JAPAN INTERNATIONAL COOPERATION AGENCY

CENTRO DE REHABILITACION DE MANABI (CRM)
THE REPUBLIC OF ECUADOR

THE DETAILED DESIGN STUDY ON THE WATER TRANSBASIN SCHEMES FOR CHONE-PORTOVIEJO RIVER BASINS

FINAL REPORT VOLUME IV

MAIN REPORT (ANNEX [illegible])

GEOLOGY AND CONSTRUCTION MATERIALS

MARCH 1995

NIPPON KOEI CO., LTD.
Tokyo, Japan

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FINAL REPORT
VOLUME IV

MAIN REPORT

(ANNEX 2)

4.GEOLOGY AND CONSTRUCTION MATERIALS

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FINAL REPORT

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GEOLOGY
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THE DETAILED DESIGN
ON
THE WATER TRANSBASIN SCHEMES
FOR
CHONE-PORTOVIEJO RIVER BASINS

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ABBREVIATION

Ecuadorian Institutions

CEDEGE	:	Committee for Guayas River Basin Development
CETUR	:	Ecuadorian Corporation for Tourism
CLIRSEN	:	Integrated Center for Remote Sensing Survey
CONADE	:	National Development Council
CPC	:	Chamber of Shrimp Producer
CRM	:	Manabi Rehabilitation Center
DIGMER	:	Directorate General of Merchant Marine
DINAC	:	National Directorate of Valuation and Cadastre
DINAF	:	National Directorate of Forestry
DITURIS	:	Directorate of Tourism
EMAPAM	:	Municipal Enterprise of Potable Water and Sewerage of Manta
ESPOL	:	Polytechnic Littoral College
GOE	:	Government of Ecuador
IEOS	:	Ecuadorian Institute of Sanitary Works
IERAC	:	Ecuadorian Institute for Agrarian Reform
IGM	:	Geographic Military Institute
INAMHI	:	National Institute of Meteorology and Hydrology
INEC	:	National Institute of Statistics and Census
INECEL	:	Ecuadorian Institute for Electrification
INEFAN	:	Ecuadorian Institute of Forestry and Natural Areas
INEN	:	Ecuadorian Institute of Standards
INERHI	:	Ecuadorian Institute of Water Resources
INIAP	:	Institute of Agricultural Investigations
INOCAR	:	Military Oceanographic Institute
JRH	:	Jipijapa and Pajan Board of Water Resources
MAG	:	Ministry of Agriculture and Livestock
MICIP	:	Ministry of Industry, Commerce, Integration and Fishery
MOP	:	Ministry of Public Works and Communications
PFI	:	Institutional Reinforcement Planning Unit of CRM

PHIMA	:	Integrated Water Resources Development Plan of Manabi
PMRC	:	Management Program of Coastal Resources

International or Foreign Institutions

ACI	:	American Concrete Institute
ASCE	:	American Society of Civil Engineers
ASTM	:	American Society for Testing and Materials
CAF	:	Corporación Andina de Fomento
CEPIS	:	Panamerican Center for Sanitary Engineering and the Environment
CIDIAT	:	Interamerican Center for Integrated Development of Water and Land
FAO	:	Food and Agriculture Organization of the United Nations
IDB/BID	:	Interamerican Development Bank
IEC	:	International Electrotechnical Commission
JEC	:	Japanese Electrotechnical Committee
JICA	:	Japan International Cooperation Agency
JIS	:	Japanese Industrial Standards
OAS/OEA	:	Organization of American States
OECF	:	Overseas Economic Cooperation Fund of Japan
SCS	:	Soil Conservation Service of USDA
UNDP	:	United Nations Development Program
USA	:	United States of America
USAID	:	United States Agency for International Development
USDA	:	United States Department of Agriculture
WHO	:	World Health Organization of the United Nations

Technical Terms

A.C.	:	Alternating Current
ACSR	:	Aluminum Cable Steel Reinforced
BOD	:	Biochemical Oxygen Demand
COD	:	Chemical Oxygen Demand

D.C.	:	Direct Current
DO	:	Dissolved Oxygen
EC/CE	:	Electrical Conductivity
EIA	:	Environmental Impact Assessment
EMMP	:	Environmental Management and Monitoring Plan
FEM	:	Finite Element Method
F.M.	:	Finess Modulus
F/S	:	Feasibility Study
FWL	:	Flood Water Level
GPS	:	Global Positioning System
H	:	Horizontal
HWL	:	High Water Level
IEE	:	Initial Environmental Examination
IPM	:	Integrated Pest Management
LACAT	:	Program for Warm Tropical Lakes
LWL	:	Low Water Level
MOL	:	Minimum Operating Level
NATM	:	New Austrian Tunneling Method
PLC	:	Power Line Carrier
RWL	:	Reservoir Water Level
SPT	:	Standard Penetration Test
ST	:	Station
T-N	:	Total Nitrogen
T-P	:	Total Phosphorus
TRMS	:	Transbasin and Reservoir Management System
TSS	:	Total Suspended Solid
V	:	Vertical
ZEM	:	Special Zone for Management

Economic Terms and Others

CIF	:	Cost Insurance and Freight

EIRR	:	Economic Internal Rate of Return
FC	:	Foreign Currency
FIRR	:	Financial Internal Rate of Return
FOB	:	Free on Board
GDP	:	Gross Domestic Product
GRP	:	Gross Regional Product
IVA	:	Sales Tax or Value Added Tax
LC	:	Local Currency
NGO/ONG	:	Non Governmental Organization

ABBREVIATION OF MEASURES

Length

mm	=	millimetre
cm	=	centimetre
m	=	metre
km	=	kilometre
masl	=	metre above sea level
EL.	=	elevation

Area

ha	=	hectare
m^2	=	square metre
km^2	=	square kilometre

Volume

l, lit	=	litre
Kl, Klit	=	kilolitre
l/s	=	litre per second
m^3	=	cubic metre
m^3/s, cms	=	cubic metre per second
m^3/min	=	cubic metre per minute
m^3/hr	=	cubic metre per hour
MCM, mcm	=	million cubic metre
m^3/d, cmd	=	cubic metre per day

Weight

mg	=	milligram
mg/l	=	milligram per litre
meq/l	=	milli-equivalent per litre
g	=	gram
kg	=	kilogram
t, ton	=	ton
t/y	=	ton per year
MT	=	metric ton

Time

sec	=	second
min	=	minute
hr, HR	=	hour
d	=	day
yr	=	year

Money

S/.	=	Ecuadorian Sucres
¥	=	Japanese Yen
US$	=	U. S. Dollars

Energy

Kcal	=	Kilocalorie
KW, Kw	=	kilowatt
MW, Mw	=	megawatt
KWh, Kwh	=	kilowatt-hour
GWh, Gwh	=	gigawatt-hour
V	=	volt
KV	=	kilovolt
KVA	=	kilovolt ampere
MVA	=	megavolt ampere
Hz	=	Hertz

Others

%	=	percent
°	=	degree
'	=	minute
"	=	second
°C	=	degree Celsius
MD, md	=	man-day
mil.	=	million
NO, Nos	=	number
pers.	=	person
Umho	=	micromho
ppt	=	parts per thousand
ppm	=	parts per million
ppb	=	parts per billion
l/h/d	=	litre per person per day
g/c/d	=	gram per capita per day
LS	=	lump sum
MPN	=	most probable numbers
O&M	=	Operation and Maintenance
p.a.	=	per annum
rpm	=	revolutions per minutes

Chapter 1 Geology

1.1 Introduction

Geological investigation for the detailed design of the Water Transbasin Schemes for Chone-Portoviejo River Basins has been carried out in the period from November 1993 to January 1994 for the routes of water ways, that is;

- Daule-Peripa ~ La Esperanza Diversion Tunnel to convey water from the existing Daule-Peripa reservoir to the reservoir of La Esperanza dam under construction.

- Severino Open Channel to lead water from La Esperanza reservoir to the portal of La Esperanza ~ Poza Honda Diversion Tunnel,

- La Esperanza ~ Poza Honda Diversion Tunnel to carry water to the existing Poza Honda reservoir,

- Poza Honda ~ Mancha Grande Diversion Tunnel to discharge water of the Poza Honda reservoir to the Mancha Grande river, a right bank tributary of the Portoviejo river,

- A route of transmission line to be newly constructed for power supply to the Severino pumping station.

The plan for this investigation was formulated based upon review and evaluation of all the geological and geotechnical information that had been obtained in the previous investigations for the feasibility study by cooperation of JICA (1992) and the foregoing study of a Daule Peripa ~ La Esperanza route by the Ecuadorian/Brazilian engineering team (1987). Location of the structure sites is shown in Figure 1.1.1.

The geological investigation at this stage has comprised geological mapping, core drilling, test pitting, sounding and laboratory testing.

1.2 Regional Geology

The land of Ecuador is geomorphologically classified into three north-to-south stretching zones, that is, from east to west, the Orient, the Mountain Range and the Coast.

The Mountain Range zone is represented by the Andean cordillera with the peak of elevation 6,310 m A.S.L. at Mt. Chimborazo, which is composed of Paleozoic metamorphic rocks, strongly folded Cretaceous to Tertiary volcano-sedimentary complex and young volcanic products.

The Orient zone covers a part of an extensive upper reaches of the Amazon river basin which develops on the east side of the Andes mountain ranges. The Coast zone covers the area from low foothills, west of the Andes to the Pacific coast.

The Project area is located in the Coast zone.

A major part of the Coast zone is the hill region lower than 300 m in ground height, while a mountain range, lower than 900 m, stretches in the direction of north to south within the distance of 60 km from the Pacific coast. Mountains of Moche, Mountains of Chindul and Mountains of Jama are members of this coastal low mountain range. A hill region of width from 60 to 100 km is developed between this low mountain range and the foot of the Andes, which is divided into two large basins of the Esmeraldas river flowing north and the Guayas river draining south. The Daule river and the Peripa river to provide water to the Project are tributaries of the Guayas river.

Some small rivers rise on the western slope of the low mountain range and flows west toward the Pacific Ocean. The Chone river and the Portoviejo river for the Project are rather large rivers of those on the west side of the low mountain range. They are located in the southern part of the Coast zone where the coastal low mountain ranges are lowered and fragmented.

The Coast zone is widely covered by Tertiary sedimentary rocks, whilst the basement rocks of Cretaceous sediments, partly of volcanic origin, are exposed on the coastal low mountains. The Tertiary sediments are conglomerates, sandstones and mudstones of Miocene Onzole Formation and Borbon Formation overlying the former, which stratify consistently sub-horizontal over a wide area.

It is deemed that the coastal low mountain range is a part of the outer arc of the tectonic zone stretching parallel with the inner arc of the Andes cordillera, with the subduction zone of the plate from the Pacific Ocean located off-shore the west coast.

In this context, the hill area between the coastal low mountains and the Andes is composed of sediments in an inter-arc shallow sea basin, which apparently were formed in a period of tectonic inactivity. The upper reaches of the Chone river and the Portoviejo river belong to the same sedimentary basin, even though those areas are now draining west breaking through parts of the low range. No major faulting is registered in this Tertiary beds in the geological literatures so far (Refer to Figure 1.2.1).

1.3 Geological Investigation

The previous geological investigations have revealed that the geological setting in the Project area is fairly simple with Tertiary sedimentary rocks monotonously bedding sub-horizontal with rare disturbance by folding and/or faulting.

A couple of problems have remained to be scrutinized and prepared with definite method of approach or criteria for the detailed design. Those problems are:

(1) Strength of the soft bedrock in the tunnel routes and appropriate method of tunnel excavation,

(2) Possibility of harmful behavior of rocks and earth embankment material in the form of slaking and swelling.

Other than these problems, it is required to confirm the geological and geotechnical conditions at exact locations of structures to be designed, e.g. strength of soil overburden and depth of bedrock.

The geological investigation of this stage has been planned and performed with these requirements in view. Its quantity is detailed in Tables 1.3.1 to 1.3.3, and as summarized below:

- Core drilling on the transbasin waterways : 370 m/11 locations
- Water pressure tests in the drill hole : 31 sections

- Standard penetration tests in the drill holes : 54 times

- Test pitting on the open channel and the transmission line routes : 15 locations

- Sounding by S.P.T. on the open channel and the transmission line routes and tunnel portals on Poza Honda : 61 locations

- Geological mapping of the transbasin waterway route : 4 locations

Eleven (11) drilling core samples have been tested in laboratory for the following items:

- X-ray diffraction analysis for clay minerals		
- Specific gravity and water absorption	: ASTM	C-127
- Natural density	: ASTM	C-29
- Uniaxial compression	: ASTM	D-2938/D-4543
- Static deformation modulus	: ASTM	D-4341/D-4405
- Poisson's ratio	: ASTM	D-3148
- Tensile strength (Brazilian test)	: ASTM	D-3967
- Swelling	: ASTM	D-4546
- Slaking durability	: ASTM	D-4644

The geological investigation has been carried out by the contractor, Hidrosuelos Cia. Ltda. The laboratory tests has been made in the National Politechnical University.

Location of the geological investigation are shown in Figures 1.3.1 to 1.3.6.

1.4 Site Geology

(1) Daule-Peripa ~ La Esperanza Diversion Tunnel (Refer to Figure 1.4.1)

The bedrock is composed of sub-horizontally bedding mudstones and sandstones which occur alternately. Unit sedimentary cycle of several metres to 15 m in thickness is roughly recognized in the form of a sequence of strata which is transitional from relatively coarse sandstone or fine conglomerate at the base to the finest material, i.e., mudstone, at the top.

Sequences of strata, however, are not well correlative among hole of the three core drillings, while bedrocks at all the drilling locations show similarly horizontal inter-beddings of moderately to weakly cemented sandstones and mudstones with minor intercalations of fine conglomerates. It seems that there is considerable change of grain size distribution in each bed within the distance of a few hundred metres.

There does not appear any substantial difference in lithologic features of the tunnel inlet site and the tunnel outlet site, whereas the Guayas plate of the Geological Map of Ecuador (scale 1/100,000) indicates that the inlet area is covered by Borbon Formation and the outlet site is in the underlying Onzole Formation. The fact is probably that the Onzole Formation occurs at the level of the tunnel formation in the inlet site, too.

In all of the new drillings DP93-1, DP93-2 and DP93-3 (Appendix 1, Logs of Core Drilling), similar silty fine sandstones were encountered at the tunnel formation height of elevation 60 m to 65 m. This does not always mean that all of those sandstones are of a single continuous bed. Nevertheless, it is highly probable that the most part of the tunnel passes through sandstones of varied component particle sizes, colors and fine material contents, as denoted in terms of fine sandstone, medium sandstone, silty or muddy sandstone, etc.

Sandstones including more silty or clayey fine components are generally more tightly cemented, probably because of good combination between particles. In contrast, pure sandstones without fine components appear more porous and friable.

(2) La Esperanza ~ Poza Honda Water Transbasin Route (Refer to Figures 1.4.2 and 1.4.3)

The bedrock is similarly composed of the alternating sandstones and mudstones of the Miocene Onzole Formation.

In the area of the Severino pumping station/head tank, the bedrock consists dominantly of sandstones. Mudstones appear only in the levels above elevation 100 m and at a part around elevation 100 m in the drillings SR93-2 at the substation site and SR93-5 at the head tank site. The 55 m long drilling SR93-1 at the pumping station which was drilled from elevation 96 m was through sandstones except its last 10 m section at the bottom. Some signs of cross-bedding in the sandstone in SR93-2 indicates that those sandstone beds have been formed by deposition near a sea shore. Shell fragments are contained in various levels of the sandstones (Appendix 1, Logs of Core Drilling).

The 6,300 m long Severino open channel route is on the similar bedrock with overburden thinner than 3 m in the most parts.

The La Esperanza ~ Poza Honda diversion tunnel route is in a thick and probably continuous sandy mudstone bed developing below elevation 120 m, whereas the tunnel formation height is elevation 107 m at the inlet and elevation 100 m at the outlet. Weathering is intensive and rather deep-reaching on both ends of the tunnel, with mudstones completely weathered to a condition of residual soil to the depth of 6 m to 8 m. Weathering can be more intensive and deeper in the higher part of the ridge above the tunnel route.

(3) Poza Honda ~ Mancha Grande Diversion Tunnel (Refer to Figure 1.4.4)

The tunnel route passes from the Poza Honda reservoir on the Portoviejo river to the Mancha Grande river, its right bank tributary, across a ridge with a peak at elevation 450 m. Similar to the other tunnel route, the upper part of the ridge, higher than a level around elevation 180 m, is occupied by sediments of Miocene Borbon Formation, whilst the lower part including the tunnel level is compose of sandstones and mudstones with minor intercalations of small grained conglomerates of the Miocene Onzole Formation.

The bedrock around the tunnel formation height is of dominantly finer components, i.e., mudstones and muddy (silty) fine sandstones which are moderately cemented or compact.

The rock is intensely weathered up to considerable depths that is not common in the other routes. Complete weathering of rock into residual soil was observed developing to the depth of 13.5 m in the drilling MG93-1 on the Poza Honda lake and 18 m in MG93-2 on the left bank slope of the Mancha Grande river (Refer to Figure 1.4.4 and Appendix 1).

The drilling MG93-3 on the Mancha Grande river bank did not encounter the bedrock till the depth of 18.8 m, or elevation 76.20 m, that is by approximately 15 m lower than the river bed.

The upper part of this deposit, 5 m in thickness and brown colored, is the recent alluvial deposit which now forms a sort of river terrace. The underlying 14 m section is occupied with grey colored silt, sand and their mixtures containing rock fragments of sandstone and mudstone origin.

The thick deposit between the bedrock surface and the river bed is deemed a sort of old colluvial deposit brought by mud flow or landsliding. The present topography of a large concave slope on the higher part (higher than the existing car road) on the left bank of the Mancha Grande may be a remnant of ancient landslide surface, though all the slopes to the river are completely stabilized at the present time as indicated by all upright trees on those slopes.

(4) Picoaza Quarry Site (Refer to Figure 1.2.1)

Basalt is quarried by private companies in low mountains between the towns of Picoaza and Montecristi. The mountains are a part of the coastal low mountain range composed of coffee-colored lutite of the Dos Bocas Member of the Oligocene to Miocene Tosagua Formation, which is stratigraphically situated lower than the Onzole Formation in the water transbasin route and covers lavas of Mesozoic Piñon Formation. The basalt in those quarries is of limited exposures to the ground surface of the Piñon Formation. Even though regionally limited, the reserve that can be quarried is sufficient for covering quantity of the project requirement.

1.5 Engineering Geology

1.5.1 Tunnels

The feasibility study has proposed the tunneling by mechanical cuttings, instead of blasting, and 10 to 15 cm thick shotcreting with wire mesh and rock bolting at 1.5 m intervals. This strategy is deemed basically appropriate in the light of findings in the geological investigations at the present stage, though the figures and criteria may need to be reviewed.

The example of tunneling the abutment galleries at La Esperanza dam, now under construction, shows that the method of blasting and steel supporting is also successfully applicable to the Onzole Formation. Shotcrete is also applied together with the steel support. The method by mechanical cutting and shotcreting, however, would be more efficient for the substantially long tunnels of the water transbasin.

As mentioned in Section 1.3, some of the sandstones with little silt and clay component are porous and friable for the lack of force to combine component particles. The sandstone of this sort can be liable to lose stability or collapse by saturating with groundwater in the periphery of the tunnel cave. Although occurrence of such sandstone is limited, it may require, under the worst condition, to provide drainage holes drilled into the bedrock surrounding the tunnel, preceding the progress of the tunneling.

In general, the soft bedrocks of the tunnel route can be categorized to the rock of medium quality at Class II or III of Bieniawskiis Rock Mass Rating (RMR), with the point ranging from 53 to 68 (Refer to Table 1.5.1). This evaluation is probably appropriate for all bedrocks of the tunnel routes except for weak parts of Class IV in the Poza Honda - Mancha Grande tunnel and rather limited parts of local disturbances or the very friable sandstone as mentioned above. Locations of those local weak sections at the tunnel level are not identifiable at the present moment. Minor adjustment to the local poor rock conditions will have to be made to meet the actual circumstances encountered in the course of the tunneling.

It seems, however, that Bienieawskiis criteria for the resistance against shearing of the Class III rock are, empirically judging, too conservative for the bedrock of this area. Criteria, applicable with safety, are 5 kgf/cm^2 for cohesion and 40 degrees for internal angle of friction. For the friable sandstone, the cohesion of 3 kgf/cm^2 shall be taken.

The tunnel cave is to be stabilized by unifying rocks over the tunnel periphery with shotcreting with wire mesh and rock bolting immediately after excavation. Drill holes for drainage will be required.

Steel supporting is not to be rejected. Sections within a hundred metres from the tunnel portals and other sections of very weak rock condition could require heavy steel supporting with close intervals up to 0.75 m.

The feasibility study has appropriately proposed the concrete lining of 30 cm in thickness with and without reinforcing bars depending upon the rock conditions.

It is duly conservative and necessary to apply concrete lining, considering that the bedrock is generally soft and vulnerable to erosion. Even though covered and protected by the shotcrete, there could still be a possibility of rock deterioration in a long time due to internal erosion or piping by seepage water through cracks which may widen the openings inside the bedrock to cause reduction of strength and loss of stability. It is also possible that the shotcrete may not be permanently intact under influence of uneven deformation of the soft rocks. Furthermore, some of the rock samples tested in laboratory showed tendency of swelling with rather low pressures less than 1.5 kgf/cm^2 under the condition of no volume change. There were two exceptional cases of 6.5 kgf/cm^2 for a sample from DP93-2 and 1.7 kgf/cm^2 for SR93-3 (Refer to Table 1.5.2). The former high pressure is for a sample of mudstone at the level of the tunnel, and could be influential upon the tunnel structure. The latter is still fairly low. The concrete lining will be an adequate and reliable solution to cope with all those problems.

The swelling is deemed due to clay minerals of the montmorillonate group (smectite) included in the rock. The swelling pressure of 1.5 kgf/cm^2 referred above is equivalent with 15 m high water head, which are supported in many cases by lining concrete of tunnels deep under the ground water table. Material pressure of 6.5 kgf/cm^2 seems uncommon and would require replacement with concrete when it causes visible damages.

Geotechnical design criteria for the tunnels to be proposed are shown in Table 1.5.3.

The geotechnical circumstances of each tunnel are as follows:

(1) Daule-Peripa-La Esperanza Diversion Tunnel (Refer to Figure 1.5.1)

Core drillings at five locations, including those in the previous investigation campaign, indicate that the tunnel route at elevation 65 m to 60 m will for the most part be in the compact silty or muddy fine sandstones which are of the common rock type of this area. They fall under Class C_M of a Japanese standard or Class II to III of Bieniawski (Refer to Tables 1.5.4 and 1.5.5).

Out of three drilling core samples taken at levels near the tunnel formation height for laboratory test, one from the drill hole DP93-1 near the outlet portal is exactly from the silty fine sandstone to cover the tunnel route. A sample from DP93-2 is of a mudstone bed in the tunnel formation level. The other is from mudstones within 5 m reaches of the tunnel level.

Deformation modulus of 10,000 kgf/cm^2 and elastic modulus of 20,000 kgf/cm^2 is to be taken as the design criteria, while the laboratory test has given a value around 12,350 kgf/cm^2 for a sandstone sample and a mudstone sample, and 21,000 kgf/cm^2 to the other mudstone (Refer to Table 1.5.6).

Poisson's ratio for the design shall be 0.20, as against 0.11 for the sandstone and 0.20 to 0.25 for the mudstones in the laboratory test.

Uniaxial compressive strength is 50 kgf/cm^2. All the samples indicate values around that (Refer to Table 1.5.6).

Swelling pressure index was zero for the sandstone, while 1.2 kgf/cm^2 for the mudstone in DP93-3. The latter is a substantial but low value. The mudstone in DP93-2 showed exceptionally high pressure of 6 kgf/cm^2.

Slaking durability index was noticeably low 0.1 percent in one of the mudstone samples, one from DP93-2, showing a strong tendency to slake. The other two samples of sandstone and mudstone had high durabilities.

It seems probable that the problem of slaking and swelling is not likely to be encountered in the silty sandstone bed which will surround the Daule Peripa-La Esperanza diversion tunnel.

The friable sandstone beds have been met by drilling at two locations. In the drilling DP93-3 at the inlet portal, it has been found in a section between elevation 79.5 m and elevation 76.2 m, which will have no effect upon the tunnel at elevation 65 m. In the drilling DP93-1 near the outlet, the friable sandstone bed has been located below elevation 60 m, immediately under the tunnel at elevation 60.6 m. The weak sandstone below the invert of tunnel, however, will not cause any substantial difficulty.

The field permeability tests around the tunnel formation height in the drill holes gave varied values. The friable sandstone at the bottom of the hole DP93-1 showed Lugeon unit of 26 and abrupt increment in the water injection rate at the pressure of 5 kgf/cm^2. This appears to be a sign of a local breakage or deformation of the friable sandstone. High permeabilities of 35 Lugeon and 60 Lugeon were observed in sandstone beds in the depths less than 20 m. In the silty sandstone beds of the tunnel level, however, the highest value observed was 15 Lugeon unit at one section in DP93-3 and the others were not more than 5 Lugeon unit.

The Daule Peripa ~ La Esperanza diversion tunnel will pass under the frequently undulating hills. The tunnel will have the thickest rock cover of 275 m under the highest part of the hill at the distance of 4,500 m from the inlet. In the other hand, it will have to pass under thin rock covers of 18 m and 16 m at the stream of Cañales at the distance of 2,100 m and the stream of Murato at the distance of 7,900 m from the inlet, respectively. Weathering does not reach deep in this area and the rocks are virtually not pervious, as indicated by the results of the permeability tests not showing more than 5 Lugeon unit at DP93-1 and DP93-2. The seepage conditions could be not seriously difficult there. Nevertheless, heavy steel supporting has to be envisaged for the tunnel to pass through these parts, in view of the possibility of overall weakening of the rock due to even slow seepage from the valley bottoms.

(2) La Esperanza ~ Poza Honda Diversion Tunnel (Refer to Figure 1.5.2)

The drilling SR93-3 at the inlet of the La Esperanza ~ Poza Honda diversion tunnel at Caña Dulce revealed that the tunnel at elevation 103.7 m will be located in a thick compact sandy mudstone bed. The similar sandy mudstone was found in the drilling SR93-4 at the outlet at Los Cuyuyes on the Poza Honda reservoir (Refer to Appendix 1).

These sandy mudstones on both ends of the tunnel are probably continuous to surround almost all the section of this tunnel. In the hole SR93-4 at the outlet, however, the tunnel formation height at elevation 100.2 m coincides the bottom of the sandy mudstone, below which lies another compact bed of silty fine sandstone. The rock is tight and moderately cemented to be classified to C_M to C_L of a Japanese standard and Class III of Bieniawskiís RMR. The rock in the outlet side is comparatively weaker than that in the inlet side, possibly because of deeper weathering in the former area.

Permeability of the rocks, observed in the field permeability test, was all higher than 40 Lugeon unit, except one section lower than the tunnel level at the inlet (SR93-3) which showed less than one Lugeon. In the section of 25 m to 30 m of depth at the outlet (SR93-4), the water injection rate had been almost zero while the pressure had been increased from 1 to 6 kgf/cm^2, but it abruptly rose up to 180 litres every minute at the pressure of 10 kgf/cm^2. The water-take remained high even after the pressure was reduced. This is deemed to indicate irreversible opening of water passage(s) in the rock by the pressured water (Refer to Appendix 2, Lugeon Test, Pressure-Injection Rate Graph, Table 1.5.7 and Figure 1.5.3).

Deformation modulus and elasticity modulus for the design are to be assumed at 10,000 kgf/cm^2 and 20,000 kgf/cm^2, respectively, similar to the case of the Daule-Peripa diversion tunnel. For the weathered rock zone near the tunnel portals, the values of a half the above figures are to be taken.

Poisson's ratio for design shall be 0.20, while the laboratory testing has given 0.21 and 0.18.

Uniaxial compressive strength gotten by the test was 64 kgf/cm^2 for the drilling core sample of the outlet portal.

Swelling and slaking tendencies are not substantial.

The bedrock of this tunnel can be deemed to fall under the same range of mechanical characteristics as that of the Daule-Peripa ~ La Esperanza diversion tunnel. No unfavorable geological member as the friable sandstone were not encountered near the tunnel formation level.

The hill covering the tunnel route is generally high. There is a thin rock cover of 22 m at one location of 11,030 m from the inlet, or only 400 m from the outlet. Considering the generally deep-reaching weathering in the outlet zone, the provisions of steel supports at close intervals and deep drainage holes for the rocks in the tunnel arch and side wall will need to be contemplated in the design for crossing under this thin rock cover.

(3) Poza Honda - Mancha Grande Diversion Tunnel (Refer to Figure 1.5.4)

This tunnel passes under a massive hill of 360 m in height, without location of thin rock cover. The bedrock encountered at the tunnel level of elevation 91.3 m by the drilling MG93-1 at the inlet on the Poza Honda reservoir is a 3 m thick compact muddy fine sandstone, intercalated between moderately cemented mudstone beds. The bedrock of the tunnel level at the outlet is mudstone. It seems that the thin sandstone bed at the inlet is a minor member which could pinch out within a short distance and the main bed along the tunnel route can be mudstone at large.

The bedrock is generally weaker than those in the other tunnel route, which might be partly because of the intensive weathering developing far deeper than that in the other routes. The rocks at the tunnel level, however, are not visibly weathered.

Permeability of the rock, observed in the field test in the drill hole MG93-1 at the outlet, is high with Lugeon units of 40 to 70. The patterns of increment in the water-take (water injection rate) as against the increment of pressure indicate that the rock has been fractured or some cracks have been opened irreversibly by the water pressure of 4 kgf/cm^2 to 6 kgf/cm^2. In the hole MG93-2 at the outlet, the permeability is lower but still at the magnitude of 18 Lugeon unit. Sign of rock deformation or fracturing under the water pressure is not clear at the outlet.

The drilling MG93-3 passed a thick slope deposit and reached to the bedrock surface at a level 10 m lower than the contemplated tunnel level. The location of this hole, therefore, is off the tunnel outlet. This hole gives information of the rock line development for the design of the tunnel outlet.

Static deformation modulus from the laboratory test was 7,730 kgf/cm^2 at the inlet and 6,100 kgf/cm^2 at the outlet, ruling out 1,900 kgf/cm^2 of the sample from MG93-3. The design value for mudstone of this tunnel route shall be 7,000 kgf/cm^2 for the deformation modulus and 12,000 kgf/cm^2 for the elasticity modulus. For the weathered rock zone near the portal, the values are to be reduced to 5,000 kgf/cm^2 and 10,000 kgf/cm^2.

Poisson's ratio for the design is 0.25.

Uniaxial compressive strength is not more than 40 kgf/cm^2.

No serious swelling has registered. Some slaking could take place, which, however, will be suppressed with shotcreting with wire mesh worked immediately after excavation.

1.5.2 Severino Pumping Station and Head Tank

The Severino pumping station to pump up water from La Esperanza reservoir to the head tank for the Severino open channel will be located at the top of a steep slope on the right bank of the Severino river, approximately 1.5 km upstream of Severino village. The Severino river is a tributary of the Barro river on which the dam of La Esperanza is being constructed.

Main structure of the Severino pumping station is a pumping shaft to be excavated in the rock at a location near the steep slope. An intake adit will be driven from the surface of the slope to be connected to the shaft.

The drilling SR93-1 shows that the shaft will be for the most part in sandstones of varied grain sizes and in mudstone only below the depth of 45 m or elevation 51 m. The bedrock is compact or moderately cemented, except for the sandstone between 14 m and 22 m of depth which is porous and friable. Considering that the groundwater table is as deep as 35 m from the ground surface or at elevation 61 m, the friable sandstone, kept dry, will not be dangerously deteriorated or weakened in the course of the excavation work. The overburden of colluvial deposit is only 1.5 m in thickness (Refer to Appendix 1).

The head tank to start the open channel will be located at elevation around 111 m in the higher part of a gentle slope above the pumping station. According to the drilling SR93-5, colluvial deposit of 3.55 m in thickness, composed of brown clayey soil containing rock debris, covers the ground surface. This deposit is underlain by a bed of completely weathered mudstone, which, though integrated into hard soil, has sufficient strength to support the structures of head tank, with test hammer blows of 22 to 56 for 30 cm of standard penetration. Sandstone and mudstone, only slightly weathered or fresh, lie under the completely weathered mudstone (Refer to Appendix 1).

The borehole permeability test in the drilling SR93-5, in muddy fine sandstone at the level immediately below the base of the head tank and the open channel, showed very low permeability of 1.7×10^{-7} cm/sec or less than 1 Lugeon unit under the pressure of 1.0 kgf/cm^2, whereas it gave 5.0×10^{-4} cm/sec or 40 Lugeon unit when the pressure was raised to 4.0 kgf/cm^2. Some cracks were forced to open or parts of the rock were fractured by the rise of pressure. For the head tank the water pressure effected at its bottom will not be higher than 1 kgf/cm^2 and will not cause serious leakage problem. Permeability of the sandstone/mudstone beds in the deeper zone turned out to be more tight against water seepage in permeability tests (Refer to Table 1.5.7, Lugeon Test Data and Appendix 2).

The pipe line to connect the pumping station to the head tank will also be able to put its foundation on bedrock under thin overburden which is strong enough to support it, even if intensively weathered. In the Test pit C-10 nearby, the overburden was only 1.4 m.

1.5.3 Severino Open Channel (Refer to Figure 1.5.5)

The Severino open channel is a canal with width of about 8 m at the top and depth of 3 m, laid out on the right bank slope of the Severino river. It leads the water pumped up from La Esperanza reservoir to the inlet of the La Esperanza-Poza Honda diversion tunnel at Caña Dulce. Its length will be approximately 6,300 m and its level will range from elevation 111 m to elevation 107 m. Closed waterways for syphon are involved at five locations to cross gullies.

The canal route was investigated with test pitting at ten locations and sounding by standard penetration test at 54 locations (Refer to Appendices 4 and 5). Soil/rock samples (disturbed) were taken from the test pits for laboratory testing of embankment material for the canal, of which outcome will be presented in the report on construction material. These investigation were intended to succeed and reinforce the previous investigations for the feasibility study with test pitting at nine locations and a core drilling at Syphon No.3.

The investigations revealed that the cover of colluvial deposit, composed of particles of varied sizes ranging from clay to rock debris, is not thicker than 5 m. Weathered bedrock was encountered within the depth of 3 m at 53 locations out of 64 where the pitting or the sounding were made. The bedrock was reached within 2 m of depth at 61 percent of those locations.

The overburden is generally thin in the open channel route, except for alluvial deposits in the bottom of gullies to be crossed by syphons. The drilling B2 for the feasibility study had found a 9.2 m thick alluvial deposit in the gully of Estero La Chontilla for Syphon No.3 (Refer to Figures 1.5.6 and 1.5.7).

A major part of the canal will be put in the bedrock. No difficulty is seen in the mechanical aspect of the foundation. Even the colluvial deposit can be expected to have sufficient strength to support the canal structure. Foundation condition for seepage or water leakage could differ for locations because the bedrock near the ground surface cannot be always homogeneous for differential weathering and slacking. The bottom and sides of the channel should be artificially covered by blanket or concrete.

A problem lies in swelling and slaking characteristics of the rock and soil, for both foundation and construction material.

The laboratory testing of samples from the test pits registered the swelling tendency classified as of medium magnitude as represented in the swelling pressure index over 1.5 kg/cm^2 for eight samples out of 27 including those in the stage of the feasibility study. The other samples indicated low swelling tendency. Even the medium index values shown by six out of those eight samples were not more than 1.65 kgf/cm^2, close to criterion for the low swelling tendency.

The slaking in the form of quick deterioration of rock under repeated dry and wet conditions usually occurs only in the superficial zone, e.g., within 30 cm depth, and does not develop deep underground. This sort of slaking can be suppressed by covering the surface with earth material or concrete to prevent contact of the rock with atmosphere.

The tendency of deterioration by swelling and slaking is originated by clay minerals of the montmorillonite (smectite) group contained in some members of the bedrock, as indicated by the X-ray diffraction analysis (Refer to Table 1.5.8 and Appendix 3).

Those unfavorable characteristics of the rock and soil will require duly cautious approach. Meanwhile, it will not be appropriately economical to apply excessively conservative measures beyond the practical necessity of maintaining its function, to this sort of water channel, specially when those unfavorable tendencies are generally low in magnitude or limited in locality. Practically, minor swelling or slaking on the surface of low cut slope will not be seriously damaging to the canal.

For the embankment material, this soil characteristics would also raise a question, while any alternative material, free from these characteristics, is not available within a reasonable distance, or in the extensive geological region of the Onzole and Borbon Formations. It is recommended to use the material obtained near the canal route with a sort of trial embankment of real size in the initial stage of the construction work and with adjustment of the design to meet the actual conditions encountered. Revision of the design, if necessitated, will be minor, and far more economical and realistic than an alternative of providing other material from very distant sources.

1.5.4 Transmission Line

The Severino substation at the end of the transmission line was investigated for the foundation condition by the core drilling SR93-2 (Refer to Appendix 1). Colluvial deposit of clayey silt with rock debris, covering the ground surface, is 2.5 m thick. The bedrock is composed of a 4 m thick sub-horizontal weathered mudstone bed and a series of sandstones underlying it. Groundwater table was met at the depth of 3.5 m.

The weathered mudstone is compact and has obviously sufficient strength to support the structures and machines of the substation of transmission line.

The colluvial deposit also is not weak soil, though the values of 23 of the standard penetration test at the depth of one metre may partly reflect effects of the contained rock debris. It can have allowable bearing strength of 4 to 6 tons every square m, if conservatively estimating. With the reliable bedrock surface at the depth of 2.5 m, however, it will be sound practice to sink the base of the important structures to reach the weathered rock surface.

Geological conditions were examined for a few important tower foundations of the transmission line, with test pitting at five locations and sounding at four locations. The tower sites in this hill region are located at such high places as tops of hills or ridges. The soils at those sites are fairly tight and strong. The bedrocks are also reached within the depth of 3 m, except only one location C-24 on a low hill between the Daule river and the Tachel river (Refer to figures 1.3.6, Appendices 4 and 5).

The geological conditions are practically similar to that of the Severino substation. The tower foundation will be placed on the bedrock which, weathered or not, is strong enough, or on the strong soil. Any soft soil as to require pile foundation will not be encountered. No problem is envisaged for the foundations of the transmission line.

1.6 Seismicity

With the subduction zone of the plate moving from the Pacific Ocean which is located in 150 km of distance off-shore, the Project area falls under a part of the circum-Pacific high seismicity zone. The given conditions as follows, however, need to be considered to determine an appropriate value for the design seismicity or the design earthquake factor:

(1) No major fault has been recorded in the area of the Miocene Onzole/Borbon Formations. This may indicate that this area of the Coast zone is in a relatively stable part in the Andean tectonic belt. The lack of major faults will lead, at least, to rule out possibility of active faults and shallow earthquakes.

(2) The Project has no large structure, like dam, which may cause disastrous damage upon the public on its destruction.

(3) Effect of earthquake is far less upon the underground structures like the tunnels than upon the structures on the ground surface.

For design of the surface structures, such as the embankment, it will be sufficiently safe and duly economical to assume an earthquake acceleration value of the higher range currently used for high seismicity zones, e.g., 0.15 g.

1.7 Conclusion

The tunnels have to be driven through soft rock composed mainly of sandstone and mudstone of Miocene Onzole Formation, which are sub-horizontally bedded and tectonically not much disturbed. Shotcreting with wire mesh immediately after excavation and rock bolting will be the effective measures for protection of these tunnels, but steel supporting will be also required to pass through highly weathered or deteriorated rock zone near the portals and at the crossings under thin rock covers of deep gullies. Concrete lining is deemed indispensable for these soft rocks, vulnerable to erosion, as the final and permanent work.

The tunneling in soft rock is not very easy and requires careful approaches, e.g., provision of drainage holes by drilling to its periphery or in advance to progress of tunneling. Nevertheless, troubles, if any, can be overcome by ordinary countermeasures used in tunneling work. No seriously difficult problem is envisaged in the tunnel construction.

The tendency of swelling and slaking in soil and some rock beds is due to clay minerals of the montmorillonite group included in the bedrock. This characteristic, therefore, prevails over all kinds of geotechnical material in the extensive region which the Onzole and Borbon Formations cover. It seems that replacement with other better material is neither practical nor duly economical, unless it is absolutely essential for safety of any important structure.

Fortunately, the swelling tendency is generally low, and found only in some rock samples. The slaking can be suppressed by preventing the rock from making direct contact with atmosphere. Those tendencies can be coped with by shotcreting and concrete lining in the tunnels. They will not substantially harmful on the low cut-slopes on the Severino open channel.

The Severino pumping station will for the most part be encased in a thick compact sandstone. A bed of friable sandstone also will remain compact because of very low groundwater table at its location close to a steep slope.

For foundations of the head tank, the open channel, the Severino sub-station and the transmission line, the bedrock, even if weathered, will be reached within 3 m of depth in the most parts, and will have no strength problem of the foundation. For the base of the open channel, the irregularly weathered rock foundation will require an artificial coating for seepage cut-off.

Chapter 2 Construction Materials

2.1 Purpose of the Construction Material Investigation

The Construction materials investigation was carried out to ascertain the quality, available quantity of the materials to be used for the Project and covered the following items.

(1) Concrete Aggregates Investigation

Considering the site condition and the lack of ready mixed concrete in the Project area, concrete aggregates will have to be obtained for its execution. Prospective sources of sand and gravel for concrete aggregate were surveyed and samples were sent to the Contractor's laboratory in Quito for various quality tests.

(2) Concrete Trial Mixing Tests

In order to ensure that concrete made from the above materials would have the required quality, trial mixing tests were carried out at a laboratory in Quito.

(3) Soil Material Tests

As recommended by the Final Feasibility Study Report drawn up in December, 1992, further soil tests were carried out. Soil samples were taken from the test pits and sent to the laboratory to ascertain the basic characteristics of the soil to be used for the embankment of Severino open channel.

2.2 Investigation Period

Investigation period is as follows;

(1)	Concrete Aggregates	:	16 November 1993	-	31 January 1994
(2)	Concrete Trial Mixing	:	3 January 1994	-	31 January 1994
(3)	Soil Materials	:	16 November 1993	-	31 January 1994

2.3 Concrete Aggregate Investigation

(1) Object of the investigation

The investigations was carried out in order to clarify the points with regard to sand and gravel to be used for concrete aggregate.

a) Potential sources of sand and gravel for concrete aggregate.
b) Available quantity of each material source.
c) Quality of sand and gravel obtained from each material source.
d) Distance from the quarry and borrow areas to the respective job site and road conditions on the way there.

(2) Location of prospective Material Sources

The survey was conducted to identify the possible sources of fine and coarse aggregates to be used for concrete structure, access road, etc.

The Prospective sources of sand and gravel are as shown in Figure 2.3.1.

a) Sand quarry/borrow areas are located at:

- Río San Pablo, Quevedo: 170 km east of Portoviejo
- Cantera Basaltica Picoazá: 16 km west of Portoviejo
- San Jacinto: 54 km north west of Portoviejo

b) Gravel quarry/river deposits are located at:

- Río San Pablo, Quevedo: 170 km east of Portoviejo
- Quarry around Picoazá area:
  - Cantera Basaltica Picoazá: 16 km west of Portoviejo
  - San Carlos/Río de Oro: 14 km west of Portoviejo
  - Carlos Poggi: 15 km west of Portoviejo

(3) Required Quantities of Aggregate

As the estimated total volume of concrete structures and shotcrete in accordance with the design is approximately 230,000 m^3. Quantities of aggregate are assumed as follows:

Fine aggregates: 230,000 m^3 x 0.4m^3/m^3 of concrete = 92,000 m^3

Coarse aggregates: 230,000 m^3 x 0.75m^3/m^3 of concrete = 173,000 m^3

(4) Test Items and Quantities

The required sampling and laboratory tests for fine and coarse aggregates were carried out as follows.

Sampling and Laboratory Test Quantities

Type of Aggregate & Location of Quarry	Fine Aggregate			Coarse Aggregate	
Name of test	Quevedo **M.O.P	Picoazá ***C.B.P.	San Jacinto	Picoazá S.Carlos	*Quevedo M.O.P.
I. Sampling					
(1) Physical Test	1	1	1	3	-
(2) Concrete Mixing	1	1	1	3	-
II. Physical Test					
(1) Grain Size Analysis	1	1	1	3	-
(2) Specific Gravity and Water Absorption (fine)	1	1	1	-	-
(3) Specific Gravity and Water Absorption (coarse)	-	-	-	3	-
(4) Organic Impurity (fine)	1	1	1	-	-
(5) Soundness with Sodium Sulphate	-	-	-	3	-
(6) Los Angeles Abrasion	-	-	-	6	-
(7) Alkali Reactivity	1	1	1	1	-

Note : * Required sizes of coarse aggregate were not available.
** M.O.P.: Ministerio de Obras Públicas
*** C.B.P.: Cantera Basáltica Picoazá

(5) Sampling

Samplings of sand were taken at Río San Pablo, in Quevedo (river deposit), Cantera Basáltica Picoazá Quarry, Picoazá, (crushed sand using rod mill), and San Jacinto (beach sand). Sample of gravel was taken at San Carlos, Picoazá. Gravel at Quevedo was not used as the gravel size did not conform with the requirements. The locations of the borrow areas and quarries are shown in Figure 2.3.1.

Two alternative sources of sand and rock materials were considered for the concrete aggregates in the Feasibility Study.

(i) Existing quarry at Picoazá. Good quality basalt in sufficient quantity at a distance of 16 km from Portoviejo.
(ii) River deposit at Quevedo. Good quality sand and gravel originating from andesite in sufficient quantity at a distance of 170 km.

The sand and gravel borrow area and quarry site mentioned above have been investigated to ascertain whether it is possible to get the required quantities and quality for the aggregate materials From the results of the investigation, it has been concluded that material from Picoazá meets the requirements.

(6) Fine Aggregate

A site reconnaissance and availability survey were carried out in the vicinity of the Project area as well as in the environs of the city of Portoviejo in accordance with description of the materials survey report in the feasibility study in 1992.

(A) Sources of fine aggregate

Possible sources of sand (grain size under 5 mm) for fine aggregate were considered as follows:

(a) Quevedo, Río San Pablo

The sand materials is available as a deposit in the Río San Pablo river near Quevedo town. The area of the deposit is approximately 100 ha, which is enough to provide the total quantity of fine aggregate required. The concession for it belongs to MOP (Ministerio de Obras Publicas). It is andesite sand. The location of the borrow area is 170 km from Portoviejo, as shown in Figure 2.3.1.

(b) Picoazá, Cantera Basáltica Picoaza

Crushed sand is produced here using rod mill with a capacity of 30 ton/hr. Small size aggregates crushed by the secondary crusher are fed to the rod mill, crushed, screened and washed using a wet type spiral classifier to control fineness modulus of the sand. It is basalt sand. At the present time, almost all of the sand produced here is supplied to La Esperanza Dam Project as fine aggregate for concrete structure. As the concession area for this quarry is more than 200 ha., it can produce the total quantity of fine aggregate required. The location of this quarry is shown in Figures 2.3.1 and 2.3.2, and 16 km from Portoviejo city.

(c) San Jacinto at Manta Beach

In consideration of concrete quality, beach sand is normally unsuitable because it contains salt. However, as quite a lot of sand for concrete aggregate used in Portoviejo City and its vicinity is from San Jacinto, Crucita, San Clemente Manta Beach, a test sample was taken from these areas because they are considered as possible sources of the materials. As sand is available over more than 10 km along the beach, the quantity of fine aggregate there is considered to be sufficient. The locations are as shown in Figure 2.3.1. The distance is 54 km from Portoviejo.

(B) Quantities of fine aggregate sampled are as follows.

(i)	Quevedo	:	100 kg
(ii)	Picoazá	:	100 kg
(iii)	San Jacinto	:	100 kg

(C) Results of laboratory tests of sand

Physical test of the sand were carried out to find a possible source of sand, samples were taken at the places mentioned above. These samples were sent to the laboratory in Quito. The results of the Laboratory tests on them are listed below and are shown in Figure 2.3.3.

(a) Sieve analysis of sand (ASTM C-136)

Sieve Size		Percentage Retained (individual)		
mm	No.	Quevedo	Picoazá	San Jacinto
10	3/8"	0.00	0.00	0.00
5	# 4	0.38	0.28	0.00
2.5	# 8	1.28	12.27	0.00
1.2	# 16	3.85	34.19	0.00
0.6	# 30	10.90	49.42	0.00
0.3	# 50	57.69	72.42	7.71
0.15	# 100	91.67	96.07	92.78
F.M.*		1.66	2.65	1.00

* : Fineness Modulus

When compared with the Standard gradation for fine aggregate shown below, it can be seen that the gradations of the samples from Quevedo and San Jacinto are too fine as fine aggregate.

Standard Gradation of Fine Aggregate

Designation of fine aggregate (mm)		Standard percentage by weight passing individual sieve (%)
10	3/8"	100
5	# 4	95 - 100
2.5	# 8	80 - 100
1.2	# 16	50 - 90
0.6	# 30	25 - 65
0.3	# 50	10 - 35
0.15	#100	2 - 10

(b) Specific gravity and absorption of sand (ASTM C-128)

	Material Sources		
Test Items	Quevedo	Picoazá	San Jacinto
- Specific gravity	2.732	2.560	2.110
	2.752	2.552	2.206
- Absorption (%)	2.25	3.80	1.16
	2.19	3.59	1.26

Apart from the sand sampled at the San Jacinto, Specific gravities of the sands shown in the above table are more than 2.5 which satisfy the requirement.

The absorption of the samples ranged from 1.16% to 3.80%. The absorption of Picoazá sand is slightly larger than the requirement (less than 3.5) mentioned in Japan Industrial Standard, but the use of this sand will not affect the quality of concrete.

(c) Organic impurities of sand (ASTM C-40)

	Material Sources		
Test Item	Quevedo	Picoazá	San Jacinto
Color indication	Dark yellow	Transparent	Transparent

The criteria of different colors is listed below.

Color	Percent decrease in concrete strength	Evaluation
Transparent	0	Good
Dark yellow	10 - 20	Usable
Red yellow	15 - 30	Limited
Light red brown	25 - 50	Not usable
Dark red brown	50 - 100	Not usable

Source: Civil Construction Materials, M. Kokubu, University of Tokyo

The above test result shows that Quevedo sand contains a lot of organic impurities which will affect a quality of concrete.

(d) Alkali reactivity test (ASTM C-289)

The potential reactivity of the aggregate with alkalies in portland cement concrete was tested using the chemical method.

From the test results shown in the table below and Figure 2.3.4, the aggregate is considered innocuous.

Quarry Site	Quantity Sc Reduction in Alkalinity (millimoles/litre)	Quantity Rc Dissolved Silica (millimoles/litre)
Quevedo	33.30	354.28
	34.63	338.65
	34.96	270.92
Basáltica	51.62	161.51
	67.93	119.83
	59.27	265.71
San Jacinto	17.98	505.37
	18.65	406.38
	22.64	343.86

(7) Coarse Aggregate

(A) Sources of coarse aggregate

The prospective aggregate sources in the Final Feasibility Study Report are as follows.

- Existing quarry at Picoazá: Good quality basalt aggregate in sufficient quantity at a distance of 16 km from Portoviejo.

- River deposit at Quevedo: Good quality andesite aggregate in sufficient quantity at a distance of 170 km from Portoviejo.

The site reconnaissance and availability investigation for coarse aggregate were carried out concurrently with the survey for fine aggregate. At San Pablo in Quevedo, river deposit of andesite cobbles is available, and a crushing/screening plant is being operated by MOP (Ministerio de Obras Públicas) there. However as only small size aggregate for asphalt pavement (maximum size: 13 mm) is produced here, sampling was considered unnecessary.

In the Picoazá area, there are four aggregate quarries operated by three firms. The rock in all of them is basalt. The production capacity (more than 150 ton/hr) and the quarry concession area (more than 100 ha.) are considered sufficient to supply concrete aggregate for the Project. The distance from the quarries to Portoviejo is 14 - 16 km.

As a result of field investigation, sampling was carried out at "San Carlos/Río de Oro" quarry and the samples were sent to the laboratory in Quito for aggregate and concrete trial mixing tests.

The quantities of coarse aggregate sampled were as follows.

-	G1:	5 - 15 mm	(1/2")	: 200 kg
-	G2:	15 - 20 mm	(3/4")	: 200 kg
-	G3:	20 - 40 mm	(1-1/2")	: 150 kg

(B) Laboratory test of coarse aggregate

Suitable aggregate for concrete is composed essentially of clean, uncoated, properly shaped particles of strong, durable material. The results of the laboratory tests carried out in the laboratory are shown below.

(a) Grain size analysis (ASTM C-136)

Sieve tests were carried out and the result is shown below and in Figure 2.3.5.

Sieve Size		Percentage retained (individual)		
mm	No.	G1 5 - 15 mm	G2 25 - 15 mm	G3 40 - 25 mm
50	2"	0.00	0.00	0.00
40	1 - 1/2"	0.00	0.00	0.00
25	1"	0.00	1.41	59.98
20	3/4"	1.55	57.20	93.67
15	1/2"	42.79	96.79	96.88
10	3/8"	67.40	98.24	97.60
5	# 4	94.24	98.98	98.85
* F.M.		2.06	3.53	4.47

Note: * Fineness Modulus

Standard gradation of the coarse aggregate is shown below.

Standard Gradation of Coarse Aggregate

Designation		Standard percentage by weight passing individual sieve (%)		
mm	No.	G1 5 - 15 mm	G2 25 - 15 mm	G3 40 - 25 mm
50	2"	100		-
40	1 - 1/2"	95 - 100		-
30		-	100	-
25	1"	-	95 - 100	100
20	3/4"	35 - 70	-	90 - 100
15	1/2"	-	30 - 70	-
10	3/8"	10 - 30	-	20 - 55
5	# 4	0 - 5	0 - 10	0 - 10
2.5	# 8	-	0 - 5	0 - 5

(b) Specific gravity and water absorption (ASTM C-127)

Specific gravity and absorption tests were carried out as routine testing, principally because of their importance in concrete mix design. Aggregates of higher specific gravity are generally more satisfactory with respect to soundness and strength.

Absorption values of higher than 3.5% indicates poor quality but this does not necessarily mean that such aggregates should be rejected.

Specific Gravity

Sample	G1	G2	G3
No.1	2.81	2.79	2.71
No.2	2.74	2.74	2.88

* Required Specific Gravity: more than 2.5

The test result shows that the specific gravities of all samples were more than 2.5 which is acceptable.

* Concrete manual, U.S. Department of the Interior, Bureau of Reclamation.

Water Absorption (%)

Sample	G1	G2	G3
No.1	4.71	3.52	2.77
No.2	4.49	3.41	3.04

* Required Value for Water Absorption: less than 3.5 %

The water absorptions were slightly higher than the required value. However, it is considered that this will not affect the quality of concrete too much.

* Concrete manual, U.S. Department of the Interior, Bureau of Reclamation.

(c) Soundness (ASTM C-88)

The sodium sulfate soundness test provides an indication of any structural weakness present in an aggregate. Aggregate samples are considered acceptable if the weighted loss of gravel is less than 10 percent after 5 years.

Soundness %

G1	G2	G3
5.59	6.29	5.59

* Required value: less than 10 %

The test result shows satisfactory value.

* Concrete manual, U.S. Department of the Interior, Bureau of Reclamation.

(d) Los Angeles abrasion (ASTM C-131)

The Los-Angeles abrasion test gives valuable information on the hardness and toughness of an aggregate. The requirement is that the coarse aggregate should not lose more than 10% and 40% after 100 and 500 revolutions, respectively.

Los Angeles Abrasion (%)

Sample	G1	G2	G3
No.1	17.00	16.40	16.10
No.2	17.80	17.00	16.70

* Required value: less than 40 %

The test results show satisfactory values

* Concrete manual, U.S. Department of the Interior, Bureau of Reclamation.

(e) Alkali reactivity test (ASTM C-289)

The potential reactivity of the coarse aggregate with alkalies in Portland cement concrete was tested by the chemical method.

The test results are shown in the below table.

Samples	Quantity Sc Reduction in Alkalinity milliomoles/liter	Quantity Rc Dissolved Silica millimoles/liter
	65.31	138.41
G1	59.42	145.72
	60.21	156.21
	68.30	122.31
G2	67.41	131.24
	52.21	145.36
	51.72	139.58
G3	59.43	148.34
	63.24	157.25

The test results indicate that all the samples are innocuous in terms of potential alkali reactivity, as shown in Figure 2.3.4.

2.4 Trial Concrete Mixing

(1) General

Concrete mixing test was carried out to estimate an adequate mix proportion of water, aggregate and admixture in accordance with specified conditions of maximum size of coarse aggregate, design compressive strength and slump, etc.

(2) Conditions of concrete mixing test

The design condition of the concrete mixing test was tentatively determined as follows.

Type of Concrete	Max.Size of Aggregate (mm)	Design Compressive Strength σ 28 (kgf/cm^2)	Slump (cm)
A - 1	25	300	10 - 14
A - 2	25	210	10 - 14
B - 1	40	300	8 - 12
B - 2	40	240	8 - 12
B - 3	40	210	8 - 12
B - 4	40	170	8 - 12

(3) Mixing materials

The following materials were used for the concrete mixing test.

a)	Water	:	Clean water
b)	Cement	:	Portland cement, type 1E (Cemento Rocafuerte)
c)	Fine aggregate	:	- Quevedo quarry; river sand - Picoaza quarry; crushed sand - San Jacinto; beach sand
d)	Coarse aggregate	:	San Carlos; quarry gravel
e)	Water reducing and AE agents	:	Airbeton, Resin Vinsol

(4) Concrete Mixing Test

The mix proportions for the concrete were as determined on the basis of the Basic Mix Proportions for Concrete Mixing Test Table in the Technical Specifications, adjusting the mixes in accordance with the procedure in ACI 318-83.

(A) Target strength and water - cement ratio

Target strength applied for concrete mixing test was the same as that for the design compressive strength.

Basically, the compressive strength of concrete depends on the water-cement ratio. The assumed compressive strength at 28 days is obtained using the following formula.

$f28 = -113 + 214\ C/W$ (Japanese Cement Technical Institute)

where, $f28$: Compressive strength of concrete at 28 days (kgf/cm^2)

C : Weight of cement (kg)

W : Weight of water (kg)

Mix Proportion Strength and Water - Cement Ratio

Type of Concrete	Design Strength f(kgf/cm^2)	Target Strength f 28 (kgf/cm^2)	Water-Cement Ratio W/C (%)
A - 1	300	300	52
A - 2	210	210	66
B - 1	300	300	52
B - 2	240	240	61
B - 3	210	210	66
B - 4	170	170	76

(B) Mix proportion for concrete mixing test

Based on approximate values of unit bulk volume of concrete aggregate, sand-aggregate ratio and unit water content of concrete, the basic mix proportions for the concrete mixing test were determined as shown below.

Basic Mix Proportion for Concrete Mixing Test

Type of Concrete	Max. Size of Coarse Aggr. (mm)	Slump (cm)	Air Content (%)	W/C (%)	S/A (%)
A - 1	25	10 - 14	5.0	52	37
A - 2	25	10 - 14	5.0	66	40
B - 1	40	8 - 12	4.5	52	33
B - 2	40	8 - 12	4.5	61	35
B - 3	40	8 - 12	4.5	66	36
B - 4	40	8 - 12	4.5	76	38

Weight (kg)

Water	Cement	Sand	G1 5-15 mm	G2 15-25 mm	G3 25-40 mm
147	283	699	584	584	-
165	250	748	550	551	-
147	283	628	417	417	417
150	246	674	409	410	410
151	229	697	405	405	406
155	204	740	395	395	395

(C) Slump test

The slump test for the above mix proportions was made in accordance with ASTM C-143.

The adjustment of the mix proportions was carried out in accordance with the following table. (Source: U.S. Department of Interior, Bureau of Reclamation, Concrete manual)

Item	Correction of S/A (%)	Correction of W (kg)
For every 0.1 increase (decrease) in F.M	Increase (decrease) by 0.5 percentage point.	No correction
For every 1 cm increase (decrease) in Slump	No correction	Increase (decrease) by 1.2%
For every 1 percentage point increase (decrease) in air content	Decrease (increase) by 0.5 - 1.0 percentage point	Decrease (increase) by 3%
For every 0.05 increase (decrease) in water-cement ratio	Increase (decrease) by 1.0 percentage point.	No correction
For every 1 percentage point increase (decrease) in s/a	---	Increase (decrease) by 1.5 kg
When using crushed sand	Increase by 3 to 5 percentage point	Increase by 6 to 9 kg

The value of unit volume of coarse aggregate is to be decreased (increased) by 1 percent point for every 0.1 increase (decrease) in the Fineness Modulus (F.M.) of sand.

(D) Preparation of specimen for compressive strength test

Six test specimens, for three different tests, were made for each type of concrete (A-1, A-2, B-1, B-2, B-3, B-4). A total of 108 specimens was thus prepared and cured in accordance with ASTM C-192. The cylinder mold for making the specimens was 15 cm in diameter and 30 cm in height.

(E) Compressive strength test

After curing the test specimens for 7 and 28 days respectively, compressive strength tests were carried out in accordance with ASTM C-39 at the Contractor's laboratory in Quito.

The testing apparatus was a hydraulically operated compression machine (Controls - Milano, Italy) having a maximum capacity of 3000 KN.

The compressive strength test results for each type of concrete are shown in Figures 2.4.1to 2.4.6.

Test samples:

Test-1:	18 Nos. Test cylinders using "Quevedo sand" for f7
	18 Nos. Test cylinders using "Quevedo sand" for f28
Test-2:	18 Nos. Test cylinders using "Picoaza sand" for f7
	18 Nos. Test cylinders using "Picoaza sand" for f28
Test-3:	18 Nos. Test cylinders using "San Jacinto sand" for f7
	18 Nos. Test cylinders using "San Jacinto sand" for f28

Test Result for the concrete specimens at the age of 28 days are shown below.

(a) Test-1: Strength at the age of 28 days using "Quevedo sand"

Test-1 Conc. Type	No. 1 fkg/cm^2	No. 2 fkg/cm^2	No. 3 fkg/cm^2	Average strength fkg/cm^2	Target strength fkg/cm^2
A1	245	168	163	192	300
A2	110	104	109	108	210
B1	201	204	148	184	300
B2	145	145	144	145	240
B3	136	126	134	132	210
B4	76	83	87	82	170

Note: * F.M. of the sand: 1.66

(b) Test-2: Strength at 28 days using "Picoaza sand"

Test-2 Conc. Type	No. 1 fkg/cm^2	No. 2 fkg/cm^2	No. 3 fkg/cm^2	Average strength fkg/cm^2	Target strength fkg/cm^2
A1	168	154	167	163.0	300
A2	134	138	145	139.0	210
B1	226	196	214	212.0	300
B2	143	143	144	143.3	240
B3	124	123	128	125.0	210
B4	86	78	85	83.0	170

Note: * F.M. of the sand: 2.65

(c) Test-3: Strength at 28 days using "San Jacinto sand"

Test-3 Conc. Type	No. 1 fkg/cm^2	No. 2 fkg/cm^2	No. 3 fkg/cm^2	Average strength fkg/cm^2	Target strength fkg/cm^2
A1	190	192	192	191	300
A2	109	107	107	108	210
B1	194	190	179	188	300
B2	124	136	129	130	240
B3	95	94	91	94	210
B4	61	94	94	62	170

Note: * F.M. of the sand: 1.00

2.5 Soil Materials Tests

(1) General

The results of the several soil tests in the Final Feasibility Study Report of December, 1992, show that the rock types in the area of the open channel are clay-silt and weathered rock. It concluded that the clay-silt along the open channel shows critical swelling and which will affect the concrete lining, the use of this material is not suitable for the embankment because of the shrinkage anticipated. The report recommended that further detailed soil investigation should be performed in D/D stage.

So, as recommended the above, soil materials to be used for the embankment of Severino open channel at the test pits and these samples were then sent to Quito for laboratory testing.

Samples were taken of the soil materials to be used for the embankment of Severino open channel at the test pits and these samples were then sent to Quito for the laboratory testing.

(2) Location of test pits and sampling

Test pits were dug along Severino open channel to take samples of the materials to be used for embankment filling in order to investigate their characteristics. Test pits were also dug along the transmission line to take samples with the purpose of obtaining parameters for the design of tower foundations.

The locations of the test pits are shown in Figures 2.4.7 and 2.4.8. There were 10 test pits for the Severino open channel and 5 for the transmission line.

Along Severino open channel, 20 disturbed samples were taken, 2 for each pit, one at the depth of between 0 to 2 m and the other taken at the depth of between 2 to 5 m.

(3) Laboratory test

The following laboratory tests were carried out on the samples taken from the pits to determine the characteristics of the soils.

- Natural Moisture Content	:	20 Nos. ASTM D-2216
- Specific Gravity	:	20 Nos. ASTM D-854
- Unit Weight	:	10 Nos. ASTM C-29
- Grain Size Analysis	:	20 Nos. ASTM D-422
- Atterberg Limit	:	20 Nos. ASTM D-423 and D-424
- Uniaxial Compression	:	10 Nos. ASTM D-2166
- Triaxial Compression (UU)	:	10 Nos. ASTM D-2850
- Consolidation	:	10 Nos. ASTM D-2435
- Proctor Compaction	:	15 Nos. ASTM D-698
- Swelling	:	20 Nos. ASTM D-4546
- Shrinkage	:	20 Nos. ASTM D-427

- Pin-hole test : 20 Nos. ASTM D-4647

(4) Result of the Test

The results of the tests are summarized in the table as shown in Table 2.4.1.

(A) Natural moisture content

The tests for natural moisture content were carried out in accordance with ASTM D-2216. The test results are shown in Table 2.4.2.

(B) Specific gravity

The specific gravities tests were carried out in accordance with ASTM D-854. The results for the natural moisture content and specific gravity tests are shown below in Table 2.4.2.

(C) Unit weight

The tests were carried out in accordance with ASTM C-29, and the results are as shown in Table 2.4.3.

(D) Grain size analysis

The tests were carried out in accordance with ASTM D-422, in order to obtain a quantitative assessment of the distribution of particle sizes in the soil samples.

The results of the test are shown in Appendix 10.

(E) Atterberg limits

The tests were carried out in accordance with ASTM D-423 as well ad D-424, to obtain consistency limits for the soils, such as liquid limit, plastic limit and plasticity index.

The test results are shown in Table 2.4.4.

(F) Uniaxial compression test (unconfined compression test)

The tests were carried out in accordance with ASTM D-2166 to obtain the unconfined compressive strength. The results for the uniaxial compression test are shown in Table 2.4.5, and test data are shown in Appendix 6.

(G) Triaxial compression test

The tests were carried out in accordance with ASTM D-2850 adopting unconsolidated-undrained test (UU test)

The results for the triaxial compression tests are shown in Table 2.4.5 and test data are shown in Appendix 7.

(H) Consolidation

Consolidation tests were carried out in accordance with ASTM D-2435, "One-dimensional Consolidation Properties of Soils", to determining the rate and degree of soil consolidation when it is being restrained laterally and loaded and drained axially.

The test results are shown in Table 2.4.6 and test data are shown in Appendix 8.

(I) Proctor compaction

Proctor compaction tests were carried out in accordance with ASTM D-698, "Moisture-density Relations of Soils and Soil-aggregate Mixture using 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop", in order to obtain the relationship between moisture content and density of the soils.

The test results are shown in Table 2.4.7 and test data are shown in Appendix 9.

(J) Swelling

Swelling tests were carried out in accordance with ASTM D-4546, "One-dimensional Swell or Settlement of Cohesive Soils" in order to determine the magnitude of swell of compacted cohesive soil at zero volume change.

The test results are shown in Table 2.4.8.

Criteria for swelling pressure are as follows;

Swelling pressure	* Criteria
Less than 15 t/m2	Low
15 - 25	Medium
25 - 100	High
More than 100	Very high

* Source of the criteria: Fu Hua Chen, Foundation on Expansive Soil" Developments in Geotechnical Engineering.

(K) Shrinkage

Shrinkage tests were carried out in accordance with ASTM D-427 "Shrinkage Factors of Soils" which are shrinkage limit, linear shrinkage, and volumatic shrinkage. The test results are shown in Table 2.4.9.

Criteria for volume change in shrinkage factors are as follows.

Volume change (%)	Criteria
Less than 5	Good
5 - 10	Medium
10 - 15	Poor
More than 15	Very poor

Note: Source of criteria: Kogler and Scheidig Baugrund und Bauwerk

(L) Pin-hole test

Pin-hole tests were carried out in accordance with ASTM D-4647. This test is a qualitative measurement of the dispersibility of clay-silt soils. The test results are shown in Table 2.4.10.

Dispersion is classified as follows; (ASTM D-4647)

Class	Degree	Description
D1	Very dispersive	Dispersive clay that fail rapidly under a 50 mm head.
D2	Highly	
ND4	Moderate	Slightly to moderately dispersive clay that erode slowly under 50 mm or 180 mm head.
ND3	Slightly	
ND2	Very slightly	Nondispersive clay with very slightly to no colloidal under 50 mm or 1020 mm head.
ND1	Non dispersive	

2.6 Conclusion and Recommendations

(1) Concrete Aggregate investigations

The investigations were carried out with a view to finding potential sources of aggregate, available quality, quantity of material, and transportation distance.

(A) Fine materials

(a) Quevedo, Rio San Pablo

This river deposit can supply a sufficient quantity of fine aggregate for the project, but, the quality of the material was found to be unsatisfactory in the laboratory tests because it contained organic impurities. Further, the transport distance from the project site is as far as 170 km.

(b) Picoaza, Cantera Basaltica Picoaza

Considering the quality of the sand, quantity available, supply capacity and transport distance, this quarry is one of the best sources of fine aggregate for the Project.

(c) San Jacinto, Manta Beach

The quality is sub-standard due to the large fine material content and considerable contamination by salt water. If this material is used for the purpose of adjusting the fineness modulus by mixing it with the Picoaza sand mentioned above, thorough washing with clean water will be necessary.

(B) Coarse aggregate

In the Picoaza area, there are four aggregate quarries operated by three mining companies. The quarried material is the same. The quality of the aggregate is acceptable for use in concrete, according to the test report. However, a test for materials finer than the 75 mm sieve (ASTM C-117) is recommended for further investigation.

Sufficient coarse aggregate can be obtained from these quarries.

(2) Concrete Mixing Test

In the tests, no specimen achieved the target compressive strength at 28 days. It is considered that this test result was caused by following reasons.

(a) Water-cement ratio for concrete mixing test was set based on target compressive strength of Portland Cement Type I. However, Portland Cement Type IE was used for the test.

(b) According to the past experience in the case of Portland Cement Type IE is used for concrete mixing, compressive strength of concrete will be about 80 % of Type I. However, test results show that all the compressive strength of test specimens are less than 80 % of the target value. It was due to water-cement ratio was increased more than designated one for adjusting the target slump.

During the Phase 2 Study in Ecuador, some additional investigations are made mainly for concrete mix, which was not satisfactorily resulted in the Phase 1 Study. Concrete mix test results are collected from the report on the construction of the Conguillo inlet structure executed in 1989-1990 and from the La Esperanza dam construction under way. These existing concrete mix data indicate that more than 300 kg of cement should be used to produce 1.0 m^3 of concrete with a compressive strength of 250 kg/cm^2. In the La Esperanza dam construction, structural pumpable concrete with maximum size of aggregate of 25 mm contains 380 kg/m^3 of cement to obtain average 28-day strength of 290 kg/cm^2 with a guaranteed strength of 253 kg/cm^2.

(3) Soil Materials

The results of the soil tests show that the soil materials are composed of fine particles (under 75 mm) from weathered mudstone, classified as MH or CH, containing ML and CL in parts. Generally, suitable materials for embankments requires to have good compactibility, low shrinkage, and only a small decrease in shear strength when the water content is increased.

The "Standard Design and Construction, Japan Road Association" and "Earth Manual, U.S. Bureau of Reclamation" recommends the materials for embankment fill have the following characteristics:-

- Maximum particle size : 100 mm
- Percentage by weight passing 4.76 mm sieve : 25 - 50 %
- Percentage by weight passing 75 mm sieve : 0 - 25 %
- Plastic limit : under 10

The test results show that almost none of the samples fulfill the above requirements and the following conclusions were made.

a) The soil has high compressive characteristics. The proctor compaction test indicated that the percentage of the optimum moisture content is very high (31 - 46 %) and gd max is very low (1.06 - 1.39 gf/cm^3).

b) The values of the swelling pressure are slightly high (2 - 25 tf/m^2), which will have a possibility to affect the concrete lining.

c) The volume change due to shrinkage is very high (25 - 57 %). There will thus be the possibility of slaking if the soil is dry and submerged repeatedly not only to the embankment fill but also the cutting surface of the waterway.

Considering the above, these soil materials are unsuitable for the embankments, but if proper and careful countermeasures are taken as described in this ANNEX, it is not necessary to reject these materials totally.