Chapter 4:
TECHNICAL INVESTIGATIONS

4.1 TOPOGRAPHY

The following topographical information was gathered during the feasibility report process.

**Aerial Photography**
- Photography, b&w, 1:6,000 along river from approximately 57 km downstream of Baynes to 46 km upstream
- Photography, b&w, 1:30,000 along river from approximately 70 km downstream of Baynes to 90 km upstream
- Photography along the river course, colour 1:10,000, from Ruacana to the Cunene river mouth
- Photography along the river course, colour enlarged to 1:5,000, from Ruacana to the Cunene river mouth
- Photography, colour 1:10,000, from Ruacana to the Cunene river mouth
- Colour oblique photography, taken from various positions of dam sites

**Mapping**
- Maps 1:10,000, 5m contour lines, of reservoir area to el. 725, upstream of Epupa.
- Maps 1:10,000, 5m contour lines, of reservoir area to el. 600, upstream of Baynes.
- Orthographic Maps 1:10,000, 5m contour lines, Epupa reservoir (40 sheets)
- Orthographic Maps 1:10,000, 5m contour lines, Baynes reservoir (12 sheets)
- Maps 1:10,000, 5m contour lines, of the surroundings of Epupa dam site alternatives.
- Maps 1:10,000, 5m contour lines, of the surroundings of Baynes dam site.
- Maps 1:1,000, 1m contour lines, of Epupa dam site A.
- Maps 1:1,000, 1m contour lines, of Epupa dam site B.
- Maps 1:1,000, 1m contour lines, of Epupa dam site upper C.
- Maps 1:1,000, 1m contour lines, of Epupa dam site C.
- Maps 1:1,000, 1m contour lines, of Baynes dam site.

4.2 HYDROLOGY

The Feasibility Study comprises the following hydrology tasks.

(a) Evaluation of the rating curves used in the conversion of daily stage data measured at Ruacana and Rundu to daily discharge. (For the Pre-Feasibility Study, the Namibian DWA rating curves were accepted as being suitably accurate.)

(b) Further exploration of the favourable correlation between Ruacana and Rundu with the view of using monthly serial and cross-correlations as an alternative to the annual linear regression used in the Pre-Feasibility Study.

(c) Extension and disaggregation into daily or weekly flows of the Ruacana flow sequence for periods outside the reliable measured period of 1961 to 1973.

(d) Production of flow sequences at sites in the Cunene upstream of Rucana through factoring and lagging of the Ruacana flows.
(e) Stochastic generation of alternate flow sequences at Ruacana to allow the severity of the Ruacana flow sequence to be evaluated.
(f) Evaluation of evaporation estimates in Angola.
(g) Extension of the flood frequency analysis presented in the Pre-Feasibility Study with evaluation of the Probable Maximum Flood (PMF) and production of a flood hydrograph.

The optimisation and design studies for the hydropower schemes at or near the Epupa Falls required long term daily natural flow sequences for the Cunene River.

The hydrological studies focused on the flow gauging station at Ruacana, for which a reasonably long term historical record was available. However, it was found that only 12 years (1961-1972) of the historical record could be considered as reliable, due to uncertainties in the upstream flow regime after the building of the Gove dam and, during some periods, inconsistent recording at Ruacana.

To overcome the deficiencies in data, and for the purpose of establishing a representative long term flow record at Ruacana, a correlation was sought between Ruacana and Rundu flow gauging station on the Okavango River which has a reliable flow record since 1945. The correlation was based on the simultaneous records at Rundu and Ruacana for the period 1961-72. The long term record at Rundu was then used to derive a long term synthesised natural flow record at Ruacana, which was applied for the Baynes project after deduction of river channel losses.

The synthesised natural flow record was cross checked against available flow data at 7 Angolan gauging stations.

For the purpose of the modelling of the river flow, partial flow records were also derived for various sub-catchments, to enable the superimposition of the Gove regulation on the natural flows from other sub-catchments.

The two gauging stations at Ruacana and Rundu were inspected by the consultants during a field visit and a complete history of speed-gaugings at each of these sites was compiled via information extracted from files maintained by the Namibian Department of Water Affairs (DWA). A special speed-gauging at the Rundu station was conducted by the Namibian DWA at the request of the consultants, before the onset of the 1995/96 wet season. This was necessary to verify the rather old historical readings in the low flow portion of the rating curve, about which a certain amount of uncertainty existed, in the light of possible backwater effects due to fluctuating sand deposits downstream of the station.

A new rating curve was suggested for Rundu, but the historical rating curve at Ruacana was found to be accurate. The Rundu mean annual runoff (MAR) was reduced by 2.5% by the new rating. A "minimum" rating, roughly equivalent to a 90 percentile, was also fitted at each station, to provide a pessimistic flow estimate for sensitivity studies. This yielded a 12-15% decrease in MAR.

The Ruacana flow sequence was synthetically extended to span the period 1945-94 by a two-stage approach. Firstly, by means of an annual regression on contemporaneous Rundu values, an annual series was created. The annual approach was accepted, because it yielded more conservative long-term flows than a more sophisticated monthly
regression also designed to preserve serial correlation. Secondly, the generated annual values at Ruacana were disaggregated to daily flows by means of Rundu daily flows via the superposition of the two sets of contemporaneous observed daily flow duration (percentile) curves. This hybrid 50-year daily flow sequence was termed the "naturalised" flows at Ruacana. A "minimum" flow sequence was also created for sensitivity studies, by means of the "minimum" rating referred to above.

Given the brevity of flow records upstream of Ruacana, these were not suitable for derivation of long-term flows throughout the Cunene river system. However, they were useful for the spatial disaggregation of Ruacana flows up the Cunene system. These flow records served to derive seasonal factors relative to overlapping values at Ruacana. By applying these factors to Ruacana daily flows and allowing for suitable lag times, 50-year daily flow sequences could be generated at upstream sites of interest such as Gove, Jamba-la-Oma and Matala. "Minimum" flow sequences were also generated at these sites, for use in sensitivity studies.

Open water evaporation estimates for the Cunene catchment done in the past were checked by comparison with three empirical methods based on use of meteorological data captured for two observation stations in the catchment. Additionally, the 1988 evaporation map by the Namibian DWA which impinges on Ruacana was also brought into the picture. On the basis of this comparison a new open water evaporation map is proposed for the Cunene catchment, which yields annual evaporation values that are 15% higher than those previously used.

Design flood determinations were performed for Ruacana on the conservative assumption that flood peaks at Baynes would be similar. The Probable Maximum Flood (PMF) was derived by means of maximised PMP depth-area-duration information transferred from South African studies and by use of a "pseudo unitgraph" approach. Lack of data and accompanying uncertainties enforced use of lower bound and upper bound PMF estimates. A design PMF of 11,000 m$^3$/s is recommended. The Regional Maximum Flood (RMF) based on the empirical area-based approach by Francou-Rodier yielded an RMF of 9,500 m$^3$/s, which approximates the lower bound PMF. Frequency floods, covering a range of recurrence intervals from 2 to 10,000 years, were derived from annual maximum discharges abstracted from the 50-year hybrid daily flow series at Ruacana.

**Table 4.1 Average monthly natural flows at Ruacana 1945-1994**

<table>
<thead>
<tr>
<th></th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
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</thead>
<tbody>
<tr>
<td>m$^3$/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>18.7</td>
<td>26.3</td>
<td>77.9</td>
<td>164.1</td>
<td>267.0</td>
<td>389.2</td>
<td>445.4</td>
<td>257.9</td>
<td>112.7</td>
<td>72.8</td>
<td>52.0</td>
<td>33.2</td>
<td>159.8</td>
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<tr>
<td>MCM</td>
<td>50.1</td>
<td>68.1</td>
<td>208.5</td>
<td>439.5</td>
<td>645.8</td>
<td>1,042.3</td>
<td>1,154.5</td>
<td>690.8</td>
<td>292.2</td>
<td>195.0</td>
<td>139.2</td>
<td>86.2</td>
<td>5,012</td>
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</tbody>
</table>
SEDIMENTATION

The objective of the sedimentation analysis is to provide an assessment of the sediment transport in the Lower Cunene River, to enable a statement on the potential effects the sediments may have on the Baynes reservoir.

Suspended sediments transported by rivers deposit as water velocities drop, as is the condition in reservoirs. The degree of deposition depends also on the grading of the sediments, as courser materials deposit at higher velocities while the fine sediments may remain in suspension and be carried through a reservoir and released downstream.

In the case of Baynes most sediments will be trapped in the reservoir due to its size and consequently low velocities at high river discharges. Therefore the following items will be examined:

- the long term effect on the reservoir storage volume
- the likely deposition pattern
- possible effects on the intake structure to the power house.

To satisfy the objective it is essential to determine the sediment load of the Cunene River as accurately as possible. Very little data exists on sediment loads in the Cunene River, and that it takes at least 5 years of continuous monitoring to determine the average annual sediment reliably by means of stream sampling, so other means were employed to determine the sediment load.

In this study four different approaches have been followed:

- An assessment of sediment yield based on existing data on sediment concentration and discharge.
• Landscape analysis and the application of sediment yield figures based on empirical
data from similar environments.

• An assessment of sediment characteristics based on a survey of the Ruacana
reservoir.

• Comparison with recorded sediment transport in Namibian rivers.

The results from the estimates of the sediment yield in the Cunene river differ
considerable between the methods employed. All methods have their drawbacks and the
conclusions of results should be cautious. However, as indicative figures on the upper limit
for the sediment yield the results are considered to be sufficiently commanding.

Table 4.2 Summary of Sediment Analysis

<table>
<thead>
<tr>
<th>Method of Approach</th>
<th>Annual Sediment Yield</th>
<th>Annual Sediment Yield</th>
<th>Sediment Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Million m³</td>
<td>Million tons</td>
<td>tonnes/km²/year</td>
</tr>
<tr>
<td>Existing Data</td>
<td>0.5-1.0</td>
<td>1-2</td>
<td>13-26</td>
</tr>
<tr>
<td>Empirical Data</td>
<td>1.5-6</td>
<td>2-8</td>
<td></td>
</tr>
<tr>
<td>Ruacana Reservoir Survey</td>
<td>3-6</td>
<td>4-8</td>
<td>50-100</td>
</tr>
<tr>
<td>Namibian Rivers</td>
<td>5-8</td>
<td>7.5-11</td>
<td>100-150</td>
</tr>
</tbody>
</table>

On the basis of these estimates it is recommended that a sediment yield value of
50 tonnes/km² be accepted for the planning and design of a the dam at Baynes. The
corresponding annual average sediment volume deposited in the Baynes reservoir
assuming an effective catchment area of 75,000 km² would be less than 3 million m³.

A sediment yield value of 100 tonnes/km² is recommended as an extreme upper
limit of the sediment yield that can be foreseen. This value has been used to test whether
sediment could possibly pose a threat to the project.

Assuming 100% trapping efficiency, which is likely, the theoretical time to fill the inactive
storage would be more than 1,000 years.

4.3.1 Deposition in the reservoir

Deposition of sediments carried by the main stream would start depositing at the inlet to
the reservoir and gradually progress downstream. In view of this both the active and
inactive parts of the reservoir would be affected. Considering the relatively low sediment
yield the effects on the reservoir would be negligible in the short and medium term. The
amount deposited during 100 years would be in the order of 300 million m³. This volume
corresponds to approximately 38% of the inactive reservoir volume or 17% of the active
reservoir volume. In relation to the total storage volume the volume of sediments would
correspond to about 12%. Accordingly the reduction in reservoir volume would be
altogether insignificant within the technical lifetime of the dam structure.
4.3.2 Dam stability

Generally sediment depositions close to the dam need to be considered in the design of the dam in respect of the stability of the dam. The estimated sediment yield in the Cunene would insignificantly affect the dam stability even in the long term. In the stability analysis conservative sediment load of 30% of the dam height has been applied and a check with a sediment load of 50% of the dam height has also been done.

4.3.3 Intake

The low sediment yield would not call for any particular facilities for silt deposition and handling at the intakes.

4.3.4 Abrasion

The contents of abrasive minerals such as quartz is low in the river. No particular problems with abrasion of hydraulic equipment are expected and have not been recorded at the existing Ruacana hydropower plant.

4.3.5 Recommendations

In view of the large variability of the river flow and sediment load in the Cunene River, a sediment sampling campaign would have to be long term to yield conclusive results. As earlier indicated, at least five (5) years or desirably 7-10 years would be needed. This is, however, not considered necessary for the construction of the Baynes project.

Unfortunately, an accident occurred during the previously mentioned reservoir survey at Ruacana and the survey was not completed. The results obtained were however sufficient to enable a conclusion on the sediment transport in the Cunene River at Ruacana. Therefore, for the purpose of the Feasibility Study no further survey is required. However, for the purpose of establishing a benchmark for future surveys, it is recommended that the Ruacana reservoir survey is completed according to the initial plan. For the same purpose a survey of Matala could be contemplated. The dam volume is in the same order of magnitude as Ruacana and it is a better defined basin. It was completed a number of years before Ruacana and further information would therefore be of value for the purpose of general assessment of sediment transport and deposition in the Cunene. It is recommended for a site survey and analyses to be undertaken together with one at Ruacana as this will broaden the general understanding of silt deposition along the Cunene River.

Following the construction of the Baynes dam, a sediment monitoring programme in the future reservoir is recommended, comprised of regular surveys and sampling, similar to the practice in Namibian and South African reservoirs. The surveys could be done from a fixed pattern of cross-sections on a regular basis. According to experience the optimum frequency of resurveying reservoir sediment deposits is about 10 years.

4.4 GEOLOGY

4.4.1 Investigations

The geological investigation can be grouped in the following classifications:
• Desk Studies
• Geological Mapping
• Field Investigations
• Testing

**Desk Studies** - The objective of the two geological desk studies is to provide a basic geological framework from available literature and maps of the project area along the Cunene River.

**Geological Mapping** - Geological mapping has been performed at the dam site and reservoir areas. The geological mapping has been divided into two different investigations: Regional description and simplified geologic mapping, and engineering geological mapping.

**Field Investigations:**

- Geological Mapping (described above)
- Seismic Refraction Surveys were used to define extent of material deposits, complement information obtained from drill holes and characterise material properties.
- Exploratory Drilling provides a concrete record of the rock structure with depth at a specific point. The rock samples are visually inspected, classified and tested. Together with the information from the geological mapping and seismic survey, the underground structure and techniques and costs for construction can be predicted.
- Stress Measurements were performed in one drillhole each at the Baynes and Epupa sites in order to determine the principle stress in the rock mass.

**Testing** - Material tests were primarily performed in connection with determining the most advantageous material type for dam construction. Material sources for potential earthfill dams and sources for fine and coarse aggregate for roller compacted concrete were investigated.

- Soil Sampling of material taken from test pits provided information as to suitability.
- Laboratory Testing of dispersivity, gradation and other indications of how the material will perform as part of a dam. In addition, point load testing of representative rock cores was performed.

The geology of the project area consists largely of a relatively old variety of quartzofeldspathic paragneisses and granitoid rocks. In addition, the present level of erosion has exposed these basement rocks virtually continuously throughout the reservoir areas. The rock types in the dam site areas are generally highly competent with excellent fabric soundness, regardless of composition. Effects of later structures such as faulting, jointing and minor shearing are present locally and will cause construction problems, but their effects overall only serve to emphasise the high degree of lithological integrity.

The results of the investigations and testing confirm that the rock types in the dam and powerhouse areas are highly competent.

**4.4.2 Geologic conditions at the project site**

**The Damsite**
The main rock types occurring on the dam site is in the central and left flank area a massive granite. The northern extension of the dam wall will be located in talus and a short section directly north of the right shoulder of the river will be on Karoo sediments which is overlying the basement rocks on this side.

No major faults were located within the dam area. However, some dykes in the gorge, within the dam area, are observed and here some displacements of the dykes are registered. The dykes can be very jointed in outcrop, but seem to be very competent at depth.

Generally the basement granite is very widely jointed. There is a distinct difference in orientation of the joints between the two sides of the gorge in the dam area. On the right flank the most prominent joint set is a north-south trending near vertical joint set. A second set dips steeply south-south-west. Both sets exhibit very wide strike direction distributions. In the left flank a prominent set with shallow north-north-westerly dip exists. A second, minor joint set dipping steeply north-east also occurs on this side.

The only notable micro-structure observed are narrow zones of net veins (termed microfractures) near some dyke contacts. These micro-fractures are normally tight or epidote-filled. Foliation texture in the metabasic rocks is due to a metamorphic texture. It does not impact a strength anisotropy to the rock.

The following interpretations of dam foundation, excavation depths and water tightness are done with a Roller Compacted Concrete (RCC) dam in the deep Cunene Gorge and also with a RCC dam for the northern extension onto the talus slope.

For foundation of the dams it is desirable to found on ground of relatively homogeneous quality in order to avoid irregular settlement of the wall.

The centreline cross-valley profile in the gorge is quite uniform with 40 degree slopes, which are slightly convex with near vertical cliffs at the toes. A few steep, or even overhanging, rock ridges of up to 15 m high occur above narrow ledges. The ledges are mostly formed by dolerite dykes. The Cunene is approximately 40 m wide at the bottom of the gorge.

The right flank is downstream the centreline partly covered with bolder scree. Closest to the river the granitic rocks are covered with a thin layer of talus. Further north on the right shoulder Karoo sediments are found between the top talus and the granitic rocks deeper down. Both talus and the Karoo sediments (tillite) increase in thickness with distance from the river.

The upper 3 metres of the scree forming the talus slope is loose. The talus is quite dense and comprises sub-rounded to sub-angular in a well graded gravel and red sand matrix, occasionally with some silt. Calcrete may occur at any depth within the talus. A poorly developed ferricrete occurs an the transition surface between old talus and loose surface scree cover.

The talus material encountered in the drillholes shows to be a weak rock and there seems to be a slight increase in strength with depth. The drill hole core recovery and RQD indicates a transition from a fractured, weak material to soft rock. This transition gets deeper further from the river. Above this depth the matrix is often friable and vuggy in
places, even though it may be weakly cemented by calcrete. Below this depth it is densely consolidated under the overburden weight.

The water absorption test (Lugeon-values) indicate extremely impermeable rock even at very shallow depths in the granite gorge. The Lugeon-values for the talus layer were also generally low, with water takes of less than 2.5 Lugeons below 9 metres depth on average. The underlying sandstone (limited intersections) and tillite proved 100% impermeable. However, the interface between the sedimentary rocks and the basement rocks below showed out in some drillholes to be quite open with rather high water takes.

The Reservoir

No specific engineering geological investigations were carried out in the dam basin area remote from the construction sites, other than for construction materials. Several excursions were made across the dam basin for this and other purposes, from which general impressions were gained of the regional engineering geology. Draft geological maps produced by Dr. Smalley were also studied to evaluate conditions in the dam basin.

The dam basin of the Baynes site is well contained within the U-shaped glacial valley and the Cunene gorge. The rock upstream of the Baynes Dam Site is, as is common for the whole region, quite impermeable. Several springs attest to this. All springs that were located and inspected originate from collection of near-surface groundwater forced to the surface due to, either the shallow scree cover petering out, or damming against and welling over impermeable dykes. The first gully upstream on the Namibian side is quite waterlogged near its confluence with the Cunene, and fountains occur in a deep gully in the tailrace portal area.

Of the Karoo sediments that occur in the dam basin, only the argillaceous sandstones (or wacke) are known to be somewhat porous. Their volumes are, however, small and should not be able to absorb large quantities of water. These sediments are underlain by either very impermeable tillite, or totally impermeable basement granites. The ability of the sandstones to absorb, or to drain, large volumes of water will thus be very limited, once fully saturated.

No regional-scale permeable structures are known to occur, that may drain water from the dam basin.

The talus (scree) slopes within the reservoir generally have gradients below 7 degrees. At this angle the talus will be stable even under conditions of cyclical saturation. Considering the shallow angle and geomorphological relation to the Baynes mountains, these qualify as pediment slopes. The true talus slopes (geomorphologically speaking) fall outside of the area to be inundated by water.

It may be anticipated that small slumps may occur in the near-vertical sides of erosion gullies. Natural cavities and “pipes” at the base of the talus in some of these erosion slopes may indicate dispersive properties. Some samples also tested slightly dispersive. Washing out of fines (dispersion) may induce the above mentioned small scale slumps. These may be regarded as insignificant.

The thin scree cover on steeper slopes is coarse, free-draining and therefore likely to be quite stable under cyclical saturation.
No large scale geological structures are known to occur in the dam basin, which may become unstable under saturation. The regional geology is quite monotonous, with very competent and structurally featureless basement rocks predominant.

**Intake, Namibian side**

The proposed position for the inlet portal will be in an area where parallel NW-SE trending dykes are intersecting the basement granite. The portal will be in a near vertical, approximately 20 m high granite cliff face, formed through erosion of one of the dolerite dykes. The portal floor should be about in the outcrop of this dyke at the toe of the cliff.

The granite is generally massive with only a thin spotty scree cover on the shoulder above the portal. The majority of joints here are steeply dipping, but vary widely in direction. Only one distinct joint set is delineated. The joints, striking almost in an east-west direction, are of moderate length and are widely spaced. The joint plane surface may generally be described as “curved to planar, rough and unaltered with surface staining only” in extremely hard rock. According to the classification systems the rock quality is good to very good.

The inlet tunnels will intersect the swarm of parallel dykes within 50 m from the portal. The dykes vary from 1 - 4 m in width and seem to have near vertical dip. In surface outcrops the members of the dyke swarm are all very closely (spacing < 50 cm) to closely jointed (50 mm < spacing < 300 mm), moderately weathered, very hard rock dolerite. The rock quality is expected to be very poor to fair.

These rock quality can be anticipated to improve at the tunnel depth. This is evident from inspection of outcrops of dykes in the floor of the upstream gully where loose blocks have been eroded away and fresh dolerite is exposed. Inside the dykes the tunnel will be excavated through massive, competent granite.

The vertical penstocks will also be located in massive granite.

**Intake, Angolan side**

In the intake area a deep gully forms a part of a local lineament. This may be a local weakness zone extending from just south of the proposed power station locality across the Cunene, trending 115°. The intake is located in the steep gorge slope west of the gully and the possible weakness zone. The bedrock here consists of massive granite.

Inside the portal area the intake tunnel will be excavated in competent granite below the granite/Karoo sediment interface. The mentioned lineament or weakness zone exposed in the gully east for the intake portal seems to trend in an east-west direction dipping towards the river. The intake tunnel may possibly intersect this zone just outside the top of the intake shaft.

The shafts themselves seem to be located in massive basement rocks.

**Power Stations Namibian side**

The power station will be located in massive granite underneath the south flank of the highest local peak. The surface is covered by patches of coarse scree and boulders, but dominated by massive granite outcrops. An east-west striking narrow dyke just north of
the power station caverns may possibly intersect the excavations. The dip of the dyke has not been determined so the point of possible intersection is unknown. The dolerite is closely jointed in outcrop, but can be anticipated to be massive and competent at the depth of the power station. This is evident from dykes intersected at depth in drillhole EL4.

The most favourable orientation of the parallel caverns, according to the mapped jointings, has been evaluated to be north-south.

Rock stress measurements show relatively higher horizontal stresses down to 100 metres depth. Further down, an almost isotropic in-situ stress field exist. Compared to the lithostatic vertical stress, the normal stresses measured across induced fractures (which is identical to the minimum principal stress) are higher.

The rock quality of the granite is estimated to be very good to extremely good, even on surface.

The proposed access tunnel portal is located in the steep gorge slope in a gully where a north-south trending dyke is intersecting the bedrock. However, the portal and the tunnel itself is expected to be located in massive, competent granite. A few dykes seem to intersect the tunnel, but no major stability problems are expected to arise.

**Power Stations Angolan side**

In the proposed power station area the surface is completely covered by scree, talus and Karoo sandstone and tillite. The bedrock in the steep northern gorge slope is however massive granite. In drillhole ER5, near the location of the proposed power station, the depth to the granite-tillite contact is 55 metres. Thus it is evident that the power station will be located in massive granite below the top sediments. The core logs from drillhole ER5 indicates a very good rock mass quality. The most favourable orientation of the power station is estimated to be TN 50°.

The proposed site for the access tunnel portal and first tunnel section is in a steep Karoo-conglomerate and sandstone (grits) slope. The tunnel must traverse the tillite before entering the granite. The sandstone is thinly bedded, closely jointed and moderate to highly weathered soft rock. The sandstone is often interbedded with thin marks, mudstone and siltstone layers. The tillite is very widely jointed (1.0 < spacing < 3.0 m), unweathered hard rock. Below these sedimentary rocks the tunnel will be located in granite, which is supposed to be competent and massive.

**Tailrace and Outlet Namibian side**

The tailrace tunnel is approximately 2.1 km long from the power station to the outlet in a bend of the river approximately 3.5 km downstream of the dam wall.

The first ± 500 m of the tunnel from the outlet portal will be in granodioritic rock, which in outcrop is a slightly to moderately weathered extremely hard rock. The tunnel will intersect two parallel dykes near the portal and another set of dykes 75 - 150 metres from the portal. The lower two, trending north-westerly, are 4 to 8 m wide and approximately 10 m apart, while the second group consists of 2 narrow, less than 2 m wide, dykes. They appear to be closely jointed and slightly weathered.
The tunnel alignment crosses the granodiorite-granite contact approximately 500 m from the outlet. As this is an intensive and gradational contact, it may shift significantly with depth. The granite is massive, very widely jointed (spacing > 3 m), fresh to slightly weathered extremely hard rock.

Approximately 650 - 1,150 metres from the outlet, the tunnel will intersect several groups of dykes. The dykes observed on surface vary in width from 2 to 15 metres. The narrow dykes are closely jointed and moderately weathered while the wider dykes are more widely jointed and moderate to slightly weathered.

At approximately 1,100 metres from the outlet the tunnel will intersect the major Karoo-age dyke which trends ± 340° and is continuous over a visible length exceeding 2.5 km. In the area of intersection it is estimated to be ± 20 m wide, closely jointed, slightly weathered very hard rock dolerite. Directly north of this area it is ± 80 m wide, but may be narrower at depth.

The rock quality of this dyke is estimated at very poor to fair. This is also anticipated to be the ratings encountered at depth for the tunnel.

Dykes will likely be intersected between 1,600 - 1,750 metres from the outlet, where the tunnel crosses under the proposed spillway. The last 400 metres the tunnel will traverse very good quality granite.

**Tailrace and Outlet Angolan side**

The tailrace tunnel alignment follows the Cunene River on the Angolan side. The length of tunnel along this alignment will be approximately 3.9 km.

The engineering geology is described below, from the outlet portal to the power station. The most important geologic structures are shown on the Updated Engineering Geological Map in Appendix 9.5.

The outlet portal will be located in a steep slope of the bank of the Cunene River. The rock in the lower bank along this section of the river is mostly basement granite which is slightly jointed to wide jointed hard rock granite. The rock quality is estimated to be fair to very good.

The tunnel will intersect a couple of dykes inside the portal. They are closely jointed and moderately weathered. Then between 600 - 700 m upstream the portal a group of closely jointed and highly weathered (moderately hard rock) dykes will be intersected. The granite between the dykes appears to be highly weathered and friable. The combined rock quality for the granite and dolerite in this zone may be extremely poor. In this area the tunnel will also traverse under a patch of Karoo sandstone with beds dipping 10 - 20° towards the river. The total thickness of this sediment patch is uncertain, but from observation it is estimated to be less than 40 m and the tunnel is supposed to be fully located in granite below the sediment pack.

From 0.8 km - 1.3 km from the outlet the tunnel will be in very good quality granite, but also here some narrow dykes will also be intersected.

From approximately 1.5 - 2.0 km the tunnel will have to intersect groups of dykes or single dykes with widths up to approximately 100 m. These are generally very closely jointed,
highly weathered moderately hard rock on surface. There will be little rock cover over this section.

A small remnant of basal Karoo strata occur in the bottom of a wide wadi between approximately 2.1 and 2.2 km. A dyke crosses through the sediments.

Just upstream of the turn in the river, the tunnel will intersect a dolerite dyke. Its width, geometry and nature are unclear, due to extensive scree cover. It trends north and appears to be up to 60 m wide. About 50 - 100 metres further another dyke will have to be intersected. This one being up to 100 m wide.

Approximately 300 m further from these dykes the tunnel will intersect the same north trending continuous dyke described for the tailrace tunnel at the Namibian side. In this locality the dyke is 20 m wide, columnar intensely jointed moderate weathered hard rock.

From here on the tunnel will be in very good quality granite up to the power station.

**Spillway**

The spillway starts at the saddle just south of the power station location on the Namibian side and follows the wide gully westwards down to the river approximately 600 - 700 metres downstream the sharp bend in the river.

In the upper section of gully the bedrock is covered by coarse boulder scree, estimated 1 to 2 m thick. A dolerite dyke outcrops in the saddle and it is anticipated that this dyke extends westwards along the gully, although no outcrop could be observed. The dyke is moderately jointed, slightly weathered extremely hard rock. The V-shaped bottom of this section of the gully suggests that the floor rocks are not prone to excessive erosion.

Approximately 600-700 m up from the river the spillway crosses over the main Karoo-age dolerite dyke, which in this area is between 75 and 100 m wide. It is closely, columnar jointed, moderately weathered hard rock. This dyke will be prone to mechanical erosion and the spillway will require excavation and backfill with concrete over its width.

Small outcrops or bands of mafic metavolcanic rock are exposed in the gully, especially in vicinity of a S-bend at approximately elevation 450 m.a.s.l. These are generally strongly foliated to schistose soft rock which is prone to desegregation during weathering. However, in fresh outcrops along the Cunene banks and in the lower part of the gully, it is massive, hard rock. From the S-bend to the Cunene the gully is narrow and deep with solid rock floor. Just downstream of the S-bend the gully crosses another 30 m wide dyke, closely jointed and moderately weathered very hard rock in the gully banks, but moderately jointed with tight, clean joints in the gully floor. This dyke is not expected to erode more than the host granite.

The last section of this gully, up to 100 m before it enters Cunene, is a very narrow crevice in the massive granite with falls up to 8 m high which indicate a very erosion resistant rock mass.

**4.4.3 Seismicity**

The Norwegian seismic institute NORSAR has made an estimate of the earthquake load for the Lower Cunene Hydropower Scheme. The estimate is based on earthquake
catalogues acquired specifically for this project for Angola and South Africa augmented by reports held in NORSAR’s data base of global earthquake observations.

The study provides estimates of PGA (peak ground acceleration) at 5% damping of critical for bedrock outcrops, for the mean horizontal component of ground motion. The location considered is the Baynes Dam Site.

The earthquake hazard computations performed for the Lower Cunene Hydropower Schemes are based on a probabilistic seismic hazard analysis techniques designed to incorporate uncertainties and to quantify the uncertainties in the final hazard characterisations.

The procedure for identifying potential seismic sources in the region comprises:

- An evaluation of the tectonic history of the region in light of available geological data, with particular emphasis on the fault pattern.
- An evaluation of the historical and recent instrumental seismicity, emphasising that these data comprise the primary empirical basis for conducting seismic hazard analyses.

Seismic hazard analysis is based on both the geological and seismological history of the region, including recent and historical seismicity, and paleoseismicity if available. The information called for here includes not only the earthquake catalogue but also data and models on source processes and characteristics, mode of faulting, focal depths, inferred stress field, etc.

### 4.4.4 Earthquake Hazard Results

Preliminary earthquake loading estimates are given for the Baynes Dam Site, in Table 4.3 for annual exceedance probability $10^2$, $10^3$ and $10^4$. 

<table>
<thead>
<tr>
<th>Exceedance probability</th>
<th>PGA (m/s²) expected - $\sigma$</th>
<th>PGA (m/s²) expected</th>
<th>PGA (m/s²) expected + $\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^2$ year</td>
<td>0.16</td>
<td>0.34</td>
<td>0.44</td>
</tr>
<tr>
<td>$10^3$/year</td>
<td>0.49</td>
<td>1.04</td>
<td>1.28</td>
</tr>
<tr>
<td>$10^4$/year</td>
<td>1.24</td>
<td>2.54</td>
<td>2.94</td>
</tr>
</tbody>
</table>

### 4.4.5 Underground design

#### 4.4.5.1 Rock Support Evaluation

For classification of the rock mass along tunnels and in the underground power station, the RMR- and the Q-systems are used. These classification systems can also be used for evaluation of the general rock stability in the excavated tunnel or rock cavern and for calculation of recommended stabilising rock support.

The rock support measures are:
Rock bolts - spot bolting and systematic bolting, bolt spacing and bolt lengths are indicated. Rock bolts can be used exclusively or in combination with shotcrete.

Shotcrete - used in combination with rock bolts. Steel fibre reinforced shotcrete is mostly used. The thickness of shotcrete is indicated.

Cast concrete lining - used in extremely poor rock masses, normally through weakness zones.

Intake tunnels and shafts are proposed to be concrete lined. The necessary rock support here has therefore only a temporary function in order to secure the excavation until concrete lining is completed. The amount of support is evaluated based on this requirement and rock mass data encountered during mapping and is based upon use of shotcrete and bolts. A certain amount of grouting of joints between tunnels is included.

The caverns in the power station area are all assumed to be located in massive granodiorite. In order to optimise the stability of the caverns, they are oriented in the most favourable direction regarding the observed joint sets. The in-situ stress pattern is not expected to be of a magnitude of significance for the orientation of the caverns.

The caverns will be excavated by the drill and blast method and the cross-section will be subdivided into top headings and benches from top down to the bottom. The permanent rock support will be installed on each excavation level prior to the next benching. Due to the difficult access to the high cavern walls after the excavation is finished, the support at each level should be dimensioned to satisfy the final wall stability situation. As for the roofs, also the walls are supposed to be supported by shotcrete and fully grouted bolts.

### 4.4.5.2 Pressure Tunnel and Shaft Design

For the main alternative two parallel intake tunnels leading the water from an intake tower to the top of two separated vertical penstocks, one for each tunnel, are proposed for this project. The level of the intake tunnels will be 50 metres below the lowest water level in the reservoir. The tunnels as well as the penstocks will be concrete lined. An important criterion for the design of the tunnels and tunnel support is that the tunnels shall be operated simultaneously or separately.

The global stability of unlined tunnels is normally taken care of by locating the tunnels in rock masses where the minimum in-situ rock stress is higher than the internal water pressure. Where this criterion cannot be fulfilled, an impervious steel lining in the tunnel is necessary. The global water control, i.e. control of water losses from the tunnel to the surface during operation, will normally be taken care of by the inherent low permeability of the rock mass and where the groundwater level is higher the water head in the tunnels.

In this project the 400 - 450 metres long intake tunnels will be excavated in hard competent rock, but it is obvious that at the outer part of the tunnels the rock overburden, i.e. the minimum in-situ rock stress, is too low to sustain the internal water pressure. However, the tunnels are located under the reservoir and the internal water pressure will always equal the external pressure and no hydrofracturing of the rock will occur and hence no steel linings at the outer part of the tunnels are required. Along the inner part of the tunnels, which are not situated below the reservoir, the weight of the rock overburden more than equals the internal water pressures in the tunnels. Stress measurement results
from drillhole EL4, show that the minimum rock stresses exceed the internal water pressure in the tunnels.

Both tunnels in operation simultaneously should therefore not require any lining of the tunnels. However, the situation where one tunnel is operated separately, as mentioned above, has to be paid special attention. In this situation one tunnel is filled with water under full pressure while the other is empty. Due to the relatively short distance between the tunnels, the water gradient will be high and especially through existing joints in the rock between the tunnels, but also through the rock mass generally water leakages will occur. In order to reduce the leakages, grouting of open joints and fractured zones during excavation is advisable. For permanent support of the intake tunnels, in order to reduce water leakages and also rock fall due to pore pressure in the rock mass close to and around the emptied tunnel, concrete lining of the tunnels is proposed. The concrete lining has to be reinforced to take the external water pressure acting on the lining of the empty tunnel.

The vertical penstocks are located so that the internal water pressure in every joint from top to the bottom is exceeded by the minimum in-situ rock stress or the lithostatic weight of the rock overburden. Steel lining should therefore not be required, but as for the intake tunnels also the penstocks are proposed to be concrete lined.

### 4.4.5.3 Excavation and Rock Support

The main excavation method considered is the drill and blast method.

Design principles for pressure tunnels and rock mass and support classification systems are described in the previous section. The classifications done here are meant to be a basis for the general design, the optimisation and the cost estimates.

The rock support design done in the tunnels during construction is in principle independent of any previous classification. Each and every part of the underground openings is subjected to an evaluation of stability and an appropriate support has to be designed. In hard rocks, a Q- or RMR-system stability evaluation is in practice frequently used. In weathered formations and more complex geology a “Hock and Bray” type stress/confined calculation will be used in addition as a basis for the design.

Normally, no or incomplete information on the physical properties of the rock mass is available at the time when decisions on support need to be made at the face concurrent with the excavation. The design is based on estimated parameters or on values available from comparable situations encountered in earlier tunnels. In the more complex cases, samples are taken from the rock mass and mechanical properties established. Likewise monitoring of support performance is done, usually in the form of convergence measurements. Based on the information thus obtained, additional support is installed as required. This methodology is called “the observational method” but is also known as NATM.

The term rock “support” may be slightly misleading concerning the design principle. The various support elements interact with the rock mass to form a composite load carrying material. In blocky rock mass only bolts may be required, while a strong and continuous surface element of concrete (sprayed or cast) will be required in faults exhibiting swelling or plastic behaviour. Combinations or rock bolts, spayed concrete and cast concrete lining are used as dictated by the rock mass quality encountered under excavation.
Conventional rock support works as described are used to increase the strength of the rock mass surrounding the opening. However, in the case of stress spalling (popping rock, rock burst) the principle is reversed. This problem is caused by high stress concentrations close to the surface. The solution is to create a “softer” ring around the opening in order to allow deformation and hence reduce the stress gradients close to the surface. This is obtained by combinations of yielding rock bolts and shotcrete. Stiff support may be installed later, when the deformation has finished. A common mistake is to install stiff support at the face. This may lead to violent and uncontrolled energy releases and may create hazardous situations.

Water control is achieved by grouting, drainage or by combinations of these methods. Grouting only is used around bulkheads and transitions to reduce the conductivity of the rock mass. In order to render the rock mass impermeable, a standard 4,000 Blaine cement in combination with 6,000 - 10,000 Blaine grouting cements is the most common combination. Where further measures are necessary, as at the interface between rock and concrete at bulkheads and transitions, chemical grout agents such as polyurethane and epoxy have proved to be an economic solution.

If the problem is water ingress into the tunnels during construction, both fine and coarse grained cement grouts and chemical grouts may be used. In such cases drainage may be employed to reduce pressures in the aquifer sufficiently to facilitate grouting. Exploratory drilling will be employed in order to locate leakage zones well ahead of the face and in order to facilitate water control.

The functional life of the tunnels is governed by the life expectancy of the support materials, in practice concrete and steel. The concrete used in the tunnels should have as high or higher quality than the structural concrete used above ground. The steel quality in the reinforcement will be as for use above ground. Rockbolts will be hot dip galvanised and epoxy coated.

The quality of materials and installation procedures for the rock support will be subject to thorough control in order to secure the quality of the underground opening.

The mineralogical investigations performed on the Baynes project have not identified harmful or unstable minerals. Minerals that may cause deterioration of concrete are anhydrite and pyrhotite. These have not been observed.

Drill and blast tunnels

For high quality rock mass normal full length rounds are used while short rounds and subdivision of rounds are used in low quality rock mass. The objective with reduced round length and span is to obtain an opening with “stand-up time” long enough to allow the installation of the necessary support.

The rock support consists of combinations of rock bolts, shotcrete, several forms of reinforcement for the shotcrete and full concrete lining as described above.

Shaft Excavation

The shaft excavation of the project consists of the intake shafts, the cable shafts and the tailrace gate shafts.
The choice of excavation method depends primarily on length and size of the shafts and on the rock mass quality and the support design.

The intake shaft is planned with length in the order of 125 m and with a cross-section of approximately 30 m$^2$.

The tailrace gate shafts are planned with a length of approximately 45 m and a cross-section of some 85 m$^2$, while the cable shafts are planned with a length of approximately 200 m and a cross-section of 40 m$^2$.

Raise Climbers (such as Alimak lifts or similar) from below combined with slashing to full size cross-section from above is a cost effective method and is a probable method for the intake and cable shaft of this project.

Shaft sinking from above is considered a safe method, but is more costly, however, and requires specialised contractors with vast experience in shaft sinking in order to complete the shaft on time and within cost. One of the major difficulties with this method is water control and muck extraction without hampering the advance and lining works.

A somewhat cheaper method which consists of a small raise bored muck chute and drainage shaft from above combined with shaft sinking to full size cross-section. is evaluated as feasible and is recommended for the project.

**Cavern Excavation**

The objective in location of the power station cavern system has been to reduce the length of intake tunnels upstream the intake shafts as well as reduce the length of the steel-lined length downstream the transition at the bottom of the shafts and to obtain as favourable rock conditions as possible for the caverns. The orientation of the caverns is determined based on joint observations during surface mapping. The in-situ rock stresses are not expected to be of magnitudes of significance for the orientation of the cavern.

The final orientation of the caverns will likewise be verified during construction and will be based on joint observations along the access tunnel and on the in-situ stress measurements in short drillholes.

The power station is designed with the objective of reducing the excavation volume to a minimum. This is a cost efficient design practice considering the rock mass quality expected.

The cavern will be excavated by the drill and blast method and the cross-section will be subdivided into top headings and benches from the top down to the bottom.

The permanent rock support has to be installed on each excavation level prior to the next benchesing. Monitoring of deformations in the crown and walls during the construction phase is recommended. The support will then be strengthened by additional support if necessary.

Fibre reinforced shotcrete in combination with long and fully grouted rock bolts (4 - 6 m) will be a major element in the support. Lattice girders in combination with shotcrete and rock bolts and/or rock anchors are suitable as support of the weakness zones expected.
4.4.6 Dam Foundation

4.4.6.1 Main Dam

The dam foundation material under the main dam is of such high quality that it is not a limiting factor in selection of dam type.

The least cost main dam at Baynes has been found to be an arch gravity RCC section, founded on basement rock formations. High on the abutments the arch gravity dam terminates on RCC gravity sections, also founded on this rock. The right abutment will have an additional section of RCC gravity dam which is described below.

The foundation excavation depth was taken to be 2 meters of rock where the rock is exposed and 1 meter of soil in the riverbed (see Chapter 14 Section 14.2.9).

4.4.6.2 Right Flank Dam

The extension of the dam on the right flank has received extra attention due to its more complicated foundation. While the main dam and abutting gravity sections are founded on granite bedrock, the right flank dam will be founded on sedimentary deposits, thus requiring special consideration.

The right flank dam foundation, at the end closest to the river, consists of granitic rocks with a thin cover of talus. A short section directly north of the right shoulder of the river will be on tillite. The northern extension of the dam will be located in the talus. Tillite occurs between the talus and the granitic rock and both the tillite and talus increase in thickness with distance from the river. These sediments are a suitable foundation stratum for an RCC gravity dam.

Two dam types have been considered for the extension of the dam on the right flank: an embankment dam with central asphaltic concrete core and RCC gravity dam. Provided the foundation conditions are adequate, the RCC dam is preferred as this avoids the complications of introducing a further technology in dam construction at this site.

Preliminary cost comparisons were performed between RCC gravity dams founded at either the top of the tillite or at the middle of the talus vs. the asphalt core embankment dam option. These showed that if the RCC foundation is in the middle of the talus, the costs would be very similar. Founding the RCC dam on the tillite layer or bedrock is far more expensive than the embankment dam since the bedrock and talus layers begin to drop off very quickly towards the end of the dam.

There are additional non-tangible advantages and potential cost savings in having only one dam material used, however, the cost benefits are not readily estimated.

The drill hole core recovery and RQD indicates a transition from a fractured, weak material to soft rock. This transition occurs in the top third of the talus near the river, trending to move deeper, located at the middle further from the river, and is at uncertain depth at the end of the dam. The dam also is retaining less water further from the river.

There is an element of uncertainty as to the final foundation depth as there is limited investigation information near the end of the dam. This uncertainty represents a small economic risk. In the event the transition in the talus layer is deeper than expected, the
dam section could be changed to reduce stress on the foundation, additional excavation or foundation treatment may also be required. Further testing is recommended during the final design phase to further define the foundation properties. Conservative excavation and dam quantities were used in the cost estimate. Foundation excavation and concrete quantities reflect a foundation level of the middle of the talus layer. The actual final level is expected to be somewhat higher than that. The conservative quantities compensate for the small uncertainty near the termination point of the right flank dam.

Consolidation Grouting is not expected to be required on the right flank area, no blasting is anticipated and therefore the foundation is expected to be relatively undisturbed. Curtain Grouting is taken to be performed in a single row spaced 5 m on centre to a depth of half the high water level. Dental Concrete, used to even out rock foundation prior to RCC placement, will be nominal. Excavation with ripping tends to be very controlled, and little treatment is needed to provide a good base for RCC.

4.5 ALTERNATIVE WATER USE

4.5.1 Background

The Cunene River has its source in the highlands of south western Angola in the vicinity of Huambo at an elevation of about 1,750 m.a.s.l. The river flows in a southward direction for a distance of 660 km to Calueque, where it turns westwards for some 390 km before discharging into the Atlantic Ocean. The catchment area of the Cunene is 106,500 km$^2$ of which 92,400 km$^2$ lie within Angolan Territory and 14,100 km$^2$ in Namibia.

Along the western divide in Angola lies an escarpment which drops steeply to the west, and forms a barrier to humid air coming from the west, giving rise to typical orographic rainfall over the upper part of the catchment. On the eastern side of the Cunene the divide towards the Okavango river basin is not pronounced, with the result that the meteorological conditions over the headwaters of the two rivers are similar, a feature of importance for the estimate of the Cunene flow record. The rainfall varies over the catchment from 1,300 mm in the upper parts to less than 100 mm per annum in the lower reaches.

The water resource of the Cunene River is substantial, being a perennial river with a mean annual flow of 50 m$^3$/s at Gove and 160 m$^3$/s at Ruacana. The annual variation is, however, considerable, the most serious circumstance being the relatively long periods of drought lasting for several years. This is particularly important at the Ruacana Hydro Power station which lacks storage for seasonal regulation and experiences large and unpredictable variations in the electric power generation. The plans for harnessing the Cunene have mainly concerned stream flow regulation, irrigation, water supply and hydroelectric power generation.

Other projects along the Cunene are located in Angola, i.e. the Gove regulating dam, the Matala hydropower and irrigation scheme, and the Calueque regulating dam and pumping scheme. These installations were implemented by the former colonial regimes in Angola and Namibia, under agreements between the Republic of Portugal and the Union of South Africa, Matala in the 1950s and Ruacana, Gove and Calueque in the 1970s. Due to the turbulence in the region the Gove dam has not brought about the intended regulation of the Cunene. The Calueque dam was never completed according to plans, and contains only the pumping plant for provision of water to Northern Namibia.
Suitable land and soils for irrigation are present along the river course in the two countries, as well as a demand for adequate water supplies for various purposes. However, the stream flow regulation has mainly been examined considering future development of the large hydropower potential notably in the Lower Cunene. Thus the implementation of the Gove Dam was an important prerequisite for the building of the Ruacana Hydro Power station and future projects downstream. Likewise the plans to develop the larger Calqueque reservoir were motivated by the same considerations. However, the pumping scheme for irrigation and water supply into Northern Namibia was the only element effectuated to its planned capacity.

4.5.2 Past Agreements

The multi-purpose use of the Cunene River has been regulated under various agreements, initially entered into between the Republic of Portugal and the Union of South Africa, that were ratified in 1990 by the People's Republic of Angola and the Republic of Namibia. The agreements of most importance are the Second Border Agreement and the First Water Use Agreement of 1926, the Agreement between the Government of Portugal and the Government of the Republic of South Africa regarding rivers of mutual interest and the Cunene Scheme of 1964, and the Agreement between the Republic of South Africa and the Government of the Republic of Portugal in regard to the First Phase Development of the Water Resources of the Cunene River Basin of 1969.

The most important provision set forth in the agreements for the future multi-purpose developments of the Cunene River was the equal rights to the water resource for the two countries and the best joint utilisation thereof in the common interest of both nations.

The Agreements also provide for the functions of the Permanent Joint Technical Commission (PJTC) established for the purpose, who meets regularly and acts as an advisory and reporting body on the development plans of the Cunene Basin.

4.5.3 Comments on the Master Plan Development

The large scale development of the Cunene River implied in the Master Plan studies in the 1960s is unlikely to be performed, at least not in the time span relevant to this study. This is a consequence of both environmental and sociological considerations as well as the competitive aspects of financial funding relating to other possible projects in both Angola and Namibia.

Generally events upstream will have an impact on the lower reaches of the river system and any interference along the river course will have impacts on the environment downstream.

Considering that the old Master Plan is lacking in certain aspects, together with today’s focus on sustainable development, it may be appropriate to look upon this study as an inventory of possible developments in the basin.

Regarding the overall development of the Cunene River basin, it is of great importance to address the findings of the Master Plan review that is being performed by Portuguese entities in parallel to the Feasibility Study of the Lower Cunene Hydropower Scheme.
4.5.4 The Assessed Development

Regarding the short-term development, until the year 2005, there seem to be a consensus on the Angolan side that efforts will be put on rehabilitation of already existing or partly developed structures, rather than developing of new projects. For the Cunene basin this will include rehabilitation of Gove, irrigation at Matala (3,000 ha), at Quiteve-Humbe (500 ha) and at Chibia (1,000 ha) as well as upgrading of the regulation capacity at Calueque.

Regarding the Neves-Humpata and the Neves schemes a development reaching almost 2,000 ha have been discussed in the short-term range.

The assessment of the long-term development, 2005-2035, is made in more general terms regarding increase in irrigated areas, water supply and industrial. In the long-term, also the development plans in Northern Namibia is an important issue.

The Jamba-Ia-Oma and Jamba-Ia-Mina projects are regarded as the major candidates in the long-term range among the identified water resource projects. They have, however, no direct effect on the water use scenarios, but their effect on the Baynes development may be investigated in simulations with the NAMANG river model.

Together with the development of the areas that will be put under irrigation, the water requirements per hectare for irrigation are the most important factors for the future water use. The three scenarios have a development rate of 600, 900 and 1,200 ha per year of irrigated land.

Altogether the figure 17,000 m$^3$/ha/a is proposed as a realistic estimate for the gross irrigation demand to be considered in the future water use scenarios.

4.5.5 Alternative Water Use Scenarios

Main sources of information have been documents of previous investigations and studies, contacts with national, regional and local concerned authorities, agencies and institutions, as well as field inspections. Additional information has been gathered from discussions with LNEC on the Master Plan Study.

A comprehensive list of existing and potential projects in the Cunene River basin has been assembled including their intended purpose and salient features. It is concluded that in the short-term no new large water resource projects, apart from Baynes, will be developed in the Cunene basin. The general trend would be to rehabilitate schemes that were fully or partly developed earlier. In the long-term the Jamba-Ia-Oma and Jamba-Ia-Mina projects are deemed to be major candidates.

Generally most of the future potential projects have only been identified and not more than briefly studied, and consequently the quality of the information varies between projects.

Three different scenarios for future development have been established, reflecting a range of possible developments from today until 2035.

The size of the areas that will be put under irrigation, and the water requirements per hectare for irrigation are the most important factors for the future consumptive water use. The three scenarios have development rates of 600, 900 and 1,200 ha per year of irrigated
land. The figure 17,000 m\(^3\)/ha/a is proposed as an average realistic estimate for the specific gross irrigation demand to be considered in the future water use scenarios. This figure would largely represent flood irrigation with an efficiency of 50-65%.

The scenarios imply that the Angolan portion of the water abstracted from the Cunene will be larger in the future. The estimates suggest that the abstraction of water in Angola may reach a quarter of the quota provided for in the 1969 Agreement in the short term. The abstraction in the long term may increase to up two thirds of the quota in the year 2035.

The most important future water consumer is likely to be irrigation, mainly in Angola but also to some extent in Northern Namibia. For Angola, irrigation use will dominate, increasing from about 3 times the consumption of all other uses combined (i.e. domestic demand, livestock consumption, mining and other industry) in the year 2000, to approximately 10 times, in the year 2035.

In absolute terms the withdrawal of water from the Cunene may increase from about 125 Mm\(^3\)/year (4 m\(^3\)/s) to approximately 600-1,100 Mm\(^3\)/year (20-40 m\(^3\)/s) in the year 2035. The large spread in the figures for the year 2035 reflects the obvious difficulties in making precise estimates on future development. The figures may also be compared to the estimated average discharge at Ruacana today of about 5,000 Mm\(^3\) (160 m\(^3\)/s).

Figure 4.2  Graphical Presentation of the Water Use Scenarios
4.1 TOPOGRAPHY ................................................................................................. 1
4.2 HYDROLOGY .................................................................................................. 1
4.3 SEDIMENTATION .......................................................................................... 3
4.4 GEOLOGY ........................................................................................................ 3
4.5 ALTERNATIVE WATER USE ........................................................................ 4