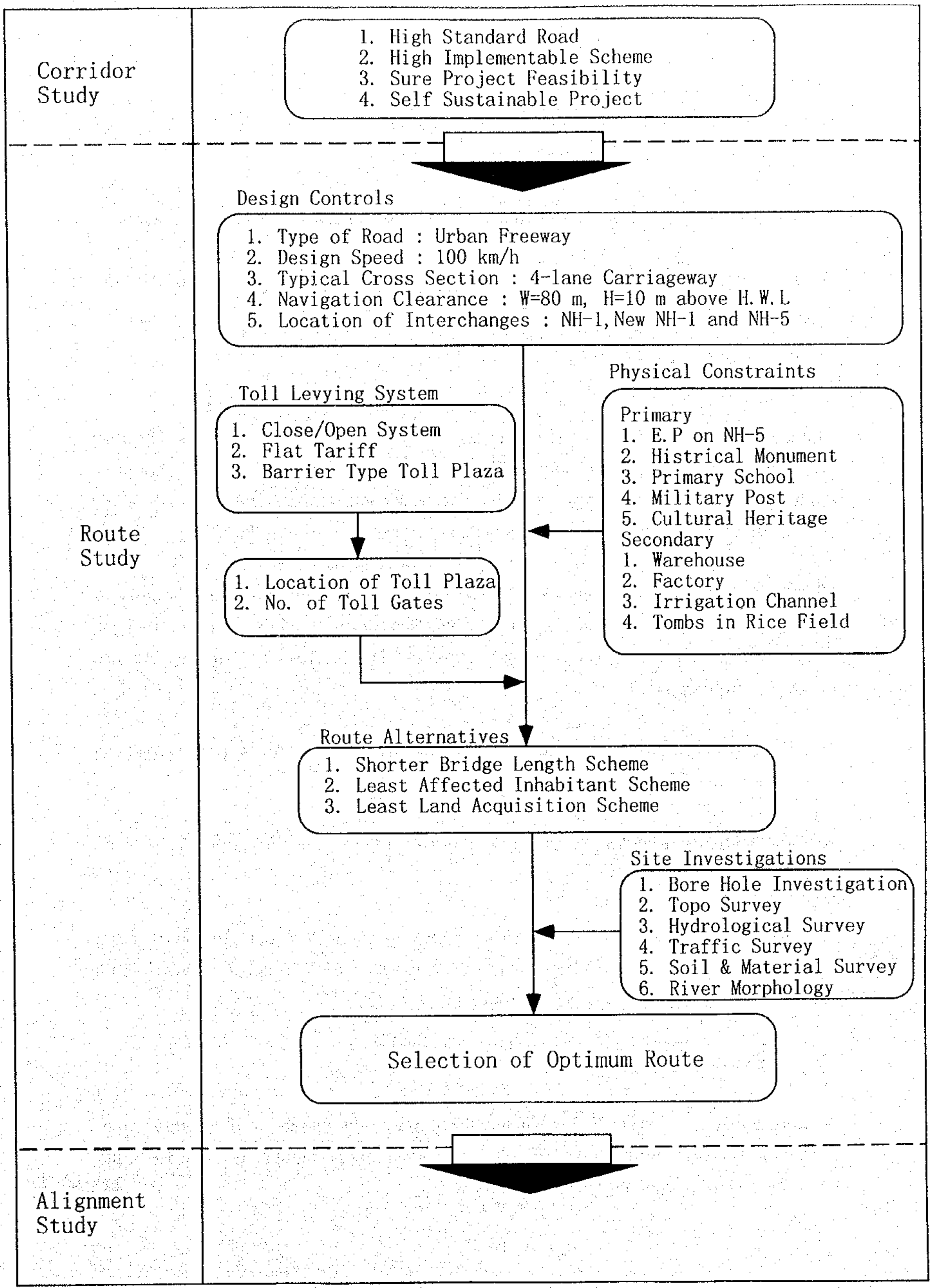

Figure 7.1.1 Methodology of Route Study

The flat tariff system may collect pre-determined tolls by type of vehicle in a certain stretch (one zone). If there is more than one zone in a whole stretch, users will pay pre-determined tolls by each zone, that is called "Zone Tariff".

Such a zone tariff system will apply to the Hanoi Third Ring Road (HTRR) as a whole.

A barrier-type toll plaza on through traveled way is the most common in case of flat tariff system due to efficiency of toll road operator and convenience of users.

(3) Physical Constraints

There usually exist many lots and properties affected by road development in the urban area. Some of those lots and properties are so important that a project road should be controlled to keep distance in order to avert adverse social impacts on them or to avoid considerable compensation costs, that is called "Primary Controlling Point".

Historical monuments, religious spots, primary school and military posts are classified as primary controlling points. On the contrary, established intersecting point on NH-5 and stable river crossing point based on the records of the Red River morphology are also primary controlling points.

"Secondary Controlling Point" is deemed desirable to keep distance if a project road can be averted from affecting lots and properties.

Factory/warehouse, tombs in rice field (locally called "mo") and irrigation channels are regarded as secondary controlling point.

To clarify the importance of cultural heritage, including religious spots, the Study Team obtained information from "Cultural and Information Division", Thanh Tri and Gia Lam district offices of Hanoi People's Committee.

(4) Route Alternatives

A route location of SHTRR has already been proposed in the pre-feasibility study conducted by TEDI. Referring to TEDI's route and aerial photographs taken in 1993, the three schemes were contemplated in the Study.

Alternative-1 : Shorter Bridge Length Scheme

Assuming that SHTRR should cross high water channel of the Red River by bridge where it is between dikes, the crossing point of the Red River is found in the north of TEDI's proposed route to make bridge 1,860 m in length as compared with 2,340 m of TEDI's route. Simultaneously, SHTRR will be the shortest among alternatives, namely approximately 450 m shorter than Alternative-2b.

Alternative-2a : Least-Affected Inhabitant Scheme (the TEDI's Route)

SHTRR is planned to cross the Red River at the most stable point observed by the records of river morphology over the past 40 years. The proposed route keeps distance from cultural heritage and passes undeveloped area as far as it is referred to the topographic map at a scale of 1 to 10,000.

Alternative-2b : Least-Affected Inhabitant Scheme

SHTRR would cross the Red River at the proposed point of the TEDI route and pass undeveloped area with minimum number of affected houses and buildings shown on the topographic maps at a scale of 1 to 5,000, and aerial photos taken in 1993 as far as required design criteria be kept.

Alternative-3 : Least Land Acquisition Scheme

SHTRR would cross the Red River at the proposed point of the TEDI's route and make full use of existing road right-of-ways to minimize additional land acquisition. However, existing causeway on the western dike cannot be used because unrealistic reconstruction of vital dike should be required to comply with design criteria of SHTRR.

(5) Site Investigations/Data Collection

Site investigations, which consist of bore hole investigation, topographic survey, hydrological survey, soil and material survey, traffic survey as well as collecting data concerning river morphology and aerial photos, were carried out to reveal natural and physical conditions of project area. These basic data were referred to for technically and financially evaluating each alternative.

7.1.4 Salient Features of Each Alternative Route

Figure 7.1.2 presents the location of route alternatives as a whole.

(1) Alternative-1 : Shorter Bridge Length Scheme (please refer to Figures 7.1.3 through 7.1.6)

The scheme of SHTRR was to cross the Red River upstream of TEDI's proposed route to make the bridge 1,860 m in length as compared with 2,340 m of the TEDI route. Simultaneously, the length of SHTRR was the shortest among alternatives, namely 450 m shorter than Alternative-2b or 700 m shorter than Alternative 3.

The starting point was set to avert a cemetery at Phap Van on the route that pass the northern part of Linh Dam lake as the western extension of SHTRR toward National Highway No. 6 should pass open space beside the Institute of Medical Herb under Ministry of Defense.

The route was planned to pass through open spaces such as agricultural land, fishery ponds, lakes and water reservoirs where possible to minimize adverse social impact, particularly relocation of inhabitants.

The route was planned to pass the center between the cemetery at Sta. 3+800 and Yen Duyen village with few disturbances, and turned toward the north separating from Alternative 2b at Sta. 4+500.

The route was also planned to pass the center between Thanh Tri pottery factory and a warehouse at Sta. 5+250 with some violation of properties, and to cross the Red River beside Thanh Tri sanitary wares factory and Ship Repair Enterprise where the distance between dykes would be the shortest within the corridor and the angle of crossing would be not right but a skew.

In spite of shorter bridge length, some buildings and houses were affected at both ends of river crossing, and several tombs in rice field in Gia Lam district were required to be relocated.

Alternative 1 was planned to join Alternative 2b and 3 at Sta. 10+400 to keep the same route.

(2) Alternative-2b : Least-Affected Inhabitant Scheme (please refer to Figures 7.1.3, 7.1.4, 7.1.9 and 7.1.10)

This route starts and passes the same route as that of Alternative-1 in the first 3.5 km long stretch to pass through open spaces in Thanh Tri area.

The route continues to pass through open space to keep distance from cultural heritage, primary school, concrete product factory and military post which were identified by using aerial photos taken in 1993 and from information obtained from "Cultural and Information Division, Thanh Tri and Gia Lam district offices of Hanoi People's Committee" and topographic survey.

The route also keeps distance to a 20 ha centralized wastewater treatment plant planned in "The Study on Urban Drainage and Wastewater Disposal System in Hanoi" conducted by JICA in 1995.

However, the route affected a cement warehouse of Chinfon factory to avoid demolishing a densely inhabited area beside it. The route followed the same route as Alternative-3 at around the hero's cemetery of Linh Nam commune and Linh Nam people's committee in Thanh Tri district.

(3) Alternative-3 : Least Land Acquisition Scheme (please refer to Figures 7.1.7 through 7.1.10)

The scheme of SHTRR was to cross the Red River at the proposed point of the TEDI route and make full use of existing road right-of-way to minimize additional land acquisition even though the length of road in Thanh Tri district became approximately 350 m longer than that of Alternative 2b. However, existing causeway on the western dyke cannot be used because vital dyke should be required to be re-aligned to comply with design criteria of SHTRR.

The route starts at existing at-grade intersection on NH-1 in the south of Linh Dam Lake where 2-lane 6 - 8 m wide 4.5 km road exists and connects the causeway on the western dyke. Accordingly, this route was planned to widen existing road to minimize land acquisition area.

The existing east-westward road forms trunk road network in Thanh Tri and the southern part of Hanoi where two groundwater pumping stations, one 35 m long concrete bridge and two crossing high voltage transmission lines were found.

Since many housing developments are underway in Thanh Tri district along existing roads, it should be pointed out that the route may cause increase of number of affected inhabitants in spite of decreased land acquisition area. However, it is likely practical to relocate such affected houses adjoining to planned frontage roads in both sides, observing actual practices in NH-5 or other projects in Hanoi.

So Thoung village at Sta. 3+800 is located at an intersection with a north-south road, and school and dense housing area were found in the vicinity of intersection.

Before connecting to the western dyke, SHTRR was planned to keep apart from the existing road and to pass in between residential areas, where church and park exist.

SHTRR runs northward along the periphery of residential area and violates the premise of cement warehouse of Chinfon factory in order to keep distance from Thanh Tri Pagoda and school at Sta. 5+500.

SHTRR manages to pass beside the hero's cemetery of Linh Nam commune at Sta. 6+800 in case of required right-of-way of 50 m.

SHTRR was planned to cross the Red River at the proposed point of the TEDI's route. However, the route turned toward 150 m north from the original intersecting point with the eastern dyke to avert historical heritage and tombs in rice field (locally called "mo") in Gia Lam district.

SHTRR runs parallel to irrigation channel and turn toward the established intersecting point on NH-5.

Total length of road between Phap Van on NH-1 and Sai Dong on NH-5 was estimated at approximately 12.2 km, while the distance between dykes is about 2,340 m.

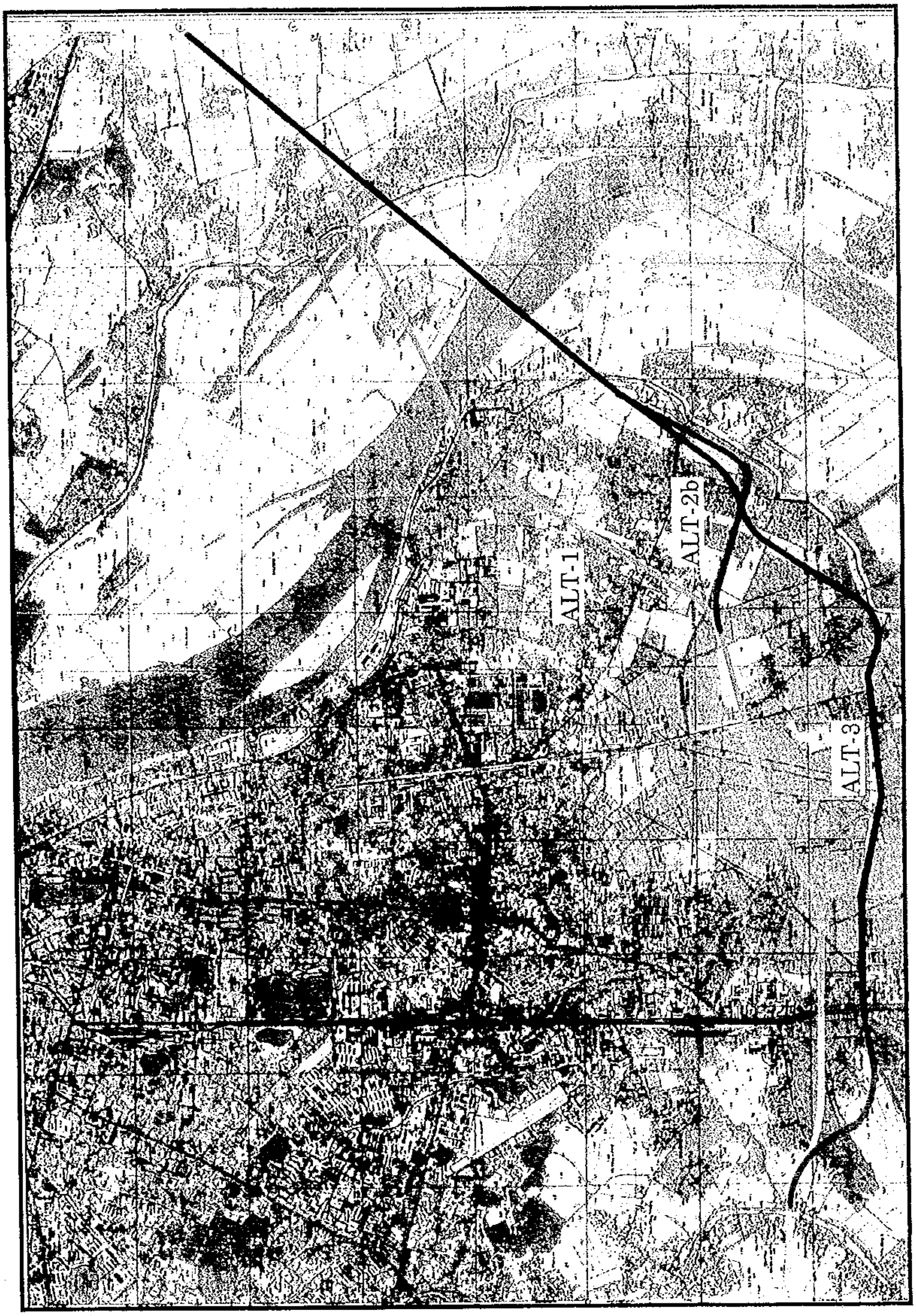

Figure 7.1.2　Location of Each Route Alternative

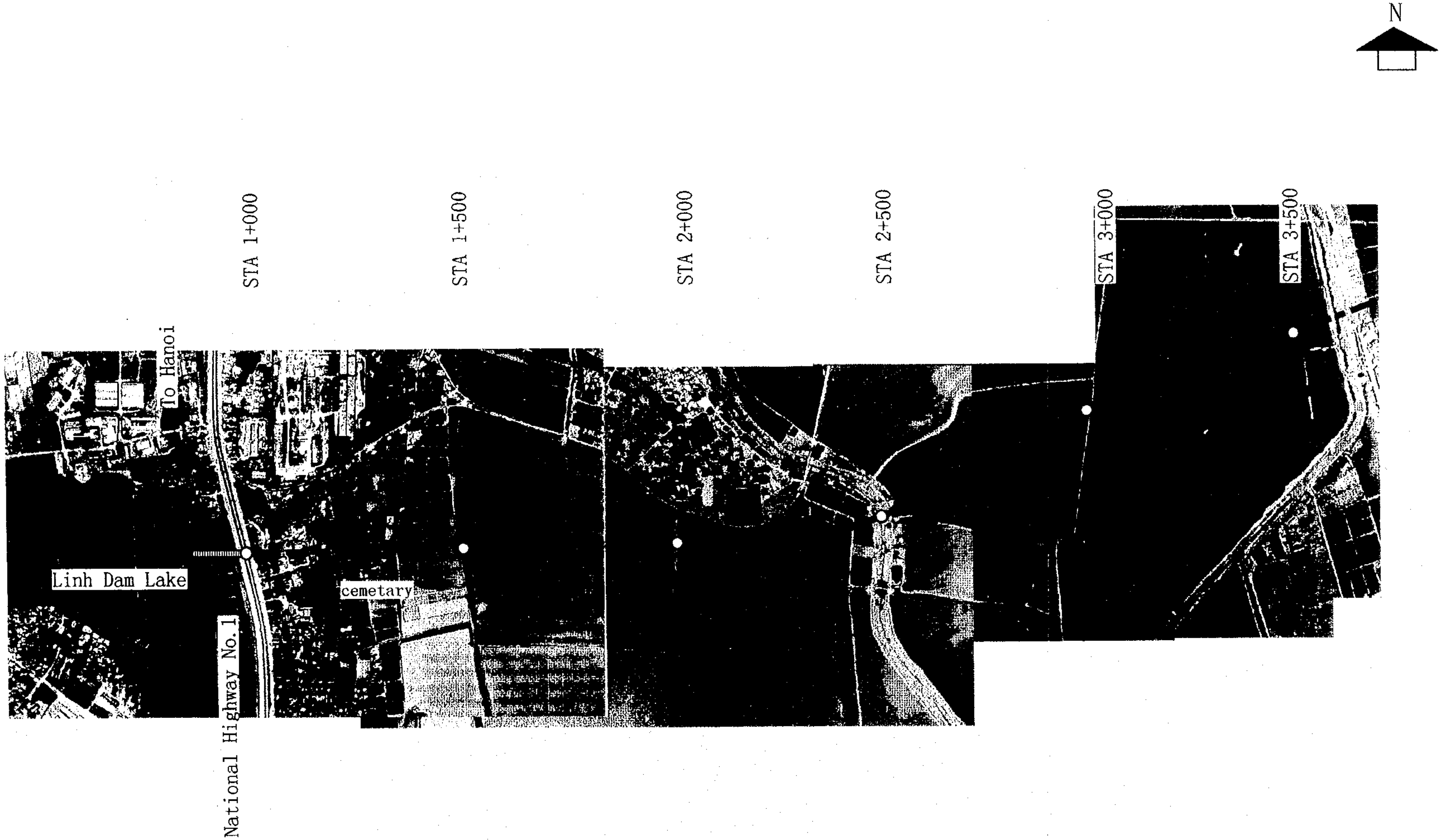

Figure 7.1.3 Alternative 1 & 2b (1)

THE FEASIBILITY STUDY ON
THANH TRI BRIDGE AND THE SOUTHERN SECTION
OF RING ROAD NO. 3 IN HANOI

ALTERNATIVE ROUTE STUDY

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

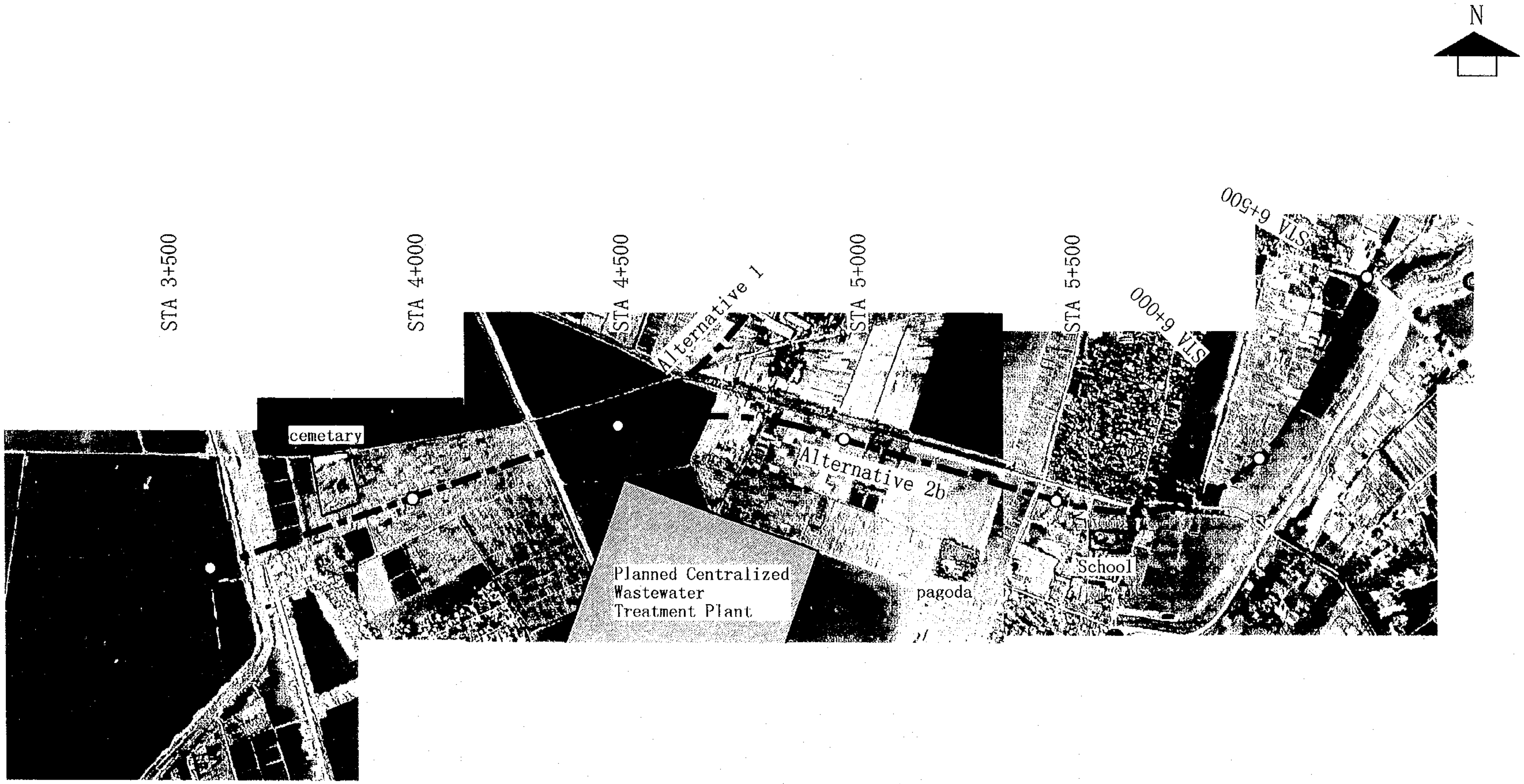

Figure 7.1.4 Alternative 1 & 2b (2)

THE FEASIBILITY STUDY ON
THANH TRI BRIDGE AND THE SOUTHERN SECTION
OF RING ROAD NO. 3 IN HANOI

ALTERNATIVE ROUTE STUDY

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

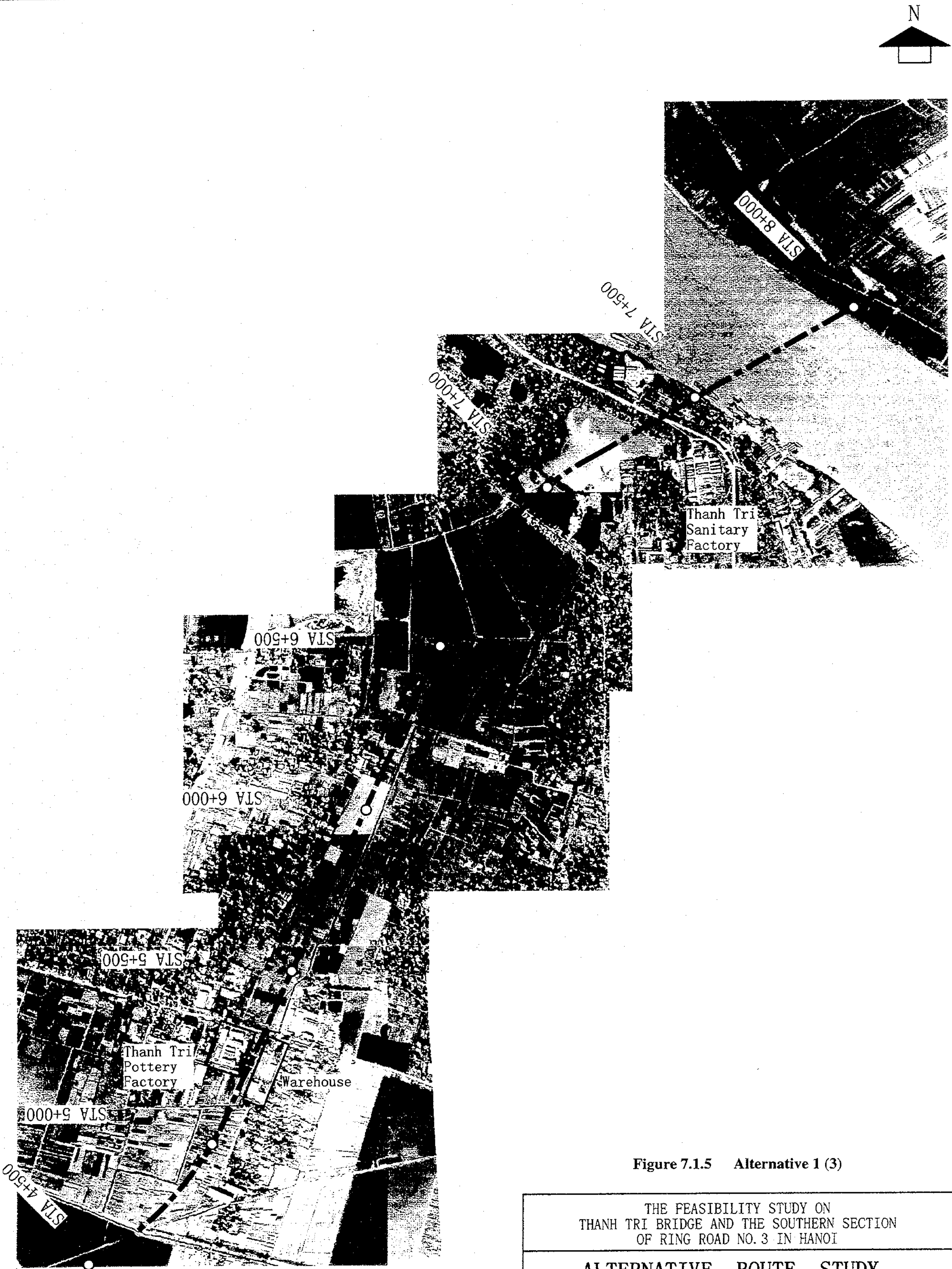

Figure 7.1.5 Alternative 1 (3)

THE FEASIBILITY STUDY ON THANH TRI BRIDGE AND THE SOUTHERN SECTION OF RING ROAD NO.3 IN HANOI
ALTERNATIVE ROUTE STUDY
PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

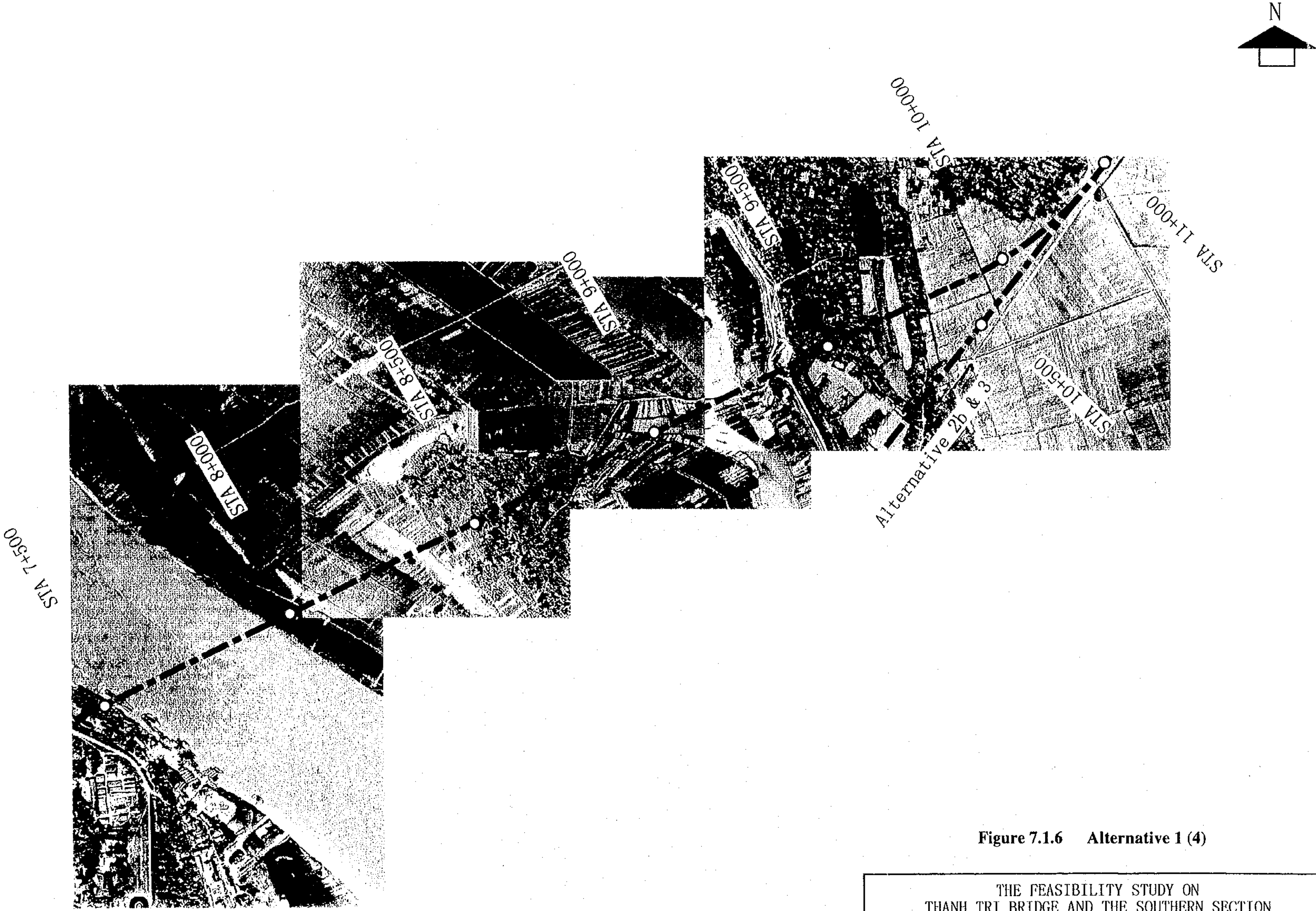

Figure 7.1.6 Alternative 1 (4)

THE FEASIBILITY STUDY ON
THANH TRI BRIDGE AND THE SOUTHERN SECTION
OF RING ROAD NO. 3 IN HANOI

ALTERNATIVE ROUTE STUDY

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

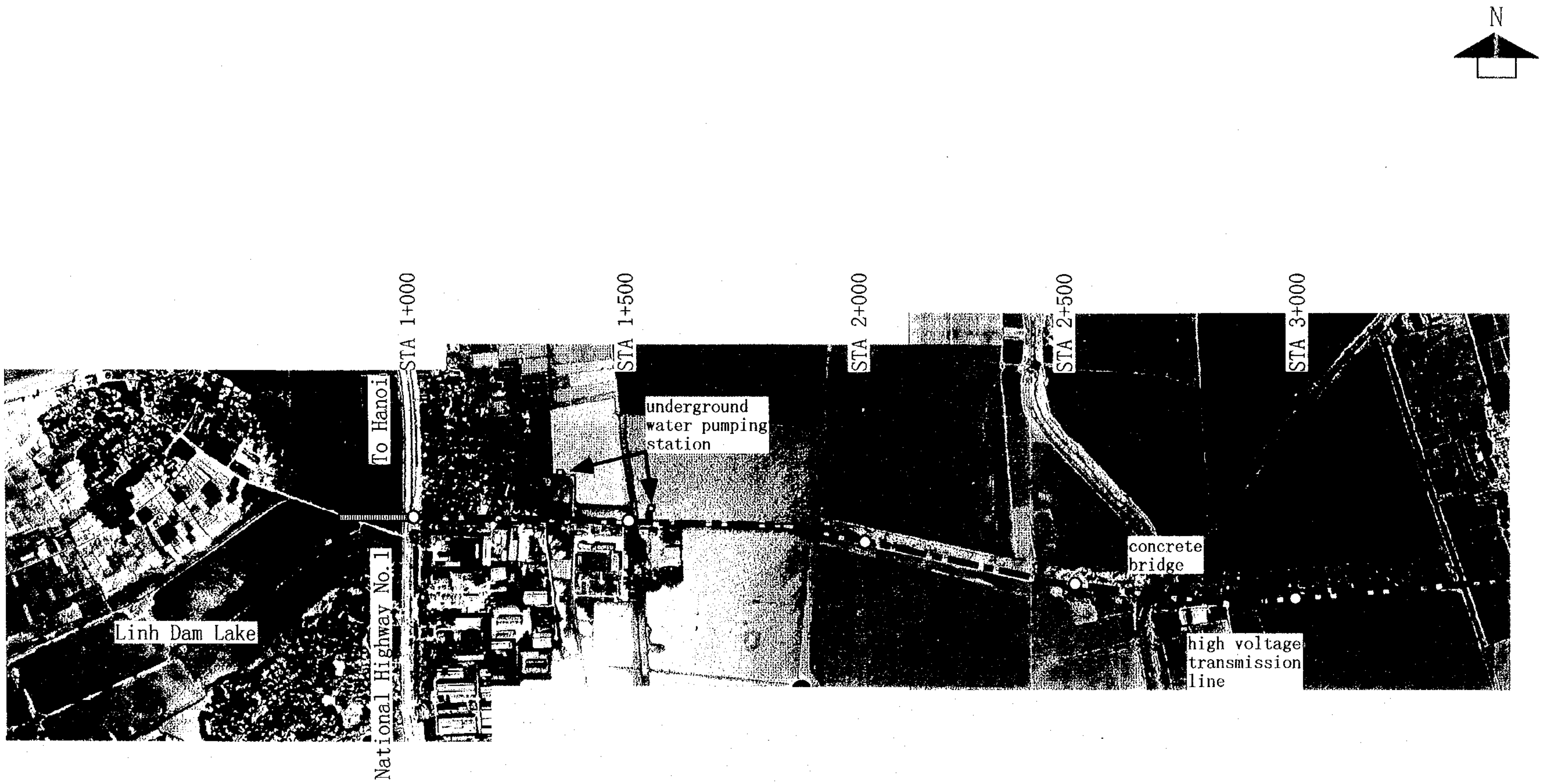

Figure 7.1.7 Alternative 3 (1)

THE FEASIBILITY STUDY ON
THANH TRI BRIDGE AND THE SOUTHERN SECTION
OF RING ROAD NO.3 IN HANOI

ALTERNATIVE ROUTE STUDY

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

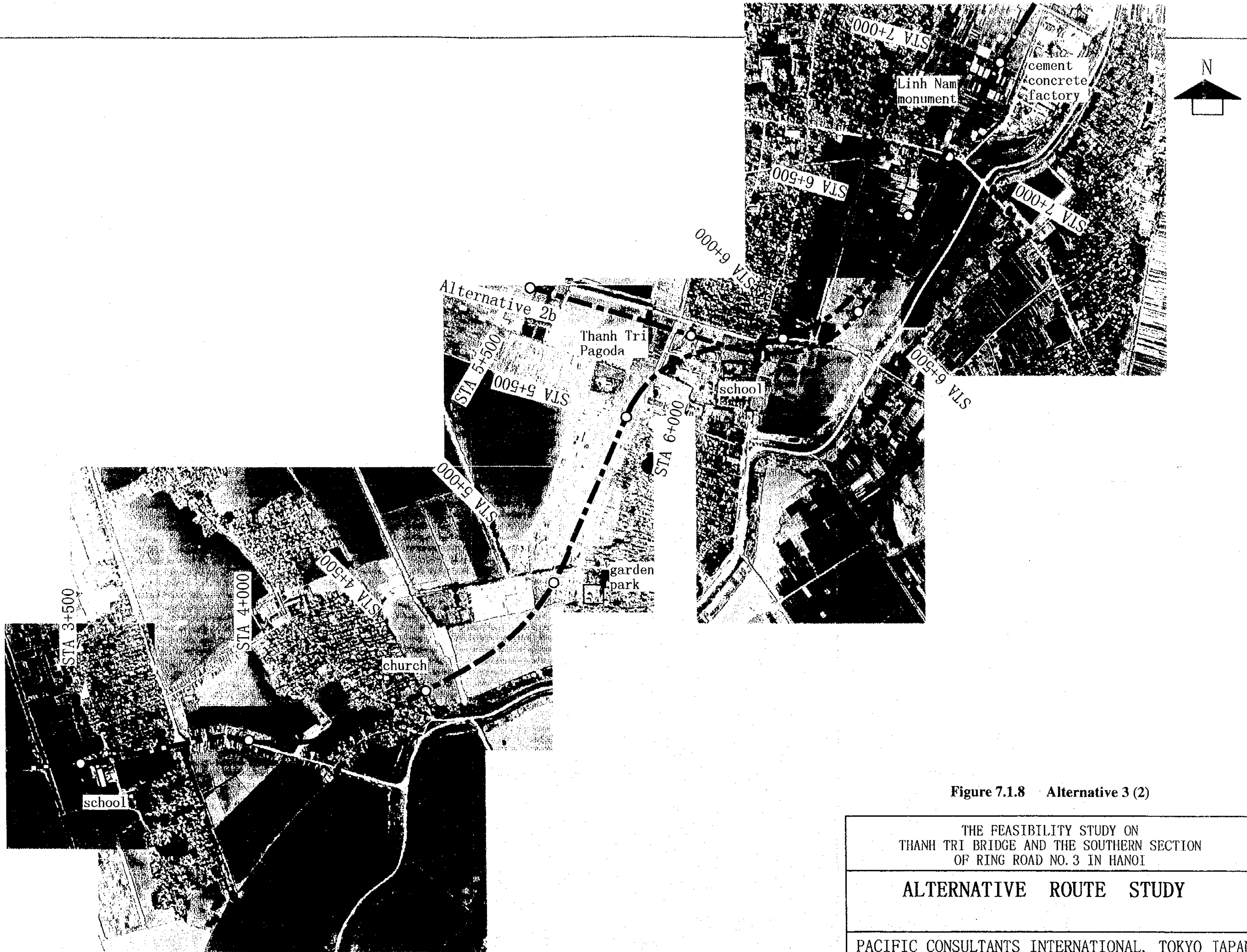

Figure 7.1.8 Alternative 3 (2)

THE FEASIBILITY STUDY ON THANH TRI BRIDGE AND THE SOUTHERN SECTION OF RING ROAD NO. 3 IN HANOI
ALTERNATIVE ROUTE STUDY
PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

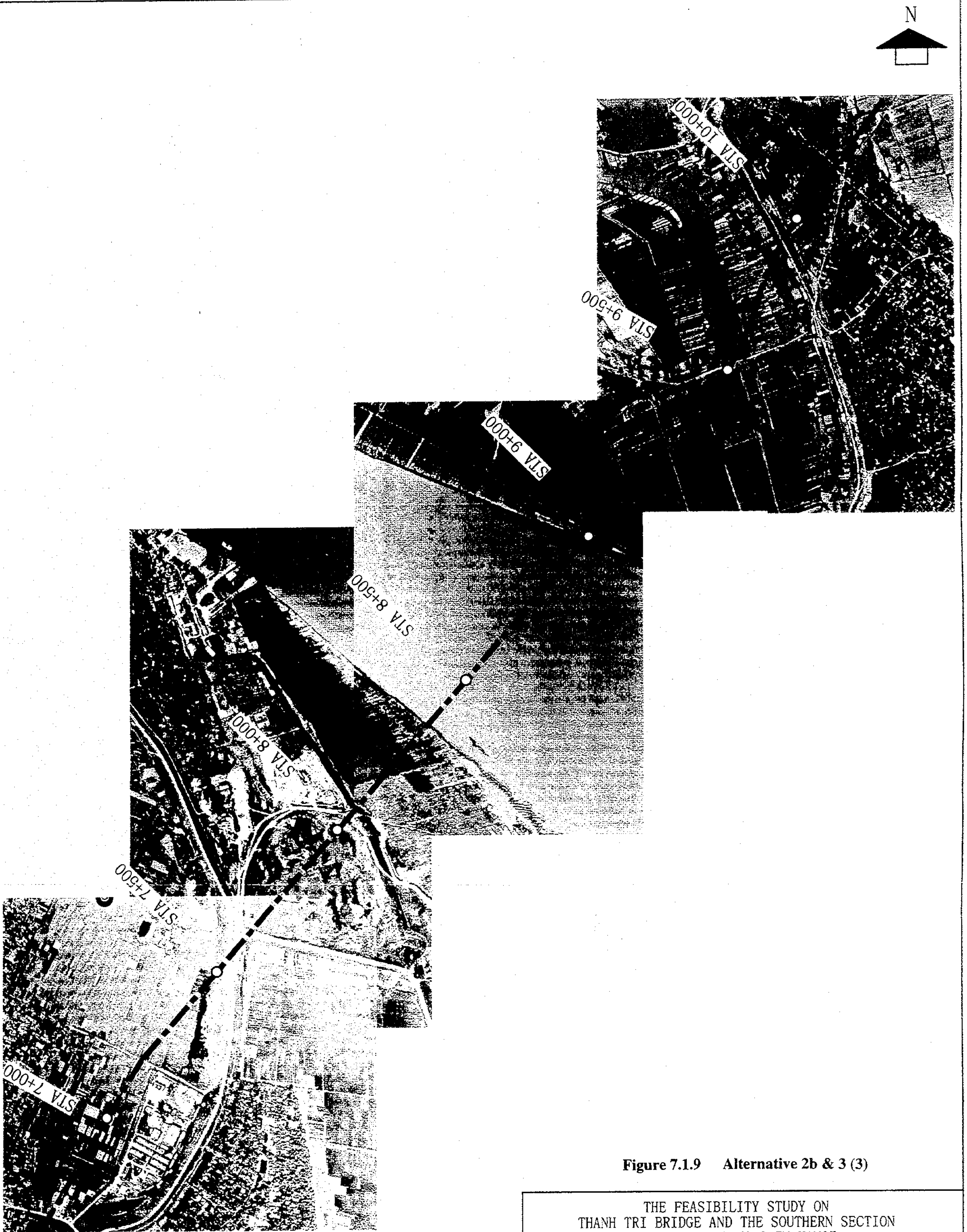

Figure 7.1.9 Alternative 2b & 3 (3)

THE FEASIBILITY STUDY ON
THANH TRI BRIDGE AND THE SOUTHERN SECTION
OF RING ROAD NO. 3 IN HANOI

ALTERNATIVE ROUTE STUDY

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN

Figure 7.1.10 Alternative 2b & 3 (4)

THE FEASIBILITY STUDY ON THANH TRI BRIDGE AND THE SOUTHERN SECTION OF RING ROAD NO. 3 IN HANOI
ALTERNATIVE ROUTE STUDY
PACIFIC CONSULTANTS INTERNATIONAL, TOKYO JAPAN