ADVENTIST DEVELOPMENT AND RELIEF AGENCY
MAPUTO - MOZAMBIQUE

CHIMOIO WATER SUPPLY PROJECT

FINAL DESIGN REPORT

Financed by:
USAID
Rua Faria de Sousa 107
Maputo

On behalf of:
National Directorate of Water
Ministry of Construction and Water
Republic of Mozambique

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JULY 1994
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. INTRODUCTION</strong></td>
<td></td>
</tr>
<tr>
<td>1.1. Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2. Project Area</td>
<td>2</td>
</tr>
<tr>
<td>1.3. Climatic Conditions</td>
<td>2</td>
</tr>
<tr>
<td>1.4. Project Objectives</td>
<td>2</td>
</tr>
<tr>
<td>1.5. Report Objectives</td>
<td>3</td>
</tr>
<tr>
<td>1.6. Report Structure</td>
<td>3</td>
</tr>
<tr>
<td><strong>2. EXISTING DATA</strong></td>
<td></td>
</tr>
<tr>
<td>2.1. Topographic Maps</td>
<td>4</td>
</tr>
<tr>
<td>2.2. Topographic Surveys and Way-leaves</td>
<td>4</td>
</tr>
<tr>
<td>2.3. Soil Surveys</td>
<td>4</td>
</tr>
<tr>
<td>2.4. Hydrology Information</td>
<td></td>
</tr>
<tr>
<td>2.4.1. Storage Capacity of Chicamba Lake</td>
<td>5</td>
</tr>
<tr>
<td>2.4.2. Design Levels of Chicamba Lake</td>
<td>5</td>
</tr>
<tr>
<td>2.4.3. Historical Water Levels in Chicamba Lake</td>
<td>6</td>
</tr>
<tr>
<td><strong>3. FIELD SURVEY</strong></td>
<td></td>
</tr>
<tr>
<td>3.1. Topographical Survey</td>
<td>7</td>
</tr>
<tr>
<td>3.2. Soil Investigation</td>
<td></td>
</tr>
<tr>
<td>3.2.1. General</td>
<td>7</td>
</tr>
<tr>
<td>3.2.2. Soil Types</td>
<td>8</td>
</tr>
<tr>
<td>3.2.3. Test Results</td>
<td>8</td>
</tr>
<tr>
<td>3.2.3.1. Sieve analyses</td>
<td>8</td>
</tr>
<tr>
<td>3.2.3.2. Compaction analyses</td>
<td>8</td>
</tr>
<tr>
<td>3.2.4. Implications of Soil Types for Excavation and Structures</td>
<td>9</td>
</tr>
</tbody>
</table>
4. COMPONENTS OF THE PROJECT

4.1. General

4.2. Intake Tower
4.2.1. Location and intake level
4.2.2. Civil work brief description
4.2.3. Structural calculation
4.2.4. Mechanical equipment
4.2.5. Electrical equipment
4.2.6. Alarms

4.3. Access Bridge to the Intake Tower
4.3.1. General
4.3.2. Method of construction
4.3.3. Structural calculation
4.3.4. Other ancillary

4.4. Transportable Pump
4.4.1. General
4.4.2. Civil work
4.4.3. Electro / mechanical work

4.5. Guard - Store House
4.5.1. General
4.5.2. Civil work

4.6. Intake Site and Transformer Tower
4.6.1. General
4.6.2. Intake Site
4.6.3. Transformer Tower
4.6.3.1. Structural Calculation
4.6.3.2. Size of transformer

4.7. Access Road
4.7.1. General
4.7.2. Method of Construction
4.7.3. Culverts
4.8. **Pressure Pipeline**

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8.1. General</td>
<td>17</td>
</tr>
<tr>
<td>4.8.2. Earthworks</td>
<td>17</td>
</tr>
<tr>
<td>4.8.3. Pipe Laying</td>
<td>17</td>
</tr>
<tr>
<td>4.8.4. Concrete Works</td>
<td>18</td>
</tr>
<tr>
<td>4.8.4.1. Valve Chambers</td>
<td>19</td>
</tr>
<tr>
<td>4.8.4.2. Anchor Blocks</td>
<td>19</td>
</tr>
<tr>
<td>4.8.4.3. Marker Posts</td>
<td>19</td>
</tr>
<tr>
<td>4.8.5. Valves and Accessories</td>
<td>19</td>
</tr>
<tr>
<td>4.8.6. Hydraulic analyses</td>
<td>20</td>
</tr>
<tr>
<td>4.8.6.1. Water Hammer / Surge Pressure</td>
<td>20</td>
</tr>
<tr>
<td>4.8.7. Pipe Material - Steel Pipes</td>
<td>22</td>
</tr>
<tr>
<td>4.8.8. Testing of the Pipelines</td>
<td>22</td>
</tr>
<tr>
<td>4.8.9. Cleansing and Disinfection of the Pipeline</td>
<td>22</td>
</tr>
</tbody>
</table>

4.9. **Header Reservoir**

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.9.1. General</td>
<td>23</td>
</tr>
<tr>
<td>4.9.2. Civil Work</td>
<td>23</td>
</tr>
<tr>
<td>4.9.3. Structural Calculation</td>
<td>23</td>
</tr>
<tr>
<td>4.9.4. Electro-Mechanical work</td>
<td>23</td>
</tr>
</tbody>
</table>

4.10 **Gravity Main**

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.10.1. General</td>
<td>24</td>
</tr>
<tr>
<td>4.10.2. Earthworks</td>
<td>24</td>
</tr>
<tr>
<td>4.10.2.1. River-Stream Crossings</td>
<td>24</td>
</tr>
<tr>
<td>4.10.3. Pipe Laying</td>
<td>24</td>
</tr>
<tr>
<td>4.10.4. Concrete Works</td>
<td>24</td>
</tr>
<tr>
<td>4.10.5. Valves and Accessories</td>
<td>24</td>
</tr>
<tr>
<td>4.10.6. Hydraulic analyses</td>
<td>24</td>
</tr>
<tr>
<td>4.10.6.1. Water Hammer / Surge Pressure</td>
<td>25</td>
</tr>
<tr>
<td>4.10.7. Pipe Material - uPVC Pipes and Fittings</td>
<td>27</td>
</tr>
<tr>
<td>4.10.8. Testing of the Pipelines</td>
<td>27</td>
</tr>
<tr>
<td>4.10.9. Cleansing and Disinfection of the Pipelines</td>
<td>27</td>
</tr>
</tbody>
</table>

5. **OPERATION & MAINTENANCE**

5.1 **Present O & M Situation**

5.2 **O & M for Extended system**

6. **CONSTRUCTION PROGRAMME AND CASH FLOW**

6.1 **Construction Programme**

6.2 **Cash Flow**
7. CONFIDENTIAL COST ESTIMATE

7.1. Summary Cost Estimate-Phase II

8. LIAISON

9. PROJECT RISKS AND CONSTRAINTS

9.1. General

9.2. Major Risks

9.2.1. Delays to Project Execution

9.2.1.1. Delay in obtaining finance on time

9.2.1.2. Delay in obtaining land acquisition approval

9.2.1.3. Delay due to possible presence of mines on the site

9.2.2. Unsustainability of the installed system

9.3. Minor Risks

9.3.1. Inability to construct system components as designed

9.3.2. Failure of the system to provide required capacity
Chimoio Water Supply Project
Final Design Report

ABBREVIATIONS

The following general abbreviations are used in this Report:

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F.D.R.</td>
<td>Final Design Report</td>
</tr>
<tr>
<td>USAID</td>
<td>United States Agency for International Development</td>
</tr>
<tr>
<td>AdC</td>
<td>Agua de Chimoio</td>
</tr>
<tr>
<td>DNA</td>
<td>National Directorate of Water Affairs</td>
</tr>
<tr>
<td>DINAGECA</td>
<td>National Directorate for Mapping</td>
</tr>
<tr>
<td>SHER</td>
<td>Chimoio Energy Company</td>
</tr>
<tr>
<td>DNEP</td>
<td>National Directorate for Roads and Bridges</td>
</tr>
<tr>
<td>DPCA</td>
<td>Provincial Directorate for Construction and Water</td>
</tr>
<tr>
<td>CPMZ</td>
<td>Companhia do Pipeline Mocambique-Zimbabwe</td>
</tr>
<tr>
<td>masl</td>
<td>Metres above sea level</td>
</tr>
<tr>
<td>m</td>
<td>Metres</td>
</tr>
<tr>
<td>m³</td>
<td>Cubic metres</td>
</tr>
<tr>
<td>m³/h</td>
<td>Cubic metres per hour</td>
</tr>
<tr>
<td>No.</td>
<td>Number</td>
</tr>
<tr>
<td>l/s</td>
<td>Litres per second</td>
</tr>
<tr>
<td>mm</td>
<td>Millimetres</td>
</tr>
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</table>
CHAPTER 1

INTRODUCTION
1. INTRODUCTION

1.1. Background

This report relates to Water Sector Study (Draft Report, Sept. 93) by the Consultant DHV and Feasibility analysis and cost estimate, Pipe line from Chicamba Dam to Chimoio (Apr. 94) by Consultant Sheladia Ass.

The city of Chimoio is the capital of Manica Province. In 1993 the Chimoio City Council estimates the total urban / peri-urban population at about 190,000 inhabitants.

The normal source of raw water for the town of Chimoio is the Mezingaze river where water is pumped out of two interconnected reservoirs to a water treatment plant and then distributed through distribution network. The upstream of these two reservoirs has been destroyed in a flash flood. The distribution network is not a part of this F.D.R. but it is worth mentioning that about 30% of total water pumped in distribution network is lost due to the leakage's in the reticulation pipes through the City.

Only 50% of the 100,000 or so residents of the urbanised part of Chimoio are connected to the water distribution network of "Agua de Chimoio". The remainders, and some of peri-urban community, are served by public taps or more often by a system of individual wells unrelated to the urban water supply system.

Due to the drought of 1990/1991, the Mezingaze River has dried with hardly any real storage capacity in the reservoir. This situation is now repeated every summer and AdC must completely cease operating six months a year.

During that period increasing pressure has been placed on boreholes wells to support the entire community. Further attempts by the AdC to supply additional water through drilled wells have not proved satisfactorily. Capacity of these sources is very limited and in no way can be considered as having potential to supply the entire City.

This drought has highlighted very clearly the extremely low security and the insufficiency of the two present sources of water supply (Mezingaze River and underground).

In response to this emergency, US Agency for International Development (USAID-Mozambique) agreed to provide funds to develop and construct an alternative source of water supply which would better safeguard the water supply of Chimoio. The adequate source of raw water has been found in Chicamba dam reservoir located 41 km from the City.
1.2. Project Area

The town of Chimoio is located in the Beira corridor which contains major infrastructures such as the railway and the EN6 national road that link the port of Beira with Zimbabwe. The distance between Chimoio and the port of Beira is approximately 200 km.

The town of Chimoio lies at an altitude of between 630 and 710. The treatment plant, where the water must be discharged is located in the lowest part of Chimoio at 635 m of altitude.

The project area is situated in the Province of Manica in Central Mozambique. The installations will be constructed between the reservoir of the Chicamba dam, where the intake is located, and the existing water treatment plant located to the north of Chimoio.

1.3. Climatic Conditions

The minimum and maximum monthly average temperatures are 18 °C and 25 °C respectively. Monthly average rainfall precipitations are:

<table>
<thead>
<tr>
<th>Month</th>
<th>Precipitation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>240</td>
</tr>
<tr>
<td>February</td>
<td>199</td>
</tr>
<tr>
<td>March</td>
<td>183</td>
</tr>
<tr>
<td>April</td>
<td>53</td>
</tr>
<tr>
<td>May</td>
<td>14</td>
</tr>
<tr>
<td>June</td>
<td>18</td>
</tr>
<tr>
<td>July</td>
<td>13</td>
</tr>
<tr>
<td>August</td>
<td>15</td>
</tr>
<tr>
<td>September</td>
<td>12</td>
</tr>
<tr>
<td>October</td>
<td>36</td>
</tr>
<tr>
<td>November</td>
<td>120</td>
</tr>
<tr>
<td>December</td>
<td>218</td>
</tr>
</tbody>
</table>

1.4. Project Objectives

The objectives of the Consultancy Services for phase I of the Project are:

- To develop a concept for the supply of raw water to the city of Chimoio
- To prepare a final design for the accepted alternative
- To prepare and issue the tender documents
- To prepare a tender evaluation and give recommendation to the client
In developing the raw water supply concept, alternative sources have been studied and the accepted alternative is supply from Chicamba dam.

1.5. **Report Objectives**

In particular the following engineering details shall be provided in this Report:

- General layout of the scheme showing all main components
- Longitudinal sections of the pipeline
- Hydraulic analysis of the pipeline
- Structural analysis of the structures
- Plans and sections of important details and the dimensions of the major components of the system such as intake tower, storage reservoir, small structures on the pipeline, all necessary electrical and mechanical equipment, etc.

1.6. **Report Structure**

This report is submitted in three volumes, 9 chapters in Volume I, Appendices in Volume II, and drawings in Volume III.

**Volume I:**
- Chapter 1. Introduction
- Chapter 2. Existing data
- Chapter 3. Field survey
- Chapter 4. Components of the project
- Chapter 5. Operation & Maintenance
- Chapter 6. Confidential cost estimate
- Chapter 7. Construction programme and cash flow
- Chapter 8. Liaison
- Chapter 9. Project Risks and Constraints
Tables and figures in this Report are numbered according to the Chapter in which they appear. They are numbered consecutively according to mention in the text. Environmental and baseline studies are subjects of separate reports.
CHAPTER 2

EXISTING DATA
2. EXISTING DATA

2.1 Topographic Maps

Three principal series of topographic maps have been consulted. These are available through the National Directorate of Water Affairs (DNA), the National Directorate for Mapping (Dinageca), and the Chimoio Energy Company (SHER/EdM).

1:50,000 (1958-1960), sheets 837 and 838
1:10,000 (1953), Bacia Hidrografica do Revue
1:10,000 (1980), DNGC, Chimoio

The 1:10,000 maps cover the whole project area, except for one section along the EN6 road of about 8 km.

2.2 Topographic Surveys and Way-leaves

Both the National Directorate for Roads and Bridges (DNEP) in Maputo and the Provincial Directorate for Construction and Water (DPCA) in Chimoio have been consulted. Both organisations were most helpful. However, neither were able to provide topographic surveys of the EN6 between Chimoio and the Messica River nor of the road from the EN6 and the Chicamba Dam. Apparently there was a serious fire in the technical archives of DNEP some years ago which destroyed the records.

Services should be at least 15 metres from the centre line of a major road. However, permission may be granted for closer passage in special cases. Route of the pipeline is crossing the major road EN6 only one place, therefore way-leave is required only for that road crossing.

The Companhia do Pipeline Mocambique-Zimbabwe (CPMZ), were also most helpful. They have provided copies of topographic survey carried out in 1964/65 along the pipeline between Chimoio and Messica River. The way-leave of oil pipeline has a width of 60 m. CPMZ have indicated that it is not acceptable to install the water pipeline within their way leave.

2.3 Soil Surveys

Relatively little information about soils along the pipe routes has been found. The CPMZ topographic surveys carries some general notes about surface soil conditions. Therefore it was necessary to execute the soil investigation along the pipe route.
2.4. **Hydrology Information**

The following information has been provided by the Hydrology Section of SHER, Chimoio, the operators of Chicamba dam.

**2.4.1. Storage Capacity of Chicamba Lake**

Chicamba dam has storage capacity for full turbine operation (50 cubic metres per second) during a period of three years. Water availability has not limited power generation since the dam crest was increased in 1973, despite two periods of serious drought. The water consumption of Chimoio is less than 0.5% of the turbine flow, a value which can be considered insignificant.

The long term plans for power generation in the Manica/Sofala region include the construction of a sub-station on the Cahora Bassa power line to provide power to the region. This will be cheaper than up-grading turbine capacity at Chicamba/Mavuzi power scheme. There is therefore likely to be a lower water consumption for electricity generation in the future and more storage capacity will be available to secure the water supply for Chimoio.

**2.4.2. Design Levels of Chicamba Lake**

Important levels (metres above sea level, masl) relevant for the design of the intake tower are:

<table>
<thead>
<tr>
<th>Description</th>
<th>Level (masl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam crest</td>
<td>626</td>
</tr>
<tr>
<td>Maximum operational level</td>
<td>625</td>
</tr>
<tr>
<td>Flood protection level, January</td>
<td>622</td>
</tr>
<tr>
<td>Flood protection level, February</td>
<td>620</td>
</tr>
<tr>
<td>Designed minimum operational level</td>
<td>590</td>
</tr>
<tr>
<td>Actual minimum operational level</td>
<td>582</td>
</tr>
<tr>
<td>Bottom discharge</td>
<td>565</td>
</tr>
<tr>
<td>River bed</td>
<td>548</td>
</tr>
</tbody>
</table>
2.4.3. Historical Water Levels in Chicamba Lake

The observed minimum in the Chicamba Lake since the dam crest was raised in 1973 are:

<table>
<thead>
<tr>
<th>Year</th>
<th>Level (masl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1973/74</td>
<td>591</td>
</tr>
<tr>
<td>1974/75</td>
<td>613</td>
</tr>
<tr>
<td>1977/78</td>
<td>618</td>
</tr>
<tr>
<td>1984/85</td>
<td>615</td>
</tr>
<tr>
<td>1991/92</td>
<td>611</td>
</tr>
<tr>
<td>1992/93</td>
<td>609</td>
</tr>
<tr>
<td>1993/94</td>
<td>610</td>
</tr>
</tbody>
</table>

A study has been carried out of the probability of the occurrence of low water levels in the Lake, this is included in the Environmental Study Report. A reasonable design level for the intake tower is 605msl.
CHAPTER 3

FIELD SURVEY
3. FIELD SURVEY

Field survey has been executed for the various aspects of the Project. Due to lack of topographic and sub-soil information both investigation were executed. The Baseline study and Environmental study are subject of separate reports.

3.1. Topographical Survey

The route of the transmission pipeline has been surveyed to establish longitudinal sections for hydraulic design. The levels along the route of the pipeline have been established on every 50 m, and along river and stream crossings every 5-10 m. Site surveying on the locations of the structures (reservoir, intake tower and bridge and intake site) established the levels on the grid 5 m x 5 m. Concrete bench marks are established every 500 m along the pipe route and on every bend.

The surveying carried out under the Project is sufficient for detailed design to be prepared. The total quantities of topographical surveying are:

- Longitudinal section levelling of transmission pipeline route 42 km
- Bush clearing for longitudinal levelling 42 km
- Site surveying (reservoir, intake site) 0.4 ha
- Bench mark establishing 92 No.

All topographical survey work has been plotted and is shown on the drawings, either as longitudinal section (transmission pipeline) or site plans (intake site, reservoir).

3.2. Soil Investigation

3.2.1. General

The sub-soil conditions for transmission pipeline route and major structures have been investigated.

For the transmission pipeline, sub-soil conditions have been investigated at a frequency of roughly 500 m. On each river, stream crossing three trial pits have been investigated, one in the middle and one on each bank of the crossing.

Seismic investigation has been done on the intake sites. First location of the intake site proved difficult for foundation construction. The alternative location proved to be suitable for required type of structure.

A total of 86 trial holes have been made, with the total excavation depth of 172 m. The full results of investigations are available in Volume II of this report.
3.2.2 Soil Types

The main soil and ground types encountered by the investigation are:

- Sand
- Silt
- Gravel
- Boulders
- Gneiss

The majority of trial holes encountered sand with varying degrees of intermixed gravel and gneiss.

Rock outcrop has been encountered in only two holes, there will be some cases where large boulders will have caused refusal and where such a boulder is classified as rock.

Standing ground water has been encountered in all the holes at the river crossings and some of the stream crossings.

3.2.3. Test Results

3.2.3.1. Sieve analyses

Sieve analyses were carried out on samples taken at sites for structures, and additionally, for selected transmission pipeline route.

The results of the sieve analyses show acceptable material for backfilling pipes and structures. Considering the presence of ground water on every river crossing there is a need for special dewatering equipment during construction and special protection of the pipes along the river and stream crossings.

3.2.3.2. Compaction analyses

Compaction analyses indicate a general suitability of all materials for compaction.
3.2.4. Implications of Soil Types for Excavation and Structures

An assessment has been made of the material to be excavated along the transmission pipeline route, based on the analysis of the results of the trial holes. The material to be excavated has been classified into:

- normally excavatable material (soft material)
- hard material, rippable
- rock requiring blasting

Based on typical trench dimensions of 600 mm width, and 1150 mm to 2500 mm depth, the following indicates the percentage of each type of material required to be excavated for transmission pipeline and intake site:

<table>
<thead>
<tr>
<th>Description</th>
<th>Soft</th>
<th>Hard</th>
<th>Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transmission pipeline</td>
<td>89%</td>
<td>9%</td>
<td>2%</td>
</tr>
<tr>
<td>Intake site</td>
<td>41%</td>
<td>31%</td>
<td>28%</td>
</tr>
</tbody>
</table>

The above percentages have been used to estimate volumes of, and cost for, hard material and rock excavation. However, it must be stressed that the frequency of test holes (every 500 m) prevents an accurate assessment and the above percentages will vary during construction.
CHAPTER 4

COMPONENTS OF THE PROJECT
4. COMPONENTS OF THE PROJECT

4.1. General

Raw water will be pumped from Chicamba Lake to header reservoir on the hill overlooking the Lake. From the reservoir the water will be transported by gravity to the treatment plant of the Water Company of Chimoio. In accordance with the original Terms of Reference the rehabilitation of the existing treatment plant was not in the scope of this Project. However it was agreed that if the funds are available the rehabilitation of the plant will take place pending on the available budget.

The following components are forming the system and will be built under this Project:

- Intake tower with high lifting pumps on the Chicamba Lake
- Access bridge to the intake tower
- Transportable pump, which will be used when the water level in the Lake falls under 605 m
- Guard, store house incorporating office, accommodation facilities and storage
- Fenced Intake site incorporated Guard, store house, transformer tower etc.
- Access road
- Pressure pipeline including valves and fittings
- Header reservoir of 100 m³ located on the hill overlooking the Lake
- Gravity pipeline including valves and fittings
- Radio communication

4.2. Intake Tower

4.2.1. Location and intake level

The location of the intake tower has been chosen in the bay of the Lake about 800m from the dam to reduce the impact of the power generation and waves. Also it has been considered the sub-soil conditions in order to minimise the
cost of foundation for the tower. Based on the available data about design levels and historical water levels of Chicamba Lake the intake level for the tower has been chosen to be 605 m.

4.2.2. Civil work brief description

The dimensions of the tower are calculated to accommodate three submersible pumps. One duty and one stand-by are in the scope of this Project, third one is for future extension. The height of the tower from the bottom of the sump to the top of the top slab is 23.20 m.

The material for construction of the tower will be reinforced concrete class 25 with high tensile bars as reinforcement. Construction of the tower can be executed with sliding formwork, depending on the contractors equipment.

As a protection of the mechanical and electrical equipment a house will be constructed on the top of the tower.

Material used for the construction of the house will be 230 mm. bricks plastered inside. Roof sheeting will be corrugated galvanised iron sheets.

Delivery pipes will be two steel pipes 300 mm Diameter encased in concrete to prevent floating of the pipes. The slope of the pipes will be 3% in direction of the Lake to enable back washing of the pipes. Back washing of the delivery pipes can be executed whenever the water level in the lake is lower than maximum level in the Lake by closing the valves on the delivery pipes, pumping the water in the tower using a mobile pump and opening the valves on the delivery pipes. Gate valve with 19 m mild steel spindle, gear box, bevel and hand wheel shall be installed on the delivery pipes to allow opening and closing from the top slab.

Minimum flow through two 300 mm steel pipes is 450m³. Which is more than adequate even for future extension.

Intake head structure is designed to allow required quantity of water in delivery pipes. Material will be concrete class 15. To prevent debris, fishes and crocodiles from entering the intake tower intake head structure will be equipped with the grid 20 mm bars at 100 mm centres.

4.2.3. Structural calculation

Structural calculation are enclosed in Annex No. 1
4.2.4. Mechanical equipment

The intake tower shall be equipped with two submersible pumps (1 duty, 1 stand-by). The pumps shall be with the following characteristics:

\[ Q=100 \, m^3/h, \, H=180 \, m. \]

**Calculation of the pump**

Required capacity \( 2000 \, m^3/day = 100 \, m^3/h \) (working time 20 hours/day)

**level difference:**

- header reservoir \( 764 \, m \)
- intake level \( -605 \, m \)
- friction losses \( +6.61 \, m \)
- \( L=2050 \, m, \, 250 \, mm \) dia steel pipe \( \sum \) \( 165.61 \, m \)

The pumps were chosen KSB BPN 374/6+10A 733/2

The outlet pipe of each pump shall be provided with all necessary non-return and isolating valve. Pumps and vertical ID 250 mm steel double flanged pipes 3000 mm long will be supported with 600 mm plate welded support on the top slab.

The intake tower house shall be equipped with a hand operated hoist, working weight 2000 kg to permit removal of equipment from the tower. Transportation of the equipment over the access bridge shall be by trolley.

4.2.5. Electrical equipment

The intake site will be electrically supplied from a distribution sub-station to be constructed by the Electricity Company. Electricity will be available with the following characteristics:

\[ 380 \, V; \, \, 50 \, Hz; \, \, 315 \, KVA \]

Distribution board and motor control panel will be installed inside the house. The house will be equipped with 2 x 36 W 1200 mm long surface mounted batten type fluorescent luminaires with single metal reflector.
The lightning protection system will be installed with 4 earth electrodes to protect the equipment from lightning.

Level switches shall be provided in the intake tower and header reservoir for the following functions:

i) Water protection against dry running of pumps-intake tower

ii) High level cut-out-header reservoir

iii) Low level cut-in-header reservoir

Level switches shall be floating mercury switches, enclosed in watertight plastic casing, FLYGT Type ENH-10. The switches shall be freely suspended and adjustable to the required level.

A pressure switch shall be installed on the discharge manifold to provide cut-out of the pumps. The pressure switch shall be adjustable and shall be set, on commissioning, at the field established "closed valve" pressure for the pump sets.

4.2.6. Alarms

The following visual and audible alarms shall be provided:

- Electricity supply-mains failure (battery operated)

- Header reservoir-overflow

- Operation of any electrical protective device or safety cut-out

- Failure of any duty motor

Visual indication of alarms shall be located on the appropriate switchboard or instrument panel. Audible indication of alarms shall be located both in the Intake Tower and in the Office of the Guard-store house. A siren shall also be installed on the outside of transformer tower.

4.3. Access Bridge to the Intake Tower

4.3.1. General

Access bridge shall be constructed as a link between intake site and the tower. Material shall be reinforced concrete class 40. Reinforcement shall be high tensile reinforcement bars.
4.3.2. Method of construction

First, the 11.183 m embankment shall be constructed using excavated rock. On the embankment shall be placed precast concrete deck. Reinforced concrete upstand restraint shall be constructed after next 11.183 m. Upstand restraint shall be based on natural stone masonry pier with class 20 concrete core. Next precast concrete deck shall be placed between embankment and a pier. Third precast concrete deck shall be placed between the pier and tower. After the precast deck is placed in the position and to required level reinforced concrete bridge deck shall be cast insitu. Every 11 m movement joints shall be constructed using Malthold DPC.

4.3.3. Structural calculation

Structural calculation are enclosed in Annex No. 2.

4.3.4. Other ancillary

Steel pipes ID 250 mm length 4000 mm internally lined and externally protected shall be placed on the bridge fixed to bridge deck using brackets and support brackets.

Electrical cables shall be fixed to a bridge beam on the galvanised cable tray.

Metal fence shall be constructed 700 mm high using 20 mm Dia. bars at 125 mm central. The handrail shall be protected with thermoplastic cover.

4.4. Transportable Pump

4.4.1. General

The water in the Chicamba Lake might fall below the elevation 605 m which corresponds to the inlet of the intake tower. It might eventually reach 585 m, the minimum level of operation of the hydro-electric plant.

4.4.2. Civil work

The reinforced ramp, on which the pump shall be transported will be constructed from the store room to the existing water level in the Lake while the execution of the Project is going on. Any extension of the ramp will be constructed separately from this Project according to necessity and when the water falls below this level.

4.4.3. Electro / mechanical work
The pump shall be mounted on the non roadworthy trailer. The pump shall be following characteristics:

\[ Q = 110 \text{m}^3 / \text{h}, \ H = 30 \text{ m} \]

The pump has been chosen KSD ELK/ELB 40.

Flexible reinforced rubber pipes 100 mm Dia shall be connected to the pump. The pump shall pump the water from the Lake to the tower sump during low water levels in the Lake. To enable operation of the pump a hand-winch with the stainless steel cable shall be mounted in the store room.

4.5. Guard - Store House

4.5.1. General

The Guard, store house shall include the following facilities:

- 1 office
- 2 bedrooms with the lounge for the guards off duty
- 1 shower and toilet
- 1 storage room for the transportable pump and pipes

The house shall be furnished with the hard furniture as follows:

- Metallic cushioned armed chair No. 2
- Metallic office desk No. 1
- 4 tier steel file cabinet No. 1
- 3 seater wooden settee with foamed cushions No. 1
- Metallic single bed with foam mattress No. 2
- Storage cupboard No. 2
- Kitchen table with 4 chairs No. 1
- Storage cupboard (lounge) No. 1
- Key storage No. 1

4.5.2. Civil work

Material shall be brickwork plastered both sides. Roof shall be galvanized iron sheeting. Sanitary waste from the toilet and shower shall be disposed in a septic tank and soakaway. Water supply for the house shall be connected to the Chicamba Township water main which crosses the intake site access road.

Sanitary wastes from the bathroom shall be disposed in a septic tank which is connected to a soakaway.
4.6. **Intake Site and Transformer Tower**

4.6.1. **General**

In order to provide efficient operation of the intake tower as well as security of the equipment, an intake site shall be built at the entrance of the bridge.

4.6.2. **Intake Site**

Intake site shall consist of Guard-store house, transformer tower, sufficient turning area for the truck in case of any operational or maintenance requirements.

The level of the intake site shall be 625.20m. To prevent soil erosion due to wave activity the edge of the site shall be protected with 500 mm free drainage rockfill. The wire fence 1800 mm high shall be built around the site because of security reasons. The erosion of the slope (if it is not rock) shall be protected with adequate vegetation.

4.6.3. **Transformer Tower**

To provide the adequate power supply transformer tower shall be built as per requirements of EdM/SHER.

4.6.3.1. **Structural Calculation**

Structural calculation are enclosed in Annex No 3.

4.6.3.2. **Size of transformer**

The size of transformer has been calculated for the worst loading case. The adequate transformer shall be 315KVA.

Detailed calculation of loading are enclosed in Annex No 4.

4.7. **Access Road**

4.7.1. **General**

Access road shall be constructed from existing road to Chicamba Dam to Intake site. Total length shall be 1850 m.
4.7.2. Method of Construction

The route of the road shall follow the existing dirt track to the "restaurante", after that it shall follow a direct path to the intake site. The potholes on the existing dirt track shall be filled with adequate material and compacted to 93% MOD AASHTO. Material for wearing course shall be selected gravel compacted to 95% MOD AASHTO.

To prevent erosion of the road on the edge of the road shall be placed free drainage rockfill 60-250 mm compacted to 95% MOD AASHTO. Drainage channels 100 mm deep shall protect the road from water flow on the slopes. Ground slopes shall be protected from erosion by vegetation.

4.7.3. Culverts

Water from the slopes shall be controlled by 4 culverts which shall be constructed on every stream, crossing the road. Culverts shall be class S culvert pipes with 100 mm. bed and haunch placed below subgrade layer with 1:100 fall.

4.8. Pressure Pipeline

4.8.1. General

First 2153 m of the transmission main from the intake to the header reservoir shall be steel pipes longitudinal welded, internally lined, externally coated, 6 m length, working pressure 22 bars.

4.8.2. Earthworks

The pipes shall be laid in the trenches 600 mm wide, with the minimum cover of 800 mm above the top of the pipe. When crossing the streams, including the portion of the pipe which is going to be laid under the maximum level of the Lake, the invert level shall be 2.0 m deep. Below the invert of the pipe 100 mm of sand shall be placed. First 200 mm above the pipe shall be sand or screened granular material compacted to 88% MOD AASHTO.

The reminder of trenches shall be backfilled with selected granular material compacted to 96% MOD AASHTO. Excavation shall be protected with all necessary planking and strutting to ensure the safety of workmen.

Material to be excavated shall be divided into three classes, Rock, Hard Material and Soft Material.

Rock shall be defined as:

(i) Solid undecomposed boulders exceeding 0.10 cubic metres; or
(ii) Solid unweathered crystalline material in bulk or in banks or ledges, which can not be broken up or removed except by the use of explosives.

**Hard Material** shall be defined as:

Material, not rock, which requires the use of pneumatic tools, mechanical breakers, or special cutting tools to mechanical excavators for its practical removal.

Excavation for concrete structures such as inspection chambers and for thrust and anchor blocks shall be to required depth and as near as possible to the external dimensions of the structure.

The excavation must be free of water at any time during the pipe laying and construction of concrete structures.

Marker tape shall be placed 300 mm above the pipeline. The material for Marker tape shall be blue PVC or polyethylene mesh or ribbon at least 50 mm wide, incorporating a corrosion resistant tracing system.

### 4.8.3. Pipe Laying

Laying instruction of the manufacturer shall be explicitly followed. All pipes shall be sound and clean before laying. When laying is not in progress the open ends of the pipes shall be closed by water tight plugs or other approved means. Good alignment shall be preserved in laying.

### 4.8.4. Concrete Works

All concrete and reinforced concrete works shall comply with the following standard specification as appropriate:

- BS 8110 The structural use of concrete
- BS 8007 Code of practice for "Design of concrete structures for retaining aqueous liquids"
- BS 12 Ordinary Portland cement
- BS 882 Aggregates for concrete
- BS 812 Water absorption of aggregate
- BS 3148 Water for washing aggregates
- BS 5775 Admixtures of concrete
- BS 1881 Strength test of concrete
- BS 4449 Carbon steel bars
- BS 4482 Cold reduced steel wires
- BS 4483 Steel fabric
- BS 1052 Finally annealed mild steel wire
BS 4466 Cutting and bending of reinforcement
BS 8100 Tolerances of placement of reinforcement
BS 8110 The minimum concrete cover of reinforcement
BS 2499 Hot poured joint sealants
BS 5212 Cold poured polymer-based joint sealant
BS 4254 Two-part polysulphide-based sealants
BS 5889 Silicone based building sealants
BS 903 Rubber waterstops

4.8.4.1. Valve Chambers

Washout chambers and air valve chambers shall be placed as per longitudinal sections. The exact position shall be indicated on site. All concrete chambers shall be painted outside with two layers of bitumen. All chambers shall be lockable.

4.8.4.2. Anchor Blocks

Anchor, thrust blocks shall be placed on all the bends and at the slopes as follows:

<table>
<thead>
<tr>
<th>gradient</th>
<th>spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>up to 1 in 2</td>
<td>6</td>
</tr>
<tr>
<td>from 1:2 to 1:4</td>
<td>12</td>
</tr>
<tr>
<td>from 1:4 to 1:5</td>
<td>18</td>
</tr>
<tr>
<td>from 1:5 to 1:6</td>
<td>24</td>
</tr>
<tr>
<td>flatter than 1:6</td>
<td>not required</td>
</tr>
</tbody>
</table>

4.8.4.3. Marker Posts

Marker posts shall be made of precast reinforced concrete and placed 1.0 m from central line of pipes. They shall be positioned on every bend of the pipeline, on the straight portion every 500 m.

4.8.5. Valves and Accessories

All valves and accessories are designed for a pressure at least equal to two times the normal working pressure where they will be installed. As far as possible all equipment of the same type shall be from one manufacturer and they must be clearly marked with the name of manufacturer, flow direction arrows and the pressure for which they are designed. All valves shall be manually operated and open left (counter clockwise).
4.8.6. Hydraulic analyses

Hydraulic calculations of friction losses for the steel pipe have been calculated based on Hazen-Williams formula as follows:

\[ J = 10,708 \times \left( \frac{Q}{C} \right)^{1.85} \times D^{-4.87} \]

Where:
- \( J \) = Friction losses (m)
- \( Q \) = Flow (l/s)
- \( C = 140 \) (Hazen-Williams coefficient)
- \( D \) = Pipe diameter (mm)

For the \( Q = 2000 \text{ m}^3/\text{day} \); \( D = 250 \text{ mm} \) and \( C = 140 \) \( \Rightarrow \)

\[ J = 6.61 \text{ m} \]

Water hammer / surge pressure is the crucial factor in the design of this pressure pipeline, therefore the calculations are given below inclusive of water hammer / surge pressure.

4.8.6.1. Water Hammer / Surge Pressure

This phenomenon occurs frequently in pressure pipelines and is defined as the periodic pressure oscillation which move back and forth along a pipeline. The phenomenon occurs when closing and opening valves, starting and stopping pumps or when any other operating condition changes in the pipeline.

The example of calculation of water hammer / surge pressure is shown below. The determining factor of the size of water hammer / surge pressure is the speed with which the pressure wave travels along the pipeline. The following formula has been used:

\[ C_p = C_w \times \frac{1}{\sqrt{1 + \left( \frac{E_w}{E_p} \times \frac{dm}{t} \right)}} \]

Where:
- \( C_p \) = Wave celerity (m/s)
- \( C_w \) = Wave celerity in a column of water (1425 m/s)
- \( E_w \) = Elasticity modus of water (2070 N/mm²)
- \( E_p \) = Elasticity modus of pipe material (N/mm²) uPVC \( E_p = 3000 \); Steel \( E_p = 210000 \)
- \( dm \) = mean diameter of pipeline which is equal to internal pipe diameter \( di \) plus pipe wall thickness \( t \) (\( dm = di + t \))
- \( t \) = Pipe wall thickness (mm)
The size of pressure oscillation after a valve closure depends on the time it takes to close the valve, and on the so called reflection time of the pipeline. If the flow is cut during a period shorter than this reflection time the maximum pressure increase may be calculated by use of the following formula:

$$H = \pm Cp \times \frac{Vo}{g}$$

Where: $H$ = Maximum pressure increase (+) or decrease (-) (mhw)
$Cp$ = Wave celerity as calculated by use of the above formula (m/s)
$g$ = Acceleration of gravity ($\approx 9,81 \text{ m/s}^2$)
$Vo$ = Flow velocity just before valve closure (m/s)

The reflection time is calculated as follows:

$$To = \frac{2L}{Cp}$$

Where: $To$ = Reflection time (s)
$L$ = Length of pipeline (m)
$Cp$ = Wave celerity (m/s)

In the case of having a longer valve closing time than the reflection time of the pipeline, the pressure increase is lower than the calculated in the above formula.

**Calculation of water hammer / surge pressure:**

Steel pipe ID 250 mm  Wall thickness 4,8 mm; $Q = 27,78 \text{ l/s}$ ;
$Vo = 0,7 \text{ m/s}$ ; Static head = 159 m; Friction losses = 6,61 m ;
Length $L = 2050 \text{ m}$  $\Rightarrow$

$$Cp = 1027,26 \text{ m/s}$$
$$H = 73,30 \text{ m (mhw)}$$
$$To = 4,00 \text{ s}$$

$$\Delta h = 159 + 6,61 + 73,30$$
$$\Delta h = 238,91 \text{ m}$$

In order to avoid installation of some form of water hammer protective device to reduce pressure, because of high operational and maintenance cost, steel pipe is accepted. The reflection time is very short therefore the pressure increase should be lower than calculated in the above formula.
4.8.7. Pipe Material - Steel Pipes

Due to high risk of water hammer and to avoid installation of water hammer protective device to reduce pressure, the material for pressure main has been designed to be GRW longitudinal welded pipe to SABS 719 Gr. B. 250 mm internal diameter, wall thickness of 4.8 mm.

The main concern regarding the use of steel pipe is corrosion. The long continuous lengths of welded steel pipelines increase the opportunities for corrosion to develop. Welded joints are a potential source of increased corrosion risk due to difficulty in applying protection after welding. Corrosion of steel pipelines does not generally cause catastrophic failure, rather a progressively increasing loss of water through leakage.

To protect steel pipe from corrosion pipes shall be internally lined with smooth, dense, centrifugally spun cement mortar lining. External corrosion protection shall be with fusion bonded low density polyethylene in thicknesses from 2.0 to 3.0 mm or similar approved by the Engineer on Site.

4.8.8. Testing of the Pipelines

The pressure in the pipeline shall be raised steadily until a pressure of 1.5 times the operating pressure is reached in the lowest part of the section, and the pressure shall be maintained at this level for a period of one hour. The pumps shall then be disconnected and no further water shall be permitted to enter the pipeline for the further period of one hour. At the end of this period the original pressure shall be restored by pumping and the loss measured.

The permissible loss in all welded steel pipeline shall not exceed 0.1 litres per mm of pipe diameter per kilometre of pipe per 24 hours for each 30 m of test head. In uPVC pipe the permissible loss shall not exceed 2.0 litres per metre nominal bore per kilometre length per metre head (calculated as the average head applied to the section) per 24 hours.

4.8.9. Cleansing and Disinfection of the Pipeline

At the conclusion of pipelaying flushing out of the pipeline shall be executed. Disinfection of the pipeline shall be carried out using chlorinated raw water from Chicamba Lake. Residual of not less than 20 mg/l of free chlorine shall be acceptable for the structures. For the pipeline a residual of at least 20 mg/l for a period of 2 hours shall be acceptable.
4.9 Header Reservoir

4.9.1. General

The header reservoir will be located on the hill overlooking the Chichamba Lake at the elevation 762 m. The head will be enough to carry the water by gravity to the treatment plant of Chimoio. It is not going to be a storage reservoir only a pressure break tank, therefore the capacity will be 100 m$^3$.

The primary function of this reservoir is to protect the gravity main from overpressure by breaking the head created by the submersible pumps in the Intake Tower. This head depends on the water level in the Chicamba Lake, which can vary by 20 m. The secondary function of the reservoir is to protect the gravity main from water hammer caused by intake pumps and to reduce the effect of water hammer at the pressure main.

4.9.2. Civil Work

The reservoir shall be built in the ground 1.0 m below original ground level. At the bottom level of the reservoir sludge disposal pipe with the valve shall be installed. Raw water has been taken from the Lake so settlement already took place. The quantity of sludge will be minor. Sludge shall be disposed in the channel filled with free drainage rockfill to allow soaking of the sludge in the ground.

On the top slab three ventilation pipes shall be installed and entrance for maintenance of the reservoir. Entrance shall be covered with cast iron manhole lockable cover.

To prevent unauthorised entry in the reservoir area it shall be fenced with 1.8 m high wire mesh fence with lockable gates. Access road to the reservoir area shall be constructed as a dirt track 3.0 m wide.

4.9.3. Structural Calculation

Structural calculation are enclosed in Annex No. 5.

4.9.4. Electro-Mechanical work

Level switches shall be installed in the reservoir to control the intake pumps. Item 4.2.4. refers. The outlet pipe shall be equipped with the gate valve in case of maintenance of the gravity main.
4.10 Gravity Main

4.10.1. General

The pipeline between header reservoir and the treatment plant will be constructed in 200 mm and 250 mm diameter unplasticized PVC pipe in class 6, 9, 12, 16 and 20 according to hydraulic gradient and altitude.

4.10.2. Earthworks

Item 4.8.2. is applicable.

4.10.2.1. River-Stream Crossings

The banks of all the river-stream crossings shall be protected with wire mesh gabions 400 mm thick. The pipes crossing the rivers and streams shall be encased in concrete to prevent possible erosion during rainy season.

4.10.3. Pipe Laying

Item 4.8.3. is applicable.

4.10.4. Concrete Works

Item 4.8.4. is applicable

4.10.5. Valves and Accessories

Item 4.8.5. is applicable.

4.10.6. Hydraulic analyses

Hydraulic calculation are based on the formula first developed by Blasius for smooth pipes, based on the premise that for smooth pipes the friction factor is solely dependent on the Reynolds number. The formula is as follows:

\[ P = \frac{158.2}{Re^{0.25}} \times \frac{L}{D} \times V^2 \]

Where

- \( P \) = Pressure loss in pascals (Pa) per metre of pipe
- \( Re \) = Reynolds number
- \( L \) = Pipe length in metres (m)
- \( D \) = Pipe diameter in metres (m)
- \( V \) = Velocity of flow in metres per second (m/s)
\[ \text{Re} = \frac{V \times d}{\eta / \delta} \]

Where

- \( \text{Re} \) = Reynolds number
- \( V \) = Velocity of flow in metres per second (m/s)
- \( d \) = Inside pipe diameter (mm)
- \( \eta / \delta \) = Kinematics fluid viscosity (m²/s)

From the flow chart which is based on the above mentioned formula friction losses for different classes of pipes for the required flow \( Q = 27.78 \text{ l/s} \) are as follows:

<table>
<thead>
<tr>
<th>pipe diameter (mm)</th>
<th>class of the pipe</th>
<th>friction losses (m/100m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>6</td>
<td>0.15</td>
</tr>
<tr>
<td>250</td>
<td>9</td>
<td>0.17</td>
</tr>
<tr>
<td>250</td>
<td>12</td>
<td>0.19</td>
</tr>
<tr>
<td>250</td>
<td>16</td>
<td>0.24</td>
</tr>
<tr>
<td>250</td>
<td>20</td>
<td>0.32</td>
</tr>
<tr>
<td>200</td>
<td>9</td>
<td>0.46</td>
</tr>
<tr>
<td>200</td>
<td>12</td>
<td>0.50</td>
</tr>
</tbody>
</table>

The highest point (level 691.99 m) on the gravity main is at km 28.675 from the header reservoir. In the table below is shown the friction losses from the reservoir to that critical point on the entire gravity main. When considering the classes of the pipes in addition to the level difference between high and low points on the pipeline the possible water hammer is taken into account.

<table>
<thead>
<tr>
<th>pipe diameter (mm)</th>
<th>class of the pipe</th>
<th>length (m)</th>
<th>friction losses (m/100m)</th>
<th>total friction losses (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>6</td>
<td>2678</td>
<td>0.15</td>
<td>4.05</td>
</tr>
<tr>
<td>250</td>
<td>9</td>
<td>5697</td>
<td>0.17</td>
<td>9.69</td>
</tr>
<tr>
<td>250</td>
<td>12</td>
<td>10500</td>
<td>0.19</td>
<td>19.95</td>
</tr>
<tr>
<td>250</td>
<td>16</td>
<td>9200</td>
<td>0.24</td>
<td>22.08</td>
</tr>
<tr>
<td>250</td>
<td>20</td>
<td>600</td>
<td>0.32</td>
<td>1.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>57.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>decrease of characteristics with age + 5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TOTAL</td>
</tr>
</tbody>
</table>
Calculation of the required level of the header reservoir:

Level of the highest point on the pipeline: 691.99 m
Total friction losses: \[ \sum 752.56 \text{ m} \]

To secure water supply, in case of some minor leakages of the pipeline, the minimum water level in the reservoir shall be 761.80 m.

In order to reduce pressure at the inlet at the treatment plant 3050 m of the gravity main before the treatment plant shall be 200 mm diameter. The pressure at the inlet of the pipeline shall be 41.91 m. Depending on the quality of the existing reservoir at the treatment plant it might be necessary to construct pressure break chamber before inlet to the existing reservoir. During the construction of the project necessary investigation of the existing reservoir should be executed.

4.10.6.1. Water Hammer / Surge Pressure

This phenomenon occurs frequently in pressure pipelines and is defined as the periodic pressure oscillation which move back and forth along a pipeline. The phenomenon occurs when closing and opening valves, starting and stopping pumps or when any other operating condition changes in the pipeline.

Although the water is transported through the pipeline by gravity due to level difference all the gravity main will be under pressure. The example of calculation of water hammer/surge pressure is shown below for the biggest level difference on the gravity main between two points. The determining factor of the size of water hammer / surge pressure is the speed with which the pressure wave travels along the pipeline. The following formula has been used:

\[
C_p = C_w \times \frac{1}{\sqrt{1 + \left(\frac{E_w}{E_p} \times \frac{d_m}{t}\right)}}
\]

Where:
- \( C_p \) = Wave celerity (m/s)
- \( C_w \) = Wave celerity in a column of water (1425 m/s)
- \( E_w \) = Elasticity modus of water (2070 N/mm\(^2\))
- \( E_p \) = Elasticity modus of pipe material (N/mm\(^2\)), uPVC \( E_p = 3000 \); Steel \( E_p = 210000 \)
- \( d_m \) = mean diameter of pipeline which is equal to internal pipe diameter \( d_i \) plus pipe wall thickness \( t \) (\( d_m = d_i + t \))
- \( t \) = Pipe wall thickness (mm)

The size of pressure oscillation after a valve closure depends on the time it takes to close the valve, and on the so called reflection time of the pipeline. If
the flow is cut during a period shorter than this reflection time the maximum pressure increase may be calculated by use of the following formula:

\[ H = (\pm) \frac{C_p \times V_o}{g} \]

Where:
- \( H \) = Maximum pressure increase (+) or decrease (-) (mhw)
- \( C_p \) = Wave celerity as calculated by use of the above formula (m/s)
- \( V_o \) = Flow velocity just before valve closure (m/s)
- \( g \) = Acceleration of gravity (= 9.81 m/s\(^2\))

The reflection time is calculated as follows:

\[ T_{o} = \frac{2L}{C_p} \]

Where:
- \( T_{o} \) = Reflection time (s)
- \( L \) = Length of pipeline (m)
- \( C_p \) = Wave celerity (m/s)

In the case of having a longer valve closing time than the reflection time of the pipeline, the pressure increase is lower than the calculated in the above formula.

Calculation of water hammer/surge pressure:

PVC 250 mm class 16 (assumption); \( Q = 27.78 \text{ l/s} \); \( V_o = 0.6 \text{ m/s} \);
Static head = 182.25 m; Friction losses = 32.96 m;
Length \( L = 17350 \text{ m m} \implies \)

\( C_p = 433.52 \text{ m/s} \)
\( H = 26.51 \text{ m (mhw)} \)
\( T_o = 80.04 \text{ s} \)

\( \Delta h = 182.25 - 32.96 + 26.51 \)
\( \Delta h = 175.8 \text{ m} \)

PVC class 20 is accepted for that area of a high pressure. Due to very long distance between two points the pressure increase at the critical point will be lower even class 16 will be sufficient, but because it is a short section of the pipeline and in the river crossing class 20 is designed.

4.10.7. Pipe Material - uPVC Pipes and Fittings

The properties of uPVC pipes allow it to be used instead of steel, cast iron asbestos cement, etc. pipes. It has the highest stiffness per unit cost of the
engineering plastics and is therefore most commonly used in load bearing applications such as pressure pipes. It has other features such as: low specific gravity, good chemical and weather resistance, very smooth bores and the long term flow characteristics vary very little from those of newly laid pipes.

Unplasticized polyvinyl chloride (uPVC) pressure pipes, joints and fittings shall comply with relevant provisions of standards as set out below:

<table>
<thead>
<tr>
<th>uPVC pipes</th>
<th>BS 3505</th>
</tr>
</thead>
<tbody>
<tr>
<td>uPVC joints and fittings</td>
<td>BS 4346</td>
</tr>
<tr>
<td>uPVC solvent cements</td>
<td>BS 4346</td>
</tr>
</tbody>
</table>

Storage of pipes must be executed strictly in accordance with manufacturer recommendation. It is essential that pipes are stacked with the socket and spigot ends alternating and with the sockets projecting so that there is a barrel to barrel contact along the lengths of pipe. During storage all pipes must be protected from ultraviolet light.

4.10.8. Testing of the Pipelines

Item 4.8.8. is applicable.

4.10.9. Cleansing and Disinfection of the Pipelines

Item 4.8.9. is applicable
CHAPTER 5

OPERATION & MAINTENANCE
5. OPERATION & MAINTENANCE

5.1 Present O & M Situation

Agua de Chimoio is responsible for all water supply system in Chimoio. The water supply scheme is uncomplicated and easy to operate and little planning is done. Even when there is enough raw water (only 6 months a year) operation of existing equipment is complicated due to lack of spare parts and necessary tools. The major problems for operation and maintenance can be summarised as follows:

- Shortage of skilled staff
- Shortage of transport for staff and materials
- Shortage of spare parts for the equipment
- Purchasing procedures entail delays in the supply of spare parts and materials and so prevent the prompt and efficient execution of repairs
- Missing system drawings, preventing proper maintenance planning and execution
- Lack of communication between intake, treatment plant, workshop and main office

These and other problems are reflected in the actual condition of system. About 40% of the pipe network is estimated to be in poor condition. Although the distribution network is not in the scope of this Project rehabilitation work is required.

5.2. O & M for Extended system

Based on the existing O & M problems proposal for efficient O & M for the extended system relate mainly to:

- More effective planning and execution of O & M activities;
- The provision of necessary vehicles and equipment;
- The adjustment of staffing requirements and working procedures to match major features of the extended system notably:
  - Intake site at Chicamba Lake
  - Pressure main
- Head reservoir site
- Gravity main

- Communication system to be established

- Continuous monitoring of the efficient operation of the system

- Change in emphasis from ad-hoc repairs to routine (preventive) maintenance in order to lengthen service life of the system

- Appropriate training to be provided, to operate the new equipment
  Detailed operation and maintenance manuals shall be prepared by the contractor. The manuals should cover the testing, operation, maintenance, dismantling and repair of all equipment and should include explanations of the function and purpose of each item supplied.

- Long term training needs should be analysed and converted into long-term training plans. Training plans for each job category should be developed, including a plan for senior management. A consultant could assist in developing the plans.
CHAPTER 6

CONSTRUCTION PROGRAMME AND CASH FLOW
6. CONSTRUCTION PROGRAMME AND CASH FLOW

Construction programme and cash flow are based on the commencement date 15/09/1994. The Christmas Holidays have been included in the Construction programme from 16/12/1994 to 02/01/1995. If the project implementation starts on any later date the construction programme has to be modified.
6.1. Construction Programme

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>15/09/94</th>
<th>15/10/94</th>
<th>15/11/94</th>
<th>16/12/94</th>
<th>02/01/95</th>
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<td>Access Road</td>
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<td>Pipelaying Pressure Main</td>
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<td></td>
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<tr>
<td>Pipelaying Gravity Main</td>
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<td>River Crossings</td>
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<td>Structures on the Pipelines</td>
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6.2 Cash Flow
CHAPTER 7

CONFIDENTIAL COST ESTIMATE
### 7. CONFIDENTIAL COST ESTIMATE

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
<th>Provis. Quant.</th>
<th>Unit Rate (US$)</th>
<th>Total (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>PRELIMINARY &amp; GENERAL</td>
<td>L.S.</td>
<td>1</td>
<td>505,000</td>
<td>505,000</td>
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<tr>
<td></td>
<td><strong>Total 1</strong></td>
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<tr>
<td>2.</td>
<td>PIPELINE</td>
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<tr>
<td>2.1.</td>
<td>Excavation of trenches in any material including backfill and removal of surplus material.</td>
<td>m</td>
<td>42,269</td>
<td>9.76</td>
<td>412,545</td>
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<tr>
<td>2.2.</td>
<td>Supply steel pipes and fittings longitudinal welded, internally coated, externally protected, 6m length, working pressure 22 bars.</td>
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<td>2153</td>
<td>70</td>
<td>150,710</td>
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<td>2.3.</td>
<td>Lay pipes and fittings including testing, flushing and disinfection</td>
<td>m</td>
<td>2153</td>
<td>15</td>
<td>32,295</td>
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<tr>
<td>2.4.</td>
<td>Supply uPVC pipes with rubber sealing rings, 6m length all classes</td>
<td>m</td>
<td>40,116</td>
<td>23.74</td>
<td>952,354</td>
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<tr>
<td>2.5.</td>
<td>Supply fittings for uPVC pipes with rubber rings</td>
<td>No</td>
<td>235</td>
<td>138.83</td>
<td>32,625</td>
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<td>2.6.</td>
<td>Lay uPVC pipes and fittings including testing, flushing and disinfection</td>
<td>m</td>
<td>40,116</td>
<td>1.5</td>
<td>60,174</td>
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<tr>
<td>2.7.</td>
<td>Construct all the chambers on the pipeline including supply of all materials, valves and fittings</td>
<td>No</td>
<td>88</td>
<td>3,400</td>
<td>299,200</td>
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<tr>
<td>2.8.</td>
<td>Provide all material and construct anchor and trust blocks, marker posts, road and railway crossings</td>
<td>L.S.</td>
<td>1</td>
<td>41,950</td>
<td>41,950</td>
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<tr>
<td>2.9.</td>
<td>Provide and construct all river and stream crossings</td>
<td>L.S.</td>
<td>1</td>
<td>46,080</td>
<td>46,080</td>
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<tr>
<td></td>
<td><strong>Total 2</strong></td>
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<td>2,027,933</td>
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</table>
### Chimoio Water Supply Project

**Final Design Report**

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
<th>Provis. Quant.</th>
<th>Unit Rate (US$)</th>
<th>Total (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.</td>
<td><strong>INTAKE TOWER AND BRIDGE</strong></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>3.1.</td>
<td>Excavation in any material and placement of rock material</td>
<td>L.S.</td>
<td>1</td>
<td>96,935</td>
<td>96,935</td>
</tr>
<tr>
<td>3.2.</td>
<td>Grouting of all fractures</td>
<td>L.S.</td>
<td>1</td>
<td>20,000</td>
<td>20,000</td>
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<tr>
<td>3.3.</td>
<td>All concrete work for intake tower and bridge including supply of all material for concrete and reinforcement bars</td>
<td>L.S.</td>
<td>1</td>
<td>231,441</td>
<td>231,441</td>
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<tr>
<td>3.4</td>
<td>Supply and installation of all electromechanical work for intake tower and bridge including all the testing and commissioning</td>
<td>L.S.</td>
<td>1</td>
<td>367,550</td>
<td>367,550</td>
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<tr>
<td>3.5.</td>
<td>House on the intake tower</td>
<td>L.S.</td>
<td>1</td>
<td>12,000</td>
<td>12,000</td>
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<tr>
<td></td>
<td><strong>Total 3</strong></td>
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<td><strong>727,926</strong></td>
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<tr>
<td>4.</td>
<td><strong>TRANSFORMER TOWER</strong></td>
<td>L.S.</td>
<td>1</td>
<td>15,000</td>
<td>15,000</td>
</tr>
<tr>
<td></td>
<td><strong>Total 4</strong></td>
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<td></td>
<td><strong>15,000</strong></td>
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</tr>
<tr>
<td>5.</td>
<td><strong>GUARD AND STORE HOUSE</strong></td>
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</tr>
<tr>
<td>5.1.</td>
<td>Construction of guard, store house</td>
<td>L.S.</td>
<td>1</td>
<td>75,000</td>
<td>75,000</td>
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<tr>
<td>5.2.</td>
<td>Construction of septic tank and soakaway including all drain pipes</td>
<td>L.S.</td>
<td>1</td>
<td>8,075</td>
<td>8,075</td>
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<tr>
<td>5.3.</td>
<td>Connection to existing water pipe</td>
<td>m</td>
<td>1,200</td>
<td>12</td>
<td>14,400</td>
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<tr>
<td>5.4.</td>
<td>Furniture</td>
<td>L.S.</td>
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<td><strong>Total 5</strong></td>
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<td><strong>117,475</strong></td>
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</table>
### INTAKE SITE AND ACCESS ROAD

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
<th>Provis. Quant.</th>
<th>Unit Rate (US$)</th>
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<tbody>
<tr>
<td>6.</td>
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<tr>
<td>6.1.</td>
<td>Excavation in any material and backfill around the structures for intake site</td>
<td>L.S.</td>
<td>1</td>
<td>114,018</td>
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<td>6.2.</td>
<td>Construction of ramp, free drainage rockfill and fence</td>
<td>L.S.</td>
<td>1</td>
<td>7,756</td>
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<tr>
<td>6.3.</td>
<td>Construction of access road</td>
<td>L.S.</td>
<td>1</td>
<td>71,705</td>
<td>71,705</td>
</tr>
<tr>
<td>6.4.</td>
<td>Supply and installation of all electromechanical work at the intake site including all testing and commissioning</td>
<td>L.S.</td>
<td>1</td>
<td>84,200</td>
<td>84,200</td>
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</tbody>
</table>

**Total 6** | 277,679 |

### STORAGE RESERVOIR

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<th>Provis. Quant.</th>
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</tr>
</thead>
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<tr>
<td>7.</td>
<td></td>
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</tr>
<tr>
<td>7.1.</td>
<td>Excavation in any material, backfilling and construction of dirt track</td>
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<td>5,752</td>
<td>5,752</td>
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<tr>
<td>7.2.</td>
<td>All concrete work for the reservoir including supply of the material for concrete and reinforcement bars</td>
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<td>12,812</td>
<td>12,812</td>
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<td>7.3.</td>
<td>Supply and installation of all pipes and fittings, wire fence and construction of chambers</td>
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<td>8,290</td>
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<td>7.4.</td>
<td>Supply and installation of float switches including 2050m of cable</td>
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<td>1</td>
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</table>

**Total 7** | 48,854 |

### SUNDARY ITEMS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
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<th>Unit Rate (US$)</th>
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<tbody>
<tr>
<td>8.</td>
<td></td>
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</tr>
<tr>
<td>8.1.</td>
<td>Calculation, construction and as-built drawings, operational manuals and 30 days instruction period</td>
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<td>36,800</td>
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<td>8.2.</td>
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<td>Notice boards</td>
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<td>Supply and install radio communication including all permissions</td>
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**Total 8** | 124,844 |
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<th>Item</th>
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<td>9.</td>
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<td>9.1.</td>
<td>Additional soil investigation</td>
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<td>Connection and installation of power supply</td>
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<td>9.3.</td>
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<td>10.</td>
<td>DAYWORK AND DE-MINING</td>
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<td>10,000</td>
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<td>10.1</td>
<td>Daywork</td>
<td>L.S.</td>
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<td>50,000</td>
<td>50,000</td>
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<td>10.2</td>
<td>De-mining survey and de-mining</td>
<td>L.S.</td>
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<td>10,000</td>
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<td><strong>60,000</strong></td>
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<td>11.</td>
<td>MANAGEMENT, SUPERVISION AND FACILITIES FOR ENGINEER</td>
<td>L.S.</td>
<td>1</td>
<td>555,000</td>
<td>555,000</td>
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<td>12.</td>
<td>SUPPLY AND INSTALLATION OF TREATMENT PLANT 2000m³/day</td>
<td>L.S.</td>
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<td>70,000</td>
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7.1. Summary Cost Estimate-Phase II

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<tr>
<td>1.</td>
<td>Preliminary and General</td>
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<tr>
<td>2.</td>
<td>Pipe line</td>
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<tr>
<td>3.</td>
<td>Intake tower and bridge</td>
<td>727,926</td>
</tr>
<tr>
<td>4.</td>
<td>Transformer tower</td>
<td>15,000</td>
</tr>
<tr>
<td>5.</td>
<td>Guard and store house</td>
<td>117,475</td>
</tr>
<tr>
<td>6.</td>
<td>Intake site and access road</td>
<td>277,679</td>
</tr>
<tr>
<td>7.</td>
<td>Storage reservoir</td>
<td>48,854</td>
</tr>
<tr>
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<td>12.</td>
<td>Supply and installation of treatment plant</td>
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CHAPTER 8

LIAISON
8. LIAISON

During the design stage of the project (phase I) Consultant maintained close contact with the relevant Officers of the following National and Provincial Directorate:

- National Directorate of Water Affairs (DNA)-Maputo
- National Directorate for Roads and Bridges (DNEP)-Maputo
- National Directorate for Mapping (DINAGECA)-Maputo
- Ministry of Finance-Maputo
- Ministry of Health-Maputo
- Provincial Directorate for Construction and Water (DPCA)-Chimoio
- Agua de Chimoio-Chimoio
- Chimoio Energy Company (SHER)-Chimoio
- City Council-Chimoio
- Companhia do Pipeline Mocambique-Zimbabwe (CPMZ)-Beira, Mutare
- Manica Provincial Government
- Rural Water Programme, Manica
- CFM-Centro (Central Region, Mozambican Railways)
- Empresa de Tobaccos de Manica
- Semoc-Mozambique
CHAPTER 9

PROJECT RISKS AND CONSTRAINTS
9. PROJECT RISKS AND CONSTRAINTS

9.1. General

As part of the services offered under this Project, the Consultant will assess the possible risks to and constrains on Project execution and sustainability.

At this point in the Project some general risks and constrains are foreseen. These are listed below and shown the reasons for the existence of the risk (or constrain) and the measures for avoidance or reduction.

9.2. Major Risks

9.2.1. Delays to Project Execution

9.2.1.1. Delay in obtaining finance on time

Water level in the Chicamba Lake is at the moment the lowest for the last 20 years, therefore the construction of the intake tower, delivery channel and intake head structure will be cheaper and should be finished before the rainy season starts.

To avoid this delay the finance construction should be closed as soon as possible to enable the Contractor to start construction immediately after letter of commencement is issued.

9.2.1.2. Delay in obtaining land acquisition approval

The Consultant strongly recommends that DNA, DPCA, AdC and Chimoio Town Council acquaints itself with the respective sites and required way-leave, and formally approaches the Land Boards with a view to securing the required sites and way-leave.

9.2.1.3. Delay due to possible presence of mines on the site

Although no official information received by the Employer indicates the presence of mines on the Site, the Consultant recommends that due precautions and investigation to be taken before commencement of work specially on the hills overlooking the Lake where the header reservoir shall be built or in the area around that location.

Such precaution and investigation may include obtaining local information, carrying out a mine survey, de-mining, or clearance of the site by scoring the surface with bulldozer equipped with a high blade as a protection in case of blast (where it is possible because of the high slope of the ground level).
9.2.2. Unsustainability of the installed system

The inadequate system maintenance can cause the unsustainability of the installed system.

To avoid that it is essential to establish the realistic operational and maintenance requirements and appropriate technology.

9.3. Minor Risks

9.3.1. Inability to construct system components as designed

During soil investigation trial pits were excavated 500 m spaced, there is a possibility of occurrence of the different material between the investigated trial pits, which might caused the re-design of some components of the system.

Close monitoring of sub-soil conditions is required by the Engineer on Site during the excavation in order to do the necessary amendments of drawings if conditions differs from original design.

9.3.2. Failure of the system to provide required capacity

The failure might occur if there is inadequate system maintenance leading to high losses.

To reduce or avoid possible high losses of the system it is essential to establish realistic operational and maintenance requirement as well as to equip the AdC to fulfil their obligations.
ANNEX No. 1

INTAKE TOWER

STRUCTURAL CALCULATION
# Bending Schedule

**CHIMOIO WATER SUPPLY PROJECT**

**INTAKE TOWER**

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All Bending Dimensions are in accordance with BS4466
## Bending Schedule

**CHIMOIO WATER SUPPLY PROJECT**

**INTAKE TOWER SLABS**

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All Bending Dimensions are in accordance with BS4466
### Bending Schedule

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<tr>
<td>$M = 200 \times 10^2$</td>
<td>Moment at the top end</td>
</tr>
<tr>
<td>$W = 100 \times 15^2$</td>
<td>Weight estimate</td>
</tr>
<tr>
<td>$C = 0.65 \times 135$</td>
<td>Calculated value</td>
</tr>
<tr>
<td>$A = 1 M$</td>
<td>Cross-sectional area</td>
</tr>
<tr>
<td>$P = \frac{24.54 \times 10^2}{0.12 \times 1000 \times 15^2}$</td>
<td>Force calculation</td>
</tr>
</tbody>
</table>

**Result:**

- $M = 200 \times 10^2$ (Moment at the top end)
- $W = 100 \times 15^2$ (Weight estimate)
- $C = 0.65 \times 135$ (Calculated value)
- $A = 1 M$ (Cross-sectional area)
- $P = \frac{24.54 \times 10^2}{0.12 \times 1000 \times 15^2}$ (Force calculation)

**Notes:**

- $A = 1 M$ (Cross-sectional area)
- $P = \frac{24.54 \times 10^2}{0.12 \times 1000 \times 15^2}$ (Force calculation)
- $C = 0.65 \times 135$ (Calculated value)
REINFORCEMENT DESIGN for a slab
for Cross Section: SUPPORT - cantilever

Width: \( b = 1000 \text{ mm} \), \( b_w = 1000 \text{ mm} \)
Depth: \( h = 200 \text{ mm} \), \( h_f = 0 \text{ mm} \)
Cover: top \( 40 \text{ mm} \), bot. \( 50 \text{ mm} \), sides \( 0 \text{ mm} \)
Span: \( l = 1.600 \text{ m} \), Width of Support \( 300 \text{ mm} \)

Materials:
Concrete \( f_{cu} = 40 \text{ N/mm}^2 \), Steel \( f_y = 460 \text{ N/mm}^2 \)

Moments and Forces:
Bending: \( M = 26.6 \text{ kNm} \)
Shear: \( V = 33.2 \text{ kN} \), \( UDL = 0.00 \text{ kN/m} \)

REINFORCEMENT:
Bending:
Bottom:
req.: \( A_s = 485 \text{ mm}^2 \) -->
provide 8 T12 at 200 mm c/c with \( A_s = 904 \text{ mm}^2 \) (0.28%)

Shear:
Shear Stress: \( \sigma_v = 0.23 \text{ N/mm}^2 \), \( \sigma_{vc} = 0.82 \text{ N/mm}^2 \)
No Shear reinforcement required!
Design at 294 mm from Cl for \( 0.0 \text{ kN} \), \( \sigma_v = 0.00 \text{ N/mm}^2 \)
\( A_{sv} = 0 \text{ mm}^2/\text{m} \)
Provide 0 legs R 0 at 0 mm --> \( A_{sv} = 0 \text{ mm}^2/\text{m} \)
Curtailment:
\(< 0 \text{ mm} >|<---0\text{ Links R 0}--->|<---0\text{ mm} ----->|
Synopsis:

1. The proposed water tank is a 
   RCC concrete square box concrete, to 
   be erected at the end of the site and 
   will have dimensions 5000 x 5000 x 
   2300. It is to be joined to the 
   building on the request.

2. Structural design has been carried 
   out to receive masonry walls.
   - Dead & imposed loads increased by 20% 
   
   - Masonry to be brickwork 7.5 by
   - Rainwater to be vertical drainage

3. Masonry thrust
   - Not necessarily, but to be considered
   
   - Masonry thrust from 1 m to 2 m. 7.5 (one metre)
   happening in all cases. 2 m.

   - Horizontal thrust of (to be used
   
   - Masonry not to exceed 6.2 or higher.

   - Concrete or bending moments
   
   - Shearing forces on the
   
   - Columns to be similar to other

   - All parts to act as lintels

   - Separate column footings should
   
   - Be connected by ties designed to
   
   - Take a thrust or pull it. 7.5.

   - Design is based on calculated
   
   - Strength and ductility to withstand
   
   - Requirements for bearings in foundation.

   - ACI Code 318 -

   - Seismic Design
### Intake Tower: Sketch

#### Calculation: Down Beta

1. **Given Data:**
   - Height = 73m above normal level
   - 10m below

2. **Material Properties:**
   - Density of Soil: 187 kN/m³
   - Angle of Friction: 34°
   - Angle of Repose: 35°
   - Yield Strength of Material: 15 kN/m²
   -地下水位: 10 m below

3. **Wind Speed Calculation:**
   - Basic wind speed: 35 m/sec
   - Design speed: 51 x 5.5 x 5 = 1,512 kN/m²
   - At the site, it is an enclosed area
   - Height above ground level = 23.0m A.B.
   - Assume S2 = 1.1
   - z = 1.1

---

#### Intake Structure:

- **Dimensions:**
  - Base: 3700 x 3700
  - Height: 73m above normal level, 10m below

---

#### Section

The following assessment constitutes:

- Characteristic of earth which has caused re-assessment of the design approach
- Re-assessment undertaken.

### Notes

- Properties of material:
  - Density of soil: 187 kN/m³
  - Angle of friction: 34°
  - Angle of repose: 35°
  - Yields strength of material: 15 kN/m²

- Loadings:
  - Roof dead: 4.0 kN/m²
  - Roof live: 4.0 kN/m²

---

### Related Properties

- Concrete:
  - fck = 25 N/mm²
  - fa = 260 N/mm²
  - fcd = 16.0 N/mm²
  - fc = 25 N/mm²

- Steel:
  - fy = 150 N/mm²

---

### Calculations

- Basic wind speed: 35 m/sec
- Design speed: 51 x 5.5 x 5 = 1,512 kN/m²
- At the site, it is an enclosed area
- Height above ground level = 23.0m A.B.
- Assume S2 = 1.1
- z = 1.1
- S2 = 4.1
- z = 4.1
### Interface Tower Foundations

- **Assume** $C_L = 0.5$  
- $C_{03}$ (LHS $= 1A$)

### Internal Forces

<table>
<thead>
<tr>
<th>$C_{03}$</th>
<th>$N_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.5$</td>
<td>$1356$</td>
</tr>
</tbody>
</table>

### Section Forces

- **Service Dead Load (Service Only)**
  - $33 \times 0.3 \times (5.1 \times 5) \times 2 \times 2.5$
  - $= 174.72 \times 2.5$

- **Service Load $N_1 = 1356 \text{ kN}$**

- **Additional Dead Load + Live Load**
  - $3 \times 300 = 900 \text{ kN}$
  - $3 \times 300 = 900 \text{ kN}$

- **Total Service load = 2366 kN**
Assumption for hydrostatic forces:

Assuming hydrostatic pressure:

Max. hydrostatic moment:

Calculation of soil pressure:

Moment: Due to water pressure:

\[ M = \frac{1}{2} b h \left( \frac{1}{2} + \sin \theta \right) \]

\[ d = \frac{369}{2} \approx 0.27 \quad H = 10 \text{ m} \]

Pressure:

Max. moment pressure:

\[ = 18 \times 0.27 \times 10 \]

\[ = 506.5 \text{ kNm}^2 \]

\[ = 10 \text{ kNm} \]

\[ = 18 \times 0.27 \times \frac{10}{16} \]

\[ = 51.0 \text{ kNm} \]
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---

**JOB**

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<tr>
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</tr>
<tr>
<td>Ref.</td>
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**SECTION**

<table>
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</thead>
<tbody>
<tr>
<td>Tower</td>
</tr>
<tr>
<td>Design</td>
</tr>
</tbody>
</table>

---

**Moment at Base**

\[ M = 2717.0 \text{ kNm} \]

**Moment, Hydrostatic**

\[ M = 57556 \text{ kNm} \]

**Moment, Soil Pressure**

\[ M = 58000 \text{ kNm} \]

**Total Max. Moment**

\[ M = 4623 \text{ kNm} \]

---

\[ B = 4 \times 1000 \times 1000 = 4000000 \text{ kNm} \]

**Area of Base**

\[ 4000000 = 529.3 \text{ kNm} \]

\[ B = 529.3 \text{ kNm} \]

**Modulus of Elasticity**

\[ E = 10000 \times 1000 = 500000 \text{ kNm} \]

\[ E = 500000 \text{ kNm} \]

**Moment from Soil Pressure**

\[ M = 58000 \text{ kNm} \]

---

**Moment from Horizontal Load**

\[ 15 \times 5931 \times 23/2 \]

\[ = 80883 \text{ kNm} \]

**Moment from Wind Load**

\[ 1568 \text{ kNm} \]

**Total Moment**

\[ M = 2717.0 \text{ kNm} \]

**Analyzing Hydrostatic Pressure Since its Always Enhanced Internally + External**

\[ 1000 \text{ kN/m}^2 \text{ on Tower} \]

---

**But Since the Base is Formed Through Continuum to Hard Rock Channel, the Interface Structures Used are Formed in**

\[ 1000 \text{ kN/m}^2 \text{ on Tower} \]

---

\[ 225.8 \text{ kNm} \]

\[ 1000 \text{ kN/m}^2 \text{ on Tower} \]

---

\[ 225.8 \text{ kNm} \]

\[ 1000 \text{ kN/m}^2 \text{ on Tower} \]

---

**Survey Report**

---

**Design Parameters**

---

**Comments**

---

---
The proposed construction plans require that walls at right angles which are provided to resist the horizontal forces should carry at least 25% of the horizontal load.

The resistance of column loads to the moment of inertia of the wall.

Deformation to be determined as shear amount

\[ \delta = \frac{P}{EI} \]

\[ a = \frac{12}{E} \times \frac{6}{2} = 1.33 \]
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JOB  
SECTION  

$500,000 Part # 1, 1985 Section 3.

A. 2.6.3.2.2. Internal support

(a) A temporary support of timber not exceeding 1.2m in length

(b) Use of supporting a wooden floor

(c) Use in such manner as best

(d) Make sure that

(e) Be so that the beam shall not be

(f) Continue for three weeks.

B. 3.2.1.1. Long units = $7/00

C. 3.2.1.1. Appearance of the members

D. 3.2.4. Deviation for members

E. 3.2.4. Deviation for members

F. 3.2.4. Deviation for members

G. 3.2.4. Deviation for members

H. 3.2.4. Deviation for members

I. 3.2.4. Deviation for members

J. 3.2.4. Deviation for members

K. 3.2.4. Deviation for members

L. 3.2.4. Deviation for members

M. 3.2.4. Deviation for members

N. 3.2.4. Deviation for members

O. 3.2.4. Deviation for members

P. 3.2.4. Deviation for members

Q. 3.2.4. Deviation for members

R. 3.2.4. Deviation for members

S. 3.2.4. Deviation for members

T. 3.2.4. Deviation for members

U. 3.2.4. Deviation for members

V. 3.2.4. Deviation for members

W. 3.2.4. Deviation for members

X. 3.2.4. Deviation for members

Y. 3.2.4. Deviation for members

Z. 3.2.4. Deviation for members

(a) An increase in the internal tension of an initial load

(b) A load of 10x10^-6/10

(c) Thus the def. in length of a sample

(d) Number of beams due to a 25% change in the

(e) To be determined by

(f) By the formula

(g) Compute stress in the beam of 7 N/mm² in an

(h) I have to wait three months

(i) I cannot do it.
(4) Design of strengthening system are in the
middle of 50 to 100 mm of 1.5 mm per 3m length

C. Cover - 30

8.3: Connection Joint - A deep cut
Discontinuity slab reinforcement in both
Concrete and reinforcement.

8.5: Beam Joint with matrix
To provide resistance connection, non-shrink
mix meeting BS 5752, BS 8110 & B.S.2218.
BS 8110: Part 1: 1985

Section Three

Table 3.1: Nominal cover to all reinforcement requirements.

Conditions of nominal cover

(see 3.2.4)

A concrete mixture capable of providing Working and Degree of
strength and workability.

Max. Free Water
Concrete Ratio
Max. Concrete Content

(kg/m³)

Largest Grade of Concrete

CAD
Design of 60 Water Tower

Considerations:

1. Handbook on the Unified Code suggest that wall at right anges which are prone to assist the unbalanced walls should carry at least 25% of the uniform load.

2. The resistance of wind loads to the moments of inertia of the

3. Deflection is assumed as same amount

\[ V = C = \frac{P H^3}{8 EI} \]

\[ P_1 = \frac{P H^3}{(H/2 + H/2) \times 2} \]

\[ f_1 = \frac{P H^3 \times 2}{2E I} \]

DESIGN:

1. Effective Heights and thicknesses:

The base and knee slabs rest on wall and ties will be provided to stabilise connection. Connection will be designed to resist moment.

For a slender wall, the thickness should not be 1/30 of the effective height.

Upper wall effective height = 7.6 m; Thickness = 300 mm

Intermediate wall effective height = 7.4 m; Thickness = 300 mm

Lower wall effective height = 6.8 m; Thickness = 300 mm

Table:

Height 300 mm

3.60 m above slender walls within the base 300 mm relocated limits.
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JOB
SECTION

Wind Loading:

Roof: Dead Load = 0.95 kN/m² = 2.5 kN/m
Upper wall: Dead = 0.85 kN/m² = 0.85 kN/m
Roof Dead = 7.6 kN/m
Lower wall = 8.68 kN/m
Total Dead Load at Top of Wall = 200.68 kN/m

Roof Uplift = 0.75 kN/m² = 0.75 kN/m
Roof Uplift = 7.3 kN/m² = 7.3 kN/m
Total Uplift = 17.73 kN/m

Wind Load = 0.35 x 1.93 x 3.7
= 3.69 kN/m

Moment at Top of Lower Wall
= 3.69 x (7.3 x 3.7)
= 90.76 kNm

Moment at Top of Upper Wall
= 2.09 x 23²/2
= 99.65 kNm

Wind Shear at Base = 3.69 x 2.3
= 84.87 kN
Checking the tension at base of wall:

Assume: \( \text{wall} = 3.7 \text{m long} \times 2 \text{m high} \)

Height = 5.2 m

Module E = \( 4.9' \times 4.9' = 0.335 \text{m}^2 \)

Maximum load due to wind

\[ 0.03 \times 6.5' = 0.205 \text{ KN/m} \]

\[ 0.02 \times 6.5' = 0.13 \text{ KN/m} \]

\[ 0.335 \times 0.205 + 0.335 \times 0.13 = 0.23 \text{ KN/m} \]

The dead load at top of fence = 200.76 \( \times 0.7 \)

or there is tension in the wall.

Shore = \( \frac{1}{4} \times 200.76 = 50.19 \geq 84.37 \)

Increase bottom section by force to 84.37 to avoid tension in the wall.

le.

upper roof load = 2.5

\[ \text{upper roof} = 67.6 \text{ (considered)} \]

\[ \text{upper wall} = 55.5 \]

\[ \text{lower wall} = 55.5 \]

lower wall = 99.10

upper roof = 7.65

lower roof = 7.65

concrete = 8.43

Total dead load = \( 337.76 \text{ KN} \times 0.7 \text{ for wind} \)

Shore = \( \frac{1}{4} \times 257.7 = 64.42 \text{ KN} \)

Complete with wind shear = 84.87 KN.

But: 13 KN/m² for wind for 30° angle

High wind shear due to wind could be considered 1388.
**WALL DESIGN**

The general wall is designed for loads at midheight.

**The most adverse combination is the dead + imposed loads.**

**The design loads are:**

\[ \text{Dead Load} = 1.4 \times (1.65 \times 3 + 0.62 \times 3) \times 1.6 \]

\[ = (28.13 + 4.96) \text{ kN/m} \]

\[ = 33.09 \text{ kN/m} \text{ acting at } 1/3 \text{ way from base} \]

\[ c_v = 75.09 \times 67. - 211.94 \times 50 \]

\[ = 287.03 \]

\[ = 19.4 \text{ mm} \]

**The thickness should not be less than 0.105 \times 300 = 31.5 \text{ mm}**

\[ = 15 \text{ mm} \]

**Allowable stress = \( \sigma_a = 31.5 \)**

\[ \text{Cleaved H/P ratio } = \frac{31.5}{6.1} = 5.1 \]

**Wall height \( h = 2 \text{ m} \)**

**Ultimate load**

\[ nh = (300 - 2 \times 1.54) \times 0.4 \times 25 \]

\[ = 2012 \text{ kN/m} \geq 287.03 \text{ kN/m} \]
The additional eccentricity due to reflection:

\[ e_M = \frac{6.2}{250} = 0.0248 \]

\[ e_M = 0.025 \text{ m} 

See \& examine wall. The ultimate load:

\[ N_u = (250 - 1.2 \times 19.4 - 2 \times 70.56) \times 0.4 \times 25 \]

\[ = 256 \text{ kN/m} \]

The ultimate applied load is:

\[ = 267.03 \text{ kN/m} \text{ at wind height of wall} \]

Hence the design is satisfactory.

Wind shear at base = 84.67 kN

Dead load of structure = 257.76

\[ \frac{1}{6} \times 257.76 = 42.96 \]

Although this is less than 84.67, the wind loading has been assumed as very conservative.

Assuming 0.7 kN/m² can compare to 1.3 kN/m²

A reduction in the shear wind force can be achieved. No further treatment will be employed into.

iv. Cover Content:

Reinforcement, tee, shear, and temperature to be provided in accordance with Clause 55.9.2

CP 120 for high yield steel.

Area required = \[ 0.25 \times 300 \times 1000 \times 100 = 10000 \text{ mm}^2 \]

But half this amount on each face.
Section faces were on sections of
Tendons at equilibrium levels.

From previous calculations:

\[ \text{Tw} = 293.0 \text{ kN} \]

Section pressures by Computer Eng.

\[ I = \frac{b_0^3}{12} \]
\[ = \frac{5.0 \times 3.7^3}{12} - 4.5 \times 3.7^3 \frac{2}{12} \]
\[ = 21.52 - 11.17 \]
\[ = 10.35 \text{ kN}^2 \]

\[ A = 6.1 \times 3.7 - 0.5 \times 3.1 \]
\[ = 19.57 - 15.95 \]
\[ = 3.62 \text{ kN}^2 \]

\[ E = \frac{1}{I} \times 12.86 \]
\[ = 0.50 \text{ kN}^2 \]

\[ \text{Min. shear} = \frac{0.94 A \text{ kN}}{\frac{A}{E}} \]

\[ M = 257 \times (5.1 + 3.7) = 2269.2 \]
\[ \text{Min. sec.} = 0.94 \times 2269.2 - 1A + \frac{218 \times 23/5}{5.59} \]
\[ = 14.93 \text{ kN} \]
\[ - 406 \text{ kN/} \text{m}^2 \]
\[ 100 \times 0.2 \text{ N/mm}^2 \text{ (tensile)} \]
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JOB

SECTION

Where force concrete is used T. thickness to be 110 cl. 3.9.3.6.1

Check on thickness of 30mm.

Total design load = 1.4 x 257.74 + 1 x 37.725

= 340.56 + 37.725

N 104.5 3.4 cN/um

Vertical load on concrete wall is 2.3 kN/m. Ac. T. Sup

The wall is used for 10% reduced by 50%

so N 0.33 kN Ac.

W capacity by unit area in

= 0.33 x 25 x 3500 x 0.3

= 2.75 kN/m

Capacity of wall thickness satisfactory.
ANNEX No. 2

ACCESS BRIDGE

STRUCTURAL CALCULATION
## Bending Schedule

**CHIMOIO WATER SUPPLY PROJECT**

**BRIDGE & INTAKE HOUSE R.C. DETAILS.**

<table>
<thead>
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<th>Member/Section</th>
<th>Bar Mark</th>
<th>Size</th>
<th>No in each</th>
<th>Total No</th>
<th>Length [mm]</th>
<th>Shape Code</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
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<td><strong>BRIDGE</strong></td>
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<tr>
<td>3# SECTIONS</td>
<td>06</td>
<td>T16</td>
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<td>20 STR</td>
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<tr>
<td>(IN 11# PANELS OF SIZE 1017 mm)</td>
<td>07</td>
<td>T12</td>
<td>4</td>
<td>12</td>
<td>11000</td>
<td>20 STR</td>
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<td>22# IN EACH PANEL</td>
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<td>414</td>
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<td>60 670</td>
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<td></td>
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<td>238</td>
<td>714</td>
<td>2000</td>
<td>60 670</td>
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<td>366</td>
<td>1050</td>
<td>81 335</td>
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All Bending Dimensions are in accordance with BS4466
Bridge deck design.

Synopsis:

Bridge deck is 2.3 m span.

300 mm precast concrete beams.

Ultimate A Class of 110 mm thickness with an additional 110 mm thickness is to be used.

Step beam shutter.

Super beam shutter.

Determination of wheel loads - the shear and moment tensor.
Nicholas O'Dwyer & Partners
Consulting Engineers

2 Robert Mugabe Road
P.O. Box UA 424, Union Avenue, Harare
Telephone 702531-5
Telefax: 702536
Telex: 22345 CODWYR ZW

JOB ..............
SECTION ..............

Table 11

- Moment Footpath + Concentrated load on
down (considering fact
in any footpath)

However, known load = 2 Tonne of Trolley = 20 kN

Inclining resistance of Trolley = 20 kN

Concrete unconfined

Load on 2.0 Tonne Swing:

20 kN

2.0 kN

\( f_k = 0.25 \text{ kN/m}^2 \)

\( f_k = 5.0 \text{ kN/m}^2 \)

\( f_k = 200 \text{ kN/m}^2 \)

Basic span/effective depth = 2.0 m

Min effective depth of span = 2.0 m x 0.7 = 1.4 m

Effective depth = 2.0 m

Vary effective depth 20 mm

Min concrete exposed to live load = 25 mm

Allowing some loss as half of
rainfall on roof, etc.

Systematic depth \( h = \frac{20 + 25 + K}{1.5} = 110 \text{ mm} \)

Shear weight of slab = 25.5 \( \times 296.9 \times 10^{-3} = 7.60 \text{ kN/m}^2 \)

Total dead load = 6.10 \( \text{kN/m}^2 \)

See in notes of design.

Ultimate load = \( \left( f_k \times 1.5 \times 1.5 \right) 2.3 \)

Ultimate load = \( \left[ \frac{1}{2} \left( 2.0 \right) \right] + \left( 1.5 \times 2.0 + 1.5 \times 6.25 \right) \times 2.3 \)

\( = 59.28 \text{ kN/m}^2 \)
\[ M = 69.28(2.3/8 + 1) \sqrt{(20/10)} \times 2.3 \]

\[ = 17.0 + 72.9/2 \]

\[ = 90.8 kN \cdot m \]

\[ S = \sqrt{H/2} \]

\[ \frac{M}{H^2} = \frac{5.83}{100^2} \times 1.06 = 14.7685 > 9.1625 \]

\[ T = 160 \]

\[ d = 160 - 23 - 8 = 117 \text{mm} \]

\[ \frac{M}{d^2} = \frac{58.3}{100^2} = 0.0342 \]

\[ = 3.93 \]

\[ h = 0.35 + \frac{(471 - f)}{120(0.9 + M/d^2)} \]

\[ Assum \ N = 1.02 \]

\[ h = 0.30 \]

\[ Horizonta shear stress \ \\max = 2300 = 16.69 \]

\[ \max \ D = 17 \text{ in adequate} \]
ENDING REINFORCEMENT:

\[
\frac{M}{Ed} \frac{E}{2} \leq \frac{53.8 \times 10^6}{1000 \times 11730} = 0.13
\]

Level XEM 2 = \( 90 - 0.95 \times 117 \)

= 90 mm

\[
A_t = M - \frac{53.8 \times 10^6}{0.875 \times 10^5} = \frac{53.8 \times 10^6}{0.875 \times 250 \times 117} = 8888 \text{ mm}^2
\]

Provide \( 81 \times 875 \text{ mm}^2 \) @ 6, 20.\% of beam. As prov = 2681.

Minimum beam beam length = \( \frac{81}{2} \times 150 = 11625 \text{ mm} \)

[Signature]

At face of support \( V = 59.625 + 20 \times 1.6 \)

= 59.625

Since \( \sigma = \frac{57.9 \times 10^3}{1000 \times 117} = 0.467 \geq 0.2 \times 30 \approx 4.5 \)

\[ \frac{100 \times 160}{1000 \times 117} = 2.29 \]

\( V_c = 1095 \text{ kN} \)

\( V_c \geq 2V_0 \) No further checks.
### Details of Structure

- **Job Number:** 
- **Section:**

#### Calculation Details

1. **Crosssection Area Calculation:**
   \[
   A = 0.69 < V_\phi (1.42/\pi)
   \]

2. **Concrete Thickness Calculation:**
   \[
   t_{concrete} = 30mm
   \]
   \[
   t_{concrete} = 16.5mm
   \]

3. **Anticracking Crack:**
   \[
   d_{crack} = 48.33 \text{ in (G6 or G8)}
   \]

4. **Description:**
   - **Concrete Slab:**
     \[
     0.25 \times 250 \times 1000 / 100
     = 62.5 \text{ mm}^2
     \]
     - Use B20 or B150, As per requirements.

### Summary

- **Bridge:**

- **Checked By:**
- **Made By:**
- **Date:**
- **Ref:**
- **Sheet No:**
- **Telephone:**
- **Telex:**
  - 22345 CODWYR ZW
REINFORCEMENT DESIGN for a rectangular beam
for Cross Section: SPAN - simply supported

Width: \( b = 300 \text{ mm} \), \( bw = 300 \text{ mm} \)
Depth: \( h = 750 \text{ mm} \), \( hf = 0 \text{ mm} \)
\( d = 698 \text{ mm} \), \( d' = 60 \text{ mm} \)
Cover: top 40 mm, bot. 40 mm, sides 30 mm
Span: \( 1 = 12.000 \text{ m} \), Width of Support 300 mm

Materials:
Concrete \( f_{cu} = 25 \text{ N/mm}^2 \), Steel \( f_y = 410 \text{ N/mm}^2 \)

Moments and Forces:
Bending: \( M = 504.0 \text{ kNm} \)
Shear: \( V = 168.0 \text{ kN} \), UDL \( = 28.00 \text{ kN/m} \)

REINFORCEMENT:
Bending:
Bottom: requ.: \( A_s = 2529 \text{ mm}^2 \) -->
provide 5 X25 with \( A_s = 3125 \text{ mm}^2 \) (1.39%)

Shear:
Shear Stress: \( \sigma = 0.81 \text{ N/mm}^2 \), \( \sigma_c = 0.72 \text{ N/mm}^2 \)
Design at \( 6000 \text{ mm} \) from Cl for 144.4 kN, \( \sigma = 0.70 \text{ N/mm}^2 \)
Minimum Links: \( A_{sv} = 552 \text{ mm}^2/m \)
Provide 2 legs R10 at 275 mm --> \( A_{sv} = 571 \text{ mm}^2/m \)
CORBEL FOR BEARER:

- Concrete mix = 300 N/mm²
- Section load = [Diagram]

\[
\begin{align*}
\text{Total load} &= 520kN \times 0.8 + 15 \times 9.3 \times 220 + 1250kN \\
&= 816.16 + 491.6 \\
&= 817.76kN
\end{align*}
\]

- Tensile force, \( f_t \) = \( 0.04 \times 817.76 = 32.7kN \)
- Maximum bearing stress = \( 0.8 / 0.76 = 32N/mm² \)

Assuming effective length of beam face is 250mm

- Minimum width = \( (320.76 \times 10^3) / (32 \times 230) \)

\[
= 110.23mm \times 0.2 = 11.05mm
\]

- Some clear at beam face:

\[
3 = \sqrt{v^2 + 0.015} \times f_t
\]

See grade of concrete 20. Concrete max = 5.05 N/mm².

\[
\begin{align*}
v &= (817.76 \times 10^3) / (320 \times 5.05) \\
&= 9.005kN/mm²
\end{align*}
\]
\[ \omega = 200, \omega_{200} = 0.268, \nu = 0.31, \alpha = 0.125 \]
\[ \theta = 0.67 \]
\[ c_0 = 0.67 \times 550 = 368.5 \text{ mm}^2 \]
\[ \text{shear} = (2.5 - 2) 	imes (550 - 350.5) \]
\[ = 101.5 \text{ mm}^2 \]
\[ F_x = 150 \times 700 = 105000 \text{ N} \]
\[ = 478.27 \text{ kN} \]
\[ \text{from strain hence } f_x = 181/350 = 0.5146 \]
\[ = 0.00172 \text{ kN/mm}^2 \]
\[ f_s = 351 \text{ N/mm}^2 \]
\[ A_s = (478.27 \times 10^3)/351 = 1363 \text{ mm}^2 \]
\[ \text{peeling: } 350 \times 2 = 700 \text{ AS poor: } 1473 \text{ mm}^2 \]
\[ 100A_s / h = 100 \times 1252/320 \times 550 = 9.4 \text{ mm} \]
\[ \text{Allowable shear stress: } \]
\[ = 0.6 \times 10^6 \text{ N/m}^2 \]
\[ = 3.3 \times 10^6 \text{ N/m}^2 \]
\[ \nu = 5.91 \text{ N/mm}^2 \]
\[ b_0 = (4.5 - 3) = 3 \times 5.91 = 177 \text{ N/mm}^2 \]
\[ \text{force: } 350 \times 150 = 14325 \text{ N} \]
**Nicholas O'Dwyer & Partners**

Consulting Engineers

2 Robert Mugabe Road
P.O. Box UA 424, Union Avenue, Harare
Telephone: 702531-5
Telefax: 702538
Telex: 22345 CODWYR ZW

**Sheet No.** NA

**Made By**

**Checked By**

**Date**

---

**JOB**

**SECTION**

**DESIGN**

---

**Ultimate Design Load**

\[ (A + 0.5) \times 1.6 = 1.6 \]

\[ = 10.5 \text{ kN/m} \]

\[ = 19.4 \text{ kN/m}^2 \]

**Reaction Each End**

\[ = \frac{2.3 \times 16.0}{2} \]

\[ = 19.35 \text{ kN for each} \]

**Design Bending Stress**

\[ = 0.4 + 2.5 \]

\[ = 10.8 \text{ kN/m}^2 \]

**Effective Beam & Section**

\[ = 100 + 100/2 \]

\[ = 600 \text{ mm} \]

**Net Bearing Width**

\[ = 19.63 \times 1000/600 \times 10 \]

\[ = 32 \text{ mm} \]

**Allowance for Spalling**

\[ = 25 \text{ mm} \]

**Allowance for Inadequacies**

\[ = 20 \text{ mm} \]

**Total Min Bearing Width**

\[ = 97 \text{ mm} \]

**Nominal Bearing Width**

\[ = 3.3 \times 2 \times 9085 \]

\[ = 97 \text{ mm} \]

**Min Perfusion**

\[ = 8+40+125 = 135 \text{ mm} \]

**Assume concrete cover, Limit of Action of Tie**

**Load**

\[ = 135 - 15 = 120 \text{ mm} \]

**Distance**

\[ = 120 + 40 + 135 = 172 \text{ mm} \]
\[ M = 19.435 \times 0.172 = 3.3 \text{ kN/m concrete} \]

**Effective Depth**

\[ d = 120 - 60 \times 12/4 = 90 \text{ mm} \]

\[ M/ bd^2 = 3.3 \times 10^5 / 1000 \times 0.09 \]

\[ A_s = 3.3 \times 12^2 / 0.87 + 14.0 + 140 = 103 \text{ mm}^2 / \text{m} \]

**Spacing per centre line**

\[ 3 \times 91 + 2 \times \text{ Base dimension} \]

**Min**

\[ 0.172 \times 1000 + 140 = 182 \text{ mm}^2 \]

Use min = 110 @ 250%

\[ v = (19.435 \times 10^3) / 1000 \times 0.09 = 0.206 \text{ N/mm}^2 \]

\[ 100A_s / bd = 35A \times 100 / 1000 \times 0.09 = 0.376 \]

\[ v_o = 0.72 \times (35/100)^{1.5} \times (200/0.09)^{4} = 0.65 \text{ N/mm}^2 \]

**Allowable Stress Stress**

\[ = 0.65 \times 200 / 172 = 0.72 \text{ N/mm}^2 \]

\[ v_{o/2} = 0.36 \times 0.206 \]

So existence of fibre is allowable.
ANNEX No. 3

TRANSFORMER TOWER

STRUCTURAL CALCULATION
## Bending Schedule

### CHIMOIO WATER SUPPLY PROJECT

**TRANSFORMER TOWER**

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</table>

*All Bending Dimensions are in accordance with BS4466*
FAX: 00 263 4 750 780

TO: MIRKO RISTIC, c/o LANTOST JOHANNESBRY.
FROM: STEPHEN HUGAN, ADRA

1) TRANSFORMER TOWERS

PLEASE FIND ATTACHED DRAWINGS FOR THE TRANSFORMER TOWER.

WE ARE REQUIRED TO PROVIDE THE CIVIL WORKS; BUILDING, PLUS DOORS, VENTILATORS, LONDRAS, FRAMED SPACE FOR INSULATORS, AND REMOVABLE SAFETY GUARD

FOLLOW DRAWINGS:
A - LAYOUT SUGGESTION
B - FRONT ELEVATION
C - RIGHT SIDE ELEVATION
D - CROSS SECTION

MAIN DOORS - STEEL, OPEN OUT, DOUBLE
SAFETY GUARD - GALVANISED MESH ON STEEL FRAME, REMOVABLE ONLY FOR INSTALLATION/MAINTENANCE OF TRANSFORMER (2.0 X 1.8m)

RADAR TOWER MAY BE FIXED TO REAR RAF TOWER.

RADIO DO AMPLER: 20 OF FMAULIACATED AUSEL

TALY PHONE YOU ABOUT PUMP?

I WILL BE IN LONDON SUNDAY 7 AND
4) Do you have detail of River Crossings?

Hurry back

Steve

ROUCH LAYOUT, TRANSFORMER TOWER

Reinforced concrete column

Render plastering 2/5
Dimension of space available for insulation space to be edged with angle-iron or similar.

Floor slope to drain

Frontal Elevation

Space for insulation

Door

1250

6250

minimum
drum +200

3/5
ANNEX No. 4

CALCULATION FOR SIZE OF TRANSFORMER
Chimato Water Supply Project

Long Estimates

Purkett Tower Pump
- Load = 80kW
  At 0.8 pf = 115kVA.

Cyclic Pump
- Load = 20kW
  At 0.8 pf = 25kVA.

Oily Basket
- Load = 10kW
  At 0.8 pf = 12.5kVA.

Ancillary Services (Lighting + Power) - Load = 5kVA.

Total Connected Load = 151.5 kVA

Worst Case Loading:

Assume Purkett Tower Pump Starting
Starting load = \( \frac{5}{8} \times 115 \text{kVA} \)
= 37.5 kVA

Cyclic Pump = 25 kVA
Other losses = 17.5 kVA

Total Starting Load = 37.5 kVA

Transformer size = 315 kVA

20% overload for short period = 378 kVA

0.8 315 kVA transformer is adequate.
ANNEX No. 5

HEADER RESERVOIR

STRUCTURAL CALCULATION
# Bending Schedule

**CHIMOIO WATER PROJECT**  
**RESERVOIR**

<table>
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<th>Member/ Section</th>
<th>Bar Mark</th>
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</table>

All Bending Dimensions are in accordance with BS4466
Description

1. Circular Reservoir 7600 m³ design capacity
2. Max. Function of Reservoir (Nominal level) = 3300
3. Total depth of Reservoir & Bed (Top to Base) = 3300

Assumption: Tank will be constructed of reinforced RCC wall thickness

1. Reinforcement charted to determine wall thickness
2. Slab of vertical bending steel
   - ADR to be determined with full polar moment of inertia, and bending steel will be limited to 10%
   - In semicircular shape of cracking, using U-type or ASPEN, ES-9000
3. Base Reinforcement is designed in the service state at the corners of the wall
4. Chair Reinforcement will be limited to 0.7
5. Base Design

Section thru Circular Tank
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Sheet No. 1

Job: 10319

Section: Determination of Wall Thickness

Figure 1

Figure 2

Figure 2 shows area A against a free surface and a circular wall subjected to fracture. At the point P, the wall is thinned. The wall and pipe maintain the same

Integrate A (A) expression, finding value of the free surface. Maximize value of the free surface.

Jaw: Westbrook. Table A shows for every layer:

\( H/\Delta t \approx 33 \text{ in} \)

For \( H/\Delta t \approx \), mixed system: Assume two curves, and assume that \( a = 0.22 \)

\( E = \sigma \) in relation to the hoop stress in the design

\( \sigma = 15 \times 10^3 \)

Equlaity: \( E = a \cdot H \cdot L \cdot c = H \cdot L \)

Since \( H/\Delta t = 1 \), then \( c = H/\Delta t \)

\( c = 0.14 \times 0.00032 \times 1.3 \times 10^{-2} \)
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JOB: Reservoir (trench)
SECTION: 

Design:

\[
M = W \times L = 0.5 \times 10 \times 0.1 = 5 \text{ kN.m} 
\]

- Buckling:
  \[
  \frac{M}{Fy} = \frac{5}{30} = 0.1667 \text{ N/mm}^2 
  \]
  
  \[
  \frac{Fy}{E} = \frac{30}{210,000} = 0.000142857 \times 10^3 \text{ N/mm}^2 
  \]

  \[
  \frac{M}{Fy} > \frac{Fy}{E} \Rightarrow \text{No buckling} 
  \]

- Torsion:
  \[
  M_t = \tau \times I = 0.01 \times 0.1 \times 0.01 = 1.0 \times 10^{-5} \text{ kN.m} 
  \]

  \[
  \frac{M_t}{G} = \frac{1.0 \times 10^{-5}}{80} = 0.00000125 \times 10^{-6} \text{ N/mm}^2 
  \]

  \[
  \frac{M_t}{G} < \frac{G}{E} = \frac{80}{210,000} = 0.000381 \times 10^{-6} \times 10^6 
  \]

  \[
  \frac{M_t}{G} < \frac{G}{E} \Rightarrow \text{No torsion} 
  \]

- Shear:
  \[
  V = 0.1 \times 0.1 \times 0.1 = 0.001 \text{ kN} 
  \]

- Bending:
  \[
  M = 0.1 \times 0.1 \times 0.01 = 0.0001 \text{ kN.m} 
  \]

- Assume: \( t = 200 \text{ mm} \)

- Check:
  \[
  t = \frac{115}{200} = 0.575 \text{ mm} 
  \]

  \[
  \frac{t}{d} = \frac{0.575}{1.2} = 0.479 
  \]

  \[
  \frac{t}{d} < \frac{1}{6} \Rightarrow \text{Safe} 
  \]

- Check:
  \[
  \frac{t}{d} = \frac{0.575}{1.2} = 0.479 
  \]

  \[
  \frac{t}{d} < \frac{1}{6} \Rightarrow \text{Safe} 
  \]

- \( f_c = \)fy / (1.5 + 0.3 \times 0.1) \times 10^3 \text{ N/mm}^2 

- Check:
  \[
  \frac{f_c}{f_y} = \frac{30}{210,000} = 0.000142857 \times 10^{-3} 
  \]

  \[
  \frac{f_c}{f_y} < \frac{f_y}{E} = \frac{210,000}{210,000} = 1 \times 10^6 
  \]

  \[
  \frac{f_c}{f_y} < \frac{f_y}{E} \Rightarrow \text{Safe} 
  \]

- Check:
  \[
  \frac{f_c}{f_y} = \frac{30}{210,000} = 0.000142857 \times 10^{-3} 
  \]

  \[
  \frac{f_c}{f_y} < \frac{f_y}{E} = \frac{210,000}{210,000} = 1 \times 10^6 
  \]

  \[
  \frac{f_c}{f_y} < \frac{f_y}{E} \Rightarrow \text{Safe} 
  \]
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Sheet No. 4

Mainly

Checked By

Date

Initials

Nicholas O'Dwyer & Partners

JOB

SECTION

Linear static design for Vertical Steel

Assumption: Use the Vertical Steel(Fig. b) for the free end, (although the material is not as rigid)

Table A (Vertical Steel) for Design:

- Moment = Weight x 0.5 x H^2 x Elevation

- Coefficient for Soil = H/Be = 334.7

- Tension = 77 mm²

Both tension and bending moments will be ignored as minimal

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Bend Load (kN)</th>
<th>M (kN.m)</th>
<th>X (m)</th>
<th>Moment (kN.m)</th>
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<td>5</td>
<td>10</td>
<td>0</td>
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</tbody>
</table>

- No. of max./min. moments
- Critical moments.
\[ Y_1 = 1.4, \quad \theta_1 = 5 \quad \text{moment} = 5.13 \times 10^3 \quad \text{N.m} \]

\[
M = 7.12 \times 10^5 \quad \text{N.m} \]

\[
I = 9.5 \times 10^3 \quad E = 2.05 \times 10^7 \quad a = 1241.2 \]

\[ A_s = 7.12 \times 10^3 \quad B_s = 1241.2 \quad 2.23 \text{mm}^2 \]

\[
\text{mean} = 0.24 + \text{mean} \times 200 \quad (480 \text{ mm} / \text{m}) \]

Check: Assuming mean steel requirement

\[ \text{mean} = 75 \text{ mm}^2 / \text{m} \quad (\text{note } 75 \text{ mm}^2 \text{ ideal}) \]

Calculation:

\[
d = \frac{1}{2} \left( \frac{75}{200} \right) \quad \text{mm} \]

\[
A_s = \frac{25}{75} \times 100 \quad \text{mm}^2 \]

Assuming \( d = \frac{x}{2} \)

\[
I = A_s \times x = \frac{5}{3} \times 125 \quad \text{mm}^2 \]

Diagram with values and calculations.
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---

### JOB

**SECTION**

**Calculation We Blake Under**

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<th>( V )</th>
<th>( v )</th>
<th>( h )</th>
<th>( V / 2h )</th>
<th>( v / 2h )</th>
<th>( V^2 / 2h )</th>
<th>( v^2 / 2h )</th>
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<td>10</td>
<td>3</td>
<td>2</td>
<td>45</td>
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</tr>
</tbody>
</table>

\[ E_6 = \frac{V}{2h} = 0.00275 \times 0.001 \Rightarrow \text{OK} \]

\[ \sigma = \frac{E_6}{E} = \frac{5}{3} \]

\[ \epsilon = \frac{\sigma}{E} = 0.00275 \times 0.5 = 0.01375 \]

\[ \epsilon = 15.4 \times 0.00554 = 0.0854 \]

\[ \frac{\sigma}{\epsilon} = 0.0854 + 0.00554 (\text{due to a}) = 0.135 \]

\[ \sigma = 0.135 \times 130 = 17.5 \text{ mm} \]

\[ E' = \left( \frac{200 - 45}{180 - 45} \right)^2 \times 0.160273 = 0.000017 \]

\[ E' = 0.160273 - 1 \times \frac{200 - 45}{180 - 45} \times \frac{2}{3} \times 0.160273 \times 154 (130 - 45) \]

\[ = 0.00047 + 0.00047 \]

\[ = 0.00094 \]

\[ E' = 0.00094 \text{ mm} \]

\[ N = 3 \times 105 = 0.1051 \]

\[ 1 = 2 (160.0 - 4.3) = 0.12 \text{ mm expansion} \]
3) Check for critical hoop tension:

By inspection, the critical hoop force is:

\[ F = \frac{1}{E} \left( \frac{1}{r_1} - \frac{1}{r_2} \right) \]

\[ = \frac{1}{60000} \left( \frac{1}{4.5} - \frac{1}{3.8} \right) \]

\[ = \frac{1}{60000} \times 0.05 \]

\[ = 8.33 \text{ kN} \]

Assuming 15 ksi for steel, 300 ksi mm² (higher for HSS):

\[ A_t = \frac{F}{E} = 72.9 \times 10^3 \text{ mm}^2 \]

\[ E = 300 \text{ ksi} \]

\[ = 500 \text{ mm}^2 \times 754 \text{ mm} \text{ As prev.} \]

\[ F = 8.33 \text{ kN} \]

Hoop tension is not critical.

P.O.B. 600

C.V. 150

C. V.: A. B.
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**Table A7: Design of Wall**

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<tr>
<th>Design</th>
<th>Shear Force</th>
<th>Bending Moment</th>
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<td>A.</td>
<td>( V = 1.4 \times 0.174 + 0.2 )</td>
<td>( M = 1.4 \times 0.174 \times 0.5 + 0.2 \times 0.25 )</td>
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<tr>
<td>B.</td>
<td>( V = 1.4 \times 0.174 \times 0.5 + 0.2 \times 0.25 )</td>
<td>( M = 1.4 \times 0.174 \times 0.5 \times 0.25 + 0.2 \times 0.25 )</td>
</tr>
</tbody>
</table>

From the calculations above:
- **Design B** is preferred.

**Design Considerations**

- **Depth of Wall**: 1.2m
- **Design Depth**: 1.2m

**Notes**

- The design is based on the assumption of a triangular load.
- The design is verified using a numerical method.
- The design is subject to further review.

**Calculations**

- **Load**: \( V = 1.8 \times 2.1 \) kN

**Verifications**

- **Shear**: \( V = 3.6 \times 1.5 = 5.4 \) kN
- **Moment**: \( M = 3 \times 7 = 21 \) kN.m
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Sheet No: 9
Made By: 
Checked By: 
Date: 
Ref: 

SECTION Reserve Section

Job: 

Rivets

1. R12 @ 150 o.c. En. man see
2. R16 @ 200 En. Top & Bottom
3. Min. R12 @ 150 Base Distance
4. R16 @ 200 Dowels
INLET PIPE DETAIL

Scale 1:25

Flat operated Control Valve

ESERVOIR

INLET PIPE

Pipe detail

Thrust block

Reservoir

186,700

\[20\] AC Pipe
JOB

SECTION

DETAIL A

Scale 1:5

100 mm Ø Overflow/Scour pipe.

2 Layers Malthold
Note: Roof to be screeded to a maximum of 75mm and to a minimum of 25mm on perimeter.
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JOB

SECTION

100mm dia.
G.S. Pipe

Mosquito gauge fixed
with M.S. strap belted
around pipe.

VENTILATOR DETAIL
Scale 1:10

Puddle flange

OUTLET PIPE DETAIL
Scale 1:25

Strainer 200 dia.
C.T. 5/F

J.C. dia. A.C. Pipe
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PLAN OF LOCKING BAR

ELEVATION OF STAPLE

PAINTING

ALL STEEL AND CAST IRON WORK TO BE WIRE-BRUSHED AND PAINTED TWO COATS BLACK EPOXY PITCH AFTER INSTALLATION.
ALL STEEL WORK IS TO BE HIGH-TENSILE RR 1875.

LIGHT DUTY RECTANGULAR COVER CAST INTO ROOF SLAB
DETAIL C

OPENING FOR 16 MANHOLE COVER

NO SUBSEQUENT GROUTING PERMITTED

STAPLE 10A, 100 MTS PLATE BENT TO SHAPE LOCKING BAR 40, 18 MTS PLATE BENT TO SHAPE

12 MTS HOLE

LOCKING STAP 10A, 100 MTS PLATE BENT TO SHAPE

INSTRUCTION

LOCKING STAP

PLATE BENT TO SHAPE

B

PLATE BENT TO SHAPE

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