- Pump capacity	:	4,000 gallons/min (approx. 15.1 m^3/min)
- Electric motor	:	553 HP (392 kW) 2,100 rpm 440/220 volts
- Discharge pipe	:	Diameter 8 inches Length 650 m

The dredging operation was commenced in 1989. However, the operation encountered some troubles and difficulties which partly arouse from the nature of reservoir deposits including wooden pieces, logs and various kind of urban and agricultural waste. Then, the dredging work has been suspended to date, while CFE plans to resume the work by improving the system and equipment of the existing facilities.

CHAPTER 4

OVERVIEW OF PROJECT AREA

CHAPTER 4 OVERVIEW OF PROJECT AREA

4.1 Topography

4.1.1 Topography of the Catchment Area

The Apulco River and its upper reach tributaries rise in a highland of elevation of around 3,000 meters to the north-east of Apixaco, State of Tlaxcala, and flow down eastward. Changing the course to north and east with frequent local meanderings, the Apulco River flows more than 120 kilometers in a general direction of northeast across a mountain zone between the central highland and the Gulf of Mexico, until it joins the Tecolutla River which pours into the Gulf near the town of Tecolutla. The location map is shown in Figure 3.1. The profile of the Apulco River is shown in Figure 4.1.

Soledad Dam and Reservoir are located in the middle reaches of the Apulco River surrounded by ridges of 1,300 meters to 2,000 meters in ground height. The riverbed is approximately at elevation of 720 meters at the damsite. From the damsite the Apulco River measures 45 km in length to the junction of the Tecolutla River and 115 km to the river mouth on the Gulf.

Soledad Reservoir collects water not only from the Apulco River basin but also from a part of the basin of the Xiucayucan River, Calapa River and other small rivers through tunnels and aqueducts, all of which join the Apulco River downstream of the damsite.

Topographically the catchment area is characterized by gently sloped hills at the head of the river basin, by ridges with steep slopes on both watersheds sides of the basins in the middle reaches and by undulating hills of low angles developed in the valley floor.

The terrain of gentle slopes at the heads of the Apulco and Xiucayucan Rivers are volcanic plateaus. Low volcanic cones of Cerro Las Tables and Cerro Huintetepetl are located behind the ultimate upstream watershed of the Apulco. The Xiucayucan and its small tributaries rise on the northern slope of the caldera of Los Humeros. Both of those areas are widely covered by Tertiary to Quaternary volcanic products.

Rugged ridges of pre-Tertiary hard sedimentary rocks form the watershed in the middle and lower parts of the Apulco River catchment area, including the zone close to Soledad Reservoir. On the Xiucayucan River, the topographic features of gorges and steep slopes are common in the area north from Tlatlauquitepec where exposed are the

basement rocks of Palaeozoic and Mesozoic ages which are hard and solid enough to maintain those steep slopes.
An area of low hills and gentle slopes is formed in the Apulco valley floor. While the very narrow river channel is incised into hard ignimbrite to form a few tens of meters high cliffs on both sides in the section between the vicinity of Zautla and the dam site, the slopes formed in the soft tuffs overlying the ignimbrite are of low angles. The valley floor widens and flattens upstream, whereas it is generally narrow with topography of high relief near the reservoir.

The narrow and deep river channel disappears upstream of Zautla and is replaced by wide and shallow river beds covered with sand and gravel deposits, including andesite boulders of one to two meters of diameter. In the middle zone of the Apulco catchment area around Zautla, a terrace is formed on the acidic tuff at the height of about 30 meters from the river bed.

4.1.2 Topographic Factors for the Sedimentation

No sign of surface erosion in a large scale is found in the lower half part of the catchment areas characterized by steep slopes of hard rocks. The slopes are stable and well-vegetated. Soil conservation seems to be good, and devastation of the land is rarely seen.

On the other hand, the vegetation is poorer upstream, especially in the flat hills of acidic tuffs upstream from Xalcomulco, 16 kilometers south of Zautla. The soft tuffs are often exposed to air on gentle slopes near the river, or covered only by thin organic soil of cultivated lands. Scars by surface erosion of the soft rocks are located at places. Such vulnerability to the surface erosion tends to increase upstream, and devastation of the ground surface is seen at many places on the sparsely vegetated hills in the area upstream from Santa Maria Coyoltepec. Several dams for sediment retention in this area are all filled up with sandy materials provided through erosion of the highly weathered lava flows and the soft pyroclastic rocks forming the low undulating hills. It is obvious that those hills in the upper reaches are the major sources of the sediment yield.

It should be noted that no evidence is found for any land sliding or slope collapses of large scale in the catchment area, even though partial slope failures are seen at rather limited locations in the soft tuffs. Sediment materials are supplied mainly by surface erosion, not by sporadic mass wasting.

4.2 Geology

4.2.1 Geology of the Catchment Area

The geological map of the catchment area is given in Figure 4.2 and some schematic geological profiles are shown in Figure 4.3.

The basement rock of this region is crystalline schist of the Palaeozoic age, which are exposed only in a few limited areas near Soledad Reservoir and the Xiucayucan River basin. The schist is overlaid by Jurassic to Cretaceous (Mesozoic) sedimentary rocks which comprises dark colored limestone, sandstone, siltstone and lutites. The hard Mesozoic rocks are widely exposed in the Apulco River basin, forming the steep watershed ridges in the downstream half of the catchment area of Soledad Dam.

In the period from Late Tertiary to Quaternary, the surface of the Mesozoic rocks has been covered by young volcanic products. An extensively developed Tertiary member of this volcanic product is the andesite lava and pyroclastic rocks, including volcanic breccias and volcanic mudflows, which covers the most part of the upstream half of the catchment area in the Apulco River basin.

The Quaternary members of the volcanic products are the sub-horizontally bedded acidic tuffs which filled the valley floors. In the lowest horizon of these tuffs lies a bed of ignimbrite, or well cemented hard acidic welded tuff. The welding is obscure in parts, but the rock is so hard and solid that it forms stable cliffs, 20 to 30 meters high and almost vertical or even overhanged at places, on the sides of the narrow river channel downstream from the village of Zautla. Less compact acidic tuffs are developed in the intra-valley hills of gentle slopes in levels above the cliffs of ignimbrite. This member is often weathered and softened to varied extents. The uppermost member of the acidic tuffs is poorly consolidated and loose. These Quaternary acidic tuffs of varied strengths cover almost all area of the valley floor along the main stream and the tributaries. The upper two members are also seen widely scattered or remaining on the slopes at fairly high altitudes, topping the Mesozoic sedimentary rocks.

The Xiucayucan basin upstream from the intake at the Atexcaco diversion dam is also composed of the similar geological units. Its major part is covered by the Quaternary basalt flow and the acidic tuffs. The area at the head of the basin is characterized by rather gentle slopes and a plateau of the basalt flows and volcanic breccias, which are extensively covered with a few meter thick pumice flow. The plateau is a remnant of a caldera, and geothermal power plants are developed at Los Humeros on this plateau. In contrast, the feature of the downstream part of the Xiucayucan catchment area is represented by steep gorges and ridges in the area north from the town of Tlatlauquitepec where the pre-Tertiary rocks are exposed.

Geological structure of this region is characterized by foldings of the pre-Tertiary rocks. The Mesozoic sedimentary rocks forms anticlinoriums, or combined foldings of large distortion and small distortion, at places, with a northwesterly trend of folding axes. A number of faults of 5 to 10 kilometers in length are recorded in the published geological map in the scale of 1/250,000. There is no sign, however, of any major fault or fractured zones that may cause substantial difficulties in geotechnical practice.

4.2.2 Geological Factors for the Sedimentation

The sediment material, carried to Soledad Reservoir by the river flow, is produced through the surface erosion in the catchment area. Liability to the erosion of each geological unit in the basin area estimated as follows;

(l) Pre-Tertiary rocks

All the pre-Tertiary rocks, i.e., the Palaeozoic schist and the Mesozoic limestone/sandstone/lutites, are fairly strong against the surface erosion, and are not likely to be the source of the large sediment supply. Intensive weathering reaches to the depth of only a few meters at most, and very often less than one meter.

(2) Tertiary andesite and pyroclastic rocks

The andesites with volcanic breccias and mudflows are located upstream of Zautla in the Apulco River basin. The surface zone of the andesites is occasionally highly weathered to the depth of several meters. The matrix of the volcanic breccia is often deteriorated so soft that it may be vulnerable to the surface erosion by flowing water.

(3) Quaternary basalt flow

The basalt at the head of the Xiucayucan basin is often highly weathered in the superficial zone, but the young pumice flow covering it is the material being eroded initially.

(4) Ignimbrite

The ignimbrite at the base of the Quaternary acidic tuffs is well-cemented and hard. Although it has been dissected to form the 30 meter deep gorges along the river channel downstream of Zautla, it still maintains stable walls on both sides of the gorges. It is difficult to count this member as a source of the large quantity of sediment material supplied to the reservoir every year.

(5) Quaternary acidic tuffs in the upper beds

These members filling the Apulco river valley floor are weak, moderately soft to very soft, and visibly vulnerable to erosion. Incision of crevices by erosion under heavy rainfalls and local collapses on the sides of hills are observed at places in the tuffs of this classification, if not very frequently. These members are very probably a major supplier of the sediment material to the river.

Other large erosion crevices of a few meters in width and several tens of meters in length are seen on the plateau at the head of the Xiucayucan basin, where a vast flat land covered by the pumice flow is exposed to air without vegetation.

(6) Overburdens

Overburden or the surface soil shows thickness of several meters at some places in gullies and depressions on the slopes, while it is ordinarily far thinner. A type of the overburden is brown clayey soil which may have its origin in recent volcanic ashes or intensive weathering of the bedrocks. The other type is of slope wash or talus deposit composed of silty organic soil and rock debris. These materials can easily be eroded if exposed to flowing water. The overburden, however, is not deemed to be a major source of the sediment supply because of their limited occurrences and seemingly good conservation by the vegetation at least in the downstream half area of the catchment area.

4.3 Meteorology

4.3.1 Meteorological Gaging Stations

Fourteen (14) meteorological stations are installed in and around the catchment area of Soledad Reservoir as shown in Table 4.1. CFE (DIVISION HIDROMETRICA

GULFO, Teziutlan) is responsible for observation of rainfall, temperature and evaporation at these stations.

4.3.2 Rainfall

Location map of the meteorological stations is shown in Figure 4.4 with isohyetal map of mean annual rainfall in the study area. A great difference is seen on the distribution of annual rainfall among these stations. The annual rainfall is 2,000 mm or more in the lowest catchment of Soledad Reservoir and the Xiucayucan River basin, and the annual rainfall at Atexcaco and La Soledad is over 3,000 mm, that is, 3,647 mm and 3,325 mm, respectively. Most of the upper catchment has the annual rainfall less than 1,000 mm and the lowest is 543 mm at Zautla.

The mean monthly rainfalls are given in Table 4.2 and Figure 4.5. The dry season falls in December to May. The rainy season begins in June and continues until November. Usually the maximum monthly rainfall occurs in September at most of the stations.

The average annual rainfall on the catchment area of Soledad Reservoir is estimated by using the Thiessen Polygon method as shown in Figure 4.6. The estimated annual basin average is 1,024 mm.

Hourly rainfall recorded at La Soledad is shown in Table 4.3.

4.3.3 Evaporation

The mean monthly evaporation is given on Table 4.4. A relatively higher evaporation rate is observed during March to May at every station. The annual evaporation is less than 1,000 mm in the lowest catchment and Xiucayucan River basin, 1,000 mm or more in the middle part of the catchment, and over 1,300 mm in the upper catchment. It is noted that at the stations located in the middle and the upper catchment, the annual evaporation rate exceeds the amount of annual rainfall.

4.3.4 Temperature

The mean monthly temperature is shown in Table 4.5. At each station, the highest temperature is recorded in May or June and the lowest is recorded in January. The mean annual temperature is 20.7°C at Tepecapan (EL. 542 m) and 9.3°C at San Antonio (EL. 3,140 m). The mean monthly temperature is less than 20°C at the station which elevation is over 1,500 m.

4.4 Hydrology

4.4.1 Runoff

(I) Runoff Records

For measuring the inflows into Soledad Reservoir, stream flow gages are installed at three (3) hydrological stations, Buenos Aires, Sontalaco and Canal Tunnel No. 1 as shown in Figure 4.4. These stations were established in 1962 when Mazatepec Power Station was commissioned for power generation. Operation and maintenance of these hydrological stations are conducted by CFE (DIVISION HIDROMETRICA GULFO, Teziutlan). CFE regularly conducts water-stage observation and makes review and updating of stage-discharge rating curves every year by discharge measurement.

Runoff data on daily basis are available at the three (3) stations for the period of 29 years from 1963 to 1991. The mean monthly runoff retrieved from the daily data is shown in Tables 4.6 to 4.8. The principal features of these stations are described below.

<u>Buenos Aires</u>

- Location: lat. 19° 57' 31", long. 97° 30' 45"
- Catchment Area: 1,405 km^2

This station is located on the mainstream of the Apulco River about 7 km upstream from Soledad Dam, where a steep-walled gorge is created. Judging from the river flow conditions observed, the backwater effect from the reservoir dose not reach to this station. The average runoff at this station is estimated at 8.85 m^3/sec.

<u>Sontalaco</u>

- Location: lat. 19° 57'12", long. 97° 29' 06"
- Catchment Area: 25 km^2 at G.S.

This station is installed on the Sontalaco River, a tributary which joins the Apulco River about 5 km upstream from the dam. This station is at a distance of about 1 km from the confluence. The river water of the Sontalaco River is rather rapid. The average runoff at this station is 1.29 m^3/sec.

<u>Canal Tunnel No. 1</u>

- Location: lat. 19° 57' 24", long. 97° 26' 54"

- Catchment Area: 370 km^2 (at the Atexcaco diversion dam)

This station is located in the outlet canal of Tunnel No. 1 which leads from the Atexcaco diversion weir on the Xiucayucan River to Soledad Reservoir. The average runoff counts for 8.05 m^3 /sec, being blessed with abundant rainfall, though the catchment area is relatively small.

In addition to the records at the above three stations, the daily runoff data of the mainstream of the Apulco River at Rancho Apulco are available as shown in Table 4.9. This gage is located about 20 km upstream of Buenos Aires. The period of record at Rancho Apulco is from 1945 to 1978.

Mean annual runoffs at these stations are summarized below.

Station	Catchment Area (km^2)	Mean Annual Runoff (m^3/sec)	Recording Period
Buenos Aires	1,405	8.85	1963 - 1991
Sontalco	25	1.29	1963 - 1991
Canal Tunnel No. 1	370	8.05	1963 - 1991
Rancho Apulco	1,204	4.49	1945 - 1978

Notes: 1) Total contributing area of Soledad Reservoir is 1,830 km^2 including the catchment area for interbasin transfer of water.
2) Catchment area from Buenos Aires to Soledad Reservoir is 55 km^2 including 25 km^2 of the Sontalaco catchment.
3) Catchment area of the diversion through Tunnel No. 1 is 370 km^2 in total including 280 km^2 of the Atexcaco catchment.

(2) Runoff Characteristic

Runoff coefficients at the respective gage stations are estimated using the basin average rainfall. The basin average rainfall is obtained by using the Thiessen Polygon method (Figure 4.6). The estimated runoff coefficients are shown below.

Station	Catchment Area (km^2)	Annual Runoff Depth (mm)	Basin Average Rainfall (mm)	Runoff Coefficient
Buenos Aires	1,405	199	942	0.21
Sontalaco	25	1,627	3,256	0.50
Rancho Apulco	1,204	122	794	0.15

Notes: Period of record: Buenos Aires and Sontalaco 1963 - 1991
Rancho Apulco 1957 - 1977

The results shows that the runoff coefficient varies from 0.50 in the most downstream catchment of Soledad Reservoir to 0.15 in the upstream catchment of Rancho Apulco. There is a great difference in runoff characteristic of the Apulco River basin in the lowest catchment and the other catchment, although the distance between Soledad Reservoir and Rancho Apulco is only about 20 km.

The diversion facilities form the Xiucayucan River consist of the intake weir at Atexcaco and a number of small tributary intakes, and tunnel and canal. It is noted that though the total catchment area of the diversion facilities is relatively small compared with that of the Apulco River, the runoff yield from the Xiucayucan River basin is relatively stable and exceeds that of the Apulco River in dry season, contributing about 40% to the annual total reservoir inflow.

(3) Reservoir Inflow

The total reservoir inflow is almost equal to the sum of flows measured at the former three station. The sum is deemed as the inflow without correction for the ungaged 30 km^2 area draining directly into the reservoir. Table 4.10 shows the mean monthly inflow into the reservoir. The total inflow is 18.19 m^3/sec on average. Figure 4.7 shows the runoff hydrograph at each station and Figure 4.8 shows the reservoir inflow hydrograph. It is noted that the runoff at Canal Tunnel No. 1 exceeds that of the Apulco River during the dry season or draught year.

The duration curve of the reservoir inflow is presented in Figure 4.9. This duration curve is prepared using the inflow series which are derived from the sum of daily data for the period of 29 years. From the curve, probability of exceedence of the inflow can be known as given below.

Probability of exceedence (%)	Flow (m^3/sec)	Probability of exceedence (%)	Flow (m^3/sec)
2	67.41	50	13.02
5	43.21	60	11.39
10	31.21	70	9.97
20	22.15	80	8.75
30	17.96	90	7.57
40	15.15	100	2.86

4.4.2 Flood

(1) Flood Records

Flood in the Apulco River basin usually occurs during the period of June to November. Table 4.11 shows the major floods observed at Buenos Aires in the recent years with the daily rainfall records in the upper area on the same dates. The hydrographs of the past major floods are shown in Figure 4.10. These data indicate the following hydrological characteristics.

- Duration of flood is from 2 to 3 days.
- Duration of rainstorm is from 1 to 2 days.
- Concentration time to flood peak is from 12 to 24 hours.
- Floods are generally caused by rainstorms in the downstream and middle catchment.

The annual maximum instantaneous runoffs at the three hydrological stations are shown in Table 4.12. The maximum peaks are 711 m^3/sec at Buenos Aires (1974), 282 m^3/sec at Sontalaco (1980) and 41.9 m^3/sec at Canal Tunnel I No. 1 (1964). Table 4.12 also presents the annual maximum flood peaks recorded at Rancho Apulco.

Review was made on the accuracy of recorded data on floods. By this review, the data of Rancho Apulco recorded in 1954 to 1956 are discarded from the study for the following reasons.

a) The floods at Rancho Apulco from 1954 to 1956 are derived from the data recorded at the other gauging station (Apulco - La Gloria) just downstream of Rancho Apulco without any calibration, where another tributary joins.

b) Runoff coefficients estimated for the above 3 years are around 0.40 which is rather high against the long term runoff coefficient of 0.15 for Rancho Apulco and 0.21 for Buenos Aires.

Relationship between the flood peak and the average basin rainfall during the three-day corresponding flood period is examined as shown in Tables 4.13, 4.14 and Figure 4.11. The flood peak at Buenos Aires fits reasonably with the basin average rainfalls. On the other hand, the correlation is very low for Rancho Apulco. These results indicate that the records at Buenos Aires are more reliable than those of Rancho Apulco.

Therefore, the records at Buenos Aires are used for estimating probability distribution of flood peak.

(2) Estimation of Probability Distribution of Flood Peak

Firstly, the probable flood peaks at Buenos Aires are estimated by applying the methods of Iwai, Gumbel and Pearson type III.

Return Period (years)	Probable Flood Peak (m^3/sec)		
	Iwai	Gumbel	Pearson Type III
5	364	404	372
10	465	513	484
20	569	616	602
50	712	750	769
100	825	851	905
200	945	951	1,050
1,000	1,247	1,183	1,429
10,000	1,747	1,515	2,081

Figure 4.12 shows the plotting position of the flood runoff peaks and the calculation results by the three methods above. No significant difference is seen among the three probability distributions and the plotting position. In this study, the results by Pearson type III will be used to for flood peaks of less frequent return period.

Secondly, the flood peaks of various return periods at the specific sites such as Soledad Reservoir and the proposed check dam site-B (to be described as an alternative countermeasure for reservoir sedimentation in Chapter 7) are estimated by using the following equation.

$$Q_2 = Q_1 \times (A_2/A_1)^{0.5}$$

where, Q_2 : probable flood runoff peak at specific site (m^3/sec)
Q_1 : probable flood runoff peak at Buenos Aires (m^3/sec)
A_2 : catchment area at specific site (km^2)
A_1 : catchment area at Buenos Aires (km^2)

The results are given below.

Return Period (years)	Probable Flood Peak (m^3/sec)		
	Buenos Aires (1,405)	Soledad Reservoir (1,460)	Check Dam Site-B (1,173)
5	372	379	340
10	484	493	442
20	602	614	550
50	769	784	703
100	905	923	827
200	1,050	1,070	959
1,000	1,429	1,457	1,306
10,000	2,081	2,121	1,901

( ): Catchment area in km^2

4.4.3 Sediment Load

(1) General

Sediment yield is defined as the total sediment discharge from a drainage basin, passing a cross section of reference for a specified period of time. It is generally expressed as weight, volume or depth per unit time (and per unit area). For Soledad Reservoir the sediment yield is defined as the mean annual sediment load entering the reservoir.

There are three sediment sources for Soledad Reservoir, (1) Apulco River, (2) Sontalaco River and other small tributaries draining directly into the reservoir, and (3) Xuicayucan River for trans-basin diversion through Canal Tunnel No. 1. The total drainage area is 1,830 km^2, which is broken down as follows.

1. Apulco River at Buenos Aires : 1,405 km^2
2. Sontalaco River at Sontalaco G.S. (25 km^2) plus other tributaries (30 km^2) : 55 km^2
3. Canal No. 1 (Xiucayucan River at Diversion weir, 280 km^2 and other small tributaries at diversion canal, 90 km^2) : 370 km^2

CFE initiated measurement of suspended sediment load in 1963 at the three gage stations at Buenos Aires, Sontalaco and Canal No. 1, probably immediately after the Mazatepec Project became operational.

(2) Sediment Sampling Equipment and Sampling Procedure

CFE has used a one liter bottle with a plastic cover having one centimeter hole to take sediment sample. The purpose of sampling is stated to take wash load only. It is assumed that the load is uniformly distributed throughout the depth of the water and a concentration rate at a few centimeters of the upper depth provides a representative value for the whole depth. However, this assumption does not appear to be correct because since 1985 the samples taken by this method have indicated a significant concentration rate of particles larger than 0.062 mm. In general, the concentration of fine sand and larger particles vary with depth as shown on Figure 4.13.

Sampling is done nearly on alternate days. Three samples are taken at the one-fourth, one-half and three-fourth of the width. The samples are combined and analyzed for estimating the total concentration of load.

(3) Analysis of Suspended Load

Before 1985, the total concentration was determined by evaporation method. The weighing balance has an accuracy of 0.01 gram. For a concentration of as low as 20 milligram per liter (mg/l), the accuracy is too low to measure sediment weight with a reasonable accuracy.

Since 1985, the samples are split into two portions: one retained on sieve No. 200 (designated as material in suspension) and the other passing through sieve No. 200 (designated as wash load). The analysis report includes time and date of sampling, gage height, discharge and weights of the two portions in grams. The CFE office, responsible for sampling and analysis, is not equipped with instruments to perform particle size analysis. However, the CFE Department of Experimental Studies, Office of Soil Mechanics Laboratories has equipment to analyze sizes finer than 0.062 mm.

Sediment load is computed in ton as the sum of daily wash load and daily load of coarse material in suspension. The concentration determined for each type of sediment is assumed to be a mean for the day and is multiplied with the daily flow in cubic meters. For the days for which the samples are not taken, the concentrations are linearly interpolated irrespective of the magnitude of intervening flows.

The monthly sediment load at the three stations computed by CFE are given in Tables 4.15 to 4.17 and are summarized below.

Gage station	Annual mean sediment load (cum/yr)	Catchment area (km^2)	Erosion rate (mm/yr)
Buenos Aires	450,000	1,405	0.320
Sontalaco	3,130	25	0.125
Canal No. 1	31,600	370	0.085

As discussed above, CFE currently computes the amount of fine and coarse material entering the reservoir. However, because of improper sampling procedure, the amount of coarse material estimated may be significantly less than the actual amount. Therefore, the above results are judged to be on a lower side. It is recommended to improve the field sampling equipment and procedures, and procedures for computations of sediment load used by CFE to obtain an accurate value for the total load entering the reservoir. This problem is discussed in Chapter 6 and Appendix B

(4) Concentration of Sediment Load

Sediment concentration of suspended load is as shown in Figure 4.14. Proportion of fine (<0.074 mm) and coarse (>0.074 mm) materials of suspended load are examined on Figure 4.15. The concentration and flow rates in the high flow period in 1988 are given in Figure 4.16. Comments on these data are described in Chapter 6.

(5) Particle Size Distribution of Bed Material Load

Review was made on the size distribution of bed material samples taken in 1989 on the Rio Apulco and Arroyo Sontalaco (Table 4.18 and Figure 4.17). Four samples of the reservoir bed materials taken by boring in 1987 are presented in Table 4.19. The location of the 1987 boring is shown in Figure 4.18. Particle size distributions of the reservoir bed material measured in 1992 and 1993 are presented in Tables 4.20, 4.21 and 4.22. The location for the sampling spots is shown in Figure 3.22.

4.5 Socio-economic Aspects

4.5.1 National Population and Economy

The total population of Mexico is 87.8 million in 1991 according to the United Nations statistics, which increased double from 42.7 million in 1965 over the period of 25 years. The average annual growth rate from 1985 to 1990 was 2.02%, indicating a decline from 2.98% for the 1975 - 1980 period and 3.49% for the 1965 - 1970 period.

The economy of Mexico took a turn for better since 1987, though the GDP (1985 base price) showed a negative growth of minus 3.6% in 1986. The annual growth rate of the GDP had increased from 1.6% in 1987 upto 4.4% in 1990. The recent economy is in a stable condition, while the growth rate dropped to 3.6% in 1991.

The growths of population and GDP are shown in Table 4.23.

4.5.2 Political Divisions/Towns

The project area is located primarily in the State of Puebla approximately 190 kilometers east of Mexico City. A small portion of the upper catchment area extends into the State of Tlaxcala. Downstream of the power plant, the Rio Apulco joins with the Rio Lajajalpan and other rivers to form the Rio Tecolutla which passes through the northern portion of the State of Veracruz on the way to discharge into the Gulf of Mexico.

The catchment area of the reservoir includes portions of ten municipalities of the State of Puebla and portions of two municipalities in the State of Tlaxcala. Table 4.24 presents information on the size of the municipalities in and adjacent to the Apulco Basin and the Xiucayucan basins, and the names and elevations of the principal community of each municipality.

4.5.3 Population of the Basin

The basin of the Apulco River upstream of Soledad Dam contains a population of approximately 107,000 with a population density of about 73 individuals per km^2. The basin upstream of the Xiucayucan diversion scheme contains a population of 63,000 with a population density of 226 individuals per km^2. Table 4.25 presents the population for each municipality in or adjacent to the catchment areas of the reservoir. Each municipality contains, in addition to the central community, many smaller communities. Table 4.26 presents population change of municipalities in the Apulco and the Xiucayucan diversion basins. A relatively larger population is seen in Tlatlauquitepec (42,447) and Zacapoaxtla (41,855). The population growth rates of Puebla and Tlaxcala from 1980 to 1990 are 19.1% and 42.9% respectively, of which the average rate is 22.6%. It is noted that the population growth of Tlaxcala is higher in recent years, though the population size is relatively small.

Overall, the region contains almost 394,000 inhabitants with a population density of 105 individuals per km^2. Some of the larger towns in the region (e.g., Libres, Zaragoza, and Tlatlauquitepec) are located along Highway 129 just outside of the Rio Apulco basin to the east, and others (Tetela de Ocampo, Cuetzalan del Progreso, and Zacatlan) are located to the north of the basin. Tlatlauquitepec and Zaragoza are the principal municipalities in the Xiucayucan diversion basin.

4.5.4 Regional Economy

Tables 4.27 shows that the level of employment for the entire male population in the municipalities in the Rio Apulco basin and in the Xiucayucan diversion basin, and that for the State of Puebla. Although agriculture accounts for only 44 percent of employment in the State of Puebla as a whole, the project study area is primarily an agricultural area. Of the employed males, 75 percent are employed in agriculture in the Rio Apulco basin. The percentage of individuals employed in other activities is also shown in Table 4.27. Employment in other activities in the project area is generally low except for an artisanal pottery industry in Zautra and relatively high employment in industry and commerce in Zaragoza, an urban area located along Highway 129.

4.5.5 Land Ownership

The land ownership in the State of Puebla (1988) is shown in Table 4.28. Nearly 80% of land in the region belongs to private ownership. In the Zacapoaxtla, Zaragoza and Zautla municipalities, group-owned land (EJIDO) is dominant.

4.6 Vegetation, Land Use and Erosion

4.6.1 Meteorology

Average rainfall, temperature, and potential evaporation for the five stations within the basin are summarized in Table 4.29. The distribution of the basin rainfall including the lower basin is shown in Figure 4.19. The stations with the lowest rainfall are Zautla (543 mm per year average) and San Francisco Ixtacamaxtitlan (584 mm per year average), located in the middle part of the basin. San Francisco Ixtacamaxtitlan also has the greatest potential evaporation at 1,488 mm per year; and potential evaporation exceeds rainfall in every month of the year. At Zautla, average rainfall exceeds potential evaporation only in the month of September. At the other stations, rainfall generally exceeds evaporation only during the months of June through September. At the Soledad dam site, however, rainfall exceeds evaporation in all months. At the dam site and along the eastern slopes of the mountains, rainfall is very high, normally exceeding 3,000 mm per year. However, towards upstream to the center of the basin, the rainfall diminishes to less than 600 mm per year, and potential evaporation exceeds 1,000 mm per year. At the higher elevations (Capuluaque and San Antonio), precipitation is somewhat higher.

4.6.2 Vegetation

Vegetation in the watershed is dependent on rainfall, land use and soil types. The rainfall pattern is reflected in the vegetation patterns and in the area of the dam and reservoir the vegetation is typically characteristic of humid areas with lush, dense growth of trees and shrubs covering the hillsides. In the upstream basin, the vegetation changes to stands of pine and oak intermixed with areas of herbaceous vegetation. The oak and pine, in turn, give way to more drought tolerant species and herbaceous species become dominant with cactus, maguey, and yucca becoming more abundant.

4.6.3 Soils

The soils of the watershed of the Rio Apulco upstream of the confluence of the Rio Zitlacuautla can be divided into three major soil groups and geographic areas. Between the confluence and Ocotzingo and upstream along the slopes of the watershed to approximately Tatenpango, the soils are listed as Cambisols. These soils tend to be a sandy loam near the surface and grade to a clayey loam with depth. In the river valley from Ocotzingo and Zautla to San Francisco Ixtacamaxtitlan, and in much of the watershed upstream from there, the soils are listed as Feozem. These soils range from a dark brown at the surface to a yellow or reddish brown at greater depth. Like the Cambisols, these soils tend to be of a sandy loam texture near the surface and grade to a clayey loam with depth.

Finally, in the area surrounding and north and west of Santa Maria Coyoltepec, the soils are listed as Regosoles. These soils are listed as being very similar to their origin which are poorly consolidated, soft volcanic tuffs. The texture of the soil is very fine, and vulnerable to erosion. Downstream of the confluence of the Rio Zitlacuautla, the soils are also Regosoles, but they are derived from well cemented welded tuff and form stable cliffs that are well vegetated and are not subject to extensive erosion.

Observations in the watershed show that the areas of the Cambisols appear to be moderately to well vegetated, frequently on moderate to steep slopes. These are also areas of moderate to high rainfall. Vegetation is much more sparse on the Feozem and Regosols in the upper part of the basin. In many locations the combination of easily erodible soil, low rainfall, and agricultural activities has resulted in wind and water erosion over extensive areas that has resulted in the loss of much of the organic matter in the soil, making natural revegetation difficult if not impossible.

4.6.4 Land Use

Erosion problems in the watershed, particularly in the river valley in the vicinity of Zautla, and in the area upstream of San Francisco, have been made worse by past and present land use practices in the area. Over grazing by livestock, particularly by goats, removes the vegetative cover and leaves the land susceptible to erosion by both wind and water. Likewise, the cultivation of the dry, easily erodible soils, particularly on slopes, destroys the binding capacity of the soils and leaves them susceptible to erosion.

Table 4.30 shows the land use patterns for the four principal municipalities in the Rio Apulco basin and for those in the Xiucayucan diversion basin. In both locations, seasonal agriculture is the predominant activity, with very little area under irrigation. Principal cultivated, annual crops grown in the area include corn, wheat, and beans. In addition, orchard crops such as apples, pears, and avocados (aguacate) are grown, and portions of the cactus and maguey are harvested for use.

However, one third of the Rio Apulco basin is subject to semi-intensive grazing or subsistence agriculture, frequently on steep slopes with shallow soils. Both of these activities can be very detrimental to the maintenance of adequate vegetative cover to retard erosion. The principal subsistence crop is corn. Livestock includes cattle, sheep, goats, turkeys and chickens. The practice of burning the old, dried vegetation, particularly on hillsides, to clear the area for subsistence cropping or to stimulate the growth of new tender shoots of grass for livestock also leads to the destruction of the organic content of the soil that helps to hold the soil in place and provides the source material for revegetation of exposed areas.

Finally, other activities in the basin, such as road construction, as presently occurring along the road from San Miguel Tenextatiloyan to Zautla, also contribute to potential erosion of the hillsides and the introduction of sediment into the Rio Apulco.

4.7 Water Use in the Apulco River Basin

In the upstream basin of the Apulco River, no intensive use of river water is seen except for a fish hatchery near Apulco which use water from a tributary on the right bank. No diversion of irrigation water nor active fishery is observed in the upstream and midstream reaches. The river does not appear to provide navigation service due to its steep and narrow topography, except on the reservoir area.

Although the downstream region of the Apulco River is a major rainfed agricultural area, only limited use of the water resources is noted downstream of the Project. Annual rainfall exceeds 1,500 mm per year and supplemental irrigation is not required. The largest direct use of water was an oil field water injection facility located at El Remolino. Other water extractions from the river are small, in the order of magnitude of 1,700 to 5,300 m^3 per day for municipal, industrial and agricultural use.

Other resource utilization in the downstream reach includes fishing and the extraction of sand and gravel from the river bed. Fishing activities are restricted to the 20 km estuarine areas of Tecolutla and Gutierrez Zamora where approximately 1,000 fishermen work in the river, its tidal tributaries, ands the Gulf north and south from the river discharge. The fishery resource is primarily marine and estuarine. No significant fishery was identified upstream of the estuarine area. Further upstream, the river is relatively shallow and movement of boats is blocked by rocks and gravel bars. The fishery resource in the Tecolutla area is characterized by various types of fish (mojarra, ostion, sabalo, and robalo), jaiba (crab), and camaron (shrimp).

The predominant resource utilization in and adjacent to the river is the extraction of sand and gravel. Extensive operations were noted in the river channel at several locations. To some extent, these operations are seasonal because they would be inundated during periods of high flows.

CHAPTER 5

ANALYSIS OF A RELATIONSHIP BETWEEN STORAGE CAPACITY AND ENERGY GENERATION AND AVAILABLE WATER FOR SAND FLUSHING

CHAPTER 5 ANALYSIS OF A RELATIONSHIP BETWEEN STORAGE CAPACITY AND ENERGY GENERATION AND AVAILABLE WATER FOR SAND FLUSHING

5.1 General

Soledad Reservoir has been affected by progressive sedimentation and will eventually end its useful life for flow regulation function, if no measure is provided. A major concern arises how much the storage capacity would influence the energy generation since generally less flow regulating function due to decrease of storage volume may lead to reduce energy production. Therefore, prior to establishing a rehabilitation plan properly against the sedimentation, it is essential to quantify effects on power generation by the reservoir sedimentation. Then, a simulation study was made to analyse a sensitivity of storage volume or reservoir operation levels to power and energy production.

Further, it is necessary to investigate the availability of water for diverting sediment laden water into the neighbouring basin and for flushing or sluicing sediment deposits downstream of the reservoir. Use of water for these purposes should be minimized since it may result in decrease of energy production of the power plant.

Through the above basic analysis, more feasible solutions against the reservoir sedimentation could be identified.

5.2 Effects of Storage Volume on Energy Output

5.2.1 General

This section presents the study results on effects on energy output of Mazatepec Power Station by sedimentation of Soledad Reservoir and alternative countermeasures against the reservoir sedimentation. The study examined the following four items which would affect energy output.

1) Reduction of reservoir storage capacity due to sedimentation
2) Change of reservoir operating levels
3) Alternative countermeasure by sediment diversion tunnel
4) Alternative countermeasure by construction of check dam

The study was conducted by using the simulation model which makes computation of reservoir water balance and energy output based on the reservoir inflow series and data of the project features. A general flow of the simulation is illustrated in Figure 5.1 and the outline of simulation model is briefly described below.

(1) Basic Data

Reservoir Inflow: The sum of daily runoff records at the three gauging stations, Buenos Aires, Sontalaco and Canal Tunnel No. 1, is used as the runoff series into Soledad Reservoir. The period of the runoff series is 29 years from 1963 to 1991.

Evaporation: Evaporation loss from the reservoir surface is estimated from the evaporation record at La Soledad station on monthly basis. Pan evaporation at the La Soledad is converted into the evaporation from the reservoir surface by multiplying a conversion factor of 0.70.

Reservoir Level-Area-Storage Curve: The reservoir level-area-storage data in 1962, 1977, 1988 and 1992 are selected for the study.

(2) Simulation Model

Reservoir Water Balance: Calculation of reservoir water balance is made by the following equation.

$$S_i = S_{i-1} + I_i - O_i - EV_i$$

where, S_i : storage
I_i : reservoir inflow
O_i : reservoir outflow
EV_i : reservoir surface evaporation

Power Plant Discharge: Power plant discharge is calculated by the following equation.

$$Q_p = Q_{max} \quad (O_i \geq Q_{max})$$
$$Q_p = O_i \quad (O_i < Q_{max})$$

where, Q_p : power plant discharge (m³/sec)
Q_{max} : maximum plant discharge $= P / (9.8 \times H_e \times \eta)$ (m³/sec)
P : maximum power output (= 220,000 kW)

H_e : effective head (m)

η : combined efficiency of generator and turbine.

assumed generator efficiency = 0.97

assumed turbine efficiency = 0.88

Energy Output: Daily energy output is calculated by the following equation.

$$E = P \times 24 \qquad (O_i \geq Q_{max})$$

$$E = Q_p \times (9.8 \times H_e \times \eta) \times 24 \qquad (O_i < Q_{max})$$

where, E : Energy Output (kWh)

5.2.2 Relationship between Storage Capacity and Energy Output

(1) Effects by Reduction of Effective Storage

The reservoir operating levels were initially set at El. 798.4 m for the normal high water level and El. 775 m for the minimum operating level as of 1962. Under this reservoir operating range, the four options of effective storage were examined by using the reservoir level-area-storage curves in 1962, 1977, 1988 and 1992. Power plant discharge, spillout and energy output were obtained by the simulation for the period of 29 years (1963-1991), and the simulation results are given below. The relationships between effective storage and energy output/spillout are shown in Figure 5.2.

Descriptions	Year			
	1962	1977	1988	1992
Effective Storage (mcm)	30.2	17.9	12.2	9.2
Annual Power Plant Discharge (mcm)	557.6	552.0	547.9	543.3
Annual Spillout (mcm)	16.4	22.4	26.6	29.2
Annual Energy Output (GWh)	615.9	610.6	608.8	604.3

The results show that annual energy output is reducing and annual spillout is increasing along with the reduction of effective storage. But when the effective storage and the energy output of 1992 are compared with those of 1962, reduction of the energy output is only about 2 % from 1962 to 1992 though the effective storage of 1992 is reduced to one-third of that of 1962. This indicates that the reservoir storage capacity within the above selected range would not affect much to energy output.

(2) Influence by Change of Reservoir Operating Levels

Soledad Reservoir was initially operated with the reservoir operating levels at El. 798.4 m for the normal high water level and at El. 775.0 m for the minimum operating level. The reservoir operating levels have been changed as the reservoir sedimentation has progressed, and the current operating levels are set at El. 804.5 m for the normal high water level and at El. 797.5 m for the minimum operating level. The simulation study was made to estimate change of energy output by the reservoir operating levels. This study also aimed at selecting proper reservoir operating levels which would influence alternative countermeasures such as dredging, rehabilitation of power intake, etc. to be provided against the reservoir sedimentation.

Reservoir operating levels for the simulation were considered on the conditions that the normal high water level be set at El. 804.5 m with the six options for the minimum operating level, which are set at El. 775, 780, 785, 790, 795, and 797.5 m. The simulations were made for four cases of the reservoir level-area-storage curves in 1962, 1977, 1988 and 1992 with each option of the reservoir operating levels. The results are shown in Table 5.1 and Figure 5.3 and are summarized below.

1) Energy output is more affected by available effective head than by effective storage. But difference of energy output is only a few percent between the minimum operating level at El. 775 m and that at El. 797.5 m. This indicates that energy output would not increase so much even if large amount of effective storage could be recovered.

2) The highest value of energy output appears to exist around the minimum operating level at El. 795 m.

5.3 Effect of Sediment Diversion

Sediment diversion tunnel which bypasses sediment flow to the neighbouring river basin is identified as one of the alternative countermeasures to decrease sediment inflow into Soledad Reservoir. This alternative would reduce the reservoir inflow due to diversion of river flow with sediment. Diversion of sediment flow is mainly operated during flood occurrence, but energy output might be decreased due to loss of water which could otherwise be used for power generation. The study was made in the following procedure to estimate energy output with the sediment diversion.

(1) Estimation of River Runoff at Sediment Diversion

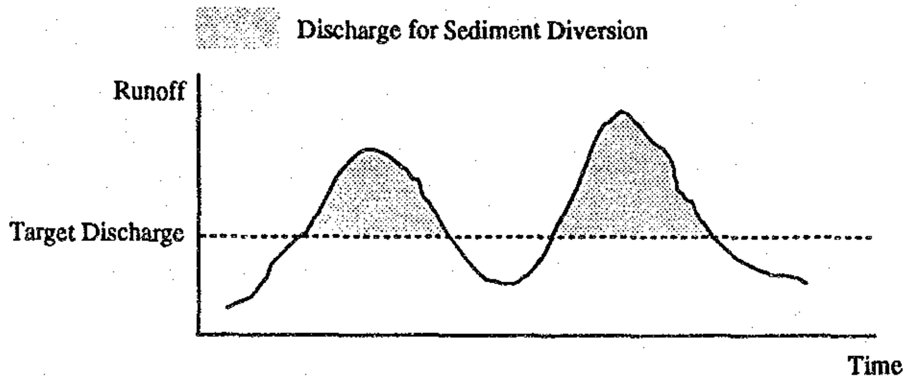
The identified sediment diversion point is located on the Apulco River near Huahuaxtla about 20 km from the reservoir upstream end. Runoff of the Apulco River at this point is estimated as follows based on the runoff records at Buenos Aires.

$$Q_d = Q_b \times (A_d/A_b) \times (R_d/R_b)$$

where, Q_d : river runoff at sediment diversion point (m^3/sec)
Q_b : river runoff at Buenos Aires (m^3/sec)
A_d : catchment area of sediment diversion point (km^2)
A_b : catchment area of Buenos Aires (km^2)
R_d : basin average rainfall at sediment diversion point (mm)
R_b : basin average rainfall at Buenos Aires (mm)

(2) Discharge for Sediment Diversion and Reservoir Inflow

It is assumed that discharge available for sediment diversion is river runoff exceeding a target discharge at which operation of the sediment diversion starts, and remaining runoff is released to downstream. This assumption is illustrated below.



Inflow into Soledad Reservoir with sediment diversion is estimated as follows.

$$Q_i' = Q_d' + Q_r$$

where, Q_i' : reservoir inflow with sediment diversion
Q_d' : outflow to the Apulco River at sediment diversion point
Q_r : inflow from the remaining catchment from sediment diversion point to Soledad Reservoir, $Q_i - Q_d$

Q_i : reservoir inflow without sediment diversion

Q_{i1} : river flow at sediment diversion point

(3) Computation of Energy Output

The optional target discharges at starting sediment diversion were selected at 30, 40 and 50 m^3/s. The reservoir inflow (Q_i') was estimated for each option. The simulation was made by using Q_i' for each option under the normal high water level at El. 804.5 m, the minimum operating level at El. 797.5 m and the reservoir level-area-storage curve in 1992. The results are given in Figure 5.4 and summarized below.

Target Discharge at Starting Sediment Diversion (m^3/sec)	Annual Energy Output (GWh)
30	625.8
40	627.9
50	628.7
(60)	(628.9)
(90)	(629.2)
without sediment diversion	629.2

The results show that less than 1% of energy reduction would occur by sediment diversion under the selected target discharges. Table 5.2 and 5.3 show the annual runoff volume and its occurrence frequency by days for sediment diversion. However, it is noted that the effectiveness for sediment diversion under these target discharges is not discussed here.

5.4 Effect by Large Check Dam

Construction of check dam on the mainstream of the Apulco River is a measure to prevent sediment inflow into Soledad Reservoir. Since a considerable large storage capacity of 27 mcm at maximum is possibly available at the proposed check dam site, depending on the dam height, flow regulation effect to the inflow into Soledad Reservoir would contribute to firm up the power plant discharge until the storage is filled up with sediment load. The following study was made to estimate the flow regulation effect by check dam.

(1) Inflow and Storage

Location of check dam is tentatively selected at site-B near Huahuaxtla. Runoff series at the check dam site-B is estimated by the same method described in Section 5.3. The normal high water level is set at El. 1,416 m tentatively if check dam with height of about 50 m is constructed at this site. Though further evaluation of dam height is required, 3 alternatives of effective storage capacity are selected for this study, assuming a drawdown range of 5, 10 and 15 m (depth to minimum operating level) for a check dam of 50 m in height. Storage capacities are estimated as follows.

Drawdown (m)	Effective Storage (mcm)
5	6.5
10	13.5
15	20.0

(2) Flow Regulation Effect

Flow regulation by check dam is estimated as follows. This concept is also illustrated in Figure 5.5.

Reservoir Water Balance

$S_i = S_{i-1} + I_i - O_i - EV_i$

where, S_i : storage

I_i : inflow

O_i : outflow

EV_i : evaporation

Calculation of Outflow

$O_i = Q_c + Q_s$ $(S = S_e)$

$O_i = Q_c$ $(0 < S < S_e)$

$O_i = I_i - EV_i$ $(S = 0)$

where, Q_c : regulated outflow (constant, m^3/sec)

Q_s : spillout (m^3/sec)

Inflow into Soledad Reservoir is calculated as follows.

$Q_i' = Q_d' + Q_r$

where, Q_i' : regulated reservoir inflow into Soledad Reservoir

Q_d' : regulated outflow to the river from check dam

Q_r : inflow from the remaining catchment from the check dam to Soledad Reservoir, $Q_i - Q_d$

Q_i : reservoir inflow inflow Soledad Reservoir

Q_d : river flow at the check dam site.

(3) Calculation of Energy Output

The simulation was made for the following cases.

Effective Storage of Check Dam : 6.5, 13.5, 20.0 mcm
Regulated Outflow (Q_c) : 5, 10, 15, 20, 25 m^3/sec

The results are shown in Figure 5.6. It is noted that the optional flow regulation rate of 20 m^3/sec gives the highest energy increment. The energy outputs under this case are given below.

Drawdown (m)	Effective Storage (mcm)	Annual Energy Output (GWh)
5	6.5	634.3
10	13.5	638.5
15	20.0	641.2
without check dam	-	629.2

Compared with the energy production without check dam, about 2% of energy increment is expected with the drawdown of 15 m. However, the effective storage under this case is 20.0 mcm which is equivalent to 74% of the gross storage of check dam for a height of 50 m. While sediment deposit volume at the check dam is roughly estimated at about 1.17 mcm per year. This means that long term effect of energy increment becomes smaller than the above value.

5.5 Sediment Flushing through Spillway

Since most of sediment inflow is brought by flood, sediment flushing through spillway during flood is considered as one of the measures to prevent sediment deposition in the reservoir. On the other hand, use of reservoir inflow for flushing sediment through spillway may cause reduction of power plant discharge. A simulation was therefore made to investigate a relationship between spillout and energy production.

It is assumed that sediment flushing is conducted when the reservoir inflow exceeds an optional discharge of 30, 40, 50 and 55.2 m^3/sec and the flow above the respective optional discharges is released through the spillway and the remaining discharge is used for power generation. The energy output was computed under the normal high water level at El. 804.5 m, and the minimum operating level at El. 797.5 m, using the reservoir level-area-storage curve in 1992. The simulation also includes the target discharge above the maximum plant discharge of 55.2 m^3/s though the maximum power output is limited to 220 MW in any case. The water not used for power generation is conserved in the reservoir or spilled out.

The results are given in Figure 5.7 and summarized below.

Target Discharge for Power Generation (m^3/sec)	Annual Energy Output (GWh)
30	560.8
40	588.9
50	604.9
55.2	610.6
(80)	(625.1)
(100)	(628.1)
without flushing	629.2

Table 5.4 and 5.5 show the discharge volume and its number of days for spillout. It is known that the energy production would decrease by 3 % under the optional discharge of 55.2 m^3/sec from the energy without flushing condition. However, it is apparent that if the optional discharge is set lower than 55.2 m^3/sec, the energy loss becomes more significant.

CHAPTER 6

ANALYSIS OF RESERVOIR SEDIMENTATION

CHAPTER 6 ANALYSIS OF RESERVOIR SEDIMENTATION

6.1 General

6.1.1 Reservoir Sedimentation

Reservoir sedimentation is inheritant to storage scheme. Depending upon the quantity and characteristics of the sediment carried by the river on which a reservoir is located, the useful life of the reservoir is eventually depleted. This phenomenon is mainly caused by an imbalance between the inflowing and outflowing sediment loads at the reservoir site.

As a stream enters a reservoir, the flow depth increases and the velocity decreases. This causes a loss in the sediment transport capacity of the stream and the deposition of the sediment moving as bed load and suspended load occurs. The quantity and pattern of sediment deposition depend upon the reservoir capacity, mean annual inflow, particle size distribution and quality of sediment, shape of reservoir and operation mode of the reservoir. Of these, the shape and operation mode of the reservoir have the greatest influence on deposition pattern.

Usually, the coarser particles deposit earlier in the reservoir. This process continues on a progressive scale towards some distance within the reservoir, and the flow velocity is sufficiently reduced such that all fine sand and larger particles are deposited. Silt and clay (particles size less than 0.062 mm) are carried further downstream and may partly deposit near the dam and partly be carried through, depending upon the operation of the outlet works.

In other words, a part of sediment loads which otherwise is transported downstreamwards by river water is trapped by the reservoir. On the contrary, in the downstream reaches, erosion, scoring and degradation often occur due to decrease of sediment supply from the upstream reaches. Major adverse effects in the downstream area are collapse of the foundation of river structures such as revetment, bridge, siphon, etc. and erosion and retrogradation of coast lands.

Then, it would be essential in planning storage schemes to select firstly favourable damsites which have less sediment inflow and to estimate the rate of sedimentation and the period of time before the sediment will interfere with the useful function of the reservoir. Provisions should be made for sufficient sediment storage in the reservoir at the time of design so as not to impair the reservoir functions during the useful or economic life of the project.

For the Mazatepec Project, emerging and possible adverse effects caused by the reservoir sedimentation would be;

- decrease of storage capacity which will degrade a regulation function of the reservoir.

- clogging of an entrance of the power intake by sediment deposition.

- shortening of the useful life of the hydraulic structures and equipment due to sand abrasion effects.

- decrease in flood discharging capacity of an approach bay or a discharge channel of the spillway due to sediment deposition.

- decrease of safety of the dam due to the increasing earth pressure by sediment deposits acting on the dam.

6.1.2 Basic Terms of Sediment Problems

For better understanding on sediment problems, some definitions on the technical terms are given below.

Sediment: a collective term meaning an accumulation of soil, rock and mineral particles transported or deposited by flowing water.

Sedimentation: a broad term that pertains to the five fundamental processes responsible for the formation of sedimentary rocks: (1) weathering, (2) detachment, (3) transportation, (4) deposition (sedimentation), and (5) diagenesis, and to the gravitational settling of suspended particles that are heavier than water.

Concentration of Sediment: the dry weight of sediment per unit volume of water-sediment mixture, that is, milligram per liter (mg/l).

Sediment Yield: the total sediment discharge from a drainage basin, passing a cross section of reference for a specified period of time, usually expressed as cum/yr, tons/yr or mm/yr.

Erosion: the loosening or dissolving and removal of soils or rock material from any part of a drainage basin.

Sheet Erosion: the wearing away of a thin layer of land surface.

Rill Erosion: the removal of soil by concentration of flowing water, with the formation of channels that are small enough to be smoothed completely by normal cultivation method.

Soil Loss: the quantity of soil actually removed by erosion.

Gully Erosion: the removal of soil by concentration of flowing water sufficient to cause the formation of channels that could not be smoothed completely by normal cultivation method.

Sediment Delivery Ratio: a measure of the decrease of eroded sediments, by deposition as they move from the point of erosion to any designated downstream location. All sediment eroded at a source do not reach to the measuring point. This, for a given drainage basin, is the ratio between the sediment yield at a measuring point to the total material eroded from the drainage system upstream from the measuring point, expressed in percentage.

Rate of Erosion: the rate at which soil is eroded from a given area, usually expressed in volume, weight or depth per unit area and time.

Geologic or Normal Erosion: the erosion in a drainage basin under natural or undisturbed conditions. Like all other drainage systems, the Rio Apulco receives sediment because of geological erosion and accelerated erosion over the normal erosion, caused by man's activities such as agricultural, grassing, urbanization, road construction, etc.

Suspended Load: the quantity of sediment passing a river cross section (usually expressed in tons per day) that moves in suspension, continuously supported in the water column by fluid turbulence. This includes both bed material in suspension and wash load.

Bed Load: the bed material moving on or near the stream bed by rolling, sliding and saltation (short jumps), usually expressed in weight per unit time.

Wash Load: the particles finer than the bed material and normally includes sediment particles smaller than 0.062 mm. CFE defines particle sizes less than 0.0724 mm as fine material. The quantity of wash load carried by a stream depends upon the supply rate rather than the hydraulic characteristics of the stream.

Bed Material: the sediment mixture of which the moving bed is composed.

Bed Material Load: the quantity of sediment in tons per day passing a stream cross section which consists of bed material moving both as bed load and moving in suspension along with the wash load.

Trap Efficiency: *the proportion of sediment inflow to a stream reach (or reservoir) that* is retained within that reach (or reservoir), computed as (inflow sediment volume - outflowing sediment volume) divided by (inflow sediment volume); positive values indicate deposition in the reach (or reservoir).

Source of sediment load is watershed overland erosion, sheet erosion, rill erosion, gully erosion, river bank erosion, and riverbed erosion. Sediment load entering Soledad Reservoir can be classified as (1) suspended load plus bed load in terms of transportation mechanism of sediment load or (2) bed material load plus wash load in terms of composition of sediment load.

6.2 Factors Affecting Sediment Production in the Basin

6.2.1 Topography and Geology

With regard to sediment production, the overland and channel slopes, and erosion potential of rocks and soils forming the basin are important. The basin is characterized by steep overland slopes typically between 30 to 50 percent with relatively flatter slopes (about 10 to 15 percent) near the banks of the Rio Apulco and major tributaries, and on some mountain tops.

The channel slopes of tributaries are steeper compared to the main river. Figure 4.1 shows the channel slopes of the main river and some tributaries. The overall channel slopes of small streams entering the check dams vary from about 6 to 40 percent. The overland slopes of the cultivated area along these streams vary between 5 and 20 percent.

The steep overland slopes are subject to sliding but no major evidence of sliding was observed during the field reconnaissance. Very steep slopes with overhanging cliffs are generally composed of hard rocks. Rill and gully erosions are seen at a few places.

The rainfall intensity recorded at Soledad Reservoir is as much as 35 mm for about half an hour (Table 4.3). Although this intensity may not be strictly applicable for the upper watershed, it does provide an indication that intensive rainfall of short durations would be also possible in the upper watershed.

The overland slopes in the upper watershed (above the confluence of the Rio Apulco and Rio Zitlalcuautla) are generally composed of young, soft rocks such as tuffs. The slopes are subjected to sheet erosion during rainy season. During the field reconnaissance of the watershed by road, washout from these slopes were observed, especially in the catchment of Rio Zitlalcuautla.

Surface soils varying in thickness from a few meters to several meters are visible at places in gullies and depressions on the slopes. These soils are easily erodible under rainfall. The source of fine particles (clay and silt) in the Rio Apulco are these soils. The fine particles give yellowish color in the river water. In comparison to this turbidity, the water of the Rio Zitlalcuautla is rather clear, probably with coarse particles.

6.2.2 Vegetation and Land Use

Vegetation in the basin decreases significantly from downstream to upstream watershed. However, near the watershed divide in the upper part of the basin, the land cover is somewhat more than that on the slopes. An area of about 440 km^2 immediately upstream from Soledad Dam is well forested with pine trees and other types of vegetation. The sediment yield from this area may be low as indicated by a yield of less than 0.2 mm per year as measured on Arroyo Sontalaco.

As shown in the rainfall pattern in the basin given on Figure 4.4 and the long-term mean monthly rainfall data in Table 4.2, the vegetation cover very much follows the rainfall pattern. Lack of vegetation on the overland slopes in the upper watershed is mainly responsible for sediment production. In this part of the basin CFE has constructed check dams.

Agriculture is practised in the nearly whole area where feasible, mostly in the vicinity of the banks of the river and its tributaries, and on relatively mild slopes. As stated previously, even these slopes may be as much as 20 percent. The farming on such slopes should be practised by contouring or terracing. Most of the farming is on the slopes except near the north-west divide of the basin. It is generally considered that the slope farming during fallow period, when rainfall occurs, can result in an erosion rate of over 2 mm per year. At a few places abandoned fields are observed. This shifting type agricultural practice removes the natural vegetation cover and replaces it with clear erodible land.

6.3 Reservoir Sedimentation

6.3.1 Survey of Reservoir Sedimentation

Since Soledad Reservoir started functioning in 1962, sedimentation has occurred in the reservoir. The reservoir operating levels have been modified from time to time as the sedimentation progressed. The current operating levels are set as given below.

Maximum operating level : 804.50 m
Minimum operating level : 797.50 m

CFE has estimated the loss of effective storage periodically by reconnaissance type or detailed reservoir sedimentation surveys. According to CFE, the 1992 survey is most reliable while the others are indicative of sediment progression in the reservoir. Some typical range lines used in the 1992 survey are shown in Figure 6.1. The elevation - surface area - storage capacity data based on these surveys were reviewed. The data considered reasonably accurate are presented in Table 3.3 and Figure 3.23. The reservoir sedimentation process and hydrological conditions in the basin are compared as shown in Figure 3.28. Table 3.3 indicates that the total sediment deposit in the reservoir for the period of 1962 to 1992 is about 40.355 mcm below EL. 804.50 m (maximum operating level). The deposit is about 37.256 mcm below EL. 797.50 (minimum operating level). The loss in reservoir storage capacity is shown in Table 3.4.

The reservoir bed profile surveyed in 1962, 1977, 1990 and 1992 are shown in Table 3.5 and Figure 3.24. The reservoir bed elevations near the dam indicated by the 1990 and 1992 profiles are higher than the invert of the power intake (EL.768.76 m) as shown in Table 3.6 and Figure 3.26. The invert of the low level outlet (EL. 750.0 m) appears to have been affected before 1977. All profiles show a progressive movement of the foreset of the deposit in the downstream direction, which is a natural phenomenon of sedimentation.

Because of power generation and some dredging operation in the past, the sediment deposit level in the vicinity of the power intake is lower than that near the non-overflow arch dam section. This is indicated by a reservoir cross section taken near the power intake, plotted on Figure 3.26. The location of this cross section is shown on Figure 3.25. These cross sectional profiles indicate that the sediment level at about 40 meters from the intake is about 772.0 meters. Within this distance of about 40 meters, the elevation drops to El. 768.76 m (crest of the intake). During the study period in 1993, the reservoir sedimentation near the dam and power intake was confirmed by using an echo sounder provided by JICA. The results are given in Figure 6.2 and 6.3. From these surveys, it is apparent that the sediment deposition is very likely to plug the intake during a large flood event or due to liquification of deposit materials near the intake in case of an earthquake event.

6.3.2 Estimate of Sediment Inflow into the Reservoir

The reservoir sedimentation survey indicates the following deposition pattern.

Year	Reservoir Volume (mcm) at EL. 804.5 m	Accumulated Deposition (mcm)
1962	58.753	
1977	28.828	29.925
1988	22.305	36.448
1990	21.171	37.582
1992	18.398	40.355

The survey conducted in 1992 reveals that the total reservoir deposition over the 30-year period is about 40.355 mcm, equivalent to 1.34 mcm per year on average.

In this study, the sediment yield into Soledad Reservoir is estimated assuming a trap efficiency of the reservoir based on the sediment surveys. The trap efficiency of a reservoir is defined as a ratio of quantity deposited sediment to be retained by the reservoir to the total inflow of sediment load into the reservoir. It is primarily dependent upon sediment particle fall velocity (a function of size and shape of sediment particle, the viscosity of water and chemical composition of water) and the rate of flow through the reservoir. Several empirical formula are used to know the trap efficiency and usually the trap efficiency is related with an index of storage capacity (C) to annual average river inflow of water (I).

Most commonly used trap efficiency curves are the Brune's and Churchill's curves. The Brune's curve is applied for large or normal ponded reservoirs. Here, the trap efficiency over the period of time was estimated using the Churchill's Curve for the capacity (C) - inflow (I) ratio developed from data in the TVA reservoirs. (Figure 6.4). The Churchill's curve is applied for settling basin small reservoirs, flood retarding structures, semi-dry reservoirs or reservoirs continuously sluiced. Because of small size of the reservoir mode of operation, and significant proportion of fine particles in water, the applied curve was considered most appropriate while other curves are also available.

The trap efficiency obtained is 0.73 to 0.61 as shown in Table 6.1 Excessive deposition occurred during the period of 1962-77, about 2.00 mcm per year. Using an estimated trap efficiency of 0.70, the sediment inflow could be about 2.9 mcm per year. For the period from 1977 to 1990, the sediment inflow rate is about 0.93 mcm per year (assumed trap efficiency of 0.64). This inflow rate increased to about 2.24 mcm per year for the period from 1990 to 1992 (trap efficiency 0.62). CFE considers that the sedimentation survey of 1992 is reliable and other survey data should not be used. Under this situation, the mean annual sediment balance is approximated as follows.

Inflow	= 2.00 mcm
Deposition	= 1.30 mcm
Outflow	= 0.70 mcm
Trap efficiency	= 65 percent

The average trap efficiency over the 30-year period was thus assumed to be 0.65. The total sediment load in terms of erosion rate over the whole watershed is about 1.417 mm/year. In other words, the sediment entering the reservoir is about 2.0 mcm per year on average, out of which about 1.3 mcm (40.355 mcm/30 yrs) is being deposited.

Estimate of sediment load by other methods including survey of deposits at the existing check dams and measurement of suspended materials are given below, though they cannot be compared simply due to difference in accuracy and investigation methods.

Estimate of Sediment Load

	Method	Annual mean sediment load (cu.m/yr)	Catchment area (km^2)	Sediment yield rate (mm/yr)	Erosion rate (mm/yr)
(1)	Survey of reservoir deposits (30 yrs from 1962 to 1992)	1,345,000	1,460	0.921	1.417[1)]
(2)	Survey of reservoir deposits (15 yrs from 1962 to 1977)	1,995,000	1,460	1.366	2.102[1)]
(3)	Survey of reservoir deposits (13 yrs from 1977 to 1990)	589,000	1,460	0.403	0.621[1)]
(4)	Survey of reservoir deposits (2 yrs from 1990 to 1992)	1,386,500	1,460	0.950	1.461[1)]
(5)	Survey of deposits at 25 check dams	70 ~ 35,500 (Total 184,000)	0.09 ~ 168.8 (Total 450.6)	0.08 ~ 8.33 (aver. 0.98)	0.41 ~ 10.2 (aver. 1.52)[2)]
(6)	Measured suspended solid at Buenos Aires (1965-90)	450,000	1,405	0.320	–
(7)	Measured suspended solid at Sontalaco (1965-90)	3,130	25	0.125	–
(8)	Measured suspended solid at Canal No. 1 (1977-90)	31,600	370	0.085	–

Remarks: 1) Erosion rate assuming the trap efficiency be 0.65.

2) Erosion rate assuming the trap efficiency be 0.35 to 0.82 depending on capacity-inflow ratio at each site.

CFE considers that the sediment surveys made prior to 1992 were of a reconnaissance level and therefore these are assumed to be indicative for mode of sedimentation. The survey data show that excessive deposition occurred during the first 15 years period from 1962 to 1977 (an erosion rate of about 2.10 mm per year). This rate decreased significantly (to about 0.62 mm per year) during the period from 1977 to 1990. This period corresponds to the time when about 25 check dams were functioning in different years. The 1992 survey indicated an erosion rate of about 1.46 mm per year for the 1990 to 1992 period. It is possible that the check dams retained a major part of coarse materials which otherwise would have settled in Soledad Reservoir. However, relatively low rainfall during this period may be also a major cause of low yield.

It is noted that a great difference is observed on the sediment yield between reservoir sedimentation survey and measurement of suspended materials at the gage stations. The measurement of sediment yield indicates a considerable lower value than the actual load entering the reservoir, giving only 20 to 25%. This difference may be caused by inadequate sediment sampling method and procedure as explained in Appendix B, though in general the sediment sampling procedure involves some technical difficulties and needs careful procedures to obtain the representative values of sediment loads with wide variety.

6.4 Characteristics of Sediment Load

6.4.1 Particle Size Distribution

Preliminary investigations show that the water entering the power intake carry particles of very fine sand, silt and clay. These particles would be non-damaging for the turbines. The coarse particles have settled upstream, but the foreset of this deposit is extending downstream. Eventually, the foreset will approach the intake area with a possibility of coarse material passing through the turbines.

CFE collected data on the particle size distribution of two bed material samples in 1989, one each on the Rio Apulco and on the Arroyo Sontalaco (Table 4.18) and four reservoir bed material samples taken by boring near the intake in 1987 (Table 4.19 and Figure 4.18).

Table 4.19 indicates that the top layer of sediment near the intake consists of mostly fine particles of size less than 0.062 mm but a small quantity of fine sand as well. This leads to the conclusion that some fine sand may have passed through the power intake. This is confirmed by the data given in Table 6.2, which shows the sediment concentration measured in the outflow from Unit No. 1.

JICA Study Team took five samples from the banks of Soledad Reservoir. These samples were analyzed for particle size distribution by the Soil Mechanics Laboratory of CFE. The results are shown in Table 4.20. The sample No. 1 is at a most upstream location and sample No. 5 is at a most downstream location as shown in Figure 3.22. The size distribution is coarser to finer from upstream to downstream.

Since 1985, CFE started computing the sediment load at Buenos Aires, Sontalaco and Canal No. 1 in terms of wash load and coarse material load. These data were reviewed and concluded that the wash load entering the reservoir during high flow months of June through October may be two to seven times higher than the coarse material in suspension, the larger ratio being for the Rio Apulco as shown in Figure 4.15. During other months, the wash load and coarse material in suspension are nearly equal.

In 1993, CFE conducted sampling, drilling and laboratory testing of the reservoir bed materials for obtaining information on particle size distribution of the sediment load and the variation in density of the deposit within the reservoir. Core drilling was made not only for obtaining the samples but also for detecting the original riverbed. The specifications used for this survey are presented in Appendix C.

The location of the sampling and drilling is shown in Figure 3.22. Geological section of Soledad Reservoir based on the above survey is given in Figure 6.5. Particle size distribution of the reservoir bed material obtained from the five drill holes and the upper fifteen surface sampling are presented in Table 4.21 and Table 4.22.

6.4.2 Density of Reservoir Bed Materials

The density of the reservoir bed materials (or commonly called the specific weight in terms of weight per unit volume) was analysed for the samples taken in 1993, though the samples were disturbed by drilling operation. Figure 6.6 shows smoothed density-depth relationships. Generally, the density increases with depth, though one exception is seen for sample No. 3 (probably due to analysis error). The density also increases from downstream to the upstream end. This is due to the fact that coarse and heavy materials settled in the upstream reaches.

6.5 Simulation for Reservoir Sedimentation Process

6.5.1 Simulation Models

Prior to establishing any countermeasure for the reservoir sedimentation, prediction of sedimentation process is required. Factors affecting the distribution pattern and deposition are reservoir capacity, mean annual inflow, mean annual sediment load, particle distribution of sediment, shape of reservoir and mode of reservoir operation. The reservoir shape and operation mode have the greatest influence to sedimentation process.

A number of computer models are available to simulate reservoir sedimentation process. The models are either one- or two-dimensional. The selection of a specific model is largely controlled by the basic data required for a model. It is generally not advisable to use a sophisticated two-dimensional model when the flow and sediment data are inadequate.

In the case of Soledad Reservoir, however, information/data in respect of quality and characteristic of suspended sediment and bed materials of the river and reservoir is not sufficient to use even one-dimensional model. Efforts were made to collect some limited data during the study period. After a review of this new information and based on assumptions for the missing information, the following two approaches were selected to simulate the sedimentation process in Soledad Reservoir.

(1) Empirical Area - Reduction Method
(2) HEC-6 Computer Model

A brief description of these approaches is given below.

(1) Empirical Area - Reduction Method

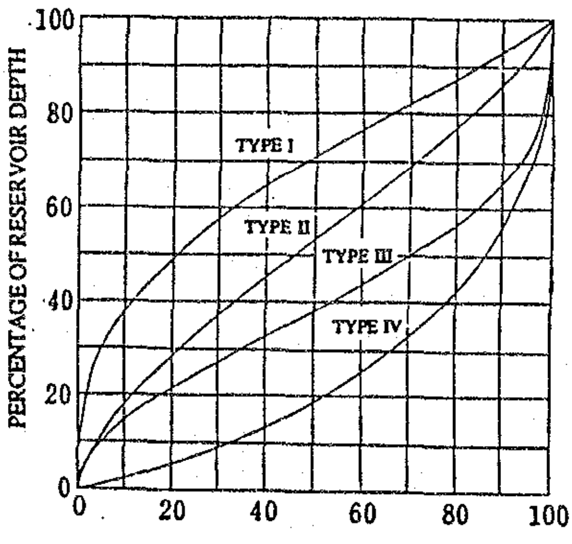
There are two methods developed by U.S. Bureau of Reclamation to distribute sediment in the reservoirs; "Empirical Area Reduction Method" and "Alternate Area-Increment Method." In both methods the sediment is distributed by depth and logitudimal profile starting from normal pool elevation. Most commonly used method is the Empirical Area Reduction Method.

Once the quantity of sediment (total sediment, sum of suspended and bed loads) has been established, the Empirical Area - Reduction Method*1, *2 can be used to estimate the distribution of sediments at any time. Although developed for large reservoirs, the method successfully been used on smaller reservoirs as well. The method was developed from extensive data gathered in the sedimentation survey of a number of reservoirs in the United States of America. The distribution of sediment is dependent upon, (U.S. Bureau of Reclamation, Design of Small Dams):

- Manner in which the reservoir is operated,
- Texture and size of deposited sediment particles,
- Shape of the reservoir, and
- Volume of sediment deposited

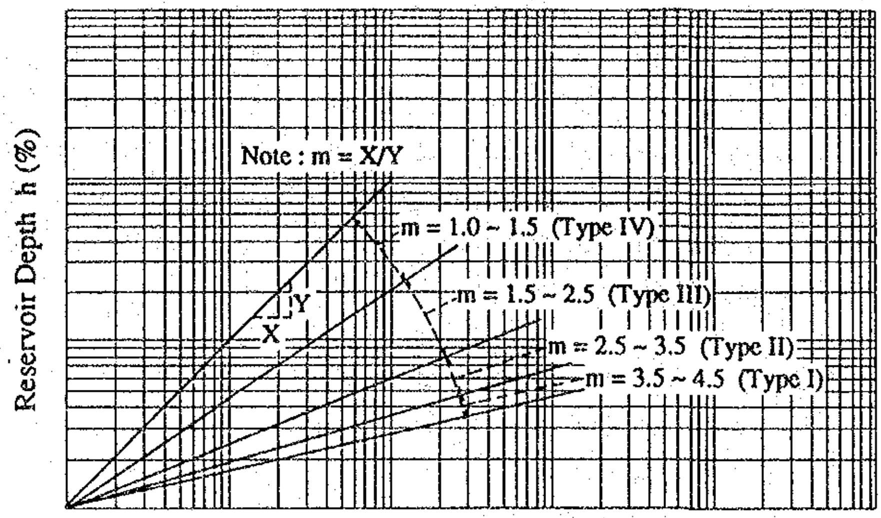
In the Empirical Area Reduction method, the reservoir is classified based on depth versus capacity relationship. The zero elevation is estimated at the dam. Sediment is distributed from this elevation to the selected normal pool elevation following the design curve derived for the reservoir. The shape factor was adopted as the major criteria for development of empirically derived design curves for distribution of the sediment. The shape of the reservoir is defined by the depth to capacity relationship, where "m" is the reciprocal of the slope of the depth versus capacity plot on a logarithmic scale. The classification of reservoirs made on this basis is given below.

Reservoir Type	Classification	"m" Value
I	Lake	3.5 - 4.5
II	Flood plain - foothill	2.5 - 3.5
III	Hill	1.5 - 2.5
IV	Normally empty	-



*1 Borland W.M., and C.R. miller, "Distribution of Sediment in Large Reservoir," Transactions, American Society of Civil Engineers, Vol. 125. Pt. 1, 1960.

*2 Lara, J.M., *"Revision of the Procedure to Compute Sediment Distribution in Large Reservoir's,"* Bureau of Reclamation, Denvor, Colorado, 1962



This method is best described in "Design of Small Dams", A Water Resources Technical Publication, U.S. Bureau of Reclamation, Third Edition, 1987. A computer program developed by the U.S. Bureau of Reclamation was used in this study.

(2) HEC-6 Computer Program

HEC-6 model was developed by U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California and is designated as "HEC-6, Scour and Deposition in Rivers and Reservoirs." The model is a one-dimensional numerical model of river mechanics that computes scour and deposition by simulating the interaction between the hydraulics of the flow and the rate of sediment transport. The model is designed to be used for the analysis of long-term river and reservoir behaviour (HEC-6, User's Manual, June 1991).

One-dimensional energy equation is used for computing water surface profile by the standard step method and Manning's equation. Expansion and contraction losses are included in the determination of energy losses. Sediment transport rates are calculated to grain sizes up to 64 mm. Sediment sizes larger than 64 mm which may exist in the bed are used for sorting computations but are not transported. For deposition and erosion of clay and silt sizes up to 0.0625 mm, Krone's method*3 is used for deposition and Ariathurai's adaptation of Parthenaides' method*4 is used for scour. The default transport option for clay and silt provides only deposition.

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*3 Krone, R.B., "Flume studies of the Transport of Sediments in Estuarial Shoaling Processes," Hydraulic Engineering Laboratory, University of California, Barkeley, CA, 1962.

*4 Parthenaides, E., "Erosion and Deposition of Cohesive Soils," Journal of of the Hydraulics Division, ASCE, March 1965.

The sediment transport function for bed material is selected by the user. Transport functions available in the program include the following (see User's Manual for reference to these functions).

(i) Toffalati
(ii) Madden's modification of Laursen relationship
(iii) Yang's stream power for sands
(iv) Duboys
(v) Ackers - White
(vi) Colby
(vii) Toffaleti and Schoklitsch
(viii) Meyer - Peter and Muller
(ix) Toffaleti/Meyar-Peter and Muller combination
(x) Parthenaides/Ariathurai and Krone for cohesive sediments
(xi) User specification of transport coefficients based upon observed data

The above methods except for the method (i) utilize the Colby method*5 for adjusting the sediment transport potential when the wash load concentration is high.

Armoring and destruction of the armor layer are simulated based upon Gessler's approach*6. For deposition or scour each point within the movable bed (i.e., the area which is allowed to vertically change due to sediment activity) is raised or lowered. The depth of deposition can be limited to the depth of the water at each time step.

6.5.2 Input Data for Simulation and Their Availability

Input data requirements for the Empirical Area Reduction Method and the HEC-6 Model are essentially water and sediment inflows, geometry of the reservoir and characteristics of the sediment. The data required for each approach and their availability are discussed below.

(1) General

For the Empirical Area Reduction Method, a computer program entitled "DISSED" was

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*5 Colby, B.R., "Practical Computations of Bed Material Discharge," Proceedings, ASCE, Vol. 90, No. Hy2, 1964.

*6 Gessler, J., "Beginning and Ceasing of Sediment Motion," Proceedings of the Institute of River Mechanics, Colorado State University, Fort Collin's, Colorado, June 1970.

used with the following input data:

- Maximum elevation up to which the estimated sediment is to be distributed;
- Elevation of the bottom of the reservoir; used for establishing a zero elevation after sedimentation
- Type of reservoir, either to be computed by the program or specified by the user; derived from the manner in which the reservoir is operated, the texture and size of deposited sediment particles, and the shape of the reservoir
- Elevation-area-storage capacity data; and
- Estimated sediment to be distributed for a specified period.

The above data were available from the reservoir sedimentation surveys and various drawings for the dam obtained from CFE.

Input data for HEC-6 computer model are grouped into four categories:

- Channel (reservoir) geometry
- Sediment inflow and characteristic, and characteristic of bed material
- Hydrologic data
- Special commands for computational procedures

(2) Geometric Data

Geometric data include: cross sectional profiles, reach lengths between successive cross sections, Manning's roughness coefficient, movable bed portion of each cross section, depth of sediment material at each cross section which may be subjected to scour and deposition, and locations of tributaries joining the main stream. The Apulco River was designated as the main stream and the Sontalaco River and Canal No. 1 were considered as the tributaries.

A total of 28 cross sections were selected from the 1962 Reservoir Sedimentation Survey. The locations of the selected cross sections are shown on Figure 6.7. The coordinates of the cross sections indicating the 1992 sedimentation conditions are given in Table 6.3.

The reservoir cross sections prior to 1962 (before the construction of the Project) were not available except for a river bed profile surveyed in 1962 (see Figure 3.24). These cross sections were required to simulate the sedimentation process and match the simulated longitudinal profiles with the surveyed profiles. The cross sectional profiles as of 1962 at the above 28 locations were estimated through trial and error procedure discussed under sub-section 6.5.3.

The length between two successive cross sections was measured on the sedimentation survey map of 1:500 scale prepared by CFE. Manning's roughness coefficients were estimated during field reconnaissance and reservoir bed material sampling. The movable bed portion at each cross section was determined based on the sedimentation surveys. The depth of sediment at each cross section was estimated using the results of reservoir bed sampling by drilling process in 1987 near the power-intake (Figure 4.18) and by drilling at five locations in the reservoir in 1993 (Figure 3.22).

(3) Sediment Inflow and Characteristics

About 95 percent of the total sediment entering to the reservoir is brought in by the Apulco River as measured at the Buenos Aires gaging station. Therefore, an attempt was made to redefine the sediment transport in the Apulco River.

A sediment rating curve was developed for the Apulco River using the sediment concentration data of 1988 and 1989 collected and analyzed by CFE. Because of wide variation of sediment concentrations with flow rates, the curve was developed through a trial and error procedure such that under the long-term flow duration curve, the long-term sediment transport was approximately same as indicated by the deposition in the reservoir.

The sediment rating curve is given in Figure 6.8. The extrapolation of this curve for higher flows was first made by limiting the sediment concentration up to 3 percent. Secondly, the sediment transports in the Apulco River for high flows were estimated using the Modified Einstein Procedure*[7,8]. Figure 6.9 shows the extrapolated curve with data points based on Modified Einstein Procedure. Sediment rating curves were also developed for the Sontalaco River and Canal No.1 as shown on Figure 6.10. These curves are based on sediment sampling performed in 1988 and 1989. It should be noted that although the curves were based on the sampling of suspended sediment, these were assumed to represent the total sediment transport entering the reservoir. This was because of some uncertainties in the data and extrapolation procedures.

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*7 Colby, B.R., and C.H. Hembree, "Computations of Total Sediment Discharge, Niobrara River near Cody, Nebraska "USGS Water Supply Paper 1357, 1955.

*8 U.S. Bureau of Reclamation, "Step Method for Computing Total Sediment Load by the Modified Einstein Procedure," Denvor, July 1955.

The HEC-6 Model required:

- Water discharge and sediment transport relationships for the streams entering the reservoir,
- Gradation of sediment in suspension and at streambed;
- Percent of sediment load for each particle size in the gradation curve; and
- Information on specific weights of sediment

Proper values of suspended sediment concentrations could not be obtained during the study period because CFE does not have any depth-integrated sampler. The laboratory analysis procedures are appropriate. During the study period, the flows in the rivers were low and relatively clear. Therefore, no data were available for the gradation of suspended sediment for all flow seasons.

Bed material sampling efforts are discussed in Appendix B. The reservoir bed material samples taken by CFE in January - February 1993 are given in Tables 4.21 and 4.22. After reviewing these date, approximate particle size distribution curves were developed for the inflowing sediment and for the reservoir bed material. These distribution curves are shown on Figures 6.11 and 6.12.

HEC-6 also required percent of sediment load for each particle size. For the Apulco and Sontalaco rivers, these percentages were estimated using the Modified Einstein Procedure and then somewhat revised during the simulation process as discussed in sub-section 6.5.3. The input data included:

- Steady state flow rates
- Area, top width, equivalent depth, sampling depth, suspended sediment concentrations and corresponding flow rates, and water temperature
- Hydraulic slope
- Particle sizes of bed material corresponding to 35 and 65 percent fine, and
- Particle size distributions of bed and suspended materials

The particle sizes in millimeter for the Modified Procedure and HEC-6 Model are defined as:

1.	Clay	.002 - .004		Coarse sand	.500 - 1.000
2.	Very fine silt	.004 - .008		Very coarse sand	1.000 - 2.000
	Fine Silt	.008 - .016	4.	Very fine gravel	2.0 - 4.0
	Medium silt	.016 - .032		Fine gravel	4.0 - 8.0
	Coarse silt	.032 - .062		Medium gravel	8.0 - 16.0
3.	Very fine sand	.062 - .125		Coarse gravel	16.0 - 32.0
	Fine sand	.125 - .250		Very Coarse gravel	32.0 - 64.0
	Medium sand	.250 - .500			

To define area, top width, equivalent depth, etc., discharge measurement notes were obtained from CFE and relationships between discharge and other parameters were developed. Figure 6.13 shows discharge - area and discharge - velocity relationships for the Apulco River at Buenos Aires. Old surveyed cross sections at Buenos Aires, Sontalaco and Canal No. 1 were also obtained from CFE to help in developing these relationships.

In summary, the sediment data input for HEC-6 included:

- Discharge-sediment transport relationship for the Apulco and Sontalaco rivers and Canal No. 1
- Percent of sediment load in each size corresponding to each discharge, and
- Bed material particle size distribution at five locations in the reservoir shown on Figure 6.11.

(4) Hydrologic Data

Daily hydrographs for the period from January 1963 to December 1991 obtained from CFE were divided into sequences of discrete steady flows of one to more days durations (HEC-6, USER'S MANUALS). The reason for generating these sequences for each stream was to minimize the number of time steps for simulation for a given time period, thus minimizing computer time. However, in generating these sequences, care was taken so that high flows are not averaged with low flows, because the sediment transports during high flows are significantly higher.

(5) *Special Commands for Computations*

Special commands and other specific data for computations were adapted as recommended in the User's Manual for HEC-6. In view of significant amount of fine material in suspension, Toffalati*9 procedure was used for sediment deposition and transport through the reservoir.

6.5.3 Prediction of Sedimentation Progression

For the prediction of future sedimentation conditions, the following basic assumptions were made:

- Sediment deposit rate will be about 1.4 mcm per year based on the incremental deposit rate estimated from the sedimentation surveys of 1990 and 1992. This rate is assumed to be representative of future conditions. It is less than the deposit rate of 1963-77 period and more than the rate of 1977-90 period.
- Sediment inflows will be represented by the discharge - sediment transport relationships based on 1988 - 89 sampling as discussed under sub-section 5.4.2, and improved if necessary during simulation:
- Particle size distributions of suspended sediment and bed material will be as discussed under sub-section 6.5.2. However, the distributions may be revised if required during the simulation.

Empirical Area-Reduction Method and HEC-6 Model were used to simulate future sedimentation. The prediction by each model is discussed below.

(1) Empirical Area-Reduction Method

Once the deposition rates are established the basic requirement of the method is to define the reservoir type. The type of reservoir was determined through a sort of calibration process as discussed below.

Elevation-area-storage capacity data of the reservoir in 1962 was assumed as the initial condition. The sediment deposit over the period from 1963 to 1992 is about 40.355 mcm. Using this deposit and 1962 capacity data, the 1992 capacity data were estimated using the computer program. The reservoir types were specified from I to III. The Type II best predicted the 1992 capacity data. The simulated points plotted on the 1992 capacity curve are shown on Figure 6.14. The sedimentation level is at EL. 780 m,

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*9 Toffalati, F.B., "A Procedure for Computation of Total River Sand Discharge and Detailed Distribution, Bed to Surface," Committee on Channel Stabilization, U.S. Army Corps of Engineers, November 1966.

about 5 m higher than in 1992 (EL. 775 m).

At El. 805.0 meters, the 1992 reservoir capacity is about 19.23 mcm. Using an annual deposition rate of 1.41 mcm for the Type II, the future sedimentation levels were predicted. It was estimated that in around year 2000, the reservoir will be practically filled. The sediment level near the dam will be about 801.3 meters compared to the 775.0 meters in 1992. This level is judged to be somewhat on the high side. Assuming a 5 m difference, the level may be assessed to be EL. 796 m.

(2) HEC-6 Model Prediction

It should be realized that HEC-6 is a one-dimensional continuous simulation model using a sequence of steady flows and does not simulate lateral distribution of sediment load across a cross section. The reservoir storage effect is not considered, that is, outflow is set equal to inflow. Deposition or scour in the reservoir is caused due to changes in the velocity.

Although sufficient basic data were not available to calibrate the model, attempt was made to check through simulation that:

- The downstream boundary condition is appropriate, because fluctuations in the reservoir may affect the deposition/scour rate;
- Hydraulic parameters and procedures of sediment transport through the reservoir are properly selected;
- Locations of cross sections, their profiles and distance between them properly represent the reservoir volume; and
- Sediment transport relationships and characteristics of suspended and bed materials are properly developed.

The reservoir bed profiles surveyed in 1977, 1990 and 1992 and the sediment deposited during the 1963-77, 1977-90 and 1990-92 periods were reviewed. The deposit during the 1977-90 period was too low, probably due to functioning of small check dams built in the upper watershed of the Apulco River. The period of 1963-77 was judged to be representative of the reservoir sedimentation process to simulate the 1977 bed profile and match with the surveyed profile.

Simulation of 1977 Reservoir Bed Profile:

Sediment transport relationship and characteristics discussed under sub-section 5.5 were used. A constant downstream boundary elevation of 798.4 meters (maximum reservoir level during 1963-77) was used for all flows. This assumption is appropriate for the Mazatepec Project because of small storage. The major problem was to obtain appropriate reservoir cross section and profiles under the 1962 conditions. These data were assumed. Through trial and error procedures and simulation of a number of computer runs, it was realized that the simulation of the 1977 bed profile was not possible because of non-availability of the 1962 cross sections. Therefore, additional simulation efforts were confirmed to match the quantity of sediment deposit and the trap efficiency during the 1963-77 period.

The following results were obtained:

1977 condition at El. 798.4

	Surveyed Data	Simulated Data
Sediment inflow, mcm	-	40.9
Sediment deposit, mcm	28.7	25.7
Trap efficiency by Churchill's curve	0.70	0.63

The above results are judged to be reasonable considering the input data uncertainties.

Prediction of Sediment Level:

Because of relatively small reservoir volume as of 1992, the sediment prediction was made for the year 2000. A period of eight years from 1981 - 91 excluding 1983, 1986 and 1987 (low flow years) was selected. During these years. the flows of the Apulco River exceeding about 25 m^3/s were assumed to carry significant sediments and sequence of these flows was used. The downstream boundary condition was represented by a constant reservoir level of 802.0 meters (estimated to be the dominant level for the period 1981 - 91). The river cross sections of 1992 conditions were used. The predicted reservoir bed profile is shown on Figure 6.15 in comparison with the 1992 profile.

The figure indicates that the foreset will move towards the intake by about 900 meters in about 8 years. The sediment level near the non-overflow section of arch dam will be about 788.5 meters. Near the upstream end, the thalweg will rise by about 10 meters.

6.5.4 Conclusions

The two approaches used for the prediction of future sedimentation provide an indication of the reservoir bed level around the year 2000. A level of 801.3 meters predicted by Empirical Area-Reduction-Method may be somewhat on the high side because of simplified assumptions in the method. The HEC-6 prediction of an elevation of 788.5 meters may on the low side because the model does not consider the storage effect. A mean elevation, about 792 meters, averaged on the predictions made by two methods is judged to be a reasonable value of sediment level, about 100 meters upstream from the non-overflow section of the dam. At the spillway, the sediment level will not exceed EL. 789.5 m, the spillay crest. At the power intake, the elevation will be less because of flow through the intake. Progress of reservoir sedimentation is illustrated in Figure 6.16.

Both methods indicate that in about 8 years the reservoir will be practically filled with sediment unless any appropriate measure is provided or dredging is done periodically. It should be realized that the predicted level is indicative of the average sediment inflow conditions for the reservoir and should not be interpreted in absolute terms. It has been observed on many rivers that quite often a high flow period of a few months or even a large single flood event can bring sediment to a reservoir about 3 to 5 times higher than the mean annual sediment inflow. This is somewhat verified by a 10-day flood of September 1 to 10, 1988 that transported about 1.3 mcm of suspended sediment, compared to the annual transport of 1.9 mcm in 1988.

CHAPTER 7

ALTERNATIVE COUNTERMEASURES FOR RESERVOIR SEDIMENTATION

CHAPTER 7 ALTERNATIVE COUNTERMEASURES FOR RESERVOIR SEDIMENTATION

7.1 Basic Approach for the Study

Countermeasures for the reservoir sedimentation were studied not only for conservation of the reservoir storage capacity for flow regulation but also for protection of hydraulic turbines and other facilities from potential abrasion effects by sediment loads. Decrease of storage volume will lead to abandonment of effective and reliable power generation, that is, failure of potential maximum output operation and waste of energy. Further, sedimentation was studied in terms of not only its volume but also its particle size or gradation.

Countermeasures against the sedimentation were studied under the following basic concepts, taking into account possible phasing for implementation.

(1) Soil conservation in the upper basin to minimize sediment production.
(2) Sediment arresting in the upper and/or middle reaches to prevent sediment loads from entering the reservoir.
(3) Bypassing of sediment loads downstream of the dam or diversion into another basin.
(4) Sluicing sediment deposits in the reservoir through the outlet and /or spillway.
(5) Removal of sediment deposits from the reservoir by mechanical method such as dredging.
(6) Provisions and improvements at the power intake to prevent sediment loads from being entrained into the waterway.
(7) Use of anti-erosion material for hydraulic turbines and appurtenant facilities and erosion reducing operation.

In addition to the above structural measures, modification of the present operation rule of the reservoir, that is, change of operation water levels and drawdown range was reviewed, while it was already tried by CFE.

7.2 Alternatives Countermeasures Identified

The alternative structural plan elements were identified in the Progress Report dated December 1992. After a brief review of the Progress Report, CFE identified additional alternatives for consideration in the study.

The following alternatives were identified in the study.

(I) Rehabilitation of Low Level Outlet:

1) Alternative A : Rehabilitation of existing low level outlet in arch dam [Figure 7.1]
2) Alternative B : Construction of new low level outlet at a higher elevation through arch dam [Figure 7.2]
3) Alternative C : Conversion of the existing power intake into a low level outlet [Figures 7.3 and 7.4]

(II) Construction of New Power Intake:

1) Alternative D : Construction of new power intake adjacent to the existing power intake [Figures 7.5 and 7.6]
2) Alternative E : Construction of new power intake just upstream of existing power intake [Figures 7.7 and 7.8]
3) Alternative F : Construction of new power intake just upstream of existing power intake [Figures 7.9 and 7.10]

(III) Construction of Other New Facilities:

1) Alternative G : Construction of new settling basin [Figures 7.11 and 7.12]
2) Alternative H : Channel improvements upstream of spillway [Figure 7.13]
3) Alternative I : Construction of new check dam [Figure 7.14, 7.15 and 7.16]
4) Alternative J : Construction of sediment diversion tunnel [Figure 7.14]

(IV) Removal of Reservoir Sediment Deposits

1) Alternative K : Removal of deposits by pump dredger and transport to spoil area

(V) Alternatives identified by CFE:

1) Alternative L : Replace existing outlet in arch dam with a larger sized low level outlet (with option of installing a new generating unit to utilize discharges) [Figure 7.17]
2) Alternative M : Construct new low level outlet directly below the arch dam abutment foundation with new intake between arch dam and existing power intake [Figure 7.18]
3) Alternative N : Construct a new low level outlet through the arch dam

below the existing outlet with installing a new generating unit [Figure 7.19]

4) Alternative O : Construct a new dam downstream of the existing arch dam to create a pool between the dams which would be supplied by water from existing Tunnel No. 1 by the construction of a new water conveyance facility. Construct a new power intake in the new pool - use the existing reservoir to collect sediment [Figure 7.20]

(VI) Erosion-reducing Measure for Turbines:

1) Alternative P : Limitation of partial load operation and use of less jet nozzles

2) Alternative Q : Use of Digipid governor

It is noted that these structural and non-structural elements may be combined with other elements or may be used independently. Specific features of the alternatives are described below.

Alternative A (Rehabilitation of existing low level outlet):

Alternative A utilizes the existing low level outlet works. The existing facility includes an intake structure with trashrack, a steel conduit through the arch dam and a 1.88 m diameter gate valve and Howell Bunger valve. The centerline elevation of the outlet is at EL. 750 m, approximately 25 m below the current sediment level and about 40 m below the spillway crest elevation (corresponding to the minimum level that the reservoir can be lowered during construction). Alternative A includes the following:

- Removal of sediment from the intake area
- Dewatering the area during construction
- Demolition and removal of the existing intake
- Construction of a new intake with invert at EL. 780 m and top elevation at EL. 806.5 m
- Construction of a concrete conduit leading from the intake to EL. 750 m
- Replacement of the existing Howell Bunger valve designed for greater vibrations due to sediment sluicing

Alternative B (New low level outlet at higher elevation):

Alternative B considers the construction of a new low level outlet through the arch dam with invert at EL. 780 m. This alternative abandons the existing low level outlet in favor of a more constructable facility. The facility would be constructed above the current sediment level, and 30 m above the existing outlet.

Alternative C (New low level outlet in existing intake):

Alternative C considers the conversion of the existing power intake into a low level outlet. This alternative would include the construction of a new outlet tunnel that would start approximately 80 m from the existing tunnel intake and exit into the river channel downstream of the arch dam. A segment of the existing power tunnel would be permanently plugged immediately downstream of the new outlet tunnel connection. Only minor modifications of the existing intake are required, such as replacement of the existing trashrack with a new trashrack designed with larger openings to facilitate the passage of sediment and smaller-sized debris. A new power intake would need to be constructed with a new tunnel segment connecting to the existing power tunnel immediately downstream of the proposed tunnel plug.

Two alternatives for the outlet works of the pressure tunnel will be considered. The first alternative will consider the extension of the tunnel to the river channel level with the construction of a stilling basin to dissipate energy. The second alternative will consider the tunnel outlet at a higher elevation (Approx. EL. 750 m) with an outlet structure designed to deflect discharges up (for energy dissipation) and then into the river channel. The latter alternative is selected because of lower cost and in consideration of the existing spillway flip bucket.

Alternative D, E and F (New power intake):

Alternatives D, E, and F consider the construction of a new power intake with invert at EL. 785 m. Modification of the existing power intake to accommodate a higher intake level was not considered to be a practical option. This conclusion was reached based on the design of the existing structure and the difficulty in dewatering the area for construction of a new facility.

Alternative D considers the construction of a new power intake adjacent to the existing intake with a new tunnel segment connecting to the existing power tunnel.

Alternatives E and F consider the construction of a new power intake just upstream of the left gravity wall and approximately 30 to 40 m away from the existing intake structure. These two alternatives are identical except for orientation of the structure. These alternatives greatly simplify construction compared to Alternative D by allowing for a convenient area for cofferdam construction and dewatering in the intake area. Alternative F would include backfilling an area around the intake to EL. 806.5 m to provide access from the dam crest and to close off the existing channel along the gravity wall.

Alternative G (Setting basin):

Alternative G considers the construction of a new settling basin to exclude sand particles above a certain harmful size (0.5 mm or greater, depending on further analysis) from entering the power tunnel. Based on preliminary estimates of total annual sediment load and grain size distribution, a preliminary size of the settling basin is estimated to be 25 m wide by 100 m long. The basin shown on the attached sketch was sized for a sediment retaining capacity of about 6,500 m^3 which would require sluicing at least three times per year. As shown on the sketch, an overflow weir and tunnel intake structure would be provided at the downstream end of the basin. A new tunnel segment would be constructed to connect to the existing power tunnel. The invert of the sluice outlet in the settling basin would be located several meters higher than the spillway crest elevation. The existing power intake could be converted into a low level outlet or closed off using the existing gate.

The sluicing of sediments from the basin into the reservoir would not be permitted since the spillway is located a considerable distance away from the settling basin. Sluicing sediment from the settling basin would be accomplished by the construction of a shaft and tunnel leading from the basin to the river channel downstream of the arch dam. However, it is noted that unless a permanent dredging scheme for the reservoir be developed, the silt level acting against the arch dam will be rising thus increasing the loads acting against the dam to unacceptable levels.

Alternative H (Channel improvement upstream of spillway):

Alternative H considers excavation of a channel directly upstream of the spillway to improve flow conditions to the spillway which may increase sediment flushing efficiency of the spillway. Consideration is also being given to excavating the island adjacent to the left side of spillway to improve flow conditions.

Alternative I(Check dam):

Soledad Reservoir has effectively stopped the movement of sediment in the river for 30 years. Construction of a facility with significant storage capacity upstream of the present reservoir could, likewise, stop further sediment accumulation into Soledad Reservoir. The costs and benefits to be derived from such a project would have to be carefully studied, but it would seem that for the short to intermediate term such a facility may offer a cost effective way of retaining the viability of the Mazatepec Project.

Some potential locations for new check dam(s) are shown on Figure 7.14. The most efficient location for the check dam would be on the main Apulco River downstream of the confluence of the Zitlalcuautla River to intercept sediment loads from both rivers. Alternative locations upstream of the confluence on one or both rivers will be considered.

The construction of new check dams on tributaries to the Apulco River or rehabilitation of the existing check dams are not considered to be practical alternatives. The reason for this is that they cannot sufficiently reduce the amount of sediment entering Soledad Reservoir.

A preliminary evaluation was performed to select the preferred large check dam site. The locations of the check dams are shown on Figure 7.14. For this evaluation the sites have been labeled starting from the downstream end as follows:

Site A - near Cuatapehual
Site B - near Huahuaxtla
Site C - near Cuauximaloyan

The riverbed gradient at these sites is approximately 1/120. Ground surface profiles (see Figure 7.15) along the dam centerline were developed for each of the three sites. Elevation-Area-Volume Curves are attached for reference (see Figure 7.16). The evaluation was made for an reservoir storage capacity of 27,000,000 m^3 (assuming the deposits to be trapped for 30 years in Interim Report).

It should be noted that the maximum dam heights for the three dam sites considered range from 45 m to 97 m. Due to the relatively high dam heights considered and the overall structural stability requirements, it is recommended that the Rolled Compacted Concrete (RCC) dam option be considered. The masonry dam option is considered for further consideration for the smaller dams.

For evaluating the high dam schemes, the RCC dam foundation was assumed to be 5 m below the ground surface. The crest of the dam was computed as 9 m above the normal maximum water surface level which is based on preliminary computations of required spillway capacity and includes 1.5 m of dam freeboard. The pertinent data for each site is given below.

LARGE CHECK DAM ALTERNATIVE - PERTINENT SITE DATA

	Site A	Site B	Site C
Storage Capacity, $m^3 \times 10^6$	27.0	27.0	27.0
Normal Max. Water Surface El.	1168	1416	1610
Dam Crest El.	1177	1425	1619
Dam Crest Length, m	225	278	433
Max. Height above Riverbed, m	97	45	69
Approximate Riverbed El.	1080	1380	1550
RCC Dam Volume, m^3 (*)	346,400	156,400	328,000
RCC Dam Volume, m^3 (**)	-	194,000	-

(*) Based on dam excavation 5 m below ground surface.

(**) Based on dam excavation 15 m (maximum) below ground surface.

The dam volume required at Site B is less than 50 % of the dam volume required at Sites A and C. Therefore, the cost of the dam would be about 50 % less at Site B than at the other two sites. In addition, construction of a dam at Site B is more favourable because of the lower dam height, shape of the valley profile, and access to the site. Site B can also be developed to provide a much greater storage capacity whereas the other two sites are fairly limited. Site B is therefore selected as the preferred site based on the reasons given above.

The proposed site (Site B) near Huahuaxtla offers a good site for construction of a new large check dam. The site can easily provide additional storage capacity with relatively small increments in dam height. The storage capacity at this site might be used not only for retention of sediment but also as regulation storage for the Mazatepec power plant, effectively restoring the storage capacity lost at Soledad Reservoir. However, the latter function is temporary only until the dam is filled up with sediment and no significant contribution to incremental energy output is expected judging from the simulation study done in Chapter 5.

Instead of large check dams, there is a possibility to build several low check dams on the main Apulco river to mainly retain coarse sediments as shown in Figure 7.21. This alternation concept is described in sub-section 7.3.

Alternative J (Sediment diversion tunnel):

There is a concept to sluice the sediment load into another basin through an appropriate diversion system. The basin divide of the Apulco River is very closely located to the river course in the midstream basin, near Huahuaxtla as shown in Figure 7.14, that is, it appears technically feasible to divert sediment load into a tributary basin on the left side through a tunnel within a few kilometers long. For this alternative, a diversion dam would be needed to be constructed across the river.

However, this alternative involves two disadvantages. One is a significant loss of energy which will be resulted from water to be used for sediment transport. The other is potential environmental impacts associated with sluicing sediments from one river to another.

Alternative K (Dredging):

Removal of the reservoir deposits by dredging is considered to be an reliable method, where the plan and operation are properly made. Dredging works which were previously performed present very useful information for further dredging in the next stage.

As probably experienced in the previous operation, it is very common that the reservoir deposits include almost all kinds of materials, natural and artificial, and it is quite important to know the physical properties and nature of the deposits. In particular, the data on gradation of deposit materials and profile and thickness of the deposit are quite essential to determine the framework for dredging work.

For planning the dredging work properly, due consideration should be also given to the following local specific factors.

- Area and volume to be dredged
- Location of spoil bank for dredged materials
- Layout and profile of discharge pipe
- Maximum head for pumping
- Location for assembling equipment and pontoon and storage of spare parts

Aiming at a reliable and effective operation of dredging against consolidated soils and solid materials, currently it is usual to furnish water-jet nozzle, wood pieces chopping apparatus, weed and root cutting unit, suction booster pump, etc. at the suction side.

Pump dredging will be a reliable measure to remove the deposits and this method is employed in many reservoirs. However, care should be paid to the impact to environment by the spoiled materials to be discharged in the river or other areas. Further, the dredging work needs a well-planned operation and maintenance program including supply of necessary spare parts.

In case of this Project, it does not appear economically feasible to use the dredged materials for other construction use due to its fineness of particle size and remoteness to the market. However, for this project dredging for small scale excavation and for rehabilitation works will be necessary.

Alternative L (Replacement by large outlet):
This alternative is identical to Alternative A except for its size of the outlet.

Alternative M (New outlet between existing intake and dam):
This alternative has a same concept with Alternative C for sluicing sediment load through a tunnel.

Alternative N (New outlet at the dam bottom):
This alternative is also identical to Alternative B to construct a new outlet in the dam body except for its location.

Alternative O (New pondage and new intake):
This alternative intends to abandon the existing dam having a storage function and to build a new dam to create a new pool.

Alternative P (Limitation of partial load operation and use of less jet nozzles):
Up until very recently, sediment that has passed through the turbines has been very fine, clayey particles. In 30 years of almost continuous operation, virtually no damage has been caused to the turbine runners by this fine sediment. The 1990 "Orr del Barreno Sl" report indicates that now there is some fine sand of small grain size (estimated at 0.0625 mm < grain size < 0.1 mm) entering the units. As the sediment delta grows and approaches to the dam, it will increasingly bring larger grain-size sand with it. Passing large-size sand particles through the turbines will cause accelerated wear of the runners as well as of the needles and seats.

H. Brekke in a discussion on sand-erosion in multi-jet Pelton turbines in an article titled, "Recent trends in the design and layout of Pelton turbines" Water Power & Dam Construction November 1987, states that:

"- the acceleration of a sand grain is dependent on the radii of curvature of the bucket; in a medium-sized bucket in a high head turbine this may be as high as 50,000 m/sec^2;
- the amount of sand in contact with the bucket is inversely proportional to the

bucket and the jet size (or the hydraulic radius of the nozzles);

- the sand erosion of a Pelton runner, for a certain rotational speed, is *proportional to the number of jets.*

Based on these statements, a more accurate calculation of the relative lifetime, for operation in silt-laden water, can be made for a certain speed as a function of the number of jets on the turbine. The difference in the lifetime of a runner (or the time between repair of sand-erosion damage) for a four- versus six-jet unit with the same speed may be expressed by the lifetime, T, as follows:

$T_4/T_6 = (6/4)^{0.5} \times (6/4)^{0.5} \times (6/4) = 2.25$; and

for a two- versus sixjet unit, with the same speed,

$T_2/T_6 = (6/2)^{0.5} \times (6/2)^{0.5} \times (6/2) = 9.00$.

The erosion of the nozzles and needles will, however, be *proportional to the hydraulic radius only.*

Based on this discussion of sand erosion, the largest units with the lowest number of jets (to obtain the largest possible jets and buckets) should be chosen if sand erosion is expected."

The same argument applies when it is desired to run the unit at two-thirds (or one-thirds) full load with sand-laden water. By using 4 jets fully open (i.e. cutting out 2 jets - a facility permitted by the "Digipid" governor) instead of 6 jets partially open, the runner life will be 2.25 times greater. And on one-thirds full load using 2 jets fully open, the runner life will be 9 times greater. Of course the turbines will not operate only at one-thirds or two-thirds full load; the analysis serves to show the increase in runner life when 2 or 4 jets are used for partial load operation with sand-laden water, instead of all 6 jets.

The Mazatepec turbines should not be used at very low-loads, eg, 10 MW. In fact, the minimum load on a turbine should be about 20 MW, or with a "Digipid" governor, which permits control of the number of nozzles in service, the minimum load corresponding to two, fully-open jets in service (i.e. approximately 18 MW).

Alternative Q (Digipid governor):

As described in the previous subsection, the "Digipid" governor can be considered as equipment which reduces erosion. It is recommended to give high priority to the installation of this type of governor (i.e. one which can control the number of nozzles in service).

The design of the needles and seats for the Mazatepec turbines is now about 35 years old. The nameplate on the turbines quotes 1959, thus the design date would be about 1957. In the intervening years Neyrpic (the turbine manufacturer) will have improved their needle shape design so that there is less cavitation damage. Material technology has also advanced during this period. Both these advances will produce worthwhile, if not dramatic, improvements in the life of the needles and seats. Thus Neyrpic should be consulted for improved needle and seat designs.

7.3 Screening of Alternatives

7.3.1 Preliminary Screening

All of the alternatives were reviewed and a screening was made to select the more favourable alternatives for further evaluation. The screening was based on technical considerations, constructability, effectiveness, and judgment regarding excessively high construction cost in relation to all of the alternative plan elements being considered in this study.

The following alternatives are recommended to be eliminated from further study:

> Alternative A, Alternative B, Alternative D, Alternative E, Alternative H, Alternative L, Alternative M, Alternative N, and Alternative O

Prior to the screening of alternatives, it should be noted that during construction the reservoir water level can only be drawn down to about El. 790 m approximately corresponding to the spillway crest elevation. Use of the power intake to draw the reservoir down below this level is not recommended because of the possibility of drawing excessive amounts of sediment materials into the power tunnel which could result in serious damage to the turbines. This has a significant impact on the constructability of several of the alternatives that were identified.

(1) Alternative A: For this alternative, the first question arises whether the existing outlet could be an effective solution for sluicing sediment materials near the power intake, even if it properly operates. The existing outlet has a capacity to release around 73 m^3/sec of discharge at maximum. It is noted that the current sediment surface near

the intake which is nearly same as the power intake sill level is presumably created by power generation over a long term, bringing sediment load with water of 55.2 m^3/sec at maximum. A possible risk of cleaning the limited area only (about 40 m reach only) *should be taken into account for any alternative designed to sluice the sediment load* near the intake.

There is no practical way to dewater the area needed for the construction of this facility. The minimum depth of water over the outlet would be about 40 m. A cellular cofferdam cannot be used because the maximum height for the cells is in the range of 25 m. In order to dewater the area during construction, a fill cofferdam would need to be constructed from the right abutment of the arch dam to the left gravity wall, encompassing the existing power intake. This would require a large and costly structure as well as the shutdown of the power station during construction.

The existing outlet is presently buried under approximately 25 m of sediment. Therefore, removal of a substantial amount of sediment from around the intake area prior to construction would be required, say approximately 400,000 to 600,000 m^3. Sediment would also need to be removed for a considerable distance upstream of the intake to reduce the possibility of surrounding sediment levels from caving in to the intake area during construction and during subsequent operation of the outlet.

The long conduit on the upstream face of the arch dam may also present maintenance problems should it become clogged with sediment.

<u>(2) Alternative B</u>: The arch dam is less than 5 m wide at El. 782 m which would be the approximate invert level of the new outlet. This alternative would require the construction of a new opening through the arch dam and the addition of large upstream and downstream cantilever structures to support the outlet facilities. Once constructed, operation of the valve may produce a considerable amount of vibrations. As the arch dam was not originally designed for these conditions, the structural stability of the arch dam would be a major concern.

In addition, with the new low level outlet located at a higher elevation, a new power intake would need to be constructed and most likely would be located upstream of the existing power intake. The new low level outlet would not likely be effective in sluicing sediment away from the new power intake due to the long distance between the two facilities.

<u>(3) Alternative D</u>: This alternative is not considered practical to construct. Like

Alternative A, there is no practical way to dewater the area during construction. A cofferdam with a maximum height of over 30 m, would need to be constructed completely around the new and existing power intake. This would also require complete shutdown of the power plant during construction. Locating the new power intake in the area identified for Alternative F would eliminate this problem and, therefore, further consideration of this alternative is not warranted.

(4) Alternative E: Alternatives E and F are basically the same except for orientation. Alternative F presents a slightly better orientation and is selected to represent this concept for further evaluation of alternatives. Alternative E is eliminated to avoid duplication of the same concept.

(5) Alternative H: The identified channel improvements as shown in Figure 7.13 would improve flow conditions to the spillway and may increase the effectiveness of using the spillway as a sluice. For the channel improvement, approximately 200,000 m^3 of excavation volume will be required. However, these improvements on their own would not likely result in an effective way to keep sediments from depositing in the vicinity of the power intake. Modifications to this alternative in order to improve its effectiveness would include the addition of a barrier dam across the main river channel and a weir across the constriction between the spillway and power intake.

The barrier dam would be constructed of large stone (derrick stone) to a sufficient level that would direct flow towards the spillway keeping sediment deposits confined in this channel and away from the power intake area. The direct flow approach to the spillway would greatly increase the effectiveness in using the spillway as a sluice. The weir would further restrict migration of sediment deposits towards the power intake.

However, technical and operational disadvantages of this alternative were identified as follows:

a) the construction of a "derrick stone" dam would need to be of a substantial height, say 20 m to 40 m, and it would have to act as an earth (silt) retaining structure. Structural stability will be very difficult to evaluate since this dam will have to be built on top of silt deposits with less bearing capacity and zero shear strength (cohesion) as observed in the recent drilling results.
b) for the channel improvement to be effective for flushing out sediments, the spillway will have to flush out large quantities of water which may result in a significant loss in power generation at the Mazatepec power plant as previously studied, and

c) long term operation of this plan may cause increased sediment deposits along the banks of the reservoir and river channel upstream of the spillway which could result in higher backwater levels. During large inflows, this could eventually result in continual overtopping of the new barrier structures which would significantly reduce their effectiveness in keeping sediments from the intake area. Higher water surface levels (above original design levels) during major flooding events may also occur causing concerns over spillway adequacy.

Therefore, it is recommended that the barrier dam and channel improvements alternative be removed from further consideration.

(6) Alternative L: This alternative would have the same problems with constructability as described for Alternative A.

(7) Alternative M: This alternative presents the same problems identified for Alternative D. Additional problems include space limitations between the existing power intake and the arch dam, concerns of dam stability with the construction of a new tunnel immediately below the foundation of the arch dam, and the presence of an existing drainage gallery in basically the same location. Also if a cofferdam was constructed to close off only a portion of the arch dam (left abutment area) for dewatering, this would cause an imbalanced load condition on the arch dam and possible warping of the dam causing major concerns over structural stability.

(8) Alternative N: This alternative would have the same problems with constructability as described for Alternative A. Construction problems would be further increased due to the lower depth of construction.

Power generation is planned at the outlet as an opitinal plan. However, this plan could not permit effective sluicing of sediment materials as already experienced in the existing plant.

(9) Alternative O: This alternative would have major technical problems, very high construction cost, and a substantial loss in power generation. Creating a new pool of water downstream of the existing arch dam would create an entirely new loading condition on the downstream face of the arch dam. Arch dams are not designed for such loads from the downstream direction and would create serious stability problems. Also additional loads on the upstream face of the dam would occur due to high sediment levels. The new downstream dam and water conveyance facility from Tunnel

No. 1 would be very expensive. Using only water from Tunnel No. 1 would result in a major loss of energy production. New diversions of water into Tunnel No. 1, if contemplated, would be limited to the maximum discharge capacity of the tunnel which is about 30 m^3/sec.

Implementation of such a scheme would best be served by a direct connection of Tunnel No. 1 to the existing power intake. However, this scheme would also be relatively expensive and would result in a substantial loss in energy production. Therefore, this scheme may remain as a future plan after the existing dam is abandoned.

7.3.2 Economic Comparison of Alternatives

In order to confirm more appropriate alternatives, comparative study was made on preliminary construction cost of structural measures which passed the preliminary screening. Cost comparison was made for the following alternatives.

- Alternative C+F : New intake + Use of existing intake as low level outlet
- Alternative G : Settling basin
- Alternative I : Large check dam
- Alternative J : Tunnel sediment diversion

Estimate of the preliminary construction cost (direct construction cost only for comparative study) was given in Tables 7.1, 7.2, 7.3, and 7.4 and was summarized below.

Alternative	Cost Estimates ($x10^6$)	
	New Peso (NP)	(U.S. $ equivalent)
Alternative C + F : (New Intake + Low Level Outlet)	31.9	(10.6)
Alternative G : (Settling Basin)	63.9	(21.3)
Alternative I : (Large Check Dam) 1)	33.0	(11.0)
Alternative J : (Sediment Diversion Tunnel)	52.5	(17.5)

Note: 1) Low dam schemes will be evaluated in further study.

The preliminary construction cost for Alternative C + F appears to be a good choice for the economic viewpoint. Alternative G is discarded with a low priority for earlier implementation due to its higher cost. Alternative G also requires additional cost for a permanent dredging scheme.

The preliminary construction cost for Alternative I is estimated at 33.0 million New Peso (US$11.0 million equivalent), and it is considered to be a cost effective alternative. In further study, comparison will be made with low dam schemes.

The preliminary construction cost for Alternative J is estimated at 52.5 million New Peso (US$17.5 million equivalent). It is recommended that Alternative J be given a low priority due to a relatively high cost, power loss and environment concerns.

7.4 Proposed Countermeasures

7.4.1 Combination of Alternative Countermeasures

In the previous technical and economical evaluations, alternative countermeasures were reviewed independently from each other or in combination. As the estimated sediment inflow is quite large, being 2.0×10^6 m^3 and the sedimentation includes a wide range of technical difficulties, it is not appropriate that the rehabilitation plan would rely only on a single remedial measure. In this respect, it is proposed to provide a combination or a package of the following countermeasures.

- Construction of a new Power Intake and Low Level Outlet (Alternatives C + F)
  as a measure to prevent sediment inflow into turbines and to sluice the deposits out of the reservoir.
- Construction of a large Check Dam (Alternative I)
  as a measure to prevent the sediment inflow into the reservoir.
- Dredging (Alternative K)
  as a measure to remove the deposit out of the reservoir.
- Sand reducing operation (Alternative P + Q)
  as a measure to provide sand - reducing operation.

The following sub-section presents a brief description of the above alternatives except for Alternatives P and Q. The preliminary designs of the alternatives were developed to include the recent information (primarily geologic and foundation conditions) gathered as a result of the field investigations during the May - June 1993 site visit.