社会開発調査部報告書

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

DIRECTORATE GENERAL OF HUMAN SETTLEMENTS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

THE DETAILED DESIGN
FOR
URBAN DRAINAGE PROJECT
IN
THE CITY OF JAKARTA

FINAL REPORT

VOLUME II

SUPPORTING REPORT

ANNEX II

No. 4 Design Criteria
No. 5 Design and Structural Calculation
No. 6 Work Quantity Calculation

DECEMBER 1997

NIPPON KOEI CO., LTD.
TOKYO, JAPAN

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The cost estimate is based on the price level of June 1997 and the monthly mean exchange rates in June 1997. The monthly mean exchange rates in June 1997 are:

US$ 1.00 = ¥ 115.00 = Rp. 2,350

ABBREVIATIONS

(1) Local Terms

BAKOSURTANAL	Badan Koordinasi Survei dan Penetaan Nasional	:National Mapping Agencies
BAPPENAS	Badan Perencanaan Pembangunan National	:National Planning and Development Board
BPS	Biro Pusat Statistic	:Central Bureau of Statistics
BINA MARGA		:Directorate General of Road Development
CIPTA KARYA		:Directorate General of Human Settlements
DGWRD		:Directorate General of Water Resources Development
DINAS TATA KOTA		:Department of City Planning, DKI Jakarta
DKI Jakarta	Daerah Khusus Ibukota Jakarta	:Special Region of Capital City Jakarta
DPMA	Direktorat Penyelidikan Masalah Air	:Directorate of Hydraulic Engineering
DPU	Departmen Pekerjaan Umum	:Ministry of Public Works
DPU DKI Jakarta	Dinas Pekerjaan Umum DKI Jakarta	:Department of Public Works, DKI Jakarta
DPUP	Dinas Pekerjaan Umum Propinsi	:Provincial Department Office of Public Works
JABOTABEK		:Jakarta-Bogor-Tangerang-Bekasi
JASA MARGA		:Indonesia Highway Corporation
Kabupaten		:Regency
Kecamatan		:Sub-district
Kelurahan		:District
Kotamadya		:Municipal City
PELITA	Pembangunan Lima Tahun	:Five-Year Development
PERUM PERUMNAS		:National Urban Development Corporation

PMG	Pusat Meteorogi dan Geofisika	:Meteorological and Geophysical Center
P.P.	Priok Pile	
P.T.	Perusahaan Terbatas	: Private Estate Enterprise (Company Ltd.)
PWSCC	Proyek Pengembangan Wilayah Sungai Ciliwung-Cisadene	: Ciliwung-Cisadane River Basin Development Project Office
RKL		:Environmental Management Program
RPL		:Environmental Monitoring Program
REPELITA	Rencana Pembangunan Lima Tahun	:Five-Year Development Plan
TTG.	Tanda Tinggi Geodesi	

(2)International or Foreign Organization

GOI	:Government of the Republic of Indonesia
GOJ	:Government of Japan
IBRD	:International Bank for Reconstruction and Development
JICA	:Japan International Cooperation Agency
OECF	:Overseas Economic Cooperation Fund

(3) Foreign Terms

EIRR	:Economic Internal Rate of Return
FIRR	:Financial Internal Rate of Return
GDP	:Gross Domestic Product
GNP	:Gross National Product
GRP	:Gross Regional Product
PMF	:Probable Maximum Flood
NPV	:Net Present Value
O&M	:Operation and Maintenance
IEI	:Initial Environmental Evaluation
B/Q	:Bill of Quantities
TOR	:Terms of Reference

B/C :Box Culvert
CAD :Computer-aided Design
EIA :Environmental Impact Assessment
ICB :International Competitive Bidding
LCB :Local Competitive Bidding
JIS :Japan Industrial Standards
ASTM :American Society for Testing and Materials

(4) Numerical Units

Length

mm	millimeter
cm	centimeter
m	meter
km	kilometer

Weight

gr	gram
kg	kilogram
ton	metric ton

Area

mm^2	square millimeter
cm^2	square centimeter
m^2	square meter
km^2	square kilometer
ha	hectare

Time

sec	second
min	minute
hr	hour
yr	year

Volume

cm^3	cubic meter
m^3	cubic meter
Ltr	liter

Others

%	percent
℃	degree centigrade
10^3	thousand
10^6	million
10^9	billion

Money

Exchange Rate

Rp.	Indonesian Rupiah	Official rate as of June, 1997
¥	Japanese yen	US$ 1= Rp 2,350 = ¥ 115
US$	US dollar	

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Design Criteria

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1. GENERAL

This Design Criteria has been prepared to be applied to the Detailed Design for Urban Drainage Project in the City of Jakarta.

The design works are required to be accomplished under the internationally accepted codes and standards as well as the Indonesian codes, standards, and practices. From this point of view, the Design Criteria provides suitable and uniform standards and guideline for the detailed design works of drainage channel and related structures.

The detailed design contains a drainage channel and related structures such as levee, revetment, sluiceway, bridge and culvert.

The following codes and standards are referred to.

(1) Technical Standards for River and Sabo Facilities, Ministry of Construction Japan, 1985

(2) Structural Specification for Design and Construction of Concrete Structures, Part 1. Design, Japan Society of Civil Engineers, 1986

(3) Specifications and Guidelines for Roadway Bridge, Japanese Association of Highway, 1991

(4) Japanese Industrial Standards, Civil Engineering Materials and Products, Japanese Standard Association, 1991

(5) Standard Specification for Geometric Design of Urban Roads, January 1988, translated in English for "Standar Perencanaan Geometrik Untuk Jalan Perkotaan, 1988"

(6) Loading Specification for Highway Bridge Design Published by BINA MARGA, 1987, for "Pedoman Perencanaan Pembebanan Jembatan Jalan Raya SKBI 1.3.28. 1987"

(7) Bridge Design Code, 9 MAY 1992, for "Peraturan Perencanaan Tehnik Jembatan, 9 MAY 1992"

(8) Bridge Design Manual 2, September 1992, for "Panduan Perencanaan Tehnik Jambatan, 1992"

(9) Standard for Bridge Superstructure Prestressed Concrete Girder, T Type-A Class for "Standard Bangunan Atas Standar Bangunan Atas Jembatan Gelagar Beton Pratekan, Tipe T-Kelas A"

(10) Standard Specification of Pile Trestle Substructure for the Girder Span 11 M - 25 M, 1991, for "Spesificasi Pilar Dan Kepala Jembatan Sederhana Bentang 11 M - 25 M Dengan Pondasi Tiang Pancang Edisi Awal 1991"

(11) Specification & Standards for Reinforced Concrete Slab Highway Bridges No.02/1969, for "Spesifikasi Dan Standard Jembatan Pelat Beton Untuk Jembatan Jalan Raya No.02/1969"

(12) Pedestrian Bridge Standard issued by Road Institute, 1979

(13) AASHTO Standard Specification for Highway Bridges, 1992, as amended by interim Specification-Bridge, 1993 and 1994

(14) Standard Specifications of the American Society for Testing and Materials (ASTM)

2. DESIGN OF DRAINAGE STRUCTURES

2.1 Design Concept

2.1.1 Drainage channel

(1) Design stretch

The detailed design of drainage channels is carried out for the Kamal, the Tanjungan, the Gede/Bor, the Saluran Cengkareng, Pedongkeran and the PIK Junction drainage channels in the Cengkareng West area and one key drainage channel in the Meruya area, of which the locations are shown in Figs. 2.1 and 2.2. The length of the design stretch in each drainage channel is as shown below:

Drainage channel	Length (km)
1) Cengkareng West area	16.0 (2.8)
- Kamal (main)	7.3 (2.8)
- Kamal (branch)	2.8 (0)
- Tanjungan	2.5 (0)
- Gede/Bor	1.2 (0)
- Saluran Cengkareng	4.2 (0)
- PIK Junction	0.8 (0)
2) Meruya area	2.3 (0)

Note: (   ) = stretch in area acquired by private sactor

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

(2) Design flood discharge

The distribution of design flood discharges to be used for the detailed design of the drainage channels is illustrated in Figs. 2.3 and 2.4 and is summarized as follows :

Drainage channel	Design discharge(m^3/sec)
1) Cengkareng West area	
- Kamal	0.9 - 48.1
- Tanjungan	9.6 - 19.0
- Gede/Bor	13.8 - 16.9
- Saluran Cengkareng	5.8 - 18.8
- PIK Junction	7.1 - 18.1
- Pedongkerang	7.6 - 24.9
2) Meruya area	1.6 - 9.4

(3) Alignment of drainage channel

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

(a) The existing drainage channels shall be used, 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.

(b) The existing river structures such as revetments and bridges which are expected to satisfactorily function in future shall be used, with some repair works if necessary.

(4) Longitudinal profile of drainage channel

The longitudinal profiles of the drainage channels shall be determined based on the following :

(a) A 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.

(b) A new channel bed gradient is designed so as not to be remarkably different from an present one, in principle.

(5) Design high water level

In principle, the design high water levels of the drainage channels should be equal to or lower than the original ground elevations or the ground elevations for the land reclamation areas. The design high water levels will be determined, through non-uniform flow calculation under the following conditions :

(a) The initial water levels for the Kamal and the Tanjungan drainage channels which discharge to Jakarta Bay is set at the highest higher tidal level of TTG System.

(b) The initial water level for the Saluran Cengkareng drainage channel which discharges to Cengkareng Floodway is set at the 25 year flood water level in the floodway at the drainage channel outlet.

(c) The initial water levels for other drainage channels are set at the water level estimated by uniform flow calculation.

(d) The roughness coefficient is 0.03 in drainage channels of which the channel slopes are protected on both sides by masonry revetment and 0.016 in concrete culvert.

(6) Cross section of drainage channel

The drainage channels are to be improved by widening the existing channel and constructing new levees. The cross sections of the drainage channels shall be determined in consideration of effective use of existing structures in and around the drainage channel such as revetments, bridges, roads, electric poles and telephone cable ducts in order to minimize compensation of the structures.

The cross section will be determined as follows:

(a) The new levees along the Tanjungan drainage channel will be constructed in the north part of Jalan Tol. Prof. Sediyatmo by dump-fill of earth materials to be obtained from excavation of the channel and in near borrow areas, since the water levels outside the channel are the same as the inside one and the levee is always saturated by water to the upper part. The side slope of the levee shall be 1:3.0.

(b) The new levees except (a) above shall be of a earth embankment type and their side slopes are to be 1:2.0 .

(c) Open culvert covered with steel mesh cover is applied to densely populated stretches of the Saluran Cengkareng drainage channel to avoid throwing dust and garbage into the channel and to keep clean water. At the upstream inlet of the open culvert, screen is provided to prevent the dust and garbage from flowing into the culvert. The stretch of the open culvert is about 400 m long in the downstream stretch of the channel.

(d) A trapezoid shape channel protected by wet masonry revetments is applied to the channels except for the stretches designated in (a), (b) and (c) above, in consideration of land and house compensation, maintenance of the side slope of the channels and convenience of water use for inhabitants. The side slope of the revetment shall be 1:0.5 .

(e) The freeboard shall be provided above the design high water level, as follows:

Drainage channel	Freeboard(cm)
1) Cengkareng West area	
- Kamal	50 (20 - 30)
- Tanjungan	40
- Gede/Bor	40
- Saluran Cengkareng	30 - 40
- PIK Junction	30
2) Meruya area	40 - 20

Note: ( ) for branch channel

2.1.2 Levee

It is required to construct new levee along the drainage channels in low land areas to meet a design high water level. 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 densely populated housing areas. The design criteria of these levees are as stated below.

(1) Earth type levee

(a) The clayey and silty earth materials which are obtained from excavation of the existing drainage channels or in near borrow areas shall be used as embankment material. Accordingly, homogeneous type levee is constructed.

(b) The levee shall be of a trapezoid shape with 3 m in crest width and its side slope shall be 1:2.0 in principle. However, the crest shall be 5 m wide, in case that the crest is used as an inspection road. The shape shall be verified by stability analysis in due consideration of physical properties of available earth materials.

(c) Two(2) m wide berm shall be provided 3 m below the levee crest on the land side slope.

(d) The foundation area of the levee shall be stripped up to 20 cm in principle. The depth to be stripped for the foundation area where the soil condition is unfavorable for levee construction shall be determined in consideration of actual soil characteristics.

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

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

The standard design is shown in Fig. 2.5.

(f) The slope of levee where revetment works are not provided will be protected by sod facing against local erosion due to rainfall and stream flow.

(2) Design of parapet wall

Parapet wall as shown in Fig 2.5 shall be designed.

The parapet wall is designed in the following stretches which encounter difficulty in construction of earth levee due to constraint in land acquisition.

i) Stretches in densely populated areas, and
ii) Stretches having narrow space between channel bank and existing road.

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

(a) Foundation Type

Spread foundation and pile foundation are conceivable as the foundation type of the parapet wall and the following criteria are applied for selection :

Type	Criteria
Spread foundation	Qmax ≤ 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)

(b) Cutoff wall of steel sheet pile

Cutoff wall is required at the foundation of parapet wall to prevent piping phenomena due to seepage water in the foundation during flood. The required length is calculated by applying the Lane's formula, which is as 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 of water levels between channel side and land side (m)

2.1.3 Revetment

Revetment shall be constructed to stabilize the slope of the drainage channels and to prevent slope erosion of earth type levee from turbulent flow of flood. Appropriate types of revetment shall be adopted in consideration of a roughness coefficient of channel bed, stability of channel slope, magnitude of flood flow velocity and available construction materials. For slope protection of the drainage channels and earth levee, wet cobble masonry revetment is employed in this design.

In case of the drainage channels, their slopes shall be protected in the whole design stretches. In case of the earth levee, the slope shall be protected at :

i) Concave sections, to be protected from direct attack of flood flows,
ii) Stretches where the levee is closely located to the drainage channel, to be protected from erosion due to turbulent flow at top of drainage channel revetment.
iii) Connection part of earth levee with parapet wall to be protected from erosion caused by turbulent flow due to change of channel cross section and roughness coefficient, and
iv) Upstream and downstream areas of the bridge and sluiceway construction sites.

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

(1) Slope protection

The revetment shall be provided at least up to the design high water level or to the top of drainage channel, depending on the local flow condition.

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

(2) Toe protection and foot protection

In order to protect the toe of the wet cobble masonry from local scouring or degradation at the channel bed, the toe protection by foot concrete shall be provided. The depth of the foot concrete shall be 0.5 m. The foot concrete shall be provided 1.0 m below the design bed in the drainage channel and 0.5 m below the original ground surface for levee revetment, as shown in Fig 2.6.

The toe portion of the wet masonry shall be also protected against the scouring and degradation, and gabion mattress shall be provided as foot protection works in front of the toe protection.

The length of foot protection to be provided perpendicular to the flow direction depends on the height of wet masonry revetment and flow velocity. The following lengths of foot protection are used :

Water depth during flood (m)	Flood flow velocity (m/sec)	
	less than 2	2 - 4
Less than 5	1.5 m	3 m
More than 5	3m	6 m

(3) Structural design of revetment

The wet masonry revetment shall be designed considering the roughness of channel bed material, hardness of channel slope, strength of flow force and available construction material. In this design, the wet masonry shall be designed to be 25 - 35 cm in thickness with cobble stone of 20 to 25 cm in diameter, and 20 cm thick gravel bedding. The contraction joint shall be provided for longitudinal direction at an interval of 10 m.

A weep hole, one (1) hole per 4 m^2, shall be provided in the revetment, because remaining water pressure behind the revetment is expected high, particularly in an excavated channel case.

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 shall be provided at the foot concrete.

2.1.4 Sluiceway

The existing sluiceway which is in operation with two (2) slide gates (approx. 2.3 m x 2.3 m) at the outlet of the Saluran Cengkareng drainage channel shall be modified due to improvement of the drainage channel.

On the other hand, the inland rain water confined due to construction of new levee shall be drained to the drainage channel through sluiceways.

(1) Location of sluiceway

The locations of sluiceways shall be determined based on the following consideration:

(a) The location of the sluiceway is to be the same as the present drainage canal site in principle. However, the location shall be determined so as no to impair the function of the existing and new facilities.

(b) The number of sluiceways shall be minimized in consideration of integration of drainage areas to avoid impairment in levee stability.

(c) The longitudinal direction of the sluiceway shall be aligned perpendicular to the axis of the levee to minimize the length underneath the levee and to simplify the structure.

(2) Hydraulic design

(a) The sluiceway structures shall be designed under the hydraulic conditions of the drainage channel with the scale of 10-year probable flood.

(b) Ten (10) year probable flood in the drainage area shall be applied as a conduit capacity under the condition of future land use. The probable flood is estimated by use of the Rational formula.

(c) The discharge capacity of the conduit is estimated by means of uniform flow calculation as given below.

$Q = V \times A$

$V = (1/n) \times R^{2/3} \times I^{1/2}$

where,
Q : discharge (m^3/sec)
A : flow area in conduit (m^2)
n : Manning's roughness coefficient (0.016 for concrete conduit)
R : hydraulic radius (m)
I : gradient of conduit

(3) Structural design

(a) Conduit

(i) A box culvert type and a circular type can be applied for sluiceway conduit design.

(ii) Standard design of the circular type sluiceway is shown in Fig. 2.7. On the other hand, a maximum size of a box culvert type conduit shall be limited to 2.3 m x 2.3 in consideration of manual gate operation

(iii) The number of conduits shall be determined so as not to remarkably reduce the flow area of the drainage channel to be connected.

(b) A contraction joint shall be provided in the conduit, if its length exceeds 20m in terms of uneven settlement of foundation.

(c) Foundation of the sluiceway shall be designed so as to safely transmit upper loads to stiff subsoil. To avoid leakage through the foundation, base concrete of 10 cm shall be provided.

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

(e) Two (2) gate types, i.e. slide gate and flap gate, shall be provided for the sluiceways to prevent drainage channel waters from inundating in the inland area during floods, in case that the conduit is lower than the design high water level of the drainage channel at the sluiceway.

(i) The slide gate shall be designed so as to allow manual operation.

(ii) The flap gate shall be applied in case that the design flood discharge is less than 0.2 m^3/sec/conduit and no serious inundation damage is presumed in malfunction of the gate

(4) Side ditch

Side ditch shall be provided along the levee on the land side if necessary to lead inland rain water to the sluiceway site. The side ditch shall be designed as a concrete-lined ditch.

(5) Revetment on the levee at sluiceway site

The revetment shall be provided on the earth levee at the sluiceway site to prevent local scouring thereon. The revetment shall be designed with a 10 m length in the upstream and downstream reaches of the sluiceway.

(6) Cut-off wall of steel sheet pile

The safety of sluiceway foundation shall be 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 soil of the foundation subject to examination is assessed based on the results of the geotechnical investigation conducted in this project.

The examination shall be made by using the Lane's formula. If the piping is assessed to occur, steel sheet piles shall be provided at several locations of the conduit bottom.

(7) Foundation

The spread foundation and pile foundation shall be applied based on the allowable bearing capacity of foundation. This allowable bearing capacity shall be estimated based on the soil properties at the sluiceway site, which are to be determined on the basis of the geotechnical investigation results of core drilling with standard penetration test and Dutch cone penetration tests conducted in the Project.

The pile foundation shall be provided in case that the estimated allowable bearing capacity of foundation is assessed to be insufficient to sustain the total design loads to be transmitted from the sluiceway side. The RC piles with 250 x 250 mm and 300 x 300 mm shall be applied for pile foundation in principle.

2.1.5 Inspection road

Existing roads along the proposed drainage channels can act as inspection roads not only for routine operation and maintenance works of such drainage facilities as drainage channels, levees, revetments and sluiceways but for periodical inspection of the facilities. In case that no existing road is available for the inspection road s along the proposed drainage channels, inspection roads at least 5 m wide will be provided on the crests of the newly constructed earth type levees or along the drainage channels.

2.1.6 Land Subsidence

Land subsidence is a serious problem in the Cengkareng west area. Extent of the land subsidence for 20 years since 1974 has been estimated by DKI Jakarta by means of leveling survey of existing bench marks in PP System. Based on the result of the leveling survey, extent of subsidence per year was estimated as follows :

- Kamal drainage at Jalan Tol. Prof. Sediyatmo highway crossing: 6cm/year
- Tanjungan drainage at Jalan Tol. Prof. Sediyatmo highway crossing: 6cm/year
- Bor drainage at confluence with Mookervaat channel: 8cm/year
- Saluran Cengkareng drainage at confluence with Cengkareng Floodway: 6cm/year

It has been presumed that subsidence takes place due to two parameters, namely settlement due to heavy burden in shallow zone and self consolidation due to pumping up of ground water in deep zone. However, the drainage facilities are designed with no allowance for the land subsidence, since the capacity and freeboard of the drainage channel and culvert can be ensured by additional heightening of the levee and heightening of the upper portion of the culvert structure .

2.2 Structural Design

2.2.1 Design loads

Drainage structures are designed so as to sustain the following loads and their combination as required :

- (i) Dead load
- (ii) Live load
- (iii) Earth pressure
- (iv) Hydraulic pressure
- (v) Uplift
- (vi) Seismic load

(1) Dead Load

Dead load consists of weights of structures and is computed by using the following unit weights of construction materials:

Material		Unit weight(ton per m^3)
Concrete	(plain)	2.2
	(reinforced)	2.5
Mortal		2.15
Brick masonry		2.0
Stone masonry		2.0
Timber		1.0
Soil	(dry)	1.6
	(wet and saturated)	1.8
Sand/gravel		1.9
Water		1.0
Structural steel		7.85
Cast iron		7.25
Asphalt pavement		2.3

(2) Live Load

The live loads dealt with herein are as itemized below:

(i) Vertical load acting on top slab of conduit embedded in embankment, which results from the live load on the embankment.

(ii) Horizontal load acting on side wall of conduit embedded in embankment or retaining wall, which results from the live load on the embankment or backfilling of a retaining wall.

(iii) Live load on sluiceway deck

(a) Vertical load on top slab of conduit

The vertical load is computed as a distributed load given by the following formula:

$$We = \frac{2P(1+i)}{a \times b} \qquad \text{(in case } b \geqq B\text{)}$$

$$We = \frac{2P(1+i)}{a} \times \frac{2B-b}{2B^2} \qquad \text{(in case } b < B\text{)}$$

$a = 2\,ha + 2.25$

$b = 2\,ha + 0.20$

where;

We : Distributed load (t/m^2)
a : Length of distributed load (m)
b : Width of distributed load (m)
B : Width of conduit (m)
P : Rear wheel load of vehicle on embankment (t)
ha : Height of embankment above conduit (m)
i : Impact coefficient as determined as follows ;

i	ha(m)
20/(50 + B)	$ha \leqq 0.6$
0.3	$0.6 < ha \leqq 1.2$
0	$1.2 < ha$

The above formula is additionally explained in Fig. 2.8.

(b) Horizontal load on side wall of conduit or retaining wall

The horizontal load is computed as a distributed load given by the following formula:

$$Pe = K \times q$$

$$q = W/(L \times B)$$

where;

Pe : Distributed Load (t/m^2)
K : Coefficient of earth pressure
q : Converted uniform load (t/m^2)
W : Total weight of vehicle (t)
L : Length of vehicle (m)
B : Width of vehicle (m)

For the coefficient of earth pressure, the paragraph (3) below shall be referred. The above formula is additionally explained in Fig. 2.9.

(c) Live load on sluiceway deck

The Live load is an uniform load of 250 kg/m^2 for deck for passage.

(3) Earth Pressure

(a) Active earth pressure

This pressure is lateral earth pressure acting on a structure which is deflected in the direction of this pressure. This earth pressure is employed for design of a retaining wall etc. and is calculated by use of Coulomb's formula expressed as follows:

$$Pa = 1/2 \times \gamma \times H^2 \times Ka - 2C(\sqrt{Ka}) \times H$$

$$Ka = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha \times \cos(\alpha + \delta)\{1+(\sqrt{\frac{\sin(\phi + \delta) \times \sin(\phi - B)}{\cos(\alpha + \delta) \times \cos(\alpha - B)}})\}^2}$$

Where;

Pa : Active earth pressure (t/m)

Ka : Coefficient of active earth pressure
γ : Unit weight of earth material behind wall (t/m^3)
H : Height of wall (m)
ϕ : Internal friction angle of earth material (degree)
δ : Friction angle of soil against back surface of wall (degree)
α : Angle between back surface of wall and vertical plane
C : cohesion of soil (t/m^2)
B : Angle between ground surface behind wall and horizontal plane (degree)

(b) Passive earth pressure

This pressure is lateral earth pressure acting on a structure which is forced to be deflected in the converse direction of this earth pressure. This earth pressure is employed for sliding check of the structure and is calculated by use of Coulomb's formula expressed as follows.

$$Pp = 1/2 \times \gamma \times H^2 \times Kp + 2C(\sqrt{Kp}) \times H$$

$$Kp = \frac{\cos^2(\phi + \alpha)}{\cos^2 \alpha \times \cos(\alpha + \delta)\{1-(\sqrt{\dfrac{\sin(\phi - \delta) \times \sin(\phi + B)}{\cos(\alpha + \delta) \times \cos(\alpha - B)}}))^2}$$

Where;

Pp : Passive earth pressure (t/m)
Kp : Coefficient of passive earth pressure

The notations for other variables are the same as those for the formula for active earth pressure.

(c) Earth pressure at rest

This pressure is lateral earth pressure acting on a structure which is rigidly fixed and not defected by this earth pressure. This earth pressure is employed for design of underground structures and is calculated by use of the following formula:

$$Ps = 1/2 \times \gamma \times H^2 \times Ks$$

$$Ks = 0.5$$

Where;

Ps : Earth pressure at rest (t/m)
γ : Unit weight of earth material behind wall (t/m^3)
H : Height of wall (m)
Ks : Coefficient of earth pressure at rest

(d) Earth pressure in seismic condition

Earth pressure in seismic condition is calculated by the Mononobe-Okabe's formula based on the Coulomb's theory. This formula is given as follows:

(i) Active earth pressure

$$Pea = 1/2 \times \gamma \times H^2 \times Kea - 2C(\sqrt{Kea}) \times H$$

$$Kea = \frac{\cos^2(\phi - \theta o - \alpha)}{\cos\theta o \times \cos^2\alpha \times \cos(\alpha + \theta o + \delta)\{1 + (\sqrt{\dfrac{\sin(\phi + \delta) \times \sin(\phi - B - \theta o)}{\cos(\alpha + \theta o + \delta) \times \cos(\alpha - B)}})\}^2}$$

(ii) Passive earth pressure

$$Pep = 1/2 \times \gamma \times H^2 \times Kep + 2C(\sqrt{Kep}) \times H$$

$$Kep = \frac{\cos^2(\phi - \theta o + \alpha)}{\cos\theta o \times \cos^2\alpha \times \cos(\alpha - \theta o + \delta)\{1 - (\sqrt{\dfrac{\sin(\phi - \delta) \times \sin(\phi + B - \theta o)}{\cos(\alpha - \theta o + \delta) \times \cos(\alpha - B)}})\}^2}$$

Where:

Pea : Active earth pressure in seismic condition (t/m)
Kea : Coefficient of seismic active earth pressure
Pep : Passive earth pressure in seismic condition (t/m)
Kep : Coefficient of seismic passive earth pressure
θo : Angle expressed as follows (degree)

$$Kep = \frac{Kh}{1-Kv}$$

Kh : Seismic coefficient in horizontal direction
Kv : Seismic coefficient in vertical direction (=0)

The notation for other variables is same as that for formula for active earth pressure. Lateral earth pressures due to earthquake are to be calculated by the use of Mononobe-Okabe formula based on the Coulomb's theory taking into account seismic factors.

(4) Hydraulic pressure

(a) Hydrostatic pressure

The hydrostatic pressure is calculated by use of the following formula:

$$Pw = 1/2 \times Wo \times H^2$$

where;

Pw : Hydrostatic pressure (t/m)
Wo : Unit weight of water (t/m^3)
H : Water depth (m)

(b) Dynamic water pressure due to flowing water

The dynamic water pressure acting on a structure in flowing water is calculated by use of the following formula:

$$P = K \times v^2 \times b \times H$$

Where;

P : Dynamic water pressure due to flowing water (t)
K : Resistance coefficient depending on figure of structure as shown in Fig. 2.10.
v : Velocity of flowing water (m/s)
b : Width of structure in direction perpendicular to flow direction (m)
H : Water depth (m)

The working point where the dynamic water pressure is concentrated is assumed as follows:

$$h = 0.6 H$$

where;

h : Height of working point above bottom (m)
H : Water depth (m)

(c) Dynamic water pressure caused by earthquake

(i)Dynamic water pressure on wall

This dynamic water pressure shall be calculated by following formula

$$P = 7/12 \times Kh \times Wo \times b \times H^2$$

Where ;

P : Dynamic water pressure (t)
Kh : Seismic coefficient in horizontal direction
Wo : Unit weight of water (t/m^3)
b : Width of structure in direction perpendicular to direction of earthquake (m)
H : water depth (m)

The working point is assumed as follows:

$$h = 0.5 H$$

Where;

h : Height of working point above bottom (m)

(ii)Dynamic water pressure on column

This dynamic water pressure and the working point are calculated by use of the following formulas. The notation for these formulas is same as that for formula for dynamic water pressure acting on wall.

$$P = 3/4 Kh \times Wo \times b^2 \times H(1\text{-}b/4H) \qquad (b/H \leqq 2.0)$$

$$P = 3/8 Kh \times Wo \times b^2 \times H \qquad (2.0<b/H \leqq 3.1)$$

$$P = 7/6 Kh \times Wo \times b^2 \times H \qquad (3.1<b/H)$$

$$h = 0.5 H$$

(5) Uplift

Uplift pressure is considered for design of a structure which is fully or partially submerged. The uplift pressure has an upward direction and its magnitude is equivalent to the water depth above the bottom of the structure.

(6) Seismic Force

In the design of drainage structures of the project except for underground structures, the seismic force is considered. The seismic force is estimated by the seismic coefficient method in which the seismic force is calculated by multiplying the weight of a structure by a seismic coefficient. The seismic coefficient is taken at 0.075 for horizontal direction and 0.0 for vertical direction.

2.2.2 Allowable stress

(1) Concrete

Concrete is classified as described in paragraph 2.2.3 (1) and the allowable stress is as follows :

(Unit : kg/cm^2)

Kind of concrete	Flexural compressive Stress	Flexural tensile stress	Shear stress	Bond stress
Type 1	90	0	4.7	8.5 (Round bar) 17 (Deformed bar)
Type 2	80	0	4.5	8 (Round bar) 16 (Deformed bar)
Type 3	60	0	4.0	7 (Round bar) 14 (Deformed bar)
Type 4	60	0	4.0	7.5 (Round bar) 14 (Deformed bar)
Type 5	35	0	-	-

(2) Reinforcement Bar

The allowable stress of reinforcement bar shall be as follows;

Allowable tensile stress and compressive stress..............1400 kg/cm^2

(3) Masonry and brick

The allowable compressive stresses of masonry and brick is respectively 15 kg/cm^2 and

8 kg/cm^2. No other allowable stress is considered for masonry and brick.

(4) Increase of allowable stress

The allowable stresses shall be increased by 50 % under load conditions during flood, earthquake and construction.

2.2.3 Reinforced concrete structure

(1) Concrete classification

Concrete is classified into six types as follows:

Type	Application
Type 1	Structure requiring high strength with relatively thin member such as RC pile.
Type 2	Bridge slab, pier, abutment, guard rail
Type 3	Secondary concrete or relatively thin structural member such as block out of sluiceway, concrete pipe, etc
Type 4	ordinary structural concrete with relatively thick member such as sluiceway, foundation concrete of revetment, concrete culvert, parapet wall, curb concrete, side walk of bridge, etc.
Type 5	Base concrete, leveling concrete

(2) Bar arrangement

Reinforcement bars are required from two aspects. One is to sustain tensile force acting in concrete structures or to compensate insufficiency in compressive strength of concrete structures. The other is to prevent occurrence of crack on concrete surface due to shrinkage of concrete.

To sustain the tensile force or to compensate insufficiency in compressive strength, the reinforcement bar is designed by use of the following formulas :

The notation for the above variables is illustrated in Fig. 2.11.

$$As' = \frac{M - (\sigma ca \cdot b \cdot x/2)(d - x/3)}{(d - d')\sigma s'}$$

$$As = \frac{1}{2}\frac{\sigma ca}{\sigma sa} b \cdot x + As' \frac{\sigma s'}{\sigma sa}$$

$$x = \frac{n \cdot \sigma ca}{n \cdot \sigma ca + \sigma sa} d$$

$$s' = n \cdot \sigma ca \frac{x - d'}{x}$$

where;

As' : Area of reinforcement bar on compression side (cm^2)

As : Area of reinforcement bar on tension side (cm^2)

M : Bending moment (kg .cm)

σ ca : Allowable flexural compressive stress of concrete (Kg/cm^2)

b : Width of member (cm)

d : Height of member (cm)

σsa : Allowable tensile stress of reinforcement bar (Kg/cm^2)

d' : Distance between reinforcement bar center and member edge on compression side (cm)

n : Ratio of elastic modulus of reinforcement bar to elastic modulus of concrete

The stresses of concrete and reinforcement bar are calaculated by use of the following formulas:

$$\sigma c = \frac{2M}{b \cdot x\,(d - x/3) + 2 \cdot n \cdot As' \;\{(x - d') \,.\, (d - d')/x\}}$$

$$\sigma s = n \cdot \sigma c \frac{d - x}{x}$$

$$\sigma s' = n \cdot \sigma c \frac{x - d'}{x}$$

$$x = -\frac{n\,(As + As')}{b} + \sqrt{\,}\{(\frac{n\,(As + As')}{b}\}^2 + \frac{2n}{b}(As \cdot d + As' \cdot d')\}$$

$$= 8/7 \cdot S/b \cdot d$$

where;

σc : Compressive stress o f concrete ((kg/cm^2)

σs : Tensile stress of reinforcement bar arranged on tension side of member (kg/ cm^2)

$\sigma s'$: Compressive stress of reinforcement bar compression side

τ : Shear stress of concrete (kg/ cm^2)

The notation for other variables in the above formula is the same as that of the formula for calculation of reinforcement bar area.

(3) Details of Bar Arrangement

Details of bar arrangement follow the following in principle.

(a) Spacing

(i) Minimum space for slab and beam

- More than diameter of reinforcement bar.
- More than four third of maximum size of aggregates.
- More than 3.0 cm.

(ii) Minimum space for wall

- More than 1.5 times of diameter of reinforcement bar.
- More than four third of maximum size of aggregates.
- More than 5.0 cm.

(iii) Maximum space for slab

- 20 cm or 2 times slab thickness for section where the maximum bending moment occurs and 40 cm or 2 times slab thickness for other section.

(iv) Maximum space for beam

- Horizontal maximum space is 15 cm.
- Vertical maximum space is 30 cm and also less than the beam width.

(v) Maximum space for wall

- Vertical maximum space is 25 cm and also less than 1.5 times the wall thickness.
- Horizontal maximum space is 40 cm and also less than 3 times the wall

thickness.

(b) Concrete cover

The minimum cover for reinforcement bar arrangement shall be as follows and also more than the bar diameter.

(unit ; cm)

Member	Case 1	Case 2	Case3
Slab	1.0	1.5	2.0
Wall	1.5	2.0	2.5
Beam	2.0	2.5	3.0

Note ;

Case 1 : The concrete surface is in conditions other than those of cases 2 and 3 below.

Case 2 : The concrete surface is exposed to sever weathering of flowing water.

Case 3 : The concrete surface is in soil.

(c) Splices

Splices of reinforcement bar shall comply with the following stipulations.

(i) Lap splices is applied in case the bar diameter is less than 30 mm and welding splices are applied in other cases. The efficiency in welding is 80%.

(ii) The bar length for lap splices is more than 20 times the bar diameter and also more than the length given by use of the following formula:

In with hook case,

$$L = \frac{\sigma sa}{6\ \tau oa} \phi$$

In without hook case,

$$L = \frac{\sigma sa}{4\ \tau oa} \phi \text{ (tension side)}$$

$$L = \frac{\sigma sa}{5\ \tau oa} \phi \text{ (compression side)}$$

Where;

L : Length of lap splices (cm)
σsa : Allowable tensile stress of bar (kg/cm^2)
τoa : Allowable bond stress of concrete (kg/cm^2)
ϕ : Diameter of bar (cm)

(d) Anchorage

Anchorage of reinforcement bar shall comply with the following stipulations.

(i) The bar length required for anchorage shall be the same as that for lap splices.

(ii) More than one third out of bars on the tension side shall be extended and anchorage beyond the fulcrums of the member.

(iii) In case a round bar is used, a semi-circular hook shall be provided for anchorage.

(4) Joint of concrete structure

(a) Construction joint

Construction joints shall be provided in accordance with the requirement in site work of concrete placement and the location and the detail of this joint shall not be shown in the design drawing.

(b) Contraction joint

Contraction joint shall be provided in order to prevent occurrence of crack due to concrete shrinkage and also to prevent crack due to such structural causes as uneven settlement of foundation, vibration etc. The location and interval of this joint and also the continuity of reinforcement bar at this joint shall be determined in consideration of these situations. The location and the detail of bottom collar, joint bar and water stop at this joint shall be determined considering foundation condition, hydraulic condition and importance of structure.

The provision for the contraction joint of major structures in the project is as follows.

(i)Sluiceway conduit

The interval of the joint is approximately 10 m. Reinforcement bars are continued at the joint. A bottom collar, joint bars and water stop shall be arranged at the joint.

(ii)Parapet wall and open culvert

The interval of the joint is 5 m for the parapet walls and 8 m for the open culvert/concrete ditches. The reinforcement bars shall not be continued and joint bars and water stop shall be provided at the joint.

(iii)Revetment

The interval of the joint is approximately 6 m and joint filler shall be provided at the joint.

2.2.4 Stability of concrete structures

(1) Overturning

For safety against overturning, the following condition shall be satisfied.

$e \leq B/6$ (in normal condition)
$e \leq B/3$ (in flood, seismic and construction conditions)

where;

e : Eccentricity (m)
B : Width of footing in overturning direction (m)

(2) Sliding

For safety against sliding, the following condition shall be satisfied:

$$Fs = \frac{C \times A + N \times F}{H}$$

where ;

Fs : Minimum safety factor 1.5 in normal condition and 1.2 in flood, seismic and construction conditions
C : Adhesion between footing and foundation, which is equivalent to the cohesion of foundation material (t/m^2)
A : Area of footing where the reaction of foundation acts on m^2
N : Total vertical force (t)
f : Friction coefficient between footing and foundation
H : Total horizontal force(t)

(3) Bearing pressure

The bearing pressure as the result of foundation reaction shall be calculated by use of the following formula:

$$q1,2 = \frac{N}{B \times L}(1 + \frac{6\,e}{L}) \qquad (\text{in case } e \leq \frac{B}{6})$$

$$q\,max = \frac{2N}{3\,L\,(B/2\text{-}e)} \qquad (\text{in case } e > \frac{B}{6})$$

where ;

e : Eccentricity (m)
N : Total vertical force (t)
B : Width of footing (m)
L : Length of footing (m)
q1,2 and q max : Reaction of foundation(t/m^2)

2.2.5 Stability of levee

(1) Stability of levee against sliding

Stability of levee against sliding decreases when high water stage lasts for a long time, shear stress of earth material decreases due to water seepage and the unit weight of the earth material increases. The stability of levee depends largely on the soil condition of the levee foundation. In order to ensure the stability of levee against sliding, the stability analysis shall be made, incorporating the condition of levee foundation.

The soil conditions are as follows, which are results of the geotechnical investigation performed in this project.

	Density(t/m^3)		Shear strengh (t/m^2, degree)							
	γ_t	γ_{sat}	Cu	ϕ_u	Ccu	ϕ_{cu}	Cd	ϕ_d	C'	ϕ'
Foundation	1.7	1.7	4	8	5	15	4.5	25	4.2	27
Embankment	1.8	1.8	4	27	-	-	-	-	-	-

To carry out the analysis calculation against sliding, friction circle method and slice method are conceivable. Out of them, slice method is employed because the site condition of levee foundation cannot meet the requirement for the calculation condition for friction circle method.

The slice method is expressed as follows :

$$F.S. = \frac{\Sigma \{ C \times l + (W - U \times b) \cos\theta \tan \phi \}}{\Sigma (W \sin\theta)}$$

where :

F.S : Safety factor > 1.3 for normal condition
> 1.1 for flood and seismic conditions
C : Cohesion (t/m^2)
ϕ : Internal friction angle (degree)
l : Arc length of sliding circle (m)
W : Weight of slice (t)
U : Pore pressure (t/m^2)
θ : See Fig. 2.12
b : Width of slice (m)

(2) Stability of levee against settlement, seepage and leakage

The stability of levee foundation against the settlement which may take place due to consolidation after levee embankment shall be examined.

The settlement of the levee foundation shall be initially calculated for the layers classified in accordance with soil condition of the levee foundation and total settlement shall be calculated by summing the values for each layer. The settlement for each layer shall be calculated by use of the following formula.

$$Sc = \frac{eo - e}{1 + eo} H$$

where :

Sc : Settlement from primary consolidation (cm).
eo : Initial void ratio
e : Void ratio after embankment
H : Thickness of compressible stratum (cm)

The extent of the extra embankment, which is specified in the previous paragraph shall be examined on the basis of the result of the settlement calculation.

The cross section of levee shall be designed so as to keep the seepage line within the levee body during flood peak. The seepage line shall be obtained by Casagrande's method as shown below:

$$X = (Y^2 - Yo^2)/2Yo$$

$$Y = (2Yo \cdot X + Yo^2)^{1/2}$$

$$Yo = (h^2 + d^2)^{1/2} - d$$

where :

h : Distance between A and B
d : Distance between B1 and A
12 : Distance between C and E
11 : Distance between A and C

The parameter necessary for obtaining the seepage line are illustrated in Fig. 2.13.

The seepage line shall be obtained by initially drawing the line on the deformed section which is prepared by reducing the horizontal dimension by the ratio of kv (coefficient of permeability in vertical direction) to kh (coefficient of permeability in horizontal direction), and then by plotting the result on the original section. For the figures of kv and kh, 1/5 and 1/25 are used for use of a tamping roller and a tire roller, respectively.

2.2.6 Design of foundation

(1) Spread foundation

Allowable bearing capacity of spread foundation is calculated by dividing the ultimate bearing capacity of the foundation by a safety factor, which is 3 for normal condition and 2 for flood, seismic and construction conditions.

The ultimate bearing capacity is calculated by use of the following formula:

$$Qu = A'(\alpha \cdot k \cdot c \cdot Nc + k \cdot q \cdot Nq + 1/2\, \gamma_1 \cdot \beta \cdot B' \cdot Nr)$$

Where;

Qu : Ultimate bearing capacity (t)
A' : Effective area of footing bottom as illustrated in Fig. 2.14.
α, β : Coefficients depending on shape of footing which are shown below:

Shape of footing	Excessively long rectangle	Circle or square	Rectangle or ellipse
α	1.0	1.3	$1 + 0.3 \dfrac{B'}{L'}$
β	1.0	0.6	$1 - 0.4 \dfrac{B'}{L'}$

Remakes ; In case B'/L' is bigger than 1.0, B'/L' shall be taken at 1.0.

B',L' : Width and length of A', as shown in Fig. 2.14 (m)
γ 1, γ 2 :Unit weights of foundation materials above and below bottom of footing (t/m^3).
e : Eccentricity, as shown in Fig. 2.14 (m)
Df : Depth from ground surface to bottom of footing (m).
K : Coefficient expressed as follows:

$$K = 1 + 0.3\frac{Df}{B'}$$

Q : Pressure expressed as follows (t/m2)

$$q = \gamma_2 \cdot Df$$

(2) Pile foundation

(a) General

The design of pile foundation is made on the assumption that the force acting on the footing is exclusively sustained by piles and no force is transmitted from the footing to the ground adjacently underneath the footing.

The arrangement of piles shall be made so that the force transmitted from the footing is distributed to each pile as evenly as possible .

The distance between the centers of piles neighboring each other shall be at least 2.5 times the pile diameter, in principle. In case piles meet the requirement, the bearing capacity of the pile foundation is calculated by accumulating bearing capacity of each pile. In other cases, the bearing capacity shall be calculated as the bearing capacity of a mass body composed of piles and foundation material among the piles. This mass body is referred to as the pile cluster explained hereinafter.

The distance between the footing edge and the pile center shall be at least 1.25 times the pile diameter for a driven pile and 1.0 time for a cast-in-situ pile.

(b) Bearing capacity per one pile

(i)Bearing capacity for intrusive force

Allowable bearing capacity for intrusive force is calculated by use of the following formula:

$$Ra = 1/n\,(Ru - Ws) + Ws - w$$
$$Ru = qd \times A + \pi \times D \times \Sigma(li \times fi)$$

where ;

Ra : Allowable bearing capacity for intrusive force (t)
n : Safety factor, as stipulated below:

	Bearing pile	Friction Pile
Normal condition	3	4
Seismic condition	2	3

Ru : Ultimate bearing capacity of pile (t)

Ws : Weight of foundation material replaced by pile body (t)

W : Weight of pile body including material in pile body (t)

qd : Ultimate bearing capacity of foundation stratum at tip of pile expressed as follows (t/m2):

- For piles other than open - tip steel pile

$$qd = N (4 \frac{Df}{D} + 10) \leq 30$$

- For open - tip steel pile

$$qd = 6 N \frac{Df}{D} \leq 30$$

Where;

Df : Driven depth into stratum at tip of pile (m)

D : Diameter of pile (m)

N : "N" Value of foundation stratum at tip of pile

A : Sectional area of pile (m2)

li : Thickness of stratum i (m)

fi : Skin friction given i, as follows (t/m2):

Foundation material	Driven pile	Cast-in-situ pile
Sandy soil	0.2 N (≤ 10)	0.5 N (≤ 20)
Clayey soil	C or N (≤ 15)	C or N (≤ 15)

Note : In case N < 2, the adhesion shall be neglected.

Where;

C : Cohesion of soil (t/m^2)

(ii)Bearing capacity for extractive force

Allowable bearing capacity for extractive force is calculated by use of the following formula:

$$Pa = 1/n \times Pu + w$$

$$Pu = \pi \times D \times \Sigma(li \times fi)$$

Where;

Pa : Allowable bearing capacity for extractive force (t)

n : Safety factor as stipulated below:

Condition	Safety factor
Normal condition	6
Seismic condition	3

Pu : Ultimate bearing capacity owing to skin friction between pile and foundation strata (t).

The notation for D, li, fi and W is the same as that for the formula for intrusive force

(iii)Bearing capacity for lateral force

Allowable bearing capacity for lateral force acting perpendicularly to a pile axis is calculated by use of the following formula:

- For pile of which the head is embedded into the ground:

$$Ha = \frac{K \times D}{\beta} \delta a$$

- For pile of which the head is sticking out above the ground ;

$$Ha = \frac{4 E \times I \times \beta^3}{1 + \beta \times h} \delta a$$

where ;

Ha : Allowable bearing capacity for lateral force (kg)

k : Lateral reaction coefficient of ground (kg/cm^3)

D : Diameter of pile (cm)

δa : Allowable deflection of pile (cm)

The location where the deflection is to be checked is shown in Fig. 2.15 and the allowable deflection is as follows;

Condition	Deflection
Normal condition	1.0 cm
Seismic condition	1.5 cm

E : Coefficient of elasticity of pile body (kg/cm2)

I : Moment of inertia of cross section of pile body (cm4)

β : Characteristic value of pile as expressed below (cm^{-1})

$$= \{\frac{k \times D}{4 E \times I}\}^{1/4}$$

h : Height of pile head above ground surface (cm).

(c) Bearing capacity of the pile cluster

(i)Bearing capacity for intrusive force

Allowable bearing capacity for intrusive force is calculated by use of the following formula:

$$Q \leq 1/n (A \times qd' - W + L \times Df \times S)$$
$$qd' = \alpha \times c \times Nc + 1/2\, \beta \times \gamma 1 \times B \times Nr + \gamma 2 \times Df \times .Nq$$

Where;

Q : Allowable bearing capacity (t)

n : Safety factor of which the value is the same as that for bearing capacity per one pile (t)

A : Effective area at bottom of pile cluster(m^2)

qd' : Ultimate bearing capacity at bottom of pile cluster (t/ m^2)

Df : Depth from bottom of footing to bottom of pile cluster (m)

S : Average shearing strength of foundation strata between bottom of footing and bottom of pile cluster (t/ m^2)

W : Weight of foundation material replaced by pile cluster (t)

α, β : Coefficients depending on shape at the bottom of pile cluster, of which values are the same as those of α and β in formula for bearing capacity of foundation.

C : Cohesion of stratum below bottom of pile cluster (t/ m^2)

Nc, Nr, Nq : Coefficients depending on angle of shearing resistance of foundation strata, of which values are given in Fig. 2.16

$\gamma 1$,$\gamma 2$: Unit weights of foundation materials above and below the bottom of pile cluster (t/ m^3)

(ii)Bearing capacity for lateral force

Bearing capacity for lateral force of the pile cluster is calculated in the same manner as that for the one pile.

The coefficient of horizontal reaction of the ground in case of the pile cluster is obtained by multiplying the said coefficient in case of the one pile by the adjustment factor given by use of the following formula:

$$\mu = 1 - 0.2 \left(2.5 - \frac{L}{D}\right) \ldots\ldots \quad (L < 2.5\ D)$$

Where ;

μ : Adjustment factor

L : Distance between piles (m)

D : Diameter of pile (m)

(d) Reaction of pile and deflection of footing

The reaction of pile and deflection of footing is calculated by employing the deflection method of which principle is described as follows ;

The deflection at the head of each pile is given as the solution of following simultaneous equation.

$$Axx \cdot \delta x + Axy \cdot \delta y + Ax\alpha \cdot \alpha = Ho$$
$$Ayx \cdot \delta x + Ayy \cdot \delta y + Ay\alpha \cdot \alpha = Vo$$
$$A\ x \cdot \delta x + A\ y \cdot \delta y + A\alpha\alpha \cdot \alpha = Mo$$

$$Axx = \sum (K1 \cos^2\theta i + kv \sin^2\theta i)$$

$$Axy = Ayx = \sum (kv - k1) \sin\theta i \cos\theta i$$

$$Ax\alpha = A x = \sum \{(kv - k1) xi \cdot \sin\theta I \cos \theta I - k2 \cos\theta I \}$$

$$Ayy = \sum (kv \;.\; \cos^2\theta I + kl \sin^2\theta)$$

$$Ay\alpha = A y = \sum \{(kv \cos^2\theta I + k1 \sin^2\theta I) xi + k2 \;.\; \sin\theta I\}$$

$$A\alpha\alpha = \sum \{(kv \cos^2\theta I + k1 \sin^2\theta I)xi^2 + (k2 + k3)xi \sin\theta I + k4)$$

where ;

Ho : Horizontal force at bottom of footing(t)

Vo : Vertical force at bottom of footing (t)

Mo : Moment due to the said resultant force around point "O" as shown in Fig. 2.17 (t.m)

δ x : Horizontal deflection of point "O" (m)

δ y : Vertical deflection of point "O" (m)

α : Angle of rotation of footing as shown in Fig. 2.17 (radian)

xi : Distance in x-coordinates from point "O" to head of pile i as shown in Fig. 2.17 (m)

θi : Angle of axis of pile i to the vertical line as shown in Fig. 2.17 (degree)

K1,K2,K3,K4 : Coefficient given by following formula.

	In case of fixed head		In case of hinged head	
	$h \neq 0$	$h = 0$	$h \neq 0$	$h = 0$
K1	$\frac{12EI\beta^3}{(1 + \beta h)^3 + 2}$	$4EI\beta^3$	$\frac{3EI\beta^3}{(1 + \beta h)^3 + 0.5}$	$2EI\beta^3$
K2, K3	$K1\frac{\lambda}{2}$	$2EI\beta^2$	0	0
K4	$\frac{4EI\beta}{1+\beta h} x \frac{(1 + \beta h)^3 + 0.5}{(1 + \beta h)^3 + 2}$	$2EI\beta$	0	0

In conformity with the deflections of δx, δy and α, the forces acting on each pile are given by use of thefollowing formula.

$$PNi = Kv \cdot \delta yi'$$

$$PHi = Ki \cdot \delta xi' - K2 \cdot \alpha$$

$Mti = -K3 \cdot \delta xi + K4 \cdot \alpha$

$xi' = \delta x \cos \theta i - (\delta y + \alpha xi) \sin \theta i$

$yi' = \delta x . \sin \theta i + (\delta y + \alpha xi) \cos \theta i$

Where ;

$\delta xi'$: Deflection of head of pile i in direction perpendicular to pile axis (m)
$\delta yi'$: Deflection of head of pile i in direction of pile axis (m)
kv : Axial force of pile, which causes the unit deflection at the pile head in direction of pile axis (t/m)
PNi : Axial force acting at head of pile i (t)
PHi : Lateral force acting at head of pile i (t)
Mti : Moment acting at head of pile i (t.m)

The notation for other variables is the same as that for aforementioned simultaneous equation for the deflection at the pile head.

(e) Design of pile body

The pile body shall be designed assuming the pile is a beam laid on the elastic foundation and the structural analysis of pile body is made by employing Chang's theory expressed as follows ;

$$E \cdot I \frac{d^4 y \cdot 1}{dx^4} = 0 \qquad \text{(For above ground portion)}$$

$$E \cdot I \frac{d^4 y^2}{d x^4} + P = 0 \qquad \text{(For under ground portion)}$$

$$P = k \cdot D \cdot y^2$$

where;

E : Coefficient of elasticity of pile (kg/cm2)
I : Moment of inertia of cross section of pile (cm4)
x : Distance from ground surface (cm)
y : Deflection at the point x cm apart from ground surface (cm)
k : Coefficient of lateral reaction of ground (kg/ cm3)
h : Height of pile head above ground surface (cm)

2.2.7 Sluiceway

(1) Transverse section of conduit

(a) Box culvert

For design of the transverse section, the following five (5) external loads acting transversely to the conduit body are combined:

- dead load
- earth pressure at rest due to levee embankment
- live load on levee embankment
- subgrade reaction
- hydrostatic pressure

The stress analysis is made by use of the slope deflection method to estimate bending moment, shearing and axial forces for each member of the conduit body and the conduit body is analyzed as a rigid frame of a box culvert.

(b) Pipe culvert

The relation between internal section stress and lining thickness of the pipe culvert is analyzed by use of the finite element method changing the diameter of culvert. The analyzed results under a unit uniform load condition on the top slab are schematically diagramed as shown in Fig. 2.18.

(2) Wing wall

(a) Design load

The design of wing walls at the inlet and outlet of sluiceway facilities is made in consideration of the earth pressure due to backfilling behind the walls. In this design, the wing wall is assumed as a cantilever sustained by the conduit body. The earth pressure at rest is applied for estimation of earth pressure.

The earth pressure at rest acts horizontally on the wing wall as schematically shown as Fig. 2.19 and is calculated by use of the following equation :

$$P_V = P_A + P_B + P_C$$

$Ph = Ko \cdot Pv$

where;

Pv : vertical earth pressure at rest (t/m^2)
PA : vertical earth pressure at rest due to embankment of "A" portion (t/m2)
PB : vertical earth pressure at rest due to embankment of "B" portion (t/m2)
PC : vertical earth pressure at rest due to embankment of "C" portion (t/m2)
Ph : horizontal earth pressure at rest (t/m2)
Ko : coefficient of earth pressure at rest (=0.5)

The above vertical earth pressure at rest of P_A and P_B are generally estimated by use of the following equations as schematically presented in Fig 2.20

$PA = (r \cdot h / \pi)(\tan\text{-}1\ B1 - \tan\text{-}1\ B2 + B3 - B4)$
$PB = (r \cdot h / \pi)(A1 - A2(\tan\text{-}1\ A3 - \tan\text{-}1\ A4))$
$PC = r \cdot x$
$A1 = x(a + b) / x2 + (a + b)2)$
$A2 = b / a$
$A3 = (a + b) / x$
$A4 = b / x$
$B1 = (a + b + c) / x$
$B2 = c / x$
$B3 = x(a + b + c) / x2 + (a + b)2)$
$B4 = x c / (x2 + c2)$

where;

r : unit weight of embankment earth material (tm3)
h : depth from embankment surface to top of wing wall (m)
π : circular constant (=3.14)
x : depth from top of wing wall (m)

Based on the above, the total earth pressure at rest acting on the wing wall is estimated as follows :

$$Ph = Ko \int^{1} (PA(x) + PB(x) + PC(x))\, dx$$

where;

Ph : total earth pressure at rest (t/m)
Ho : height of wing wall (m)
Ko : coefficient of earth pressure at rest (=0.5)

The bending moment and shear stress are therefore estimated at the edge of wing wall fixed on the conduit body as follows :

$$M = \int^{1} Ph(x) \cdot x\, dx$$
$$S = \int^{1} Ph(x)\, Dx$$

Where;

M : bending moment (t·m)
S : shearing stress (t)
x : distance from edge of wing wall (m)

(3) Foundation

(a) Selection of foundation type

The following criteria are used for selection of spread foundation and pile foundation types :

	Criteria
Spread foundation	$Qmax \leqq Qa$
Pile foundation	$Qmax > Qa$

where;

Qmax : maximum ground reaction under full design load condition (t/m2)
Qa : allowable bearing capacity of spread foundation (t/m2)

(b) Ground reaction of spread foundation

The distribution of ground reaction at sluiceway site is schematically shown in Fig. 2.21.

$$Qmax = \sum Ni\,(1 + 6\,e / B) / B$$

$$Qmin = \sum Ni\,(1 - 6\,e / B) / B$$

where;

Qmax : maximum ground reaction (t/m2)
Qmin : minimum ground reaction (t/m2)
Ni : design load (t/m)
B : conduit length (m)
e : eccentricity (m)

The design load mainly consists of the following:

- total weight of sluiceway structure,
- total weight of embankment materials, and
- water pressure under filled-up condition

(c) Pile foundation

(i)Number of piles

In order to estimate the required number of piles, the reaction of piles is estimated and thereby the required number of piles is evaluated to sustain the total load transmitted from the 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 sluiceway body and its levee embankment to pile foundation is vertical load only. Accordingly, the intrusive vertical load is herein considered for the design of pile foundation.

The number of piles for foundation is calculated 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)

(ii)Arrangement of piles

In general, when the distance between pile centers is smaller than 2.5 times the pile diameter, the bearing capacity of piles tends to decrease compared with the capacity under the condition of single pile. This might be due to the adverse effect caused under the condition of group of piles. Accordingly, the foundation piles shall be arranged at least at the distance of 2.5 times the pile diameter.

In addition, the distance between the center of pile and the edge of sluiceway body shall be arranged at 1.25 times the 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 load condition, the reaction of each pile is determined by the following equation :

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

where;

Vi : reaction of pile (allocated load to pile) (t)
Wi : vertical design load (t)
N : total number of piles
e : eccentric distance between gravity center of design load and that of pile (m)
Xi : distance between pile and gravity center of piles (m)
Ig : geometrical moment of inertia of piles (m2)

(4) Cutoff wall of sheet pile

(a) Necessity of sheet pile

The safety against piping is examined by use 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)

The Lane'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, necessity of sheet pile is examined by use of the said formula under without sheet pile condition ($\sum Lv = 0.0$).

(b) Design of sheet pile

The sheet piles are to be provided at two locations of the inlet and outlet. The required total depth of sheet piles is calculated by use of the aforementioned formula. The required depth of sheet pile is obtained by the following equation :

$$ls = (\Delta H \times Cw - L/3 - t1 - t2) / 4$$

where;

ls : length of sheet pile (m)
ΔH : difference of water levels at both sides of sluiceway (m)
L : length of conduit (m)
t1, t2 : thickness of bottom slab of conduit (m)

3. DESIGN OF BRIDGE AND CULVERT

3.1 General

This section describes the detailed criteria for the replacement plan of the bridges and culverts brought about due to the channel widening.

The design of the bridge and culvert will be performed basically in accordance with Indonesian Codes and Standards related to the bridge and culvert design and planning issued by BINA MARGA, however for the requirements of design not covered by the above Codes and Standards, Japan Standards or AASHTO for Highway Bridges will be applied properly.

3.2 Bridge Design

3.2.1 Methodology

The final design shall be proceeded on the information cleared in the definitive plans in the interim report, together with the information included in all the associated supplementary documents, which were prepared through the study period.

3.2.2 Applicable codes and standards

The followings are considered main codes and standards:

(1) Indonesian Standards

1) Standard Specification for Geometric Design of Urban Roads, January 1988, translated in English for "Standar Perencanaan Geometrik Untuk Jalan Perkotaan, 1988"

2) Loading Specification for Highway Bridge Design Published by BINA MARGA, 1987, for "Pedoman Perencanaan Pembebanan Jembatan Jalan Raya SKBI 1.3.28. 1987"

3) Bridge Design Code, 9 MAY 1992, for "Peraturan Perencanaan Tehnik Jembatan, 9 MAY 1992"

4) Bridge Design Manual 2, September 1992, for "Panduan Perencanaan Tehnik Jembatan, 1992"

5) Standard for Bridge Superstructure Prestressed Concrete Girder, T Type-A Class for "Standard Bangunan Atas Standar Bangunan Atas Jembatan Gelagar Beton Pratekan, Tipe T-Kelas A"

6) Standard Specification of Pile Trestle Substructure for the Girder Span 11 M - 25 M, 1991, for "Spesificasi Pilar Dan Kepala Jembatan Sederhana Bentang 11 M - 25 M Dengan Pondasi Tiang Pancang Edisi Awal 1991"

7) Specification & Standards for Reinforced Concrete Slab Highway Bridges No. 02/1969, for "Spesifikasi Dan Standard Jembatan Pelat Beton Untuk Jembatan Jalan Raya No. 02/1969"

(2) Japanese Standards

1) Japan Bridge Standard issued by Japan Road Institute, 1990

2) Bridge Standard Design, Volume 18 - 20, subtitled as Pre-tension type, Simple Slab Bridge and T Girder Bridge standardized by Ministry of Construction, March 1996

3) Bridge Design Manual, Volume 2 issued by Japan Highway Authority, 1980

4) Pedestrian Bridge Standard issued by Road Institute, 1979

(3) American Standards

1) AASHTO Standard Specification for Highway Bridges, 1992, as amended by interim Specification-Bridge, 1993 and 1994

2) Standard Specifications of the American Society for Testing and Materials (ASTM)

3.2.3 Design specification

(1) Fundamental design policy

Except as modified herein, the selected structure element will be designed in according with the above mentioned Standards.

(2) Superstructure

The service load Method (Allowable Stress Design) will be used for design of the girder bridge and ultimate capacity will be properly checked using the Strength Design Method (Load Factor Design).

(3) Substructure

The strength Design Method (Load Factor Design) will be used for the design of substructure elements.

(4) Ancillary structures

Most suitable structural type for the ancillary works of the bridge will be studied and selected among the typical structure drawings depicted in the Indonesian and Japan Standards.

(5) Pedestrian bridge

This is designed according to the Indonesian Bridge Design Code, referring to Japan Pedestrian Bridge Standard.

3.2.4 Design loads

It is proposed to apply BINA MARGA's Standard Loading Specification for the structural design:

(1) Live load

(a) Application of "D" and "T" loads

Application and distribution method of the loads are as follows:

Traffic loads for design of road bridges consist of the "D" lane loading and the "T" truck loading. The "D" lane loading is applied across the full width of the bridge roadway and produces effects in the bridge equivalent to a queue of real vehicles. The total amount of "D" lane loading applied depends upon the width of the bridge roadway.

The "T" truck loading is a single heavy vehicles with three axles which are applied in any position in a Design Traffic lane. Each axle comprises two patch loading which are intended to simulate the effects of the wheels of heavy vehicles. Only One "T" truck may be applied per design traffic lane.

(b) Use of non-full live load

The loading specification stipulates about the use of non-full live load as follows, which will be applied to all road class except National Highway under the management of BINA MARGA:

In the case of the use of non-full live loads due to particular conditions (such as semi-permanent bridge below-standard bridge, temporary bridge), the live load shall be calculated in the following way:

1) 70% of load
70% of "T" load and 70% of "D" load

2) 50% of load
50% of "T" load and 50% of "D" load

Where the rules of using "T" and "D" loads shall bee as described in the previous article.

In the above mentioned classification for the reduction of live load, the following application method is widely used in recognition of the Authorities: 100% of load is applied to National Roads and State Road, which are belonged to Type I Class I and II and Type II Class I in the road classification table according to Road Geometric Standard, 70% of the load is used for the road class of Type II Class II and III, and also 50% of the load for the Road Class IV of Type II.

On the other hand, however, it's differently described in the Bridge Design Code that "in special circumstances approved by the Authority, a reduced value of 70% of the "D" loading may be used".

There are found contradictory descriptions among two Standards i.e. concerning the classification of percent, there is no description about 50 % in the latter and for applicable Loads it is not clear to be use "T" and "D" or only "D" .Taking into account the actual traffic situation such as traffic volume and the possibility of heavy car's passage, the following criteria would be recommendable:

1) The bridges on the national roads are to be designed by the 100% of the Live Load according to the Loading Specification Standard.

2) Other bridges except the national roads are designed by the 70% of the Live Load, "T" and "D" irrespective of road classes.

Additionally speaking, it is reasonable that since all bridges planned in the Project are not built for temporary use, the maximum reduction is to be limited at least 70% of the Live Load.

(c) Impact

The impact coefficient used for bridges:

$$K = 1 + 20 / (50 + L)$$

where: K is dimension of impact coefficient and
L is length of effective span in meter

(2) Wind load

The wind load is taken as 150 kg per square meter on the structure, computed on the horizontal side area of the bridge, in a perpendicular direction to the longitudinal axis of the bridge.

The sum of the vertical areas of the bridge superstructure assumed to be effected by the wind load is computed as a percentage of the total side area of the bridge and the total vertical area of the live load which is not protected from the wind by the bridge members. The vertical live load surface is considered a continuous vertical surface, 2.00 m high above the bridge deck. In calculating the vertical side area of the bridge, the following assumptions are made:

Case - 1 No live load on bridges

- For girders and beams the area is 100% of the vertical side area which is directly affected by the wind, plus 50% of the vertical area of the other side

- For trusses and arches the area is 30% of the vertical side area directly affected by the wind, plus 15% of the vertical area of the other side

Case-2 Live load on bridges

- For the superstructure the area is 50% of the vertical area defined above

- For the live load the area is 100% of the vertical area directly affected by the wind.

(3) Thermal forces

The range of temperature is generally as follows:

- Steel structures .... 30°C
- Concrete structure.. 15°C

(4) Earthquake force

Earthquake Force on bridges are computed according to item 2.4.7 Earthquake Effects, Bridge Design Code, 14 May, 1992.

(5) Longitudinal forces induced by friction at movable bearings

The friction forces are considered as induced by the dead load only and the following friction coefficients are used:

- Roller bearing in steel

With 1 or 2 rollers	0.01
With 3 or more rollers	0.05

- Friction bearing

Between steel and copper alloy steel	0.15
Between steel and cast iron	0.25
Between rubber pad and steel/concrete	0.15-0.18

(6) Brake and traction

The effect of breaking forces may be considered as being five percent (5%) of the "D" loading, without impact, for all lanes carrying traffic headed in the same direction. The position of the breaking forces is assumed to be 1.80 m above the bridge deck.

(7) Centrifugal forces

Structures on curves are designed for a horizontal centrifugal force equal to a percentage of the "D" loading without impact, in all traffic lanes. The centrifugal force is applied 1.80 m above the bridge deck and the rate is determined by the following formula:

$$KS = 0.79 \times v^2/R$$

Where; KS: The coefficient of centrifugal force
V: The Design speed in kilometres per hour
R: The Radius of the curve in meters

(8) Collision force

To calculate the collision forces between pier and vehicles, one of the following two static horizontal collision force criteria is applied:

- In the direction of traffic lanes : 100 tons
- Perpendicular to traffic lanes : 50 tons

The position of collision forces is considered to be 1.80 m above the roadway surface.

(9) Combination of Loads

The members of the roadway bridge structure are designed to withstand all combinations of possible working loads and forces. In accordance with the loading possibilities and characteristics of each stress combination under consideration, the allowable stresses of the structure may be increased.

The applicable stresses, expressed as a percentage of the allowable stresses for some combinations of loads and forces, are as follows:

Combination of loads/force	The applicable stresses expressed as a percentage of the allowable stresses
I. M + (H + K) + Ta + Tu	100%
II. M + Ta + Ah + F + A + SR + Tm	125%
III. Comb. (I) + Rm + F + A + SR + Tm + S	140%
IV. M + Gh + Tag + F + Ahg + Tu	150%
V. M + PI	130%
VI. M + (H + K) + Ta + S + Tb	150%

Where:

A = Wind load
Ah = Water flow & floating material forces
Ahg = Water flow & floating material forces due to earthquakes
F = Friction force at expansion bearing
Gh = Equivalent horizontal force due to earthquake
(H + K) = Live load with impact
M = Dead load
PI = Forces during construction
Rm = Braking force
S = Centrifugal force
SR = Shrinkage force & creep
Tm = Thermal force
Ta = Earth pressure
Tag = Earth pressure due to earthquake
Tb = Collision force
Tu = Buoyancy

3.2.5 Special provisions

To cope with the land subsidence the special criteria for the bridge planning, which are generally differed from the common bridges, are set up in this separated section. The present land situation to be considered with care for the bridge planning are as follows:

1) The elevation difference between the specific flood level and the ground level is so small that the vertical clearance required for the bridge construction can not be secured without taking some measures.
2) And the land subsidence is going on as ever and the land will be submerged at wide stretch in near future.

Against the above adverse circumstances for the bridge planning special criteria are presented herein:

(1) Vertical clearance

The vertical clearance between the lowest point of bridge so fit and the design high water level shall meet at least the minimum clearance allowed for normal stream flow and freeboard of the levee, and moreover the allowance for land subsidence has to be added to the above minimum clearance:

Allowance for the vertical clearance is totally as follows,

$$H \geq h1 + h2$$

where;

h1: Allowance for free boad, which is varied by the river width:

For the river width $W \geq 15$ m: $h1 \geq 0.5$ m
$5 \text{ m} < W < 15$ m: $h1 \geq 0.4$ m
$W < 5$ m: $h1 \geq 0.3 - 0.2$ m

While: since Minimum Vertical Clearance required by normal stream flow is 0.3 meters according to the Bridge Design Code, 1.4.4, the minimum clearance is to be considered 0.3 meters.

h2: Allowance for land subsidence of 14 years after, where 14 years is established under the following assumption that:

- (i) The construction will be completed 4 years after the planning.
- (ii) Lifting work will be carried out at 10 years interval.

The total subsidence up to the next rehabilitation time of 14 years after is estimated under the following computation:

1) Kamal drainage at the highway crossing:
mm/years x 14 years = 840 mm

2) Tanjungan drainage at the highway crossing:
idem

3) Bor drainage at confluence with Mookervalt:
mm/year x 14 years = 1,120 mm

4) Saluran Cengkareng at confluence with Cengkareng floodway:
mm/year x 14 years = 840 mm

As mentioned above "h2" is input by 840 mm at the bridge location other than Bor Drainage.

(2) Lifting method of bridge girders

To cope with the large land subsidence, the lifting measures shall be introduced at design stage for re-positioning the bridge girder on the heightened abutments and piers, being followed simultaneously by the rehabilitation of the access road with additional embankment and new pavement.

The bellow consideration for designing lifting measures shall be taken:

Structural measures for lifting the girder shall be taken at the space between superstructure and substructure. Twenty four years are assumed as the design year for the rehabilitation work by use of lifting.

(3) Grade of access road

Maximum grades allowable under normal conditions shall be maintained according to Geometric Design Standard as stated bellow:

Road class Type II	Design speed (unit: km/hour)	Maximum grade (%)
Class I	60	5
Class II	60, 50	5, 6
Class III	40, 30	7, 8
Class IV	30, 20	8, 9

Since there are a variety of construction conditions for the replacement especially in the residence zone, the consideration on the surrounding conditions of the access road shall be taken.

(4) Debris

Since the channels are considered very small as river classification and the water basin of these channels is limited to the residence plane with grass land and small trees around the urban, the hydrodynamic forces acting to the bridge side would be negligible due to small debris mats and nothing of log impact.

Since the stoppers which are shaped in rectangular wall will be constructed at the both end of piers and abutment Trestles to resist horizontal force by seismic and simultaneously to be able to resist to the debris force, the stress check by the lateral force due to the debris will not be required .

(5) Drawings

Drawings for the bridge construction should be prepared using the following principles:

1) Standard methods of detailing, particularly for steel and reinforced concrete, shall be used consistently for all drawings;

2) Bridge components should be drawn as they actually appear; avoid mirror images and opposite hand views;

3) Each dimension should only be shown once;

4) Each bridge component should be detailed on one sheet as far as possible;

5) All drawings should be to scale, and the scales should be shown on all drawings;

6) Standard procedures shall be used for setting out the bridge and dimensioning components. Where abbreviations are used, these shall also be standardised.

3.3 Culvert Design

(1) Definition

In order to avoid the confusion of term usage for culvert it is defined for the report that the term, "Culvert" is a crossing structure constructed in subsurface for water stream or that of constructed close to surface with the opening size of less than 2 m x 2 m.

(2) Applicable codes and standards

There is only one Indonesian Code, i.e. Standard Drawings of Culvert translated in English for "Standard Duiker", in which the principal dimensions of box type culvert with the opening of less than 2 m x 2 m are tabulated. In connection with the Standard, Japan Bridge Design Manual issued by Japan Highway Authority set out the design procedure on the various type of culvert not only for the box culvert but also pipe culvert, arch, portal and so on, consequently it is recommendable to use the Japan Standard properly in addition to the Indonesian Standard above stated.

(3) Design load

As to the live load, the same load as previously described for bridge criteria will be used and soil pressure and bearing strength under the culvert will be computed based on the soil data.

(4) Special provisions

For the measure of subsidence the following consideration shall be taken:

1) Syphon system shall not be used to obtain smooth stream of water without concern about accumulation of debris mats.

2) At the reach of water to under the upper slab of the box culvert, the slab shall be supported on heightening of both walls or another culvert shall be provided for the measures.

3.4 Design Specification

Detailed design specification concerning road bridge including access road, pedestrian bridge and culvert are described in Table 3.1, 3.2 and 3.3 respectively.

4. DESIGN OF GATES AND OTHER METAL WORKS

4.1 Design Loads

4.1.1 General

The equipment shall be designed with the worst combination of the acting loads. The loads specified herein shall be considered as minimum requirements and the Contractor shall use additional loads and combination thereof which the Contractor considers to be applicable and necessary.

4.1.2 Gate leaf and stoplog

(1) Hydrostatic load

Hydrostatic load shall be of the water head difference between upstream and downstream sides of the gate and stoplog.

(2) Dead weight

Reaction due to self weight.

(3) Operating load

Operating load shall conform to the equipment of "(4.1.4) Hoist"

(4) All loads imposed during operating the stoplog due to the overload hoist or stoplog jammed conditions.

4.1.3 Trashrack

(1) Hydrostatic load

Hydrostatic load shall be of the water head difference between upstream and downstream sides of the Trash rack.

(2) Dead weight

4.1.4 Hoist

The hoist shall be designed taking into account of the following loads:

- Dead weight of the gate leaf. Such connecting devices with the hoist as spindle, etc., shall be included into the dead weight of the gate.
- Friction force due to sliding parts
- Friction force due to seal rubbers
- Buoyancy
- All loads imposed during raising the gate due to the overload hoist or gate jammed condition

4.2 Design Stresses

4.2.1 Gates and other steel structures

(1) Structural steel members

The allowable stresses for normal loading condition of structural steel members with a thickness of 40 mm or less shall be as shown in the following table:

	Kinds of Stresses	SS 400 or Equivalent (thickness<40 mm) SM 400
1)	Axial tensile stress (per net sectional area)	$1{,}200 \text{ kgf/cm}^2$
2)	Axial compressive stress (per gross sectional area) Compressive members	On condition of $(l/r) < 20$, $1{,}200 \text{ kgf/cm}^2$ On condition of $20<(l/r) \leqq 93$ $1{,}200-7.5\ \{(l/r)-20\} \text{ kgf/cm}^2$ On condition of $93 \leqq (l/r)$, $10{,}000{,}000/\{6{,}700+(l/r)^2\} \text{ kgf/cm}^2$
	where, l : buckling length of member (cm) r : radius of gyration of sectional area of member (cm)	
	Compressive splice member	$1{,}200 \text{ kgf/cm}^2$
3)	Bending tensile stress	$1{,}200 \text{ kgf/cm}^2$

	Kinds of Stresses	SS 400 or Equivalent (thickness<40 mm) SM 400
4)	Bending compressive stress (per gross sectional area)	On condition of $(1/b) \leqq (9/K)$ 1,200 kgf/cm^2
	Compressive members	On condition of $(9/K)<(1/b) \leqq 30$ $1{,}200 - 11\ (K.1/b - 9)$ kgf/cm^2

where,
l : distance between fixed point of compressive flange (cm)
b : width of compressive flange (cm)
K : Squ. Root {3 + (Aw/2Ac)}
Aw : sectional area of web plate (cm^2)
Ac : sectional area of compressive flange (cm^2)
In case of (Aw/Ac)<2, K is taken as 2

	On condition that compressive flange is directly fixed to skin plate, etc.	1,200 kgf/cm^2
5)	Shearing stress (per gross sectional area)	700 kgf/cm^2

In case the thickness exceeds forth (40) mm, the allowable stresses for normal loading condition of the structural steel members shall be 0.92 times that of the allowable stress as mentioned above.

6)	Combined stress resulting from combination of biaxial or triaxial principal stress	1,800 kgf/cm^2

(2) Stress in the bar elements of trashrack shall not exceed the following critical stress.

$$Fcr = 0.6.\ fy \times (1.23 - 0.0153\ L/t)$$

where,
fcr : critical allowable stress (kg/cm^2)
fy : yield stress of the material (kg/cm^2)
L : laterally unsupported length of bar elements (cm), but $L \leqq 70t$
t : thickness of the bar elements (cm), decreased a corrosion allowances as specified.

Notes:

(a) Bj 44 (SNI 0722-89-A) is equivalent to SS 400 (JIS G 3101).

(b) The allowable stresses in case of overloading condition and/or the combined stresses resulting from combination of biaxial or triaxial principal stresses may be increased by fifty (50%) percent than those for normal loading condition. In no case, however, shall any stress exceed ninety (90%) percent of the yield point strength and/or minimum elastic limit of the steel material used.

The combined stress shall be calculated by the following formula as developed by Mises, Hencky and Hubber:

$$fg = \text{Squ. Root } \{fx^2 + fy^2 - fx.\,fy + 3\,fq^2\}$$

where,

fg : Combined stress (kgf/cm^2)

fx : Direct stress (tension is considered as positive) (kgf/cm^2)

fy : direct stress acting perpendicular to axis of fx (tension is considered as positive) (kgf/cm^2)

fq : Shearing stress (kgf/cm^2)

(3) When steel material other than those mentioned in the table is used, its allowable tensile stress for normal loading condition shall not exceed fifty (50%) percent of the yield point strength of the steel material used.

All other allowable stresses shall be computed in proportion to the allowable stresses given in the table based on the yield point and/or ultimate strength of steel material used whichever is the least.

4.2.2 Machine parts of hoisting equipment

All mechanical parts of the equipment subjected to normal or rated capacity loading condition shall be designed with the following factors of safety (FS) against the ultimate strength of the steel material used:

Material	FS for tensile stress	FS for compressive stress	FS for shearing stress
Rolled steel for general or welded structure	5	5	8.7

Material	FS for tensile stress	FS for compressive stress	FS for shearing stress
Carbon steel forgings	5	5	8.7
Carbon steel for machine structural use	5	5	8.7
Corrosion-resisting steel	5	5	8.7
Carbon steel castings	5	5	8.7
Gray iron casting	10	3.5	10
Bronze casting	8	8	10

4.2.3 Concrete

The allowable concrete bearing and shearing stresses shall not exceed 60 kgf/cm^2 and 3.6 kgf/cm^2 respectively.

4.3 Design Particulars

4.3.1 Minimum thickness

The thickness of all structural members shall not be less than six (6.0) mm, except those of the following parts or part as approved by the Engineer:

Parts	Min. thickness (mm)
- Skin plate of gates	6.0
- Corrosion resisting steel plate for sealing plates	6.0
- Steel sections	5.0
- Bar element	6.0

4.3.2 Critical slenderness ratio

The critical slenderness ratio for major compressive members shall be less than 120, and 150 for secondary members. The said ratio in case of tension members shall be read as 200 and 240 respectively.

4.3.3 Maximum deflection

The maximum deflection of each horizontal main beam member shall be less than the following value at full load:

Equipment	Max. deflection
Slide gate	1/800 of supporting span
Flap gate	1/1,000 of supporting span
Supporting beam of trashrack	1/600 of supporting span

4.3.4 Corrosion allowance

The corrosion allowance shall be 1.0 mm to the water contact face members of gates and trashrack.

4.3.5 Friction coefficient

For the purpose of designing the equipment, the coefficient of friction shall not be less than the following values:

Kinds of friction force	Frictional coefficient
- Sliding friction force of bearing plate	0.4
- Sliding friction force of seal rubber (Against stainless steel plate)	
*Dry condition	1.2
*Wetted condition	0.7
- Sliding friction force of sedimentary silt	0.4

4.3.6 Rubber seals

Seals shall be designed and mounted in such a manner that they are adjustable, water tight and shall be readily removed and replaced. Seals shall be molded. Extruded seals shall not be permitted. Where seals are installed curved, they shall be clamped in a jig which will form them to the proper radius before the holes are laid out and drilled, and the ends trimmed. Holes in related parts of the seal assemblies shall be carefully drilled, using a template, to ensure proper matching when the seal units are assembled.

All adjusting screws, bolts and washers for securing the seals and seal assembly in place shall be of corrosion resisting steel.

Seals shall be made of natural or synthetic rubber suitable for the temperature ranges and conditions at the Site and shall be of a material that has proven successful in similar applications. The seal materials shall have the following physical properties as determined by tests made in accordance with the relevant standards:

Property	Limits
- Tensile strength	210 kgf/cm^2 minimum
- Ultimate elongation	450 % minimum
- Durometer hardness (shore, type A)	60 to 70
- Specific gravity	1.1 to 1.3
- Water absorption (70 °C for 48 hours)	5% maximum by weight
- Compression set (as a percent of total original deflection)	30 % maximum
- Tensile strength after oxygen bomb aging for 48 hours at 70 °C	80 % minimum of ensile strength before aging
- Adhesion of metal insert to rubber	
*Sheared test	16 kgf/cm^2
*Tension test (90 degree to axis)	2 kgf/cm^2

The gate seal shall be spliced at the corners by shop vulcanizing to provide a single continuous seal. The tensile strength of all shop splices shall not be less than 50% of the tensile strength of unspliced material.

4.3.7 Materials

The materials used in structures of the steel gate shall be new and of high quality, selected particularly to meet the duties required for the proper operation of the gate, and indicated fully in the Contractor's detailed drawings for approval. The materials shall meet with the requirements mentioned in the latest issue of the following standards, and in accordance with those as listed below or equivalent and/or better in quality or as approved in advance by the Engineer, unless otherwise specified:

(1) Steel plates, bars, etc.

-	Steel plates for structural main parts of the equipment, except for rolled steel shapes	JIS G3106 SM400B, JIS G3101 SS 400, SNI 0552-89-A or equivalent
-	Steel plates for general structures	JIS G 3101, SS 400, SNI 0552-89-A or equivalent
-	Rolled steel shapes	JIS G 3192, SNI 0945-89-A or equivalent
-	Steel bolts, nuts and washers	JIS B 1180, B 1181 and B 1256, SNI 0541-89-A or equivalent
-	High-strength steel hexagon bolts, hexagon nuts and plain washers	JIS B 1186, SNI 0541-89-A or equivalent
-	Spring lock washers	JIS B 1251, SNI 0571-89-A 9 or equivalent
-	Corrosion-resisting steel plates and bars, etc.	JIS G 4304, G 4305 G 4306 and G 4307 or equivalent
-	Corrosion-resisting clad steel plates	JIS G 3601 or equivalent

(2) Castings

-	Iron castings	JIS G 5501, FC 200, SNI 0813-89-A or equivalent
-	Steel castings	JIS G 5101, SC 410, SNI 1812-90-A (to be fully annealed) or equivalent
-	High tensile strength carbon steel castings and low alloy equivalent steel castings	JIS G 5111, SCMn Cr 3B or

	- Bronze castings	JIS H 5111 or equivalent
	- Phosphor bronze castings	JIS H 5113 or equivalent
(3)	Forgings	
	- Carbon steel forgings	JIS G 3201, SF 490 A, SNI 1855-90-A or equivalent
(4)	Wire rope	JIS G 3525, Grade G (galvanized), SNI 0076-87-A or equivalent

Tables

Table 3.1 DESIGN SPECIFICATION FOR ROAD BRIDGE

Item	Specification	Remarks
1. General	• This specification is applied for the design of road bridges with over 6 meters span. • For less than 6 meters, slab bridge of in-situ concrete is to be used.	
2. Applicable Codes and Standards	• Indonesian Codes and Standards properly supplemented by Japan Standards and AASHTO.	
3. Bridge Location	• Same place as existing bridge location.	
4. Design Conditions		
4.1 Principal Dimension		
4.1.1 Width of bridge	• To be designed according to "Standar Parencanaan Geometrik Untuk Jalan Perkotaan, 1988".	• Refer to Table* 5.4 and Fig.* 5.16.
4.1.2 Bridge length	• To span the channel width newly planned.	• Refer to Table 5.3.
4.1.3 Span length	• Span length shall be designed with consideration of construction condition at bridge locations.	• Refer to Table 5.4. • Maximum span is tentatively less than 16 meters.
4.1.4 Bridge angle	1) Basically right angle with both levee. 2) In case of skew, the angle is noted on the drawings.	
4.2 Superstructure		
4.2.1 Structure on deck slab		
(1) Rail	• Concrete wall with hand rail putting on it.	• Refer to Fig. 5.22(1).
(2) Side walk	• Mound-up type with 1 meter width of both sides.	• Same as above.
(3) Pavement	• 50 mm at curb stone with 2% crossfall. (100 mm in average for computation)	

Remarks for *: Number of Table and Figure is that of main report.

Item	Specification	Remarks
4.2.2 Loads (1) Dead loads	• Cast steel .... 7.85 t/m^3 • Cast iron .... 7.25 t/m^3 • Alloy aluminum .... 2.80 t/m^3 • Prestressed reinforced concrete .... 2.50 t/m^3 • Plain, cyclops concrete .... 2.20 t/m^3 • Stone/brick masonry .... 2.00 t/m^3 • Wood .... 1.00 t/m^3 • Compact earth, sand and pebble .... 2.00 t/m^3 • Asphalt pavement .... 2.00 to 2.50 t/m^3 • Water .... 1.00 t/m^3 The materials not included in the above list shall be accounted for in their real weights.	

Item	Specification	Remarks
(2) Live load	1) T load and D load, and distribution method are applied according to "Pedoman Perencanaan Pembebanan Jembatan Jalan Raya SKBI as follows:	• Refer to the textual criteria.

Table "T" Load

Class of bridges/load	I	II	III
Bina Marga's load	B.M. 100	B.M. 70	50% of B.M. 100
Total load (t)	45.0	31.50	21.50
Front wheel load (t)	2.5	1.75	1.25
Middle and rear wheel load (t)	10.0	7.00	5.00
Design load of a wheel "P" (t)	10.0	7.00	5.00
Ground contact of tire:			
Length "a" (cm)	20.0	14.00	10.00
Width: - Front wheel (cm)	12.5	8.75	6.25
- Rear wheel "b" (cm)	50.0	35.00	25.00

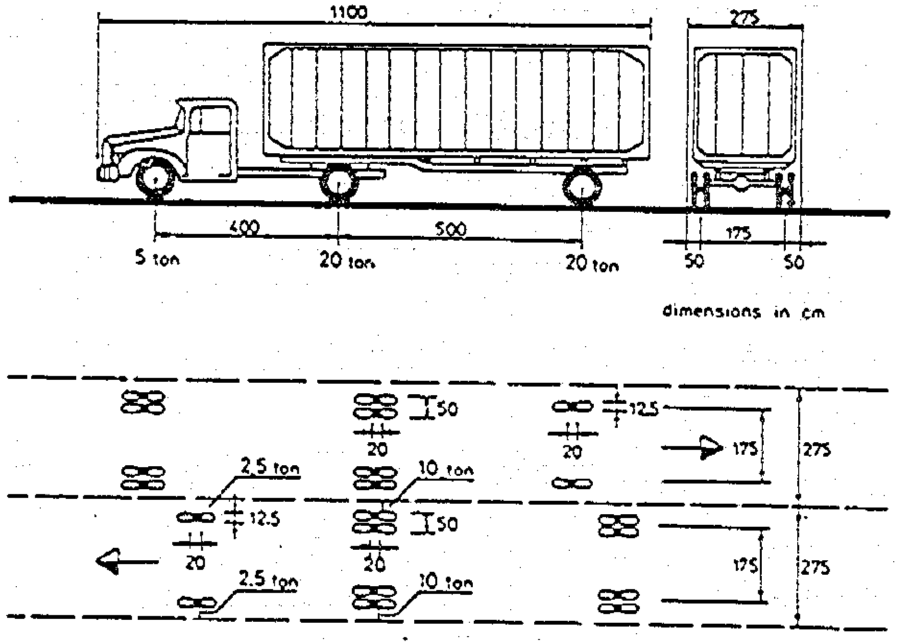


Figure Bina Marga "T" Load for BM 100

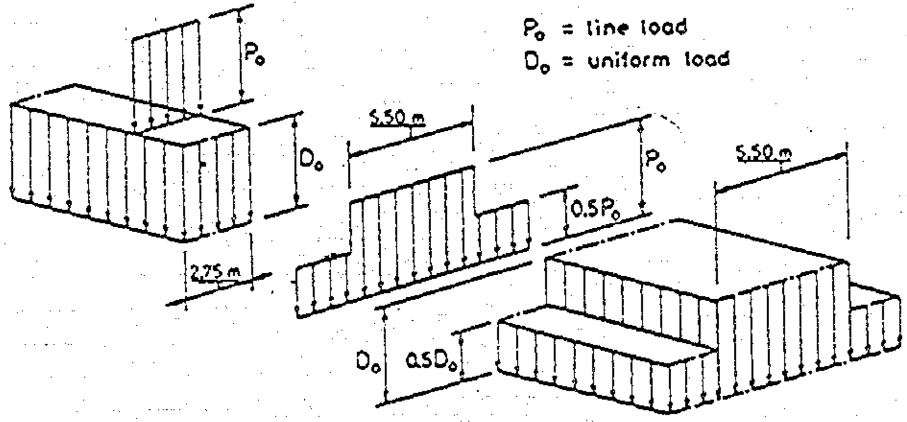
Item	Specification	Remarks

Table "D" Load

Class of bridges		I	II	III
Effective width of bridge (m)		7.0	6.0	3.0
Load		B.M. 100	B.M. 70	50% of B.M. 100
Line load				
P	(t/lane)	12.00	8.40	6.00
$P^* = P_o/2.75$	(t/m)	4.40	3.08	2.20
A half line load				
P/2	(t/m)	2.20	1.54	-
Uniform load$^{*)}$				
D	(t/m/lane)	2.20	1.54	1.10
$D^* = D_o/2.75$	(t/m)	0.80	0.56	0.40
A half uniform load				
D/2	(t/m)	0.40	0.28	-

Note *): In case the effective span "L" is larger than 30 m, uniform load "D_o" shall be modified by:

- Up to 60 m : $D_o = 2.20 - 1.1 (L - 30)/60$
- More than 60 m : $D_o = 1.1 (1 + 30/L)$



for one-lane road

for road of two lanes or more

Figure Bina Marga "D" Load for BM 100

Item	Specification	Remarks
	2)Load on the sidewalk, curb and railings a) Sidewalk construction : 500 kg/m^2 b) Girder strength : 60% of 500 kg/m^2 c) Curb : 500 kg/m at 25 cm high d) Railing : Horizontal load of 100 kg/m at 90 cm high 3)Use of non-full live load a) National Road under the administration of Bina Marga: 100% of load b) Other road : 70% of "T" load and 70% of "D" load	• Refer to Fig. 5.16.
(3) Impact load	To take account of the effects of vibrations and other dynamics, the stresses effected by "P" line load shall be multiplied by the shock coefficient which will produce a minimum result, while the design load "q" and the load "T" is not multiplied by the coefficient. The impact coefficient shall be determined using the formula: $K = 1 + 20/(50 + L)$ where: K = Shock coefficient L = Length of span in meter, determined by the construction type of the bridge (static condition) and the position of "P" line load as in table III of the standard. Shock coefficient is not accounted for the substructure if the substructure and superstructure are not monolithic. If the substructure and the superstructure are monolithic, the shock coefficient is accounted for the substructure.	

<table>
<tr><th>Item</th><th>Specification</th><th>Remarks</th></tr>
<tr><td>4.2.3 Materials</td><td></td><td></td></tr>
<tr><td>(1) Concrete</td><td>Main girder: Design strength, $\sigma_{CK} = 500$ kgf/cm^2 {49.1 N/mm^2}
Transverse beam, in-situ concrete: $\sigma_{CK} = 300$ kgf/cm^2 {29.4 N/mm^2}</td><td>• Standard Design by Japan Civil Engineering Institute.</td></tr>
<tr><td>(2) PC Tendon</td><td>a) Main girder
PC cable (SWPR 7BN) 7 wires spiral - ø12.7 mm (L ≤ 11 m)
7 wires spiral - ø15.2 mm (L > 11 m)
b) Transverse tension
<table><tr><th>Tendon System</th><th>JIS Note</th><th>Cable</th></tr><tr><td>40 ton</td><td>SWPR19N</td><td>19 - ø17.8 mm</td></tr><tr><td>50 ton</td><td>SWPR19N</td><td>19 - ø19.3 mm</td></tr><tr><td>60 ton</td><td>SWPR19N</td><td>19 - ø21.8 mm</td></tr></table>c) Reinforcing bar (SD295) - D 10 mm, D 13 mm
Lmax = 12 m
Lap length $= \frac{\sigma sa}{4\tau oa} \cdot \varphi$
la: lap joint (cm) {mm}
σsa: allowable tension stress (kgf/cm^2) {N/mm^2}
τoa: allowable adhesive stress (kgf/cm^2) {N/mm^2}
φ: diameter of Re-bar (cm) {mm}</td><td></td></tr>
</table>

<table>
<tr><th>Item</th><th>Specification</th><th>Remarks</th></tr>
<tr><td>4.2.4 Material strength and allowable stress
(1) Concrete</td><td>
Unit: kgf/cm² {N/mm²}
<table>
<tr><th colspan="3">Strength and allowable</th><th>Girder</th><th>In-situ</th></tr>
<tr><td colspan="3">Design strength</td><td>500 {49.1}</td><td>300 {29.4}</td></tr>
<tr><td rowspan="4">Allowable bending</td><td rowspan="2">Just after prestressing</td><td>Rectangle</td><td>210 {20.6}*</td><td>150 {14.7}*</td></tr>
<tr><td>T section</td><td>200 {19.6}</td><td>140 {13.7}</td></tr>
<tr><td rowspan="2">In case of design loading</td><td>Rectangle</td><td>170 {16.7}</td><td>120 {11.8}</td></tr>
<tr><td>T section</td><td>160 {15.7}</td><td>110 {10.8}</td></tr>
<tr><td rowspan="2">Allowable bending tension stress</td><td colspan="2">Just after prestressing</td><td>18 {1.77}</td><td>0 {0}</td></tr>
<tr><td colspan="2">In case of design loading</td><td>18 {1.77}</td><td>0 {0}</td></tr>
<tr><td rowspan="2">Allowable shear stress</td><td colspan="2">Check for service loading</td><td>6.5 {0.637}</td><td>-</td></tr>
<tr><td colspan="2">Maximum at ultimate loading</td><td>60 {5.88}</td><td>-</td></tr>
<tr><td colspan="3">Allowable diagonal tension stress</td><td>12 {1.18}</td><td>-</td></tr>
<tr><td colspan="3">Strength at prestressing</td><td>350 {34.3}</td><td>250 {24.5}</td></tr>
</table>
Remarks for *: real stress shall be less than 1/1.7 of strength at prestressing
For main girder: $a = \frac{350\ \{34.3\}}{1.7} = 205\ \mathrm{kgf/cm^2}\ \{20.1\ \mathrm{N/mm^2}\}$
In-situ: $a = \frac{250\ \{24.5\}}{1.7} = 145\ \mathrm{kgf/cm^2}\ \{14.2\ \mathrm{N/mm^2}\}$
</td><td></td></tr>
</table>

Item	Specification	Remarks

(2) PC tendon

Unit: kgf/mm^2 {N/mm^2}

Item \ Wire		PC cable		
		SWPR 7 BN		SWPR 19 N
		7 wires spiral 12.7 mm 15.2 mm	19 wires spiral 17.8 mm 19.3 mm	19 wires spiral 21.8 mm
Tension strength		190.0 {1,863}	190.0 {1,863}	185.0 {1,814}
Yield stress		160.0 {1,569}	160.0 {1,569}	160.0 {1,569}
Allowable tension stress	At design loading	114.0 {1,118}	114.0 {1,118}	111.0 {1,089}
	After tensioning	133.0 {1,304}	133.0 {1,304}	129.5 {1,270}
	During tensioning	144.0 {1,412}$^{1)}$	144.0 {1,412}$^{2)}$	144.0 {1,412}$^{2)}$

Remarks for 1): Including relaxation loss of tendons at setting and additional stress allowance of 8 kgf/mm^2 {78 N/mm^2}, the following equation is computed:

σ_{PI} = 144.0 {1,412} - 8.0 {78} + 136.0 kgf/mm^2 {1,334 N/mm^2}

2): For transversely tensioning, 3% down of above stress is computed with the consideration of friction at anchorage and additional stress allowance.

(3) Re-bar

Allowable tension stress	1,400 kgf/cm^2 {137 N/mm^2}
Yield stress	3,000 kgf/cm^2 {295 N/mm^2}

4.2.5 Design coefficient

1) Standard setting length and diameter of sheath are as follows:

Tendon	Setting length (mm)	Diameter of sheath (mm)
40 ton, SWPR19N 19 wire spiral 17.8 mm	3.0	42
50 ton, SWPR19N 19 wires spiral 19.3 mm	3.5	42
60 ton, SWPR19N 19 wires spiral 21.8 mm	4.0	42

Item	Specification	Remarks

2) Other design coefficient is subject to Japan Bridge Standard and JIS

a) Young's modulus and relaxation coefficient are as follows:

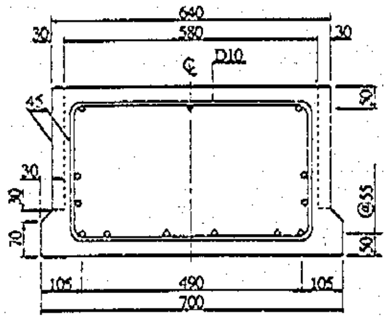
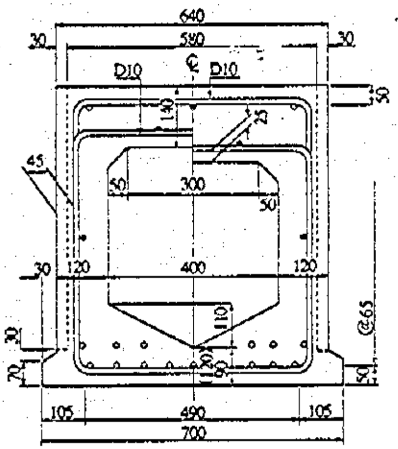
Item			Coefficient
Young's modulus	σ_{ck} = 500 kgf/cm^2 (girder) {49.1 N/mm^2}		3.3 × 10^5 kgf/cm^2 {3.24 × 10^4 N/mm^2}
	σ_{ck} = 300 kgf/cm^2 (In-situ) {29.4 N/mm^2}		2.8 × 10^5 kgf/cm^2 {2.75 × 10^4 N/mm^2}
Young's modulus of PC tendon			2.0 × 10^6 kgf/cm^2 {19.6 × 10^4 N/mm^2}
Relaxation of cable	Girder	Before prestressing	6%*
		After prestressing	5%
	Transverse tension		5%

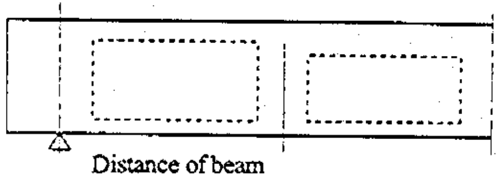
Remarks for *: 2% is added as influenced by curing under high temperature

b) Creep coefficient and shrinkage coefficient for slab bridge:

Item			Coefficient
Creep coefficient	At girder designing	Before composite with filling concrete	1.2
		After filling	1.8
	At transverse beam designing		2.8*
Shrinkage coefficient	At girder designing	Before composite with filling concrete	7 × 10-5
		After filling	13 × 10^{-5}
	At transverse beam designing		20 × 10^{-5}

Remarks for *: Presumed early strength Portland cement

Item	Specification	Remarks
4.2.6 Check for ultimate strength	• The following load combinations are considered: (1) $1.3D + 2.5\ (L + i)$ (2) $1.7\ (D + L + i)$ (3) $1.0\ D + 2.5\ (L + i)$ where, D: Dead load L: Live load i: Impact load	
4.2.7 Design of girder		
(1) Analysis method	1) Plate analysis method established by Dr. Guyyon Massonet is applied for live load, pavement and curb stone etc. loaded after the completion of the deck slab. 2) Self weight of girder and filling concrete are equally loaded on each girders.	
(2) Girder distance	• Installed basically at the distance of less than 0.77 meters.	
(3) Size of section and arrangement of re-bar and PC tendon	 (a) Solid type ● : Re-bar D10 o : PC cable SWPR7BN 7 wires spiral, 12.7 mm  (b) Hollow type ● : Re-bar D10 o : PC cable SWPR7BN 7 wires spiral, 15.2 mm	

<table>
<tr><th>Item</th><th>Specification</th><th>Remarks</th></tr>
<tr><td>(4) Back length of girder</td><td>
<table>
<tr><th>Span (m)</th><th>Back length (m)</th></tr>
<tr><td>L ≤7</td><td>0.15</td></tr>
<tr><td>7 < L ≤9</td><td>0.20</td></tr>
<tr><td>9 < L ≤ 14</td><td>0.25</td></tr>
<tr><td>14 < L ≤ 19</td><td>0.30</td></tr>
<tr><td>19 < L</td><td>0.35</td></tr>
</table>
</td><td></td></tr>
<tr><td>4.2.8 Design of transverse beam
(1) Distance of beam</td><td>
<table>
<tr><th>Span (m)</th><th>Distance (m)</th><th>Span (m)</th><th>Distance (m)</th></tr>
<tr><td>5</td><td>2.5</td><td>15</td><td>3.7</td></tr>
<tr><td>6</td><td>2.0</td><td>16</td><td>4.0</td></tr>
<tr><td>7</td><td>2.3</td><td>17</td><td>4.2</td></tr>
<tr><td>8</td><td>2.6</td><td>18</td><td>4.5</td></tr>
<tr><td>9</td><td>3.0</td><td>19</td><td>3.8</td></tr>
<tr><td>10</td><td>3.3</td><td>20</td><td>4.0</td></tr>
<tr><td>11</td><td>2.7</td><td>21</td><td>4.2</td></tr>
<tr><td>12</td><td>3.0</td><td>22</td><td>4.4</td></tr>
<tr><td>13</td><td>3.2</td><td>23</td><td>4.6</td></tr>
<tr><td>14</td><td>3.5</td><td>24</td><td>4.8</td></tr>
</table>


</td><td></td></tr>
<tr><td>4.2.9 Ancillary structure</td><td>• The specification of the ancillary structure is described in the textual report and related figures.</td><td>• Refer to Figs. 5.22</td></tr>
</table>

Item	Specification	Remarks
4.3 Substructure		
4.3.1 Substructure type	• Pile Trestle according to Indonesian Standard Design "SPESIFIKASI, PILAR DAN KEPALA JEMBATAN SEDERHANA BENTANG 11M - 25M DESIGN PONDASI TIANG PANCANG".	• Figs. 5.20 and 5.21.
4.3.2 Pile shaft		
(1) Principal dimensions		
(a) Width and depth	• In case of more than 11 meters span. - Abutment : Width - 1.8 meters, depth - 0.65 meters - Pier : Width - 2.0 meters, depth - 0.60 meters (11m < L < 16m) depth - 0.65 meters (L > 16m) • In case of less than 11 meters span. - Abutment : Width - 1.2 meters, depth - 0.60 meters - Pier : Width - 2.0 meters, depth - 0.60 meters	
(b) Length	• To be designed according to the width of superstructure.	
(2) Materials		
(a) Concrete	• fc = 22.5 MPA	
(b) Re-bar	• Deformed bar of BJTD24. • Arrangement of re-bar is made subject to the Standards. • Stiffening re-bar will be added according to Indonesian Codes.	
(3) Special provision	• Space for the installation of oil jack is built by means of cutting the corner in shape of rectangular.	

<table>
<tr><th>Item</th><th>Specification</th><th>Remarks</th></tr>
<tr><td>4.3.3 Pile</td><td></td><td></td></tr>
<tr><td>(1) Bearing strength</td><td>• To be designed according to the Indonesian Codes and Standards.</td><td></td></tr>
<tr><td>(2) Diameter</td><td>• Diameter of pile will be selected among the range of 350 to 600 mm.</td><td></td></tr>
<tr><td>(3) Materials</td><td>
<table>
<tr><th>Type</th><th>Code/Standards</th><th>Description</th></tr>
<tr><td>a. Cement</td><td>SII 0013-81</td><td>Ordinary Portland Cement Type I</td></tr>
<tr><td>b. Aggregates</td><td>JIS A 5308</td><td>Aggregates for Ready Mixed Concrete, for coarse aggregate, max. size 20 mm</td></tr>
<tr><td>c. Chemical Admixture</td><td>ASTM C 494</td><td>Standard Specification for Chemical Admixture Type G, Calcium Chloride free</td></tr>
<tr><td>d. Prestressing Steel</td><td>JIS G 3536</td><td>Uncoated Stress Relieved Steel Wire & Strand for Prestressed, Concrete SPWD 1 - 7 mm SPWD 1 - 9 mm</td></tr>
<tr><td>e. Spiral Wire</td><td>JIS G 3532</td><td>Low Carbon Steel Wire SWM-B or equivalent</td></tr>
<tr><td>f. Joint Plate</td><td>JIS G 31011</td><td>Rolled Steel for General Structure SS-41</td></tr>
<tr><td>g. Water</td><td>-</td><td>Shall not contain any detrimental amount of oils, acids, salts, etc.</td></tr>
</table>
</td><td></td></tr>
<tr><td>(4) Concrete strength</td><td>• Compressive Strength test will be done for each daily production work for the age of 1 day (before stress introduction), 7 days and 14 days (at shipment/delivery period) and 28 days accordingly. Characteristic cube strength in accordance with Indonesian Concrete Code (PBI) 1981 should be 600 kg/cm^2 (K 600) or equivalent with minimum cylinder strength (fck') of 500 kg/cm^2.</td><td></td></tr>
<tr><td>(5) Pile bonding test</td><td>• Pile bending test of mainbody shall be made in accordance to clause 8 Bending Strength Test of JIS A 5335 - 1987. Unless specified otherwise, one pile of every 500 piles of the same diameter and type produced will be proof tested by Bending Strength Test. The Test will be considered as satisfactory if no visual crack occurred at the load corresponding to its M Crack.</td><td></td></tr>
</table>

<table>
<tr><th>Item</th><th>Specification</th><th>Remarks</th></tr>
<tr><td>(6) Allowance</td><td>• Appearance and dimension check are done for each finished product with the following criterions:
<table>
<tr><th>Description</th><th>Tolerance</th></tr>
<tr><td>Crack</td><td>No Visual Crack</td></tr>
<tr><td>Outside</td><td>+5 mm
-2 mm</td></tr>
<tr><td>Wall thickness</td><td>-0 mm
+ not specified</td></tr>
<tr><td>Length</td><td>0.3% of PC Pile Length</td></tr>
<tr><td>Angle between joint plate and pile axis</td><td>90 ± 20'</td></tr>
</table></td><td></td></tr>
<tr><td>4.3.4 Access road</td><td></td><td></td></tr>
<tr><td>(1) Standard</td><td>• To be designed according to Indonesian Codes and Standard related to road geometric alignment and material qualities.</td><td></td></tr>
<tr><td>(2) Design</td><td></td><td></td></tr>
<tr><td>(a) Alignment</td><td>• To be designed basically according to the related Standards.
• The grade of access road shall be decided after the study of environmental conditions such as topographical relation with residence, traffic volume and car classification.</td><td>• Refer to 4.2.5(2).</td></tr>
<tr><td>(b) Road layer composition</td><td>• To be designed subject to prevailing specification.</td><td></td></tr>
<tr><td>(c) Slope protection</td><td>• Wet masonry for less than 1 meter high and block masonry for over 1 meter high.</td><td>• Fig. 5.23</td></tr>
</table>

(1/2)

Table 3.2 **DESIGN SPECIFICATION FOR PEDESTRIAN BRIDGE**

Item	Specification	Remarks
1. General	• This specification is applied for the design of pedestrian bridges. • Principal items are described in this papers.	
2. Applicable Codes and Standards	• Since there is no codes and standards issued for the pedestrian bridge in Indonesia, the related items to the pedestrian bridge in the existing Codes and Standards are referred jointly with Japan Pedestrian Standard published by Japan Road Association.	
3. Bridge Location	• Same place as existing bridge location.	
4. Design Conditions		
4.1 Principal Dimensions	• Same place as existing bridge location.	• Refer to Tables 5.3, 5.4 and 5.5.
4.2 Superstructure		
4.2.1 Type of structure	• Pre-cast deck slab supported with pre-tension girders.	
4.2.2 Structure on deck slab		
(1) Rail	• Steel railing.	
(2) Pavement	• 20 mm cement mortar.	
4.2.3 Load		
(1) Dead loads	• Same as described for road bridge.	
(2) Live load	• Deck slab and beam : 500 kg/m^2 • Girder : 350 kg/m^2 • Railing : H = 250 kg/m (Horizontal force)	

Item	Specification	Remarks
4.2.4 Material	• Same as described for road bridge.	
4.3 Substructure	• Description differed from that of road bridge is as follows: • Abutment : Width - 1.2 meters, depth - 0.5 meters Pier : Width - 1.2 meters, depth - 0.5 meters • PC pile will be used.	
4.4 Access Road	• Slope or stairs will be chosen taking into account the traffic classification.	

(1/2)

Table 3.3 DESIGN SPECIFICATION FOR CULVERT

Item	Specification	Remarks
1. General	• This specification is applied for the design of culvert.	
2. Codes and Standards	• Indonesian Codes and Standards properly supplemented with Design Manual of Japan Highway Authority.	
3. Opening	• To be designed according to the widening plan of channel. • Same vertical clearance at H.W.L. as in existing culvert will be basically adapted for new culvert.	• Refer to Fig. 5.16. • Refer to Fig. 5.24.
4. Measures for Land Subsidence	• Measures for land subsidence can be selected among two method, i.e. heightening of both supporting walls for box culvert or installation of new culvert for pipe culvert.	• Refer to Fig. 5.25.
5. Culvert Type	• Box culvert - in case of being positioned close to road surface or relatively shallow place. • Pipe culvert - in case of being positioned at deep subsurface where new installation is possible to install in double with keeping distance required for structural stability.	
6. Design		
6.1 Box Culvert		
(1) Earth covering	• Basically more than 50 cm (compounds of pavement layers). • In case of positioned close to surface, deck slab shall be designed as a part of bridge.	
(2) Loads	• Dead load: Self weight of earth and deck slab, earth and water pressure. • Live load: Same as bridge loading.	

Item	Specification	Remarks
(3) Analyse method	• According to the Design Manual.	
(4) Design detail	• Same as above.	
(5) Materials	• Concrete: fc = 240 kg/cm^2 {24 MPA} • Re-bar: D10 and D13	
6.2 Pipe Culvert		
(1) Pipe classification	• Centrifugal RC Pipe or PC Pipe produced according to JIS, which is checked by Indonesian Industrial Standards.	
(2) Foundation	• It's selected among three options depend upon soil conditions., i.e. concrete, sand and existing bearing layer.	
(3) Loads	• Same as stipulated in Box Culvert.	
(4) Analyse method	• According to the Design Manual.	
(5) Design detail	• Same as above.	

Figures

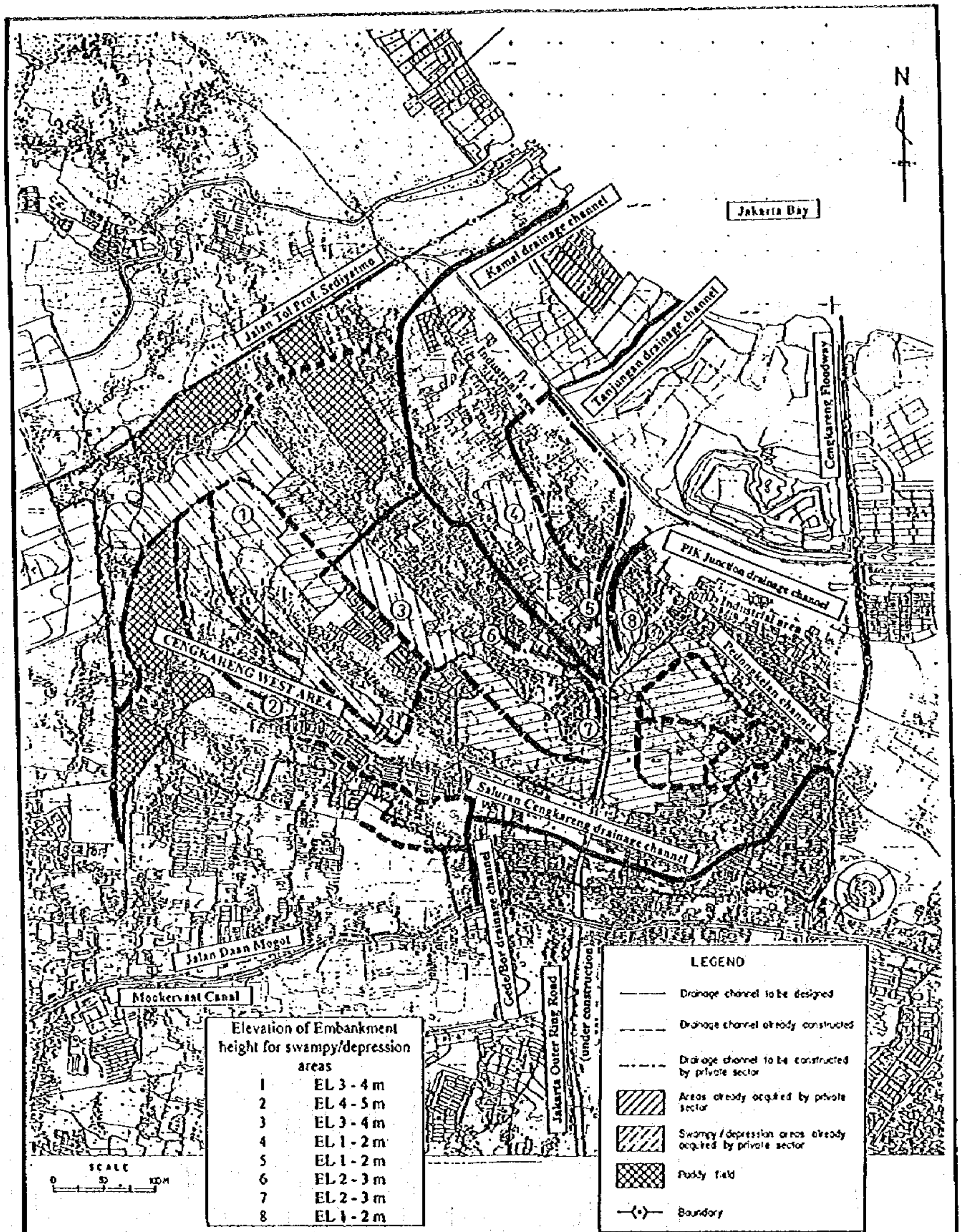


Fig. 2.1 Design Stretches of Drainage Channels in Cengkareng West Area

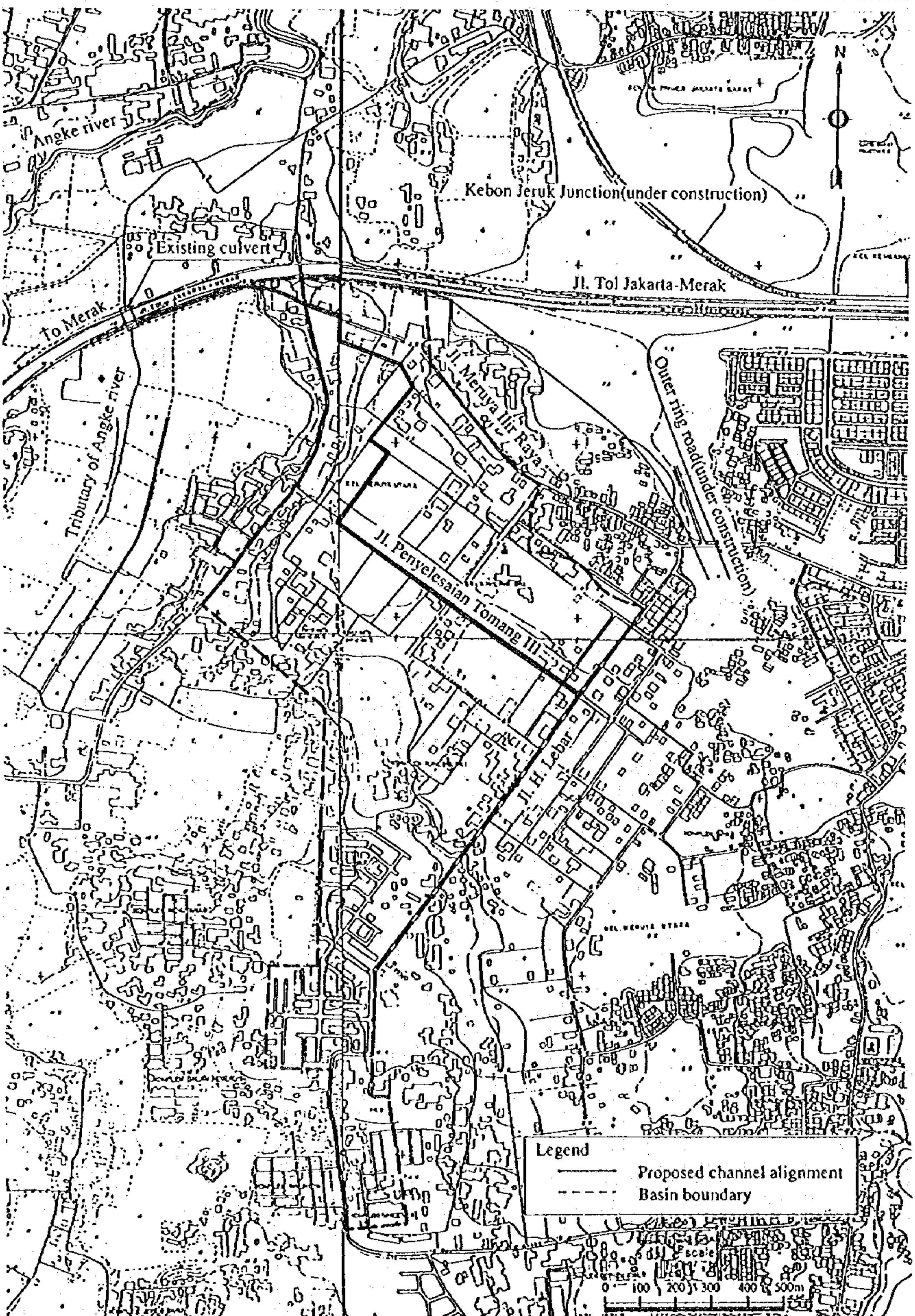
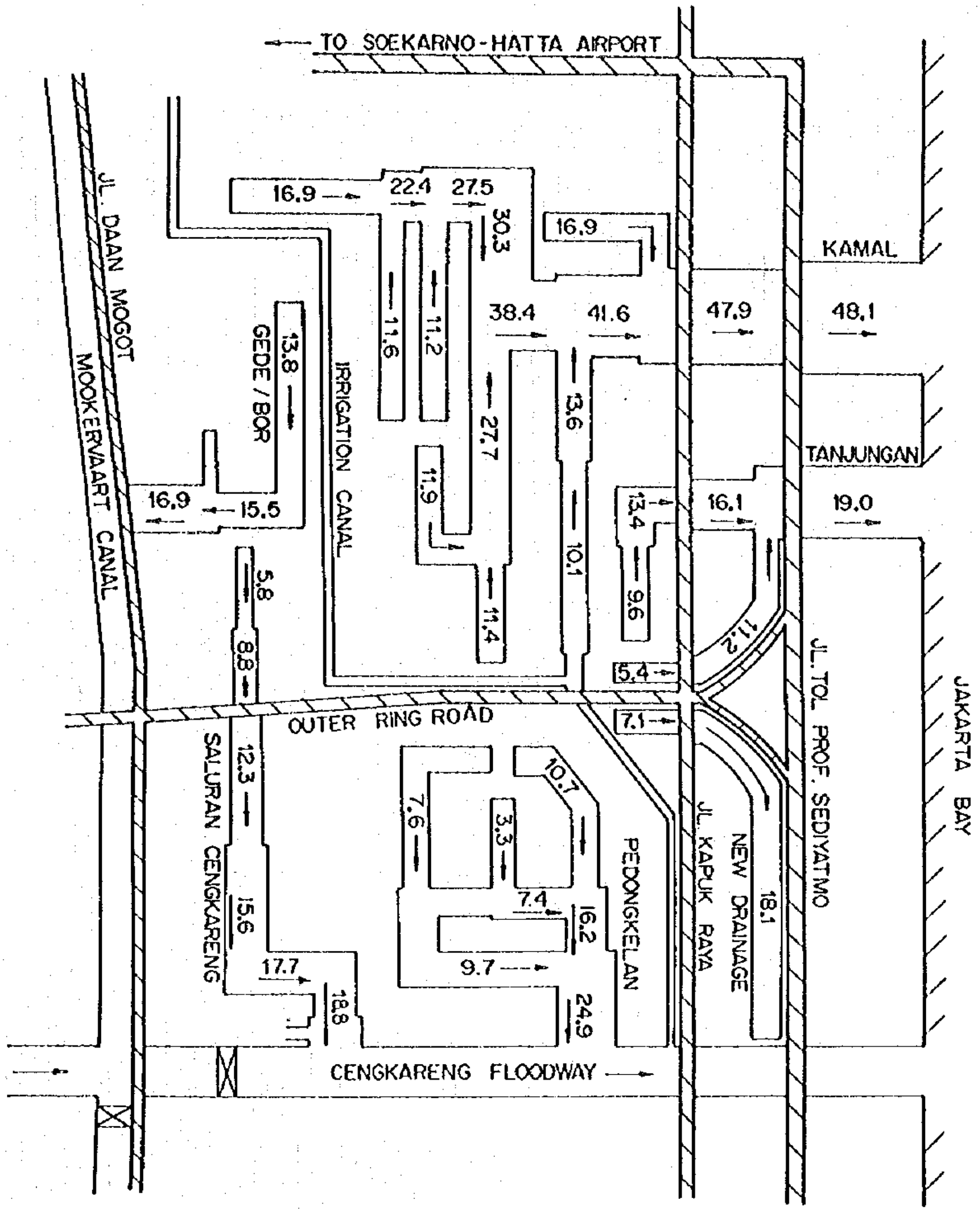
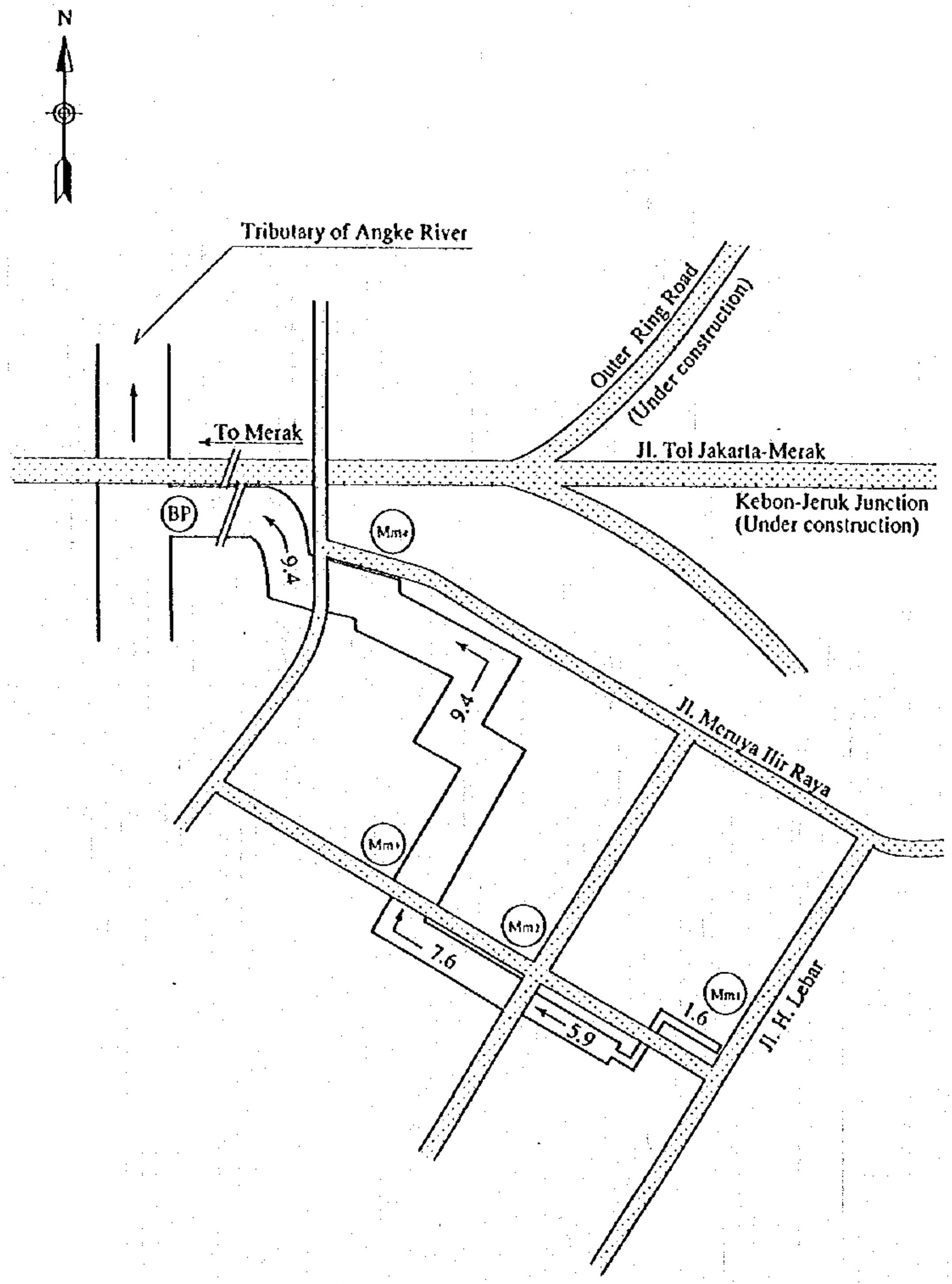


Fig. 2.2 Design Stretches of Drainage Channels in Meruya Area



(unit : m³/s)

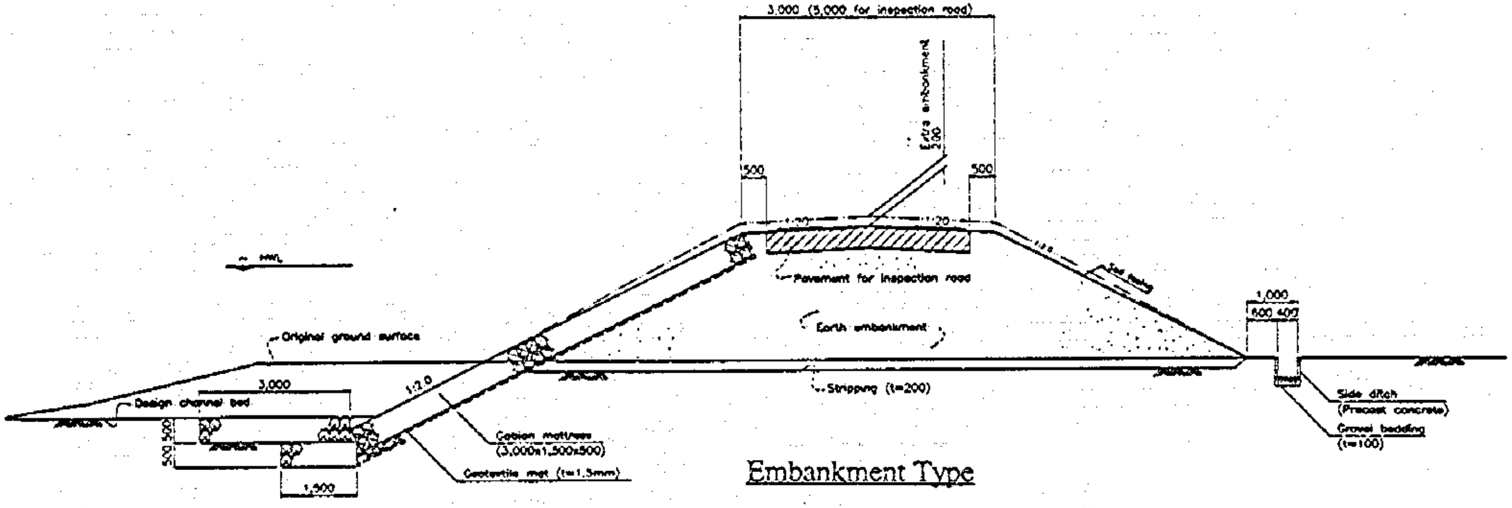
Fig. 2.3 Design Flood Discharge Distribution in Cengkareng West Area



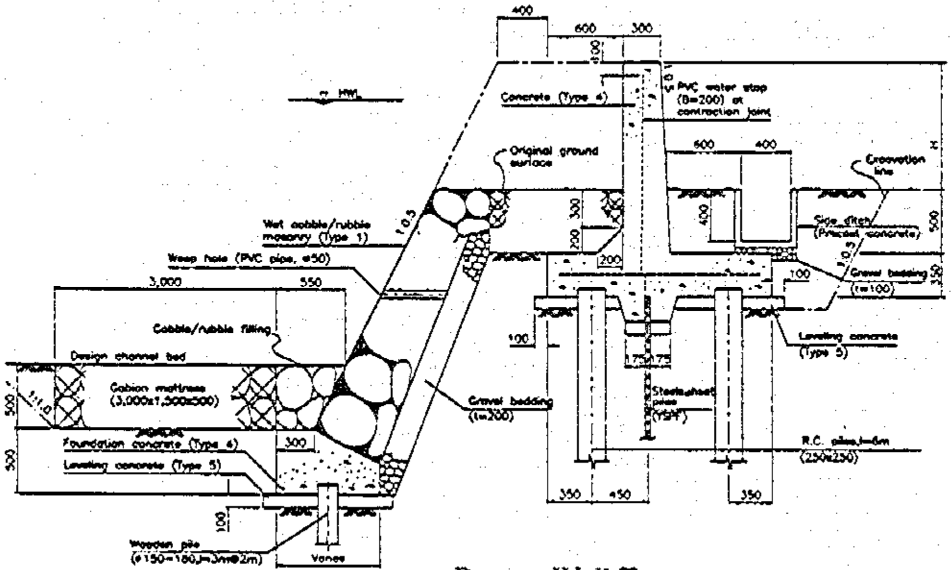
Note: Value : Design Discharge (m^3/s)

Mm : Code No.

Fig. 2.4 Design Flood Discharge Distribution in Meruya Area



Embankment Type



Parapet Wall Type

Fig. 2.5 Typical Section of Levee

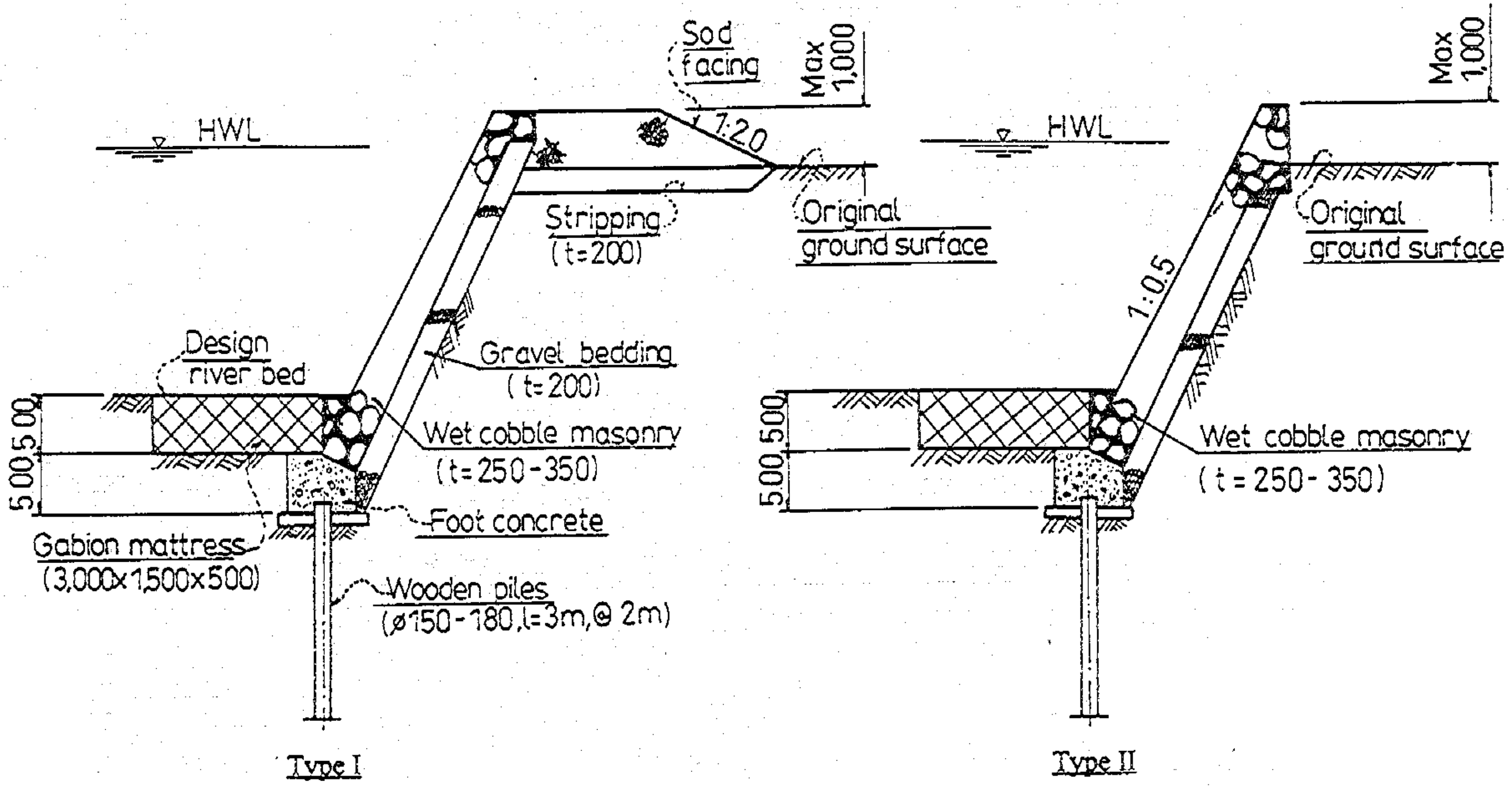
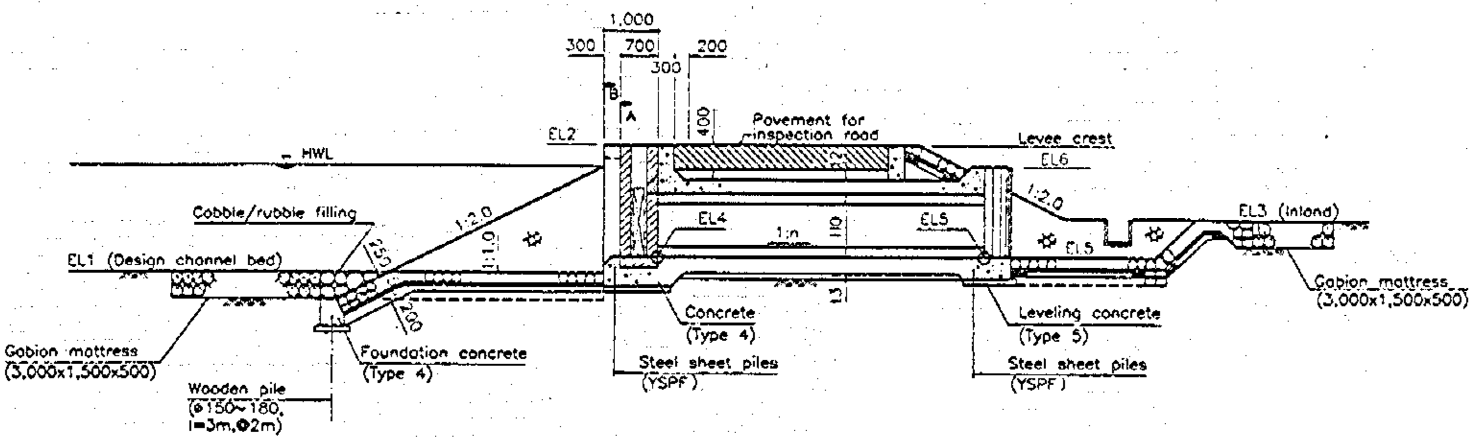
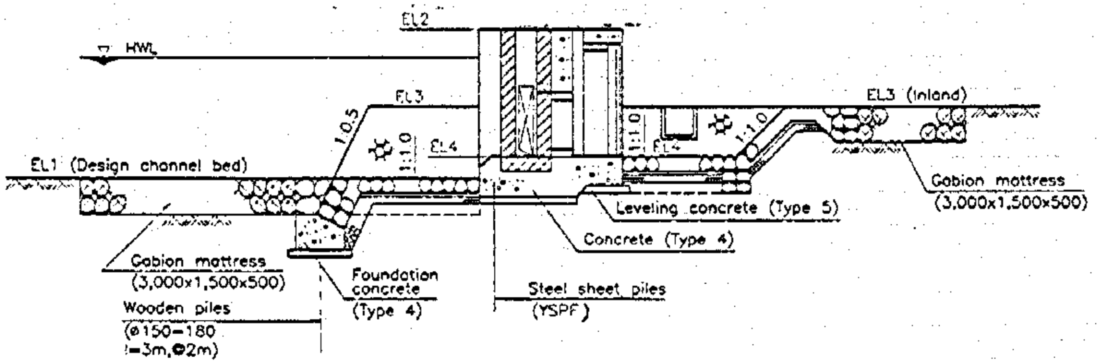


Fig. 2.6 Typical Section of Revetment



in levee embankment type



in levee parapet wall type

Fig. 2.7 Typical Section of Sluiceway

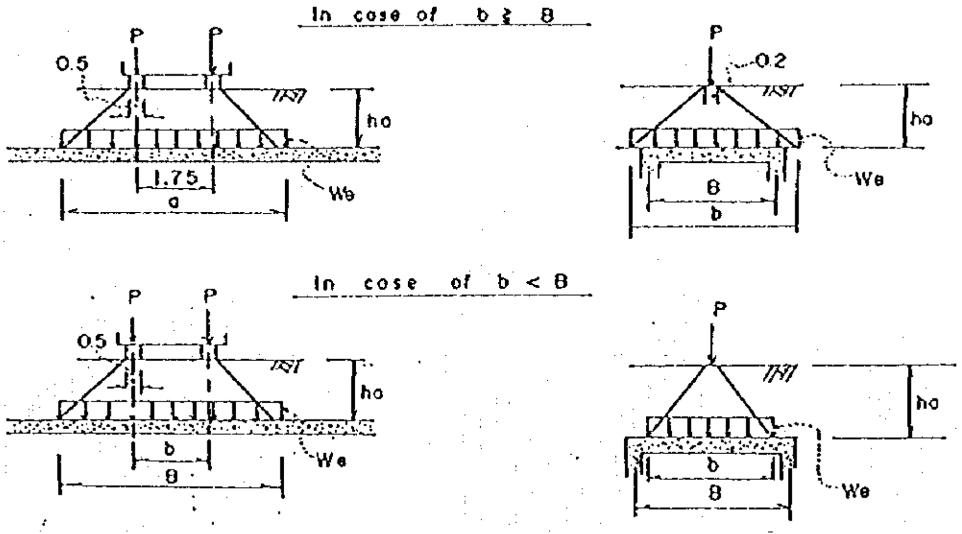


Fig. 2.8 Vertical Load on Top Slab of Conduit due to Live Load

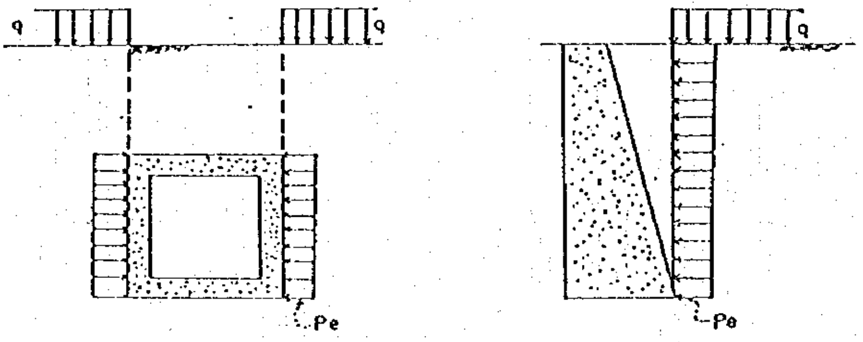


Fig. 2.9 Horizontal Load on Wall due to Live Load

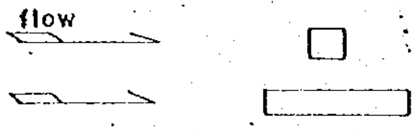
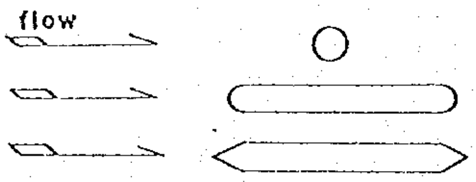
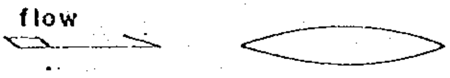
Cross section of Pier	K
	0.07
	0.04
	0.02

Fig. 2.10 Coefficient of Dynamic Water Pressure due to Flowing Water

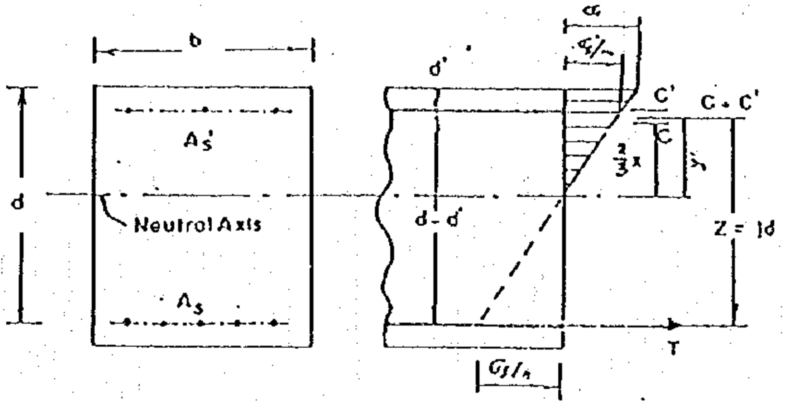


Fig. 2.11 Reinforcement of Rectangular Section

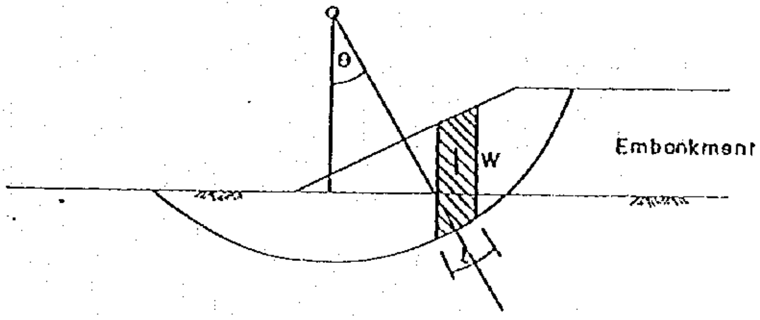
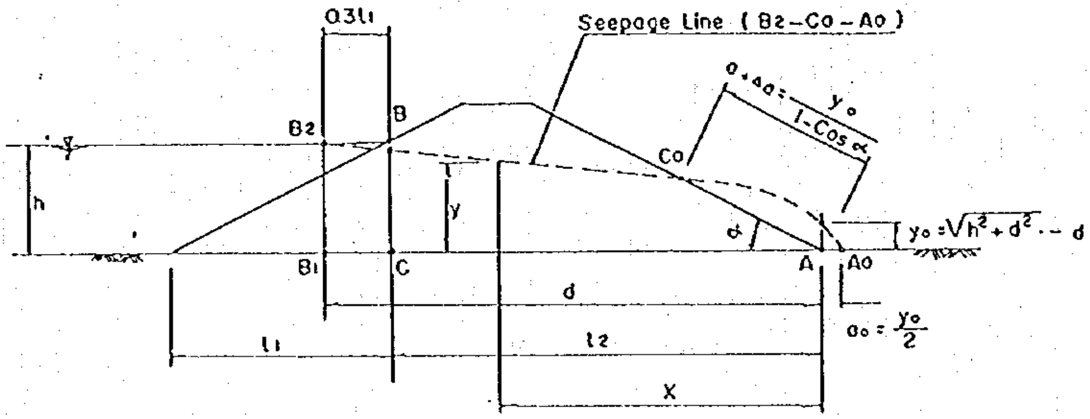
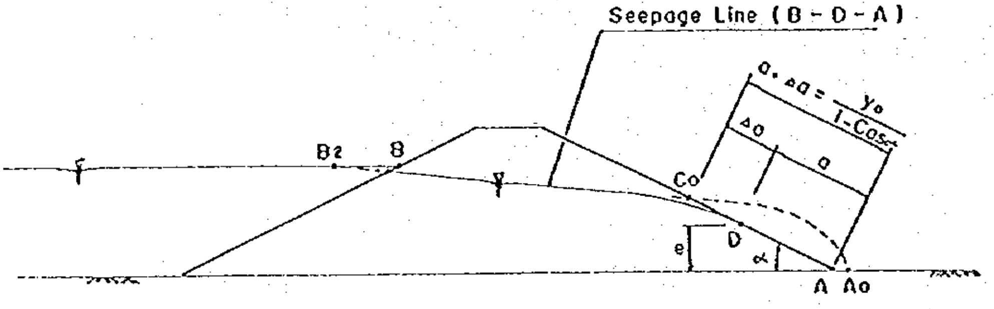


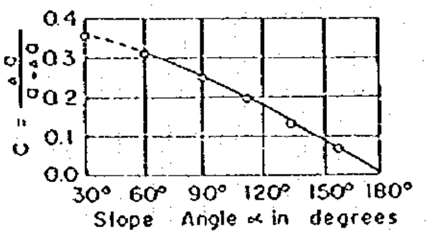
Fig. 2.12 Stability Analysis of Leveeing



(a) Seepage line of Basis



(b) Seepage line of Modification



(c) Relation between α & C

Fig. 2.13 Seepage Analysis in Levee

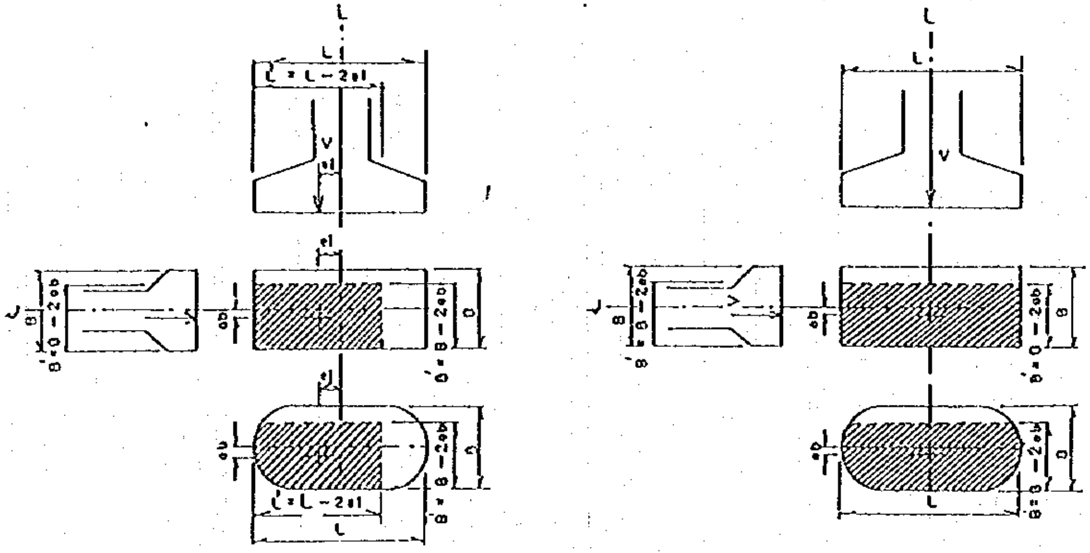


Fig. 2.14 Effective Area of Footing for Bearing Capacity

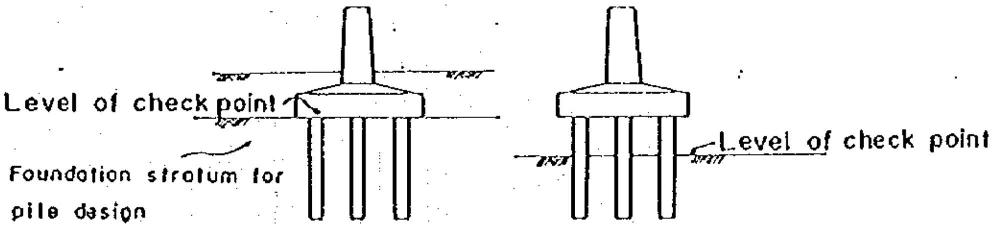


Fig. 2.15 Check Point of Pile Deflection

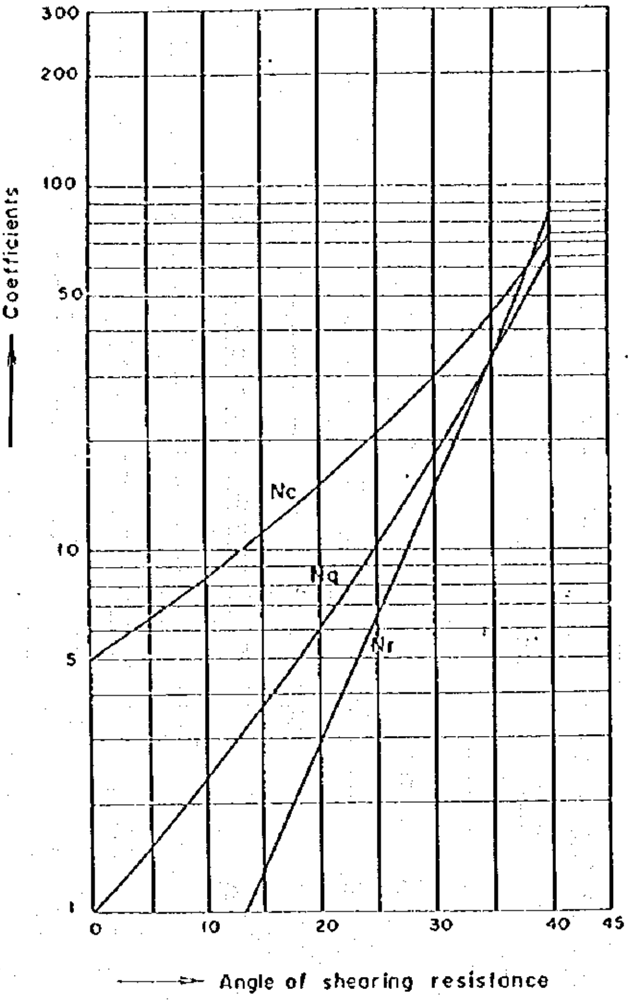
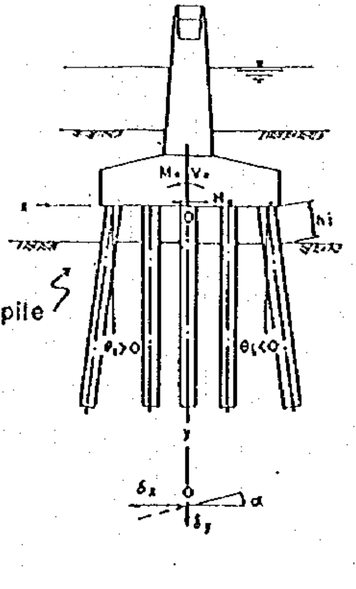


Fig. 2.16 Nc, Nr, Nq and Internal Friction Angle



Foundation stratum for pile design.

Fig. 2.17 Deflection of Pile Foundation

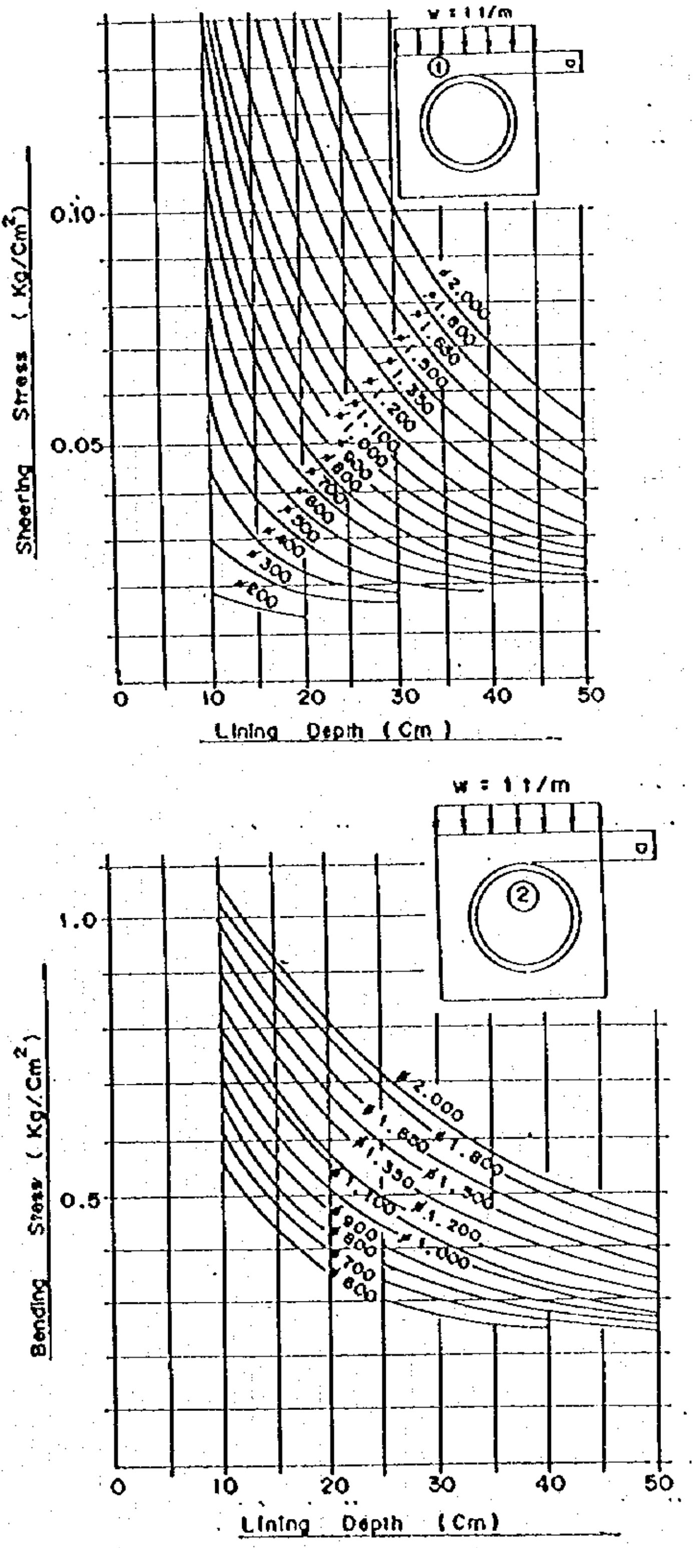


Fig. 2.18 Design Diagram of Circular Type Conduit

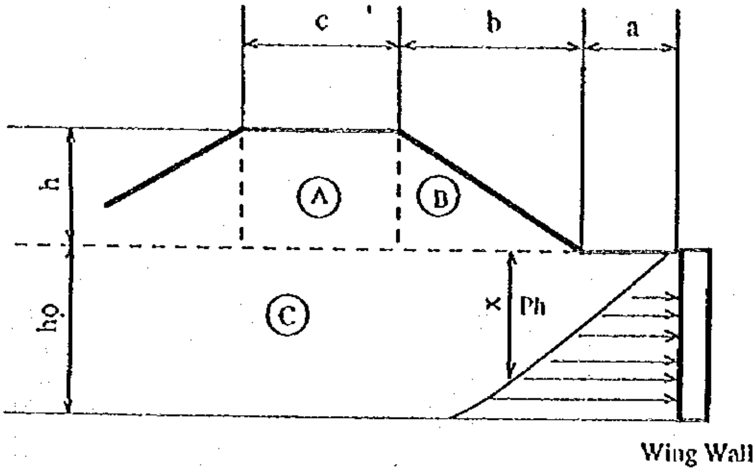
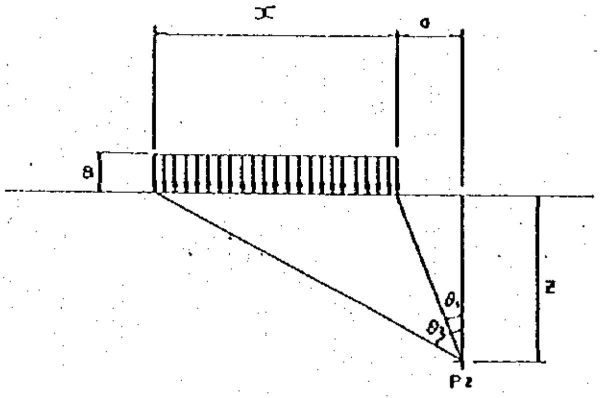


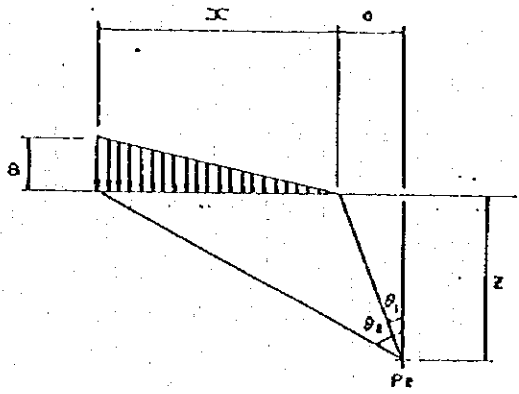
Fig. 2.19 Earth Pressure at Rest on Wing Wall

Embankment Ⓐ



$$Pz = \frac{q}{\pi}\left[\left\{\tan^{-1}\frac{(x+a)}{z} - \tan^{-1}\frac{x}{z}\right\} + \left\{\frac{z(x+a)}{z^2+(x+a)^2} - \frac{zx}{z^2+x^2}\right\}\right]$$

Embankment Ⓑ



$$Pz = \frac{q}{\pi}\left[\frac{z(x+a)}{z^2+(x+a)^2} - \frac{x}{a}\left\{\tan^{-1}\frac{(x+a)}{z} - \tan^{-1}\frac{x}{z}\right\}\right]$$

Fig. 2.20 Vertical Earth Pressure at Rest due to Embankment

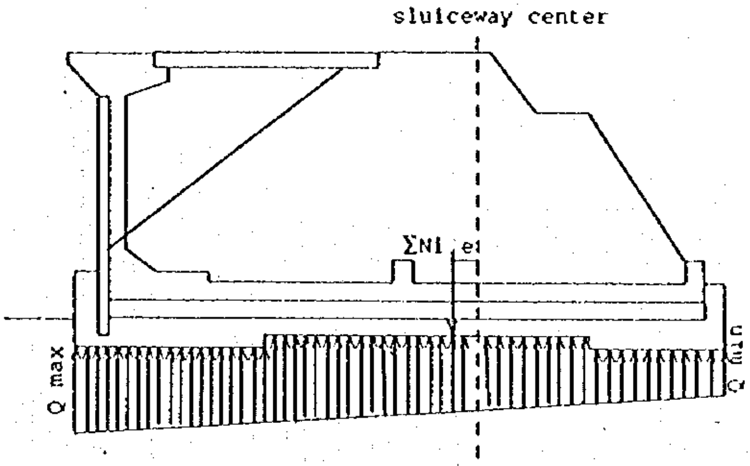


Fig. 2-21 Distribution of Ground Reaction at Sluiceway