Appendix

DRAFTING STANDARDS

Appendix
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1. General Standard

1.1 General

This drafting standard aims to standardize the drafting of drawings to be prepared for the Detailed Design of Urban Drainage Project in the City of Jakarta.

All the drawings prepared for the detailed design shall be drafted in conformity with this standard in principle.

1.2 Drawing size

The sheet size shall conform to size A-1 and size A-0 specified in JIS P0138 as shown in Fig. 1, but it is allowed to expand the dimension of "b" in this figure if required.

1.3 Line and letter

The line for the drafting shall be used in accordance with the classification of line as shown in Table 1.

The lettering for the drafting shall comply with the standard as shown in Table 2.

1.4 Dimension

The dimension shall be expressed in mm of metric system. For the description of dimension, following care shall be taken.

(i) Unit of dimension ;

- Not to be shown except for the case that the dimension is expressed in the unit other than mm

(ii) Dimension line

- To be drawn in parallel with the direction of dimension.
- To be drawn outside of outline of structure.

(iii) Description of dimension

- To be arranged upward or on left side of dimension line.
- To be arranged 2 mm apart from and in parallel with dimension line.

(iv) Description of particular dimensions

- Description of such particular dimensions as slope, angle, etc. shall comply with the standard as shown in Fig. 2.

1.5 Abbreviation and symbol

(i) Abbreviation

The abbreviation to be used for the drafting shall be as described in Table 3.

(ii) Symbol

The symbol to be used for drafting shall be as shown is Tabl.4 for description of materials.

(iii) Indication of reinforcement bar

The indication of reinforcement bar shall principally follow to Indonesian Concrete Code, P.B.I. 71.

Reinforcement bar shall be indicated by solid line and the cross section of reinforcement bar shall be shown by dots.

(iv) Title block

The title block as shown in Fig. 3 shall be arranged at right and bottom corner of the drawing.

2. Drawing for Structural Design

2.1 Kind of Drawings

The drawings shall be classified into following three kinds.

(i) General plan and layout

Location and vicinity maps and general layout and profile covering whole design stretch are included in this category.

This kind of drawings shall indicate the location of structures and the principal feature of drainage channel improvement work.

Applied scale shall be 1/10,000, 1/5,000 and 1/1,000 for general layout and profile.

(ii) Structural drawings

Plan, profile and cross section for channels and plan, profile and sections for structures are included in this category.

This kind of drawings shall indicate the detailed dimensions of drainage channels and feature and principal dimensions of structures.

Applied scale shall be 1/500, 1/200 and 1/100 for drawings for channels and 1/200, 1/100 and 1/50 for drawings for structures.

(iii) Detail drawings

Drawings for the detail of structures and drawings for bar arrangement are included in this category.

Applied scale shall be 1/20, 1/10 and 1/5 for drawings for structure detail and 1/100, 1/50 and 1/20 for drawings for bar arrangement.

2.2 Arrangement of Drawings

2.2.1 Orientation

Location and vicinity maps shall be oriented with north to the top of drawings.

Drawings showing drainage channel shall be oriented so that the flow of channel water is directed as follows.

- From right side to left side of drawing
- From bottom to top of drawing, or
- From drawer's side to side of drawing

Drawing showing main feature of hydraulic structures except for riparian structures shall be oriented so that the flow direction is situated as follows ;

- From left side to right side of drawing
- From bottom to top drawing, or
- From drawer's side to side of drawing

Drawings other than above mentioned drawings shall be oriented properly in consideration of the consistency with the orientation in relevant maps or drawings.

2.2.2 Arrangement of figures in drawing

In case plural figures are to be shown in one drawing, principal view which shows main feature of structure shall be arranged at top and left corner of the drawing. Secondary views which show the side view or sectional view of the structure shall be arranged at right side of or below the principal view. The views or sections explaining the specific details of structure shall be arranged at right side of or below the principal and secondary views.

Table 1 Standard of Linework

L i n e	U s e s	Thickness
	Reinforcement bar	0.6 mm
	Invisible reinforcement bar	0.6 mm
	Boundary, Outline Visible part of works	0.4 mm
	Originall ground line Earthfill, Backfill Excavation line	0.2 mm
	Dimension line, Contour line	0.1 mm
	Invisible, Hidden part	0.4 mm
	Center line	0.1 mm
	Imaginary line	0.1 mm
	Curtailment of length or distance	0.2 mm
	Original contour line inside structure	0.1 mm

Table 2 Standard of Lettering

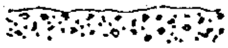
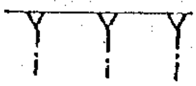
Application	Size	Style
(1) Dimensions and notings	2mm, 3mm	Abcde Fghijk
(2) Main titles	4mm	ABCDE
(3) Subtitle and heading of Note, Reference, etc.	3.5mm	NOTESECT
(4) Title of drawing in title block	3.5mm 4mm 3.5mm	DAM GALLERY SECTION
(5) Drawing No. in title block	3.5mm	A-C-7500
(6) Scale	2mm	SCALE A

Table 3 ABBREVIATION

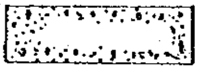
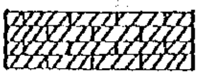
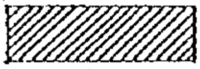
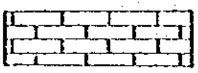
&	and	Fig.	figure
approx.	approximate	g	gram
B.C.	beginning point of curve	H.V.L.	High water level
B.P.	beginning point	I	I-beam
B.M.	bench mark	I.D.	inside diameter
B.R.	bridge	I.A.	intersection angle
C.I.	cast iron	I.P.	intersection point
C.S.	cast steel	kg	kilogram
ctc	center to center	km	kilometer
CL	center line	max.	maximum
cm	centimeter	m	meter
cm/sec	centimeter per second	m/sec	meter per second
conc.	concrete	mm	millimeter
C.J.	construction joint	min.	minimum
Cont.J.	contraction joint	no.	number
C.D.	cross drain	G.L.	ground xxxxx
cm3	cubic centimeter	O.D.	outside diameter
m3	cubic meter	%	percent
C.V.	culvert	PL	plate
C.L.	curve length	P.V.C.	poly vinyl chloride
D	deformed reinforcement xxxx	r	radius
dia.	diameter	reinf.	Reinforced or reinforcement
DWG. or Dwg.	drawing	sec	second
DWGS or Dwgs	drawings	xxx	spaced at
EL.	elevation	cm2	square centimeter
E.P.	ending point	m2	suare meter
E.C.	ending point or curve	Sta.	Station
		T.L.	tangent length
		W.R.	weir

Table 4 Symbols for Materials

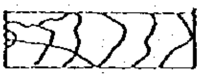
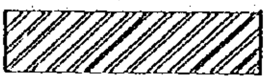
a) Earthwork

	Earth		Sand
	Rock		Sand and gravel
	Gravel		Cut slope
	Embankment slope		

b) Masonry and Concrete

	Concrete		Existing brick
	Existing concrete		Existing stone masonry
	Later stage concrete		Dry stone masonry
	Brick		Wet stone masonry

c) Miscellaneous Materials

	Cast iron		Bronze, brass and copper
	Timber		Cast steel

d) Water

	Water surface

Drawing Size	A1 (mm)	Ao (mm)
a x b	594 x 841	841 x 1188
c	15	15
d	30	30

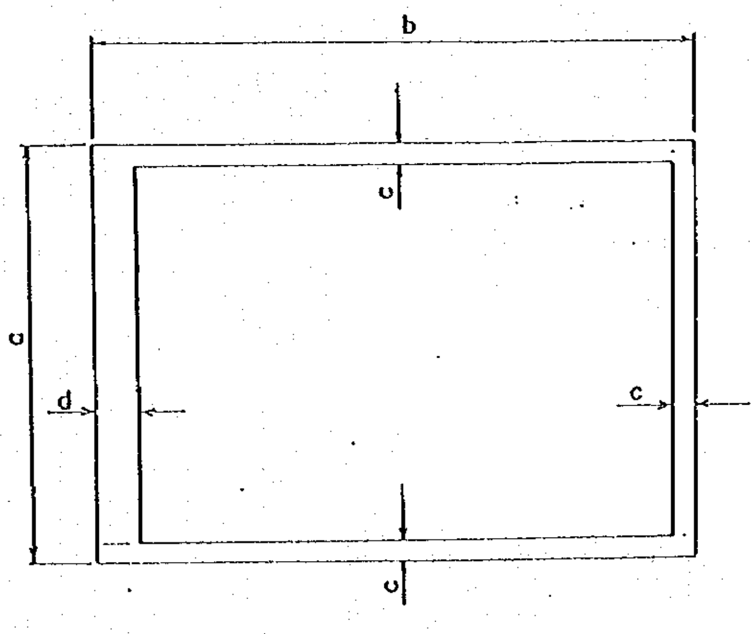


Fig. 1 Sheet size and Side Lines

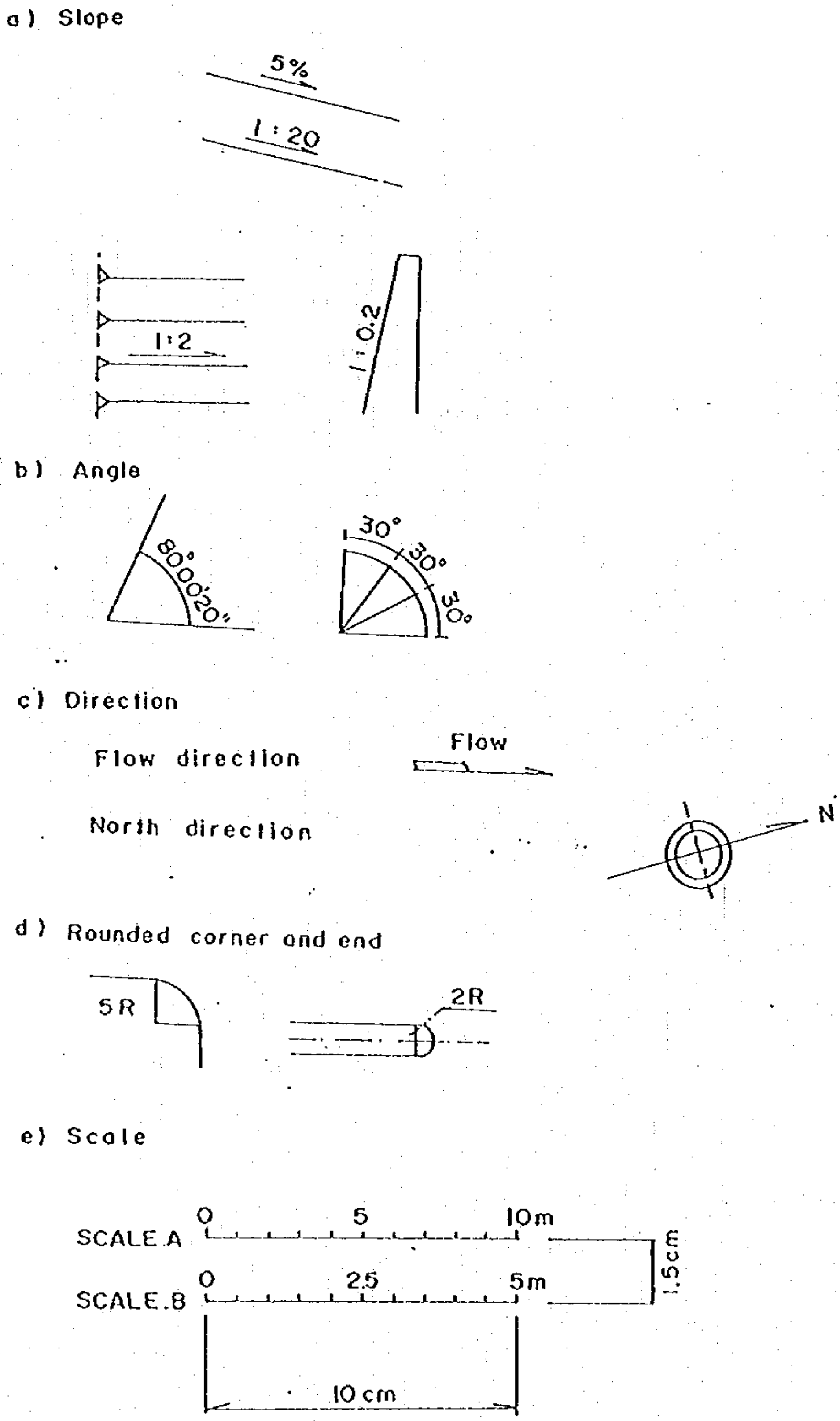


Fig. 2 Standard of Particular Dimensions

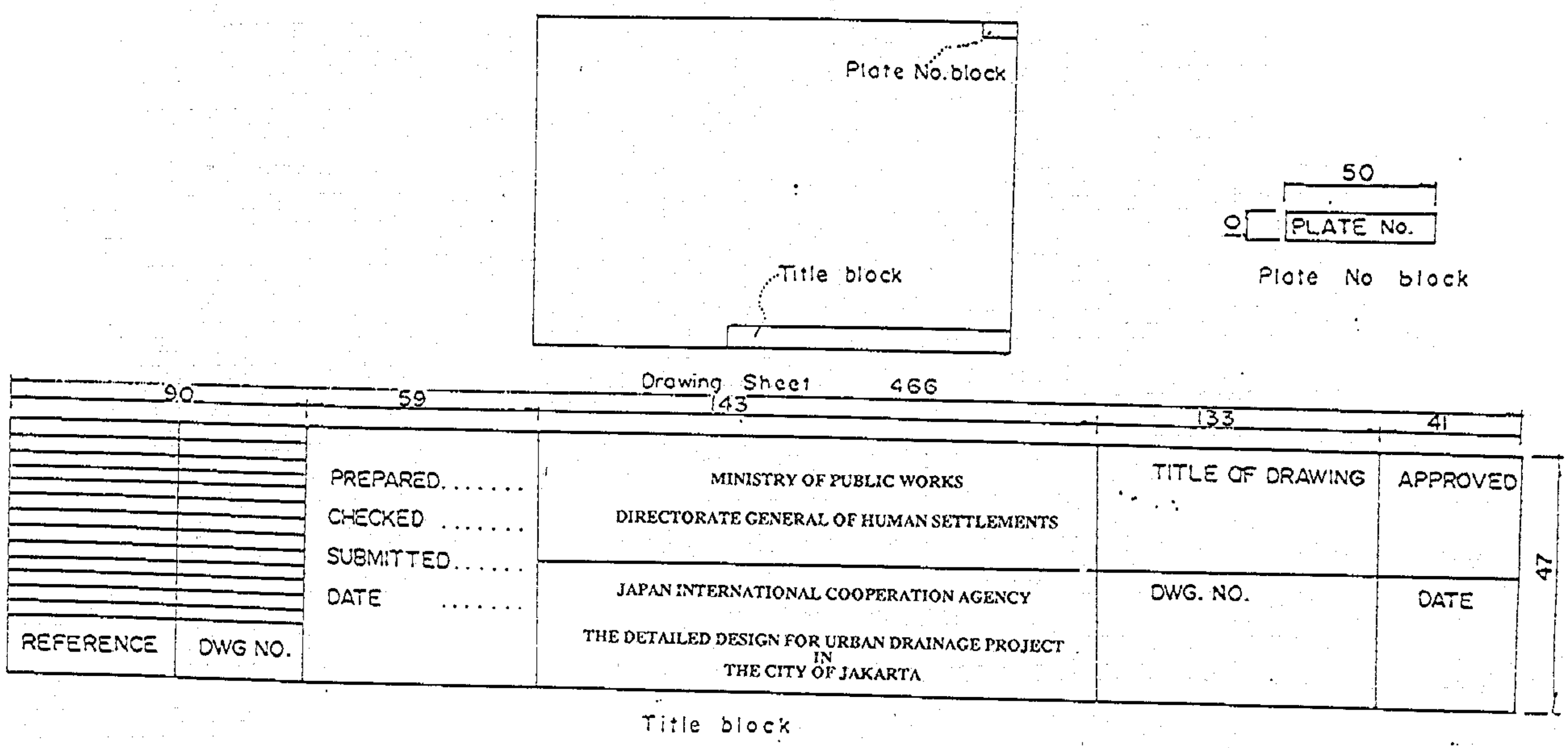


Fig. 3 Standared of Title Block

No. 5

Design and Structural Calculation

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1 GENERAL

This is " Supporting Report : No. 5 Design and Structural Calculation ", which constitutes one of eleven supporting reports for the Detailed Design for Urban Drainage Project in the City of Jakarta.

This supporting report is divided into four chapters, as follows:

Chapter 2	Drainage Channel
Chapter 3	Channel Structures
Chapter 4	Bridge and Culvert
Chapter 5	Gates and Other Metal Works

This report contains the concept of the urban drainage project, of which the objective areas are the Cengkareng West area and the Meruya area locating in the northwestern area of Jakarta, hydraulic calculation for the design of drainage channels in the objective areas, stability analysis on levees, parapet walls and sub-structures of bridges and structural calculation of such channel structures as sluiceways and drain-ditch structures, bridges and gate facilities.

2 DRAINAGE CHANNEL

2.1 Basic Concept for Proposed Urban Drainage Project

The basic concept for the urban drainage project for two objective areas, Cengkareng West area and Meruya area, was established, considering rapid urbanization in the objective areas and anticipating future land use condition. The rapid urbanization is represented by house development and drastic variation of the existing drainage networks due to construction of the new junctions of Jakarta Outer Ring Road. The established basic concept for the proposed urban drainage project is as follows:

(1) A future land use plan in the objective areas was established, referring to the land use plan set out by DKI Jakarta and anticipating future land use condition from present land use progressing situation. The target year for this drainage plan is set at 2010. Comparison of the land use in the present and target year, 2010 is as follows:

(Unit: km^2)

Category of Area	Present Land Use	Land Use in 2010
- Residential area	22.44	27.98
- Industrial area	2.31	3.48
- Paddy field	5.55	2.27
- Fish pond	3.28	2.98
- Swamp/depression area	3.13	0
Total	36.7	36.71

(2) The drainage channels are designed with a flood protection level of a 10-year probable flood for the Cengkareng West area and a 5-year probable flood for the Meruya area.

(3) Drainage water is drained by gravity flow in principle to minimize operation and maintenance cost for the drainage facilities.

(4) In the Cengkareng West area, only primary (main) drainage channels are designed. For the Meruya area, all of the major drainage channels are secondary level. In this design, the major secondary drainage channels are designed.

(5) Special consideration for land subsidence and clean water management was made for planning and designing of the proposed urban drainage project.

(6) In order to flush out the contaminated water in the drainage channels in dry seasons, it is contemplated to release a part of irrigation water to the drainage

channels as maintenance flow. The main and branch channel of the Kamal drainage channel are connected with the existing irrigation canals. The connections of the irrigation canal and the drainage channels are remained in this study for flushing out.

(7) There are many swamp and depression are as in the Cengkareng West area and these areas have been already acquired by the private housing developers/industrial enterprises. The drainage channels in the private owned areas should be constructed by themselves and the constructed drainage facilities will be handed over to DKI Jakarta for routine maintenance works. In the definitive plan, design criteria to be applied to the drainage channels for the private sector areas such as the design discharge, the design channel bed slope and the design channel bed elevation are clarified.

(8) Jl. Tol. Prof. Sediyatmo (the Highway) is aligned in the swamp area in the northern part of the Cengkareng West area and it has suffered in many times from submergence in the rainy season. Cause of this road submergence is attributable mainly to the low elevation of the Highway in about 4 km long stretch in the Cengkareng West area. To cope with this submergence and also to cope with future increase in the traffic volume, it has been planned by JASA MARGA that the Highway will be widened to four lanes and raised up in the stretch between the Kamal drainage crossing site and the Cengkareng flooway crossing site, and bridges crossing the Kamal and Tanjungan drainage channels are designed by JASA MARGA themselves. Thus, in this design, design of the bridges for the Kamal and Tanjungan drainage channels, which cross the Highway, is excluded.

(9) The high tension electric lines and water supply pipe lines are crossing the existing drainage channels. Due to expansion of the drainage channels, treatment of these lines are needed. Besides, it is obliged to shift the existing telephone poles, road signs, traffic signals, etc. due to expansion of the existing drainage channels. Planning, designing and construction of expansion/shifting works of these structures will be made by authorities concerned under the condition that the cost necessary for expansion/shifting works is born by the project.

2.2 Design of Drainage Channels

2.2.1 Basic concept for design of drainage channels

(1) Design stretch

The detailed design was carried out for the proposed Kamal, Tanjungan, PIK Junction, Gede/Bor and Saluran Cengkareng drainage channels in the Cengkareng West area and key drainage channels in the Meruya area. The length of a design stretch in each drainage channel is as shown below:

Drainage Channel	Length (km)
1) Cengkareng West area	18.8(2.8)
- Kamal (main)	7.3(2.8)
- Kamal (branch)	2.8(0)
- Tanjungan	2.5(0)
- PIK Junction	0.8(0)
- Gede/Bor	1.2(0)
- Saluran Cengkareng	4.2(0)
2) Meruya area	2.3(0)
Total	21.1

Note: Figures in brackets show stretches in the areas acquired by private sectors

Out of the above design stretches, those in the areas acquired by the private sectors were examined only from a hydraulic viewpoint to clarify main features of the drainage channels such as the cross sections and channel gradients which will be required as a part of the drainage network to be newly constructed in the Cengkareng west area.

(2) Alignment of drainage channel

The alignments of the drainage channels are designed, based basically on the present channel alignments, in consideration of the following:

(i) The existing drainage channels shall be used as much as possible to minimize compensation cost, so far as the widening of the existing channels is economically feasible from the viewpoint of the compensation of lands, houses and other existing facilities, namely electric poles, water pipes and telephone cable ducts.

(ii) The channel structures such as revetments and bridges which are expected to satisfactorily function in future shall be used as much as possible, if necessary with some modification works.

(3) Longitudinal profile of drainage channel

The longitudinal profiles of the drainage channels are determined based on the following criteria :

(i) The drainage channel bed is designed so as to gradually change from a steep slope in the upstream stretch to a gentle slope in the downstream one. In order to realize a stable channel bed having no detrimental scouring and sedimentation, the ratio of the channel bed gradients between the upstream and downstream stretches to be connected each other shall be smaller than 2.

(ii) The longitudinal profile of the drainage channel shall be determined under the condition that the newly designed channel bed gradient is not remarkably different from the present one.

(4) Design high water level

In principle, the design high water level of the drainage channel should be equal to or lower than the original ground elevation or the ground elevation for the land reclamation area. The design high water level is determined, through non-uniform flow calculation under the following conditions:

1) Boundary water level

(i) Tidal level

The water level of Kamal, Tanjungan and PIK Junction drainage channels, which discharge to the Jakarta Bay, are affected by the tidal level of the Jakarta Bay. The tidal movement in Jakarta Bay at the Tanjun Priok harbor on P.P. system has been analyzed in the Master Plan Study on Drainage and Flood Control in Jakarta (NEDECO, 1973). It has been reported that the tidal movement is a single day with one high tide and low tide in 24 hours. While, the relationship between the datum level of P.P. system and TTG system was clarified in the topographic survey in this study, and it was confirmed that the value of the elevation of P.P. system is 1.003m higher than that of TTG system. A series of the tidal movement established in the Master Plan Study is presented by both P.P. and TTG systems as follows:

Tidal movement	P.P. system	TTG system
- Spring tide(high high water)	P.P.+1.15m	+0.15m
- Average high water	P.P.+0.9m	-0.1m
- Neap tide high water	P.P.+0.8m	-0.2m
- Mean Sea Level(M.S.L)	P.P.+0.6m	-0.4m
- Neap tide low water	P.P.+0.4m	-0.6m

-	Average low water	P.P.+0.25m	-0.75m
-	Spring tide(low low water)	P.P.+0.00m	-1.00m

The Feasibility Study on the Jakarta Urban Drainage Project (JICA, 1991) employed the spring tide (high high water) of P.P+1.15m (round up at P.P.+1.20m) as the design tide level for the Kamal and Tanjungan drainage channels.

This detailed design determined to employ the same design tide level that had been employed in the Feasibility Study for the design of the Kamal and Tanjungan drainage channels.

(ii) Water level of the Cengkareng Floodway

The water level of the Saluran Cengkareng drainage channel is affected by the water level of the Cengkareng Floodway.

100-year, 25-year, 2-year flood discharge and water level for each discharge of the Cengkareng Floodway is estimated in Cengkareng Drain System Study (NEDECO, 1981). The design discharge of the Cengkareng Floodway is summarized below:

100-year flood	Q_{100} = 390m³/s (design flood)
25-year flood	Q_{25} = 280m³/s
10-year flood	Q_{10} = 250m³/s (estimated by the Study Team)
2-year flood	Q_2 = 150m³/s

The water level for the respective flood discharge at the estuary of the Saluran Cengkareng located at 4.3km upstream from the estuary of the Cengkareng Floodway is estimated as follows:

100-year flood	1.87 TTG.m
25-year flood	1.21 TTG.m
10-year flood	1.13 TTG.m
2-year flood	0.62 TTG.m

In this study, the initial water level for the Saluran Cengkareng drainage channel is determined for the 25-year flood water level of the Cengkareng Floodway.

(iii) Water level of the Mookervaat Canal

The Gede/Bor drainage channel flows into the Mookervaat Canal at 3.3 km upstream from the confluence with the Cengkareng Floodway. The Mookervaar Canal itself has designed for 25-year flood. But the water level of the Mookervaat Canal is affected by the back water of the 100-year flood of the Cenkareng Floodway which has designed for 100-year flood. The design water level of the outlet of the Gede/Bor drainage channel are set 2.5 TTG.m considering the 100-year flood water level of the Cengkareng Floodway.

(iv) Tributary of Angke river

Since the Meruya drainage channel flows into the tributary of the Angke river, the design water level of the Meruya drainage channel is affected by the water level of the tributary of the Angke River and also by the main stream of the Angke river.

The Study Team estimated the water level of the tributary of the Angke river at the outlet of the Meruya drainage channel for 25-year flood.

2) The roughness coefficient

The roughness coefficient is 0.03 in a drainage channel of which the channel slopes will be protected on both sides by masonry revetment and 0.016 for concrete structures.

2.2.2 Design flood discharge distribution

The design flood discharge prepared in the feasibility stage (1991) was reviewed by incorporating the data obtained in this design stage. Procedures of preparation of the design flood discharge distribution diagram are as follows:

(1) Division of drainage area

The drainage area for the proposed drainage channels in the Cengkareng West area and Meruya area was divided into sub-basins based on the topographic maps with a scale of 1:5,000 which was prepared in the JABOTABEK Study(1995) considering the present drainage network conditions. The division of drainage basin for the Cengkareng West area and Meruya area are shown in Figs. 2.1 and 2.2. Length of the drainage and the drainage area of sub-basins are shown in Tables 2.1 and 2.2 respectively.

(2) Calculation method

It has been determined to apply a 10-year probable flood for the design of the Cengkareng West area and a 5-year probable flood for the Meruya area. The flood peak discharges of the objective urban drainage channels were calculated by applying the following rational formula:

$$Qp = \frac{1}{3.6} \cdot f \cdot ra \cdot A$$

Where,

Qp	:	Flood peak run-off (m^3/s)
f	:	Run-off coefficient
ra	:	Basin average rainfall intensity during flood concentration time (mm/hr)
A	:	Catchment area (km^2)

(i) Run-off coefficient (f)

The flood run-off coefficient varies according to the land use patterns of the basin. The run-off coefficients for the respective land use categories are as follows:

- Residential Area	0.5
- Commercial Area	0.7
- Industrial Area	0.7
- Paddy Field	0.2
- Fish Pond	0.2

The flood run-off coefficients for the respective drainage basins are calculated based on the land use pattern in 2010 in the drainage area. The value of the run-off coefficients for each drainage basin and sub-basins are shown in Table 2.3.

(ii) Basin rainfall intensity

Point rainfall intensity-duration relationship was studied in ANNEX-I, No 1, Meteorology and Hydrology and point rainfall intensity- duration curves are prepared, as shown in Fig 2.3. In this figure, the basin rainfall intensity (ra) was obtained by multiplying the point rainfall intensity by the rainfall reduction factor as estimated in No. 1, Meteorology and Hydrology, ANNEX-I.

(iii) Concentration time (tc)

The concentration time (tc) of a flood discharge consists of an overland time(t_1) and a drain time (t_2) as follows:

$$tc = t_1 + t_2 \quad \text{(min.)}$$

The overland time(t_1) is a flood concentration time along the longest time route from the base point to the uppermost point of the objective urban drainage channel. The overland time was estimated by dividing the route length by a flow velocity, assuming that :

$$\text{Route length} = \frac{\text{Catchment Area}}{\text{Length of Objective Drainage Channel}}$$

Flow velocity = 0.4 m/s.

The drain time(t_2) is a flood concentration time to the objective urban drainage channel. In the drain time calculation, a flow velocity is assumed as follows.

Velocity(m/sec)	Channel gradient
0.5	1:5,000<I < 1:1,000
0.4	I < 1:1,000

(3) Estimation of flood peak discharge and preparation of flood discharge distribution

The flood peak discharges at several points of the proposed drainage channels were estimated by using the rational formula. Based on the flood peak discharge, specific discharge- catchment area relationship was studied as shown in Fig 2.4. The figure shows that the specific discharge- catchment area relationship is divided into two groups, depending on the shape of the drainage area. Thus, the flood peak discharges at the selected points for respective drainage channels were estimated based on the width and length of the drainage area. The flood peak discharge at the selected points is listed in Table 2.3. And the flood discharge distribution based on this estimation for the Cengkareng west area (10-year probable flood) and Meruya area(5-year probable flood) are given in Figs 2.5 and 2.6.

2.2.3 Alignment of drainage channel

In accordance with the above basic concept, the drainage channel alignments and basic drainage plan were determined as follows and these are illustrated in Fig 2.7 for the Cengkareng West area and Fig 2.8 for the Meruya area.

(1) Cengkareng West area

1) Kamal drainage channel

The drainage channels in the depression areas and swampy areas which had been acquired by the private sectors have already been constructed, and it is presumed that the drainage channels in the remaining depression and swampy areas, which have been acquired by private sectors, will be constructed by private sectors themselves by reclaiming the lands up to EL. 4~6 m, under the condition that the drainage water is discharged by gravity flow. To drain the water from the upstream drainage areas to Jakarta bay, the drainage channel is aligned by expanding the existing drainage channel and reconstructing a bridge at Jl. Tol. Prof. Sediyatmo (the Highway). The gate facilities will be provided along the Kamal drainage channel in locally low land areas in southern part of the Highway.

2) Tanjungan drainage channel

The drainage channel in the depression area in the center part of the drainage area, which has been acquired by the private sector, will be constructed by a private sector under the condition that the depression area is reclaimed up to EL.2 m and the drainage water is drained by gravity flow. A low land is located at the southern part of the Highway. In order to drain water in this low land by gravity flow and to cope with land subsidence, it is necessary to heighten this low land up to EL. 2 m. The drainage water in this low land will be discharged through sluiceways until completion of land reclamation in this low land. In order to drain water from the upstream depression areas and other drainage area to Jakarta bay, the Tanjungan drainage channel is aligned straightway to Jakarta bay by providing a bridge at the Highway and constructing the drainage channel in the fish pond.

3) PIK Junction drainage channel

The drainage condition in this area was drastically changed from that stated in the feasibility study (1991): drainage in this area is interrupted by a newly constructed junction road (Penjaringan Junction) which divides the drainage area into two zones. The drainage channel in the depression area, which is located along the Jakarta Outer Ring Road and already acquired by a private sector, will be constructed in the land to be reclaimed up to EL 2 m. Since the junction road network crosses the proposed drainage channel, the proposed drainage channel will be aligned by avoiding this junction road. The northern part of this drainage area is acquired by a private housing developer. The drainage water collected along the Highway is planned to be discharged to Jakarta bay through the existing 6 culverts with diameters of 0.80m~1m under the condition that the land already acquired by the private sector is heightened up to EL. 1~2 m.

4) Gede/Bor drainage channels

Since upstream stretch of the Gede/Bor drainage channel was already constructed by a private housing developer, the remaining stretches of the Gede/Bor drainage channel will be widened to discharge the design flood to the Mookervaat canal. The Gede/Bor drainage channel is designed by gravity flow without any gate facilities.

5) Saluran Cengkareng drainage channel

The Saluran Cengkareng drainage channel is connected with the Gede/Bor drainage channel at its upstream end and drainage water flowing in the Gede/Bor drainage channel is discharged to two directions at the connection point; namely the southern direction to the Mookervaat canal flowing down the Gede/Bor drainage channel and the eastern direction to the Saluran Cengkareng drainage channel. In order to minimize the width of the Saluran Cengkareng drainage channel in the downstream stretch in the densely populated area, the Saluran Cengkareng drainage channel is separated from the Gede/Bor drainage channel. However, at the upstream end of the Saluran Cengkareng drainage channel, sluice gate facilities are provided to flow the water from the Gede/Bor drainage channel into the Saluran Cengkaren drainage channel as maintenance flow in dry seasons. The Saluran Cengkareng drainage channel is designed by gravity flow without any pumping facilities. To prevent further expansion of inundation in the locally low land area along the drainage channel in case a large magnitude flood occurs in the Cengkareng Floodway and the flood flows into the Saluran Cengkareng drainage channel, sluice gate facilities will be provided at the outlet of the Saluran Cengkareng drainage channel by modification of the existing gate facilities. Both banks of the present Saluran Cengkaren drainage channel along the low land area 1.5 km upstream of the outlet will be heightened to prevent drainage water from flowing into the low land area and sluiceways will be provided.

6) Pedongkelan drainage channel

In the center part of this drainage area, a large scale house complex is located and drainage channels surrounding this house complex and extending to the eastern direction to drain water to the Cengkareng Floodway are being constructed by the National Urban Development Corporation (PERUM PERUMNAS) at present. It is planned in this drainage channel to provide a regulation pond and pump facilities to discharge drainage water to the Cengkareng Floodway. Since construction of this drainage channel belongs to the National Urban Development Corporation, design of this drainage channel was deleted.

(2) Meruya area

Since the ground elevation of Meruya area is lower than the surrounding area topographically, the habitual inundation areas are formed in the center part of the drainage area. A part of the drainage water from Meruya area flows into the existing drainage ditch located in the private housing complex in the eastwards of Meruya area. Since the capacity of the existing drainage ditch in this private owned housing area is limited, the drainage water caused by the future development of Meruya area could not be drained to eastern direction. In addition, due to the construction of the new junction of Jl. Tol Jakarta-Merak (Kebon Jeruk Junction) in the northeast part of the project area, the drainage to the existing culverts under the embankment of Jl. Tol Jakarta-Merak will be interrupted. Because of these reasons mentioned above, it is considered that the only outlet of the drainage from Meruya area is a tributary of Angke river located at about 480m West from the drainage area. The new drainage channel is aligned across the inundation areas in upstream and middle stream stretch, along Jl Tol Jakarta-Merak in downstream stretch, and connected to the tributary of the Angke river.

2.2.4 Longitudinal profile of drainage channels

The design longitudinal profiles determined in accordance with following concepts are given in Fig.2.9.

(1) Cengkareng West area

1) Drainage channel to be drained to Jakarta bay

In order to drain the discharge as much as possible within the limited channel width, a steeper channel bed slope would be applied generally. But the water level of the Kamal, the Tanjungan and the PIK Junction drainage channels discharged to the Jakarta Bay could not be lowered by means of the selection of steeper slope especially in the downstream stretches, since the water level is affected by the tidal level. In addition, it is anticipated that the deepened channel bed alters to the original channel bed due to sea sand and sedimentation from upstream side. In view of the above, the design channel bed slope are determined same as the present channel bed slope. The determined channel bed slope is as follows:

- Kamal drainage channel : 1/3,200 and 1/1,800
- Tanjungan drainage channel : 1/5,000

- PIK Junction drainage channel: 1/600

2) Drainage channel to be drained to Cengkareng floodway

The Saluran Cengkareng drainage channel discharges to the Cengkareng Flooway. The Saluran Cengkareng drainage channel is affected in its downstream stretch by the water level of the Cengkareng Floodway. Even if the channel bed at outlet portion of the Saluran Cengkareng drainage channel is lowered, the design water level could not be lowered. Thus, the design channel bed slope is at the original channel bed slope. The determined channel bed slope is 1/3,000.

3) Drainage channel to be drained to Mookervaat canal

The Gede/Bor drainage channel discharges to the Mookervaat canal. Since the water level of the Gede/Bor drainage channel is affected by the water level of the Mookervaat canal, the design channel bed slope is set same with the present channel bed slope. The determined channel bed slope is 1/1,600.

4) Verification of safety in design water level

In connection with the design water level, check study was made from two aspects, namely, fluctuation of the tidal level in Jakarta bay and occurrence of larger floods exceeding the design flood.

In this design, the spring tide(high high tide) of P.P 1.2m in Jakarta bay is applied for determination of the design water levels of the Kamal and the Tanjungan drainage channels, while the maximum tide level of P.P 1.54m has been applied for the design of East Jakarta Flood Control Project. To verify the safety of the drainage channel in case of tide level occurrence of P.P 1.54m, check study was made. The result of the study verified that both drainage channels are able to discharge their design floods within the freeboard, receptively, even when the tide level of P.P 1.54m occurs in the Jakarta bay.

The check study was also made to verify the safety of the drainage channels in case that a 25-year flood takes place in the drainage channels in the Cengkareng west area. The study resulted in the verification that even if a 25-year flood occurs in the Cengkareng west area, it can be safely drained within the designed drainage channels due to the freeboard. Fig 2.10 exemplifies comparison of flood water levels for 10 and 25-year floods and the freeboards.

(2) Meruya area

In order to drain water from the drainage area by gravity flow to the tributary of the Angke river, following conditions were contemplated:

(i) In the partly high elevated areas in the drainage area, box culverts are designed under the existing roads to drain water to the westward of the drainage area.

(ii) All of the objective drainage area was already acquired by residents and provision of a new drainage channel in this acquired area is rather difficult because of compensation problem with residents. To avoid this compensation problem, the proposed drainage channel is to be aligned along the existing roads.

(iii) The proposed drainage channel is to have alignment crossing the city road along the west fringe of the drainage area, so as not to cross a pier portion of the road bridge across Jakarta-Merak highway.

(iv) Foundation of the existing houses located along the proposed drainage channel in the center part area has been heightened by 0.5~0.7 m to avoid inundation. Since the ground elevation of the center part area is about 2m lower than those of neighboring areas, the design water level corresponding to a 5-year flood becomes about 0.5m higher than the foundation of the existing houses. Under such drainage conditions, it was contemplated to proceed with the drainage works by two stages. In the initial stage, a drainage channel with the same design water level as the foundation of the existing houses is designed considering convenience of inhabitants, on condition that inundation is allowed at a part of the center part area against a 5-year probable flood. In the final stage when substantial land reclamation will be carried out, the drainage channel will be heightened to discharge a 5-year probable flood.

Comparative study on the relationship among channel width, ground elevation along the drainage channel and channel bed slope was made. Among several alternatives, drainage plan with the lowest design water level was selected. The determined channel bed slope is 1:2,000, 1:260 and 1:700.

2.2.5 Cross section of drainage channels

A single cross section is applied for all of the drainage channels. The cross section was designed considering the following conditions:

a) The design water level should be lower than the ground elevation of the drainage area in principle,

b) In the drainage area of the Tanjungan drainage channel, it is necessary to heighten the low land in the southern part of Jl. Tol. Prof. Sediyatmo up to EL 2 m to discharge water by gravity flow and to cope with land subsidence.
c) Compensation for lands and houses due to widening the drainage channel should be minimized.
d) The freeboard as specified in the design criteria is applied.

Comparative study on alternatives of the channel cross sections was made for respective drainage channels as shown in Table 2.4 by means of non-uniform flow calculation. Among several alternatives, the cross section with the maximum channel depth to satisfy the above condition and the minimum width was selected for each channel stretch, except for locally low land areas. The proposed design water level, channel width, channel bed slope of each drainage channel are listed in Table 2.5. The proposed general plan of drainage channels are shown in Fig.2.11 together with the principal dimensions of the standard cross sections.

2.2.6 Capacity check of other drainage channels

In the private sector areas, 5 major drainage channels have been constructed by the private sectors themselves. On the other hand, one major secondary irrigation canal, which changes its function to the drainage channel in the downstream stretch after feeding irrigation water, joins the Kamal main drainage channel. The 6 major channels/canal, of which the locations are shown in Fig. 2.12, were examined in terms of flood capacity, because the channels/canal are expected to function as a part of the drainage channel network in the Cenkareng West area.

The examination revealed that the capacities are insufficient for the 10 year flood, except for the channel GA. Accordingly, it is recommended that the channels will be improved with the following channel features:

Channel name	Design discharge (m3/s)	channel slope	channel width (m)	side slope	Remarks
KM-C	30.3	1/1,800	12	1:0.5	
KC-C	27.7	1/1,800	20	1:0.5	
KH-C	16.9	1/250~ 1/1,800	15	1:0.5	
TM-C	9.6	1/5,000	10	1:0.5	
GM-C	13.8	1/1,600	7	1:0.5	

2.2.7 Measures against land subsidence for Kamal, Tanjungan and Saluran Cengkareng drainage channels

The Kamal, Tanjungan and Saluran Cengkareng drainage channels are designed by gravity flow. However, it is anticipated that the design water levels of these drainage channels will become higher than the elevations of the riparian areas along the drainage channels after the target year, 2010, if all the riparian areas along the drainage channels are settled down at a rate of 6cm/year and the tidal level in Jakarta bay is unchanged. To cope with this land subsidence problem in the future stage, a pump drainage plan including a regulation pond was studied at a preliminary level. The study results in the following table:

Drainage channel	Installation year	Pump capacity (m^3/s)	Regulation pond	
			Scale (length x width)	Storage capacity (m^3)
Kamal (main)	2018	20	400m x 400m	480
Tanjungan	2016	5	200m x 200m	80
Saluran Cengkareng	2018	3	270m x 270m	150

As seen above, pump drainage is required in and after 2016, for the Kamal (main),Tanjungan and Saluran Cengkaren drainage channels. It is conceived that pump facilities and a regulation pond shall be located near the downstream end of each channel, in consideration of land availability, drainage efficiency and possibility in future basin development. Location of regulation ponds in after 2016 is shown in Fig.2.13.

3 CHANNEL STRUCTURES

3.1 Design of Levee

(1) General

It is required to construct new levees along the drainage channels in some low land areas. An earth type levee is adopted, if there is no special restriction in land for levee construction. However, a parapet wall is adopted if there is constraint in land acquisition in a densely populated area. Design criteria of the earth type levee are as stated below.

(i) Clayey and silty materials which will be obtained from excavation for drainage channel widening and in near borrow areas shall be used as levee embankment material. Accordingly, a homogeneous type levee is designed.

(ii) The earth type levee shall be of a trapezoid shape with 3 m in crest width and its side slopes shall be 1: 2.0 in principle. However, the crest width is to be 5 m, in case that the crest is used for an inspection road of the drainage channel because no existing road will be available for the inspection road.

(iii) A two(2) m wide berm shall be provided 3 m below the levee crest on the land side slope.

(iv) The foundation area of the levee shall be stripped up to 20 cm in principle.

(v) Extra embankment is necessary for levee construction to cope with future settlement of levee body and foundation, and the following values shall be applied:

Height of levee(m)	Extra embankment(cm)
Less than 3	20
3 to 5	30

(vi) The slopes of the earth type levee are protected by gabion mattress and sod facing on the channel and land sides, respectively, against local erosion due to rainfall and stream flow.

(vii) The levees along the Tanjungan drainage channel in the north part of Jl. Tol. Prof. Sediyatmo are provided by dump fill of earth materials, because the channels are aligned in fish ponds. The side slopes of the levees are 1:3.0 and are protected with rip-rap only on the channel side.

The typical cross section of the earth type levee is shown in Fig. 3.1.

(2) Location

The earth type levee is adopted for the channel stretches in rarely populated areas along the Kamal, the Tanjungan and the Saluran Cengkareng drainage channels. The location and length of each levee are presented in Table 3.1 and Fig. 3.2. The total lengths are summarized below.

Drainage channel	Levee length (m)			Remarks
	Left bank	Right bank	Total	
Kamal (main)	1,893	2,770	4,663	
Tanjungan	1,737	1,743	3,480	
	(1,454)	(1,663)	(3,117)	( )dump fill
Saluran Cengkareng	1,912	1,537	3,449	

(3) Stability analysis

(a) Properties of embankment materials

Findings on the properties of available levee embankment materials on the basis of the geotechnical investigation are summarized below.

- Most of the soil distributed along the drainage channels is classified into CH/OH by the ASTM Unified Soil Classification.
- The compaction test results showed 18 % OMC for the silty material and 30 % OMC for clay material at maximum dry densities of 1.575 t/m3 and 1.350 t/m3, respectively. In both cases, the optimum water contents are much lower than the natural water contents, meaning that when used as embankment material such soils have to be dried.

(b) Stability analysis

Stability analysis of the levees against sliding failure is carried out to determine dimensions of the levees.

The long term stability of the levees is herein analyzed by the total stress method, and therefore, the following physical and mechanical material properties are employed for the analysis, duly referring to the results of the geotechnical investigation.

Materials	Cohesion (t/m^2)	Friction Angle (Degree)	Unit Weight (natural) (t/m^3)	Unit Weight (saturated) (t/m^3)
Embank. Material	4	27	1.8	1.8
Foundation	4	8	1.7	1.7

The analysis was made under the following conditions:

- Levee dimension
  - Side slope : 1 : 2.0
  - Width of crest : 3.0 m
  - Height : 2 m
- Loading condition :
  - Case 1 : Loading under the design high water level (land side)
  - Case 2 : Loading under draw down of water level. (channel side)

The minimum safety factor which has to be ensured in both cases is determined at 1.5 for which a slightly larger allowance is given in consideration of uncertainties involved in the poor quality of levee foundation and available embankment materials.

The stability analyses clarify that the side slope of 1 : 2.0 meets the requirements, as shown in Fig. 3.3. The Figure results from analysis under the following condition of water level, which covers water level conditions for both cases above:

1) in levee body : 0.4 m below levee crest
2) in land area : equal to land surface
3) in channel area : equal to channel bed

3.2 Design of Parapet Wall

(1) General

The parapet wall is designed in the following stretches having difficulty in construction of an earth type levee due to constraint in land acquisition:

1) stretches facing difficulty in land acquisition in densely populated areas, and
2) stretches having no space for earth type levee construction because of narrow space between channel bank and existing road.

The parapet wall is an inverted T-type of reinforced concrete. The height of the wall is designed based on the design water level and necessary freeboard above the design water level. Other dimensions such as the width and thickness of the wall and footing take into consideration the loading conditions after heightening the wall due to future land subsidence.

In the upstream stretch of the Tanjungan drainage channel, a concrete L-type wall is employed, in consideration of (i) needlessness in construction of masonry revetment (ii) shortening of foundation pile length and (iii) future land reclamation in the inland area.

(2) Location

The concrete parapet wall was applied to the channel stretches in the densely populated areas in 3 drainage channels. The location and length of each parapet wall are presented in Table 3.1 and Fig. 3.2. The total lengths are summarized below.

Drainage channel	Parapet wall length (m)			Remarks
	Left bank	Right bank	Total	
Kamal (main)	493	0	493	
Tanjungan	567	567	1,134	L-type wall
Saluran Cengkareng	569	851	1,420	

(3) structural design

(a) Wall

Classification

The parapet walls are classified into the following three (3) types by their heights and types, which are determined in accordance with requirement at each site.

Unit : m

Wall type ∠1	Height	Remarks
Pw 1	1.5 (2.5) ∠2	
Pw 2	2.0 (3.0)	
Pw 3	1.85 (2.85)	L-type

∠1 Pw-1 and Pw-2 walls are an inverted T-type.
∠2 Figures in parentheses show future anticipated wall heights.

Stability and stress analyses

i) Loading condition

The parapet walls are designed under the future loading condition until the target year of 2010.

ii) Design loads and allowable stress

The design loads are composed of dead load, live load, earth pressure, hydrostatic pressure, uplift and seismic force. Details of the design loads and allowable stress to be applied to the structural design of the parapet wall are as referred to in Supporting Report No. 4.

iii) Stability analysis

The stability on the parapet walls was examined for the following three items :

- Overturning
- Sliding
- Bearing pressure

iv) Stress analysis

The stress analysis on the vertical walls and bottom slabs is made as follows.

Vertical wall :

each vertical wall is designed as a cantilever fixed at the lowest section and earth pressure, live load and hydrostatic pressure are considered as design loads.

Bottom slab :

each bottom slab is designed also as a cantilever fixed at the lowest portion of the wall and reaction of foundation, uplift, live load and such dead loads as concrete and back filling materials are considered as the loads.

Results of the stability analysis on design of the parapet walls are presented in Table 3.2.

(b) Foundation type

(i) Criteria

Two (2) types of spread and pile foundations are conceivable for the parapet walls and the following criteria are applied for selection:

Type	Criteria
Spread foundation	$Qmax \leq Qa$
Pile foundation	$Qmax > Qa$

where, Q_{max}: maximum bearing pressure(t/m^2) under full design load condition(t/m^2)
Qa: allowable bearing capacity of foundation (t/m^2)

(ii) Allowable bearing capacity

The allowable bearing capacity of foundation is estimated at 3.5 t/m^2, based on the geological conditions at the proposed parapet wall sites, where soil properties are clarified through the geotechnical investigation of this Project, by the Terzaghi's formula as expressed below.

$$Qa = (\alpha \cdot C \cdot Nc \quad + \beta \cdot r \cdot B \cdot Nr + r \cdot Df \cdot Nq) / Fs$$

where, Qa	:	allowable bearing capacity (t/m^2)
C	:	cohesion (t/m^2)
α,β	:	shape factors of foundation
r	:	unit weight of ground materials (t/m^2)
B	:	minimum width of loaded surface area (m)
Df	:	foundation depth from ground surface (m)
Fs	:	safety factor (=2)
Nc,Nr,Nq	:	bearing cpacity factors

The shape factors of α and β depending on the shape of each parapet wall footing are 1.0 and 0.5 respectively as the continuous footing is provided.

(iii) Foundation type.

The examination through the stability analysis on the parapet walls revealed that the maximum bearing pressure of each parapet wall is higher than the allowable bearing capacity of foundation. Accordingly, the footing concrete is provided with friction piles of 250 mm x 250 mm, as follow:

Wall type	R.C. pile	
	Length (m)	Interval (m)
Pw1	6.0	1.1 x 2.0
Pw2	6.0	1.5 x 1.5
Pw3	6.0	1.8 x 1.25

(c) Cutoff wall of steel sheet pile

A cutoff wall is required at the foundation of ech parapet wall to prevent piping phenomena due to seepage water in the foundation during flood. The required depth of the cutoff wall is calculated by applying the Lane's formula, which is presented below:

$$Cw \leq (\sum Lh/3 + \sum Lv)/\Delta H$$

where,	Cw	:	Lane's creep ratio (clay, Cw=3 to 5)
	Lh	:	length of horizontal creep line (m)
	Lv	:	length of vertical creep line (m)
	ΔH	:	difference in water level between channel side and land side (m)

The examination is carried out under the load conditions until the targety year of 2010 in consideration of occurrence in future land subsidence. The length of steel sheet piles required to protect piping phenomena is summarized below.

Wall type	Sheet pile length (m)
Pw1	3.5
Pw2	4.0
Pw3	3.0

The typical cross section of the concrete parapet is given in Fig. 3.3.

3.3 Design of Revetment

(1) General

For slope protection of the drainage channels and earth type levees, revetments of wet masonry are employed in this design.

In case of the drainage channels, their bank slopes will be protected in the whole design stretches, excluding those provided with concrete ditches/walls and earth type levees. In case of the earth type levee, the slope shall be protected at :

1) the connection part with the parapet wall, to be protected from erosion caused by turbulent flow due to change of channel cross section and roughness coefficient, and
2) the upstream and downstream areas of bridge and sluiceway construction sites.

(2) Structural design

The revetment of wet masonry consists of slope protection, toe protection and foot protection works. Their structural design conforms to the following criteria.

(i) Slope protection

The slope protection of wet masonry with a 1:0.5 slope shall be provided at least up to the design high water level. However, the protection shall be raised up to the top of the drainage channel or levee, depending on the local flow condition. Contraction joints with rubber joint filler will be provided at a 6 m interval in longitudinal direction and weep holes will be provided at a rate of one hole per 4 m^2.

A transition of 3 m in length is provided at upstream and downstream ends of the revetment, to avoid abrupt change in roughness and hardness between the wet masonry and levee slope which may cause unfavorable flow turbulence. For this transition, gabion mattress is to be placed.

(ii) Toe protection and foot protection

In order to protect the toe potion of the slope protection from damage due to local scouring or degradation at the channel bed, the toe protection by foot concrete will be provided. The depth of the foot concrete is 0.5 m. The foot concrete shall be provided 1.0 m below the design channel bed in the drainage channel and 0.5 m below the original ground surface for the levee revetment.

In order to avoid destruction of the wet masonry after uneven settlement of foundation due to durability of foundation material and future local subsidence, wooden piles having a 3 m length and a 2 m interval are provided at the foot concrete.

The toe portion of the slope protection will be also protected against scouring and degradation of the channel bed, and gabion mattress will be provided for foot protection in front of the toe protection.

The typical cross section of the wet masonry revetment is given in Fig. 3.4.

(3) Location

The location and length of each revetment are presented in Table 3.1 and Fig. 3.2. The total lengths are summarized below.

Drainage channel	Revetment length (m)			Remarks
	Left bank	Right bank	Total	
Kamal (main)	1,315	1,580	2,895	
Kamal (branch)	1,869	1,474	3,343	
Tanjungan	204	143	347	
Saluran Cengkareng	1,341	1,435	2,776	
Gede/Bor	1,183	1,183	2.366	

3.4 Design of Drainage Facilities

3.4.1 General

The channel improvement works bring about the following negative effects, due to construction of levee, revetment and other channel structures:

(i) to close outlets of existing drainage canals which discharge domestic waste water and inland rain water to the drainage channels, and
(ii) to confine inland rain water in locally low land areas along the drainage channels.

To solve these negative effects, it is necessary to provide drainage facilities such as sluiceway and drain-ditch structures at the existing drainage canals and the locally low land areas. The sluiceways will be equipped with slide or flap gates and at each sluiceway rain water will be discharged to the drainage channel through the gates when the water level in the drainage channel is lower than that in the inland water level at the sluiceway.

Details of the design criteria are presented in Supporting Report No. 4 and the outline is as follow.

(1) Location of sluiceway

The location of each sluiceway is determined based on the following consideration:

(i) The location is, in principle, to be the same as the site of the present drainage canal. However, the location shall be determined so as no to impair the function of the existing and/or new drainage facilities.
(ii) The number of all sluiceways shall be minimized with the concept of minor drainage area integration to realize effective and efficient drainage.
(iii) The longitudinal direction of the sluiceway shall be aligned perpendicular to the axis of the levee to minimize the sluiceway length and to simplify the structure.

(2) Hydraulic design

(i) The sluiceways are designed under the hydraulic condition of the drainage channels having a protection level of a 10-year flood.
(ii) The conduit capacity of each sluiceway is equal to the ten (10) year discharge in the drainage area at the sluiceway under the future land use condition. The probable discharge is estimated by use of the Rational formula.

(3) Structural design

(i) Conduit

1) A box culvert type is applied because of its high drainage capacity.
2) The maximum size of a box culvert type conduit is limited to 2.3 m x 2.3 m at the outlet in consideration of manual gate operation.
3) The number of conduits is determined so that the total areas of the conduit may not reduce remarkably the original flow area of the drainage channel to be connected.
4) In the case that lanes of more than four (4) conduits are required at a larger drainage area, the combination of two lanes and/or three lanes of conduits shall be used because of working efficiency of concrete pouring of a long floor slab of the conduit body.

(ii) A contraction joint shall be provided in the conduit, if its length exceeds 20m, in terms of uneven settlement of foundation. The contraction joint is structurally required to play water tightness performance with somewhat flexibility for allowable displacement. In addition, a joint collor shall be less reinforced compared with the reinforcement arrangement of the box culvert to get smaller rigidity than the culvert body.

(iii) Foundation of the sluiceway is designed so as to safely transmit upper loads to stiff subsoil. To avoid leakage through the foundation, base concrete of 10 cm is provided instead of gravel bedding.

(iv) The outlet sill elevation of the conduit is determined based on the design elevation of the channel bed and topographic conditions of the inland area.

(v) A slide gate or a flap gate is provided with the sluiceway to prevent drainage channel water from inundating in the inland area during floods, in case that the inland surface is lower than the design high water level of the drainage channel at the sluiceway site.

The slide gate, in principle, has the following advantages compared with the flap gate :

1) The flap gate tends to malfunction due to clogging by water driftage and trash.
2) The slide gate has higher reliability of operation performance, although it shall be operated manually.

Accordingly, the gate is designed based on:

1) The slide gate is designed so as to allow manual operation.
2) The flap gate is applied in case that the design conduit discharge is less than 0.2 $m^3/sec/$ and no serious inundation damage is presumed in the inland area even in the case of malfunction of the gate.

(4) Foundation

Two (2) types of spread and pile foundations are conceivable and the following criteria are applied for selection:

Type	Criteria
Spread foundation	$Qmax \leq Qa$
Pile foundation	$Qmax > Qa$

where, Q_{max}: maximum bearing pressure(t/m^2) under full design load condition(t/m^2)
Qa: allowable bearing capacity of foundation (t/m^2)

(5) Cut-off wall of steel sheet pile

The safety of sluiceway foundation is carefully examined against piping by seepage

water around the conduit because of water level difference between the upstream and downstream sides of the sluiceway. The foundation soil property subject to examination is assessed based on the results of the geotechnical investigation conducted in this project.
The examination on the piping is made by using the Lane's formula. If the piping is assessed to occur, steel sheet piles will be provided at the inlet and outlet of the conduit.

3.4.2 Location

Conceivable sites for construction of the drainage facilities were thoroughly examined through extensive site reconnaissance along the drainage channels from topographic, geotechnical and hydraulic viewpoints to minimize the number of the sluiceway/drain-ditch sites and to realize effective drainage.

Consequently, 51 and 13 sites are determined for construction of the sluiceways and drain-ditches respectively, as shown in the following table. The present outlet sites of major drainage canals are included in the sites.

Drainage channel	Left bank		Right bank		Total
	S	D	S	D	
Kamal (main channel)	8 sites	2 sites	7 sites	-	17 sites
Kamal (branch channel)	5 sites	3 sites	3 sites	1 sites	12 sites
Tanjungan	4 sites	-	3 sites	-	7 sites
Gede/Bor	3 sites	3 sites	2 sites	3 sites	11 sites
Saluran Cengkareng	8 sites	-	7 sites	1 sites	16 sites
PIC drainage channel	-	-	1 site	-	1 sites
Total	28 sites	8 sites	23 sites	5 sites	64 sites

Notes: S = Sluiceway, D = Drain-ditch

The locations of the above sites are illustrated in Fig. 3.5.

The drain-ditch structure is employed at the site where a drain-ditch is currently provided and the inland elevation is higher than the design high water level (HWL) of the drainage channel, because the inland area will have no inundation from the drainage channel during floods and gate facilities are not required.

3.4.3 Design flood

A 10 year runoff discharge in a inland area is set as criteria for the design of the sluiceway and drain-ditch structures. The 10 year runoff discharge was calculated by Rational Formula, as in the case of the design discharge calculation of the drainage channels. The formula used in this calculation is as follows:

$$Qp = f \times ra \times A/3.6$$

where,

Qp :peak discharge (m3/s)

f :run-off coefficient (0.5 for residential area, 0.7 for industrial area)

A :drainage area (km2) of sluiceway/drain-ditch site

ra :basin average rainfall intensity during concentration time (mm/h)

$ra = a \times rp = a \times 8571/(t^{1/1.02} + 50.1)$ $(t \leqq 180min)$

$ra = a \times rp = a \times 8973/(t^{1/1.02} + 68.0)$ $(180min < t)$

rp :point rainfall intensity (mm/h)

a :reduction coefficient

t :concentration time (min)

The locations of sluiceway and drain-ditch sites are shown in Fig. 3.5, together with their drainage areas. The computed design discharges are in the wide range from 0.06 m^3/s to 7.5 m^3/s, as referred to in Table 3.3.

3.4.4 Structural design

(1) Number and size of sluiceway conduit and drain-ditch

The conduit number and size of each sluiceway are determined so that the design discharge may be safely drained through the conduit without overtopping from the levee even in the case that flood water in the drainage channel reaches the HWL at the sluiceway site.

The drain-ditches are also designed so as to safely drain inland waters in their drainage basins up to the design discharges

The required number and size of conduit are determined based on the following :

i) It is assumed that the drainage water in the conduit will be of a pipe flow, when the flood water in the drainage channel reaches the HWL.
ii) Generally, the design velocity of the conduit is to be 1.0 m/sec to 2.0 m/sec in case of a box culvert. However, 3.5 m/sec was applied in this design, because the flood peak discharge estimated by the Rational formula shows a rather large value compared with those estimated by other methods which take into account flood inundation.

The number and size of the conduit are compiled in Tables 3.4.

(2) Sluiceway gate

Two types of gates, namely slide and flap gates, are used for the sluiceways. A flap gate is selected, at the site which meets the following requirements:

(i) Design discharge $\leqq$ 0.2 m^3/s

(ii) EL1-EL2 $\geqq$ 0.6 m

where, EL1 = outlet sill elevation of sluiceway conduit
EL2 = drainage channel bed elevation

Consequently, the sluiceways will be equipped with 50 slide gates and 5 flap gates in total, as referred to in Table. 3.4.

(3) Transverse section of sluiceway conduit

(a) Design load

For designing the transverse section of the conduit, the following five (5) external loads act transversely to the conduit body :

- dead load of conduit body
- earth pressure at rest due to levee embankment
- live load on levee embankment
- ground reaction
- hydrostatic pressure

The loading conditions of major design loads in the above are briefly explained below.

Earth pressure

The earth pressure at rest due to levee embankment acts vertically and horizontally to the culvert as shown in Fig. 3.6.

Live load

i) Vertical live load

Case A : The embankment depth from the top slab of the conduit is less than 3.5 m.

The unit load of rear tires of track in the longitudinal direction of the culvert is given by the following equation:

$$Pl+i = 2 \times Pt \times (1 + i) / Bt = 2 \times 8.0 \times (1 + 0.3) / 2.75 = 7.56$$

where,	Pl + i	:	unit load of rear tires of track (t/m)
	Pt	:	concentrated load per rear tire of track (=8.0t)
	i	:	impact coefficient (=0.3)
	Bt	:	width of track (=2.75 m)

The distribution of the live load due to a track is estimated based on the assumptions as schematically shown in Fig. 3.6.

Case B :Embankment depth from the top slab of the culvert is more than 3.5 m.

The constant uniform load (Pvl) of 1.0 t/m^2 shall act as a live load.

ii) Horizontal earth pressure due to live load

The horizontal earth pressure due to the above live load (Phl) shall be applied to be constantly Ko (=0.5) t/m2 acting laterally on both side walls of conduit with no relation to the depth from the embankment surface.

Ground reaction

The reaction from ground acts upward on the bottom slab of the conduit. Although the conduit body is supported by piles, the ground reaction is expressed by uniformly distributed reaction, assuming that the reaction from ground is dispersed uniformly.

The reaction is expressed as follows :

$$Pvb = Pve + Pvl + Db/B$$

where

Pvb	:	ground reaction (t/m^2)
Pve	:	vertical earth pressure at rest on top slab (t/m^2)
Pvl	:	Live load due to track (t/m^2)
Db	:	unit weight of conduit excluding bottom slab portion (t/m)
B	:	external width of conduit (m)

(b) Combination of design loads

The combination of design loads is arranged based on the following conditions:

i) The load due to hydrostatic pressure is not taken into account, because (i) the difference between internal and external hydrostatic pressures is negligibly small at the top slab and side walls of the conduit and (ii) the hydrostatic pressure acts as a negative load against the external load on the bottom slab of the conduit.

ii) The seismic load is also neglected, because the conduit body is considered as an underground structure embedded in levee embankment.

iii) In case that the conduit is separated with provision of a contraction joint, design of the transverse section of the conduit is to be made for each span, independently.

iv) The transverse section of the conduit under the most critical design load condition to the conduit body is to be applied.

In view of the above, the combination of design load are summarized below.

Vertical load on the top slab of culvert

The following loads act on the top slab of the conduit :

i) earth pressure (t/m^2) : Pve = r x hl
ii) live load (t/m^2) : Pvl = 7.56 /(2 x hl +0.2)
(hl of less than 3.5 m)
Pvl = 1.0
(hl of more than 3.5 than 3.5 m)
iii) dead load of top slab (t/m^2) : Pvd = Dt/B

where,
r : unit weight of embankment soil material (t/m^2)
hl : depth from embankment surface to top slab of conduit (m)
Dt : total weight of top slabs per longitudinal length (t/m)
B : total width of culvert (m)

Lateral load on side and partition walls of conduit

The following loads act on both side walls of the conduit :

i) earth pressure (t/m^2) : Phe = Ko x r x h
ii) live load (t/m^2) : Phl = 0.5
iii) dead load of side walls (t) : Ds
iv) dead load of partition walls (t) : Dt

where,
Ko : coefficient of earth pressure at rest (=0.5)
r : unit weight of embankment soil material (t/m^2)
h : depth from embankment surface (m)
Ds : total weight of side walls per longitudinal length (t/m)
Dt : total weight of partition walls per longitudinal length (t/m)

Upward load on bottom slab of culvert

The following load acts upward on the bottom slab of the culvert :

i) ground reaction (t) : Pvb

(c) Stress analysis

The stress analysis is made by means of the finite element method to estimate bending moment, shearing and axial forces for each member of the conduit. In view of a large number of sluiceway facilities to be designed, namely 51 sluiceways, these stress values are estimated by grouping the 51 sluiceways into the 3 categories as follows:

I. 1 lane sluiceway
1-1 Category 1 ... conduit size : 1.0 x 1.0 ~ 0.7 x 0.7 (represented size : 1.0 x 1.0)
1-2 Category 2 ... conduit size : 1.3 x 1.3 ~ 1.1 x 1.1 (represented size : 1.3 x 1.3)
II. 2 lane sluiceway
2-1 Category 3 ... conduit size : 1.2 x 1.2 ~ 1.0 x 1.0 (represented size : 1.2 x 1.2)

Structural dimension of members

The stress analysis is performed in terms of the rigid frame of a box culvert which is expressed by connected lines in the section center of members as shown in Fig. 3.7. The structural dimensions of each member for the stress analysis are compiled in Table 3.5.

Design loads

The combination of design loads for the stress analysis is schematically shown in Fig. 3.8. Table 3.5 summarizes the combined design loads under respective design conditions.

Result of analysis

The stress analysis was performed under respective design load conditions. The analysis results are compiled in Table 3.6, which shows the maximum sectional forces in each member of the conduit, consisting of a bending moment (M), a shearing force (S) and an axial force (N). The diagrams of bending moment and shearing force are shown in Fig. 3.9.

(d) Examination on thickness of concrete slab

The thickness of the conduit slab shall be examined, comparing the predetermined design thickness with the required one estimated based on the results of the stress analysis. The thickness shall be finally determined based on the following judgment :

$$Td > Te$$

where, Td : design thickness of concrete slab (cm)
Te : required thickness of concrete slab (cm)

The required thickness of concrete slab is expressed as follows :

$$Te = d + d'$$

$$d = C1 \sqrt{(Msmax / b)}$$
$$C1 = \sqrt{(2 / \sigma ca \times m (1 - m/3))}$$
$$m = n \times \sigma ca / (n \times \sigma ca + \sigma sa)$$

where, d : effective thickness of concrete slab (cm)
d' : concrete cover (cm)
Msmax : maximum bending moment taking into account axial force (kg cm)
b : width of concrete slab
σca : allowable compressive stress of concrete (Kg/cm^2)
σsa : allowable tensile stress of reinforcing bar (Kg/cm^2)
n : ratio of elastic modulus of steel to that of concrete (=15)

The design bending moment taking into account the axial force (Ms) is given by the following equation :

$$Ms = N \times (e + c)$$
$$e = M/N$$
$$c = h/2 - d'$$

where, N : axial force (t)
M : bending moment (t m)
h : thickness of concrete slab (m)
d : concrete cover (m)

The results of examination are summarized in Table 3.7.

(e) Examination of shearing stress

The shearing stress of the conduit member shall be examined against the allowable shearing stress of concrete. The average shearing stress is estimated by the following equation :

$$\tau = S / (b \times d)$$

where, τ : average sheering stress (kg/cm^2)
S : shearing stress (kg)
b : width of concrete slab (cm)
d : effective thickness of concrete slab (cm)

The average shearing stress is estimated at several significant sections and the examination results are summarized in Table 3.8.

(f) Examination on required volume of reinforcing bar

The required volume of reinforcing bar shall be estimated using the following equation :

$$As = C1 \times C2 \times Ms/d - N/\sigma sa$$
$$C2 = 0.5 \times \sigma ca \times m \times C1/\sigma sa$$
$$m = n \times \sigma ca /(n \times \sigma ca + \sigma sa)$$

where,
- As : required volume of reinforcing bar (cm^2/m)
- Ms : bending moment taking into account axial (kg x cm)
- N : axial force (kg)
- n : ratio of elastic modulus of steel to that of concrete (=15)
- d : effective thickness of concrete slab (cm)

Taking into account the estimated required volumes of reinforcing bar under several load conditions, the arrangement of designed reinforcing bar is carried out as summarized in Table 3.9. This arrangement is examined at the same sections where examined for thickness of concrete slab of the conduit.

(4) Wing wall

The design of wing walls at the inlet and outlet of the sluiceway shall be made taking account of the earth pressure due to back filling behind the wall and the live load. In this design, the wing wall is assumed as a cantilever sustained by the conduit body.

The stress analysis is carried out for the wing wall at the outlet of the sluiceway, considering that the larger sectional stress acts on it compared with that at the intlet of the sluiceway.

The arrangement of reinforcing bar is determined based on the analyzed stress of the wing wall. The internal section stress are estimated under the designed arrangement of reinforcing bar. The structural safety of wing wall is thus examined comparing the estimated stress with the allowable stress values as summarized in Table 3.10.

(5) Foundation

According to the geotechnical investigation conducted in this project, the N values (SPT values) of sluiceway foundation, composed mainly of clayey/silty material, are in the low range of 2 ~ 8 blows, and the allowable bearing capacity is estimated to be as low as 3.5 t/m^2.

In view of the above, all sluiceways are provided with the pile foundation with R.C. piles of 250 mm x 250 mm, as the estimated allowable bearing capacity is assessed to be insufficient to sustain the total loads to be transmitted from the sluiceway body.

(a) Selection criteria of foundation type

The following criteria are applied for selection of the foundation type, namely spread foundation and pile foundation:

Type	Criteria
Spread foundation	$Qmax \leq Qa$
Pile foundation	$Qmax > Qa$

where, Qmax : maximum ground reaction under full design load condition (t/m^2)
Qa : allowable bearing capacity of ground for foundation (t/m^2)

(b) Estimation of maximum ground reaction

The distribution of ground reaction at each sluiceway site is schematically shown in Fig. 3.10.

The design load condition mainly consists of the following loads :

i) total weight of sluiceway structure,
ii) total weight of embankment materials, and
iii) water pressure load under the filled-up condition

Based on the above, the total design load is estimated and Table 3.11 shows the gravity center and ground reaction of each sluiceway.

(d) Results on selection of foundation type

Based on the above, the estimated allowable bearing capacity, say 3.5 t/m^2 is assessed to be insufficient to sustain the loads to be transmitted from the sluiceway body. Accordingly, pile foundation is adopted.

(6) Design of Pile Foundation

As mentioned earlier, the pile foundation shall be provided for the sluiceway site. The design procedure for pile foundation is briefly explained below.

(a) Estimation of allowable bearing capacity of pile

For the estimation of allowable bearing capacity of a pile, the following formula is applied, which is, in principle, based on the so-called Mayerholf's formula and modified and widely applied by the Ministry of Construction of Japan through in-situ tests.

$$Ra = (Ru - Ws) / Fs + Ws - W$$
$$Ru = Qd \times A + U \times \sum(Li \times Fi)$$

where, Ra : allowable bearing capacity for intrusive force at the top of pile (t)

Ru	:	ultimate bearing capacity of pile (t)
Ws	:	weight of soil for the portion to be replaced by pile body (t)
W	:	weight of pile body including the poured inside materials (t)
Fs	:	safety factor as given below bearing pile : 3 friction pile : 4
Qd	:	ultimate bearing capacity of ground stratum at tip of pile (t/m^2)
A	:	section area of pile tip (m)
U	:	round length of pile (m)
Li	:	thickness of ground stratum (m)
Fi	:	frictional resistance between pile and ground stratum (t/m^2)
Σ	:	symbol of summation

As for the above factors, the ultimate bearing capacity of ground stratum at the tip of pile (qd) is given by the following equation :

$$Qd = N \times (4 \times Df / D + 10)$$

Where,

N	:	N-value at the bearing ground stratum (less than 40)
Df	:	penetration depth of pile in bearing ground stratum (m)
D	:	diameter of pile (m)

The frictional resistance between a pile and ground stratum (Fi) is given below.

Sandy soil : $Fi = 0.2 \times N$ (t/m^2)
(N-value is less than 10)

Cohesive soil : $Fi = C$ or $Fi = N$ (t/m^2)
(N-value is less than 15)

The estimated allowable bearing capacity of pile of each sluiceway site is summarized in Table 3.12.

(b) Estimation of required number of piles

In order to estimate the required number of piles for foundation, the reaction of piles is estimated and thereby the required number of piles is evaluated to sustain the total load transmitted from sluiceway body and levee embankment.

Since the sluiceway body is embedded in levee embankment and the lateral loads acting on both sides of the conduit body is symmetric, the load transmitted from sliceway body and its levee embankment to pile foundation is vertical load only. Therefore, the intrusive vertical load is herein considered for the design of pile foundation.

The required number of piles for foundation is estimated as follows :

$$N = \sum Wi / Ra$$

where,

N	:	required number of piles
Wi	:	vertical design load (t)
Ra	:	allowable bearing capacity for intrusive force at the top of pile (t)
$\sum$	:	symbol of summation

(c) Arrangement of piles

In general, when the distance between piles is smaller than 2.5 times of pile diameter, the bearing capacity of piles empirically tends to decrease compared with the capacity under the condition of a single pile. This might be due to the adverse effect to be caused under the condition of a group of piles.

From the engineering point of view, the determined number of piles shall be arranged at the minimum distance of 2.5 times of the pile diameter between pile centers.

In addition, the distance between the center of pile and the edge of sluiceway body shall be arranged at 1.25 times of pile diameter in the case of a drift pile.

Consequently, piles shall be arranged so that they might equally receive a long lasting load. Under such loaded condition, the reaction of each pile is determined by the following equation :

$$Vi = \sum Wi / N + \sum Wi \times e \times Xi / Ig$$

where,

Vi	:	reaction of pile (allocated load to pile) (t)
Wi	:	vertical design load (t)
N	:	total number of piles
e	:	eccentric distance between the gravity center design load and that of pile (m)
Xi	:	length between pile and gravity center of piles (m)
Ig	:	geometrical moment of inertia of piles (m^2)
$\sum$	:	symbol of summation

The required number of piles is finally determined so that the estimated reaction of pile might not exceed the estimated allowable bearing capacity of the pile. The calculation results are summarized in Table 3.12.

(6) Cut-off wall

To ensure the safety against piping due to the difference in water level between the sluiceway inlet and outlet during floods, sheet piling cut-off walls (depths of 2 ~ 2.5 m) are provided at the inlet and outlet bottoms.

(a) Examination on necessity of sheet pile

The safety against piping shall be examined by means of the Lane's formula as presented below.

$$Cw \leq (\sum Lh / 3 + \sum Lv) / \Delta H$$

where,	Cw	:	Lane's creep ratio
	Lh	:	length of horizontal creep line (m)
	Lv	:	length of vertical creep line (m)
	ΔH	:	difference of water levels at both sides of sluiceway (m)
	Σ	:	symbol of summation.

The Leins's creep ratio (Cw) is as follows :

Soil condition	Cw
Very fine sand or silt	8.5
fine sand	7.0
medium sand	6.0
coarse sand	5.0
fine gravel	4.0
medium gravel	3.5
coarse gravel	3.0
plastic clay	3.0
medium clay	2.0
heavy clay	1.8
solid clay	1.6

Firstly, the necessity of sheet pile is examined by use of the said formula applying that there is no sheet pile ($\sum Lv = 0.0$). The water level in the land side of a sluiceway is assumed to be 1.0 m above the ground line.

(b) Design of sheet pile

The sheet piles are to be provided at two locations of the inlet and outlet of the conduit. The required total depth of sheet piles is estimated by means of the aforementioned formula.

In addtion, the minimum depth of sheet pile is set at 2.0 m in view of working efficiency for construction.

(7) Main features of sluiceways

Main features of the sluiceways and drain-ditch structures are compiled in Table 3.4.

3.5 Other Channel Structures

(a) Open culvert with mesh cover and screen

The open culvert is applied to the 390 m long channel stretch from outlet of the Saluran Cengkareng drainage channel. The open culvert is divided into 3 lanes and is provided with mesh cover at the top over the whole stretch in order to prevent dust and garbage from being thrown into the drainage channel. A screen made of I-type steel members is provided at the upstream end of the open culvert to prevent dust and garbage from flowing into the culvert. The open culvert is designed as shown in Fig. 3.11.

(d) Open U- type concrete ditch

The open U-type concrete ditch was adopted to the Kamal (branch) and the tanjungan drainage channels and the Meruya area. all the proposed ditch in the Meruya area except for highly elevated portion in the center part of the drainage area, where box culvert is provided.

Drainage channel	Concrete ditch length (m)	Remarks
Kamal (branch)	452	upstream stretch
PIK Junction	765	whole stretch
Meruya area	1,477	

(3) Inspection road

The drainage channels need an inspection road on one side. The inspection road is aligned in the crest of the earth type levee or in the area along the channel, except for the channel stretches where the existing public road will be available for the inspection road., as shown below:

Drainage channel	Length of inspection road(m)			Remarks
	Left bank	Right bank	Total	
Kamal (main)	2,264	632	2,896	
Kamal (branch)	0	1,357	1,357	
Tanjungan	0	495	495	
Saluran Cengkareng	2,602	1,338	3,940	
Gede/Bor	265	0	265	

4 BRIDGE AND CULVERT

4.1 Preliminary Study

4.1.1 Superstructure

(i) Road classification and width of roadway

The road classification and width of roadway for new bridges has been studied according to the Indonesian Road Geometric Design Standard as shown in Fig. 4.1. The following points are taken into account to make the typical configuration of the bridge section:

(1) Applicable road classification to the replacement have been chosen in the classification tables of the standard width of the above design standard. As the result of the study for application it was clarified that the roads of the project are all belonged to Type II of the primary road classification, which is defined as the road with partial access control or non access control.

(2) Lane number of the road is examined on both cases of one and two lanes, i.e. two lanes with sidewalk and without sidewalk which belong to for Classes I, II and III, and 1-lane is shown only for Class IV.

(3) In the column of particular case another aspects for the application has been introduced according to the Bridge Design Code as follows: (i) the width of Class I and Class II are composed of the total width including full width of the shoulder of approach road and (ii) Classes III and IV have 6 m for securing minimum width of two lanes for safety car operation so that the bridge width of between 5 and 6 m should be avoided because they give drivers the mistaken impression that passage is possible.

(4) Concerning loading class noted in the remarks, BD 100 (100% of live load) and BD 70 (actuary 80% of the load is applied) are used to each classifications in accordance with the criteria proposed in this report.

As to the width of pedestrian bridges, 2 m is taken as a minimum standard width between the both inside of railing on which 2 bicycles can pass each other with keeping safety distance. The selection of the above width is explained in the stipulation of the

Road Geometric Design Standard that minimum width of bicycle lane shall be 1 m. Fig. 4.2 illustrates the typical width of pedestrian bridge: 2 m is provided for Type-1 and m for Type-2, which is widened to make equal to that of a existing bridge.

(ii) Width of existing bridges and new bridges

Road Bridge and Culvert of Fig. 4.3 a) shows the summary of measurement of existing bridge dimension obtained by field survey: the total number of frequency in respect to width of bridge and culvert and also accumulation percent line.

Since the construction history of the bridges such as construction year and applied standards and codes are obscure, the dependence on the standard currently in use will be a most proper way to reflect the present traffic situation and introduce modern developed technology for the construction plan. The bridge widths for the replacement in Fig. 4.3 are shown on the upper of the graph with the arrow marks to let clear the relation between the existing width and new one. For the Pedestrian, the width is grouped into two types as described in Fig. 4.3 b).

(iii) Bridge type

According to the channel widening plan the widest width of the channel is estimated at approx. 40 m of the Kamal channel after widening, for which the PC concrete girder type can be selected among various kind of bridge type as a most optimal as center pier can be constructed in the channel on the relatively shallow bed.

Table 4.1 illustrates the sub-classification of PC concrete girder type, in which pre-tension girder type is recognized most applicable to short spans less than 24 m long, while post-tension bridges is suitable for long spans more than 20 m long. Taking into consideration the bridge construction conditions of small river crossing and working at narrow space in the residential zone, the solid slab bridge type and hollow slab bridge of five types will be the best selection for new bridge. These pretension beams can be produced and stocked in factory in advance to the election so that there can be expected several advantages such as high quality of concrete, simplification of field works and shortening of construction period.

Fig. 4.4 is a graph of showing the relation between span length and girder height. Although these data are obtained based on the trial design performed according to Japan Bridge Standards due to similar live load between the two countries, the relation of them will be almost same as designed on the Indonesian Standards, therefore the graph can be effectively used for setting up the definitive plans of girder section.

The ratio of girder height to bridge span is roughly computed as follows by use of the relation as shown in Fig. 19:

Solid Slab	:	h/s = 1/17* - 1/22
Hollow Slab	:	h/s = 1/24 - 1/25
Reinforce Concrete Girder (for reference)	:	h/s = 1/13 - 1/16

where: Symbol "h" indicates a girder height and "s" is a span length.

Remarks: Since minimum height of girder section for small span in case of solid slab is decided by the minimum space of reinforcing bars arrangement and PC cables irrespective of stress, the figure of the ratio become large.

With an overall evaluation as mentioned above, the slab bridge with pre-tension girder will be considered most suitable selection of the bridge type.

4.1.2 Substructure

(i) Pile Trestle

Fig. 4.5 shows the typical foundation type with it's applicable height ranges. Among these types, Pile Trestle Type will be recommended as a most suitable substructure due to the following reasons:

(a) Channel depth is so shallow with 1.5 to 1.6 m deep that Pile can support the trestle and bearing forces in stability.

(b) More economical than other types due to the simple works and small bill of quantities.

(c) Occupied area by Pier in river width is smallest.

(d) It is standardized in Indonesian Collected Drawings as illustrated in Fig. 4.6.

As one of disadvantage of pile trestle type it is generally pointed out that this type is unable to resist against the lateral force by earthquake due to the flexibility of the pile, however designing conditions for the pile are greatly differ from those of normal case, i.e. the city of Jakarta is located in the lower earthquake coefficient zone according to the Bridge Design Standard and the length of pile exposed on river bed is very short as mentioned above.

(ii) PC pile

The design conditions for the substructure is not serious due to the following reasons: since the bridge size of the Project is very small, the bearing force conveyed from the superstructure is not large due to proportion to the span length and moreover bearing stratum composed of the clay with N value of 20 which meets design requirement for the pile are lying on 12 to 15 m deep at most under the ground. The soil information are suitable ground condition for the selection of pre-cast pile foundation such as RC pile and PC pile, in which the latter will be requested to use due to the property of bigger lateral resistance itself against earthquake force and lateral components of vertical force due to slanted piles.

4.1.3 Ancillary structures

(i) Guard-rail

Guard-rail shall have fundamental functions to prevent pedestrians and vehicles from falling down into the channels. The typical section of guard-rail is shown in the collected drawings of "Standard for Bridge Superstructure Prestressed Concrete Girder" as shown in (1) of Fig. 4.7. The standard type features two functions, i.e. the strong resistance of concrete wall against car collision and the external slender appearance by use of steel hand rails fabricated on the concrete walls.

(ii) Expansion joint

There are three types of Expansion Joint applicable to small bridges, i.e. a) open type is suitable for small bridge with concrete pavement, b) closed type for small bridge with asphalt pavement and c) rubber type for heavy traffic passage refer to (2) of the figure. The above three types will be selected according to the bridge specifications.

(iii) Bearing type

For the bearing neoprene rubber is usually used, as shown in (3) of the same figure.

(iv) Drainage

Drainage pipes can be installed outside the girders or at the space between the adjacent girders as shown in Fig. 4.8(4).

(v) Lifting measures of bridge girders

To cope with the land subsidence, the lifting measures are to be considered at the design stage as specified in the criteria of the report. (5) of Fig. 4.8 (5) illustrate an idea of the device, by which the girder can be lifted with the operation of oil jacks temporally installed on the concrete floor built in shape of rectangular cut at the corner of the Trestle. In case of the usage of 30-40mm depth flat jack, the rectangular cut is not required.

(vi) Retaining wall

The bridge elevation is computed approx. 2.0 m higher than existing one due to securing the specific vertical clearance as described in the criteria. In order to minimize land area occupied by the access road especially for residential area, the retaining wall shall be constructed with suitable type as shown in Fig 4.9, gravity type can be recommendable as a most optimal.

4.1.4 Culvert

As previously mentioned on the definition of the culvert in the criteria, the crossing structure built close to the road surface can be classified into bridge or culvert depending on the span length whether it is over 2 m or less than this. Fig. 4.10 illustrates the classification of crossing structure including subsoil culvert.

4.1.5 Span length

Table4.2 tabulates the provisional division of span length in each channel set up based on the basic study of channel widening. Span length are set up based on the following ideas:

(i) Slab bridges with solid beam and hollow beam are presupposed to select span division.

(ii) Maximum span length is less than approx. 16 meters long due to needs of lower height of girder, transportation condition on the roads and capacity of factory equipment in the local.

(iii) For the crossing structure of small channel culverts are applied according to the classification previously stated.

4.1.6 Principal dimensions of bridge and culvert

The principal dimensions of bridge and culvert have been studied based on the information of the existing bridges and their circumstantial conditions obtained in the field investigation such as scale measurements of roads and bridges, traffic volume observation at bridge sites and road network together with the preliminary study results as previously stated in this report. Results of the study is summarized in Table 4.3 and location of the bridges and culverts are illustrated in Fig. 4.11.

4.2 Design

4.2.1 Superstructure

(a) Primitive plan of the structure

The detailed design was made based on the primitive plan proposed in the preliminary study of which the main points are confirmed for conducting the detail design, i.e. they are summarized as follows:

- The bridge locations are to be same as those of existing bridges to so that the road alignment of new bridge shall be basically on the center of existing bridges unless otherwise specified.
- The bridge span is to cross over between the both levees planned according to the widening plan.
- The bridge width and configuration are planned in accordance with Indonesian Code with keeping at least those of existing bridges.
- As the gradient and cross fall of bridges, 1.5% and 2.0% respectively are adopted as a bridge surface drainage, which are commonly used to maintain a natural flow of rain on the bridge deck covered with asphalt pavement of 5 cm depth.
- Ancillary works such as railings, expansion joints and bearings are selected among the typical type used in Indonesia as illustrated in the preliminary study.
- The pre-tension girders are adapted to have as lower girder height as possible and further the maximum girder length is limited to less than 17.5 m taking into account the severe construction conditions to be executed in narrow space at residential area and transportation restraints on the national road.

(b) Comparison of live load

Concerning the application of "T" load to the bridge which is generally used for the design of small bridge with the span of less than 17.5 m, since the both codes, i.e. load standard and design code are different in the stipulations that the reduction of T load for local road is not allowed in the latter code against the allowance of load standard, other two standards, i.e. the standard drawing collection of in-situ slab bridge with small span of less than 6 m in Indonesia and Japan Bridge Standards are tried to compared with the Indonesian load standard:

As the results of the study the followings were made clear:

- According to the drawing collection issued by BINA MARGA, BM 100 and BM 80 are recommended instead of BM 100 and BM 70.
- In comparison with Japan Bridge Standard, "A" load is equivalent to BM 80 and "B" load is just same as BM 100.

It is finally recommended to design the girder to cover BM 80 load for the nominal BM 70 bridges taking consideration the international trend of car weight getting bigger in addition to the above study. The increase in bill of quantities due to the change of live load are substantially very small. Table 4.4 shows the comparison of the moment and shear force by live load of Indonesia and Japan.

(c) Grouping of girder length for the calculation based on the primitive plan

The total number of structures to be designed are classified as shown below:

Drainage Channel	Girder type	Slab type	Total
- Kamal(main)	9	-	9
- Kamal(branch)	17	2	19
- Tanjungan	5	-	5
- PIK Junction	-	4	4
- Gede/Bor	10	-	10
- S. Cengkareng	13	-	13
- Meruya	-	16	16
Total	54	22	76

Among these, 54 are normal sized bridges with pre-tension girders, while 22 bridges are small to be designed as an in-situ slab bridge respectively. There are classified in respects of bridge width and girder length belonged to each channel to make clear the breakdown of principal dimensions for the first step and further grouped in same length irrespective of the division by channel. Finally the total number of structures to be designed are counted as follows:

Bridge	31
In-situ slab	6
Total	37

Namely, 37 bridge structures shall be designed to decide the structural sections which can be applied to the remained 39 structures. Table 4.5 shows the classification of structures belonged to each channel and Table 4.6 tabulates the grouping in same dimensions so select the representatives of structures for the computation.

(d) Stress summary

The assessment of each girder section is summarized in the stress sheets showing the stress by service loads under the allowable stress at girder center and safety ratio of ultimate moment by combined load to destructive strength as well. These assessments certificate that each girder has been designed safely with specific safety ratio. The reaction shown at same sheets can be used for the design of substructure. Table 4.7 shows the stress summary of all bridges.

(e) Design section

Girder has an uniform rectangle section height through the girder length which is composed of 700 mm in width and 400 mm in height for the shortest span of 7.0 m and 700 mm x 700 mm for the 17.5 m length respectively. The slenderness ratio of the girder varies from 1/17.5 for 7.0 m length and 1/25 for 17.5 m respectively, which are fare less than that of I girder type, i.e. approx. 1/16, that contributes to shortening of the access road due to the smaller gap with existing road level than I girder, while the height of pedestrian bridge varies from 350 mm to 500 mm with 700 mm in width, of which the slenderness ratio is from 1/20 to 1/30.6, being far smaller depth than those of road bridges. Table 4.8 depicts the typical section of each girder.

4.2.2 Substructure

The following consideration has been taken for the design of substructure:

(i) Pile trestle type which consists of piles and hammer head (shaft) has been selected due to the economical and structural reasons as stated in the section of preliminary study as follows:

- Reactions from superstructure are comparatively small due to shorter span as a bridge scale.
- Since the height of the piles exposed over the channel bed is only about 3 m to 3.5 m, the transverse flexibility is not so significant to support earthquake from in stability.
- In addition to the above item, the bridge location is situated in the region of small seismic coefficient.

(ii) The structural detail of hammer head has been designed subject to the local standard, where vertical piles are used instead of battered as distributed in Table 4.9 piles to ease the driving work of piles with small construction tolerance, while PC Pile is chosen to increase more transverse ultimate strength rather than RC Pile.

(iii)Since there are so many bridges to be located along the channel in wide stretch and more over the test results of Dutch-cone sounding shall be used, it is not necessarily precise for evaluation, for reading the final position of pile tip to be settled, the smaller allowable capacity of piles i.e. 44.1 tons at 16 blows of N value has been selected to take safer side among 3 cases of trial computation proposed in the interim report and further 20 blows of N value which is generally considered reliable bearing layer of clay is used for pointing out the position of pile tips. Although the safer side has been taken as a pile capacity here, the pile number is mainly decided by the balanced arrangement of piles to support the hammer head in sense of stability and safety ratio of stress check instead of minimum number assessed only by the pile capacity. From the same reasons as above, the consolidation settlement will be far less than the figures reported in the interim report as several millimeters and in this connection 1 cm is generally accepted as a allowable settlement of bridge foundation.

(a) Typical section of hammer head

The typical section of hammer head is composed of 2 m x 0.65 m (width x height) of rectangular for the pier and 1.8 x 0.65 m for the abutment. These rectangles are supported in balance with double squads, which are used for the bridges with large reactions such as comparatively long span more than 11 m Class I road (national road) and pier of multi-spans. For shorter bridge span less than 11 m and pedestrian bridges are designed 1.2 m x 0.65 m section supported with single squad to save construction cost. For the countermeasures of subsidence in future, the superstructure can be lifted with the operation of flat jack (30 to 42 m/m depth) by the installation of it between the substructure and superstructure. Fig. 4.12 illustrates the typical section of abutment and pier.

(b) Pile strength

Concerning the pile strength for computing the numbers of piles required to support the reaction from the superstructure and the self weight of hammer head, the allowable bearing capacity of piles is recommended in the section of topographic survey for three cases possibly occurred based on the soil exploration as follows:

Tip of pile	N-Value at pile tip	Type of pile	Depth	Allowable bearing strength
Stiff Clay	24 blows	friction	15 m	88 t
C. Clay	15 blows	friction	9 m	41 t
C. S	32 blows	end bearing	8 m	85 t

Amount three cases above stated the lowest capacity of 41 tons has been selected due to the below reasons;

- Since many bridge are planned to be located in wide stretch, it is practical to choose the safer side of pile strength as a typical capacity among 6 core borings points.
- The pile number is substantially decided by the strength of hammer head from the view points of stability of pile trestle, namely the number has been decided for the strength balance of the trestle and stress check rather than designed by the pile strength capacity.

In addition to the above consideration for the pile capacity, the geological configuration of stratum along the channels was investigated by way of Dutch Cone Test at along all channels length, and for the tip of pile to be drove 20 blows of N value has been chosen, which is generally considered stiff clay suitable for bearing layer of pile.

(c) Arrangement of pile

The number of pile to support superstructure and hammer head can be obtained with dividing the total load acting on the piles by the allowable pile capacity. The number to be practically used are studied from respects of the stability of pile trestle and the strength of hammer head as stated above and finally assessed with the consideration of suitable arrangement of piles connected safely to the lower portion of the hammer head. The usage of single or double squads of piles is decided based on the following criteria;

- The bridge with the span of more than 11 m is supported with double squads according to the Indonesian Standard.
- The bridges on National Highway (Road Class - I) administered by BINA MARGA are of double squads to resist heavy traffics.
- Piers of multi-spans of road bridge are to be supported by double squads to avoid the eccentric load at pile top.
- Those of simple span with under 11 m and pedestrian bridges are put on single squad of piles to make the construction cost lower.

Table 4.9 shows the pile number and arrangement deployed within the hammer head.

(d) Pile length

The geological data are fully used to presume the bearing layer penetrated with pile tips. In the report the geological cross section connecting the bore data of 6 points and geological profiles of Dutch Cone Sounding along the channels are depicted, which can be effectively used to point out the tip location of pile to be drove by way of the insert method of putting bridge location at between two sounding positions of the profiles. The layers with more than 20 blows of N Value were read on the profiles as a bearing stratum, in which the following depth are roughly assessed in each channels by the geologist of this project:

Channel	D (N:20) (m)
Kamal(main)	10 to 16
Kamal(branch)	7 to 19
Tanjungan	9 to 17
PIK Junction	8 to 11
Saluran Cengkareng	5 to 9
Gede/Bor	9 to 10

Table 4.10 tabulates the depth of the pile tip at 20 blows of N value inserted between the positions of sounding test which are located with in the above indications.

4.2.3 Approach road

The elevation of new bridges is positioned on the vertical clearance given by the height of free board and the allowance of land subsidence, therefore the elevation gap between bridge surface and existing ground line shall be smoothly connected by the access road with transition curve. For the design of transition curve and ancillary belongings related to the access road the followings are taken into consideration:

- Gradient angle, transition curve and length of the access road are designed basically according to Indonesian road alignment standard.
- For the pedestrian bridge the straight line of 10% or stairs for an alternative will be used.
- In case of difficult to obtain the specific alignment of standard because of technical reasons or from the inconvenience of daily life for residents, straight curve is inevitably taken just at the place of few traffic volume for the access road although it is out of the standard.
- Storm sewage's are provided on both side of the access road.
- If there is narrow space between houses and road side at resident area the guard-rails are installed on the retaining walls to prevent the residents against car accidents.

(a) Gradient and transition curve

As stated above, maximum gradient and transition curve and their length are stipulated in the standard for Geometric Design of Urban Road. Table 4.11 shows the specific figures for maximum gradient, transition curve radius and minimum length of curvature in respect of each design speed and further applied road classes to the specific design speed are recommended in remarks column.

(b) Bridge elevation

The elevation of deck surface has been calculated as the following equation:

Bridge Elevation = H.W.L. + Free Board + Land Subsidence + Girder Height + Pavement Depth + Cross-fall

In line with the elevation gap between bridge surface and ground line, the ground line has been read on the field survey data, by which the gap are computed as outlined below:

Channel	Elevation Gap (m)
Kamal(main)	1.1~2.1
Kamal(branch)	1.4~2.0
Tanjungan	1.5~2.4
Saluran Cengkareng	0.8~2.6
Gede/Bor	0.5~2.9

The locations of bridges with big elevation gaps over 2 m mainly are brought about due to originally existing with existing bridges. These elevation gaps are shown in Table 4.12 together with length of the access road.

(c) Approach road length

The total length of access road can be computed mechanically according to the standard as follows;

Channel	Road length (one side) (m)	
	Carriage way	Pedestrian
Kamal(main)	30.5~60.5	15.7~21.3
Kamal (branch)	31.2~50.1	18.7~21.6
Tanjungan	49.2~68.8(83.5)*	
Saluran Cengkareng	27.6~57.0(84.0)*	13.8~26.0
Gede/Bor	43.9~79.0	19.1

* : National Road (Class 1st)

As the above table indicates, the approach length is 50 to 70 m for local road, approx. 85 m for National Highway and approx. 25 m for Pedestrian bridges. The length has been adjusted and modified after the detailed study on the topographical map and field investigation. The breakdown of the length is shown in Table 4.12, however, these figure will be re-adjusted to existing road after the completion of widening detailed drawings.

(d) Typical section of access road

The typical section of access road is planned in two different types i.e. retaining wall or slope protection type, whether the bridges are situated at field area or resident area. The embankment in field can be provided with gentle slope for side protection, while in resident area the retaining wall of concrete is build to make the total width of access road narrow as far as possible to secure the space wider before houses and furthermore the installation of guard-rail and storm drainage shall be provided on the retaining walls for improving the comfortable life of residents. Fig 4.13 shows typical section of access road.

4.2.4 Design of culvert

(a) Small bridge and culvert

In this report the definition of border line between bridge and culvert is put on the 2 m span length, however, this definition is not necessarily internationally known. The classification by span length is, however, generally accepted in Indonesia because it is stipulated in two standard, i.e. in-situ bridge for 2 to 6 m long span and culvert for 2 m span.

According to the above definition in-situ bridges are estimated at 22, i.e. 2 in the Kamal(branch), 4 in the PIK Junction and 16 in the Meruya. Besides, two types of box culvert are planned under ground for the Meruya area. These are 2.2 m x 1.6 m size and 288 m long, and 3 m x 2.2 m size and 504 m long.

(b) Typical section

Typical section of In-situ slab bridge and culvert are illustrated in Fig 4.14 according to Indonesian standard. The detail design is being carried out.

4.3 Study of Remained Items

4.3.1 Allowable girder length

In consideration of the possibility of minor changes of widening plan in construction stage, the allowable span length of bridge has been computed to cope with unexpected changes. The maximum span of girder allowed for the change are tabulated in Table 4.13 together with those of basic design.

Table 4.14 shows the stress check of the girders in case of use of the maximum length allowed for each girders that certificates each girder has been designed under the condition less than allowable stress.

4.3.2 Principal dimensions of bridges of detailed design

The span length of basic design has been reviewed by adjusting the span length with the varieties of drainage width. The updated dimension of all bridge is given in Table 4.15 together with that of the basic design, in which allowable maximum girder and bridge length are presented for reference for the case of possibility of span change occurred from unknown reasons in future.

Major reasons of the change except Meruya channel of which the channel line has been changed after having reviewed according to final widening plan are as follows:

(a) At downstream close to the channel mouth of the Kamal (main) and Tanjungan, the slope grade of levee becomes gentle from 1:0.5 to 1:2.0 in design, for which the span of BKM 1. 2. 4 and BTM 1 was changed.

(b) The intersection angle of skew bridge, i.e. BKM 5 to road center line has been newly found to be less than that of basic design.

(c) Five bridges(BKM9,BKE12,BCM1,BGM11 and BGM12) are canceled due to the final channel plan and new road network.

(d) For other several bridges out of above mentioned, minor change was made due to the technical reasons of drawings for BCM 11, 12, and 13.

(e) A bridge(BTM6) has been newly added

Concerning Meruya channel 3of 9 structure (BMM3.4 and 7) are same location as existing ones and 22 bridges or culverts and 2 underground culverts have been newly added, counting 24 structures in total

All other bridges are currently as same dimension as that of the basic design.

4.3.3 Number of bridge

Number of the bridge to be designed was estimated at 76 after the reviewing works and 7 structures were increased from the basic design as follows:

Structure type	Basic design	Detailed design
Pre-tension girder	62(31)	54(36)
In situ slab bridge(L>2)	7(4)	22(6)
Total	69(35)	76(42)

Note: (1) Number in parenthesis indicates the number of bridges with different girder dimensions.
(2) Culvert including 14 in-situ bridges($L \leqq 2$).

The differences between both design stages have been inevitably brought about due to the advancement of design work from rough to detail.

Table 4.16 tablets the classification of girder length in respect of bridge width used for detail design.

4.3.4 Earthquake-proof design of substructure

(a) Design standard

The seismic design of substructure has been carried out properly according to Indonesian Standards, "Procedure of Designing Earthquake Proof for Highway Bridges, SNI 03-2833-1992.

(b) Equivalent horizontal seismic coefficient (Kh)

Kh can be obtained from below equation:

$$Kh = Kr \cdot f \cdot p \cdot b$$

where:

Kr = 0.15 (combined response coefficient, natural period of bridge (Tg) is considered less than 0.4 second per cycle in Zone 4 in the above standard, which is the safer side of evaluation.)

f = 1.0 (structural factor, for the bridge of BM 100 located on national road)

f = 0.8 (same as above, for that of BM 70 located on local road)

p = 1.3 (material factor, for PC structure)

Consequently,

$$Kh = 0.15 \cdot 1.0 \cdot 1.0 \cdot 1.3 = 0.195 \text{ for BM 100}$$
$$Kh = 0.15 \cdot 1.0 \cdot 0.8 \cdot 1.3 = 0.156 \text{ for BM 70}$$

Horizontal force can be computed by the multiplication of "Kh" to the reaction of dead load as follows:

$$H = Kh \times Rd$$

where:

H : Equivalent horizontal force (t)

Rd : Reaction by dead load (t)

(c) Equation to compute the section force of pile

Since there are usually differences much or less between presumption of calculation and real structures, two methods with different preconditions were applied to check the safety of pile stability, for which the bigger section force has been selected. Table 4.17 illustrates the equation of above two methods with figures to make clear the differences:

Method-1 Equations to obtain section force of pile are simplified with the consideration of portal shape of pile trestle and also the convenience of manual computation as follows:

(i) Fundamental structural system has been set up based on the condition that a column driven in soil is supported with the elasticity of soil.

(ii) Horizontal seismic force acting on the top of pile is transmitted to the pile top at ground surface with two forces i.e. moment and shearing force, in which the moment is supposed to add to the maximum moment brought about by the shearing force at under ground. This equation is established for the analysis of section force of pier.

(iii) For abutment, a hinge connection at pile top is presumed as a boundary condition for analyses model that is considered as safer side computation.

Method-2 The equation of the table can be operated with personal computer, the preconditions of which are based on the following model:

(i) Structural system is a single column supported downward with the elasticity of soil.

(ii) For the analysis of single squad piles irrespective of foundation type i.e. pier or abutment, the boundary condition is set up that the horizontal force is acting at column top in free rotation.

(iii) For that of double squad piles, single column is presumed to be fixed at the top for moment.

In the two methods above stated, the section force of moment and shearing force are considered for computation but not normal force because moment force is considered to be predominant on the ultimate strength of pile.

(d) Calculation of section force

The sectional force obtained with Method-1 is tabulated in Table 4.18 in which the elastic spring coefficient of soil is computed with the precondition that 3 blows of N value of upper layers is taken as typical soil condition due to affecting the section force more than lower layers. The specific computation procedures are explained at the end of the Table as commentary notes.

Instead for Method-2: The coefficient of two layers, i.e., upper and lower layers has been used to calculate the section force in detail. All input data of brides for Method-2 are shown in Table 4.19.

(e) Stress check of pile

Stress check has been carried out based on the operation of two methods.

Bigger moment out of two output have been selected to compare with the allowable resistant moment of pile defined as ultimate strength.

Table 4.20 shows the selected maximum moment which are marked with an asterisk and pile strength applied to each foundation, where three grades of pile come to be employed, i.e. 350-A, -B and 400-B of JIS A 5335 which can be purchased in Indonesia.

4.3.5 Stress check of hammer head

Hammer head comprising one of trestle structure transmitting the reaction from superstructure to pile head has been designed under the following preconditions according to Indonesian Standard:

(a) Beam theory was applied for the analysis of the structure since a beam is supported longitudinally with mullet-piles.

(b) Reaction of dead load from superstructure is acting uniformly and longitudinally on the hammer head.

(c) Both live loads, i.e. BM 100 (10 tons concentrated load) and BM 70 (8 tons instead of 7) are included for impact load.

(d) Horizontal force by earthquake is loaded on the top of hammer head. Typical loading cases and deployment of piles which induce maximum moment has been selected in respect of both squads, single and double among all bridges to check stress.

Table 4.21 tabulates the computation results such as section force by service and seismic load, section modulus, stress and allowable stress that certifies the safety of the structure. Pile arrangement has been decided after the following examinations:

(a) Stress check of pile in respect of not only service loads but also seismic ones, which affect the pile strength and number.

(b) Strength of hammer head which effects the distance of pile support.

The final deployment of pile is illustrated in Table 4.22.

4.3.6 Design of access road

(a) Classification of mound up height

The classification of mound up height for the access road is as follows:

	Mound up height (m)						Total
	0<h<0.5 and h=0	0.5<h<1.0 and h=0.5	1.0<h<1.5 and h=1.0	1.5<h<2.0 and h=1.5	2.0<h<2.5 and h=2.0	2.5<h<3.0 and h=2.5	
Number of bridge	18	4	10	21	19	4	76

Remarks: Kamal(branch) is counted as a lifted structure for land subsidence in this table as it will be decided after having negotiation with the resident of the area.

This table shows that number of the bridge in a range between the mound height of 1 m to 2.5 m is 54, amounting to 67 % of the total number. Twenty three bridges with less than 0.5 m in height are in-situ slab bridge with small span less than 6 m are planned without consideration of the allowance for land subsidence due to the easy work of heightening the deck slab of bridges and culverts with relatively light weight.

(b) Classification of side protection

The side protection works of the access road will be classified into following types, namely, sod facing-6 retaining wall-43 stairs for pedestrian-12 and no need of embankment-23. It shows that the retaining wall type occupies 71 % of al of the access roads because the bridge is remarkably approached to the residential areas. Table 4.23 tablets the regional classification of above statement.

4.3.7 Others

(a) Remarks on the technical specification

Technical specification from the viewpoint of designing will be annexed to the volume of drawings in preface as common technical remarks. Table 4.24 is the summary of the draft of technical specification.