CHAPTER 3 STABILITY ANALYSIS

3.1 Stability Analysis of Canal Bank
3.2 Stability Analysis of Dike Create Detention Pond
3.3 Optimum Slope of Excavation of No.7 Pumping Station
3.4 Foundation Design of No.7 PS and Other Structures
3.5 Pavement Design of Access Road and OM Road

CHAPTER 3 STABILITY ANALYSIS

3.1 Stability Analysis of Canal Bank

The stability analysis is made of the canal bank slope by the slip circle method. The following equation is used for necessary calculation.

$$F.S. = \sum[(N - U) \cdot \tan\phi + C \cdot \ell] / \sum T$$

Where,	F.S.	;	Factor of safety
	N	;	Normal force acting on slip circle of each slice
	U	;	Residual pore pressure acting on slip circle of each slice
	ϕ	;	Internal friction angle of materials on slip circle of each slice
	C	;	Cohesion of materials on slip circle of each slice
	ℓ	;	Arc length of slip circle of each slice
	T	;	Tangential force acting on slip circle of each slice

The allowable safety factor taken in the stability analysis should be comprehensively determined taking into consideration input constants of materials, methods of slope trimming, importance of structures, etc. In this study, the value of the safety factor is taken to be 1.2 with an allowance of 0.2.

The slope stability analysis was carried out for the following two sections.

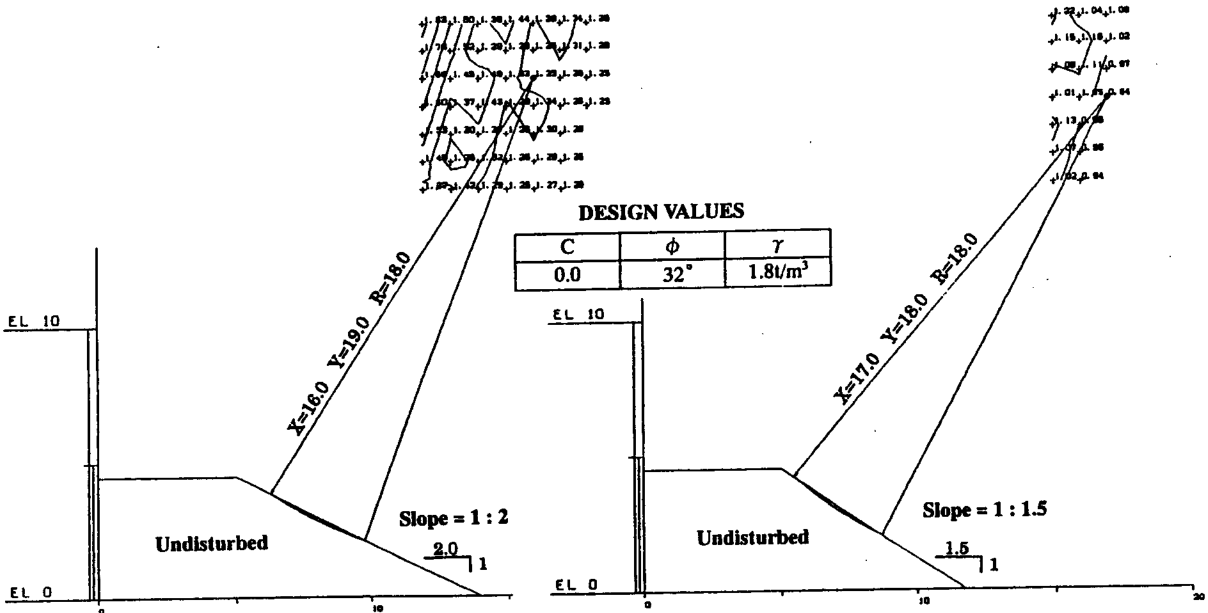
Section 1 : Side Slope of Concrete Lined Section
Section 2 : Slopes other than Concrete Lined Section

The results of the analysis are shown in Figure 3.1-1 and Table below.

Table 3.1-1 Results of Slope Stability Analysis of Canal Banks

Canal Banks	Slope	Safety Factor	Remarks
Section 1 : (Concrete Lined Section)	1 : 2.0	◎ 1.25 > 1.2	Adopted
	1 : 1.5	▲ 0.94 < 1.2	
Section 2 : (Other Banks)	1 : 3.0	◎ 1.33 > 1.2	Adopted
	1 : 2.5	▲ 1.11 < 1.2	

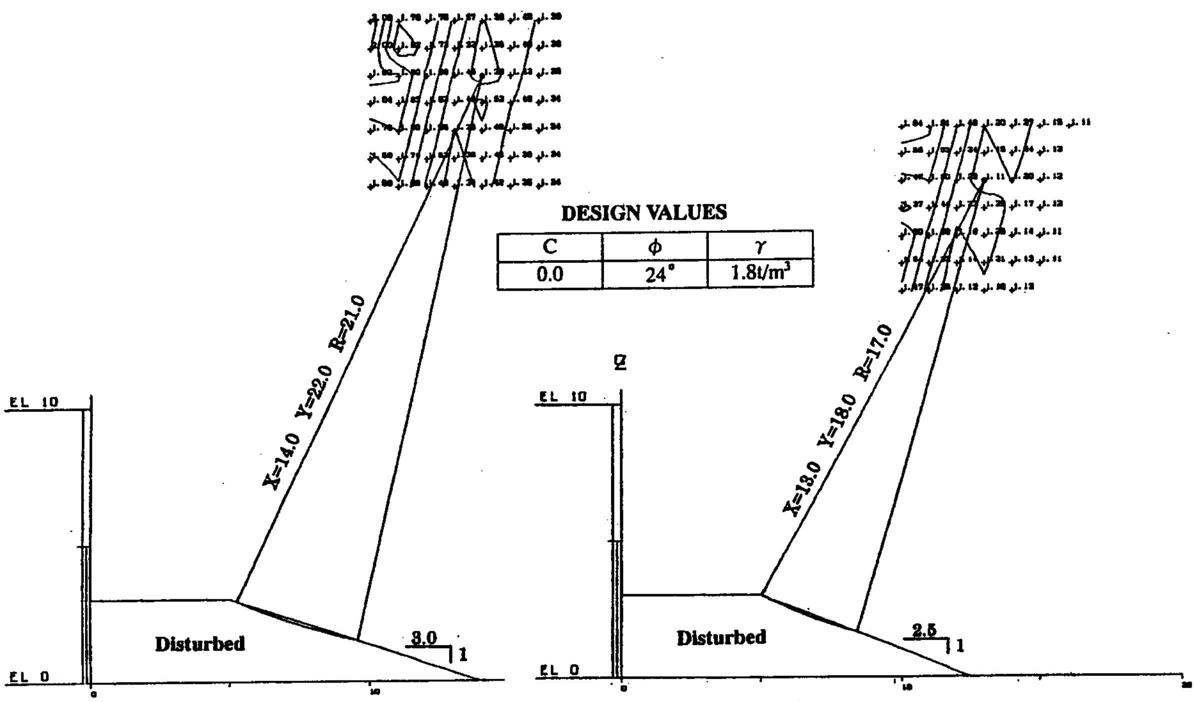
NOTE : For detail of stability analysis, see Appendix B.3.1-1.



DESIGN VALUES

C	ϕ	γ
0.0	32°	1.8t/m^3

Section 1 : Side Slope of Concrete Lined Section



DESIGN VALUES

C	ϕ	γ
0.0	24°	1.8t/m^3

Section 2 : Slopes other than Concrete Lined Section

Figure 3.1-1 Stability Analysis of Canal Banks

3.2 Stability Analysis of Dike to Create Detention Pond

The stability analysis is made of the dike slope as shown in Figure 2.4-7 by the slip circle method as shown in Chapter 3.1. The following two water level conditions are taken for stability analysis.

Case-1 : Detention Water Level = EL. 3.4 m (Upstream Slope)
Case-2 : No Stored Water (Empty) = EL. 0.0 m (Downstream Slope)

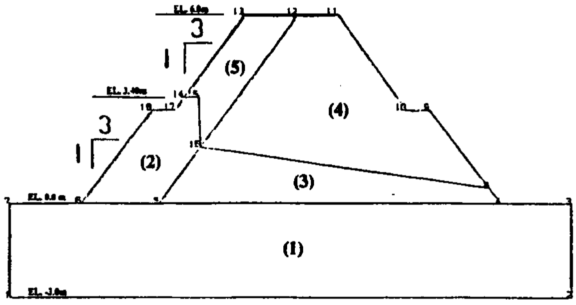
The calculations of stability analysis were carried out repeatedly with different slip circles to obtain the smallest value of the safety factor. The results of the analysis are shown in Figure 3.2-1 and Table 3.2-1.

Table 3.2-1 Results of Slope Stability of Dike

Case	Slope	Safety Factor	Water Level
Case-1	1 : 3.0	◎ 1.37 > 1.2	EL.3.4 m
Case-2	1 : 3.0	◎ 1.32 > 1.2	EL.0.0 m

NOTE : For detail of stability analysis of dike, see Appendix B.3.2-1.

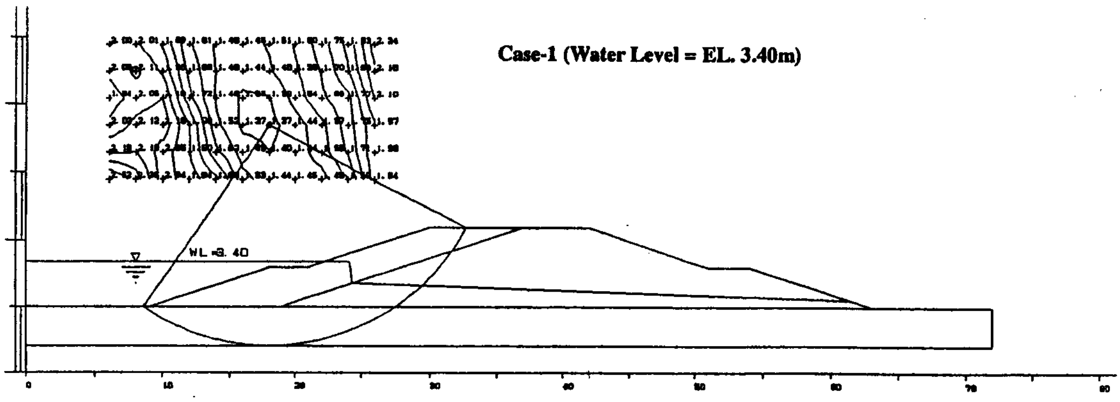
The safety factors resulting from this analysis are more than the allowable value for both Case-1 and Case-2.



ZONING OF DIKE

DESIGN VALUES FOR STABILITY ANALYSIS

Zone	Case-1 (W.L. EL.3.40m)			Case-2 (W.L. EL.0.0m)		
	C	ϕ	γ	C	ϕ	γ
1	0	22°	0.8t/m³	0	22°	0.8t/m³
2	0	35°	1.0t/m³	0	32°	1.8t/m³
3	0	32°	0.8t/m³	0	32°	1.8t/m³
4	0	32°	1.8t/m³	0	32°	1.8t/m³
5	0	35°	2.0t/m³	0	32°	1.8t/m³



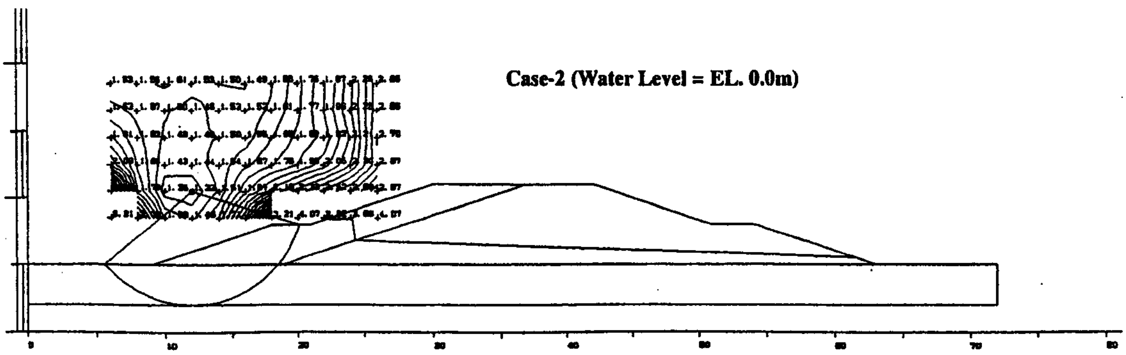


Figure 3.2-1 Stability Analysis of Dike

3.3 Optimum Slope of Excavation of No.7 Pumping Station

The stability analysis was made for proposed excavation slope by the slip circle method as shown in Chapter 3.1.

3.3.1 Analysis Conditions

In the results of the geological investigation at the No.7 Pumping Station, N-value of the upper layer (EL.13.40 m ~ EL.- 2.50 m) and the lower layer (below EL.- 2.50 m) are 12 ~ 50 and more than 50, respectively. The analysis conditions of each layer are as follows;

The internal friction angle of materials shall be estimated by Dunhum's equation as follows;

$$\phi = \sqrt{12N_0} + (15 \sim 25)$$

where, ϕ ;Internal friction angle of materials on slip circle of each slice (degree),
N_0 ; N-value of standard penetration tests

Table 3.3-1 Analysis Conditions of Excavated Layers

Description	Upper Layer (EL.13.40m ~ EL.-2.50m)	Lower Layer (below EL.-2.50m)	Remarks
N-value	$N_0 = 12$	$N_0 = 50$	At No.5 Bore-hole
Internal friction angle	$\phi = 30°$	$\phi = 40°$	
Cohesion of materials	$C = 0$ tf/m^2	$C = 0$ tf/m^2	
Soil unit weight	$\gamma = 1.80$ tf/m^3	$\gamma = 1.80$ tf/m^3	

3.3.2 Stability Analysis

The water level conditions in analysis are in the below of the excavation surface. The value of the safety factor for excavated slopes is taken to be 1.1 with an allowance of 0.1.

The excavated slope stability analysis was carried out for the following two sections.

Section 1 : Upper Layer (between EL. 13.40 m and EL. – 2.50 m)
Section 2 : Lower Layer (below EL. – 2.50 m)

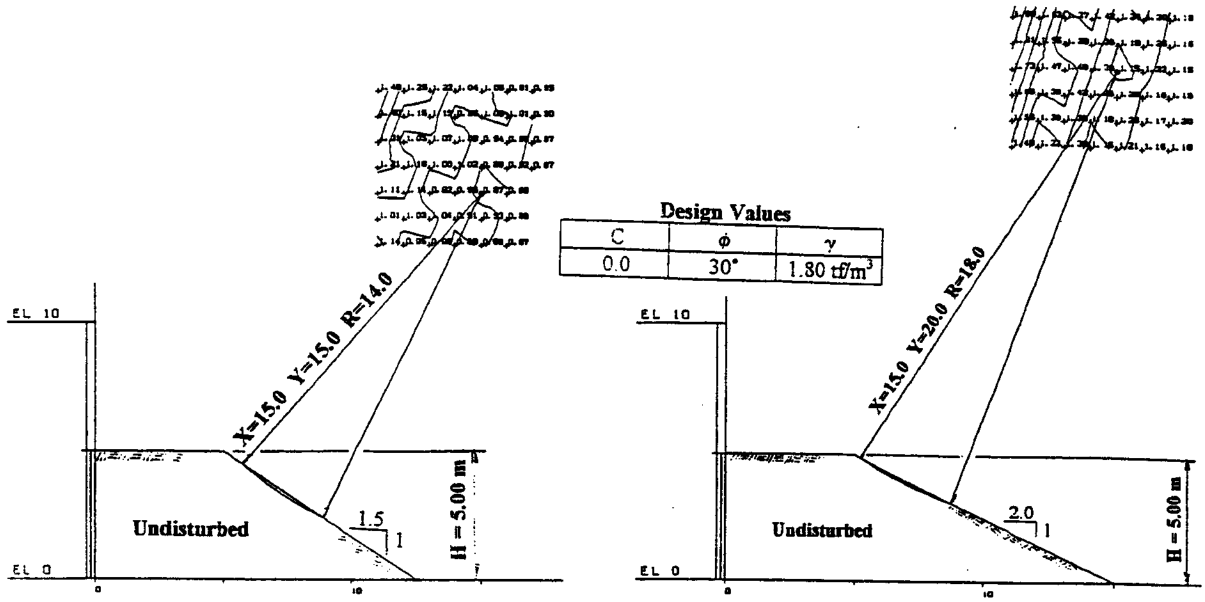
The results of the analysis are shown in Table 3.3-2 and Figure 3.3-1.

Table 3.3-2 Results of Slope Stability Analysis of Excavation

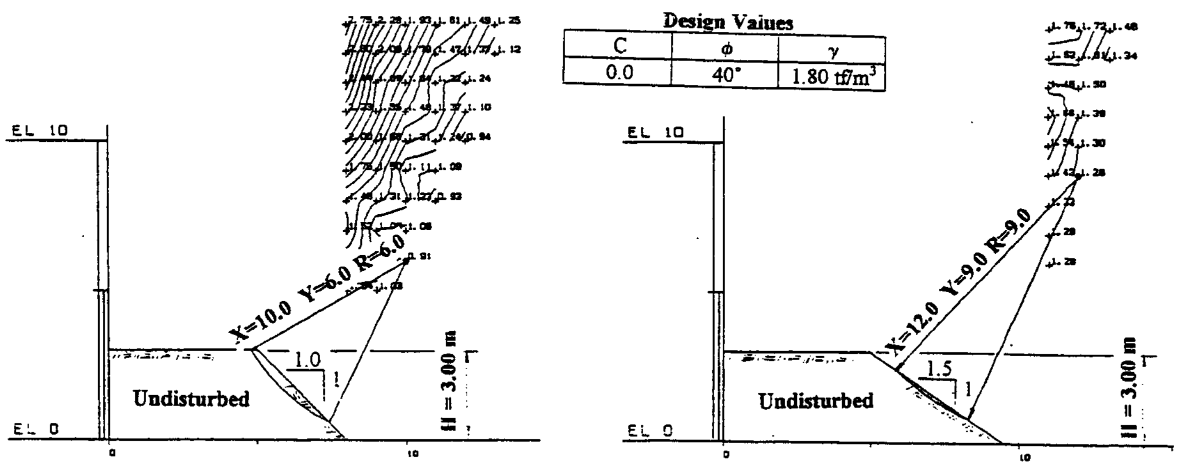
Slopes of Excavation	Slope	Safety Factor	Remarks
Section 1:	1 : 1.5	▲ 0.87 < 1.1	
(Upper Layer)	1 : 2.0	◎ 1.16 > 1.1	Adopted
Section 2:	1 : 1.0	▲ 0.91 < 1.1	
(Lower Layer)	1 : 1.5	◎ 1.27 > 1.1	Adopted

Note : For detail of stability analysis, see Appendix-B.3

In the above results of stability analysis of the excavated slopes, the optimum excavation slopes of the upper and lower layers are at 1 : 2.0 and 1 : 1.5, respectively.



Section 1: Excavated Slope of Upper Layer



Section 2: Excavated Slope of Lower Layer

Figure 3.3-1 Slope Stability Analysis of Excavation of No.7 P.S.

3.4 Foundation Design of No.7 PS and Other Structures

3.4.1 Foundation Design of No.7 Pumping Station

This section will describe the foundation treatment of the pumping station based on the results of the bore hole drilling survey.

(1) Sub-surface Condition of Pumping Station

Ten bore holes with depth of 40 m were drilled in and around the site of the pumping station. Most of the bore holes give the similar character of sub-surface condition. The geological log at the pumping house is shown in Figure 3.4-1. The foundation consists of fine-medium sand partly with thin silty clay. The sub-surface condition is judged as favorable for the foundation of pumping station with N value of standard penetration test more than 50.

(2) Examination of Bearing Capacity

(a) Examination Conditions of Pumping Station

The examination conditions of No.7 Pumping Station are in the following table.

Angle of internal friction of sand can be estimated by Dunhum equation: $\phi = \sqrt{12N} + 15$,and it of clay is 25 % of Dunhum equation

Table 3.4-1 Examination Conditions of Pumping Station

Description	Symbol	Suction Sump	Pumping Station	Main. Space
Location ;	KM	108.865	108.900	108.900
Width ;	B	20.00 m	21.50 m	16.00 m
Length	L	35.50 m	52.50 m	21.50 m
Elevation of foundation	EL.	EL. 2.90 m	EL. - 6.70 m	EL. - 6.70 m
N-value of foundation (sandy layer)	N_1	50	50	50
N-value of foundation (clay layer)	N_2	50	50	50
Internal friction angle	ϕ	39°	39°	39°
Depth from deepest ground surface	D_f	0.00 m	9.60 m	9.60 m
Unit weight of below foundation	γ_1	1.00 tf/m^3	1.00 tf/m^3	1.00 tf/m^3
Unit weight of above foundation	γ_2	1.80 tf/m^3	1.00 tf/m^3	1.00 tf/m^3
Vertical load	V	25.00 tf/m^2	40.00 tf/m^2	50.00 tf/m^2
Horizontal load	H	0 tf/m^2	0 tf/m^2	0 tf/m^2
Inclination of load ($\tan\theta$ = H / V)	θ	0.00	0.00	0.00
Maximum Soil Reaction	Q	25.00 tf/m^2	40.00 tf/m^2	50.00 tf/m^2
Shape of foundation plate		Square	Square	Square

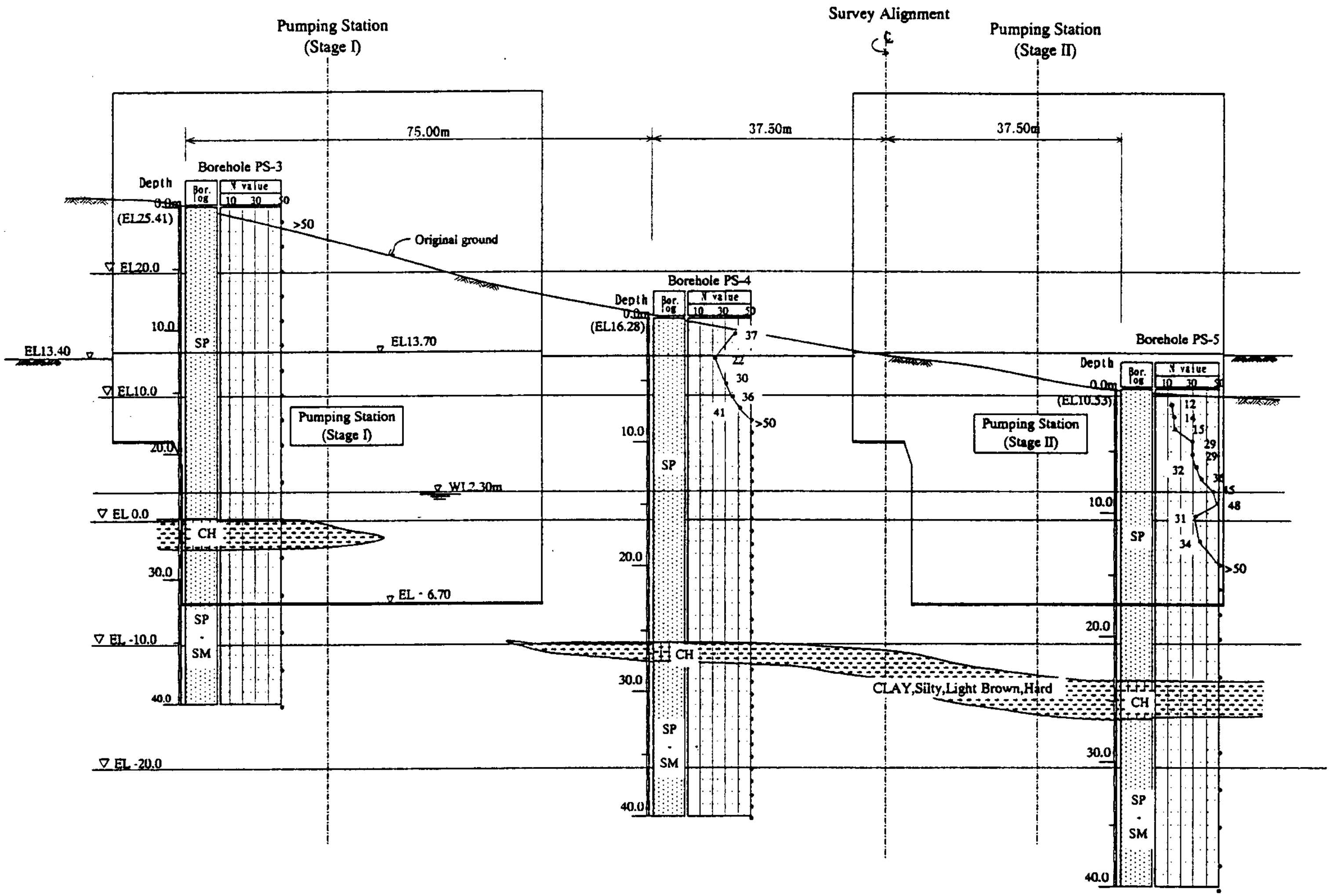


Figure 3.4-1 Results of Drilling Works at No.7 Pumping Station

(b) Examination of Bearing Capacity

The bearing capacity of the foundation will be estimated by the modified Terzaghi equation.

- Long term allowable bearing capacity

$$q_a = \frac{1}{3} \cdot \left(\alpha \cdot C \cdot N_c + \beta \cdot \gamma_1 \cdot B \cdot N_\gamma + \gamma_2 \cdot D_f \cdot N_q\right) \text{ (tf/m}^2\text{)}$$

- Short term allowable bearing capacity

$$q_a = \frac{2}{3} \cdot \left(\alpha \cdot C \cdot N_c + \beta \cdot \gamma_1 \cdot B \cdot N_\gamma + \frac{1}{2} \cdot \gamma_2 \cdot D_f \cdot N_q\right) \text{ (tf/m}^2\text{)}$$

where, q_a : Allowable bearing capacity (tf/m^2)

C : Cohesion of ground below foundation load surface (tf/m^2)

Sand: C = 0 tf/m^2

Clay: C = 1/1.6 · N tf/m^2

γ_1 : Unit weight of ground below foundation load surface (tf/m^3) When it is below groundwater level, use the submerged unit weight.

γ_2 : Average unit weight of ground above foundation load surface (tf/m^3). When it is below groundwater level, use the submerged unit weight

α, β : Shape factor

N_c,N_γ,N_q : Coefficient of bearing capacity, which is a function of angle of internal friction

D_f : Depth from deepest ground surface adjacent to foundation to foundation load surface (m)

B : Minimum width of foundation load surface. In case of circular shape, use diameter (m).

Table 3.4-2 Shape Factor

Shape of Foundation / Shape Factor	Continuous	Square	Rectangle	Circle
α	1.0	1.3	1.0 + 0.3 × B/L	1.3
β	0.5	0.4	0.5−0.1 × B/L	0.3

Note; B : Length of short side of rectangle

L : Length of long side of rectangle

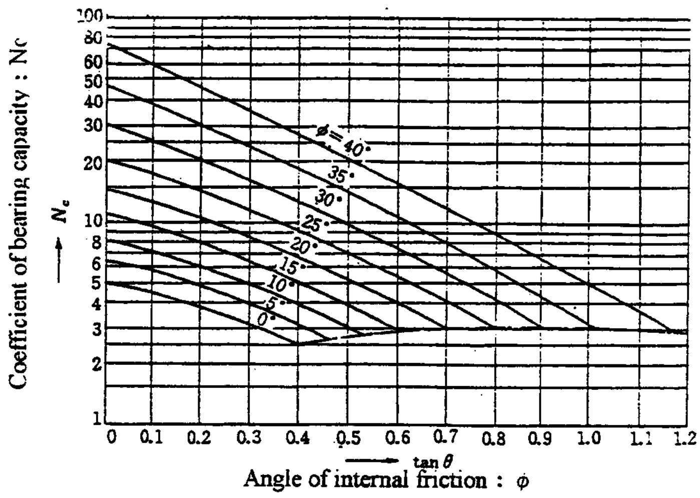


Figure 3.4-2 Coefficient of Bearing Capacity : Nc

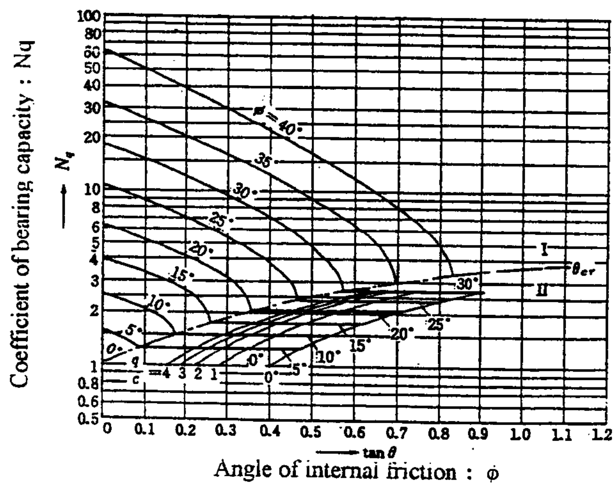


Figure 3.4-3 Coefficient of Bearing Capacity : Nq

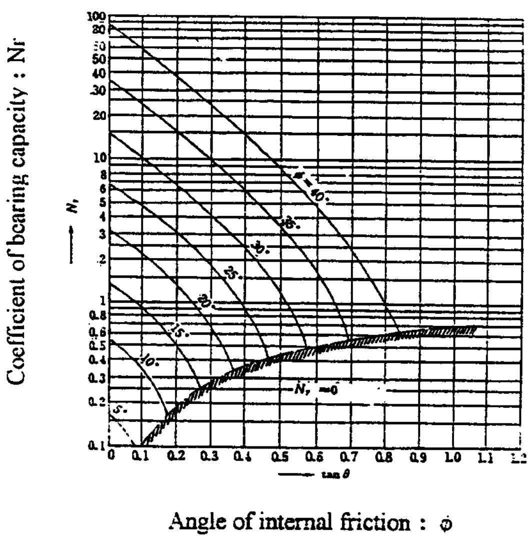


Figure 3.4-4 Coefficient of Bearing Capacity : Nr

The calculated long term allowable bearing capacities of the suction sump, pumping station and maintenance space are as following table.

All structures have enough allowable bearing capacities against their load. There are thin silty clay layers with 1 ~ 3 m thickness around EL. - 10.00 m. These silty layers are gray or brown color and calcareous soil. The character of this soil is to harden by calcareous soil. In the results of standard penetration test, N-values are 50 or more. These silty clay layers can be judged good safe bearing layer. As these character of soil, these silty clay layer can be judged that dangerous consolidation settlement does not occur by their loads. Therefor, spread foundation is adopted for the suction sump, pumping station and maintenance space.

Table 3.4-3 Bearing Capacities of No.7 Pumping Station

Name of Structure	B (m)	L (m)	α	β	N	ϕ (degree)	C (tf/m^2)	tan θ	Nc	Nr	Nq	γ_1 (tf/m^3)	γ_2 (tf/m^3)	D_f (m)	qa (tf/m^2)		Q (tf/m^2)
Suction Sump (sand)	20.00	35.50	1.17	0.44	50	39	0.00	0.00	74.00	86.00	64.00	1.00	1.80	0.00	254.37	>	25.00
Suction Sump (clay)	20.00	35.50	1.17	0.44	50	10	30.00	0.00	8.00	0.55	2.50	1.00	1.80	0.00	95.15	>	25.00
Pumping Station (sand)	21.50	52.50	1.12	0.46	50	39	0.00	0.00	74.00	86.00	64.00	1.00	1.00	9.60	487.73	>	40.00
Pumping Station (clay)	21.50	52.50	1.12	0.46	50	10	30.00	0.00	8.00	0.55	2.50	1.00	1.00	9.60	99.64	>	40.00
Maintenance Space (sand)	16.00	21.50	1.22	0.43	50	39	0.00	0.00	74.00	86.00	64.00	1.00	1.00	9.60	400.00	>	50.00
Maintenance Space (clay)	16.00	21.50	1.22	0.43	50	10	30.00	0.00	8.00	0.55	2.50	1.00	1.00	9.60	107.11	>	50.00

3.4.2 Foundation Design of Other Structures

This section will describe the foundation treatments of the Box Culvert, Bridge, Spillway, Sand Settling Basin, Discharge Tank, Surge Tank and Air Valve Chamber based on the results of the bore hole drilling survey.

(1) Examination of Bearing Capacity

(a) Examination Conditions of Other Structures

The examination conditions of other structures are as following table.

Angle of internal friction can be estimated by Dunhum equation: $\phi = \sqrt{12N} + 15$.

Table 3.4-4 Examination Conditions of Pumping Station (1/2)

Description	Symbol	Box Culvert	Bridge	Spillway	Settling Basin (O/M Bridge)
Location ;	KM	96.500	122.000	106.211	108.670
Width ;	B	16.80 m	6.00 m	5.05 m	10.00 m
Length	L	—	15.60 m	13.00 m	23.50 m
Elevation	EL.	EL. 8.80 m	EL. 86.60 m	EL. 6.60 m	EL. 6.30 m
N-value	N_1	12	22	16	31
Internal friction angle	ϕ	27°	31°	29°	34°
Depth from G.S.	D_f	5.70 m	1.70 m	1.00 m	1.10 m
Unit weight of below	γ_1	1.80 tf/m^3	1.80 tf/m^3	1.80 tf/m^3	1.80 tf/m^3
Unit weight of above	γ_2	1.80 tf/m^3	1.80 tf/m^3	1.80 tf/m^3	1.80 tf/m^3
Vertical load	V	17.86 tf/m^2	328.74 tf	75.82 tf/m	15.71 tf/m^2
Horizontal load	H	0 tf/m^2	57.78 tf	21.80 tf/m	0 tf/m^2
Tanθ = V / H	θ	0.00	0.18	0.29	0.00
Maximum Reaction	Q	17.86 tf/m^2	28.76 tf/m^2	18.63 tf/m^2	15.71 tf/m^2
Shape of foundation		Continuous	Square	Square	Square

Table 3.4-4 Examination Conditions of Pumping Station (2/2)

Description	Symbol	Settling Basin (Wall – 4)	Discharge Tank	Surge Tank	Air Valve Chamber
Location ;	KM	108.820	118.370	110.300	115.800
Width ;	B	5.80 m	20.00 m	6.00 m	3.10 m
Length	L	–	24.90 m	29.00 m	12.40 m
Elevation	EL.	EL. 2.00 m	EL. 87.60 m	EL. 47.90 m	EL. 55.55 m
N-value	N_1	50	21	50	32
Internal friction angle	ϕ	39°	31°	39°	35°
Depth from G.S.	D_f	0.90 m	0.60 m	4.30 m	4.10 m
Unit weight of below	γ_1	1.00 tf/m³	1.80 tf/m³	1.80 tf/m³	1.80 tf/m³
Unit weight of above	γ_2	1.80 tf/m³	1.80 tf/m³	1.80 tf/m³	1.80 tf/m³
Vertical load	V	91.79 tf/m	8.94 tf/m²	12.50 tf/m²	10.50 tf/m²
Horizontal load	H	34.08 tf/m	0 tf/m²	0 tf/m²	0 tf/m²
$\tan\theta$ = V / H	θ	0.37	0.00	0.00	0.00
Maximum Reaction	Q	26.67 tf/m²	8.94 tf/m²	12.50 tf/m²	10.50 tf/m²
Shape of foundation		Square	Square	Square	Square

(b) Examination of Bearing Capacity

The bearing capacity of the foundation will be estimated by the modified Terzaghi equation.

- Long term allowable bearing capacity

$$q_a = \frac{1}{3} \cdot \left(\alpha \cdot C \cdot N_c + \beta \cdot \gamma_1 \cdot B \cdot N_\gamma + \gamma_2 \cdot D_f \cdot N_q\right) \ (\mathrm{tf/m^2})$$

The calculated long term allowable bearing capacities of the box culvert, bridges, spillway, sand settling basin, discharge tank, surge tank and air valve chamber are as Table 3.4-5.

(2) Foundation Treatment of Other Structures

All structures have enough allowable bearing capacities against their load. Therefor, spread foundation is adopted for the. box culvert, bridges, spillway, sand settling basin, discharge tank, surge tank and air valve chamber

Table 3.4-5 Bearing Capacities of Other Structures

Name of Structure	B (m)	L (m)	α	β	N	ϕ (degree)	C (tf/m^2)	tan θ	Nc	Nr	Nq	γ_1 (tf/m^3)	γ_2 (tf/m^3)	D_f (m)	q_a (tf/m^2)		Q (tf/m^2)
Box Culvert	16.80	—	1.00	0.50	12	27	0.00	0.00	24.00	9.00	13.00	1.80	1.80	5.70	89.82	>	17.86
Bridge	6.00	15.60	1.12	0.46	22	31	0.00	0.18	23.00	9.00	14.00	1.80	1.80	1.70	29.23	>	28.76
Spillway	5.05	13.00	1.12	0.46	16	29	0.00	0.29	17.00	5.00	10.00	1.80	1.80	2.20	20.19	>	18.63
Settling Basin (O/M Bridge)	10.00	23.50	1.13	0.46	31	34	0.00	0.00	45.00	33.00	31.00	1.80	1.80	1.10	111.03	>	15.71
Settling Basin (Wall-4)	5.80	—	1.00	0.50	50	39	0.00	0.37	30.00	18.00	25.00	1.00	1.80	0.90	30.90	>	26.67
Discharge Tank	20.00	24.90	1.24	0.42	21	31	0.00	0.00	32.00	16.00	19.00	1.80	1.80	0.60	87.42	>	8.94
Surge Tank	6.00	29.00	1.06	0.48	50	39	0.00	0.00	74.00	86.00	64.00	1.80	1.80	4.30	313.51	>	12.50
Air Valve Chamber	3.10	12.40	1.08	0.48	32	35	0.00	0.00	46.00	34.00	32.00	1.80	1.80	4.10	108.76	>	10.50

3.5 Pavement Design of Access Road and OM Road

3.5.1 Design Conditions

- Daily traffic capacity (large size vehicles) : A-traffic (100 - 250/day)
- Design CBR : 5 %
- Minimum thickness of each paved layer
  - Surface course (Asphalt concrete) : 5.0 cm
  - Upper sub-grade (Macadam) : 10.0 cm
  - Lower sub-grade (Laterite soil) : 10.0 cm

3.5.2 Pavement thickness required (T_A)

$$T_A = \frac{3.84 \cdot N^{0.16}}{C.B.R.^{0.3}}$$

where, T_A : Thickness required (cm)

N : Number of wheels of vehicles passing for 10 years. (150,000 wheels/dir.)

C.B.R. : C.B.R. for road grade (5 %)

T_A is obtained as about 16 cm.

3.5.3 Design Pavement Thickness

$$T'_A = a_1 \cdot T_1 + a_2 \cdot T_2 + a_3 \cdot T_3$$

where, T'_A : Design pavement thickness (cm)

T_1, T_2 & T_3 : Thickness of surface course, upper sub-grade and lower sub-grade, respectively (cm)

a_1, a_2 & a_3 : Equivalent conversion coefficient of each course by 1.00, 0.35 and 0.20, respectively.

The pavement thickness of T_1, T_2 and T_3 will be 5 cm, 30 cm and 25 cm, respectively in accordance with the practice in Egypt.

T'_A = 1.00 x 5.0 + 30 x 0.35 + 25 x 0.20 = 20.5 cm > 16.0 cm OK.

3.5.4 Typical Cross Section of Access Road and OM Road

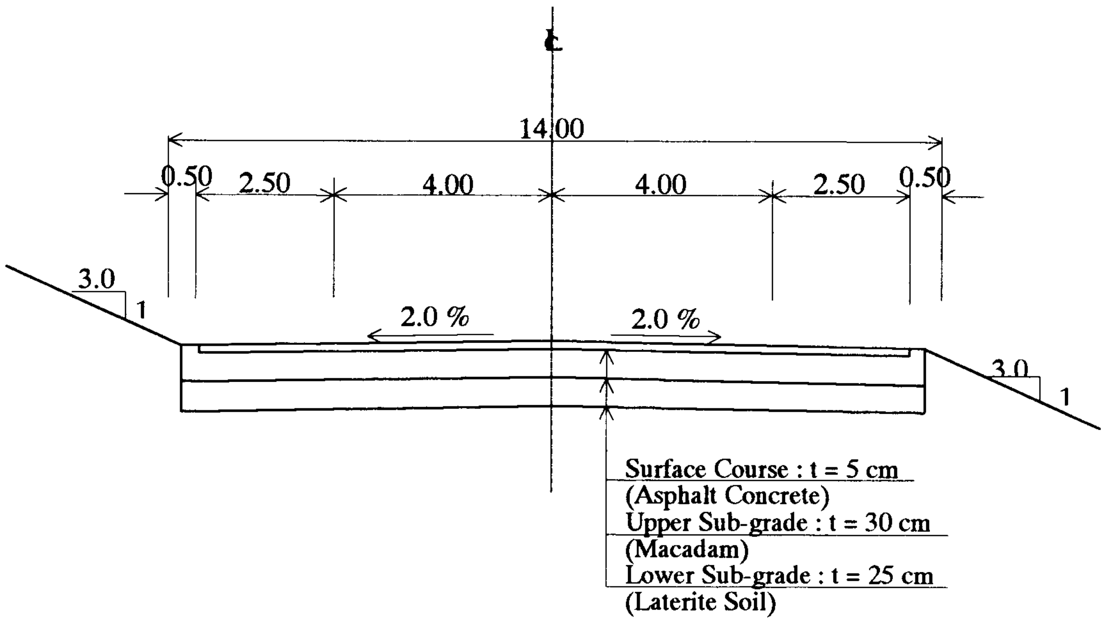


Figure 3.5-1 Standard Cross Section of Access Road

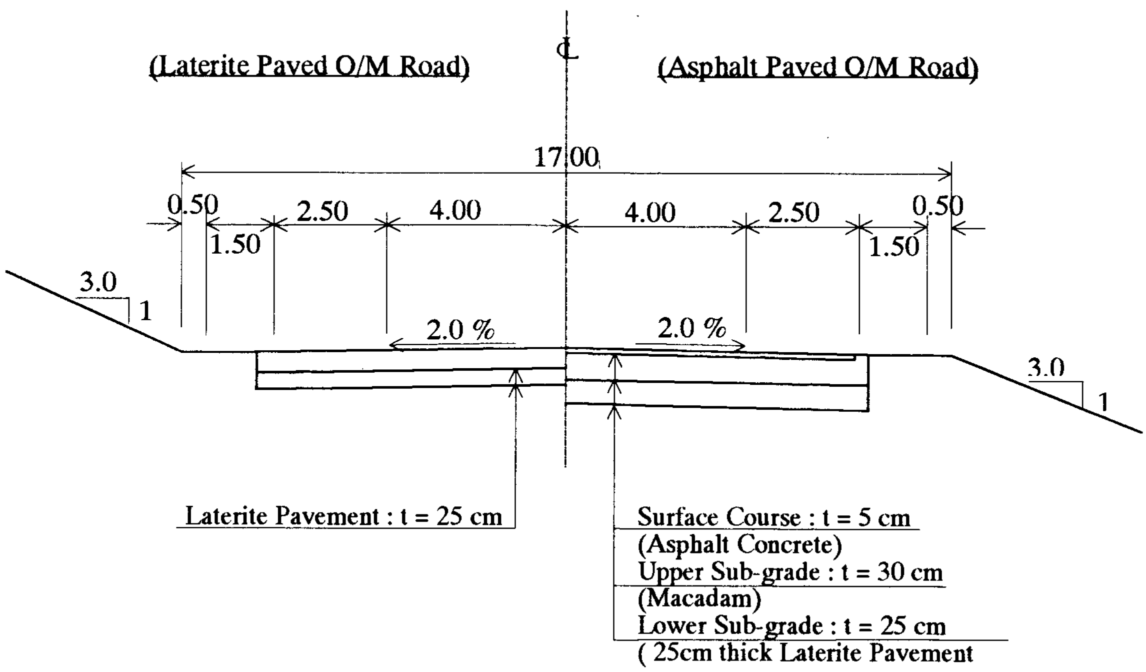


Figure 3.5-2 Standard Cross Section of Maintenance Road

CHAPTER 4 STRUCTURAL DESIGN

CHAPTER 4 STRUCTURAL DESIGN

4.1 General Description

4.1.1 Design Standards for Structure

The project facilities are designed according to the design standard given in this section. The other design standard necessary for the complete design of the project facilities can be found in the following design standards. In additions to the above, applicable Egyptian design standards shall be applied preferentially over the Japanese design standards.

- Design standards for Canal works, Pipeline, Pump facilities and Fill dam in land improvement projects, established by Agricultural Structure Improvement Bureau, Ministry of Agriculture, Forestry and Fisheries of Japan.
- Japanese government ordinance for road structures and Japanese design standard for road bridges.
- Technical Standards for Gate and Penstocks, established by Hydraulic Gate and Penstock Association.

4.1.2 General Design Conditions for Structure

(1) Allowable Stresses of Construction Materials

The allowable stresses of construction materials, that are agreed upon and decided between NSDO and the Study Team, are summarized as follows:

(a) Allowable Stress of Reinforced Concrete

Table 4.1-1 Allowable Stress of Reinforced Concrete

Allowable Stress (kgf/cm^2)		28 day Concrete Strength (kgf/cm^2)	
		225	275
Bending Compressive Stress		75	85
Shear Stress	Beams	7	8
	Slabs	9	10
Bond Stress	Round Bar	7	8
	Deformed Bar	9	10
Bearing Stress		60	70
Structures to be applied		Others	Slabs, walls, beams, columns and piers of main structures

Note; The modular ratio (modulus of elasticity of steel/modulus of elasticity of concrete) of 10 will be used for the design of the project facilities.

Source: Reinforced concrete design handbook established 1984 by Dr. Shaker El Behairy.

(b) Allowable Stress of Plain Concrete

Table 4.1-2 Allowable Stress of Plain Concrete

Allowable Stress (kgf/cm^2)	28 Day Concrete Strength (kgf/cm^2)	
	160	180
Bending Compressive Stress	60	65
Bending Tensile stress	-	-
Bearing Stress	45	50

Source: Reinforced concrete design handbook established 1984 by Dr. Shaker El Behairy.

(c) Allowable Tensile Stress of Steel

- Deformed bar Slabs (Steel 52) σ_{sa} = 2,000 kgf/cm^2
  Beams (Steel 52) σ_{sa} = 1,800 kgf/cm^2
  All (Steel 37) σ_{sa} = 1,400 kgf/cm^2
- Round bar All (Steel 37) σ_{sa} = 1,400 kgf/cm^2
- Structural steel (SS400) σ_{sa} = 1,200 kgf/cm^2
- Structural steel for Pipe (STW400) σ_{sa} = 1,400 kgf/cm^2
- Steel sheet pile (SY295) σ_{sa} = 1,400 kgf/cm^2

(2) Loadings

(a) Dead Loads

The dead-load unit weights are as follows;

- Reinforced concrete γ_c = 2.45 tf/m^3
- Plain concrete γ_c = 2.30 tf/m^3
- Water γ_w = 1.0 tf/m^3
- Dry earth γ_e = 1.6 tf/m^3
- Wet earth γ_e = 1.8 tf/m^3
- Saturated earth γ_e = 2.0 tf/m^3
- Steel γ_s = 7.85 tf/m^3

Source: MOAFJ's standard (Canal Works)

(b) Live Loads

(i) Crowd Loads

1) Walk-way

- Live load on walk-way : 300 kgf/m^2

2) Pumping Station

- Live load on floor of pumping station : 500 kgf/m

(ii) Live Loads on Bridge

Following the Egyptian code of practice 1994:

- Main lane (3.5m width) : consider 70t vehicle + 500 kgf/m^2 as uniform distributed load proceeding and exceeding the vehicle.
- Secondary adjacent lane (3m width) : consider 30t vehicle + 300 kgf/m^2 as uniform distributed load proceeding and exceeding the vehicle.
- A uniform distributed load : 300 kgf/m^2 covering the rest of the bridge area.
- Live load on side-walk : 300 kgf/m^2
- Impact coefficient : $I = 0.4 - 0.008\ L_1$
- Braking force : 25 % from main lane load without impact.

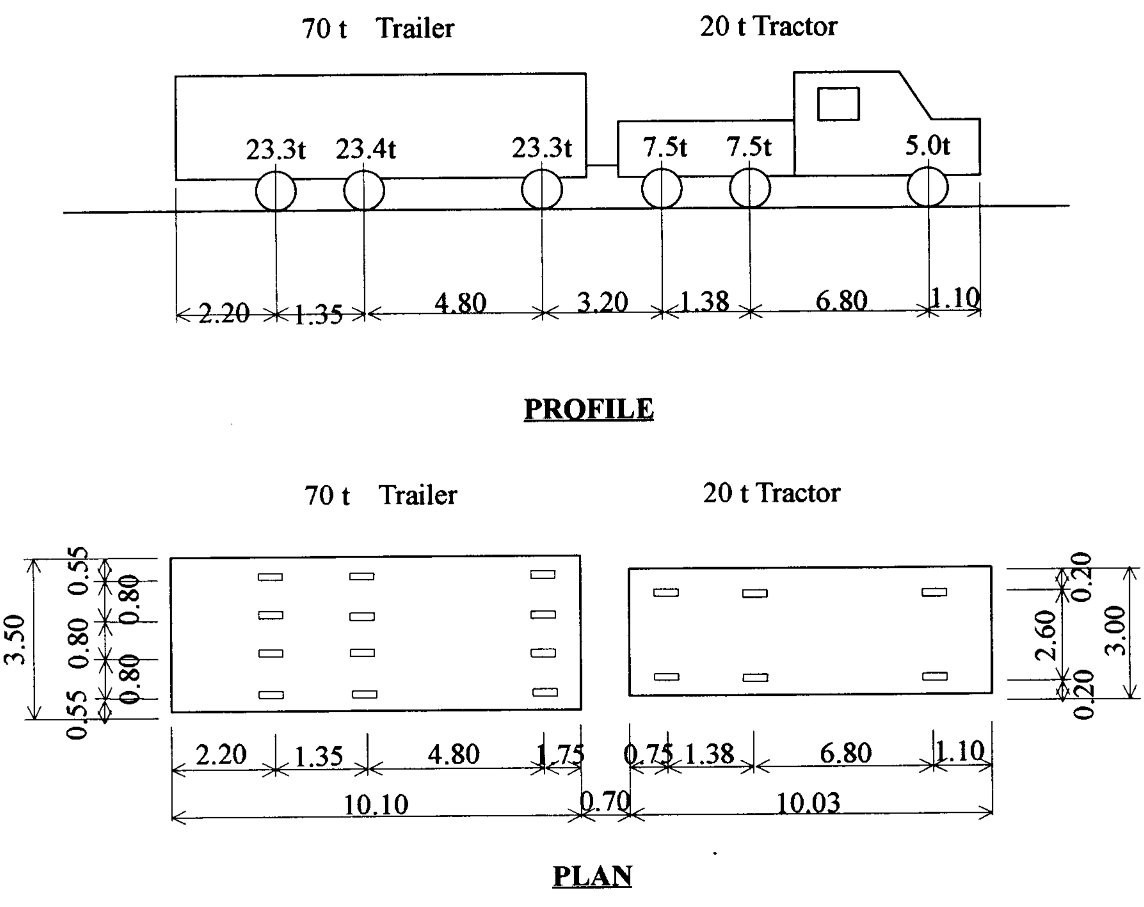


Figure 4.1-1 Truck Type of Wheel Load

(c) Seismic Loads

In the design of reinforced concrete COLUMNS according to new Egyptian code concept 1990 established by structural design engineer Khalil Ibrahim Waked, the seismic loads are as follows.

$$K_h = 0.4\ K \cdot C \cdot I$$

where, K_h : Seismic horizontal acceleration for design.

K : 1.0 for structural system contains both ductile space frames and shear walls, both to resist the effect of horizontal force.

C : Factor calculated from following equation.

$C = 1/15 \cdot \sqrt{T} = 1/15 \cdot \sqrt{0.285} = 0.036$

T : Fundamental period of vibration of the structure under consideration in seconds.

I : Degree of importance for the structure.

1.5 for structure with special importance

$K_h = 0.4 \times 1.0 \times 0.036 \times 1.5 = 0.02$

Therefore, the seismic loads are not considered for the design of the project facilities.

(3) Reinforcement Bars

(a) Cover over Reinforcement Bar

The depth of the concrete cover reinforcement bar shall be determined in consideration of the importance of the structure, site conditions and other factor.

- The depth of the concrete cover shall be defined as the minimum distance from the surface of the reinforcement bar to the surface of the concrete.
- Concrete covering shall be more than the bar diameter or 1.25 times the maximum size of the coarse aggregate used.
- Concrete cover shall generally be not less than the relevant value given in Table 4.2-3.

Table 4.1-3 Minimum Concrete Covering (Unit : cm)

Conditions	Slab	Beam	Pillar
General Environment	2.5	3.0	3.5
Corrosion Environment	**4.0**	**5.0**	**6.0**
Severe Corrosion Environment	5.0	6.0	7.0

The concrete covering of slab, beam and pillar shall be 4.0 cm, 5.0 cm and 6.0 cm, respectively.

(b) Interval of Reinforcement Bars

Reinforcement bars shall be arranged at intervals varying according to the type and size of members, maximum size of aggregate, dimensions of steelworks and other factors. In short, bar arrangement intervals shall be determined taking into consideration the assembly of reinforcement, placing of concrete and strength between the reinforcement and concrete.

- Maximum net horizontal intervals of reinforcement bars arranged in a beam shall be not less than 2.0 cm, and at the same time not less than three fourth of the maximum size of coarse aggregate used and not less than the bar diameter. Vertical net intervals of main reinforcement bars arranged in two tiers shall be not less than 2.0 cm and at the same time not less than the bar diameter.

- Center-to-center intervals of main reinforcement bars arranged in a slab shall be not more than twice as thick as the slab and at the same time not more than 20 cm in a section in which the maximum bending moment occurs. In other sections, intervals shall be not more than three times as thick as the slab and at the same time not more than 25 cm.

4.2 Open Canal and Its Appurtenant Structures

(1) Open Canal Section

The soils in the site are classified into fine and/or medium sand that are lose and have a high permeability ranging 2.9 x 10^{-2} to 5.5 x 10^{-2} cm/sec. The concrete lined canal, that has been constructed upstream from KM 86.5 of the beginning point of the conveyance canal, is planned for preventing excessive leakage from the conveyance canal. The concrete lining, which is placed on PE sheet of 1.5 mm thick laid on 0.10 m thick mortar lining, is 0.25 m thick for side walls and 0.20 m for bottom slab as shown on Figure 4.2-1. (For selection of type of conveyance structure over depressions in the route of the box culvert conduit, see Appendix C.4.2-1.)

(2) Road Bridge

The road bridges of cast-in place concrete T beam spans has a bridge width of 14 m composed of side walks (2m x 2) and a roadway of 10m wide as shown on Figure 4.2-2 and their location, lengths and numbers of beam spans, etc. are shown in Table 4.2-1.

Table 4.2-1 Location and Span Length of Road Bridge

Intersectional Angle	90°	58° 03' 45"	112° 57' 20"
Nos. of Bridges	5	1	1
Location	1. Spillway (10.48m x 5 spans) 2. Emergency Spillway (10.48m x 3 spans) 3. KM 122.80 (10.48m x 3 spans) 4. KM 127.90 (10.48m x 3 spans) 5. KM 132.40 (10.48m x 3 spans)	1. KM 90.566 (11.47m x 3 spans)	1. KM 105.70888 (12.434m x 3 spans)

The structural analysis was carried out following the Egyptian code of practice, 1994 as shown in the design standard and the results are shown in Appendix C.4.2-2.

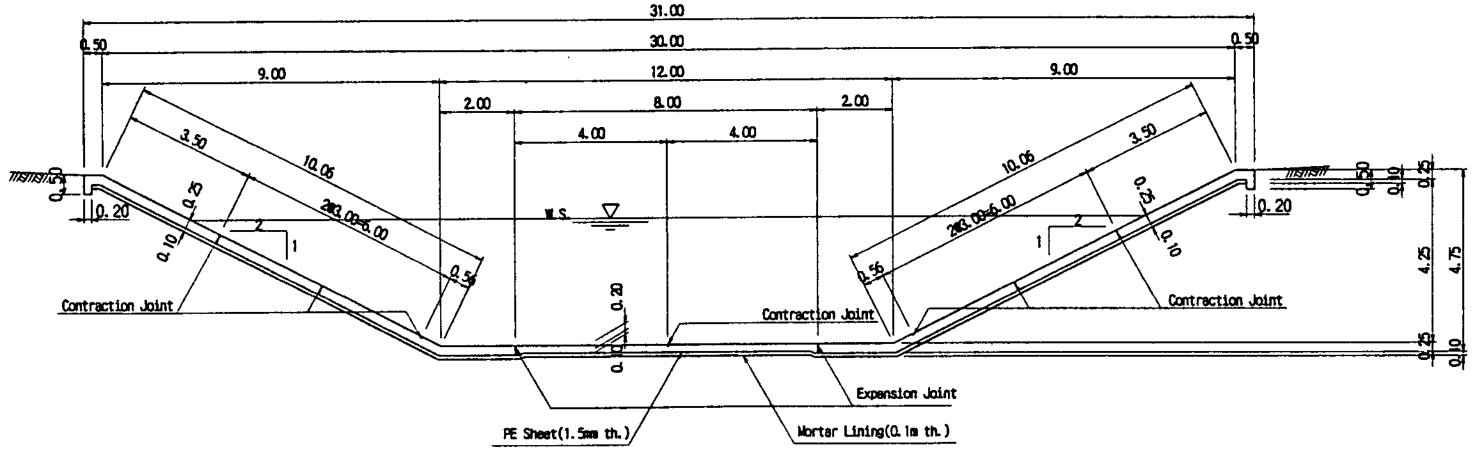
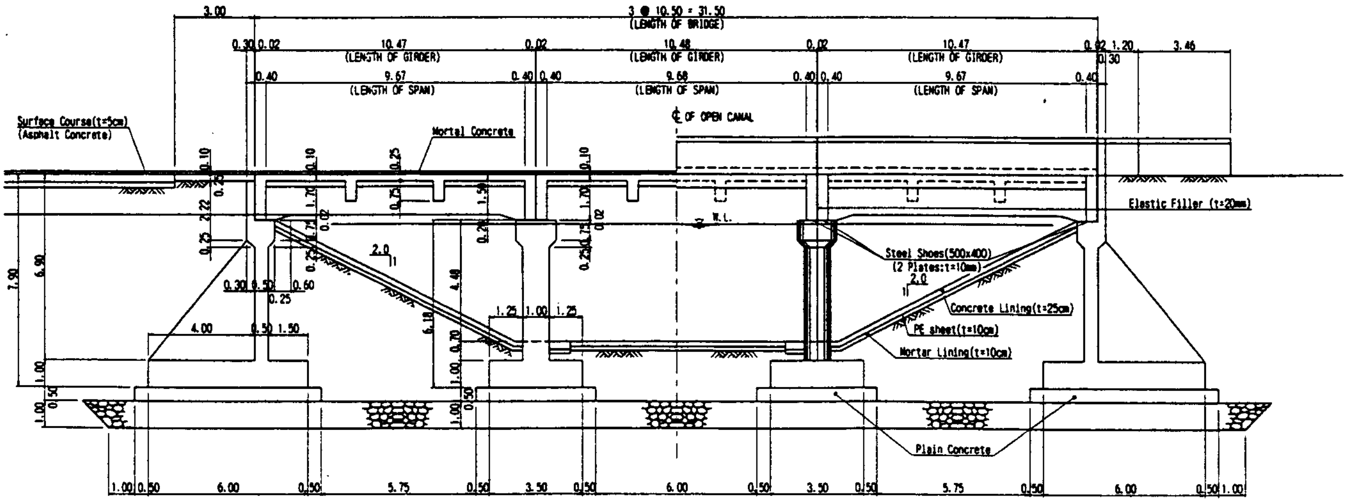
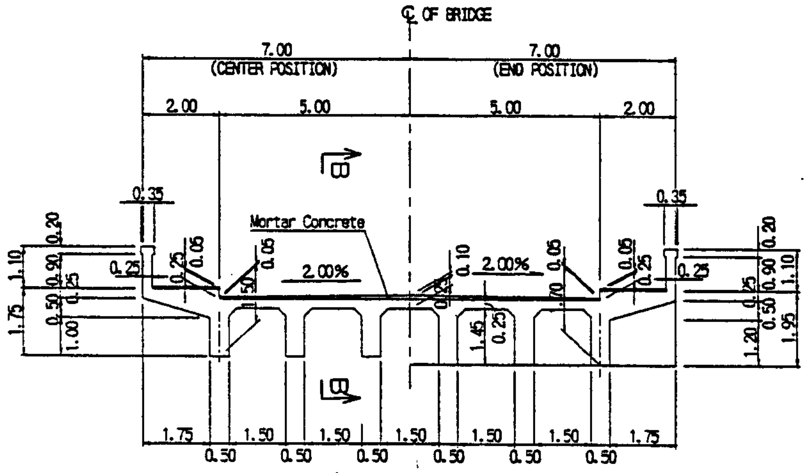


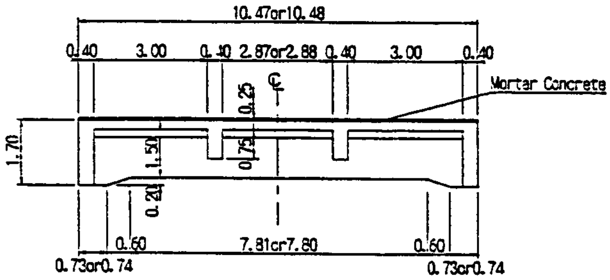
Figure 4.2-1 Typical Section of Concrete Lined Canal



ELEVATION



SECTION A-A



SECTION B-B

Figure 4.2-2 Elevation and Section of Road Bridge

4.3 Box Culvert and Its Appurtenant Structures

(1) 4-Cell Box Culvert

The 4-cell box culvert section (3.7 m x 3.7 m x 4) has been planned in the sections between KM 94.3 and KM 101.8 where drift sand dunes are prevailing. The structural analysis of 4-cell box culvert covered with drifting sand of 5 m deep is carried out as shown in Appendix C.4.3-1 and necessary reinforcements are shown on Figure 4.3-1.

(2) Openings

The 8 openings are designed on the route of box culvert conduit for easy maintenance and easy supply water necessary to irrigate trees planted along O/M roads. The structural analysis of the openings are carried out as reinforced concrete flat slab supported by walls and the computation of openings are shown in Appendix C.4.3-2.

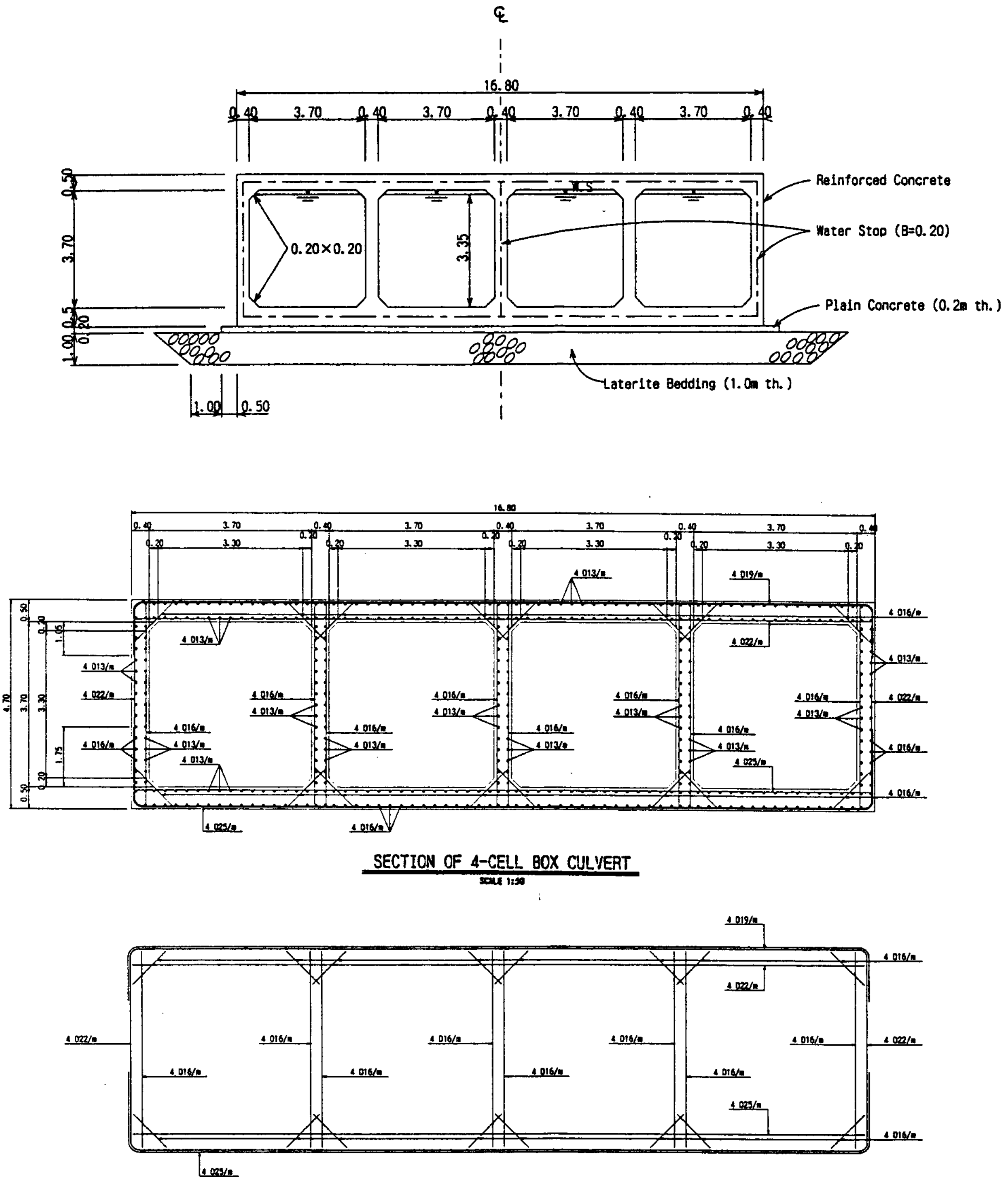


Figure 4.3-1 Typical Section of 4-Cell Box Culvert