FINAL REPORT

VOLUME III　SUPPORTING REPORT

H. FACILITIES PLAN

STUDY
ON
INTEGRATED WATER RESOURCES DEVELOPMENT
IN THE CAÑETE RIVER BASIN
IN
THE REPUBLIC OF PERU

FINAL REPORT
VOLUME III
SUPPORTING REPORT

H: Facilities Plan

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Chapter 1 General Information about Peru

1.1 Land and Population

Peru, a Latin American country, with a coastal length of about 2 100 kms. on the Pacific Ocean, has an area of 1 285 000 s.q. km. and the population of the country is about 22 million. The great majority of the population lives on the coastal line or quite close to it, with only 2% of the overall population living to the east of the Andes. The population of Lima, the capital city of Peru, is about 7.5 million, housing about one third of the population of the whole country.

1.2 Runoff of the Pacific Slope of the Andes

The mean annual volume of runoff of the Pacific slope of the Andes is about 36,000 MCM and about 1/3 of it is used in agricultural activities.

Chapter 2 Previous Studies

2.1 Chronological Order of Studies

2.1.1 Studies by Panedile Peruana S.A.

The first study on the Cañete Basin was made in 1955 by Panedile Peruana S.A. with the consultancy of the Edison Milan Company of Italy. The purpose of the study was to meet the agricultural requirements of the Cañete Valley and it principally concentrated on the supply of water to an area of 26 000 ha. in the Pampas of Concon - Topara - Chincha Alta.

The study came up with the proposal of damming of 11 lakes located in the upper basin and the transfer of water from Rio Cochas, a tributary of Rio Mantaro.

2.1.2 Studies by Electricite de France

In 1956, a study was carried out for the Government of Peru by Electricite de France on the national electrification of the country. This study recommended the development of the El Platanal Hydropower Project with the regulation of the river runoff mainly in the headwaters of the basin.

2.1.3 Studies by Motor Columbus

The third study was made by Motor Columbus INC. of Switzerland for Electrolima in 1966 and it was on the "Hydroelectric Development in the Valley of River Cañete". This study came up with the recommendation of the development of two large hydroelectric schemes. The regulation of the river runoff was recommended through the damming of the lakes in the headwaters of the basin. The transfer of water from Rio Cochas, a tributary of Rio Mantaro, was also proposed in this study.

2.1.4 Studies by Lahmeyer - Salzgitter Consortium

In 1978, a consortium of Lahmeyer and Salzgitter, commissioned by the Ministry of Energy and Mines for evaluating the national hydroelectric potential of the country studied the Cañete River Basin recommending a series of stages for hydropower development in the basin.

2.1.5 Studies by Motor Columbus and Electrowatt

One of the most recent studies on the potential of Rio Cañete is the one made by a consortium of Motor Columbus and Electrowatt of Switzerland and three local consulting companies. This study made for Electro-Peru in 1987, recommended the El Platanal Hydropower Project with the intake weir at a locality called Chavin. The regulation of the river runoff was proposed to be made through the damming of some of the lakes in the upper basin. One of the most predominant remarks in this study is the indirect implication of the potential landslip problems in the basin. Another important remark made in the report is about the difficulty of using the secondary energy of the hydropower projects. This is a factor which is quite important in the determination of the installed capacity of the hydropower schemes in countries like Peru.

2.1.6 Study by a Local Consultant.

The latest study on Rio Cañete is the one made in 1995 for Sedapal by a local consulting company. This study analyzed the possibility of transfer of water to Lima for domestic and industrial use. The discharge considered was 10 m^3/sec and the regulation of the river runoff was reported, referring to the 1987 study made for Electro-Peru, to be provided by the damming of a number of lakes in the upper basin, in stages, in conformity with the increase in the domestic water demand. This study analyzed two routes for the transfer of water to Lima, these routes are called a) the mountainous route and b) the coastal route. The proposal in the study was the mountainous route due to its costwise advantages in comparison with the coastal one.

2.1.7 Study by Binnie and Partners on Mantaro Transfer Scheme

A study was made by “Binnie and Partners” of United Kingdom in 1981 for the Ministry of Housing and Construction of Peru on the possibility of transfer of water from the Upper Catchment of Rio Mantaro to Lima. The study came up with the proposal of diverting water at Atacayan in Rio Mantaro, pumping it to an intermediate reservoir at Carispacha from where it would again be pumped to the existing Marcapomacocha reservoir suitably enlarged. Flows regulated in the Marcapomacocha reservoir would flow via the existing Trans-Andean Conveyance Conduit to the headwaters of Rio Santa Eulalia, a tributary of Rio Rimac lying on the Pacific slope of the Andes. The study proposed the development of the ‘Project’ in two stages with a transfer of 16 m^3/sec in the first stage reaching 35 $m^{3/}$sec at the final stage of final stage of development.

2.1.8 Study on Mantaro Transfer Scheme by a Local Consultant

The Mantaro Transfer Scheme proposed by Binnie and Partners has not been implemented. One of the main reasons for it is that the population of Lima has not grown to the extent as forecasted in the population projection studies. It is understood tat after 1990 Sedapal, Water and Sewerage Administration of Lima, has undertaken studies to supply domestic water to Lima from various sources. One of these studies was made by a local consulting company on the Mantaro

Transfer scheme. This study is in fact a modification of the 1981 “Binnie and Partners” study where the main difference is the elimination of one of the pumping stages proposed in the 1981 study. It is understood that the study also considered the developments which took place between 1981 and 1995.

2.2. Involvement of Cementos Lima for the Development of Water Resources of Rio Cañete.

Cementos Lima is a major cement producer based in Lima. This private company is involved in the development of the water resources of Rio Cañete for hydropower development and transfer of water to the nearby basin to irrigate a 27 000 ha of land in the Concon-Topara-Chincha Alta area. This area, for the time being, is a desert and no agricultural activities can be carried out.

The hydropower project recommended in the 1987 study made for Electro-Peru is called the El Platanal Hydropower Project. Cementos Lima has shifted the intake site of this project further upstream to a site called Capillucas. The riverbed elevation at the site is 1 521.50 m.a.s.l. based on actual measurements. The height of the dam above the riverbed is reported to be 37 m. The power system consists of a 1.25 km. long power conduit and a 12.15 km long power tunnel. The head available in this project is about 650 m.

The storage capacity of Capillucas Dam is very small so as to have any contribution to the regulation of the river runoff. The regulation of river runoff which is required both for the irrigation of the Concon-Topara-Chincha Alta area and secondly for the Capillucas Powerhouse has been considered by Cementos Lima, in the beginning, to be provided by the damming of the Paucarcocha Lake and a high dam at Morro de Arica site. The total effective storage capacities considered at those sites were 55MCM and about 175 MCM for Paucarcocha and Morro de Arica Dams, respectively.

The riverbed elevation at Morro de Arica site is about 2 785 m.a.s.l. and the total storage capacity of the reservoir at elevation 2 987 m.a.s.l. is 205.3 MCM. This dam, located at a very narrow gorge has been considered by Cementos Lima mainly for regulation of the river runoff. However, they have also come up with a powerhouse at the toe of the dam using the diversion tunnel as the power tunnel. The cost of the power facilities is marginal with respect to the benefits from it. It appears that they want to lower the minimum operation level as much as possible within the operation range of turbines. The dam will also be equipped with a bottom outlet to release water downstream in exceptionally dry years when the water level drops below the level for the operation of the powerhouse.

The studies of Cementos Lima are still in progress. The designs they have for both of the dams are at a preliminary basis. Besides, an extensive geotechnical investigation is in progress at Morro de Arica site consisting of audits driven at both banks. The recent approach of Cementos Lima is to increase the full supply level of Morro de Arica Dam by another 10 m so as to have a total storage capacity of about 235 MCM out of which 200 MCM could be used as the effective storage capacity.

As for Paucarcocha Dam, the previous studies have come up with a total net storage capacity of 55 MCM above the long term lake level. The reservoir capacity recommended is about the size to store the inflows between January and April which is the rainy period in all the basin. Recommended reservoir operation policy appears to be to release the water downstream at a constant rate during the rest of the year. This appears to be a reasonable approach since the river runoff drops dramatically in the dry period.

It is understood that Cementos Lima aims at obtaining the storage capacity required, with Morro de Arica Dam only. They want to increase the dam height by another 10m so as to gain another 30 MCM of storage capacity. The policy of Cementos Lima is understood to have a sufficient storage capacity so as to have an acceptable amount of deficit of water for irrigation for both agricultural lands, the Cañete Valley and the Concon-Topara-Chincha Alta area in exceptionally dry periods. This is a remark worth to be made.

The debris material at the abutments of the Morro de Arica reservoir seems to be relatively less compared to the downstream reaches of the basin. As for the Capillucas dam site, the right bank of the reservoir just to the upstream of the dam axis is covered with debris in the upstream-downstream direction. Capillucas Dam is provided with radial gates at riverbed level to release downstream monthly or seasonally, the sediment that will accumulate in the reservoir area. These gates will also be used as spillway gates.

The depth of alluvium at Morro de Arica site is rather shallow. On the other hand it is more than 70 m at Capillucas damsite.

Capillucas Dam, a 37 m high concrete gravity dam, has been designed so as to rest on concrete replacing alluvium for a depth of 7m. This concrete extends both to the upstream and downstream of the concrete gravity dam to avoid a shear failure of the alluvial foundation material.

The installed capacities of the Morro de Arica and Capillucas powerhouses due to recent information are 46MW and 220 MW, respectively.

Chapter 3 Existing Facilities in the Basin

3.1 Irrigation Facilities

At present there is an area in the Cañete Valley irrigated for a long time with water diverted by an intake structure at Nuevo Imperial. The total area irrigated presently is in the order of 24,000 ha. It is probable that there is a deficit of water for these farmlands only in exceptionally dry years.

3.2 Hydropower Facilities

There exists a small scale hydropower plant about 5 km. to the upstream of the Morro de Arica Damsite. The plant discharge of this powerplant is 1.25 m^3/sec and exploits a gross head of 100m through a head pond. The installed capacity of this plant is about 1000 kW. This plant operating as a base plant to generate hydropower for a nearby mining facility will be inundated upon the impounding of the Morro de Arica Dam. Water to the headpond is conveyed by a rectangular crosssection canal following the intake structure which is about 5 kms to the upstream of the powerhouse. The desilting basin is a few hundred meters to the downstream of the intake structure.

Chapter 4 Scope of the Study

4.1 General

The scope of the study is mainly to determine the possibility and approximate cost of transfer of water to Lima for domestic and industrial use to cover part of the deficit to arise after 2022. The magnitude of this water demand is 5 m3/sec. The other demands for water is the domestic water demand of the Cañete Corridor and the irrigation water demand of the Pampas Altas de Imperial area which would be developed for agricultural activities with the supply of water. This land has a total area of 2 475 ha.

The alternative scheme to meet the domestic water demand of Lima after 2022 is the Marcapomacocha-Carispacha scheme which is mainly a water transfer project from the Atlantic slope of the Peruvian Andes to its Pacific slope in the headwaters of Rio Rimac. The two alternatives will be compared for technical and economic considerations.

4.2 Total Water Demand

The annual water demand in the basin for irrigation and domestic water is as given below.

1.- The Cañete Valley	340.20 MCM
2.- Pampas Altas De Imperial	30.17 MCM
3.- Pampas De Concon-Topara	351.41 MCM
4.- Domestic Water Demand (Q = 6.5 m^3/sec.)	205.00 MCM
	926.78 MCM

The total demand in the basin would be 1,062.38 MCM with consideration of Q=4.3 m^3/sec for the maintenance flow in Rio Cañete to the downstream of Nuevo Imperial.

4.2.1 Demand Pattern

The demand for domestic water is a constant value all throughout the year. On the other hand, irrigation water demand is variable throughout the year with higher values occurring in summertime when the runoff of Rio Cañete has the highest values. The monthly demand values for irrigation and domestic water are given below in a tabulated form.

Monthly Demand in m^3/Sec

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Cañete Val.	19.19	22.27	17.73	12.38	3.77	4.01	4.09	4.33	5.72	7.55	12.49	16.75
Pampas De Altas De Imperial	1.33	1.68	1.32	0.98	0.63	0.62	0.59	0.57	0.55	0.81	1.19	1.25
Concon-Topara and Chincha Alta	15.21	19.45	15.20	11.21	6.80	7.40	6.97	6.37	5.73	9.14	14.15	16.72
Domestic Water Demand	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50
Total	42.23	49.90	40.75	31.07	17.70	18.53	18.15	17.77	18.50	24.00	34.33	41.22

4.3 Water Potential in the Basin

The mean annual discharge in Rio Cañete based on measurements at the most downstream site, Socsi, is 51.03 m3/sec for an observation period between January 1985 and December 1997. In fact, the observation period of the runoff gauging station at Socsi is much longer dating back to 1925. The mean discharge at Socsi corresponds to an annual water potential of 1,609 MCM.

4.3.1 Inflow Pattern of Rio Cañete

Inflow pattern of Rio Cañete if very irregular with a greater percentage of the runoff occurring in January to April. Following are the mean monthly discharges in m3/sec for Socsi site covering the period between 1986 and 1997.

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Aver.
88.83	137.86	147.75	72.46	38.24	20.73	15.03	12.11	10.31	11.74	21.23	36.03	51.03

About 73% of the mean annual runoff occurs in the first four months of the year, this percentage is 61% for the first three months of the year which is the rainy season. The above table indicates clearly that the inflow pattern is quite irregular.

4.4 Requirements for Additional Storage Dams

The irrigation water demand of the already irrigated Cañete Valley and the Concon-Topara-Chincha Alta area to be promoted by Cementos Lima will be supplied through the regulation of the river runoff in the reservoir of Morro de Arica Dam and the inflow from the subbasin between Morro de Arica Dam and Socsi. The additional demands due to the domestic water requirement of Lima and the irrigation water demand of Pampas Altas de Imperial make it necessary to have additional storage dams.

4.5 Possible Dam Sites.

The development of water resources in a basin for hydropower development or for water supply either for domestic purposes or for irrigation would have different approaches depending on the water potential of the basin and the topographical conditions.

The topographical conditions of Rio Cañete, ignoring the geological conditions, provide a good source for hydropower generation. The head that could be utilized for hydropower generation is the head between the full supply level of Morro de Arica Dam and the tailwater level of Capillucas Powerhouse. This head is roughly 3 000 - 900 = 2,100m. Other than that, there are narrow gorges which have been considered as potential dam sites in this study. These are Paruco and Calachota sites which are located between Morro de Arica site and Auco site. The problems in the reservoir area of Paruco Dam and the problems of the same nature existing in the reservoir area of Calachota Dam have resulted in disregarding these sites for erection of dams.

It would usually be more advantageous to look for dam sites in the downstream reaches of a basin in case the main purpose is water supply. Dams in the downstream reaches of a basin would control a bigger percentage of the catchment area and would usually be sufficient to serve the purpose. In this regard, surficial geological studies have come up with the conclusion that the Auco site and San Jeronimo site, considered as potential dam sites at the beginning of the study, are suitable for erection of dams. San Jeronimo site is the most downstream site in the basin suitable for the erection of a dam. In case the geological conditions were favorable in the upstream reaches of the basin, it would be possible to erect dams which would help meet some of the regulation requirement of the river runoff and thus be very useful for power generation at a reasonably low cost and secondly to help meet the demand of domestic water and irrigation.

The probable set of dams that would be promoted by Cementos Lima and the additional ones to meet mainly the demand for domestic water of Lima is given below in the upstream-downstream direction.

Name of the Dam	Catchment Area (Km²)	Riverbed Elevation (m.a.s.l.)	Mean Annual Water Potential (MCM)
1. Morro de Arica (*)	1,689	2,785	681
2. Auco	2,713	1,955	987
3. Capillucas (*)	3,288	1,521.50	1,094
4. San Jeronimo	4,880	996	1,507
5. Zuñiga	5,188	775	1,637
6. Lunahuana (**)	~ 5,400	425	1,643
7. Nuevo Imperial	5,800	~ 200	1,643

Water potential given in the above table is the natural ones.

San Jeronimo Dam lies in Rio Cañete between Capillucas Dam and the powerhouse of this dam. The tailwater elevation of Capillucas Powerhouse is about 900 m.a.s.l.

Zuñiga, Lunahuana and Nuevo Imperial are the intake sites respectively for water transfer to Lima, delivery of water to Concon-Topara-Chinca Alta area and the Cañete Valley already irrigated.

Morro de Arica and Capillucas Dams and the intake structure at Lunahuana will be implemented by Cementos Lima.

4.6 Determination of Dam Heights

The full supply level of the dams will be determined assuming a dead storage volume due to sediment accumulation in the reservoir for a period of 50 years and superposing to it the active storage requirement and the freeboard, with the type of spillway assumed to be gated at this stage of study.

4.6.1 Sediment Yield

It appears that the tributaries of Rio Cañete carry a lot of sediment in rainy periods which can be considered to be a characteristic of the basin. The sediment yield based on this observation has been assumed as 600 m^3/sq. km/year for Auco Dam and San Jeronimo Dam.

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* Cementos Lima dam sites

** Cementos Lima diversion dam site

4.6.2 Determination of Active Storage Requirement

The inflow pattern of Rio Cañete is quite irregular as mentioned in numeral 4.3.1. Other than that, the annual runoff of Rio Cañete has big fluctuations in some years. Following are the annual mean discharges and the annual total water potential values of Rio Cañete between 1986 - 1997 based on measurements at Socsi site:

Year	Annual Mean Discharge (M^3/Sec)	Annual Runoff (MCM)
1986	80.67	2,544
1987	39.96	1,260
1988	32.80	1,034
1989	51.66	1,629
1990	26.98	851
1991	51.25	1,616
1992	20.36	642
1993	73.41	2,315
1994	82.33	2,596
1995	49.00	1,545
1996	62.75	1,979
1997	41.17	1,298

The active storage requirement of a dam is greatly dependant on the inflow pattern of the river and the outflow pattern from the dam reservoir which is actually the demand. As for the development of Rio Cañete for water supply and irrigation, the demand is all throughout the year and besides the months of peak demand usually match with the period the river runoff is bigger than the demand. This is a factor to be effective to reduce the active storage requirement of the dams. The rainy period that is summertime, except dry years, is the period in this basin to fill the reservoirs and have spillage at the same time. However variation of the river runoff annually is more influential in this respect. That is why the exceptionally dry year 1992 plays a vital role in increasing the active storage requirement. In such periods usually a shortage of supply for irrigation would be foreseen to reduce the cost of the storage dam or dams. This approach would also be valid to a lesser extent for domestic water supply.

The height of a dam for a given storage capacity is closely dependant on the physical characteristics of the reservoir area and mainly on the riverbed. Rio Cañete has an average slope of 2.5% and 4% along certain reaches of the river.

This is a big slope resulting in the need for a higher dam so as to fulfill the requirements.

Reservoir operation studies conducted have covered different scenarios of demand where the storage dams to meet the demand mainly consisted of a combination of Morro de Arica + auco or Morro de Auco + San Jeronimo. The demand given has been increased due to maintenance flow requirement of the river. The scenario which takes into consideration the total water demand given as well as the demand of water for maintenance flow of the river is called as “Scenario 3”. The demand for Scenario 3 is met by the following set of dams:

Name of the Dam	Active Storage Requirement (MCM)
Morro de Arica	245
San Jeronimo	250

Another set of dams comprising of (Morro de Arica + Auco) dams has not been sufficient to meet the demand.

4.6.3 Height of Morro de Arica Dam

This is a project to be implemented by Cementos Lima. Studies on this project are still in progress and the present status of study is at preliminary level. The tendency of Cementos Lima is to further heighten the dam. Some of the characteristics of Morro de Arica dam which meets the demand of Scenario 3 are given below.

Riverbed Elevation	:	2,785	m.a.s.ℓ
Catchment Area	:	1,689	sq.km
Mean Annual Runoff	:	681	MCM
Sediment Yield	:	300	m^3/sq.km/year [Cementos Lima assumption]
Dead Volume	:	19	MCM
Effective Storage capacity	:	245	MCM
Total Storage Capacity	:	264	MCM
Reservoir Elevation at F.S.L.	:	2,785	m.a.s.ℓ
Minimum Operation Level	:	2,870.50	m.a.s.ℓ
Crest Level	:	3,012	m.a.s.ℓ
Height above the Riverbed	:	227	m

This dam is also for hydropower generation and the powerhouse is located at the end of the diversion tunnel converted to a power tunnel. The minimum operation level of the reservoir for the operation of turbines is higher then the reservoir elevation corresponding to dead storage. The diversion tunnel has another branch to be used as a bottom outlet structure and the valve to be placed in here will enable the release of water downstream when the reservoir level falls below the level to run the powerhouse.

4.6.4 Height of San Jeronimo Dam

The second dam proposed in the study is San Jeronimo dam. Some of the characteristics of this dam are given below.

Riverbed Elevation	:	996	m.a.s.ℓ
Catchment Area	:	14,880	sq.km
Mean Annual Runoff	:	1,507	MCM
Sediment Yield	:	600	m^3/sq.km/year
Dead Volume	:	95	MCM
Effective Storage capacity	:	250	MCM
Total Storage Capacity	:	345	MCM
Reservoir Elevation at F.S.L.	:	1,180	m.a.s.ℓ
Minimum Operation Level	:	1,110	m.a.s.ℓ
Crest Level	:	1,185	m.a.s.ℓ
Height above the Riverbed	:	189	m

4.6.5 Height of Auco Dam

Auco Dam is not proposed in the final set of dams but it is a potential site and has been studied as an alternative. Some of the characteristics of this dam are given below:

Riverbed Elevation	:	1,952	m.a.s.ℓ
Catchment Area	:	2,713	sq.km
Mean Annual Runoff	:	987	MCM
Sediment Yield from the Sub basin	:	600	m^3/sq.km/year
Dead Volume	:	30	MCM
Effective Storage capacity	:	300	MCM
Total Storage Capacity	:	330	MCM
Reservoir Elevation at F.S.L.	:	2,137	m.a.s.ℓ
Minimum Operation Level	:	2,015	m.a.s.ℓ
Crest Level	:	2,142	m.a.s.ℓ
Height above the Riverbed	:	190	m

4.7 Determination of Dam Types

Both Auco and San Jeronimo damsites are suitable topographically for a fill type of dam. The riverbed is wide being about 170m at Auco site and 140m at San Jeronimo site. At Auco damsite the average slope of the abutments is close to 1/1 at the right bank whereas it is a little bit steeper at the left bank. The abutments of San Jeronimo Dam are flatter than those of Auco Dam and this is more predominant at the left bank.

The possible fill type of dams, ignoring the geological conditions of the basin, are a) rockfill with a central clay core, b) rockfill with concrete facing and c) rockfill with an asphaltic core.

A dam of rockfill type with a central clay core is out of consideration due to the inavailability of clayey material.

Rockfill dams with an asphaltic core have so far been designed and constructed up to a height of 150 m. This type of dam would also be out of consideration at both sites due to the need of higher dams above the foundation level.

Concrete faced rockfill dams also have a height limitation and besides this type of dam is reported to have a leakage problem being important at sites where the mean annual water potential is relatively not so much.

However, the main consideration in the selection of the dam type has been the geological conditions in the reservoir area. The abutments of Rio Cañete between elevations 3,500 m.a.s.l. and 500 m.a.s.l. are in generally covered with debris, exposed rock being observed at spots. This material is like rockfill at certain spots whereas it is in the form of a slopewash at other spots. It appears to be rather medium to heavily consolidated most probably due to drying. It is most probable that this material may lose its cementitious character when it comes in contact with water. This debris seems to be quite thick at certain spots whereas it is a thin cover over the bedrock at other spots which would make it easy to remove. This material has the potential to slide due to fluctuation of the reservoir water level depending also on its slope at present. Any slide in the reservoir area would possibly result in a slide of the overlying debris. Such occurrences would be effective in reducing the effective storage capacity of the reservoir and what is more it may cause overtopping of the dam which would be quite disastrous in case of a fill type of dam.

A concrete gravity type of dam is proposed at both sites, Auco and San Jeronimo to handle the above-mentioned problems. This type of dam is much more advantageous to place sand flushing units in the dam body. These units may be placed in the blocks at the right and left banks. A suitable elevation would be one a little above the riverbed elevation.

4.8 Appurtenant Structures of the Dam

4.8.1 Diversion

The diversion of the river both at Auco and San Jeronimo sites will be designed for a flood with a 10-year recurrence interval period. The diversion system is a combination of a tunnel and a cofferdam and the optimum dimensions of this sytem will be determined in the next stages of the study. The diversion tunnel of the Auco dam lies at the right bank whereas it is at the left bank at San Jeronimo site. The diversion tunnel can also be used as a sand flushing tunnel during the operation stage of the reservoir with the introduction of a high level intake structure at the tunnel inlet. In this case there would not be need for a regulatory type of valve and a slide gate would be sufficient.

4.8.2 Spillway

Spillways of both Auco and San Jeronimo dams will be designed for the probable maximum flood. The type of spillway considered at this stage of study is a gated one utilizing a big percentage of the width of the valley so as to reduce the discharge per unit length. An alternative type of spillway which appears to fit the operation policy of such dams would be an ungated spillway. This type of spillway would not need any operation staff and besides routing effect of the reservoir might be more pronounced so as to result in a smaller outflow from the spillway. However, an ungated spillway to be provided at both sites would definitely result in an increase in cost since the dam height would be bigger in this case. It is worth to remark that this is not only a matter of cost and operational considerations should not be overlooked. The terminal structure of spillway at both dams has been considered as a flip bucket. This would no doubt cause scouring in the downstream of the bucket. Spillways of dams in this basin are expected to operate almost continuously during summertime. This subject needs further consideration in the latter stage of the study.

4.8.3 Bottom Outlet

The development of the basin is mainly for water supply and irrigation. Release of water downstream from a dam for water supply is a constant discharge all throughout the year. Irrigational water requirement in Peru in the coastal area is also for 12 months a year with the highest demand occurring in February when the runoff in the basin in general has the peak value.

The operation of Morro de Arica dam and Capillucas dam, both of them being projects to be implemented by Cementos Lima, will be a little bit different than Auco or San Jeronimo dams. Water release from Morro de Arica reservoir will be through the powerhouse intended to be operated as a peak plant from turbine efficiency point of view. However, the reservoir of Morro de Arica dam, in normal and wet years, appears to have an inflow sufficient to operate the power plant continuously for almost three months a year in average. Powerhouse of Capillucas dam is also planned to be operated as a peak plant and daily regulation of the reservoir will be sufficient for it. This powerhouse will also operate continuously for almost three months a year. The tailwater channel of Capillucas powerhouse is to be connected to a reservoir so that it would be possible to release water downstream from this reservoir to meet the demand for water supply and irrigation. Auco dam which is not proposed in this study is located to the upstream of Capillucas Dam. Therefore, in case it was proposed, the bottom outlet system would be operated continuously or it would be stopped in summertime when the spillage from the dam was in excess of the demand. The case is a little bit different with San Jeronimo Dam. The source of water to be impounded in the San Jeronimo reservoir is spillage from Capillucas dam and release from the intermediate basin. Even in summertime the release from the powerhouse of Capillucas dam would be insufficient to meet the demand. This is explained below:

Maximum plant Discharge of Capillucas powerhouse: 42 m^3/sec.

Total Deabd at Socsi in February including Maintenance Flow: 54.2 m^3/sec.

This would necessitate releases from the San Jeronimo dam even in summertime and it would be as spillage for most of the time. During the rest of the year, the release from San Jeronimo dam would be continuous but in continuous coordination with the operation of Capillucas powerhouse.

The bottom outlet capacity of the San Jeronimo dam will be determined so as to release downstream the peak demand, at full opening at the minimum operation level. The bottom outlet valve can be operated at partial opening at higher reservoir levels.

The bottom outlet valve of San Jeronimo dam would be of a regulatory type, either a Hollow-Jet type of valve or a Howell-Bunger type of valve with a hood liner. The bottom outlet structure can be placed in one of the blocks adjacent to spillway blocks both at Auco and San Jeronimo dams.

4.8.4 Sand Flushing Gates

The abutments of the reservoir of both Auco and San Jeronimo dams are covered with debris of a variable thickness. The possibility of landslides in the reservoir area due to fluctuation of the reservoir level would make it necessary to have sand flushing systems incorporated in the dam body. A concrete gravity type of dam is the most convenient type of dam for this purpose which would enable to have intake structures at different levels.

Three sand flushing systems are provided at both Auco and San Jeroino dams. Two of them have the same inlet elevation but in different blocks at the right and left banks of the dam adjacent to the spillway blocks. The other one is at a higher elevation. The gates of the sand flushing system will be placed in a hall to the downstream of the dam blocks and have an emergency gate adjacent to it. They will both be slide gates and operated periodically at full opening. The size considered at this stage for the gate is 2.50 m. It may be necessary to provide a gate also at the intake structure to be controlled from the dam crest.

Chapter 5 Water Transfer to Lima

In 1995, a study was made for Sedapal by a local consulting company to investigate the possibility of conveying water to Lima from Rio Cañete. The discharge considered in the study was 5 m^3/sec, 7.5 m^3/sec and 10 m^3/sec and the regulation of the river runoff to meet the demand was proposed to be made by damming a number of lakes in the upper basin of Rio Cañete referring to the study made for Electro-Peru in 1987 by a joint venture of Motor Columbus and Electrowatt of Switzerland and three local consulting companies. The study, then, came up with the recommendation of implementation of the project in two stages. The recent information available from the Master Plan Study made for Sedapal in 1998 is that the demand in Lima for domestic water after 2022 is 5 m^3/sec and Rio Cañete is one of the two alternatives for the transfer of water. The water transfer scheme from io Cañete will therefore be considered to be implemented in one stage.

5.1 Routes of Water Transfer to Lima

In this study two routes were studied for costwise comparison purposes: a) the mountainous route and b) the coastal route.

5.1.1 The Mountainous Route

This is mainly a free flow system consisting of tunnels and channels including pressure flow conduits consisting of pipes and siphons. The use of pipes in the system is mainly due to topographical conditions.

The intake structure in this alternative is close to a locality called Zuñiga. The elevation of water at the end of the desilting basin is 775 m.a.s.l. There is only one route up to chainage 146+000 and three close routes called A, B and C up to chainage 206+000. The elevation of the water treatment plant has been assumed as 250 m.a.s.l., 200 m.a.s.l. and 150 m.a.s.l. in the alternatives A, B and C. The recommended alternative in the study is C.

This alternative with an exploitable head of 775-150=625 m is based on losing head so as to reduce the size of the elements of the transmission line within acceptable limits. The hydraulic gradient available is 0.003 which is usually much higher than that which would be available in most of the water supply projects.

The alignment recommended in the study is reasonable targeting a relatively short length to achieve. The elements of the transmission line are as follows :

1.	Open Channel	125 km
2.	Pipe (D = 1.60 m)	53 km
3.	Siphons	8 km
4.	Tunnels	18 km
5.	Drop	2 km

The open channel recommended in the study has a slope of 0.001 and it is of rectangular crosssection. Freeboard assumed is 0.30 m. The width of the maintenance road is 3 m with a provision of a berm, 0.50 m in width on the mountains side of the platform. The slope of cut has been assumed as 1:V-0.1:H in sound rock conditions to reduce the amount of excavation.

The tunnels are of modified horseshoe type with an internal diameter of 3.00 m. The longitudinal slope of the tunnel is 0.002. The number of tunnels in this alternative is 11.

There are eight number of siphon crossings along the transmission line and three of them at crossings of Rio Omas, Rio Mala and Quebrada Chilca are as high as about 250 m. These water courses are dry during wintertime. It appears as if the location of siphon crossings has been the guiding factor in the determination of the route. In this regard, a drop of about 140m has been foreseen about 10 km to the upstream of Rio Omas crossing for the purpose of shortening the length of the line. The siphons which are all subject to a high head are composed of pipes resting on footings at the abutments and encased in concrete along the riverbed crossings. Concrete surrounding the pipe would be reinforced, to take some percentage of the internal pressure the pipe would be subject to.

The pipe crossings along the reach of the transmission line have been determined mainly on topographical conditions and on geological conditions along a certain reach of the line. The first 12.8 km long stretch of the line is a pipe crossing along Rio Cañete. It is most probable that why such a preference has been made lies on the fact that the right abutment is covered with debris at certain spots and secondly the slope of the abutment is quite variable. The second pipe crossing is between chainage 64+800 and 74+540 and it is due to topographical conditions and secondly due to achieving a shorter line. The third pipe crossing is proposed between the chainages 171+400 and 206+000 where the topographical conditions would only allow a pipe crossing.

5.1.2 The Coastal Route

This route is not a separate one between the intake structure at Socsi and the treatment plant at Lurin.

The elevation of water at the exit of the desilting basin at Socsi is 340 m.a.s.l. The line extends for 105 km with a 1.80 m diameter pipe and then pumped up for 110 m to be connected to alternative C of the mountainous route at an approximate elevation of 275 m.a.s.l.

The total tunnel crossing in this alternative within the first 105 km which is a separate line is about 6 km. The size of the tunnels appears to have been selected so as to accommodate two pipes installed in stages. The rest of the line between chainage 105+000 and 165+000 is exactly the same as in alternative C of the mountainous route.

5.2 Comments on the Transmission Line of 1995 Study

The 1995 study made for Sedapal considered siphon crossings only at big river crossings and ignored them along the gullies. Some of the gullies at the abutments have a slope in the order of 7.5 % and there is a high probability that loose material at the bottom of these gullies may fall into the channel even if there is no rainfall.

The width of the maintenance road adjacent to the channel appears to be inadequate. Besides, the berm on the mountainous side of the channel is too narrow with a width of 0.50 m. The slope of cut considered as 1:V-0.1:H is too steep and no recommendations have been made to protect the slope of the excavated surface of slope.

The head available for the coastal route is 340 - 150 = 190 m. Assuming that the total length of the transmission line is 165 km as given for the coastal route in the 1995 study, the hydraulic gradient that could be used for a separate line would be 0.00115 which is more than sufficient for a pipe to discharge 5 m3/sec. It is not clear why a separate line has not been considered all through. The pipe size would be bigger in this alternative but there would not be a need for the pumping station.

5.3 Alternative Routes

Alternatives route study was made for both the mountainous and coastal routes. More emphasis was given in this study to the mountainous route.

5.3.1 Alternative Mountainous Route

The intake site is still at Zuñiga but the water level has been assumed as 780 m.a.s.l. at the end of the desilting basin.

In principal, the route in this alternative study is similar to the route recommended in the study made for Sedapal in 1995, but there are modifications at certain stretches.

The elevation of the water treatment plant at Lurin has been assumed as 200 m.a.s.l. so as to be able to supply water to the surrounding area by gravity.

The recommended channel crosssection in this study is also rectangular which appears to be the crosssection with the least cost per meter due to resulting in a lesser excavation to be made for the platform. The slope of the channel is 0.0008. This results in a rectangular crosssection with a width of 2.60 m and a water depth of 1.28 m for a Manning’s coefficient, n, of 0.014 for a discharge of 5 m^3/sec. Freeboard assumed is 40 cm and the thickness of concrete is 15 cm.

The other free flow element in the system is the tunnels with a modified horseshoe crosssection with an internal diameter of 3.00 m. The slope of the tunnel is 0.0007 and the corresponding water depth is 1.17 m.

The thickness of concrete lining assumed is 30 cm. with an additional 10 cm. assumed for probable overexcavation.

The main difference in this alternative study is the introduction of siphons at gully crossings. The number of siphon crosssings along the 175 km long alternative route is 42 including the high head siphon crosssings along the main rivers. The velocity in the siphons is about 1.3 times as much as it is in the open channel.

The slope of the abutments along the transmission line is variable, it varies between 1:V-1:H to 1:V-3:H.

The elements of the alternative transmission line for the mountainous route are as follows:

1.	Channel	70.5 km
2.	Tunnels	29.3 km
3.	Siphons	19.5 km
4.	Pipe, D = 2.00 m	30.4 km
5.	Pipe, D = 1.60 m	19.5 km
6.	Drop, D = 1.60 m	2.8 km

The number of tunnels is 40 among which the length varies between 90 m and 4730m. All the tunnels are of free flow type except for one which is a pressure tunnel with a length of 1 450 m. This tunnel is circular in crosssection with an internal diameter of 3.00 m.

Among the 42 siphons existing in the system, seven of them are subject to an internal pressure above a 100 meters. These crossings will be of steel pipe with encasement of steel pipe with concrete along the riverbed. The rest of the siphons are subject to a variable head with a maximum head of about 55 m. These siphons are considered to be of reinforced concrete.

The D = 2.00 m steel pipes are recommended to be in the system in the first 63 km due to geological and topographical conditions. The hydraulic gradient used for those pipes for a discharge of 5 m3/sec and a Manning's coefficient n of 0.012 for a middle aged pipe with an additional 10% head loss due to horizontal and vertical bends is 0.001.

A drop has been foreseen in the system for about 240 m. between the chainages 65+250 and 68+050. This drop is a potential source for hydropower generation with a plant factor of 1.00 in case standby units are provided. The reason why such a drop is recommended lies in the fact that it is a requirement so as to reach the siphon crossing at Rio Omas within the shortest length.

The introduction of more tunnels in this alternative study with respect to that in the 1995 study is due to topographical conditions. Besides, it is considered to be advantageous from maintenance point of view.

The platform of the open channel considered in this study is wider, the width of the maintenance road is 5.00 m and the width of the berm on the mountain side of the open channel is 1.00 m. Presplitting method of blasting is recommended so as to get a smooth surface with a minimum number of loose rock fragments on the excavated surface. Installation of a wire net on the excavated slope is foreseen so as to prevent rock fragments to fall to the channel.

The number of 1/25 000 scale maps covering the alternative mountainous route is 14.

5.3.2 Alternative Coastal Route

In this alternative study emphasis has been given so as not to have a pipe crossing along the already irrigated farmlands.

The intake site is still at the same locality called Socsi and the elevation of water at the end of the desilting basin is 340 m.a.s.l. as it is in the original study. The elevation of the water treatment plant at Lurin has been assumed as 200 m.a.s.l. which is 50 m higher than the elevation given in the original study.

The total length of the alternative coastal route is 145 km and the line includes eight tunnels with a total length of about 6 000 m. The tunnels to operate under pressure have an internal diameter of 3.00 m and circular in section. These tunnels may partly or wholly need to be steel lined for static internal pressure due to inadequacy of rock overburden.

The hydraulic gradient available for the pipes in this alternative study is about 0.00095.

5.4 Interpretation of the Conveyance Lima

The mountain route proposed in the 1995 study made for Sedapal aimed at acquiring the shortest conveyance line between the intake site at Zuñiga and the water treatment plant at Lurin.

The main item that needs further consideration is the gully crossings at the abutments, the 1995 study appears not to have taken them into consideration. Besides, there is not a consistency with the elements of the conveyance line in other words, the crossing of a similar topography is considered with a pipe at certain reaches whereas it is a canal along another reach of the conveyance line. The introduction of more tunnels would definitely reduce the total length of the system.

The alternative mountain route studied during the field work in Peru includes more tunnels and siphons thereby resulting in a smaller overall length. This would no doubt have advantages during operation and besides be a safer alignment. The increase in cost due to the alternative mountain route would be minor compared to the mountain route proposed in the study made for Sedapal in 1995.

The coastal route is not only more costly, the main disadvantage of this route is that it would not be possible to divert water to the Pampas de Imperial where 2,475 ha of land is planned to be irrigated with the io Cañete Scheme.

The construction of the conveyance line will be made at one stage for Q=5 m^3/sec which is the most recent target for supply of domestic water to Lima. However, the chainage where water could be diverted from the mountain route to Pampas de Imperial near Quilmana corresponds to about 40+000 at the conveyance line. The maximum demand for irrigation in this area is about 4 m^3/sec, that is why the conveyance line must be designed for 5+4=9 m^3/sec up to this chainage. This would require the installation of either two pipes for the first 12.8 km reach of the conveyance line or a bigger single pipe.

Chapter 6 Mantaro-Carispacha Scheme for Water Transfer to Lima

The latter stages of the field work in Peru have resulted in the inclusion of a new task in the study. Sedapal has another alternative to transfer water to Lima to meet the demand after 2022 and this project is called as “Marcapomacoha-Carispacha Scheme”. The discharge is 5 m^3/sec and that is why the Rio Cañete Scheme has been modified to transfer 5 m^3/sec to Lima.

6.1 General

Rio Mantaro lies to the east of the ‘Continental Divide’ in Peru. It appears that there have been studies since 1950’s to divert water from Rio Mantaro Basin to the basins on the Pacific slope of the Peruvian Andes. This scheme has been called as Mantaro Transfer Scheme since the beginning of the said studies. The purpose of this diversion had focused on supply of water to Lima for domestic and industrial use. On the other hand, water diverted is an additional source of flow for hydropower generation in the existing power plants on the Pacific slope of the Andes. However, water diverted results in a reduction in the hydropower generated in the power plants in Rio Mantaro.

A study was made for Sedapal in 1995 on transfer of water to Lima from the headwaters of Rio Mantaro to meet the domestic water demand of Lima and the discharge considered for the final stage was 10 m^3/sec whereas it is 5 m^3/sec for the initial stage of development.

The study made for Sedapal is based on the study “Final Study of the Transfer of Water from the Upper Catchment of the River Mantaro to Lima” carried out by “Binnie and Partners” in 1981.

6.2 Description of the Mantaro-Carispacha Scheme

Following is a brief explanation of the Mantaro Transfer Scheme as studied a few years ago for Sedapal. This study is a modified version, mainly based on deviations from the projected populations, of the 1981 Binnie and Partners study.

The project area lies roughly between elevations 4,200 m.a.s.l and 4,550 m.a.s.l and the system is based on diverting water from a number of streams to be conveyed to a reservoir, of a very small useful storage capacity, created by damming Rio Carispacha, pumping the water from this reservoir for an approximate static head of 250 m to the beginning of a conveyance canal from where it would be conveyed to the reservoir of Marcapomacohca Dam for regulation. Water regulated in the reservoir of Marcapomacocha Dam, to be enlarged in accordance with the requirements of the ‘Project’, will be released downstream and reach, through the Cuevas Canal and the Trans-Andean Tunnel already existing, to the headwaters of Rio Santa Eulalia, a tributary of Rio Rimac on the Pacific slope of the Andes.

The total catchment area of the system as called the “Mantaro-Carispacha Scheme” is 767 sq.km and the mean annual amount of water to be diverted from this system is estimated to be 360 MCM corresponding to a mean discharge of 11.4 m^3/sec. However, the system will be developed in two stages, compatible with the growth of demand, with a first stage target to supply a regulated flow of 6.2 m^3/sec corresponding to an annual total of 196 MCM. The total catchment area of the streams from which water will be diverted in the first stage is 299 sq.km.

A certain percentage of the catchment area of this scheme has been used for the development of the Marca III Project. However, it will still be assumed at this stage that the first stage development of the scheme as proposed will still be sufficient to supply in average a flow of 5 m^3/sec.

The stream water will be diverted with the erection of a weir in them lie to the north and south of Carispacha reservoir. These streams for the final stage of development are as follows:

I. Southern Streams

	Streams Diverted	Catchment Area (sq.km)
1.	Rio Yanacancha	39.5
2.	Rio Richis	
3.	Rio Shoclay	94.5
4.	Rio Morada	
		ΣC.A=134.0

II. Northern Streams

	Streams Diverted	Catchment Area (sq.km)
1.	Rio Huascacocha	127
2.	Rio Tambo	140
3.	Qda Casacacha	201
4.	Qda Taprasa	25
5.	Qda Atapa	62
6.	Qda Macuri	
		ΣC.A=555

In addition to this, the catchment area of Carispacha Dam which will lie in Rio Carispacha is 8 sq.km.

Total Catchment Area = 134+555+78 = 767 sq.km

The first stage development of the scheme considers all the southern streams and Qda Taprasa, Qda Atapa and Qda Macuri. In this case,

Total Catchment Area = 134+25+62+78 = 299 sq.km

6.2.1 Criteria of Comparison of Alternatives

As mentioned earlier in the report, the demand in Lima for domestic water after 2022 is estimated as 5 m^3/sec. That is why both alternatives to be compared for the transfer to Lima, of the same amount of water.

6.2.2 Amount of Water that Can be Diverted

The first stage development of this scheme considers the diversion of water from a catchment area of 299 sq.km.

Catchment Area = 299 sq.km

Mean Rainfall = 800 mm

Runoff Coefficient = 0.51

Mean Runoff = $299{,}000{,}000 \times 0.80 \times 0.51$ 122 MCM

It will be assumed that at most 95% of this runoff can be diverted to the system within economic limits.

Then Mean Runoff Diverted = 0.95×122 115.9 MCM

Qmean = 3.68 m^3/sec < 5 m^3/sec

Flows from a bigger catchment area need to be diverted to supply the amount of water required.

6.2.3 Elements of the Conveyance System

The free flow conveyance conduits to transmit water to Carispacha Reservoir are given below with their types and lengths:

I. Southern Streams

	Location	Type of Conduit	Length (m)
1.	Rio Yanacancha – Rio Richis	Canal	3,800
2.	Rio Richis – Tunnel Inlet	Canal	800
3.	Tunnel Inlet – Tunnel Outlet	Tunnel	2,400
4.	Rio Shoclay – Rio Morada	Canal	5,900
5.	Rio Morada – Carispacha Reservoir	Canal	5,700

II. Northern Streams

	Location	Type of Conduit	Length (m)
1.	Qda Taprasa – Tunnel No.2 Inlet	Canal	6,600
2.	Tunnel No.2 – Tunnel No.2 Outlet	Tunnel	1,600
3.	Tunnel No.2 Outlet – Qda Atapa	Canal	3,600
4.	Qda Atapa – Qda Macuri	Canal	400
5.	Qda Macuri – Tunnel No.3 Inlet	Canal	500
6.	Tunnel No.3 Inlet – Tunnel No.3 Outlet	Tunnel	2,200
7.	Tunnel No.3 Outlet – Tunnel No.4 Inlet	Canal	1,300
8.	Tunnel No.4 Inlet – Tunnel No.4 Outlet	Tunnel	1,300
9.	Tunnel No.4 Outlet – Tunnel No.5 Inlet	Canal	2,600
10.	Tunnel No.5 Inlet – Tunnel No.5 Outlet	Tunnel	3,200
11.	Tunnel No.5 Outlet – Carispacha Reservoir	Canal	4,800

Total Canal and Tunnel Lengths

	Location	Total Canal Length (m)	Total Tunnel Length (m)
1.	Southern Streams	16,200	2,400
2.	Northern Streams	19,800	8,300
		ΣL=36,000	ΣL=10,700

(1) Design Discharges

The design discharges assumed in the study made for Sedapal are reasonable for a study at this stage. It is 4.90 m^3/sec for the southern system for the downstream canals. It can be assumed to be in the order of 3.0 m^3/sec for the northern system proposed. Rio Carispacha with a catchment area of 78 sq.km at Carispacha Dam site is considered to have a mean discharge in the order of 1.0 – 1.5 m^3/sec.

(2) Probable sizes of the Conduits

There are six tunnels along the alignment of the conveyance line. The maximum discharge to be transmitted by a tunnel is in the order of 3 m^3/sec. However, the tunnel size is to be selected as 3.00 m due to consideration of construction practice although a tunnel of this size can accommodate discharges up to about 15 m^3/sec. The type of the tunnel considered is modified horseshoe.

The sections of canals in the system are interpreted to be determined based on the slope of the abutments. They are usually trapezoidal in section with side slopes of 1:1. This is the steepest slope for placement of concrete without the need to use a formwork. However, concrete to be used in this case would be required to have a rather low slump. Thickness of concrete assumed as 0.0.75 m by Sedapal for the lining of the canals should preferable be 0.10 m which is the usual practice.

6.2.4 Carispacha Dam

The reservoir to be created by the Carispacha Dam will have the function of hourly to daily regulation through balancing the flows entering the reservoir form the southern and northern canals and the flows pumped to the Upper canal which conveys the flows to the reservoir of Marcapomacocha Dam.

The effective storage requirement of Carispacha Dam is very small but it will be assumed as 2.5 MCM as given for a maximum fluctuation of the reservoir level by 1 m. The function of Carispacha reservoir is not seasonal regulation.

The characteristics of Carispacha Dam are given below:

Full Supply Level	:	4,185 m.a.s.ℓ
Min. Operation Level	:	4,184 m.a.s.ℓ
Riverbed Elevation	:	4,160 m.a.s.ℓ
Total Storage Capacity	:	25 MCM
Active Storage Capacity	:	2.5 MCM
Crest Width	:	10 m
Crest Elevation	:	4,189 m.a.s.ℓ
Maximum Height of Dam	:	29 m
Type of Dam	:	Zoned earthfill
Spillway Design Discharge	:	150 m^3/sec
Size of Diversion Tunnel	:	3.00 m
Type of Tunnel	:	Horseshoe

The dam site is in a highly active region from seismicity point of view. The depth of glacial deposits at the dam site is not clear according to the Binnie and Partners Study. On the other hand, the left bank is covered with thick deposits of glacial and fluvioglacial materials overlying the rock and it is understood that the andesitic rocks at the right bank are at a shallow depth.

The foundation treatment at Carispacha Dam is recommended to be made by grouting in glacial deposits and in rock. Diaphragm walls are not recommended due to the presence of big boulders in the glacial deposits.

Materials to be used in a zoned type earth fill dam are available in the close vicinity of the dam site.

(1) Comments on Foundation Treatment and Dam Type

Alluvium grouting as proposed in Binnie and Partners Study is rather a measure for temporary structures such as cofferdams. However, it is possible to have two or three rows of grout curtain. The possibility of using diaphragm walls should be checked and studied again.

The possibility of adopting a zoned earth fill dam with an upstream blanket may prove to be a better alternative in which case there would not be a need for the grouting of glacial deposits in the valley bottom.

The removal of some of the fluvioglacial material at the left bank may be another alternative. In this case the volume of the dam embankment would increase but the depth of grouting would reduce.

6.2.5 The Pumping System

All the water diverted from the southern and northern streams and conveyed to Carispacha Reservoir by free flow conduits will be pumped to a head pond, with an approximate static head of 250 m, from where it will be transmitted by a 2.7 km long canal to the reservoir of Marcapomacoha Dam.

Inflow to Carispacha Reservoir will be variable, however it is estimated at this stage that a design discharge of 10 m^3/sec would be sufficient for the withdrawal of 5 m^3/sec continuously from the reservoir of Marcapomacocha dam.

The number of pumps considered by Sedapal is five with a capacity of 2.50 m^3/sec each where one of the pumps is a standby unit.

Both the pumping head and the discharge are rather big, therefore it is recommended, in future stages, to study the possibility of having seven pumps with one of them using a standby unit. This alternative would slightly increase the size of the pumping station but its effect on the cost would be minor. Increasing number of pumps would provide more flexibility in operation.

The steel pipe to convey the design discharge of 10 m^3/sec to the head pond is 1,805 m in length 2.00 m in diameter. It can be studied in future whether a bigger pipe should be used.

6.2.6 The Upper Conveyance System

The upper conveyance lineis a free flow conduit to transmit, to Marcapomacocha Reservoir, a maximum design discharge of 10 m^3/sec pumped from the Carispacha Reservoir. The line consists of a 2,600 m long rectangular canal and a 100 m long tunnel. The characteristics of the conduit are given below:

Width of Canal	:	3.20 m
Height of Canal	:	2.10 m including freeboard
Tunnel Diameter	:	3.00 m
Type of Tunnel	:	Modified Horseshoe
Slope	:	0.001

6.2.7 Marcapomacocha Dam

There is already a reservoir of 80 MCM storage capacity at the site created by a dam and this storage is used by Electro-Lima to provide releases, during the dry season, to the Transandean System, conveying water to the Pacific slope of the Andes.

All the water entering the Carispacha Reservoir from the streams diverted will be pumped to Marcapomacocha Reservoir for regulation. Water pumped to this regulation reservoir will vary between an approximate minimum discharge of 1 m^3/sec and a design maximum of 10 m^3/sec. The outflow from the reservoir is intended to be 5 m^3/sec all throughout the year to meet the additional domestic demand of Lima after 2022. The effective storage capacity of the Marcapomacocha Reservoir is to be determined so as to provide a continuous release of 5 m^3/sec for domestic water supply to Lima and the release of 4 m^3/sec for Electro-Lima during the dry period. This would necessitate to have the data of daily inflows to Marcapomacocha Reservoir pumped from the Carispacha Reservoir.

The report prepared by Sedapal in the recent years on this transfer scheme considers that an effective storage of 100 MCM would be sufficient at Marcapomacocha Reservoir for the first stage development of the Transfer Scheme, in other words, a withdrawal of a continuous discharge of 5 m^3/sec. This means the dam to be rebuilt at the same site will only have an additional storage capacity of 20 MCM.

It will be considered, with an assumed inflow pattern of the pumped discharges to Marcapomacocha Reservoir, that the additional effective storage requirement would be about 50 MCM. In this case the total effective storage capacity of Marcapomacocha Reservoir would be 130 MCM. Some of the characteristics of the Marcapomacocha Dam are given below:

Full Supply Level	:	4,430 m.a.s.ℓ
Min. Operation Level	:	4,412 m.a.s.ℓ
Riverbed Elevation	:	4,408 m.a.s.ℓ
Effective Storage Capacity	:	130 MCM
Crest Width	:	10 m
Crest Elevation	:	4,434 m.a.s.ℓ
Maximum Height of Dam	:	26 m
Type of Dam	:	Zoned earthfill
Spillway Design Discharge	:	25 m^3/sec
Diversion/Bottom Outlet	:	2 no. of 2.5 m diameter concrete culverts controlled by radial gates at the downstream end

The dam rests on thick glacial deposits exceeding 60 m as given in the “Binnie and Partners Report”, Volume 1. The glacial deposits, as reported, contain permeable lenses. It is reported that a fault passes through the foundation of the dam.

The treatment of the stream during construction is considered to be made by twin circular culverts beneath the highest part of the dam. This is the consideration by Binnie and Partners in 1981 when the ‘Study’ came up with a proposal of releasing 22 m^3/sec from the Marcapomacocha Reservoir at the final stage of development of the scheme.

Construction materials to be used in the zoned earthfill type of dam are available within a short distance to the dam site.

(1) Comments about Marcapomacocha Dam

The foundation conditions of Marcapomacocha Dam are reported to be similar to that of Carispacha Dam or even less favorable. The conditions of seismicity are similar as well.

It is recommended at this site as for Carispacha Dam, to study the possibility of adopting a zoned earthfill dam with an upstream blanket. This would prove to be more economical and technically more effective than a partial cutoff formed either by grouting or a diaphragm wall.

The diversion conduit to be founded on a thick glacial deposit may undergo settlements due to the weight of the dam embankment. It is understood, according to the study made by Binnie and Partners, that there is not another alternative. This subject needs further consideration.

The diversion conduits shall, during the operation stage of the dam, be used as bottom outlet conduits and the conduits are equipped with radial gates at the downstream end of the conduits for flow adjustment. The conduits are then connected to the Cuevas canal.

Sedapal has made some modification in the Master Plan for supply of water to Lima after the study of Binnie and Partners in 1981. This may result in the need of a smaller capacity for the bottom outlet system. This is to be reviewed as well.

6.2.8 The Transandean Aqueduct

The Transandean Aqueduct is a conveyance system comprising of the 11.9 km long Marcapomacocha-Cuevas canal and the 10.1 km long Cuevas-Milloc tunnel to transmit the discharges, released from Marcapomacoha Reservoir, to the upper basin of Rio Rimac on the Pacific slope of the Andes. The bottom outlet structure of Marcapomacocha Dam is connected to the Cuevas canal.

The capacity of both the Cuevas canal and the Cuevas-Milloc tunnel is 14 m^3/sec.

The discharge in the system at present is as follows:

1. Release for Electro Lima in the dry period, $Q = 4\ m^3/sec$
2. Release due to Marca III Project, $Q = 3.1\ m^3/sec$

$\Sigma Q = 7.1\ m^3/sec$

The regulated discharge from the Mantaro-Carispacha Scheme is intended to be 5 m^3/sec. In this case the total discharge to pass from the Transandean Aqueduct would be about 12 m^3/sec still being less than the ultimate capacity given as 14 m^3/sec. There would not be a need for enlargement of the Cuevas canal and remedial works in the Cuevas-Milloc tunnel in case the Mataro-Caripacha Scheme is developed for a release of 5 m^3/sec.