CHAPTER III DESIGN CONDITIONS AND STANDARDS

CHAPTER 3 DESIGGN CONDITIONS AND STANDARDS

3.1 Topography of Canal Route and Appurtenance Structures

3.1.1 Topography of Canal Routes

(1) Location

The Study area is located in El Sir and El Kawareer Zone within North Sinai governorate, 400 km northeast of Cairo. It is some 50 km west from El Arish, the largest city in Sinai Peninsula, to the middle portion of the proposed conveyance canal where No.7 pumping station, one of the major facilities in the project is to be planned. The Study area lies roughly between latitude 30° 55' N and 31° 03'N, and longitude 33° 07'E and 33° 32'E.

(2) Topography

The beginning point (BP) of the proposed conveyance canal at latitude 30° 59'42"N and longitude 33° 07'27"E, KM 86.5, which corresponds to the end point of the preceding upstream canal under construction now, is covered with sand dunes. The drifting sand dunes undulating with a height of about 20 m prevails in the area south of latitude 31° N. Therefor, the alignment of the canal is selected to the north runs about 7.8 km through plains and stable sand dunes covered with vegetation. The ground surface on No.1 Open Canal route has an elevation between EL. 5 m and EL. 24 m.

The alignment of the canal is placed in parallel with the national highway from KM 94.3 to KM 101.8 where sand dunes predominate and are presumed to be shifting and there might be stable sand dunes in some part among the shifting sand dunes. The canal alignment is selected to east-southeast ward from KM 101.8 to KM 108.9 where stable sand dunes with some bush vegetation are observed.

The No.7 pumping station is planned near the section of KM 108.9 which is an end of the small valley surrounded by rather old stable sand dunes with vegetation. The ground elevation is around 15m amsl and gradually goes up to the downstream of southeastern direction.

Pipelines with length of 9.6 km including discharge tank of 0.2 km long are proposed in the area where very undulating topography with combination of active drifting sand dunes and stable sand dunes is predominant.

No.3 Open Canal with 13.9 km long is to be connected to the end of the pipeline and extend to the entrance point to the beneficiary area which is the intersection point of elevation 90 m contour line and the northwestern boundary of the beneficiary area. The topographical condition in these area is similar to the area of canal section located between the beginning point and KM 94.3.

3.1.2 Topography of No.7 Pumping Station

The No.7 pumping station including sand settling basin is planned at the section between KM 108.48 and KM 108.985 which is an end of the small valley surrounded by hill in northeast direction and rather old stable sand dunes with vegetation in south direction. The original ground elevation is assumed to be at around EL5～30 m and gradually rises up to the downstream toward the southeastern direction.

Pipelines with length of 9.4 km are proposed to set up in the area where very undulating condition with combination of predominant active drifting sand dunes and stable sand dunes is predominant.

3.1.3 Topography of Access Road

The access roads connecting from national road to maintenance road of conveyance canal are planned three routes. No.1 and No.2 access roads are designed to connect at near beginning point and end point of box culvert for the purpose of both access of the construction works and post project maintenance. Length and distance of three roads are summarized as follows.

Road No.	Length	Location of end point	Distance of conveyance canal
No.1	1.05 km	BP of culvert	KM 94.767
No.2	2.84 km	EP of culvert	KM 102.304
No.3	5.10 km	N0.7 pumping station	KM 107.926

The routes of road are located semi-desert area with topographically rolling plain. Ground surface covered with drifting sand is found at anywhere. Access roads pass through small hill, sand dune and low depression area. Alignment of No.1 and No.3 road would be used exiting roads.

No.1 access road route with elevation of EL.1.5 to 12.5 would be used exiting village road paved by gravel which have slightly gentle ascent or descent all the way from national road to proposed conveyance canal. However topographical condition of No.2 road with elevation of EL.1.1 to EL.25.9 is undulated. Route of new road is crossed major five (5) small sand dune. No.3 road route is selected at the trace of existing road which is paved by asphalt concrete at 500 meters only from starting point (national road) and after branched off from existing road, new road route may pass through up and down area of with sand dune.
The elevations of national road surface, existing ground level of the routes and proposed O/M road are as follows.

Road No.	National road surface	Ground level of route		O/M road surface
		Minimum	Maximum	Proposed
No.1	EL. 6.00 m	EL. 1.5 m	EL. 12.5 m	EL. 14.56 m
No.2	EL. 7.27 m	EL. 1.1 m	EL. 25.0 m	EL. 13.41 m
No.3	EL. 0.84 m	EL. 0.8 m	EL. 21.5 m	EL. 12.96 m

3.2 Geological Conditions of the Project Site

3.2.1 General

It is of primary importance to have geological and soil information available prior to beginning the detailed design of the facilities. For this purpose, the following geological investigations for the subject facilities such as the proposed conveyance canal and pipeline, and the spillway site, No.7 pumping station and discharge tank sites, and soil laboratory tests were performed by Sunrise Engineering & Trading Co. under the supervision of JICA study team.

Borehole drilling	28 boreholes (total drilling depth 710 m)
Sounding test	80 places
Electric resistivity test	10 places
Test pit excavation	10 pits
Soil laboratory test	107 samples (physical property test)
	10 samples (dynamic property test)
	38 samples (chemical property test)
Aggregate and concrete tests	7 samples (aggregate test)
	6 mix plans (concrete test)

3.2.2 Geological Investigation and Laboratory Test

(1) Borehole Drilling

Borehole drillings were required for obtaining necessary geological information for proper design of the project. On the occasion of borehole drillings, the following in-situ tests were carried out simultaneously for confirmation of N-value, degree of permeability, including measurement of water table. In addition, soil laboratory tests were performed using drilling samples in order to confirm soil characteristics. The technical means to achieve these purposes consist of :

- Coring (sampling)
- Standard penetration test;
- Permeability test
- Soil laboratory test (by drilling sample)

The standard penetration tests, which were performed on the basis of ASTM rule, were carried out at testing intervals of 1.0 m in depth as a general rule. Furthermore, the permeability tests were performed by the unstable waterhead method, in other words, falling waterhead method, at the saturated layer, where testing section is limited under the ground water table.

Location and drilling depth of each borehole are shown in Table 3.2-1.

Table 3.2-1 Location and Drilling Depth of Borehole

No. of Boring	Location		G.H of Borehole (EL.)	Drilling Depth (m)
CC - 1	Conveyance canal	BP + 1.3 km	8.2	15.0
CC - 2	- ditto -	BP + 4.5 km	12.0	15.0
CC - 3	- ditto -	BP + 10.0 km	11.3	15.0
CC - 4	- ditto -	BP + 17.5 km	10.5	15.0
CC - 5	---	---		---
CC - 6	Pipeline	BP + 23.8 km	58.3	15.0
CC - 7	- ditto -	BP + 28.2 km	66.4	15.0
CC - 8	Conveyance canal	BP + 35.0 km	95.2	15.0
CC - 9	- ditto -	BP + 40.0 km	94.8	15.0
CC - 10	- ditto -	BP + 46.0 km	90.6	15.0
SP - 1	Spillway	Spillway	10.0	20.0
SP - 2	- ditto -	Outlet channel	11.5	20.0
SP - 3	- ditto -	Dike (left side)	(-) 0.3	20.0
SP - 4	- ditto -	Dike (right side)	2.9	20.0
SP - 5 (CC - 5)	- ditto -	Dike (center part)	(-) 0.5	15.0
PS - 1	Pumping station	Left side of C.L	17.5	40.0
PS - 2	- ditto -	Right side of C.L	9.2	40.0
PS - 3	- ditto -	Left side of C.L	25.4	40.0
PS - 4	- ditto -	Left side of C.L	16.3	40.0
PS - 5	- ditto -	Right side of C.L	10.5	40.0
PS - 6	- ditto -	Center part of C.L	12.2	40.0
PS - 7	- ditto -	Left side of C.L	18.8	40.0
PS - 8	- ditto -	Left side of C.L	17.2	40.0
PS - 9	- ditto -	Right side of C.L	16.1	40.0
PS - 10	- ditto -	Left side of C.L	23.8	40.0
DT - 1	Discharge tank	Left side of C.L	91.0	20.0
DT - 2	- ditto -	Right side of C.L	91.0	20.0
DT - 3	- ditto -	Left side of C.L	91.7	20.0
DT - 4	- ditto -	Right side of C.L	92.7	20.0
Total				710.0

Remarks : 1. G.H : ground height (elevation), C.L : center line of longitudinal survey

2. BP means beginning point of this study and distance from Suez Siphon is 86.5 km.
3. Borehole No. CC-5 was sifted to the spillway site.
4. Borehole No. CC-6 is located at the proposed surge tank No.1 (survey point TP60).
5. Borehole No. CC-7 is located at the proposed surge tank No.2 (survey point TP72).

(2) Sounding Test

Sounding tests were performed along the proposed conveyance canal route and pipeline route by the use of Dutch cone penetration test (10-tf class, tip angle 60 degrees). The basic purpose of this sounding test is to confirm cone resistance values (qc values) in the soil. In this case, target depth of each sounding was set as 10 m at least taking earth works for conveyance canal and pipeline at the construction stage into consideration.

Total quantities of performed sounding test are 80 places, which are situated at 0.5 to 1.0 km intervals. Details as for location of each testing point are as follows ;

Table 3.2-2 Location of Sounding Test

No. of Test	Location	No. of Test	Location	No. of Test	Location
1	BP + 0.0 km	31	BP + 15.0 km	61	BP + 30.0 km
2	BP + 0.5 km	32	BP + 15.5 km	62	BP + 30.5 km
3	BP + 1.0 km	33	BP + 16.0 km	63	BP + 31.0 km
4	BP + 1.5 km	34	BP + 16.5 km	64	BP + 31.5 km
5	BP + 2.0 km	35	BP + 17.0 km	65	BP + 32.0 km
6	BP + 2.5 km	36	BP + 17.5 km	66	BP + 33.0 km
7	BP + 3.0 km	37	BP + 18.0 km	67	BP + 34.0 km
8	BP + 3.5 km	38	BP + 18.5 km	68	BP + 35.0 km
9	BP + 4.0 km	39	BP + 19.0 km	69	BP + 36.0 km
10	BP + 4.5 km	40	BP + 19.5 km	70	BP + 37.0 km
11	BP + 5.0 km	41	BP + 20.0 km	71	BP + 38.0 km
12	BP + 5.5 km	42	BP + 20.5 km	72	BP + 39.0 km
13	BP + 6.0 km	43	BP + 21.0 km	73	BP + 40.0 km
14	BP + 6.5 km	44	BP + 21.5 km	74	BP + 41.0 km
15	BP + 7.0 km	45	BP + 22.0 km	75	BP + 42.0 km
16	BP + 7.5 km	46	BP + 22.5 km	76	BP + 43.0 km
17	BP + 8.0 km	47	BP + 23.0 km	77	BP + 44.0 km
18	BP + 8.5 km	48	BP + 23.5 km	78	BP + 45.0 km
19	BP + 9.0 km	49	BP + 24.0 km	79	BP + 45.5 km
20	BP + 9.5 km	50	BP + 24.5 km	80	BP + 46.0 km
21	BP + 10.0 km	51	BP + 25.0 km		
22	BP + 10.5 km	52	BP + 25.5 km		
23	BP + 11.0 km	53	BP + 26.0 km		
24	BP + 11.5 km	54	BP + 26.5 km		
25	BP + 12.0 km	55	BP + 27.0 km		
26	BP + 12.5 km	56	BP + 27.5 km		
27	BP + 13.0 km	57	BP + 28.0 km		
28	BP + 13.5 km	58	BP + 28.5 km		
29	BP + 14.0 km	59	BP + 29.0 km		
30	BP + 14.5 km	60	BP + 29.5 km		

Note : BP means beginning point of this study and distance from Suez Siphon is 86.5 km.

(3) Electric Resistivity Test

Electric resistivity tests were performed along the pipeline route, with total 10 places. On the occasion of testing, an electrode arrangement by Wenner was adopted in consideration of shallow investigation. Furthermore, the results of this testing have clarified the resistivity in the ground and can be utilized to study as to corrosion of material for pipeline.

Locations of each measuring point are shown in the following table ;

Table 3.2-3 Location of Electric Resistivity Test

Measuring Point	Location	Measuring Point	Location
ES-1	BP+22.5 km	ES-6	BP+27.0 km
ES-2	BP+23.0 km	ES-7	BP+28.0 km
ES-3	BP+24.0 km	ES-8	BP+29.0 km
ES-4	BP+25.0 km	ES-9	BP+30.0 km
ES-5	BP+26.0 km	ES-10	BP+31.0 km

Note : BP means beginning point of this study and distance from Suez Siphon is 86.5 km.

(4) Test Pitting

Test pittings were performed along the proposed conveyance canal route and pipeline route, with total 10 places. In addition, excavation depth for each test pitting was planned as 2.0 m at least in principle. The objectives of test pitting are to confirm geological condition at the shallow depth and to collect soil sample for laboratory test.

Locations of each test pitting are shown in Table 3.2-4.

Table 3.2-4 Location of Test Pitting

No. of T.P.	Location	No. of T.P.	Location
TP - 1	BP + 3.0 km	TP - 6	BP + 27.5 km
TP - 2	BP + 7.5 km	TP - 7	BP + 32.5 km
TP - 3	Spillway site	TP - 8	BP + 37.5 km
TP - 4	BP + 17.5 km	TP - 9	BP + 42.5 km
TP - 5	BP + 22.5 km	TP - 10	EP (BP46.0 km)

Note : BP means beginning point of this study and distance from Suez Siphon is 86.5 km.

(5) Soil Laboratory Test

Soil laboratory tests are performed for confirmation of soil characteristics at/around proposed facility sites. Soil samples for soil laboratory test were prepared using the drilling samples and test pit samples. In this case, test data include at least one sample for each borehole and test pit, respectively.

Test items for soil tests can be classified into three kinds, which signify physical property tests, dynamic property test and chemical test. The physical property tests and the chemical tests were used for sampled materials by drillings and test pits. On the other hand, the dynamic property tests were restricted to sampled materials by test pits. Methods for soil tests are based on the following designation of ASTM in principle.

< Physical property tests >

- Particle size distribution (Grain size analysis) test........ D422-63
- Water content test... D2216-80
- Specific gravity test.. D854-83

- Liquid limit/plastic test.. D427-83 and D4318-84

< Dynamic property tests >

- Compaction test.. Compaction energy method
- Direct shear test (CD test)
- Permeability test.. D2434-68

< Chemical property tests >

- TDS, Cl^-, SO_4^-, pH (injurious chemical compounds) and HCO_3 (water analysis)

(6) Aggregate and Concrete Tests

Around the project site, the proposed quarry and borrow area for construction materials, which signifies to use principally for the concrete as coarse and fine aggregate, are widely distributed at the west side of the Suez Canal, especially around the Suez City (for example Ataqa quarry etc.), and the El Sir area where is located on around end point of conveyance canal.

Aggregate tests are performed by the use of the above mentioned materials, which were collected at the existing concrete plants attached to the No.6 Pumping Station and the NSDO Branch Office of Bir El Abd. In addition, testing methods for aggregate tests are based on the designation of 'Concrete manual (8th edition), U.S. Department of the Interior' in principle. Testing items are as follows.

- Sieving test (Screen analysis test) : Concrete manual, designation 4 and 5
- Unit weight test : -ditto-, designation 12
- Specific gravity and water absorption test : -ditto-, designation 9 and 10
- Surface moisture test : -ditto-, designation 11
- Stability test (Soundness test) : -ditto-, designation 19
- Abrasion resistance test : -ditto-, designation 21
- Injurious chemical compounds test

Furthermore, concrete tests are performed by the use of aggregate materials of the above 2 sampling points. Three kinds of tentative mix plan, which correspond to those for canal lining concrete, reinforced concrete and building concrete respectively, were adopted for concrete testing.

Testing items for concrete tests are as follows ;

- Slump test : Concrete manual, designation 22
- Air content test : -ditto-, designation 24
- Compressive strength (at the age of 3 days, 7 days and 28 days) test : -ditto-, designation 33

3.2.3 Geological Conditions of the Project Site

(1) Conveyance Canal between B.P and No.7 Pumping Station

The results of borehole drillings (CC-1 to CC-4) revealed that geological conditions along the proposed conveyance canal route between B.P (beginning point) and No.7 pumping station are characterized by the existence of poorly graded and medium to fine sand (SP* by USCS classification) with yellow or brown colour. Furthermore, the N values by the standard penetration test show ranging from 12 to 20 blows up to about 4 to 5 m in depth. At the deeper depth from about 8 to 9 m, N values indicate 40 blows or more as a whole. It is judged that the confirmed bearing layer for conveyance canal exists underneath of about 4 to 5 m in depth. The above relationship between N values and depth obtained from the standard penetration tests is shown in Figure 3.2-1.

* SP : Poorly graded sands and gravelly sands, little or no fine

In addition, the boreholes (CC-1 to CC-4) also clarified that ground water tables are located underneath the invert level of conveyance canal. Moreover, the coefficient of permeability in saturated layers, which exist underneath the ground water table, show high permeable characteristics with ranging from $2.9*10^{-2}$ to $5.5*10^{-2}$ cm/sec ($3.8*10^{-2}$ cm/sec on an average).

The cone resistance values (qc values) by sounding tests also indicate 80 kg/cm^2 or less up to 2 to 7 m in depth as a whole. This qc value (80 kg/cm^2) corresponds to about 20 blows of N value based on the Meyerhof's formula. The results of sounding tests are indicated similar tendencies with those of standard penetration tests.

Taking the results of performed geological investigations into consideration, it is concluded that the conveyance canal (between B.P and No.7 pumping station) site has hardly serious hazard as for civil engineering works at the construction stage. In addition, the liquefaction effect by the earthquake will not occur in the Study area because of very small seismic horizontal acceleration for design of 0.02g and very dense foundation having N-value of more than 40 underneath the ground water table.

(2) Spillway

Locations of borehole drillings at the proposed spillway site are classified into the following two places, at the proposed outlet channel and along the proposed dike axis. Two boreholes (SP-1 and SP-2) are located on the former place, and three boreholes (SP-3, SP-4 and SP-5) are situated at the latter place.

The geological conditions along the outlet channel are mainly made up of poorly graded and medium to fine sand (SP by USCS) with yellow or brown colour. Moreover, the confirmed sand layer in borehole SP-1 intercalates a thin silty clay layer (CL* by USCS) of about 1.2 m in thickness.

* CL : Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays

At both abutments along the proposed dike axis, the geological conditions also consist of poorly graded and medium to fine sand (SP by USCS) with yellow or brown colour up to about 12 m to 13 m in depth. On the other hand, at the center part of dike, the geological conditions are composed of silty sand (SM* by USCS) with gray to yellow colour up to 7.3 m in depth. At the deeper depth, lean silty clay (CL by USCS) with brown to gray colour exists underneath the above sand layer. These clay layers are characterized by stiff and hard facies as a whole.

* SM : Silty sands, sand-silt mixture

N value data of SP-1 and SP-2, which is located on the outlet channel, are similar to those of conveyance canal (CC-1 to CC-4). On the other hand, N values at the proposed dike axis have a little difference in comparison with those of the outlet channel. At both abutments of the dike axis (SP-3 and SP-4), N values show 30 blows or more from the shallow depth. On the contrary, at the center part (SP-5), N values indicate 20 blows or less up to 5 m in depth, and N values show about 50 blows or more at the deeper depth from 5 m. The above relationship between N values and depth obtained from the standard penetration test is shown in Figure 3.2-2.

Taking overload conditions by dike body and thickness of that layer into consideration, it is considered that serious settlement does not result in the foundation after embankment nevertheless this point have to be further studied in future.

At the outlet channel, the ground water tables of boreholes (SP-1 and SP-2) exist at EL.3 m to EL.5 m, on the other hand, at the dike axis, those of boreholes exist in relatively shallow depth 0.3 m (EL.0 m approximately). In addition, the coefficient of permeability of sand layers in saturated layers, which exist underneath the ground water table, show high permeable characteristics with ranging from $1.9*10^{-2}$ to $3.3*10^{-2}$ cm/sec ($3.3*10^{-2}$ cm/sec on an average).

(3) No.7 Pumping Station

The results of borehole drillings (PS-1 to PS-10) clarified that geological conditions at the proposed No.7 pumping station site are mainly composed of poorly graded and medium to fine sand (SP by USCS) with yellow or brown colour. This sand layer intercalates several stiff and hard silty clay layers, which is about 1.5 to 5 m in thickness, at the deeper depth. These silty clay layers are classified as inorganic clays of high plasticity (CH by USCS) and indicate 50 or more as N value. Furthermore, the results of borehole drilling suggest that clay layers occur at the following three stratigraphic horizons.

Horizon	EL. of clay layer	Confirmed borehole No.
The 1st horizon	EL.0.4 - EL.(-)5.3	PS-3, PS-6, PS-8, PS10
The 2nd horizon	EL.(-)6.0 - EL.(-)12.9	PS-1, PS-4, PS-9
The 3rd horizon	EL.(-)12.9 - EL.(-)18.7	S-2, PS-5, PS-6, PS-7

From occurrences of different stratigraphic horizons for clay layers, it may be inferred that those were formed as the results of original lenticular deposition.

N value data show 10 to 30 blows up to about 10 m in depth and indicate ranging from 30 to 50 between about 10 and 20 m in depth, so that it is judged that the clarified bearing layer (supporting layer) exists underneath of about 10 m in depth. At the deeper depth from about 20 m, N values shows 50 blows or more. As the results of these data, it is judged that this site has not serious problem as for bearing capacity. In addition, the geological characteristic of foundation indicates that potential hazard for liquefaction of sandy soil hardly exists for the reason of showing high N values. The above relationship between N values and depth obtained from the standard penetration test is shown in Figure 3.2-3.

The ground water tables at the performed boreholes are located on the level of approximately EL.2.3 m and these water tables are in harmony with that of existing shallow well beside the site. In this case, there is a large possibility that the plateau around the El Sir area serves as an important recharge area for the ground water around the site. Furthermore, it might be stated that interbedded stiff clay layer in the sand layer plays a part of the important role as retaining layer for ground water. In addition, the coefficient of permeability of sand layers in saturated layers, which exist underneath the ground water table, show high permeable characteristics with ranging from $1.6*10^{-2}$ to $4.5*10^{-2}$ cm/sec ($3.1*10^{-2}$ cm/sec on an average).

Taking the above ground water condition at the site into consideration, much attention should be paid to drain out ground water on the occasion of excavation works under the ground water level.

(4) Pipeline

Along the proposed pipeline route, total 2 boreholes and sounding tests, including 10 sites of electric resistivity test, as geological investigations were carried out. The results of borehole drillings (CC-6 and CC-7) clarified that geological conditions along the proposed pipeline route are also characterized by the existence of poorly graded and medium to fine sand (SP by USCS classification) with yellow or brown colour.

Furthermore, N values by the standard penetration test show 30 blows or more at the deeper than 2 m in depth, and at the deeper depth from 4 m, N values indicate 40 blows or more as a whole. The above relationship between N values and depth obtained from the standard penetration tests is shown in Figure 3.2-1. In addition, the ground water tables are not recognized up to the bottom of boreholes (CC-6 and CC-7).

The cone resistance values (qc values) by sounding tests also indicate 80 kg/cm^2 or less, which correspond to about 20 blows of N value based on the Meyerhof's formula, up to 2 to 6 m in depth as a whole. Furthermore, the electric resistivity test revealed that the resistivity conditions in the ground range from 1,100 to 4,700 ohm-cm (in the case of pole distance 3.0 m), and show about 3,300 ohm-m on an average. The histogram of confirmed resistivity values is shown in the following figure.

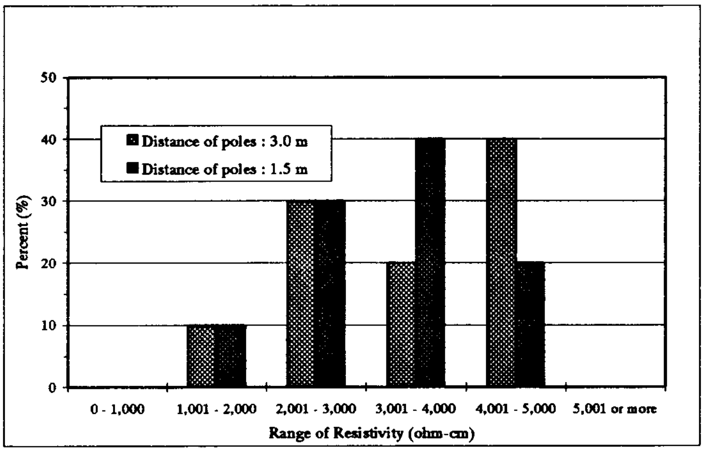


Figure 3.2-1 Histogram of Resistivity Data at the Pipeline Route

Taking the results of performed geological investigations into consideration, it is concluded that the pipeline route has hardly serious problem as for the bearing capacity at the construction stage. In addition, taking conditions of somewhat low resistivity in the ground into consideration, in order to prevent the corrosion of pipe material, proper precaution or measures should be carefully studied.

(5)Discharge Tank

The results of borehole drillings (DT-1 to DT-4) revealed that geological conditions at the proposed discharge tank site are mainly made up of fine sand (SP by USCS classification) and silty sand (SM by USCS) with yellow or brown colour. The particle sizes of these soils are characterized by poorly graded shape on the particle-size distribution curve.

At the DT-1 and DT-2, the N values by the standard penetration test show 30 blows or more in relatively shallow depth (about 2 to 5 m). On the other hand, at the DT-3 and DT-4, N values range from 20 to 30 blows up to about 5 to 7 m in depth. Furthermore, at the deeper depth from about 8 to 9 m, N values indicate 40 blows or more as a whole. These relationship between N values and depth obtained from the standard penetration tests is shown in Figure 3.2-4. In addition, ground water tables are not recognized up to the bottom of boreholes (DT-1 to DT-4).

As the results of the above geological investigations, it can be stated that the proposed discharge tank site has hardly serious hazard as for civil engineering works in the construction stage.

(6) Conveyance Canal between Discharge Tank and E.P

Along the proposed conveyance canal route between discharge tank and E.P (end point), total 3 boreholes and sounding tests as geological investigations were carried out. The results of borehole drillings (CC-8 to CC-10) clarified that geological conditions along route are also characterized by the existence of poorly graded and medium to fine sand (SP by USCS classification) with yellow or brown colour, except for around E.P. The results of borehole CC-10, which is located around E.P, indicate that the existence of predominant silty gravel layer (GM by USCS classification) up to about 8.4 m in depth. Taking the topographical condition as flat plain area around E.P into consideration, there is a large possibility that this gravel layer corresponds to Wadi sediments as the Pleistocene formation.

GM : Silty gravels, gravel-sand-silt mixture

The N values by the standard penetration test of CC-8 and CC-9 indicate ranging from 20 blows or more in relatively shallow depth (2 and 4 m) and show 40 blows or more at the deeper depth to 8 m. On the other hand, N values of CC-10 show 50 blows or more at the deeper depth from 2 m resulting in the existence of thick gravel layer. The above relationship between N values and depth obtained from the standard penetration tests is shown in Figure 3.2-1.

In addition, the ground water tables are not recognized up to the bottom of boreholes (CC-8 to CC-10).

Moreover, the cone resistance values (qc values) by sounding tests also indicate 80 kg/cm^2 or less, which correspond to about 20 blows of N value based on the Meyerhof's formula, up to 2 to 5 m in depth as a whole. In this case, the results of sounding tests are indicated similar tendencies with those of standard penetration tests.

Taking the results of performed geological investigations into consideration, it is concluded that the conveyance canal (between discharge tank and E.P) site has hardly serious problem as for civil engineering works at the construction stage.

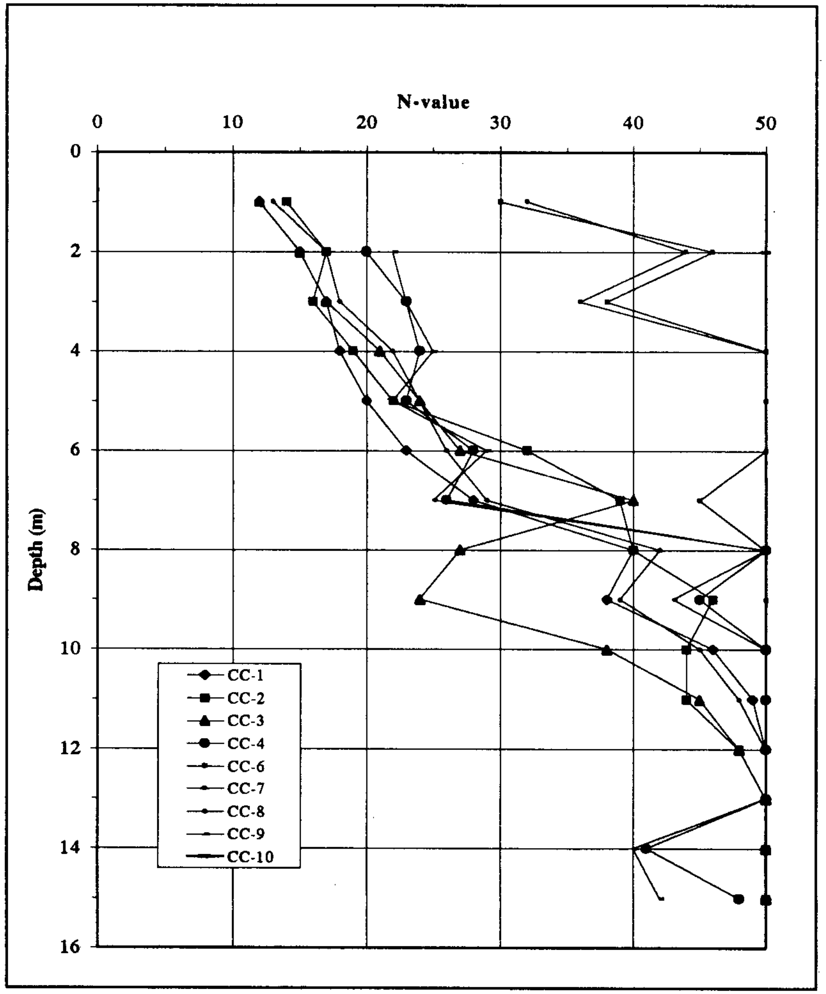


Figure 3.2-2 Relationship between N-value and Depth (Conveyance Canal)

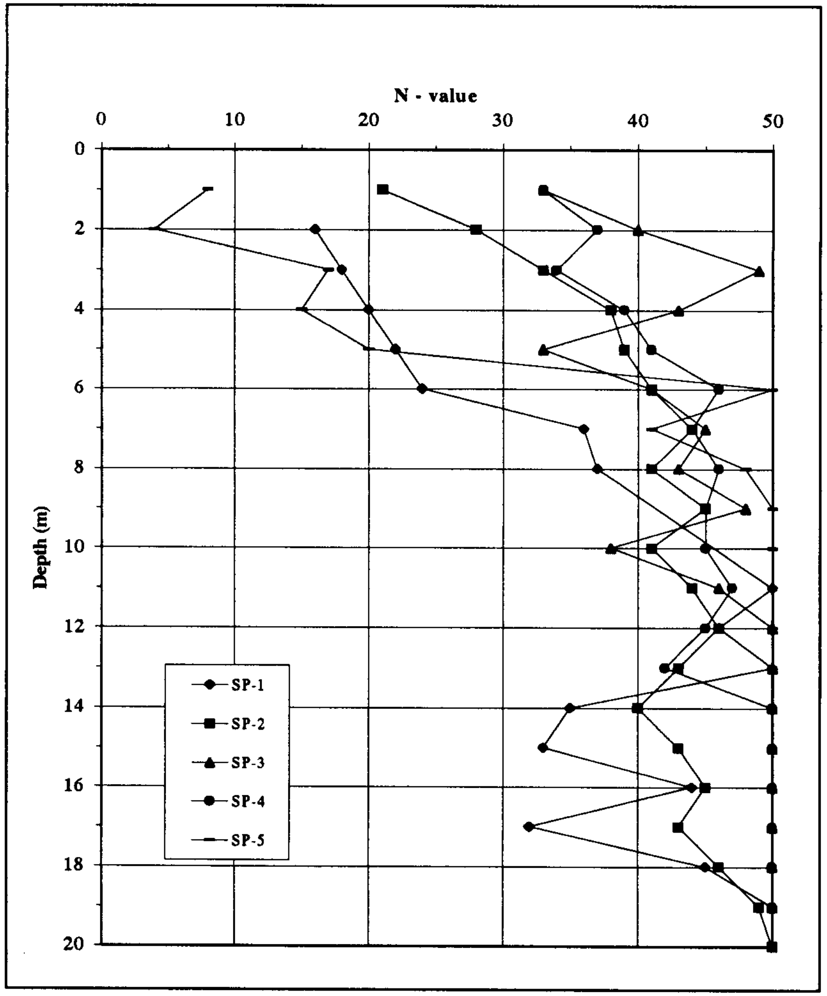


Figure 3.2-3 Relationship between N-value and Depht (Spillway)

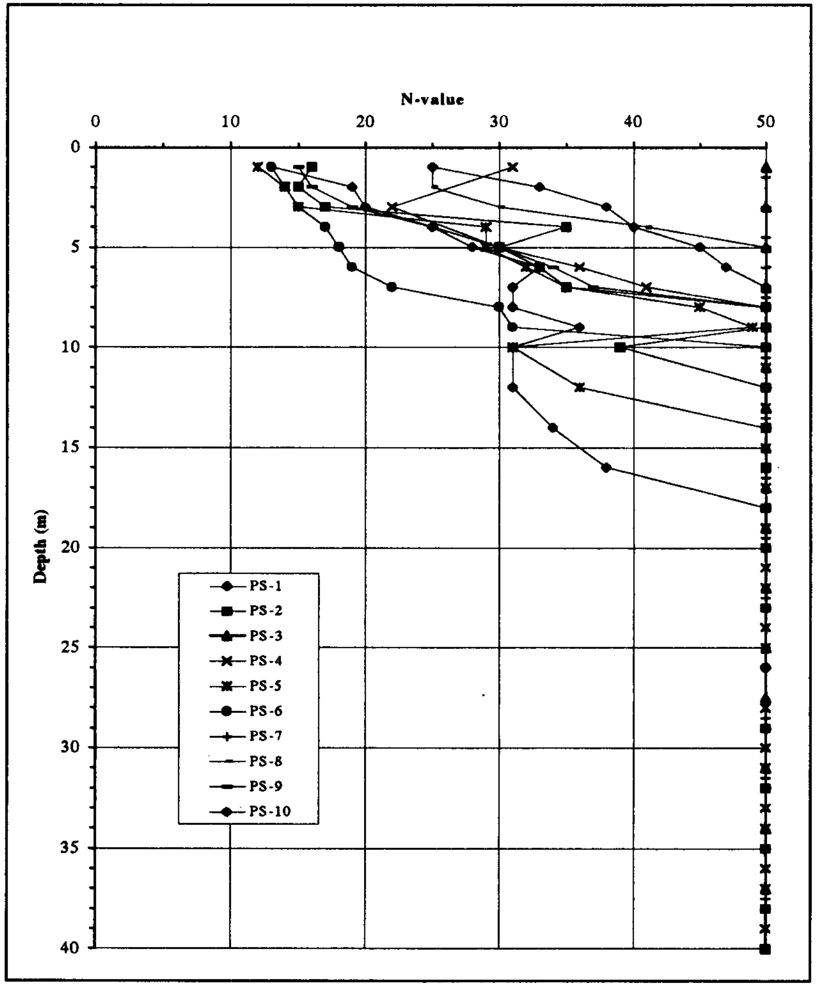


Figure 3.2-4 Relationship between N-value and Depth (No. 7 Pumping Station)

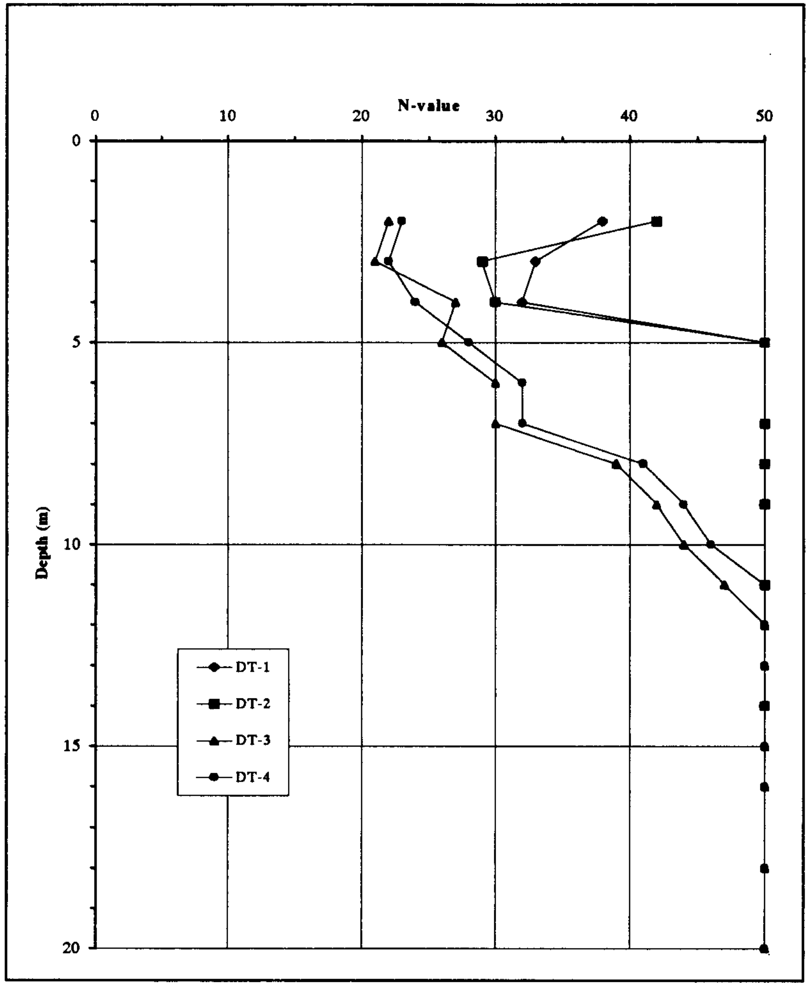


Figure 3.2-5 Relationship between N-value and Depth (Discharge)

(7) Soil Properties at the Project Site

The confirmed soil properties by the soil laboratory test are summarized as follows.

(a) Physical property of soil

< Water content >

The water contents of samples range from 2 to 6 % at the upper part of ground water and from 30 to 50 % at the lower part of that. In addition, the water contents of test pit samples (2 m in depth) indicate less than 3 %.

< Specific gravity >

The specific gravity of all samples range from 2.61 to 2.71 (2.66 on an average of sand, 2.68 on an average of stiff clay).

< Liquid limit and plastic limit >

The whole sand materials indicate non plastic (NP) characteristics, on the other hand, the clay materials show liquid limit ranging from 41 to 108 %, plastic limit ranging from 17 to 37 % and plastic index ranging from 21 to 71.

< Grain size distribution >

The summaries of grain size distribution of soils are shown in Table 3.2-5 and Figure 3.2-6 to 3.2-9. As easily understood from the above table and figures, the distributions of grain size at the site are rich in ranging 0.2 mm to 0.8 mm and they correspond to the fine sand and medium sand. Furthermore, the soil conditions are also characterized by low uniformity coefficient (3.0 or less on an average, except for the samples of conveyance canal (CC-10)), which signifies that sand materials are composed of remarkable uniform soil size.

Table 3.2-5 Summary of Grain Size Distribution of Soil

Sample	Location	Num.	Grain size distribution (%, in average)					Uc	Uc'
			Gravel	Sand			Silt/clay		
				Coarse	Medium	Fine			
Bore-hole sample	Conveyance canal (CC-1 to CC-4)	11	0.0	0.1	59.0	39.4	1.5	2.11 - 3.33 (2.604)	0.89 - 1.60 (1.205)
	Conveyance canal (CC-10)	2	9.0	10.5	17.0	57.5	6.0	8.50 (8.500)	0.36 (0.360)
	Spillway	12	0.8	1.7	39.6	50.7	7.3	2.21 - 5.33 (3.446)	0.90 - 1.98 (1.346)
	No.7 Pumping station	23	0.1	0.1	23.9	68.8	7.1	1.72 - 3.73 (2.431)	0.82 - 1.38 (1.014)
	Discharge Tank	4	0.0	0.3	16.5	78.8	4.5	1.63 - 3.77 (2.528)	0.90 - 1.79 (1.183)
Test pit samples (along the canal)		19	0.0	0.1	38.1	58.8	3.1	1.68 - 5.37 (2.861)	0.47 - 1.50 (1.041)

* Uc : Uniformity coefficient, Uc' : Coefficient of curvature

Figure3.2-6 Grain Size Accumulation Curve of Soil along the Conveyance Canal

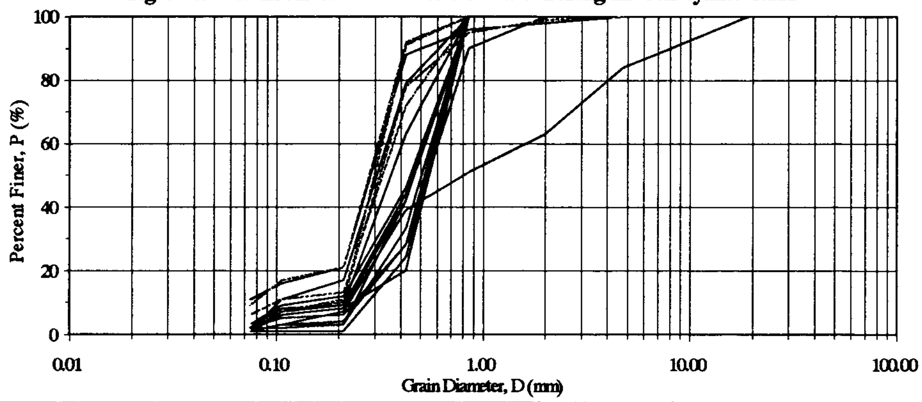


Figure3.2-7 Grain Size Accumulation Curve of Soil at the Spillway

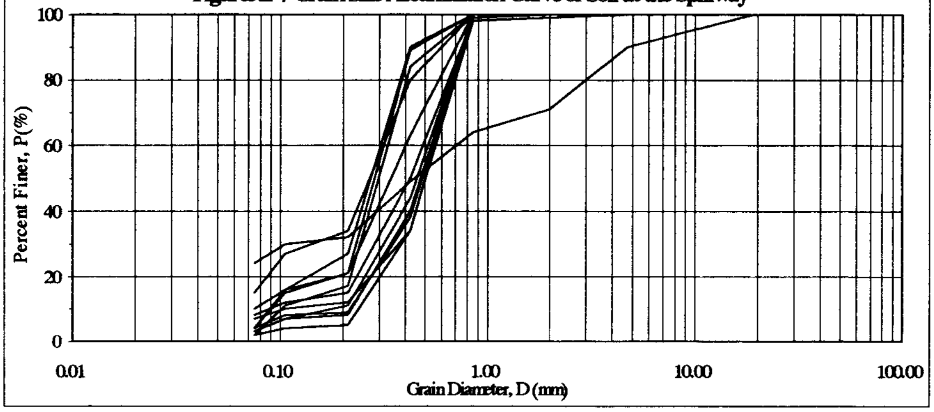


Figure3.2-8 Grain Size Accumulation Curve of Soil at the No.7 Pumping Station

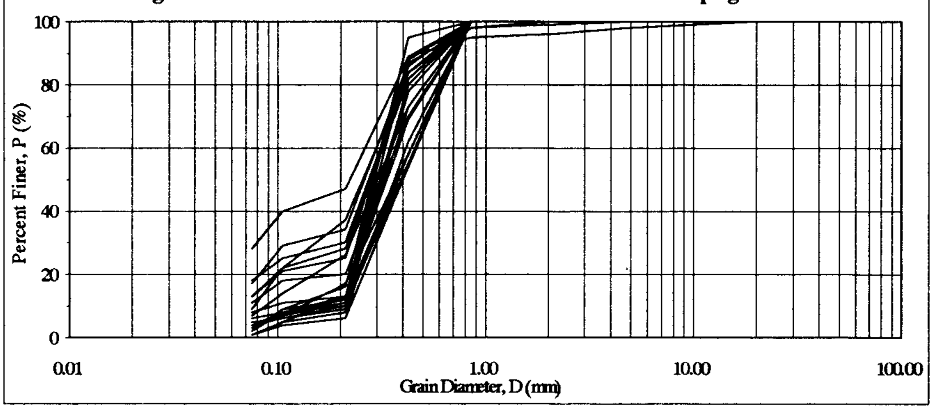


Figure3.2-9 Grain Size Accumulation Curve of Soil by the Test Pit Sample

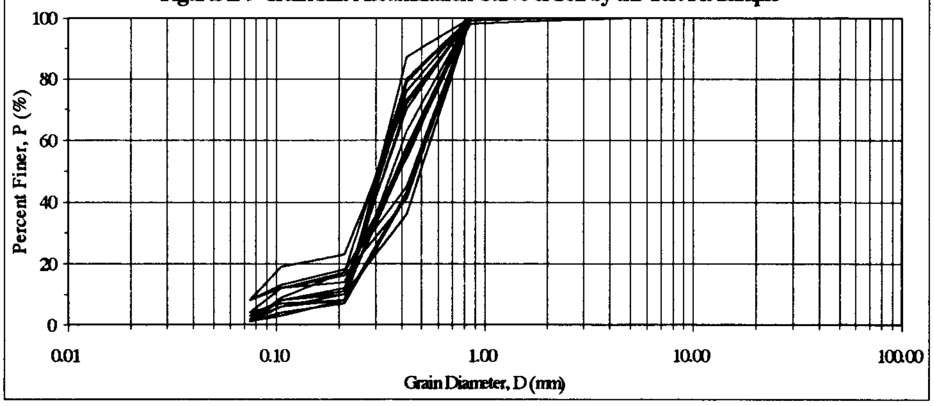
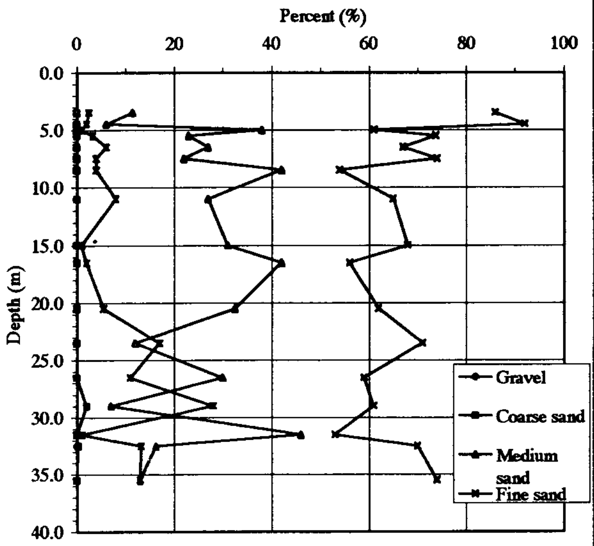


Figure 3.2-10 shows relationship between grain size and depth at the No.7 pumping station. As easily visualized from the figure, the fine sand is remarkably dominant (about 90 %) up to about 5 m in depth, on the other hand, at the deeper depth from 5.0 m, the proportions of grain size between fine sand and medium sand indicate the proportion of fine sand is 50 to 70 % and that of medium sand is 20 to 40 % up to 40 m in depth.

Figure3.2-10 Relationship between Grain Size and Depth (No.7 Pumping Station)



(b)Dynamic property of soil

< Results of compaction test >

The compaction tests were performed by the compaction energy method (50 %, 100 %, 150 % and 200 % as the compaction energy) at the water content 2 % (field moisture content) and 7 % (addition of water), taking soil conditions into consideration. The results of compaction test are shown in Figure 3.2-11 and 3.2-12.

Figure3.2-11 Results of Compaction Test (Wf = 2 %)

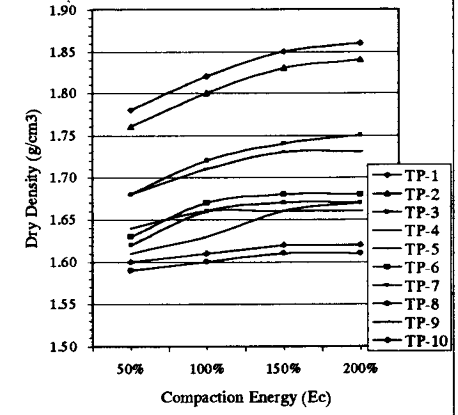
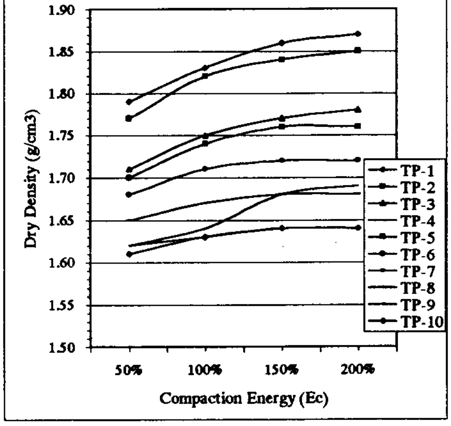


Figure3.2-12 Results of Compaction Test (Wt = 7 %)



As the results of compaction tests, it can be stated that increases of dry density by the soil compaction, which signify increase of compaction energy and/or addition of water, are not dramatic on the compaction curve on the whole, and 150 % or more of compaction energy does not make a large contribution with regard to the increase of dry density.

< Results of permeability test >

The test specimens for permeability test were prepared at the testing point of moisture content 2 % and 7 % on the condition of compaction energy 100 %. The coefficient of permeability range between $2.4*10^{-2}$ cm/sec and $5.2*10^{-2}$ cm/sec ($4.1*10^{-2}$ cm/sec on an average), and the effectiveness by the increase of compaction energy are not apparent on the coefficient of permeability.

< Results of direct shear test >

The test specimens for direct shear test were also prepared at the testing point of moisture content 2 % and 7 % on the condition of compaction energy 100 %. The confirmed shear strengths are as follows.

Cohesion	0.0
Friction angle	32.2 degrees (in the case of moisture content 2 %)
	31.8 degrees (in the case of moisture content 7 %)

(c)Chemical properties of soil

The injurious chemical compounds by the results of chemical test are summarized in the following table.

Table 3.2-6 Summary of Injurious Chemical Compounds (in Average)

Sample	Location	Num.	TDS (%)	Cl^- (%)	SO_4^{--} (%)	pH
Borehole sample	Conveyance canal	6	0.197	0.062	0.048	7.18
	Spillway	2	0.275	0.112	0.054	7.10
	No.7 pumping station	5	0.174	0.059	0.060	7.15
	Discharge tank	4	0.178	0.056	0.063	7.29
Test pit samples		10	0.168	0.053	0.053	7.42

At the above table, the major difference among the whole samples is the presence of relatively high TDS and chloride concentrations at the spillway site, which can be explained that spillway site closes to the shore line and the sea water intrusion occurs at the dike site of spillway.

Furthermore, the chemical water analyses, which were performed by the use of water sample of total 11 boreholes, also revealed that the samples of spillway site is characterized by abundantly containing TDS, carbonic acid, chloride and sulfate in comparison with those of No.7 pumping station.

(8) Results of Aggregate and Concrete Test

The aggregate tests were performed by the use of the following samples, which were collected at the two sampling points.

Table 3.2-7 Sample Descriptions for Aggregate Test

Sampling point	Sample lot No.	Classification of Aggregate	Size of aggregate	Location of collected quarries
Sampling point 1	Sp 1-1	Coarse aggregate	Size I	Crush stone by Attaqua quarries
	Sp 1-1	-ditto-	Size II	-ditto-
	Sp 1-3	Fine aggregate	-	Sand material by West side of Suez Canal
Sampling point 2	Sp 2-1	Coarse aggregate	Size I	Crush stone by El Sir quarries
	Sp 2-1	-ditto-	Size II	-ditto-
	Sp 2-2	Coarse aggregate	Size II	Gravel aggregate by El Sir quarries
	Sp 2-3	Fine aggregate	-	Sand material by West side of Suez Canal

* Sampling point 1 : Concrete plant of No.6 pumping station, Sampling point 2 : Concrete plant near the Bir El Abd branch office of NSDO
* Size I : max. 1.0 cm, size II : max. 2.0 cm
* Crush stones are composed of limestone. Sand materials are derived from the Abassa quarry etc. at the west side of Suez Canal.

The results of aggregate test are summarized as follows.

Table 3.2-8 Summary of Aggregate Test

	Specific gravity	Water absorption (%)	Unit weight (kg/litter)	Surface moisture (%)	Stability (%)	Abrasion resistance (%)	Sieve analysis (FM)
Coarse aggregate	2.50 - 2.66 (2.53)	0.8 - 2.4 (1.7)	1.60 - 1.63 (1.61)	0.3 - 1.2 (0.7)	0.5 - 1.2 (0.9)	8.3 - 11.4 (10.3)	5.7 - 9.5 (7.4)
Fine aggregate	2.58 (2.58)	0.3 - 0.7 (0.5)	1.65 (1.65)	1.0 - 1.6 (1.3)	0 - 0.4 (0.2)	-	1.8 - 1.9 (1.9)

* Figures in the parenthesis show the mean value.

From the above test results, it is concluded that both aggregates are sufficiently suitable for the use as that in concrete. In addition, the results of chemical test indicate that the contents of chloride ion in fine aggregate are not larger than 0.05 % in weight and this value satisfy values of the applied standard.

Furthermore, the concrete tests were performed by the three kinds of tentative mix proportion plan, which correspond to those for canal lining concrete, reinforced concrete and building concrete, respectively. Applied tentative mix proportion plans are as follows.

Table 3.2-9 Tentative Mix Proportion Plan for Concrete Test

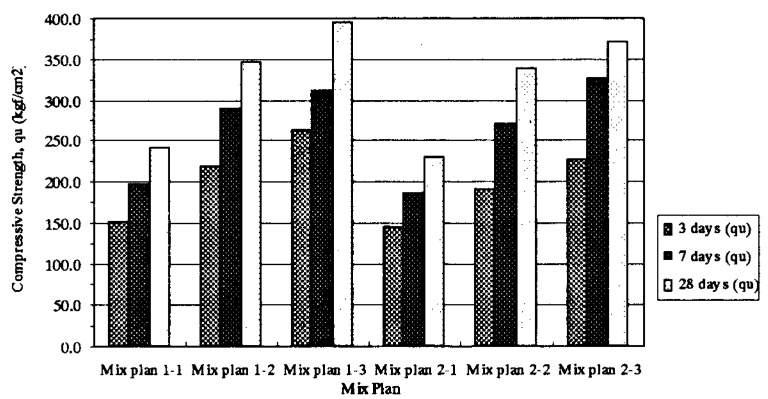
Mix Plan 1-1		Mix Plan 1-2		Mix Plan 1-3	
Mix plan for concrete of canal lining		Mix plan for reinforced concrete		Mix plan for concrete of building	
Materials	Weight	Materials	Weight	Materials	Weight
Aggregate	1,150 kg (sample lot 1-1) Small 480 kg (size 1) Large 670 kg (size 2)	Aggregate	1,237 kg (sample lot 1-1) Small 495 kg (size 1) Large 742 kg (size 2)	Aggregate	1,149 kg (sample lot 1-1 (size 1))
Sand	650 kg (sample lot 1-3)	Sand	633 kg (sample lot 1-3)	Sand	620 kg (sample lot 1-3)
Cement	300 kg	Cement	425 kg	Cement	400 kg
Water	195 liter	Water	205 liter	Water	195 liter

Mix Plan 2-1		(i)Mix Plan 2-2		Mix Plan 2-3	
Mix plan for concrete of canal lining		Mix plan for reinforced concretem[3]		Mix plan for concrete of building	
Materials	Weight	Materials	Weight	Materials	Weight
Aggregate	1,150 kg (sample lot 2-1) Small 480 kg (size 1) Large 670 kg (size 2)	Aggregate	1,237 kg (sample lot 2-2)	Aggregate	1,149 kg (sample lot 2-1 (size 1))
Sand	650 kg (sample lot 2-3)	Sand	633 kg (sample lot 2-3)	Sand	620 kg (sample lot 2-3)
Cement	300 kg	Cement	425 kg	Cement	400 kg
Water	195 liter	Water	205 liter	Water	195 liter

* Kind of cement is sulfate-resisting cement.
* Workability retention admixture (ex. Conplast SP337) was added for concrete of building.

The results of compressive strength test (compressive strength of 3 days, 7 days and 28 days) as concrete test are shown in the Figure 3.2-13.

Figure3.2-13 Results of Concrete Test (Compressive Strength Test)



The whole results of compressive strength test have exceeded tentative target values, which was planned as 220 kgf/cm^2 (concrete of canal lining), 285 kgf/cm^2 (reinforced concrete) and 300 kgf/cm^2 (concrete of building). In this case, it is judged that the above aggregates can be used as that for concrete, in other words, as for the matter about supply of aggregates in this project, it is considered that locating it has not become a problem.

3.3 Hydraulic Conditions

3.3.1 Design Discharges

Based on the Stage Development Plan, the required discharge of each stage is given in the following table.

Required discharge Unit : m^3/sec

Discharge	Stage I	Stage II	Total
Irrigation Water (GIA)	29.52 (85,000 fed.)	17.36 (50,000 fed.)	46.88 (135,000 fed.)
Industrial Water	2.96	2.82	5.78
Required Discharge	32.48	20.18	52.66

Note; GIA: Gross irrigation area.

The beneficial area is to be developed in two stages. The required discharge of the Stage I is 32.48 m^3/sec and the required discharge of the full development in Stage II is 52.66 m^3/sec. The design discharge of the conveyance canal that is agreed upon and decided between NSDO and the Study Team are as follows:

Design Discharge

Conveyance Canal	Stage I	Stage II
No.1, No.2 & No.3 Open Canal and Box Culvert Sections	52.66 m^3/sec	-
No.7 Pumping Station and Pipeline	32.48m^3/sec	52.66m^3/sec

Note : For location of open canals, box culvert, etc., see Figure 4.2-2.

The increase of the design discharge of No.7 pumping station and pipeline will be coped with the increase of numbers of pumping station and row of pipeline.

The minimum design discharge is the minimum monthly water demand that occurs in November and is given in the following table.

Minimum Design Discharge Unit : m^3/sec

Discharge	Stage I	Stage II	Total
Irrigation Water	5.13	3.02	8.15
Industrial Water	2.96	2.82	5.78
Min. Design Discharge	8.09	5.84	13.93

The most frequent discharge could be defined as the mid-value of monthly discharges in descending or ascending order, and it occurs in April and September as follows;

Most Frequent Discharge Unit : m^3/sec

Discharge	Stage I	Stage II	Total
Irrigation Water	12.42	7.30	19.72
Industrial Water	2.96	2.82	5.78
Most Frequent Discharge	15.38	10.12	25.50

3.3.2 Hydraulic Conditions

(1) Beginning Point

The beginning point was discussed at the commencement of the detailed design study and confirmed as the point of the KM86.5 on El Sheikh Gaber Canal from Suez Canal Siphon as follows;

Point : KM86.5 from Suez Canal Siphon
Latitude : 30° -59'-42" N
Longitude : 33° -07'-27" E
Full Water Supply Level : EL.13.64 m after Stage II development

This KM point coincides with the original plan made by the Egyptian side and the JICA F/S report followed this point. However, the modification of canal alignment of the preceding canal under construction in Bir El Abd Zone and the consequent change of the full supply water level has been made. Therefor the coordinates and the full water supply level are different from the F/S report although the KM length is same as the report.

(2) End Point

The full water supply level at the end point of the conveyance canal was determined at EL 90 m for Stage I development so as to guarantee the water supply to the gross irrigation area of 85,000 feddans partially with some small scale re-lifting pumps. The elevation of the ground surface at the end point was selected at the approximately same elevation of the full water supply level. Therefor, the end point of the conveyance canal was selected at the point of intersection of the elevation 90 m contour line and the northwestern boundary of the beneficial area.

End Point : KM 132.50 from Suez Canal Siphon
Full Water Supply Level : EL. 90 m for Stage I development

The rough coordination of the end point will be as shown below.
Latitude : 30° -55'-06" N
Longitude : 33° -31'-35" E

3.4 Design Standards

3.4.1 Hydraulic Design

(1) Open Canal

(a) Allowable Velocity and Manning's Roughness Coefficient

The allowable flow velocities for design of canal structures, that agreed upon and decided between NSDO and the Study Team, are summarized as follows:

Structure	Allowable Flow Velocity	Roughness Coefficient
(i) Concrete Lined Canal	V = 0.5 – 1.5 m/sec	1/n = 55 (n=0.018) I = 8 cm/1km
(ii) Box Culvert	- more than 1.2 times of Concrete Lined Canal	1/n = 67 (n=0.015)

(b) Design Formula

The Manning's formula shall be applied for calculating a velocity of open canal flow.

Manning' Formula : $V = 1/n \times R^{2/3} \times I^{1/2}$

$Q = A \times V$

where :
- V : Velocity in m/sec
- R : Hydraulic Radius in m (= A / P)
- I : Hydraulic Gradient
- n : Roughness Coefficient
- A : Flow Area in sq.m
- Q : Discharge in cu.m/sec
- P : Wetted Perimeter in m

In case of exceeding critical velocity in open canal flow, the Bernoule's theorems will be applied.

(c) Head Losses

Major head losses in open canal are classified as follows:

(i) Friction head loss
(ii) Entrance head loss
(iii) Transition head loss

(i) Friction Head Loss

Friction head loss in a constant cross section of open canal will be computed by the following equation:

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

where ;
- hf : Friction Head Loss in m
- I : Hydraulic Gradient
- L : Length of Canal in m
- R : Hydraulic Radius in m
- V : Mean Velocity in m/sec

(ii) Entrance Head Loss

Entrance head loss, when approaching velocity head can be neglected, will be computed by the following equation:

$hi = fi \times (V_2^2 / 2g)$

where: hi : Entrance Head Loss in m

fi : Entrance Loss Coefficient (refer to Table 3.4-1)

V_2 : Velocity at downstream section in m/sec

Table 3.4 -1 Entrance Loss Coefficient

Shape of Entrance	Coefficient (fi)
Square corner opening	0.5
Circular bell mouth	0.1
Square bell mouth	0.2

Source: MOAFJ's standard (Canal Works)

(iii) Transition Head Loss

Transition loss can be computed by the following equations:

- Expansion head loss : $he = fe \times (V_1^2 - V_2^2) \times 1/2g) + Im \times L$
- Contraction head loss : $hc = fc \times (V_2^2 - V_1^2) \times 1/2g) + Im \times L$

where: he : Expansion Head Loss in m

hc : Contraction Head Loss in m

fe : Expansion Loss Coefficient (refer to Table 3.4 -2)

fc : Contraction Loss Coefficient (refer to Table 3.4 -2)

V1 : Velocity at upstream section in m/sec

V2 : Velocity at downstream section in m/sec

Im : Mean Gradient between upstream end and downstream end of transition

L : Length of transition

Table 3.4-2 Coefficient of Transition Head Loss

Shape of Transition	Coefficient	
	Expansion (fe)	Contraction (fc)
Gradual change	0.2	0.1
Sudden change	1.0	0.5

Source: MOAFJ's standard (Canal Works)

(2) Pipeline

(a) Mean Velocity Formula

Mean velocity of flow in pipe is given by Hazen-Williams formula as,

$V = 0.849\, C \cdot R^{0.63} \cdot I^{0.54}$

where, V : mean velocity (m/s = Q/A)

C : flow coefficient (see Table 3.4-3)

R : hydraulic radius (m)

I : hydraulic gradient

From above equation, following equations are derived for flow in circular pipe.

$$V = 0.355\,C \cdot D^{0.63} \cdot I^{0.54}$$
$$Q = 0.279\,C \cdot D^{2.63} \cdot I^{0.54}$$
$$D = 1.626\,C^{-0.38} \cdot Q^{0.38} \cdot I^{-0.21}$$
$$I = h_f/L = 10.67\,C^{-1.85} \cdot D^{-4.87} \cdot Q^{1.85}$$

where, D: diameter (m)
h_f: friction loss head (m)
Q: discharge (m^3/s)
L: pipe length (m)

Table 3.4-3 Flow Coefficient (C)

Pipe & Inner Coating	Flow Coefficient C		
	Max.	Min.	Standard
Cast iron pipe (no coating)	150	80	100
Steel pipe (no coating)	150	90	100
Coal tar coated (cast iron pipe)	145	80	100
Tar-epoxy coated (steel pipe)			
≧ φ800	-	-	130
φ600 to 700	-	-	120
φ350 to 500	-	-	110
≦ φ300	-	-	100
Mortar lined (steel or cast iron pipe)	150	120	130
Reinforced concrete pipe (factory-made)	140	120	130
Pre-stressed Concrete pipe	140	120	130
PVC pipe	160	140	150
Polyethylene pipe	170	130	150
Fiber-reinforced plastic mortar pipe	180	-	150

Note : Standard value of Flow Coefficient C shall be adopted in general.

Source : MOAFFJ's standard (Pipeline)

(b) Hydraulic Calculation

(i) Calculation method

One of objectives of hydraulic calculation is to find the minimum cross section to allow design discharge by making full use of available hydraulic energy head. The minimum cross section can only be found by firstly assuming the pipe diameter and secondly calculating friction head loss and other head losses produced by the design maximum discharge in the following manners.

①Friction loss heads: by Hazen-Williams Formula

②Other loss heads: to be calculated in meeting cross-sectional and longitudinal design alignment

- by flow entry

- by outflow
- by bend or curve
- by change of cross sectional flow area
- by flow diversion
- by confluence
- by valve
- by flow-meter
- by trash screen

③Total loss head: sum of friction and other loss heads

(ii) Friction loss heads

Friction loss heads shall be calculated from Hazen-Williams Formula to the design discharge by selecting values of flow coefficient (C) and applying the equation below.

$$h_f = 10.67\, C^{-1.85} \cdot D^{-4.87} \cdot Q^{1.85} \cdot L$$

where, h_f : friction loss head (m)
C : flow coefficient (see Table3.4-3)
D : pipe diameter (m)
Q : design discharge (m^3/s)
L : pipe length (m)

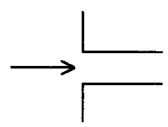
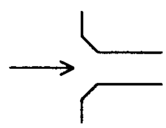
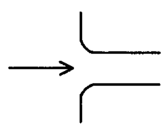
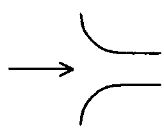
(iii) Other loss heads

①Loss heads by flow entry

$$h_e = f_e \cdot \frac{V_2^2}{2g}$$

where, h_e : loss head by flow entry (m)
V_2 : mean velocity after entry (m/s)
g : gravity acceleration (m/s^2)
f_e : entry loss coefficient (see Table3.4-4)

Table 3.4-4 Entry Loss Coefficient by Entrance Shape

	Right angle edge	chamfered edge	Rounded edge	bell-mouth
Entry Shape				
fe	0.5	0.25	0.1(circular) to 0.2(square)	0.01 to 0.05

Source : MOAFFJ's standard (Pipeline)

② Loss head by outflow

$$h_o = f_o \cdot \frac{V_1^2}{2g}$$

where, h_o : loss head by outflow (m)
V_1 : mean velocity before outflow (m/s)
g : gravity acceleration (m/s^2)
f_o : outflow loss coefficient

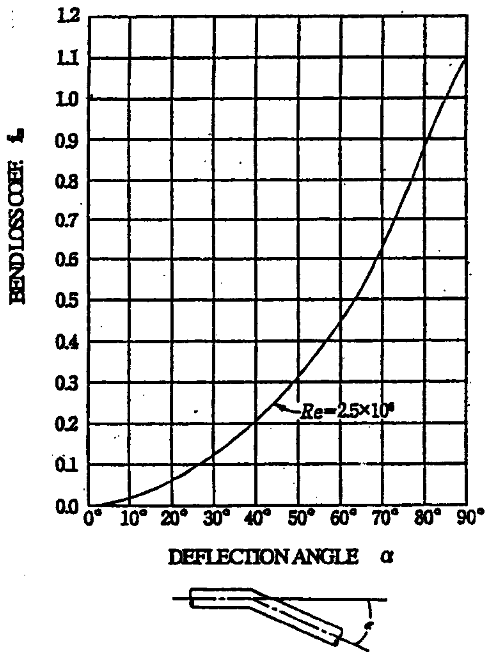
In case water is discharged into a tank or in the air, the velocity head is totally lost as f_o=1.0 in general.

③ Loss head by bends

$$h_{be} = f_{be} \cdot \frac{V^2}{2g}$$

where, h_{be} : loss head by bend (m)
V : mean velocity (m/s)
G : gravity acceleration (m/s^2)
f_{be}: bend loss coefficient

Bend loss coefficient is dependent on Reynolds number (Re) and deflection angle α as given by Figure 3.4-1.



Note : Loss head to be multiplied by number of deflection.
Source : MOAFFJ's standard (Pipeline)

Figure 3.4-1 Bend Loss Coefficient Value

(3) Analysis and Countermeasures of Water Hammer

(a) General Description of Water Hammer

(i)Hydraulic environment of No.7 pumping station

As it discusses in the Chapter 5, hydraulic conditions of No.7 pumping station are lather large scale pumping plants e.g. total lifting head of approx. 100m and design discharge of 10.728 cu.m./sec per one unit with 7 units installation including 2 units standby and 5 rows, 2,400mm diameter delivery pressured steel pipelines with approx. 9.4 km long in total. Therefore, careful study and countermeasures for water hammer phenomenon shall be carried out at the detailed design stage.

(ii)Necessity of water hammer study

As the general tendencies of the pumped delivery pipelines, the study of water hammer shall be carried out to provide adequate countermeasures when the pump facilities of feeding water through pressured pipelines suddenly stops and or check valves is suddenly operated with following general conditions.

- Length of pressured pipelines is more than about 20 times of the actual heads and also actual head is more than 10m.
- There is a raised portion in the pipelines which air trap and or water column separation may occur.
- The pump is started while the discharge valve is open
- Air remains between the pump and the discharge valve for vertical shaft pump or is insufficiently vented.
- The valve operation time is shorter than the time of one cycle of pressure wave reciprocation.

(b) Methodology of water hammer analysis

(i) Empirical estimate of water hammer pressure

Water hammer pressure in a pumped pipeline system is calculated based on the different static pressures depending on delivery systems.

- Water hammer pressure is taken as 100% of water pressure during pumping (dynamic head) if the head is below 4.5 kgf/cm^2
- Larger value of 60% of the head or 4.5 kgf/cm^2, if the head exceeds 4.5 kgf/cm^2. (Refer to Figure 3.4-2) Source : MOAFFJ's standard (Pipeline)

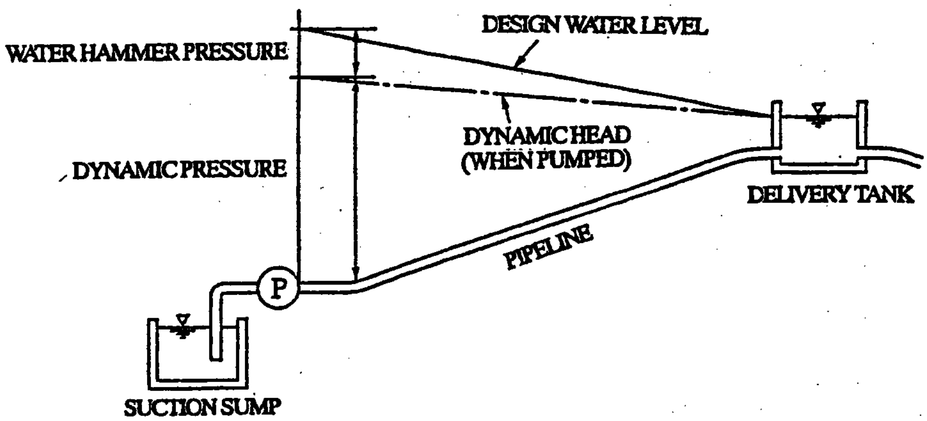


Figure 3.4–2 Water Hammer Pressure in Delivery Tank System

(ii)Estimation by Elastic Theory-Based Unsteady Flow Analysis

Water hammer analysis, in general, are simulated based on the equations of kinetic equation as unsteady flow analysis and equation of continuity considering with time lag simulation for the analysis of transient flow. Actual analysis of water hammer considering phenomenon of "water column separation" and "predetermine of check valve operation programmes" when the pump with pressured pipelines suddenly stops for electrical and or mechanical failures will be carried out using computer systems at the detailed design stage.

(c) Countermeasure to Reduce Water Hammer Pressure

In order to prevent water hammer phenomenon, two category countermeasures shall be considered e.g. protection of pressured pipelines and main pumps for up surging, and countermeasures of negative pressure for the pressured pipelines in the case of down surging.
There are many countermeasures to prevent and or mitigate up surging and down surging water hammer pressures. The methods indicated in the Figure 3.4-3 to 3.4-6 are common for prevention of down surging. The general characteristics of respective methods can be summarized as follows ;

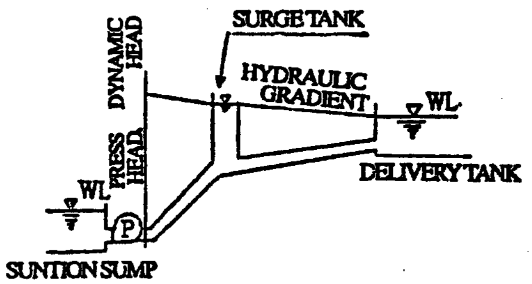
(i) Conventional surge tank : The conventional type with relatively large sectional area is installed midway in the pipelines in case where a decrease in the pressure of the pipeline requires a large quantity of water to be supplied. Conventional surge tank, however, requires the same height as the hydraulic grade for pressured water. Therefore, it is not practicable for this project case due to high cost of construction of tank.

(ii) One-way surge tank : This system supplies water in one direction only from the tank to the pressured pipelines when the pressure in the pipelines unusually decreases. Usually, check valve is installed between the tank and pressured pipelines to prevent flow in the reverse direction. This valve opens only decreasing pressure in the pressured pipelines.
The one-way surge tank is more economical to install than conventional surge tank because it is not so tall facility than that of conventional surge tank.

(iii) Pressure tank (Air chamber) : Pressure tank, holding a pressure equal to that generated by the pump, affects the entire length of the pressured pipelines. In general, this tank shall be applied commonly for lather small scale pump with sharing large portion of friction losses to the static head.

(iv)Air Valve or Air Vent : In a water pressured pipeline of convex form, the water downstream of the convex points in the pressured pipelines continue to flow even after the sudden pump stop. Air valve and or air vent is installed at the convex point to supply air when the pressure in the pipeline decreases.

- In case of the installation point is close to the hydraulic grade, an air vent is used to supply air,
- In case of the installation point is too far from the hydraulic grade, an air valve is installed to supply air. If air is supplied to the pipelines, the air must be removed when water is fed. Therefore air supply should be used to prevent water hammer only there are no way applicable.



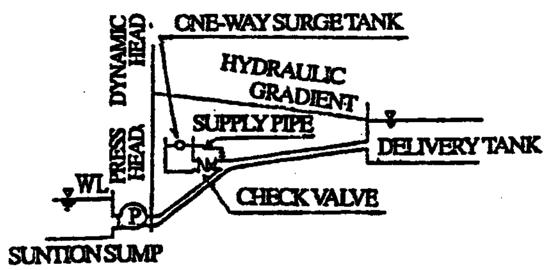
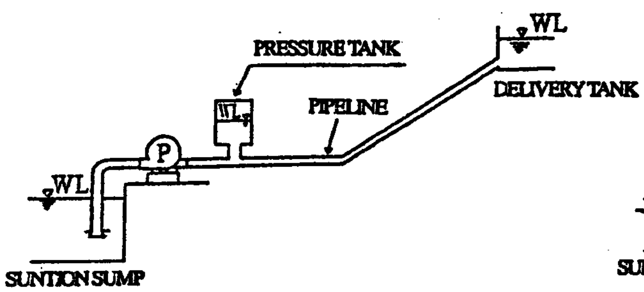


Figure 3.4-3 Conventional Surge Tank   Figure 3.4-4 One-way Surge Tank



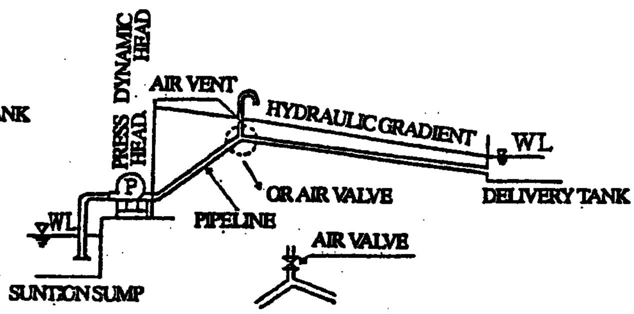


Figure 3.4 -5 Pressure Tank (air chamber) Figure 3.4 -6 Air Valve or Air Vent

On the other hand, mitigation manners of up surging pressure to the pipelines and or pumps are provision of flywheel and control butterfly valves. Function of the former is to increase the inertial effect (GD square) of the rotating pump portion to prevent too sudden a decrease in rotation speed after sudden pump stop. However, this countermeasure is not applicable for vertical shaft type pump. The latter is installed butterfly check valve with pre-programmed operation manners between the pump and delivery pressured pipelines.

3.4.2 Structural Design (Civil and Building)

(1) General

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.

(2) General Design Conditions for Structure

(a) 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:

(i)Allowable Stress of Reinforced Concrete

Table 3.4-5 Allowable Stress (1)

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.

(ii) Allowable Stress of Plain Concrete

Table 3.4-6 Allowable Stress (2)

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.

(iii)Allowable Tensile Stress of Steel

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

(b) Loadings

(i) Dead Loads

The dead-load 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)

(ii)Live Loads

Structures on which heavy wheels pass over the side of the structure should be designed for the wheel loads, and where heavy wheels do not pass through the side of the structure, a live load of 300 kg/m^2.

(iii) 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. Especially, concrete structures under the ground level like the box culverts, as it is not considered. The liquefaction effect caused by the earthquake does not apply in the Study area, becouse the moisture content of the soil in the area is well below the danger point of 85 %.

(3) Box Culvert

(a) General

In the structural calculations for the box culverts, conditions of loading on the ground, geology, ground water conditions, construction conditions, etc. shall be taken into consideration and the magnitude and distribution of loads on the structures calculated, and the required strength of members determined for maximum loading conditions.

(b) Combination of Loads for Box Culverts

Combination of loads for the box culverts shall be in accordance with the combination shown in Figure 3.4-8 and Table 3.4-8.

(c) Formula for Earth Pressure

The earth pressure application to a box culvert is, as a rule, determined by using the following formula:

Vertical earth pressure (Vertical earth pressure's formula):

$$W_v = \gamma \cdot H$$

Horizontal earth pressure:

$$P_h = K_0 \cdot \gamma \cdot H$$

Where, K_0 : earth pressure coefficient $K_0 = 0.5$

Source: MOAFJ's standard (Canal Works)

γ : Unit volume weight of backfill earth or embankment (tf/m^3)

H : Depth from the top of the box culvert to the surface of backfill earth or embankment (m)

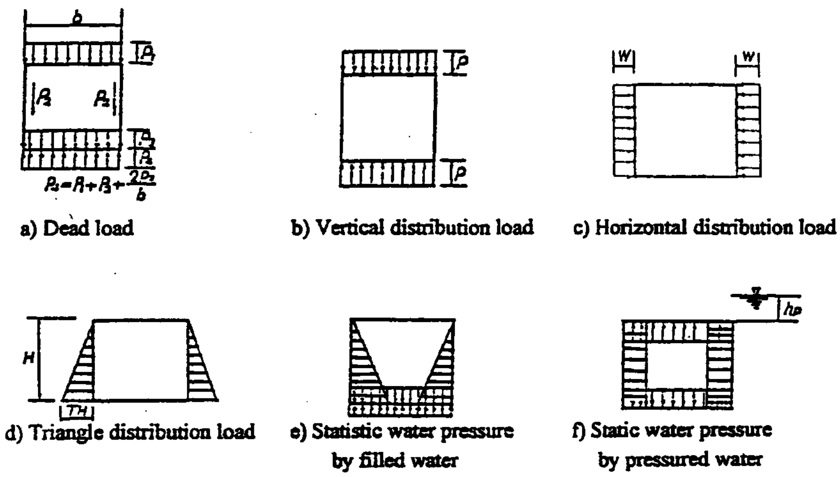


Figure 3.4-7 Combination of Loads Applied on Box Culverts

Table 3.4-7 Combination of Loads for Box Culverts

Loads			Combination					
			I	II	III	IV	V	VI
a)	Dead Load		○	○	○	○	○	○
b)	Uniformly distributed vertical Load	Dried earth			○	○	○	○
		Saturated earth	○	○				
		Live load	○	○				
c)	Uniformly distributed Horizontal Load	Dried earth			○	○	○	○
		Saturated earth	○	○				
d)	Triangular horizontal Load	Dried earth					○	○
		Saturated earth	○	○	○	○		
e)	Static water pressure due to replete water		○			○	○	○
f)	Static water pressure due to pressurized water					○	○	○

Note; ① Condition I causes the maximum tensile stress inside the top slab and floor slab.
② Condition II causes the maximum tensile stress outside each corner point.
③ Condition III causes the maximum tensile stress inside the side walls.
④ Condition IV causes the maximum tensile stress outside the center of each member.
⑤ Condition V causes the maximum tensile stress inside the center of each member.
⑥ Condition VI causes the maximum tensile stress inside the each corner point.

(4) Pumping Station

(a) Design Conditions

(i) Internal Friction Angle of Backfill or Embankment

Internal friction angle of backfill or embankment is 30 degree.

(ii) Live Load (Truck)

- At outside of pumping station;
  Wheel load: q = 1.00 tf/m^2
  Crowd load: q = 0.30 tf/m^2
- At inside of pumping station;
  Crowd load: q = 0.50 tf/m^2

(b) Design Policy

(i) Suction Sump

- Structural frame: Box Rahmen of crossed section
- Load: Load by building and machine
- Hydraulic pressure: Empty in suction sump
- Locations of checking: At the maximum load by machine
- Case of checking: Normal period

(ii) Side and Intermediate Wall

- Structural frame: 4 sides fixed slab by top slab, both side walls and bottom slab
- Case of checking: Normal period
- Hydraulic pressure: Empty in suction sump (Full water in one-side for intermediate wall)

(5) Bridge

The design conditions of bridge, that are agreed upon and decided between NSDO and the Study Team, are as follows;

(a) Width of Bridge

The width of bridge is as follows;

For vehicles	2 lanes	x	4.0 m	=	8.0 m
For shoulder	2	x	1.0 m	=	2.0 m
For side-walk	2	x	1.5 m	=	3.0 m
For guardrail	2	x	0.5 m	=	1.0 m
Total width					14.0 m

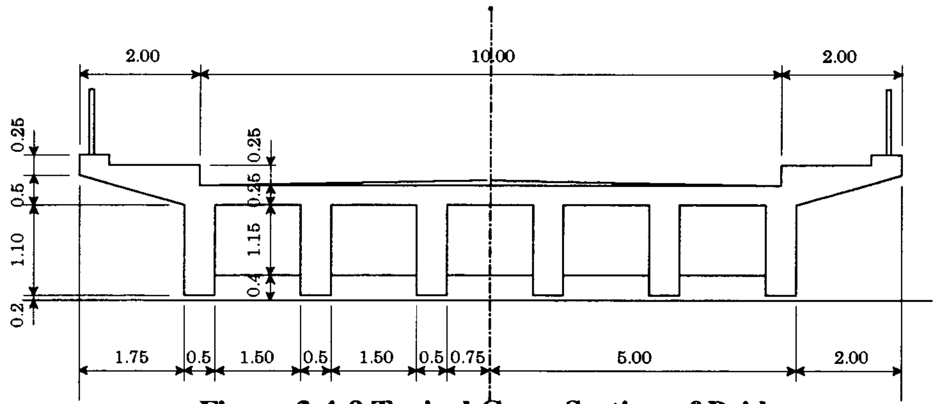


Figure 3.4-8 Typical Cross Section of Bridge

(b) Live Loads on Bridge

Following the Egyptian code of practice 1994:

- Main lane (3m 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.

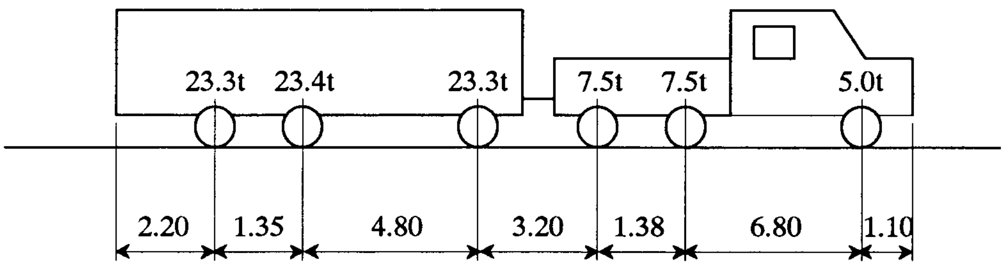


Figure 3.4-9 Track Type for High Way

(6) Cross-sectional Design of Pipe Laying

(a) Foundation and Fill Material

Foundation and fill materials for laid pipes shall in principle be sand or fair sandy soil. Attentions to be paid on the materials are as follows;

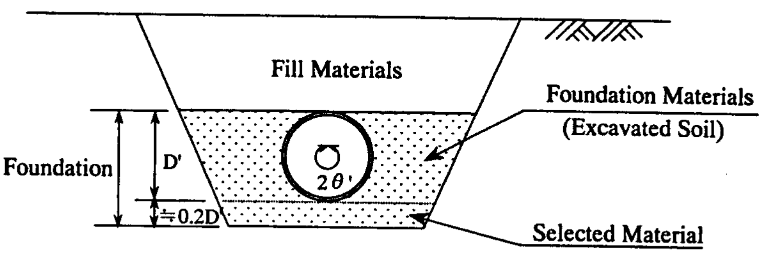
① Foundation materials shall not give any negative effects on pipe body and joints.

② Upon back-filling, if the excavated materials at site is improper for back-filling, fair materials (e.g. purchased soil) shall be used.

③ Bedding material shall be adequate as foundation of pipe

Support angle of foundation shall be, in accordance with Spangler's horizontal earth pressure theory (homogeneous material on pipe sides) and for simplification of design, be $2\theta'$=360°

Deflective Pipe



$2\theta'$: Support Angle $2\theta' = 360°$

Figure 3.4-10 Foundation and Fill Materials in Laying Cross-section

For cross sectional design of pipe laying, simultaneous actions by inner and outer pressures shall be carefully examined to assure due safety.

(b)Design Support Angle

Standard values of design support angle for calculation of bending moments by soil type are given as follows.

Table 3.4-8 Design Support Angle of Compacted Foundation for Deflective Pipe

Soil type	Soil Class	Design Support Angle (Laid 360 °)
Gravel Soil	G, GS	120
	GF	90
Sandy Soil	SW,SW-G or SGW in S and SG	120
	SP,SP-G or SGP in S and SG	**90**
	Other S, SG and SF	90

(c)Selection of Deflective Pipe and Pipe Thickness

(i) General considerations

Pipe thickness is solved by taking static water head, pressure arised from water impulse, soil pressure, traffic load, and the outside pressure due to vacuum in the pipe.

(ii) Deflection Ratio

Deflection of deflective pipe can allow up to several % of diameter at center of pipe thickness. In case horizontal deflection Δx becomes abnormally large, it may cause leakage at joints, cracks on coating on both faces or insufficient cross sectional flow area.

For prevention of these, allowable deflection ratio δ shall be defined as a ratio of deflection to pipe diameter at center of pipe thickness ($\delta = \Delta x / 2R \times 100\%$). A value of δ shall be determined by taking coating conditions into account. Standard value of the design deflection modulus is given in Table 3.4-11, wherein, design deflection is $\Delta X = \delta \times 2R/100$.

Table 3.4-9 Standard Value of Design Deflection Ratio

Coating of Interior Surface	Paint	Mortar
Allowable deflection ratio (%)	5%	3%

For selection of pipe type, required two values of pipe thickness, one from allowable stress of pipe material and another from design deflection, shall be calculated and then a safer value shall be employed.

(iii) Formula for Calculating Thickness of Steel Pipe by Inner Pressure

In case all outer loads are borne by concrete mass totally embedded inside, the thickness can be determined only by inner pressure as;

$$t \geq \frac{H \cdot D}{2\sigma_a}$$

where, t : required thickness of pipe by stress (cm)

D : inner diameter of pipe (cm)

H : design inner pressure (kgf/cm^2); H=H1+H2

Wherein, H1: static pressure

H2: water hammer pressure

σ_a: allowable tensile stress (kgf/cm^2)

In case fittings are totally embedded within concrete mass such as thrust block and outer loads are all borne by the mass, the calculation shall be by the above equation. In case partly embedded, the calculation shall be made onto the portion not embedded in the mass.

(iv) Formula for calculating pipe thickness from outer loads

The formula for computing the bending stress and deflection of pipe is shown as follows;

・ Bending stress $\sigma_b = \frac{2}{fZ}(Wv + Wt)\frac{Kb \cdot R^2 \cdot EI + (0.061Kb - 0.083Kx)E' \cdot R^5}{EI + 0.061E'R^3}$

・ Displacement $\Delta X = \frac{2Kx \cdot (Wv + Wt) \cdot R^4}{EI + 0.061E' \cdot R^3}$

where, σb : bending stress at the bottom of pipe (kgf/cm^2)
ΔX : horizontal deflection of pipe (cm)
Kb : coefficient for bending moment at the bottom of pipe
Kx : bedding constant, its value depending on the bedding angle
R : mean radius of the pipe (cm)
E : modulus of elasticity of the pipe material (kgf/cm^2)
I : moment of inertia per unit length of cross-section of the pipe wall
$I = t^3/12$ (cm^4/cm)
Wv : vertical soil load on conduit (kgf/cm^2)
Wt : vertical traffic load (kgf/cm^2)
E' : modulus of soil reaction (kgf/cm^2)
f : coefficient by shape (1.5)
Z : section modulus $Z = t^2/6$ (cm^3/cm)

The coefficients for bending moment Kb and the beddin g constant Kx are shown in the following table, the values of which charge by tube supporting angle.

Table 3.4-10 Standard value of Kb,Kx

Design Support Angle	Kb	Kx	(0.061Kb-0083Kx)
60°	0.189	0103	0.00307
90°	0.157	0.103	0.00171
120°	0.138	0.096	0.00107
150°	0.128	0.085	0.00082

①Earth Pressure Distribution Model

As the earth pressure distribution model, Spangler's distribution model shall be applied for deflective pipe.

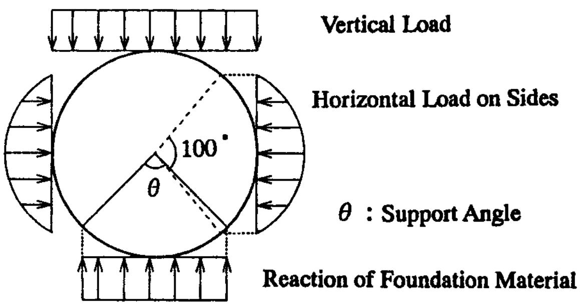


Figure 3.4-11 Load distribution

②Vertical Earth Pressure Formula for Deflective Pipe

Following earth pressure formulae shall be applied for deflective pipe.

Vertical Earth Pressure
- $H \leqq 2.0$ m — $Wv = w \cdot H$ (vertical earth press. F.)
- $H > 2.0$ m
  - ditch type: $Wv = Cd\ w\ B$ (Marston's F.)
  - projected type: $Wv = Cc\ w\ B$ (Marston's F.)
  - sheet pile work: $Wv = w \cdot H$

(v)Allowable Stress of Steel Pipe

allowable stress of steel pipe shall be applied as following table.

Table 3.4-11 Allowable Tensile Stress of Steel Pipe

Material (JIS)	Tensile Stress (kgf/cm2)	Allowable Stress σ_a (kgf/cm2)	Related Other Standards	
			ASTM	ISO
SM490	5,000	1,710	A516	
STW400, STPY400	4,100	1,400	A283, A285 A570, A36	ISO559

(7) Buildings

The extent of the structural design will be to provide the necessary structural system to support all proposed buildings loads and at the same time to be compatible with the architectural scheme and electrical and mechanical systems. This includes the design of all superstructures such as slabs, beams, columns, and walls. The general design criteria and considerations for the structural design of buildings are as follows:

(a) Scope of works

Structural design for Stage I Pump House in line with the architectural design concepts as stated in Sub-clause 5.6.6 basic Design of Pump House.

Structural design will include structural calculation, general notes, framing plans and standard details and so on.

(b) Codes and Standards

Structural design for buildings shall be carried out in accordance with the latest edition of Egyptian Code of Practice 1994 (E.S.S).

(c) Design Method

The design method for all buildings shall be "Ultimate Strength Design" and the Ultimate load U shall be taken as follows:

- Dead load D.L. and Live load L.L. : U= 1.4 D + 1.6 L
- Dead Load D.L. and Wind Load W: U= 0.9D + 1.3 W
- Dead load D.L. and Live load L.L. plus Wind Load

  W = 0.8 (1.4D + 1.6L + 1.6W)

(d) Design Loads

(i) Dead Loads:

Dead loads shall include the weight of the structure itself plus superimposed loads such as partitions, flooring, ceiling, roofing, mechanical equipment bases, services and all other permanent constructions. Dead loads will be calculated from the actual weight of the materials used. The following list is a summary of the typical uniform dead loads in the design of the building:

- Weight of reinforced concrete	2,500 kg/cu.m
- Weight of steel	7,850 kg/cu.m
- Concrete finish topping	2,200 kg/cu.m
- Hanging load (ceiling, mech. elect.,..etc)	75 kg/cu.m
- Hollow block	1,150 kg/cu.m
- Mortar	2,100 kg/cu.m

(ii) Live Loads:

The following list is a summary of the uniform live loads applicable to the design of the building:

<u>Type of Floors / Rooms:</u>

- Roof

  Variable according to roof function (Accessible: 200 kg/sq.m Inaccessible: 100 kg/m^2).

- Office rooms	250 kg/sq.m
- Stairs	400 kg/sq.m
- Balconies	400 kg/sq.m
- Toilet rooms	300 kg/sq.m
- Corridors	400 kg/sq.m

- Mechanical and Electrical Room

  Variable according to type of equipment (taking dynamic effect and vibration into consideration)

- Workshops:

  Variable according to the usage of the room in addition to dynamic effect due to machine vibration which shall be taken into consideration

(iii) Wind Loads:

Buildings shall be designed to resist all horizontal wind loads acting on all surfaces subjected to wind load. Wind effect is taken as not equivalent static pressure, Wi, uniformity distributed along the whole width of the elevation exposed to wind where,

Wi = C.Ks.Wd (kg/sq.m)

Wi: total wind pressure kg/cm^2 (includes pressure and suction)

C: factor taking into consideration the relationship between Building Height and its plan dimensions and degree of inclination of its surface.

Ks: 1 for normal location, 1.3 for locations, heavily exposed to high wind pressure.

Wd: static load pressure equivalent to wind effect (kg/sq.m), it is dependent on height above ground level as follows:

Height (m)	Kg/sq.m
0 to 10	50
10 to 20	60
20 to 30	65
30 to 40	75
40 to 60	80

Souse : Design of reinforced concrete "COLUMNS" according to new Egyptian code concept. 1996.

(iv) Seismic Loads:

As stated in Section 2.3.2 of the Interim Report (1), seismic load will not be considered for the design of the buildings.

(v) Accidental and other loads

- Dynamic loads:

The structure shall be designed and constructed to resist the effect of dynamic load such as rotating and/or vibrating machines.

The use of factored imposed static loads equivalent to the dynamic effects as specified by the manufacturer is adequate.

(e) Materials of Construction

The following information is a summary of the materials of construction proposed for use in the structure. All materials described below shall conform to the applicable standards listed in E.S.S.

(i) Concrete:

- Columns, walls, beams, girders and slabs: fcu = 300 kg/sq.cm
- Plain concrete fcu = 200 kg/sq.cm
- Foundations: Refer to Structural Design (Civil).

(ii) Steel:

-Normal mild steel: Grade 24/35 fy =2400 kg/sq.cm

-High grade steel: Grade 36/52 fy =3600 kg/sq.cm

(iii) Cement:

Cement shall conform to E.S.S.

Sulphate Resistant Cement (SRC) shall be used in all concrete in contact with soil. Ordinary Portland Cement (OPC) shall be used for building superstructures.

(iv) Aggregate:

Concrete aggregates shall conform to E.S.S.

(v) Water:

Water used for mixing concrete shall be clean and free from injurious amounts of soils, acids, alkaline, salts, organic materials or other substances that may be deleterious to concrete or reinforcement.

3.4.3 Structural Design of Mechanics

(1) Design Standard

Since ISO (International Organization for Standardization) and IEC (International Electrotechnical Commission) are worldwide federation of national standards institutes, pump and related equipment shall be manufactured in accordance with these codes/standards. However, it will be specified in parallel on the tender documents that codes/standards which have been adopted by various associations of manufacturer and engineering societies shall be acceptable, when the supplier requires to apply the codes /standards of his country upon they shall submit the codes/standards of the English version. These codes/standards used for materials and methods as well as design, manufacturing and testing should be agreed by MED. In this connection, the Team shall apply the following standards in principle.

(a) Standards to be Applied

Manufacturing	
ISO/TC 1	Screw threads and ISO/TC 2 Fasteners (TC means technical committee)
ISO/TC 3	Limits and Fits
ISO/TC 4	Rolling bearing
ISO/TC14	Shafts for machinery and accessories
ISO/TC 5	Ferrous metal pipes and metallic fittings
ISO/TC11	Pressure vessel
ISO/TC17	Steel
ISO/TC26	Copper and copper alloys
ISO/TC44	Welding and allied processes
ISO/TC57	Metrology and properties of surfaces (ISO 468)
ISO/TC60	Gears
ISO/TC100	Chains and chain wheels for power transmission and conveyors
ISO/TC108	Mechanical vibration and shock (ISO1940 & ISO3945)
ISO/TC153	Valves
Testing	
IEC-497	International code for model acceptance test of storage pumps
IEC-198	International code for the field acceptance test of storage pumps

(b) Severity of Vibrations

Quality of balancing of the rotating parts shall be kept to Q2.5 or better according to ISO1940. The vibration severity of the machinery shall correspond to grade "Good" or better in ISO3945.

(c) Rating

All parts of the plant and machinery shall be rated for 24 hours continuous running all year round at a maximum ambient temperature of 45 ℃.

(2) Requirements for Pump Design

As a result of the studies, the pump shall be of the vertical-shaft, single-suction, centrifugal type with direct connection to a vertical shaft motor. The pump shall be designed and constructed to meet the following general requirements.

1) The impeller should be removable without disturbing the suction cover and the entire rotating element should be able to be removed from the casing when the top cover is taken away.
2) The impeller and shaft should be movable and enough space for adjusting and dismantling both the thrust bearing and clearing the motor shaft without coupling bolts, should be made.
3) An oil-lubricated thrust bearing as a part of motor should be bearable against both the total weight of the rotating parts in pump and maximum unbalanced hydraulic thrust which is occurred in case that clearance between impeller and wearing expands more than two times of design value.
4) All parts of pump should be designed for safety operation against the stress by the maximum reverse speed in power failure without consideration of unrestricted reverse flow, friction and windage.

(a) Each Individual Part

(i) Impeller

1) General

The impeller shall be of the enclosed type and shall be made in entirely one piece of stainless steel (ASTM designation; A743M Grade CF-8) or equivalent, selected for and resistance、to cavitation and abrasion. Impeller shall be equipped with replaceable wearing rings.

Austenitic stainless steel (ASTM A743M previous A296) Grade CF-8 is selected as suitable material for the impeller in this project because fluid to be lifted contains slightly mild acid corrosives such as chloride, carbonate which discharged from upstream irrigation area.

This material is best known and widely used for most application. It is markedly superior in corrosion resistance compared to the chromium stainless steel, and may make unnecessary the high temperature water quench after welding. Following shows chemical composition and minimum mechanical properties of CF-8.

Carbon	Manganese	Phosphorus	Sulphur	Silicon
max.0.08	max.1.50	max.0.04	max.0.04	max.2.00
Nickel	Cromium	Tensile, MPa	Yield, MPa	Elongation,min%
8.0~11.0	18.0~21.0	485	205	35

2) Design and fabrication

The impeller shall be thoroughly inspected by a non-destructive method (liquid penetrant examination). The impeller shall have sufficient strength to withstand forces by runaway speed and to support its own weight and the weight of the main shaft when the latter is disconnected from the motor shaft and the impeller is resting on a ledge or shoulder in the suction cover or suction tube liner. The finished impeller shall be dynamically balanced.

(ii) Shaft

1) General

The pump shaft shall be made of forged, open-hearth carbon or alloy steel properly heat treated. The shaft shall be provided with integrally forged flanged couplings for connection to the impeller and to the intermediate shaft/ motor shaft.

2) Design and fabrication

The shaft shall be of ample size to operate at any speed up to full reverse runaway speed without excessive vibration or objectionable distortion. The size of the shaft and the construction of the main bearing support shall be such that any shaft deflection or unbalanced radial thrust on the impeller under any condition of speed up to full runaway speed or under any condition of discharge from maximum capacity to shutoff. Shall not cause contact between the impeller and casing ring.

(iii) Casing

1) General

The pump casing shall be of diffuser type construction as shown in Figure 3.4-1 and shall be fully embedded in concrete. The casing shall be constructed in radial sections and shall have the least number of sections practicable for shipment and handling. The casing shall be of welded plate steel.

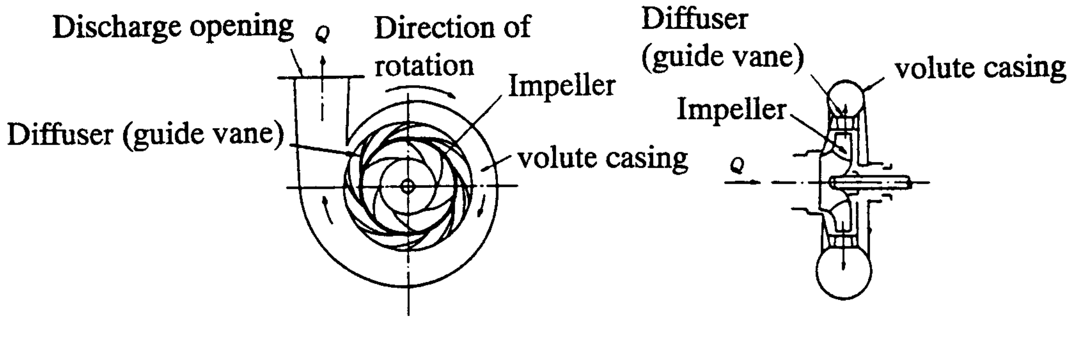


Figure 3.4-12 Diffuser Type Pump

2) Design and fabrication

Unless otherwise specified, the pump casing shall be designed and fabricated in accordance with a pressure vessel code approved by MED.

(iv) Guide bearing

The main guide bearing shall be of the babbit-lined oil lubricate type, and shall consist of a bearing housing and a removable bearing shell. The bearing shall be located above the stuffing box and as close to the impeller as possible and shall be suitable for contiguous operation with the shaft rotating in either direction.

(v) Casing and impeller wearing ring

Removable and renewable wearing rings shall be provided where there are close running clearance between the impeller and the stationary parts of the pump. To avoid biting each other, one of each pair of wearing rings shall be made of stainless steel. Brinell hardness approx. 300 shall be applied. The other wearing ring of each pair shall be made of aluminum bronze with Brinell hardness of approx. 160.

(vi) Stuffing box

A stuffing box designed so that it can be repacked and adjusted without disturbing the guide bearing or dismantling any other part of the pump, shall be provided where the shaft passes through the casing cover. The stuffing box shall be constructed to permit minimum seal water flow without excessive pressure on the packing and shall be packed with high-grade, Teflon type, hydraulic packing.

(3) Requirement for Auxiliaries

(a) Discharge Valves

(i) Type and description

The type of valves placed in the discharge lines from pumps must be carefully selected to give the most economical installation while providing the desired operating conditions. Hydraulically cylinder-operated butterfly valves, electrically controlled, in the discharge lines from vertical type centrifugal pumps should be selected for automatic start and stop of pumps and automatic closure following power failure or shutdown of the units.

(ii) Structural general

The bi-plane type butterfly valve with nominal bore of 1500 mm (rated flow speed will be approx. 6 m/s) should be selected for high head in this project, which is suitable for operating system using hydraulic oil. Valve size, maximum unbalanced pressure expected at the discharge valves, and time of opening or closing the discharge valves should be suitable to operate under the most severe normal and abnormal operating conditions including emergency shutdown for water hammer protection.

Valve body- The valve body shall be designed to withstand the maximum occurring combined forces of operating cylinder or counterweight, pump suction head, pump shutoff head, and the bulkhead load imposed by the full discharge pipe line. The valve body shall be provided with a structurally adequate base and a concrete pier will support mating sole plate that.

Valve shaft-The valve shaft shall be made of carbon steel coated with stainless steel and be of sufficient size to transmit the forces imposed on the shaft by the disk, cylinder operator and counterweight without distortion or undue stress. The valve shaft shall be of the stub shaft type, which comprises two separate shafts inserted into the valve disk hubs or be combination of a separate shaft and a shaft combined with the valve disk.

Valve disk-The valve disk shall be welded plate of steel or cast steel. Valve seat-The valve seats shall be of a design that permits removal, replacement and adjustment without removing the valve from the discharge line. The valve seats shall provide tight shutoff with full discharge line pressure of 9.1 bar at maximum on the downstream face of the valve and zero bar on the upstream face of the valve. Rubber seats shall be clamped to the disk. The mating seat surface shall be 18.8 stainless steel.

Bearings-The valve shall be fitted with sleeve type. Self-lubricating bearings in the hubs of the of the valve body. The inside bearing support shall be designed to allow ready access to the stuffing box. Either two thrust bearings or a two-way thrust bearing shall be provided to insure centering of the disk.

Operator-The rated torque capability of the operator shall be sufficient to seat, unseat and hold the valve disk rigidly in any intermediate position under any operating condition.

(b) Isolating Valves

Manually and electrically operated bi-plane valve shall be selected for maintenance shutoff service and positioned at downstream side of the discharge valve. And spare parts shall be interchangeable with the discharge valve spares. Structural design of the valve shall be same as the discharge valve.

(c) Discharge Pipe Joints

The sleeve coupling or the slip-on type flange will be used to provide tightness and strength with flexibility for disassembly and assembly of the valves.

3.4.4 Structural Design of Electric

(1) Design Standards

All electrical equipment shall be designed, manufactured and tested in accordance with the latest IEC Standards (International Electro-Technical Commission) and/or the equivalent standards in the country of Manufacture. This was agreed by MED.

The following specific standards shall be used for the design of all electrical equipment.

(a) Main pump motor

IEC-34	Rotating Electrical Machine
IEC-72/72A	Dimensions and Output ratings
IEC-85	Recommendations for the classification of insulating materials in relation to their thermal stability in service.

(b) Power supply switchgear

IEC-298	A.C. Metal-enclosed switchgear and control-gear for rated voltages above 1KV
IEC-470	High-voltage alternating current contractors
IEC-632	High-voltage motor starters
IEC-694	Specification for A.C. switchgear for voltages above 1KV
IEC-439	Low-voltage switchgear and control-gear assemblies
IEC-947	Low-voltage switchgear and control-gear
IEC-76	Specifications for transformers and reactors

(2) Design Conditions

(a) Service conditions

All electrical equipment shall be suitable for operation in No.7 pumping station which is located in hot, humid and tropical atmosphere polluted by wind-drift sand.

The service conditions in the pumping station shall be as follows:

Maximum ambient air temperature:	45°C
Minimum ambient air temperature:	5°C
Maximum relative humidity:	85%
Maximum altitude above sea level:	1,000m below

(b) Features of major equipment

Features of the major electrical equipment are as described below. The detailed technical description for each item of electrical equipment is described in Chapter V, Clause 5.5, "Basic Design of Electrical Equipment".

(i) Main pump motor

Type of motor: Synchronous motor

Ratings: *13MW, 11KV, 375rpm, 50Hz

Starting mode: Auto-transformer (Kondorfer) starting

Excitation system: Brushless type (A.C exciter)

*: to be confirmed during the detailed design phase.

(ii) Main motor starting panel

Nominal voltage: 11 kV

Rated voltage: 12 kV

Rated frequency: 50 Hz

Circuit current: 1,200 A

Insulation level Impulse: 75 kV

Insulation level Power frequency: 28 kV

Breaking current: 40 kA

(iii) Exciter panel

Type: Self-supported, metal-enclosed type

Rated voltage: 380V AC, 3-phase

Rated frequency: 50 Hz

(iv) Auto-transformer

Nominal voltage: 11 kV

Rating: 3-phase, 50Hz, 3 minutes rating

Number of taps: 80%, 65% and 50%

(v) Medium and Low voltage switchgear

- Medium voltage switchgear

Nominal voltage	11 kV
Rated voltage	12 kV
Rated frequency	50 Hz
Rated current	1,200 A
Insulation level Impulse:	75 kV
Insulation level power frequency:	28 kV
Breaking current	40 kA

- Low voltage switchgear

	Load Center (1000KVA Trans.)	Motor Control Center (MCC)
Nominal voltage	380/220 V	380/220 V
Rated Insulation voltage	1000 V	1000 V
Power frequency test voltage	2.5 kV	2.5 kV
Type of system	3Ph, 4W	3Ph, 4W
Neutral grounding	Solidly	Solidly
Breaking current	50 kA	50 kA
Rated current / Bus bar	2000 A	-
For main pump MCC	-	400 A
For common A MCC	-	1200 A
For common B MCC	-	1200 A

(vi) Station batteries and charger

Batteries:

Alkaline (Ni-Cd) type 5 hours discharge rate, mounted on a steel rack and installed in a cubicle.

Battery charger:

Full wave silicone controlled rectifier type with fully automatic controls or thyristor controls.

Input power source: 3-phase, 380 V, 50 Hz

Output power source: 220 V DC

(3) Main Pump Unit Operation

(a) Location of the operation

The main pump units shall be operable from the following three positions:

(i) Main Pump Unit Starting Panel – Located in the auxiliary substation

(ii) Remote Control Panel (desk type) – Located room of the auxiliary substation

(b) Operation Mode

(i) Manual (Individual) operation mode

In this method the main pumps and the auxiliary machines (such as the discharge valves and related equipment) shall be operated individually by an operator.

(ii) Linking Operation (One-man Control)

In this method, operator operate only start or stop switch of the main pump then all auxiliary machines (such as the discharge valves and related equipment) have to be operated in accordance with the sequence linked to the start or stop order for the main pump units.

3.4.5 Structural Design of Power Supply

(1) Applicable Standards

For main equipment of the power supply system the following standards will be applied.

(a) For Main transformers (66/11 kV, 25 MVA)

IEC 60076-1 (1993) Power transformer Part 1: General
IEC 60076-2 (1993) Power transformer Part 2: Temperature rise
IEC 60076-3 (1980) Power transformer Part 3: Insulation levels and dielectric tests
IEC 60076-5 (1976) Power transformer Part 5: Ability to withstand short-circuit
IEC 60076-8 (1997) Power transformer Part 8: Application guide

(b) For Station transformer (11 kV/380-220 V, 1000 kVA)

IEC 60726 (1982) Dry-type Power transformers

(c) For 66 kV and 11 kV Circuit breakers

IEC 60056 (1987) High-voltage AC current circuit-breakers
IEC 60427 (1989) Synthetic testing of high-voltage AC current circuit breakers
IEC 61166 (1993) Guide for seismic qualification of high-voltage AC current circuit

(d) For 66 kV Disconnectors and Earthing switches

IEC 60129 (1984) AC Disconnectors and earthing switches

(e) For 66 kV Surge arresters

IEC 60099-1 (1991) Surge arresters Part 1: Non-linear resistor type gapped arresters for a.c. system

(f) For Instrument transformers

IEC 60044-1 (1996) Instrument transformers Part 1: Current transformers
IEC 60044-2 (1997) Instrument transformers Part 2: Inductive voltage transformers
IEC 60186 (1987) Voltage transformers

(g) For Relays

IEC 60255-6 (1988) Measuring relays and protection equipment

(h) For Insulators

IEC 60120 (1984) Dimensions of ball and socket couplings of string insulator units
IEC 60168 (1994) Test on indoor and outdoor post insulators of ceramic material or glass for systems with nominal voltage greater than 1000 V
IEC 60305 (1995) Insulators for overhead lines with a nominal voltage above 1000 V
IEC 60383 (1993) Insulators for overhead lines with a nominal voltage above 1000 V
IEC 60433 (1998) Insulators for overhead lines with a nominal voltage above 1000 V

(i) For 11 kV Switchgears

IEC 60694 (1996) Common specification for high-voltage switchgears and controlgears standards

(j) For Batteries

IEC 61434 (1996) Secondary cells and batteries containing alkaline or other non-acid electrolytes

(2) Design Conditions

(a) Allowable current carrying capacity of cable conductors

The allowable current carrying capacity of each cable type, conductor size and installation method taking into the design works of the power supply system is shown on the following tables.

Table 3.4-12 Heat Resistance Aluminum Alloy Stranded Cable

Cable Size	Current Carrying Capacity (A) (for 50 Hz)	
(mm^2)	Conductor temperature: 70℃	Conductor temperature: 80℃
240	383	483
400	515	661
660	687	897
850	791	1044
980	855	1136
1030	881	1173
1260	976	1312
1600	1096	1494
2020	1214	1677
2500	1321	1815

Table 3.4-13 11 kV XLPE Insulated Single core Cable (Copper Conductor)

Conductor Size (mm^2)	Installed in Cable pit (A)	Installed in Conduit (A)	Direct Buried (A)
60	225	210	245
100	345	280	325
150	440	355	410
200	525	415	480
250	605	465	540
325	710	550	625
400	805	610	695
500	920	670	785
600	1020	745	855
800	1200	865	985
1000	1440	1035	1160

Table 3.4-14 Aluminum Pipe (Aerial Conductor for 66 kV Substation)

Conductor Size (mm)	Outdoor Installation (A)	Indoor Installation (A)
50	1185	1155
60	1380	1255
80	1910	1805
100	2300	2235
120 (thick:8 mm)	3050	3035
120 (thick:12 mm)	3585	3585
140	4090	4550

Table 3.4-15 600 V XLPE Insulated 3-Core Cable (Copper Conductor)

Conductor Size (mm^2)	Installed in Cable pit(A)	Installed in Conduit (A)	Direct buried (A)
1.5	22	19	31
2.5	30	24	40
4	38	32	52
6	47	40	65

Table 3.4-16 600 V XLPE Insulated 4-Core Cable (Copper Conductor)

Conductor Size (mm^2)	Installed in Cable pit(A)	Installed in Conduit (A)	Direct buried (A)
1.5	21	17	26
2.5	30	25	34
4	36	30	44
6	47	40	57
10	63	53	78
16	81	69	99
25	104	90	128
35	125	107	157

Table 3.4-17 600 V XLPE Insulated 3-Core + Neutral Cable (Copper Conductor)

Conductor Size (mm^2)	Installed in Cable pit(A)	Installed in Conduit (A)	Direct buried (A)
35+16	125	107	157
50+25	149	129	187
70+36	183	161	229
95+50	220	196	276
120+70	251	226	313
150+70	283	258	350
185+95	321	295	395
240+120	372	346	458
300+150	420	394	516

(3) Permissible Voltage drop

Each feeder and branch circuit shall be designed to be the voltage drop of the line less than the permissible value as shown on the following table.

Table 3.4-18 Permissible Voltage Drop

Section of power supply line		Supply Voltage	Voltage drop (V)	%
From	To			
Main transformer	11 kV Switchgear	11 kV	200	2
11 kV Switchgear	Main motor Control Panel	11 kV	300	3
Aux. Transformer	Distribution board	380 V	11.4	3
Distribution board	Light fixture/Socket outlet	220 V	11	3
Aux. Transformer	Building equipment panel	380 V	11.4	3
Building equipment panel	Building equipment	380 V	11.4	3