Table 27 Dimensions of Design Vessels

	Self-propelled vessel						Towed barge		
	2000	1000	300	200	100	40	500	400	100
Maximum Length (LOA) (m)	90	75	38	34	15	8	40	41	27
Maximum Breadth (m)	12.0	10.5	7.0	6.6	5.0	3.0	10.0	11.2	6.4
Laden draught (m)	3.5	2.8	2.2	1.7	1.0	0.8	1.7	1.3	1.0

(6) *Design Collision Velocity*

The recommended design impact velocity, V, to be used with each design vessel shall be as given in Table 28, where:

Vs = mean annual stream velocity adjacent to the bridge element under consideration (m/s)

Table 28 Design Impact Velocity for Desian Vessels

Design vessel	Design impact velocity, V (m/s)
Self-propelled Vessel >= 1000 DWT	3.3 + Vs
Self-propelled Vessel < 1000 DWT	2.5 + Vs
Towed Barge	1.6 + Vs

(7) *Vessel Collision Energy*

The kinetic energy of a moving vessel to be absorbed during a non-eccentric collision with a bridge pier shall be taken as:

$KE = 500CHMV^2$ where:

KE = Vessel collision energy (joule)

M = Vessel displacement tonnage (Mg)

The vessel mass, M, shall be based on the loading condition of the vessel and shall include the empty mass of the vessel, plus consideration of the mass of cargo, for loaded vessels, or the mass of water ballast for vessels transiting in an empty or lightly loaded conditions.

CH = Hydrodynamic mass coefficient

= 1.05 Underkeel clearance >= 0.5*draft

= interpolate

= 1.25 Underkeel clearance <= 0.1*draft

V = Vessel impact velocity (m/s)

(8) *Ship Collision Force on Pier*

The head-on ship collision impact force on a pier shall be taken as:

$PS = 1.2*10^5 V(DWT)^{0.5}$ where;

PS = Equivalent static vessel impact force

DWT = Deadweight tonnage of vessel

V = Vessel impact velocity

(9) ***Barge Collision Force on Pier***

The collision impact force, N, on a pier for a standard hopper barge shall be taken as:

$PB = 6.0*10^4 aB$	$aB < 100$ mm	where;
$= 6.0*10^6 + 1600aB$	$aB >= 100$ mm	PB = Equivalent static barge impact force (N)
		aB = Barge bow damage length specified in the following equation (mm)
		$aB = 3100 [(1 + 1.3*10^{-7}KE)^{0.5} - 1]$

The impact force for design barges larger than the standard hopper barge shall be determined by increasing the standard hopper barge impact force by the ratio of the larger barge's width to the width of the standard hopper barge.

(10) ***Application of Impact Forces***

1) Substructure Design

For substructure design, equivalent static force, parallel and normal to the centerline of the navigable channel, shall be applied separately as follows;

- * Parallel to the alignment of the centerline of the channel: 100% of the design impact force
- * Normal to the alignment of the centerline of the channel: 50% of the design impact force

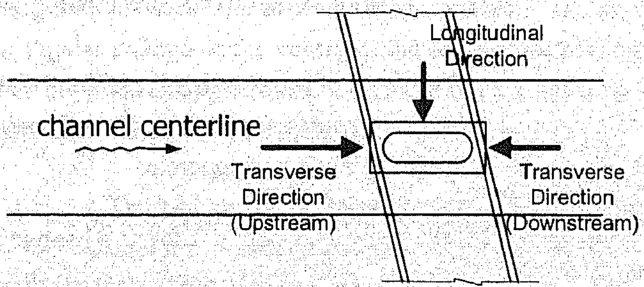

Figure 10 Application of Vessel Collision Force on Pier

The impact force shall be applied to a substructure in accordance with the following criteria:

- * For overall stability, design impact force is applied as a concentrated force on the substructure at the mean annual high water level of the waterway, as shown in Figure 11.
- * For local collision forces, design impact force is applied as a vertical line load equally distributed on the depth of the head block (HL), as shown in Figure 12.

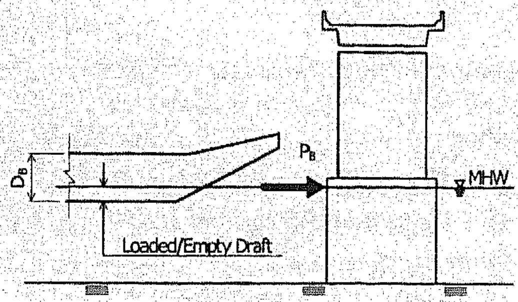

Figure 11 Load Application for Overall Stability

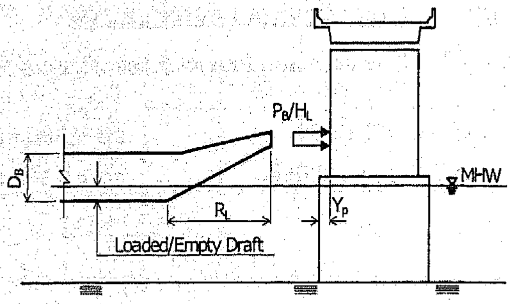

Figure 12 Load Application for Local Collision

2) Superstructure

For superstructure design, design impact force may be applied as an equivalent static force transverse to the superstructure component in a direction parallel to the centerline of navigable channel, if necessary.

1.3.8. Prestress Force

The Prestressing force shall be computed using the equation as bellow.

$$P(x) = Pi - [\triangle Pi(x) + \triangle Pt(x)]$$

where,
P(x) :Prestressing force of cross section under consideration
Pi :Prestressing force at prestressing work at the tensioning end of tendon
△Pi(x) :Loss of prestressing force immediately after prestressing to be computed considering the following effects
Elastic deformation of concrete
Friction between tendon and duct
Anchorage seating loss, or set loss
Others
△Pt(x) :Loss of prestress force over time to be computed considering the following effects
Relaxation of prestressing steel
Creep of concrete
Shrinkage of concrete

To calculate the indeterminate forces at the serviceability or the fatigue limit state, the prestress force may be taken to be the characteristic value of the prestressing force.

[Commentary] (1) The following effects shall be considered when calculating the prestress losses, namely △Pi(x) and △Pt(x).

(1) *Elastic deformation of concrete*

The prestress loss due to elastic deformation of concrete shall always be considered for the pre-tensioning system. When post-tensioning tendons are tensioned one by one, the prestress loss due to elastic deformation of concrete shall be calculated, and the average prestress loss may be computed.

for pre-tensioning system △pEs = np*f'cpg

for post-tensioning system △σp = 1/2np*f'cpg(N-1)/N

where, △σp : Prestress loss in prestressing tendon

np :Young' modulus ratio of prestressing tendon to concrete = Ep / Ec

f'cpg :Concrete compassive stress due to prestressing at the centroid of prestressing tendons

N :Number of tensioning times (number of groups of the tendon)

(2) *Friction between prestressing tendon and duct*

The prestress loss in prestressing tendon due to friction varies considerably on condition of the inner surface of the duct and type, degrees of rusting, and alignments of the prestressing tendon.

Loss of prestressing tendon force due to friction can generally be separated into two terms – one related to the angular change of the centroid line of the prestressing tendons, and the other related to length of the prestressing tendon. tension in the prestressing tendon at cross section under consideration can be expressed by following equation.

Px = Pi*e(μα+λx) where,

Px:Tension force of tendon at considered cross section
Pi:Tension force at the tensioning end of tendon
μ:Friction coefficient for angular change of 1 radian
α:Angular change of the tendon in radian
λ:Friction coefficient per unit length
χ:Length from the end of the tendon to the considered cross section

Though, μ and λ should be determined by site measurement, but values shown in next table may generally be used for calculation for prestressing force in a tendon encased in a sheath.

Table 29 Friction Coefficient

	μ	λ
Prestressing wire, Prestressing wire strand	0.3	0.004
Prestressing steel bar	0.3	0.003

Because external tendons are arranged outside of the concrete and are free from friction except at anchorages or deviators, friction loss at a section may not be considered. Depending on the material of the duct, the value of the coefficient of friction at anchorages or deviators given in following table may be used.

Table 30 Friction coefficient between prestressing wire strand and duct

	μ	λ
Steel Polyethylene	0.30 0.15	0.004 0.004
(for prestressing wire and prestressing wire strand)		

The wobble coefficient may be determined on the basis of prior experimental or other data in case a special sheath or spacer is used in order to reduce the friction, or the prestressing steel is specially processed, or the friction is reduced by giving impact to the prestressing bar.

(3) ***Anchorage seating, or set***

If during the anchoring of the tendon 'set' occurs, the ensuing loss in prestress shall be taken into account. Especially in the case of a wedge-type anchorage system, since the amount of set is relatively large, the loss of prestress and the affected length, shall be determined prior to tensioning on the basis of previous experience or available data. The "set" refers to the pulling in of a prestressing tendon at the anchoring device during anchoring. As the actual amount of set varies depending on the anchoring device used, the actual amount corresponding for each device shall be determined (See "Guidelines for Design and Construction of Prestressed Concrete Structures", 1991, JSCE).

When there is no friction between the prestressing tendon and duct, loss of the prestressing tendon force due to set may be calculated using the following equation;

$\Delta P = (\Delta\iota)/\iota A_P E_P$ where,
ΔP :Loss of tension force due to set of tendon
$\Delta\iota$:Setting length
ι :Length of tendon
A_P :Area of tendon
E_P :Young's modulus of tendon

1.4. Pile Foundation Design

The bridge is located in the soft ground area, which bearing stratum is about -50 m. Therefore, the pile foundation shall be adopted for foundation type.

Pile foundation design shall be made for service limit states, strength limit states and extreme event limit states respectively. Each limit state include the followings:

Table 31 Verification Items for Limit States

	Verification Items	Remark
Service Limit States	Adequate Bearing resistance	Allowable Bearing Resistance
	Structural Resistance	Control of Cracking
	Tolerable Settlement	Considered Bridge Performance
	Tolerable Horizontal Displacement	
Strength Limit States	Adequate Bearing Resistance	Considered punching failure
	Structural Resistance	
	Horizontal Displacement	P-Y curve
Extreme Event Limit States	Bearing Resistance	
	Structural Resistance	
Service Limit state	Overall stability	Considered Lateral Flow

1.4.1. Transverse Spring Coefficients of Pile

Transverse Spring Coefficients of Pile for the analysis are shown in Table 32.

Table 32 Transverse Spring Coefficients of Pile

	$h \neq 0$	$h = 0$	$h \neq 0$	$h = 0$
K1	$\dfrac{12EI\beta^3}{(1+\beta h)^3+2}$	$4EI\beta^3$	$\dfrac{3EI\beta^3}{(1+\beta h)^3+0.5}$	$2EI\beta^3$
K2, K3	$K_1\dfrac{\lambda}{2}$	$2EI\beta^2$	0	0
K4	$\dfrac{4EI\beta}{1+\beta h}\dfrac{(1+\beta h)^3+0.5}{(1+\beta h)^3+2}$	$2EI\beta$	0	0

where:

Kv = $\dfrac{A_p E_p}{L}$ (kN/m)	= characteristic value of a pile $\beta = \sqrt[4]{\dfrac{k_H D}{4EI}}$ (1/m)
γ =	$h + \dfrac{1}{\beta}$
kH =	coefficient of lateral ground spring (kN/m^3)
D =	pile diameter (m)
EI =	flexure rigidity of a pile (kN.m^2)
h =	axial length of a pile above the ground level (m); if h < 0, h = 0
a =	0.014(L/D) + 0.72 for driven pile by percussion 0.017(L/D) - 0.014 for driven pile by vibro hammer 0.031(L/D) – 0.15 for cast-in-place conrete pile
Ap =	net area of a pile (mm^2)
Ep =	modulus of elasticity of a pile (kN/mm^2)
L =	pile length (m)
D =	pile diameter (m)

1.5. Materials

In this section, the specified values for the concrete, reinforcing bar and prestressing tendon on the TVCN and AASHTO are described. In the detail design, these values should be reviewed and modified if necessary.

1.5.1. Concrete

Although concrete strength for each structural element shall basically follow the Vietnamese Standard considering local conditions, they may be modified based on the AASHTO LRFD and Japanese Specifications for reasons of required properties. The followings are concrete strengths for each structural element to be used in this Project.

Table 33 Concrete Strength by Structural Member

Compressive Strength at 28 days (MPa) (Cylinder Specimen)	Structural Member	Remarks
50	Pretensioned Slab/Girder	During design period, these may be modified due to requirements.
45	Free Cantilever PC Girder	
40	Post-tensioned PC I-Girder Cast-in-situ PC Slab/Girder	
35	Cast-in-situ PC Slab Cast-in-situ PC Crossbeam	
28	RC Girder Diaphragm (Crossbeam) RC Deck Slab Substructure (Pier, Abutment, Pile Caps, Wingwall) Retaining Wall, Box Culvert Precast Reinforced Concrete Plate Precast Pile Precast Parapet	
21	Approach Slab Pipe Culvert Precast Concrete Curb	
30	Cast-in-situ Bored Pile	
18	Non-reinforced Concrete Structure Lean Concrete	

In this project, only normal density concrete shall be used. The properties of concrete are as shown below.

Table 34 Concrete Properpies

Modulus of Elasticity (MPa)	Poisson's Ratio	Modulus of Rupture (MPa)
Ec = $0.043\gamma_c^{1.5}\sqrt{f'_c}\quad(1440 \le \gamma_c \le 2500)$	0.20	fr = $0.63\sqrt{f'_c}$
γc = density of concrete (kg/m^3) f' c = specified strength of concrete (MPa)		flexure tensile stress

Stress limits for concrete in Service Limit State in PC are shown in Tables 35 and 36. For RC, as the width of flexure cracks is controlled by distributing the reinforcement over the region of maximum concrete tension, stress limit for concrete is not described.

Table 35 Temporary Tensile Stress Limits in PC before Losses

Bridge Type	Location		Stress Limit (MPa)
Other Than Segmentally Constructed Bridges	*	In precompressed tensile zone without bonded reinforcement	Not Applicable
	*	In areas other than the precompressed tensile zone and without bonded reinforcement	$0.25\sqrt{f'_{ci}} \leq 1.38$
	*	In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 fy, not to exceed 210 MPa.	$0.63\sqrt{f'_{ci}}$
	*	For handling stresses in prestressed piles	$0.415\sqrt{f'_{ci}}$

Table 36 Compressive Stress Limits in PC at Service Limit State after Losses

Location		Stress Limit (MPa)
*	In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	0.45 f'c
*	In other than segmentally constructed bridges due to live load and one-half the sum of effective prestress and permanent loads	0.40 f'c
*	Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	0.60 φw f'c

Table 37 Tensile Stress Limits in PC at Service Limit State after Losses

Bridge Type	Location		Stress Limit (MPa)
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, assuming uncracked sections		
	*	For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.50\sqrt{f'_{ci}}$
	*	For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.25\sqrt{f'_{ci}}$
	*	For components with unbonded prestressing tendons	No tension

1.5.2. Reinforcing Bar

Two types of Grade 300 and Grade 420 shall be used. The properties and strength are as shown below.

Table 38 Properties and Stress Limit of Reinforcing Bars

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)
Grade 300	300	500	200,000
Grade 420	420	620	200,000

1.5.3. Prestressing Steel

Uncoated, stress-relieved or low-relaxation, seven-wire strand, or uncoated plain or deformed, high-strength bars, shall have the following properties and strength as shown in Table 39.

Table 39 Properties of Prestressing Strand and Bar

Material	Grade or Type	Diameter (mm)	Tensile Strength fpu (MPa)	Modulus of Elasticity Ep (MPa)	Yield Strength fpy (MPa)
Strand	1725 MPa (Grade 250)	6.35 – 15.24	1725	197,000	0.85 f pu for stress-relieved 0.90 f pu for low-relaxation
	1860 MPa (Grade 270)	9.53 – 15.24	1860		0.90 f pu
Bar	Type 1, Plain	19 – 35	1035	207,000	0.85 f pu
	Type 2, Deformed	16 – 35	1035		0.80 f pu

Stress limits for each tendon type are as shown in Table 40.

Table 40 Stress Limits for Prestressing Tendons

	Tendon Type		
	Stress-relieved Strand / Plain high-strength bars	Low Relaxation Strand	Deformed High-strength Bars
Pretensioning			
* Immediately prior to transfer ($f_{pt} + \Delta f_{pES}$)	0.70 f pu	0.75 f pu	-
* At service limit state after all losses (f pe)	0.80 f py	0.80 f py	0.80 f py
Post-tensioning			
* Prior to seating–short-term fs may be allowed	0.90 f py	0.90 f py	0.90 f py
* At anchorages and couplers immediately after anchor set ($f_{pt} + \Delta f_{pES} + \Delta f_{pA}$)	0.70 f pu	0.70 f pu	0.70 f pu
* At end of the seating loss zone immediately after anchor set ($f_{pt} + \Delta f_{pES} + \Delta f_{pA}$)	0.70 f pu	0.74 f pu	0.70 f pu
* At service limit state after all losses (f pe)	0.80 f py	0.80 f py	0.80 f py

1.5.4. Durability of Concrete

(1) Environmental condition

It is considered extremely severe environment for a concrete structure that a region where saline moisture, seawater etc., are splashed or sprayed constantly, such as a location for pier at the coast.

It is because the increment of the density of salinity occurs in such a region where supply and dry of salt water being repeated.

It seems appropriate to take some counter measures, considering the following conditions;

The bridge crosses the sea.

It is an especially important structure.

The design service life shall be 100 years.

(2) ***Proposal of counter measure concerning durability of bridge structurres***

We would propose to provide following counter measures as recommendable.

1) The cover of the reinforcing bar of main girder is to be 45mm that is 10mm more than the designed cover.

2) The cover of the reinforcing bar of pier & abutment is to be 40mm when the structure is under water, and is to be 60mm when the structure is at elevations of tidal water.

3) Painted reinforcing bars, performance of which is equal to or greater than that of epoxy painted reinforcing bars, shall be used for the reinforcing bars at the most outside perimeter of structure, excluding that of upper slab structure.

Regarding 1)

The standard cover is 35mm according to the Specifications for Highway Bridges of the Japan Road Association.

Here, 45mm cover would be proposed with additional thickness of 10mm, in consideration of a margin for the construction error etc.

Regarding 2)

The standard cover is 40mm and 60mm according to the Vietnamses standard TCXDVN 327:2004.

Regarding 3)

It was anticipated from the beginning stage that the structure would be affected by saline splash. As for the Dinh Vu-Cat Hai Bridge, it would be proposed to adopt the painted reinforcing bars for the most outside perimeter bars so as to secure the durability which is equal to or greater than that of above-mentioned bridge.

The bars painted with "Magne line", which is categorized as polyacrylic ester paint, may be adopted as the said painted reinforcing bar.

Regarding "Magne line"

This material is widely adopted as coating material on the surface of concrete and rust prevention material for steel material, and it retains the following features;

Certain rust prevention action

The salt water atomization tests have been performed by official body for the steel material painted with the said material.

The result is obtained as "Rust will not be generated after 4000 hours of the salt water atomization".

This result excels the required standard of the epoxy painting reinforcing bars to the same tests, which is "Rust shall not be generated by 1000 hours or more and within 1100 hours under the salt water atomization "

Therefore, it could be expected that the performance of reinforcing bars painted with the said material would be more than that of the epoxy painted reinforcing bars for the rust prevention of the steel material when the said material is applied and spread on to steel surface.

Excellent adhesion strength

The adhesive pulling out test also have been performed by the official body, for the reinforcing bars painted with Mange line and for the reinforcing bars without paint.

Result shows, the reinforcing bars painted with Magne Line has approximately 1.4 times larger adhesive strength comparing to bare reinforcing bars.

Therefore, it could be expected that the reinforcing bar painted with this material has more than or the equal adhesion strength to bare reinforcing bar.

Mechanism of rust prevention

The epoxy painted reinforcing bar secures the effect of rust prevention by intercepting salinity. Therefore, there is no effect of rust prevention, once salinity invades from the pinhole etc. In addition, once rust would be generated, it is considered that behavior to pull the epoxy painting apart from the inside of the paint will occur.

On the other hand, as for the mechanism of the rust prevention of reinforcing bar painted with Magne-Line, the stable rust formed on the surface of the reinforcing bar secures rust prevention effect. Therefore, if salinity invades from the pinhole and rust is generated, it is less likely to extend its surrounding part. Thus, it can be said that reinforcing bar painted with Magne-Line has grater performance against the unanticipated scratch comparing than the epoxy painted reinforcing bar.

It is therefore considered that the durability of a main girder can be secured by applying the counter measures of 1), 2) & 3) as stated above

(3) ***Regarding execution management***

It is necessary to provide the construction management standard, since the said material shall be applied at the site. For this purpose, the quality standard of the epoxy painting reinforcing bars specified by Japan Society of Civil Engineers shall be adopted.

And that can be defined by "The number of pinholes in 1 m shall be no more than 5 for D19 o1 smaller diameter, and no more than 8 for D22 or bigger diameter." The standard for D19 refers to D20.

< Reference literature >

"Concrete and reinforced concrete structures – Requirements of Protection from Corrosion in Marine Environment" TCXDVN 327:2004.

"Preliminarily design & construction guideline for the reinforced concrete which utilize epoxy resin painting " Japan Society of Civil Engineers: November, 2003

1.5.5. Steel Pipe

Two types of steel pipe Grade SKK400 and Grade SKK 490 based on the Japanese Standard JIS 5525 or equivalent international standard shall be used. The properties and strength are as shown below.

Table 41 Properties and Stress Limit of Steel Pipe

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)
Grade SKK 400	235	400	200,000
Grade SKK 490	315	490	200,000

Appendix 4: List of Construction Equipment

For reference, list of construction equipment for the smooth implementation of the construction works are as follows but not limited to. Number, capacity and specifications of each equipment will be optimized when the scale of contract package is finalized.

1. General

Air Compressor
Bar Bending Machine
Cargo Truck
Center Hole Jack
Forklift
Four Wheel Drive Car (Jeep)
Fuel Truck
Generator
High Pressure Pump
Lighting Tower
Material Testing Laboratory
Submergible Pump
Truck Crane
Vehicle
Water Tanker
Welding Generator
Welding Machine

2. Road Works

Backhoe
Bulldozer
Dump Truck
Motor Grader
Wheel Loader
Tired Roller

Appendix-7: Prediction of Impact on Ambient Air Quality and Impact of Noise during Operation Phase

1. Impact on Ambient Air Quality during Operation Phase

1.1. Pollutant emission source intensity of automobile tail gas

a) Pollutant emission source intensity of automobile tail gas

Sources of air pollution during operation phase are mainly from emission of vehicle engines, emission of friction between vehicle tires and road pavement.

The following formula is used to calculate gaseous pollutant (SO2, NOx, CO, and TSP) discharge source intensity of the planned highway[1].

$$Q_t = V_w \cdot \frac{1}{3600} \times \frac{1}{1000} \times \sum_{i=1}^{2} (N_{it} \times E_i)$$

In which, Q_t : discharge intensity of gaseous pollutant (ml/m・s (or mg/m・s))
E_i : air emission coefficient of i type vehicle (g/km・vehicle)
N_{it} : hour traffic volume by i type vehicle (vehicle/h)
V_w : conversion coefficient (ml/g (or mg/g))

Regarding the air emission coefficient E_i, the following formulation is used[1].

$$Ei = a/x + bx + cx^2 + d$$

In which, x is vehicle average hour speed (km/h), and a, b, c, d are regression parameters.

Table 1 shows the results of calculation of E_i in cases of x= 50km/h, 60km/h and 80km/h for small car and big car.

[1] Referred to "道路環境影響評価の技術手法（Technical Handbook for Environmental Impact Assessment of Roads）", 2007 Revision, Japan Highway Environment Research Institute (HERI).

Table 1 Air emission coefficients of vehicles

(Unit: g/km per vehicle unit)

Pollutants	Vehicle size	a	b	c	d	Average speed of vehicle		
						50km/h	60km/h	80km/h
NOx	Small car	-0.902	-0.00578	4.39E-05	0.261	0.0637	0.0572	0.0683
	Big car	-7.12	-0.0895	0.000735	3.93	1.1501	1.0873	1.3850
SPM	Small car	-0.0687	-0.000385	2.87E-06	0.017	0.0036	0.0031	0.0037
	Big car	0.0318	-0.0031	2.27E-05	0.158	0.0604	0.0543	0.0557
CO	Small car	-12.5	-0.0559	0.000448	2.2	0.2750	0.2505	0.4390
	Big car	10.9	-0.0168	0.000115	1.19	0.8555	0.7777	0.7183
SO2	Small car	0.0783	-0.000162	1.31E-06	0.0112	0.0079	0.0075	0.0076
	Big car	0.0411	-0.000699	5.51E-05	0.0424	0.1460	0.1995	0.3396

Source of data: Technical Handbook for Environmental Impact Assessment of Roads, 2007 Revision, HERI

Table 2 Daily traffic volume - TanVu-DinhVu Section

(Unit: vehicle /day)

Year	2015	2020	2030
(a) Bicycle	42,400	64,800	40,500
(b) Motorcycle	65,800	91,200	108,067
(c) Car	3,960	13,540	48,000
(d) Trucks of 2 axles and mini bus with less than 25 seats 4	1,243	2,571	8,107
(e) Truck of more than 3 axles and large bus	1,246	2,920	13,851
(f) Trailer and bus with trailer	71	129	436
Total	114,720	175,160	218,961
Small car [(b) + (c)]	69,760	104,740	156,067
Big car [(d)+(e)+(f)]	2,560	5,620	22,394

Source of data: JICA Preparatory Survey Team, May 2010

Table 3 Daily traffic volume – DinhVu-Cat Hai Section

(Unit: vehicle /day)

Year	2015	2020	2030
(a) Bicycle	26,900	40,300	16,800
(b) Motorcycle	41,533	56,733	44,667
(c) Car	2,500	8,420	19,860
(d) Trucks of 2 axles and mini bus with less than 25 seats 4	214	536	1,350
(e) Truck of more than 3 axles and large bus	789	1,817	5,731
(f) Trailer and bus with trailer	46	79	179
Total	71,982	107,885	88,587
Small car [(b) + (c)]	44,033	65,153	64,527
Big car [d)+(e)+(f)]	1,049	2,432	7,260

Source of data: JICA Preparatory Survey Team, May 2010

Table 4 Tan Vu - Dinh Vu Section, forecasted air pollutant emission

(Unit: g/km ·day)

Year	2015	2020	2030
SO2	927.7	897.8	8,791.5
NOx	7,388.7	12,102.6	41,672.7
CO	21,374.1	30,604.4	84,590.1
TSP	402.3	628.2	1,825.7

Source of data: JICA Preparatory Survey Team, May 2010

Table 5 Dinh Vu – Cat Hai Section, forecasted air pollutant emission

(Unit: g/km ·day)

Year	2015	2020	2030
SO2	502.8	537.2	2,956.1
NOx	4,011.8	6,371.6	14,461.3
CO	13,006.5	18,209.9	33,538.6
TSP	219.7	333.1	643.6

Source of data: JICA Preparatory Survey Team, May 2010