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

SHELADIA Associates Inc.
15825 Shady Grove Road, Suite 100
Rockville, Maryland 20850
USA

JULY 1994

TABLE OF CONTENTS

ABBREVIATIONS

The following general abbreviations are used in this Report:

F.D.R.	Final Design Report
USAID	United States Agency for International Development
AdC	Agua de Chimoio
DNA	National Directorate of Water Affairs
DINAGECA	National Directorate for Mapping
SHER	Chimoio Energy Company
DNEP	National Directorate for Roads and Bridges
DPCA	Provincial Directorate for Construction and Water
CPMZ	Companhia do Pipeline Mocambique-Zimbabwe
masl	Metres above sea level
m	Metres
m^3	Cubic metres
m^3/h	Cubic metres per hour
No.	Number
l/s	Litres per second
mm	Millimetres

CHAPTER 1

INTRODUCTION

Vi

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:

January	240 mm
February	199 mm
March	183 mm
April	53 mm
May	14 mm
June	18 mm
July	13 mm
August	15 mm
September	12 mm
October	36 mm
November	120 mm
December	218 mm

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 Constrains

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:

Description	Level (masl)
Dam crest	626
Maximum operational level	625
Flood protection level, January	622
Flood protection level, February	620
Designed minimum operational level	590
Actual minimum operational level	582
Bottom discharge	565
River bed	548

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:

Year	Level (masl)
1973/74	591
1974/75	613
1977/78	618
1984/85	615
1991/92	611
1992/93	609
1993/94	610

A study has been carried out of ˙ıe probability of the occurrence of low water levels in the Lake, this is included in the Environmental Study Report. A resonable 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 the 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:

Description	Soft	Hard	Rock
Transmission pipeline	89%	9%	2%
Intake site	41%	31%	28%

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

10A

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 m^3 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:

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

<u>Calculation of the pump</u>

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
	159	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:

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

The pump has been chosen KSB 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 galvanised 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 wit 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 5075	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:

gradient	spacing (m)
up to 1 in 2	6
from 1:2 to 1:4	12
from 1:4 to 1:5	18
from 1:5 to 1:6	24
flatter than 1:6	not required

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:

$$J = 10,708 \times \left(\frac{Q}{C}\right)^{1,852} x D^{-4,87}$$

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

For the Q = 2000 m^3 / day; D = 250 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:

$$Cp = Cw x \frac{1}{\sqrt{1+\left(\frac{Ew}{Ep} x \frac{dm}{t}\right)}}$$

Where Cp = Wave celerity (m/s)
Cw = Wave celerity in a column of water (1425 m/s)
Ew = Elasticity modus of water (2070 N/mm^2)
Ep = Elasticity modus of pipe material (N/mm^2) uPVC Ep = 3000 ; Steel Ep = 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 ($\approx 9{,}81\ m/s^2$)
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 l/s ;
Vo = 0,7 m/s ; Static head = 159 m; Friction losses = 6,61 m ;
Length L = 2050 m ⇒⇒

Cp = 1027,26 m/s
H = 73,30 m (mhw)
To = 4,00 s

Δh = 159 + 6,61 + 73,30
Δh = 238,91 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 than 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)

$$Re = \frac{Vxd}{\eta / \delta}$$

Where Re = Reynolds number
V = Velocity of flow in metres per second (m/s)
d = Inside pipe diameter (mm)
η / δ = Kinematics fluid viscosity (m^2 / 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 l/s are as follows:

pipe diameter (mm)	classof the pipe	friction losses (m/100m)
250	6	0,15
250	9	0,17
250	12	0,19
250	16	0,24
250	20	0,32
200	9	0,46
200	12	0,50

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.

pipe diameter (mm)	class of the pipe	length (m)	friction losses (m/100m)	total friction losses (m)
250	6	2678	0,15	4,05
250	9	5697	0,17	9,69
250	12	10500	0,19	19,95
250	16	9200	0,24	22,08
250	20	600	0,32	1,92
			total friction losses (m)	57,69
			decrease of characteristics with age + 5%	2.88
			TOTAL	60,57

Calculation of the required level of the header reservoir:

Level of the highest point on the pipeline	691,99 m
Total friction losses:	60,57 m
	$\sum$ 752,56 m

To secure water supply, in case of some minor lekages 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 moveback 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:

$$Cp = Cw x \frac{1}{\sqrt{1+\left(\frac{Ew}{Ep} x \frac{dm}{t}\right)}}$$

Where Cp = Wave celerity (m/s)
Cw = Wave celerity in a column of water (1425 m/s)
Ew = Elasticity modus of water (2070 N/mm^2)
Ep = Elasticity modus of pipe material (N/mm^2) uPVC Ep = 3000 ; Steel Ep = 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)
Vo = Flow velocity just before valve closure (m/s)
g = Acceleration of gravity (= 9,81 m/s^2)

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:

PVC 250 mm class 16 (assumption); Q = 27,78 l/s ; Vo = 0.6 m/s ;
Static head = 182,25 m; Friction losses = 32, 96 m ;
Length L = 17350m ⇒⇒

Cp = 433,52 m/s
H = 26,51 m (mhw)
To = 80,04 s

Δh = 182,25 - 32,96 + 26,51
Δh = 175,8 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:

uPVC pipes:	BS 3505
uPVC joints and fittings:	BS 4346
uPVC solvent cements:	BS 4346

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

- Heac r 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.

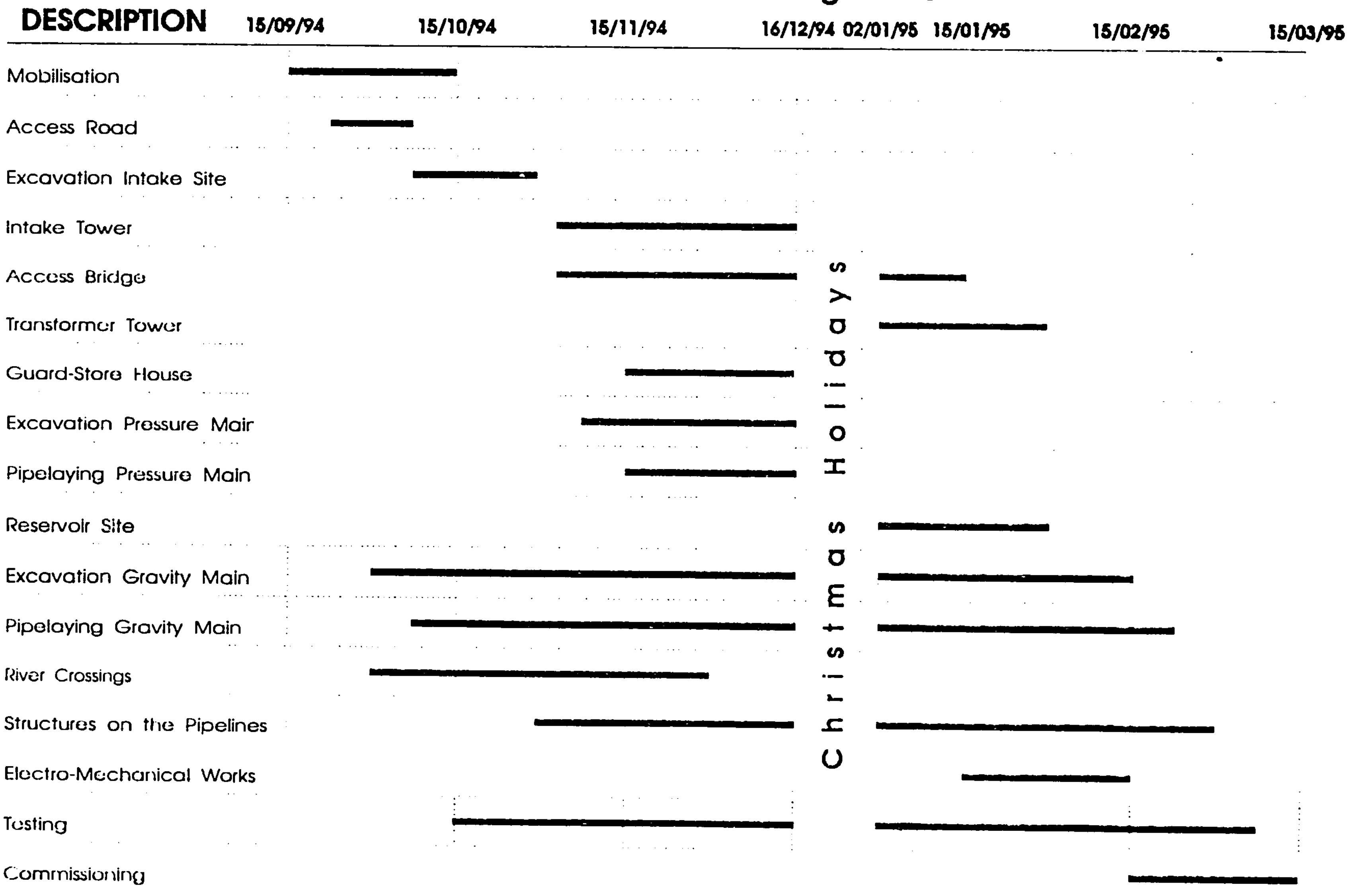
CHAPTER 6

CONSTRUCTION PROGRAMME AND CASH FLOW

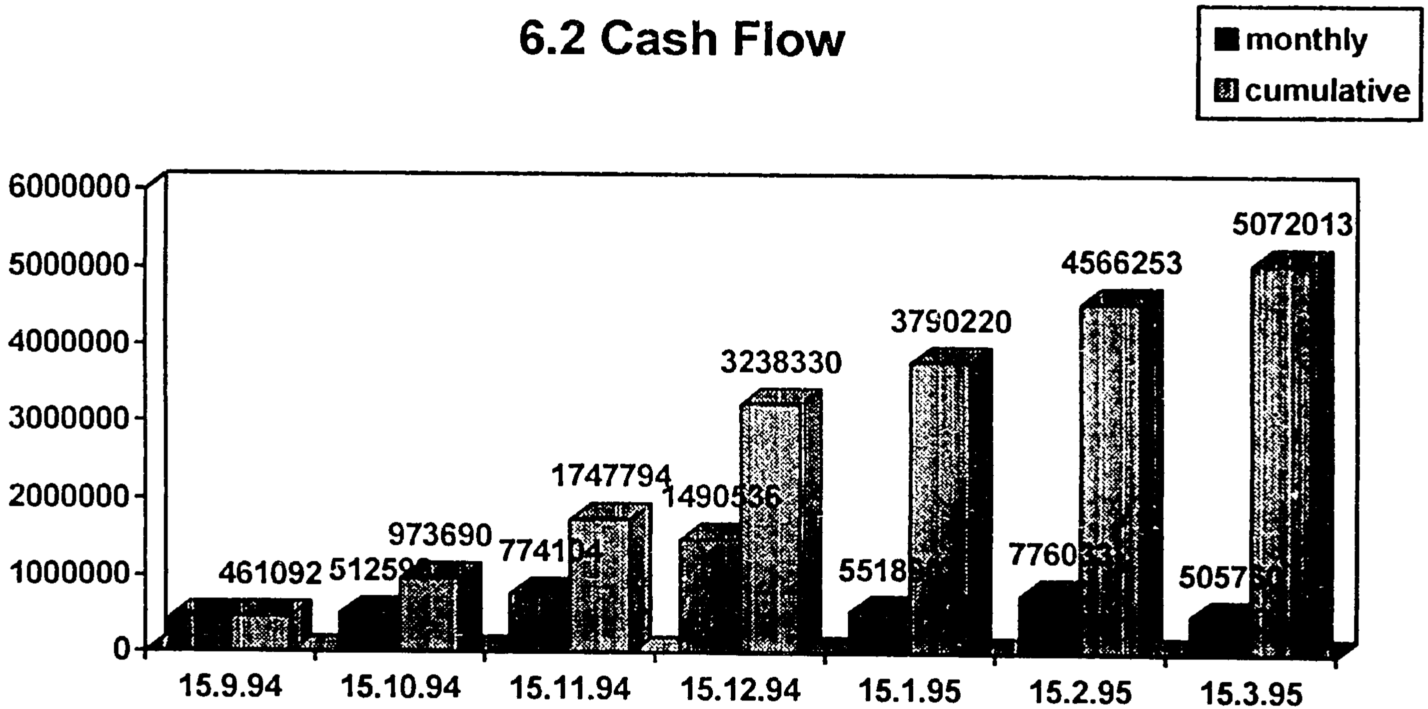
6. CONSTRUCTION PROGRAMME AND CASH FLOW

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



6.2 Cash Flow



32B

CHAPTER 7

CONFIDENTIAL COST ESTIMATE

7. CONFIDENTIAL COST ESTIMATE

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
1.	**PRELIMINARY & GENERAL**	L.S.	1	505,000	505,000
				total 1	**505,000**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
2.	**PIPELINE**				
2.1.	Excavation of trenches in any material including backfill and removal of surplus material.	m	42,269	9.76	412,545
2.2.	Supply steel pipes and fittings longitudinal welded, internally coated, externally protected, 6m length, working pressure 22 bars.	m	2153	70	150,710
2.3.	Lay pips and fittings including testing, flushing and disinfection	m	2153	15	32,295
2.4.	Supply uPVC pipes with rubber sealing rings, 6m length all classes	m	40,116	23.74	952,354
2.5.	Supply fittings for uPVC pipes with rubber rings	No	235	138.83	32,625
2.6.	Lay uPVC pipes and fittings including testing, flushing and disinfection	m	40,116	1.5	60,174
2.7.	Construct all the chambers on the pipeline including supply of all materials, valves and fittings	No	88	3,400	299,200
2.8.	Provide all material and construct anchor and trust blocks, marker posts, road and railway crossings	L.S.	1	41,950	41,950
2.9.	Provide and construct all river and stream crossings	L.S.	1	46,080	46,080
				total 2	**2,027,933**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
3.	**INTAKE TOWER AND BRIDGE**				
3.1.	Excavation in any material and placement of rock material	L.S.	1	96,935	96,935
3.2.	Grouting of all fractures	L.S.	1	20,000	20,000
3.3.	All concrete work for intake tower and bridge including supply of all material for concrete and reinforcement bars	L.S.	1	231,441	231,441
3.4	Supply and installation of all electromechanical work for intake tower and bridge including all the testing and commissioning	L.S.	1	367,550	367,550
3.5.	House on the intake tower	L.S.	1	12,000	12,000
				total 3	**727,926**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
4.	**TRANSFORMER TOWER**	L.S.	1	15,000	15,000
				total 4	**15,000**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
5.	**GUARD AND STORE HOUSE**				
5.1.	Construction of guard, store house	L.S.	1	75,000	75,000
5.2.	Construction of septic tank and soakaway including all drain pipes	L.S.	1	8,075	8,075
5.3.	Connection to existing water pipe	m	1,200	12	14,400
5.4.	Furniture	L.S.	1	20,000	20,000
				total 5	**117,475**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
6.	**INTAKE SITE AND ACCESS ROAD**				
6.1.	Excavation in any material and backfill around the structures for intake site	L.S.	1	114,018	114,018
6.2.	Construction of ramp, free drainage rockfill and fence	L.S.	1	7,756	7,756
6.3.	Construction of access road	L.S.	1	71,705	71,705
6.4.	Supply and installation of all electromechanical work at the intake site including all testing and commissioning	L.S.	1	84,200	84,200
				total 6	**277,679**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
7.	**STORAGE RESERVOIR**				
7.1.	Excavation in any material, backfilling and construction of dirt track	L.S.	1	5,752	5,752
7.2.	All concrete work for the reservoir including supply of the material for concrete and reinforcement bars	L.S.	1	12,812	12,812
7.3.	Supply and installation of all pipes and fittings, wire fence and construction of chambers	L.S.	1	8,290	8,290
7.4.	Supply and installation of float switches including 2050m of cable	L.S.	1	22,000	22,000
				total 7	**48,854**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
8.	**SUNDRY ITEMS**				
8.1.	Calculation, construction and as-built drawings, operational manuals and 30 days instruction period	L.S.	1	36,800	36,800
8.2.	Construction and removal of coffer dam	L.S.	1	23,000	23,000
8.3.	Notice boards	No.	2	1,000	2,000
8.4.	Spare pipes and fittings	L.S.	1	28,044	28,044
8.5.	Supply and install radio communication including all permissions	L.S.	1	35,000	35,000
				total 8	**124,844**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
9.	**PROVISIONAL SUMS**				
9.1.	Additional soil investigation	L.S.	1	9,280	9,280
9.2.	Connection and installation of power supply	L.S.	1	40,000	40,000
9.3.	Miscellaneous plumbing and electrical works, accounts certified by the Engineer	L.S.	1	31,930	31,930
				total 9	**81,210**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
10.	**DAYWORK AND DE-MINING**				
10.1.	Daywork	L.S.	1	10,000	10,000
10.2.	De-mining survey and de-mining	L.S.	1	50.000	50,000
				total 10	**60,000**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
11.	**MANAGEMENT, SUPERVISION AND FACILITIES FOR ENGINEER**	L.S.	1	555,000	555,000
				total 11	**555,000**

item	description	unit	provis. quant.	unit rate (US$)	total (US$)
12	**SUPPLY AND INSTALLATION OF TREATMENT PLANT 2000m^3/day**	L.S.	1	70,000	70,000
				total 12	**70,000**

7.1. Summary Cost Estimate-Phase II

ITEM	DESCRIPTION	TOTAL (US$)
1.	Preliminary and General	505,000
2.	Pipe line	2,027,933
3.	Intake tower and bridge	727,926
4.	Transformer tower	15,000
5.	Guard and store house	117,475
6.	Intake site and access road	277,679
7.	Storage reservoir	48,854
8.	Sundry items	124,844
9.	Provisional sums	81,210
10.	Daywork and De-mining	60,000
11.	Management, Supervision and facilities for Engineer	555,000
12.	Supply and installation of treatment plant	70,000
	SUB-TOTAL item 1 to item 12	**4,610,921**
	Contingencies (10% of sub-total)	**461,092**
	TOTAL	**5,072,013**

CHAPTER 8

LIAISON

37A

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 CONSTRAINS

9. PROJECT RISKS AND CONSTRAINS

9.1. General

As apart 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 tehnology.

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

Consulting Engineers and Project Managers
Telephone +263-4-750775/9 Telefax +263-4-750780

Sheet Nº:	4007/94/C/104
Made by:	AP
Checked by:	A.P.
Date:	JULY 1994
Drawing Reference:	4007/94/C/2.14

Bending Schedule

CHIMOIO WATER SUPPLY PROJECT
INTAKE TOWER

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
INTAKE TOWER	01	T12	600	600	1975	38	915	200			
	02	T12	336	336	2275	38	1015	300			
	03	T12	300	300	4400	20	STR				
	04	T12	300	300	3000	20	STR				
	05	T12	168	168	4400	20	STR				
	06	T12	168	168	3000	20	STR				
	07	T16	180	180	5150	37	4340				
	08	T16	90	90	2125	38	965				
	09	T16	90	90	5950	41	1040				
	10	T16	90	90	5925	20	STR				
	11	T16	180	180	8225	20	STR				
	12	T16	180	180	7325	20	STR				
	13	T16	16	16	8225	20	STR				
	14	T16	90	90	2200	39	1065				
Intake Tower Top Slab	15	T12	38	38	3000	20	STR				
	16	T12	63	63	1975	39	945	160			
1 # thus	17	T12	44	44	1925	39	925	150			
	18	T12	10	10	3100	39	1505	175			
	19	T12	10	10	1600	82	495	125			
	20	T12	18	18	4700	20	STR				
	21	T12	8	8	3425	20	STR				
	22	T12	6	6	1350	20	STR				
	23	T12	9	9	1975	20	STR				
	24	T12	15	15	2100	39	1005	165			
Intake Tower	17	T12	23	69	1925	39	925	150			
Intermediate Cantilever	18	T12	25	75	3100	39	1505	175			
Slabs	19	T12	10	30	1600	82	495	125			
	20	T12	15	45	4700	20	STR				
3 # thus	22	T12	25	75	1350	20	STR				
cont'd on next page	25	T12	16	48	1825	37	925				
	26	T16	8	24	2450	39	1175	205			

All Bending Dimensions are in accordance with BS4466

Consulting Engineers and Project Managers
Telephone +263-4-750775/9 Telefax +263-4-750780

Sheet Nº:	4007/94/C/105
Made by:	AP
Checked by:	A.P.
Date:	JULY 1994
Drawing Reference:	4007/94/C.2.14

Bending Schedule

CHIMOIO WATER SUPPLY PROJECT
INTAKE TOWER SLABS

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
Intake Tower	27	T16	8	24	4500	20	str				
Intermidiate Cantilever	28	T16	8	24	3400	20	str				
Slabs (Cont'd from	29	R8	116	348	1250	61	210	360			
previous page)											

All Bending Dimensions are in accordance with BS4466

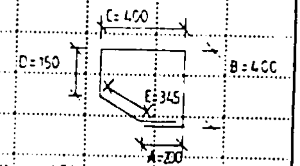
43

Consulting Engineers and Project Managers
Telephone +263-4-750775/9 Telefax +263-4-750780

Bending Schedule

Sheet Nº:	BS 106
Made by:	R.S.B.
Checked by:	A.P.
Date:	JUNE 1994
Drawing Reference:	4007/94/C.2.13

CHIMOIO WATER SUPPLY PROJECT
INTAKE HOUSE ABOVE INTAKE TOWER

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
BEAM 2	19	T16	3	6	3150	20	STR				
2# THUS	20	T12	2	4	3150	20	STR				
	21	T16	4	8	1950	39	865	430			
	22	R8	54	108	1400	61	195	445			
BEAM 3	21	T16	4	8	1950	39	865	430			
2# THUS	23	T16	2	4	4550	20	STR				
	24	T16	2	4	4550	20	STR				
	22	R8	32	64	1400	61	195	445			
	23	T16	2	4	4500	20	STR				
	24	T16	2	4	4500	20	STR				
	26	T16	2	4	1625	60	200	450			
	27	T20	3	6	1400	99C					
				A + B + C + D + E + 200 + 7.5d							
	28	T12	2	4	1550	60	200	450			
	29	T12	1	2	1200	60	200	275			
COLUMN	29	T16	4	16	4825	37	4700				
4# THUS	30	R8	32	128	775	60	150	150			

All Bending Dimensions are in accordance with BS4466

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

Sheet No: TS 1
Made By:
Checked By:
Date:
Ref:

JOB:
SECTION: Intake Tower Slabs

DESIGN OF FLOOR AT LEVEL +1085.20

Loading

BS 8110 Pt 1 1985	Dead loads - concrete	=	2400 kg/m³
CP 3 Ch V Dead & Imposed Loads	Dominating - assume		250 RC slab
	Dead load (G_k)	=	0.25×25
		=	$6.25\ kN/m^2$
	Imposed load		
Table 1	Industrial - Pumphouse		
	Use ref Table 10.		
	Imposed load	=	$7.5\ kN/m^2$
	Design loads DL	=	$1.0G_k + 1.0Q_k$ (service state)
		or	$1.4G_k + 1.6Q_k$ (ultimate limit state)
	$\therefore$ DSL $= 1.0 \times 6.25 + 7.5 \times 1.0$	=	$13.75\ kN/m^2$
	$\therefore$ DUL $= 1.4 \times 6.25 + 1.6 \times 7.5$	=	$20.75\ kN/m^2$
	$\frac{l_y}{l_x} = \frac{5.1}{3.7} = 1.37 < 2.0$		
	$\therefore$ Slab is 2 way spanning.		
BS 8110 3.5.3.4	Slab designed as restrained with corners prevented from lifting, provision for torsion made		
eq. 14 & 15	$M_{sx} = \beta_{sx} n l_x^2$, $M_{sy} = \beta_{sy} n l_x^2$		
	For edge discontinuous: $\beta_{sx} = 0.085$, $\beta_{sy} = 0.056$		
	$\therefore M_{sx} = 0.085 \times 20.75 \times 3.7^2$	=	24.15 kN.m
	$M_{sy} = 0.056 \times 20.75 \times 3.7^2$	=	15.9 kN.m

Notes.
$\therefore$ As for torsion = 3/4 As in midspan.

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

Sheet No TS 2.

Made By

Checked By

JOB

Date

SECTION

Ref.

As the Tie [illegible] [illegible] :

$$\frac{M}{bd^2 f_{cu}} = \frac{24.15 \times 10^6}{1000 \times 185^2 \times 25} = 0.028495$$

$$\therefore z = 0.95d = 0.95 \times 185 = 175.75\ mm$$

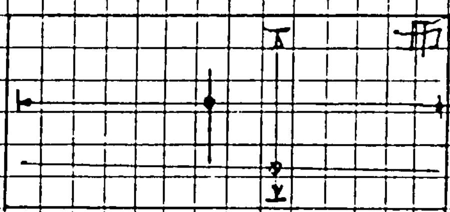
$$A_s = \frac{M}{0.87 f_y z} = \frac{24.15 \times 10^6}{0.87 \times 410 \times 175.75} = \underline{385\ mm^2}$$

Prov T12 @ 150 as main

Table 3.27 Min % of Ref.

Min Reinforcement Required

$$= \frac{0.13 \times 1000 \times 250}{100} = \underline{325\ mm^2}$$



T12 @ 175 in centres [illegible] [illegible] [illegible]

T12 @ 150 (same) > 325

~~T12 @ 150 325 mm²~~

T12 @ 75 (449 mm²) > 325 mm² min steel

LAMONT Consulting Engineers

1031/20	Page :
Chimoio Water Supply project	Job No.:
rc cant. slab	Made by: ap
30/07/94	Date :
rc design	checked:

beam/slab design acc. BS 8110

REINFORCEMENT DESIGN for a slab
for Cross Section: SUPPORT - cantilever

```
Width:   b = 1000 mm,      bw = 1000 mm
Depth:   h =  200 mm,      hf =    0 mm
         d =  144 mm,      d' =   60 mm
Cover:   top   40 mm,      bot.   50 mm,      sides    0 mm
Span:    l = 1.600 m,      Width of Support  300 mm
```

Materials:

Concrete fcu = 40 N/mm², Steel fy = 460 N/mm²

Moments and Forces:

```
Bending: M =   26.6 kNm
Shear:   V =   33.2 kN,     UDL =   0.00 kN/m
```

REINFORCEMENT:
Bending:
Bottom:

```
requ.: As =  485 mm² -->
   provide  8 T12 at 200 mm c/c  with As =  904 mm² (0.28%)
```

Shear:

```
Shear Stress: v = 0.23 N/mm²,          vc = 0.82 N/mm²
 No Shear reinforcement required !
Design at 294 mm from Cl for   0.0 kN, v = 0.00 N/mm²
                                              Asv =     0 mm²/m

Provide  0 legs R 0 at   0 mm --> Asv =    0 mm²/mCurtailment:
|<   0 mm >|<--- 0 Links R 0 --->|
                                |<----------    0 mm ---------->|
```

47

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

Sheet No: one
Made By: AP
Checked By:
Date: June
JOB: Chinoio Water Supply Project
SECTION: Intake Tower Structure - Design: 031.20

Synopsis:

1. The proposed intake tower is of RC concrete grade 25 concrete. Its located at the end of the site and is of gross dimensions 5100 x 3700 x 23m odd high. Apart from the bridge, it forms the largest structure on the project.

2. Earthquake design has been considered in the following manner:

- Dead & imposed loads increased by 20% to allow for vertical movement.
- Magnitude of horizontal thrust (not necessarily wind load) depending on magnitude of acceleration of the tremor varying from 1m/s² to 4m/s² (the latter happening in alluvial soil & the former in firm soil)
- Horizontal thrust of 1/10 of mass of building sufficient for all but major shocks when building does not exceed 6.0m height.
- Calculation of bending moments and shearing forces on the columns and floor beams similar to that for wind loading.
- All parts to act as unit and to be effectively bonded.
- Separate column footings should be connected by ties designed to take a thrust or pull of 1/10 of the load on the footing.
- Present day procedures assume the structure to possess sufficient strength and ductility to withstand tremors by behaving inelastically

ACI Code 318 - Seismic Design

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

Sheet No: Two
Made By:
Checked By:
Date:
Ref:

JOB: Chimoio Water Supply Project
SECTION: Intake Tower: Sketch

The following assessment constitutes calculation done prior to the survey on geological characteristics of site which has called for a reassessment of the design after the survey was in hand.

Proposed intake structure:

- Dimensions = 5500 × 3700
  Height = 23m above NGL & 10m below NGL.

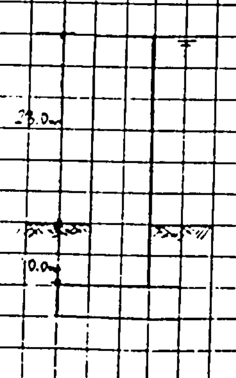
- Properties of materials:
  γ_s Density of soil = 2000kg/m³
  = 18kN/m³
- ϕ Angle of friction = 35°
- ϕ Angle of repose = 35° say
- γ_w Density of water = 10 kN/m³

- Loadings → Roof dead = 4.8 kN/m²
- Roof live = 4.0 kN/m²

Material properties

f_{cu} = 25N/mm²
f_{yv} = 250N/mm²
f_y = 460 N/mm²
γ_{conc} = 25kN/m³

Safe soil pressure
ABB. = 150N/mm²



Calculation of Wind Speed/L

Basic wind speed = 35m/sec. for Chimoio
Design speed = $V_s = S_1 \times S_2 \times S_3 \times V$
S1 for high degree of exposure as in the case of an intake tower, S1 = 1.1
S2 Height above ground level = 23.0m A.G.
Assume S2 = 1.1
S3 Take S_3 = 1.1

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

Sheet No (3)
Made By
Checked By
Date
Ref 1031 I.T.

JOB

SECTION Intake Tower Foundations:

So $q = 0.613 V_s^2 = 0.613 \times 46.585^2 / 10^3$

$= \underline{1.33 \text{ kN/m}^2}$

Assume $C_f = 0.5$ (CP3 CH.5 Table 14)

Wk: $W_k = (C_f \times q \times A_e)$

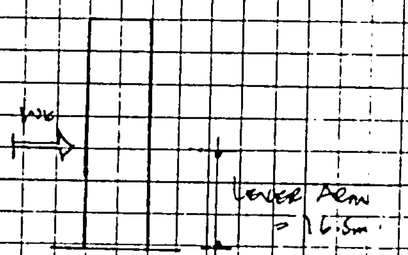
$= 0.5 \times 1.33 \times 33 \times 5.1$ (33m H. Assumed)

$= 58 \text{ kN} \times 5.1$

$= \boxed{298.0 \text{ kN}}$

Wk UDL = 298/33 = 9.03 kN/m

Service Wind Moment



M = 298 × 16.5

= $\underline{4917.0 \text{ kNm}}$.

Calculation of Dead Load of Intake Tower:

• Service Dead Load (Walls only) Ass. 300 TH. Walls

= 33 × 0.3 × (5.1 + 3.7) × 2 × 25

⇒ 174.24 × 25

Service Load N1 = 4356 kN

o Additional Floor Load + Pumps

= 118.0 kN × 2 = 236 kN.

Pumps = 3 × 300 × 10/1000

= $\underline{9.0 \text{ kN}}$

Total Service Load = 4601 kN

Nicholas O'Dwyer & Partners
Consulting Engineers

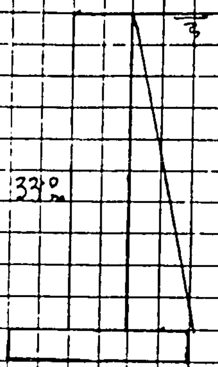
2 Robert Mugabe Road
P.O. Box UA 424, Union Avenue, Harare
Telephone 702531-5
Telefax: 702536
Telex: 22345 CODWYR ZW

JOB ....

SECTION ....

Sheet No ④
Made By ....
Checked By ....
Date ....
Ref ....

Calculation for Hydrostatic Pressure



Assuming hydrostatic pressure from one side as an active force:

$q_{ep} = \gamma h$

$= 9.81 \times 33.0$

$= 323.73 \text{ kN/m}^2$

Max. hydrostatic moment

$= \text{pressure} \times \frac{1}{2} \times h \times h/3$

$= 58756 \text{ kNm}$

Calculation of Soil Pressure:

Moments for the earth pressure

$p = \gamma . k . H$

$\gamma = 18 \text{ kN/m}^3$

$k = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$

H is the depth of earth

$\phi = 35^\circ$ $k = 0.27$ $H = 10 \text{ m}$

$\text{Pressure} = 18 \times 0.27 \times 10$

$= 48.6 \text{ kN/m}^2$

Max Mom. $= \text{Pressure} \times h^2/6$ kNm.

$= 48.6 \times 10^2/6$

$= 810 \text{ kNm}$

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

Sheet No ⑤
Made By
Checked By
Date
Ref

JOB
SECTION Stack Tower Design:

Moment at base W_k = 4917.0 kNm.

Moment, Hydrostatic = 58756 kNm
Moment, Soil pressure = 810 kNm

Total max. moment = 64483 kNm.

Assuming base size = 8000 × 6600 × 1000 Deep.

Area of base = $\frac{BD^2}{6}$ = 52.8 m²

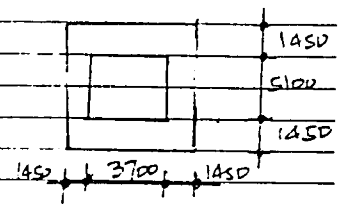
Modulus $Z = \frac{BD^2}{6}$ = 58.08 m³

- Moment from soil press = 810 kNm
- Moment from notional load

$= \frac{1.5 \times 5921}{100} \times 33/2$

$= 88.8\text{ kN} \times 33/2$

$= 1465\text{ kNm}.$



- Moment from wind analysis

= 4917.0 kNm.

Total moment M = 7192 kNm.

Ignoring hydrostatic pressure since its always balanced internally & externally

$\frac{M}{Z} + \frac{N}{A} = \frac{7192}{58.08} + \frac{4601+1320}{58.08}$

$= 123.836 + 101.945$

$= 225.8\text{ kN/m}^2 \ll 1000\text{ kN/m}^2$ or more as see Geological Survey Report

But since the base is founded through colluvium to hard rock strata, the intake structure walls are founded in deep excavated rock.

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

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

JOB ....

SECTION Intake Tower

The previous calculation pages (4)-(5) were to serve as a stability and bearing pressure check.

Design of R.C. walls for water tower:

Considerations:

(1) Handbook on the unified code suggest that wall at right angles which are provided to assist the unbraced walls should carry at least 25% of the lateral load.

(2) For the resistance of wind loads, the loads are divided to the moment of inertia of the wall.

(3) Deflection is assumed as same amount as deflection $\delta = pH^3 / 8EI$

(4) $P_3 = pl^3 / l_3^3 + l_4^3 \rightarrow$ load

$P_1 = P_i H^3 \times 6 / 2R l_3^3 = P_3 H^3 / 4 l_3^3$

Design:

i) Effective heights and thickness

U3

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 ....

Sheet No NOTES 1

Made By ....

Checked By ....

Date ....

Ref ....

BS8110 PART 2, 1985 Section 2.

Cl. 2.6.3.2.2 Lateral support

(a) a stiffened section of the wall (not exceeding 1,0m in length) capable of resisting a horizontal force in kN/m height of wall) of 1,5Ft, or

(b) where Ft is the lesser of (20 + 4No) or 60 where No is the no of storeys in the structure.

Cl. 3.2.1.1 Defl limit = l/250

Cl. 3.2.4.1 Appearance → For visible members, cracking should be kept within reasonable bounds by attention to detail. Max. crack width not exceed 0,3mm.

Cl. 3.2.4.2. Corrosion. For members in aggressive environments, calculated max crack width not to exceed 0,3mm.

BS8110 PART 2: 1985 PART 5

Table 5.2. Cover.

BS8110 PART 2 1985 - Movement joints

(a) Av. coefficient of thermal expansion of normal wt concrete = $10 \times 10^{-6}/°C$; thus the diff. in length of a concrete member 30m long due to a 33°C change in temp. would be approximately 10mm. If this is restrained this could cause a stress in the order of $7 N/mm^2$ in an unreinforced concrete member made ...

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 ....

Sheet No NOTES 2

Made By ....

Checked By ....

Date ....

Ref ....

8.2

(b) Drying shrinkage strains are in the order of 500×10^{-6} or 1,5 mm per 3m length in any unreinforced section.

(c) Creep - nil

8.3 - Contraction joint - - A deliberate discontinuity or interruption in both the concrete & reinforcement.

8.5, Paint joint with material to provide resistance corrosion, weatherproof or building mastic satisfying BS 3712, BS 6093 & BS 6213.

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 ....

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

BS8110: PART 1: 1985

SECTION THREE

TABLE 3.4 NOMINAL COVER TO ALL REINFORCEMENT (INCLUDING LINKS) TO MEET DURABILITY REQUIREMENTS.

CONDITIONS OF EXPOSURE (SEE 3.3.4) — NOMINAL COVER.

SEVERE — Concrete surfaces exposed to severe rain, alternate wetting & drying or severe condensation. — 40 mm

MAX. FREE WATER/CEMENT RATIO — 0,55

MIN. CEMENT CONTENT (kg/m³) — 325

LOWEST GRADE OF CONCRETE — C40

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

Sheet No ....
Made By ....
Checked By ....
Date ....

JOB ....
SECTION INTAKE TOWER DESIGNS

DESIGN OF RC WATER TOWER.

CONSIDERATIONS:

1, HANDBOOK ON THE UNIFIED CODE SUGGEST THAT WALL AT RIGHT ANGLES WHICH ARE PROVIDED TO ASSIST THE UNBRACED WALLS SHOULD CARRY AT LEAST 25% OF THE LATERAL LOAD.

2, FOR THE RESISTANCE OF WIND LOADS, THE LOADS ARE DIVIDED IN PROPORTION TO THE MOMENTS OF INERTIA OF THE WALLS.

3, DEFLECTION IS ASSUMED AS SAME AMOUNT AS $\delta = pH^3 / 8EI$

4, $p_1 = \dfrac{p l_1^3}{l_1^3 + l_2^3}$ → LOAD

$f_1 = \dfrac{p_1 H^2 \times 6}{2 h l_1^2} = \dfrac{3 p_1 H^2}{h l_1^2}$

DESIGN:

i, EFFECTIVE HEIGHTS AND THICKNESSES

THE ROOF AND FLOOR SLABS REST ON WALL AND TIES WILL BE PROVIDED FOR STABILITY. 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 = 7.4m; THICKNESS = 300mm
INTER/LOWER WALL EFFECTIVE HEIGHT 7.4m; THICKNESS = 300mm
LOWER WALL EFFECTIVE HEIGHT 8.4m; THICKNESS = 300mm

$\left.\begin{array}{l} \dfrac{7400}{30} = 246\text{mm} \\ \dfrac{8400}{30} = 280\text{mm} \end{array}\right\}$ ∴ SLENDER WALLS WITHIN THE PRESCRIBED LIMITS.

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 ....

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref. ....

iii, LOADING:

ROOF: DEAD LOAD = 0.95 kN/m² = 2.5 kN/m
UPPER WALL (300 th. × 7.4 m high) = 55.5 kN/m
FLOOR DEAD = 7.65[redacted] kN/m
INTERMEDIATE WALL = 7.65 kN/m
LOWER WALL = 8.69 kN/m
TOTAL DEAD LOAD AT TOP OF FDN = 200.48 kN/m.
ROOF IMPOSED = 0.75 kN/m² = 0.95 kN/m
FLOOR IMPOSED = 7.0 kN/m² = 8.95 kN/m.
TOTAL IMPOSED LOAD = 27.725 kN/m.

WIND LOADS: SEE PREVIOUS PAGE FOR FULL ANALYSIS.

FOR TRANSVERSE WINDS

$l/w = 5.1/3.7 = 1.378$

$h/b = 23.2/5.1 = 4.55$

FORCE COEFFICIENT $C_f = 0.75$

WIND LOAD = 0.75 × 1.33 × 3.7

= 3.69 kN/m.

MOMENT AT TOP OF LOWER WALL

$= 3.69 \times (7.4 \times 2)^2/2$

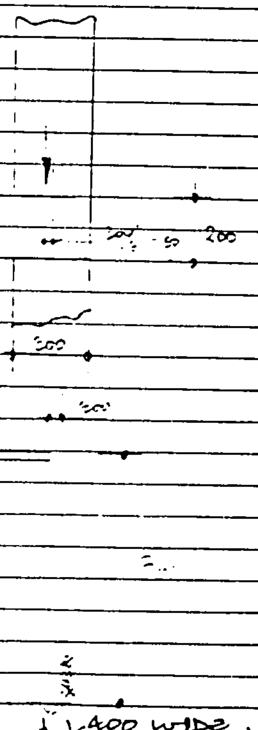
= 404.2 kNm

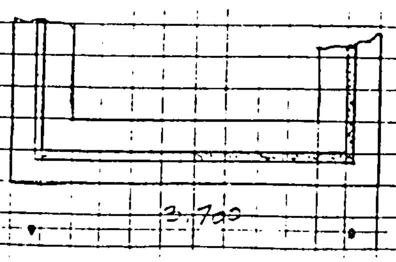
MOMENT AT TOP OF FOUNDATION

$= 3.69 \times 23.2^2/2$

= 993.05 kNm

WIND SHEAR AT BASE = 3.69 × 23
= 84.87 kN.





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 .... Sheet No ....
SECTION .... Made By ....
Checked By ....
Date ....
Ref. ....

CHECKING FOR TENSION AT BASE OF THE WALL:

ASSUME WALL = 3.7m LONG × say, unit THICKNESS.

MODULUS Z = $\frac{5.1^2}{6}$ = 4.335 m³

MAXIMUM LOAD DUE TO WIND
= $\frac{993.05}{4.335}$ = 229.07 kN/m.

THE DEAD LOAD AT TOP OF FOUND = 200.48 < 229.07

∴ THERE IS TENSION IN THE WALL.

SHEAR → 1/4 × 200.48 = 50.12 < 84.37.

INCREASE BOTTOMMOST SECTION OF TOWER TO 600mm THICKNESS SO AS TO AVOID TENSION IN THE WALL.

i.e. UPPER ROOF LOAD = 2.5
UPPER WALL = 27.6 (BRICKWORK)
UPPER WALL = 55.5
INTER WALL = 55.5
LOWER WALL = 84.0
UPPER FLOOR = 7.65
INTER FLOOR = 7.65
LOWER FLOOR = 8.68
FOUNDATIONS = 8.68

TOTAL DEAD LOAD = 257.76 kN/m > 229.07 kN/m FOR WIND.

SHEAR → 1/4 × 257.7 = 64.25 kN.
COMPARED WITH WIND SHEAR = 84.87 kN.

BUT 1.3 kN/m² FOR WIND CONSIDERED QUITE HIGH AND THE SHEAR DUE TO WIND COULD BE CONSIDERABLY LESS.

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 ....

Sheet No ....

Made By ....

Checked By ....

Date ....

Ref. ....

ii, WALL DESIGN:

THE LOWER WALL IS DESIGNED FOR LOADS AT MIDHEIGHT.

THE MOST ADVERSE COMBINATION IS FOR DEAD & IMPOSED LOADS:

THE DESIGN LOADS ARE:

~~Roof & upper wall + internal wall = (2.5 + 27.6 + 55.5)1.4 + (0.95 + 3.95)×1.6~~

~~= 119.84 + 15.84~~

~~= 135.68 kN/m load~~

= (1.4 × 141.1) + (1.6 × 9.00)

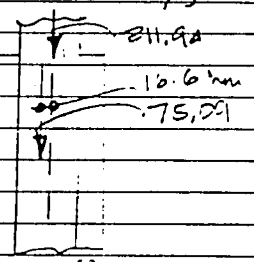
= 197.54 + 14.4

= 211.94 kN/m acting centrally.

Slab load = 1.4 × (7.65 × 3) + (8.95 × 3) 1.6

= (32.13 + 42.96) kN/m

= 75.09 kN/m acting at 1/3 wall thickness



$e_x = \dfrac{75.09 \times 67 - 211.94 \times 50}{287.03}$

= 19,4mm

This should not be less than 0,05 × 300

= 15mm.

Clear H/L ratio = $\dfrac{8.4}{5.1}$ = 1.64.

Wall coeff. K_w = 0,4

Ultimate load n_w = (300 − 2 × 19.4) 0,4 × 25

= 2612 kN/m > 287.03.

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 ......

Sheet No ......
Made By ......
Checked By ......
Date ......
Ref. ......

The additional eccentricity due to deflection

$e_a = h_e^2 / 2500h = (\ldots 84)^2 / 2500 \times 300$

$= 70.56\,mm$

For a slender wall, the ultimate load $N_u = (300 - 1.2 \times 19.4 - 2 \times 70.56)\,0.4 \times 25$

$= 2356\ kN/m$

The ultimate applied load is $= 287.03\ kN/m$ at mid height of wall.
Hence the design is satisfactory.

Wind shear at base $= 84.87\ kN$

Dead load of structure $= 257.76$

$1/4 \times 257.76 = 64.44$.

Although this is less than 84.87, the wind loading has been assumed as very conservative assuming $0.7\ kN/m^2$ say compared to $1.3\ kN/m^2$. A reduction in the shear wind force can be achieved. No further treatment will be entered into.

iv. Crack control:

Reinforcement for shrinkage & temperature is provided in accordance with clause 5.5.9.2 CP110 for high yield steel.

Area required $= 0.25 \times 400 \times 1000 / 100 = 1000\ mm^2$.
Put half this amount on each face.

[illegible]

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 ODWYR ZW

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

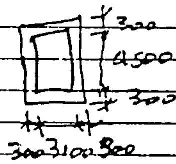
JOB ....

SECTION INTACO Tower:
Limit state check on stability of Tower at foundation level.

From previous calculation:

$W_k = 298{,}0$ kN

Section properties of annular ring.



$I_{max} = bd^3/12$

$= 5{,}10 \times 3{,}7^3/12 - 4{,}5 \times 3{,}1^3/12$

$= 21{,}52 - 11{,}17$

$= 10{,}35\ m^4$

$A = 5{,}1 \times 3{,}7 - 4{,}5 \times 3{,}1$

$= 18{,}87 - 13{,}95$

$= 4{,}92\ m^2$

$Z = I/y = 10{,}35/1{,}85$

$= 5{,}59\ m^3$

Min stress $= \dfrac{0{,}9\ G_k}{A} - \dfrac{1{,}4\ W_k y}{Z}$

$G_k = 257{,}76 \times (5{,}1 + 3{,}7) \times 2$

$= 2268{,}29$

Min stress $= \dfrac{0{,}9 \times 2268{,}29}{4{,}92} - \dfrac{1{,}4 \times 298 \times 23/2}{5{,}59}$

$= 414{,}93 - 820{,}9$

$= -406$ kN/m² (tension)

ie $0{,}4$ N/mm² (tension) ∴ Tension develops

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

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

JOB ....

SECTION WATER TOWER; CHECK ON WALL THICKNESS TO BS8110 Cl. 3.9.3.6.1

CHECK ON A THICKNESS OF 300mm.

TOTAL DEAD LOAD = 257.76 kN/m
TOTAL IMPOSED LD = 27.725 kN/m

TOTAL DESIGN LOAD = 1.4 × 257.76 + 1.6 × 27.725

= 360.86 + 44.36

N = 405.22 kN/m

VERTICAL LOAD ON CONCRETE WALL ⊁ 0.35 fcu. Ac. IF SLIP FORMING IS USED fcu IS REDUCED BY 5%

∴ N ⊁ 0.33 fcu Ac.

∴ Capacity of wall per m
= 0.33 × 25 × 300 × 10³/10³
= 2475 kN/m

25 = fcu to be changed to 30 fcu.

CAPACITY > N ∴ WALL THICKNESS SATISFACTORY.

ANNEX No. 2

ACCESS BRIDGE

STRUCTURAL CALCULATION

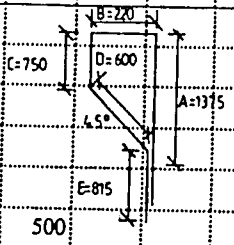
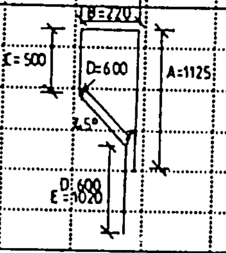
Consulting Engineers and Project Managers
Telephone +263-4-750775/9 Telefax +263-4-750780

Bending Schedule

Sheet Nº:	BS 103
Made by:	R.S.B.
Checked by:	A.P.
Date:	JUNE
Drawing Reference:	4007/94/C/2.13

CHIMOIO WATER SUPPLY PROJECT

BRIDGE & INTAKE HOUSE R.C. DETAILS.

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
BRIDGE	06	T16	4	12	11000	20	STR				
3# SECTIONS	07	T12	4	12	11000	20	STR				
(IN 11# PANELS OF	08	R10	138	414	2000	60	670	220			
SIZE 1017mm)	09	R12	238	714	2000	60	670	200			
22# IN EACH PANEL	10	R12	122	366	1050	81	335	35			
	11	R12	242	726	1125	83	900	150	615		
	12	R12	238	714	1550	39	750	90			
	13	T25	10	30	11000	20	STR				
	15	T12	162	486	1975	20	STR				
	16	T16	32	972	2200	20	STR				
	17	T12	30	90	11000	20	STR				
	18	T12	22	66	2050	39	1015	75			
CORBEL STUB COLUMN	01	T25	3	6	3375	99A					
2# THUS				A + B	+ C + D + E - 5d						
	02	T25	2	4	1900	60	200	500			
	03	T12	4	8	1650	60	200	500			
MAIN CORBEL	04	T20	21	21	3375	99B					
			A + B + C	+ D + E - 5d							
	05	T20	7	7	3650	33					
3600 x 400 BEAM WITH	14	T20	8	8	1975	38	840	400			
300 X 350 X 940 STUB	30	T20	6	6	5075	38	840	3500			
COLUMN AT MOVEMENT	31	R10	20	20	2500	60	300	840			
JOINT.	32	R10	6	6	2400	60	840	250			
2# THUS											

All Bending Dimensions are in accordance with BS4466

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

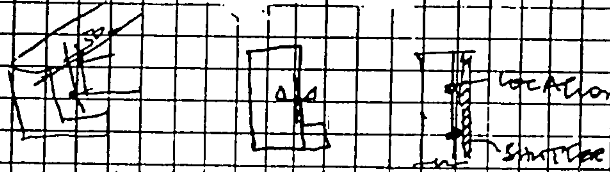
Sheet No BD 1
Made By
Checked By
Date
Ref

JOB CHIMOIO WATER SUPPLY PROJECT
SECTION DESIGN OF BRIDGE DECK

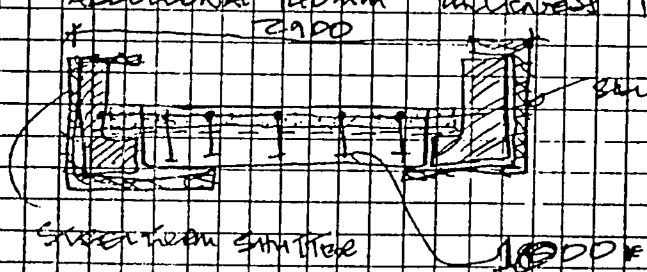
BRIDGE DECK DESIGN.

Synopsis:

BRIDGE DECK IS 2.3 m SPAN ON 300 × 750 PRECAST CONCRETE BEAMS.



ULTIMATELY A SLAB OF 110mm THICKNESS WITH AN ADDITIONAL 140mm THICKNESS IS TO BE USED.



Re 9.
gnolds:

DETERMINATION OF WATER LOADS FOR FOOTBRIDGES & CYCLE TRACK

EXCEEDING 30m OF LOADED LENGTH

LOAD $(kN/m^2) = 25\left(\frac{1}{L}\right)^{0.475} =$ ~~[illegible]~~ $4.557\ kN/m^2$.

FACTOR OF SAFETY = 1.5 FOR Ult. Lim. state.

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

Sheet No BD2
Made By
Checked By
Date
Ref

JOB

SECTION

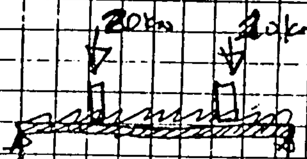
[illegible] Table 11,
[illegible]

FOOTPATH → CONCENTRATED LOAD OF 40 KN (INCLUDING IMPACT IN ANY POSITIONS)

HOWEVER KNOWN LOAD = 2 TONNE OR 20 KN.

INCLUDING SELFWEIGHT OF TROLLEY = 20 KN SEEMS REASONABLE.

LOADING ONTO SLAB:



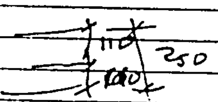
$Q_k = 6.25\ kN/m^2$
$G_k = 5.0\ kN/m^2$.

$f_{cu} = 30\ N/mm^2$ $f_y = 250\ N/mm^2$

BASIC SPAN/EFFECTIVE DEPTH RATIO = 20

MIN EFFECTIVE DEPTH $d = \frac{SPAN}{20 \times MOD.\ FAC.}$

$= \frac{2300}{20 \times MF}$



TRY EFFECTIVE DEPTH 80mm

FOR MILD EXPOSURE THE COVER IS 25mm
ALLOWING SAY 10mm AS HALF OF REINFORCING BAR,

OVERALL DEPTH $h = 80 + 25 + 5 = 110mm$
SELF WEIGHT OF SLAB $= 250 \times 24 \times 10^{-3} = 6.0\ kN/m^2$
TOTAL DEAD LOAD $= 6.0\ kN/m^2$

FOR 1m WIDTH OF SLAB.

Ultimate load $= (1.4 g_k + 1.6 q_k)\ 2.3$.
Ultimate load $= [1.4\ (6.0) + (1.6 \times 8 + 1.5 \times 6.25)]\ 2.3$
$= \underline{59.2825\ kN/m^2}$

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 ....

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref. ....

$M = 59.2825 \times 2.3/8 + 1/4 \times (80 \times 1.6) \times 2.3$

$= 17.0 + 73.6/2$

$= \cancel{90.6}$

$= 53.8$ kNm.

Span - Effective Depth Ratio.

$M/bd^2 = \frac{\cancel{90.6}\ 53.8 \times 10^6}{1000 \times 80^2} = \cancel{14.1625}\ 8.1625$ N.A.

Try Depth of 150

Effect. D = $150 - 25 - 8 = 117$ mm

$M/bd^2 = \frac{\cancel{90.6}\ 53.8 \times 10^6}{1000 \times 117^2} = \cancel{6.62}\ 3.93.$

Table 3.11
BS8110

Modif. Factor $= 0.55 + \frac{(477 - fs)}{120(0.9 + M/bd^2)}$

Assume MF = 1.02 for fs = 200

Therefore Limiting Span/Effect. D $= 20 \times 1.02$
$= 20.4$

Actual $= \frac{2300}{117} = 19.65$.

Thus D = 117 is adequate.

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 ....

Sheet No ....

Made By ....

Checked By ....

Date ....

Ref ....

BENDING REINFORCEMENT:

$$M/bd^2 f_{cu} = \frac{53.8 \times 10^6}{1000 \times 117^2 \times 30}$$

$$= 0.13$$

Lever Arm $z = l_a d = 0.95 \times 117$

$= 111mm$

$$A_s = \frac{M}{0.87 f_y z} = \frac{53.8 \times 10^6}{0.87 \times 250 \times 111}$$

$= 2228 mm^2$.

Provide R13 @ 75mm c/c. Bottom. As prov = 2681.

Minimum from Beam RCDD = ½ ie. R16 @ 150 or $\boxed{R12 @ 100}$

ADOPT.

~~Shear~~.

At Face of Support $V = \frac{59.825}{2} + 20 \times 1.6$

$= 57.9 kN$.

Shear Stress $v = \frac{57.9 kN \times 10^3}{1000 \times 117} = 0.49. < 0.8\sqrt{30}$ (4.38)

$100 A_s/bd = 100 \times 2681 / 1000 \times 117$

$= 2.29$

$v_c = 1.23$ say.

$v < v_c$ ∴ no further checks.

[illegible]

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

JOB ....

SECTION ....

Sheet No ....

Made By ....

Checked By ....

Date ....

Ref ....

END ANCHORAGE

$v = 0.49 < V_c/2 \quad (1.73/2)$

Therefore anchorage length ≥ 30mm or end bearing/3

end bearing = 145mm

Therefore

anchorage length 145/3 = 48.33 beyond C/L of support

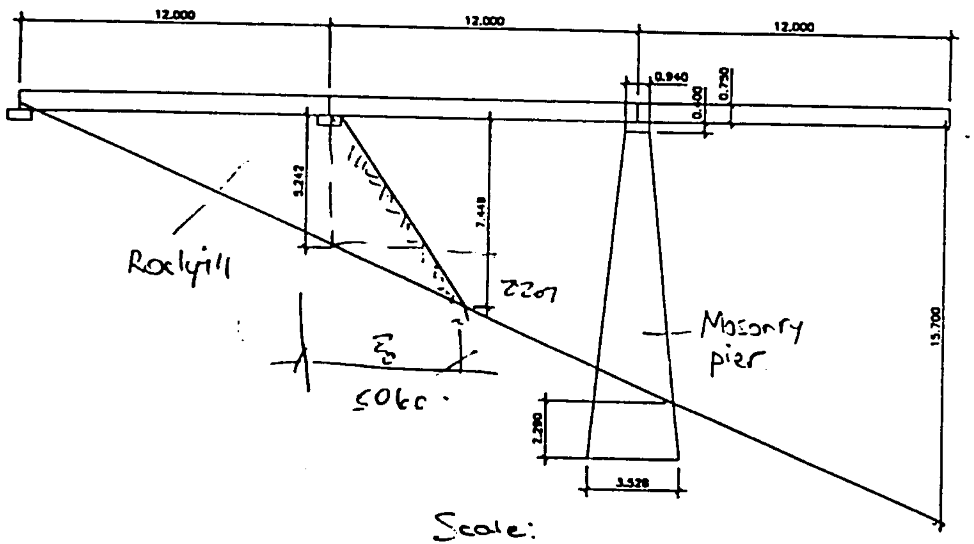
Distribution steel → 0.25*350*1000/100

= 625mm²

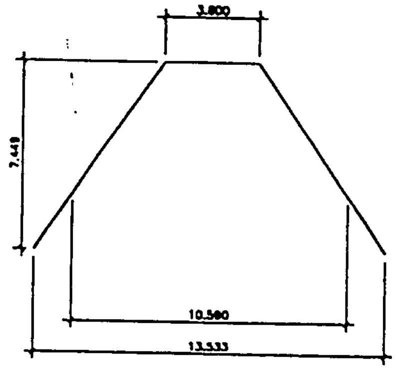
use Y12 @ 150 As prov = 754mm².

Summary:

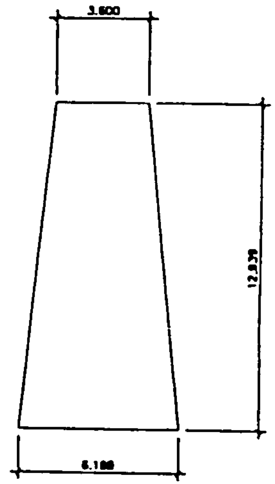
BRIDGE



Scale:



ROCK FILL CROSS-SECTION SCALE 1:250.



PIER CROSS SECTION

SCALE 1:250

beam/slab design acc. BS 8110

REINFORCEMENT DESIGN for a rectangular beam
for Cross Section: SPAN - simply supported

Width:	b = 300 mm,	bw = 300 mm		
Depth:	h = 750 mm,	hf = 0 mm		
	d = 698 mm,	d' = 60 mm		
Cover:	top 40 mm,	bot. 40 mm,	sides	40 mm
Span:	l =12.000 m,	Width of Support	300 mm	

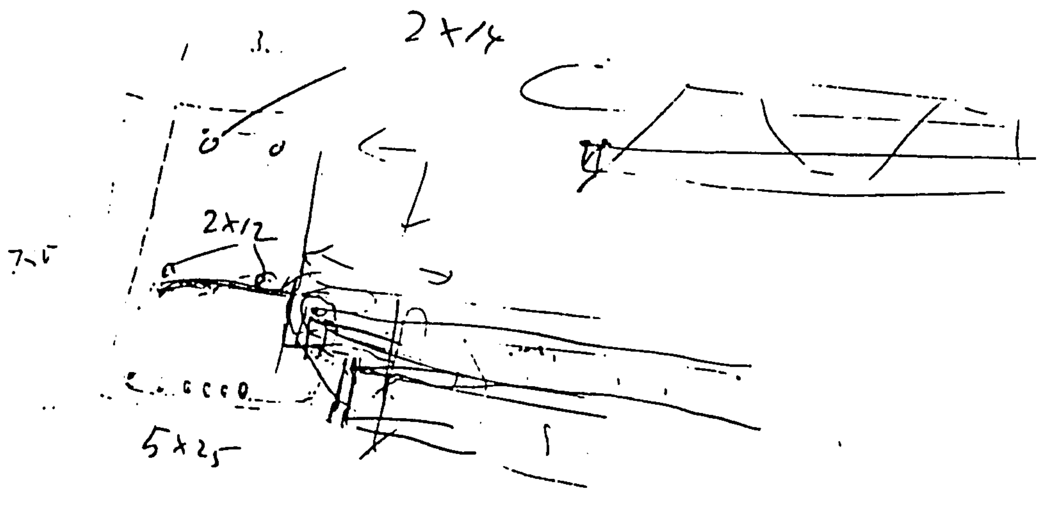
Materials:
Concrete fcu = 25 N/mm², Steel fy = 410 N/mm²

Moments and Forces:
Bending: M = 504.0 kNm
Shear: V = 168.0 kN, UDL = 28.00 kN/m

REINFORCEMENT:
Bending:
Bottom: requ.: As = 2529 mm² -->

provide 5 X25 with As = 3125 mm² (1.39%)

Shear:
Shear Stress: v = 0.81 N/mm², vc = 0.72 N/mm²
Design at %6000 mm from Cl for 144.4 kN, v = 0.70 N/mm²
Minimum Links: Asv = 552 mm²/m
Provide 2 legs R10 at 275 mm --> Asv = 571 mm²/m



Nicholas O'Dwyer & Partners
Consulting Engineers

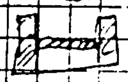
2 Robert Mugabe Road
P.O. Box UA 424, Union Avenue, Harare
Telephone 702531-5
Telefax: 702536
Telex: 22345 CODWYR ZW

JOB: CORBEL
SECTION: BRIDGE DESIGN.

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

CORBEL FOR BRIDGE:

Assuming width = 320mm
Vertical load =



$= 39.2 \times \frac{12 + 2.3}{2} + 1.5 \times 0.3 \times 24 \times 12/2$

$= 816.96 + 64.8$

$= 881.76\,kN.$

$f_{cu} = 40\,N/mm^2$, $f_y = 460\,N/mm^2$

Max. Bearing stress = $0.8 \times 40 = 32\,N/mm^2$

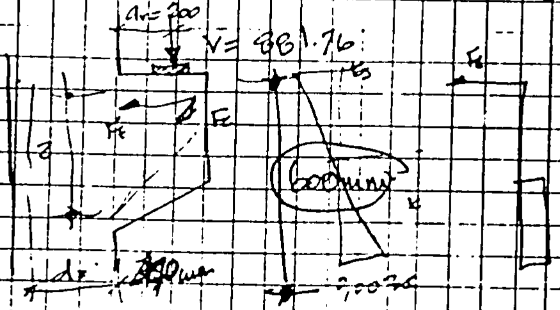
Assuming effective length of bearing plate is 250mm

Min. width = $(881.76 \times 10^3)/(32 \times 250)$

$= 110.22\,mm$, say 115mm

Shear stress at beam face:

$v = V/bd \ngtr 0.8\sqrt{f_{cu}}$
for grade 40 concrete max = $5.05\,N/mm^2$.



CORBEL = 600mm

$V = (881.76 \times 10^3) / 320 \times 550 = 5.01\,N/mm^2$

73

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

Sheet No ....
Made By ....
Checked By ....
Date ....
Ref ....

JOB ....
SECTION CORBEL

$a_v/d = 200/550 = 0{,}363$, $v/f_{cu} = 5{,}01/40 = 0{,}125$

$z/d = 0{,}67$

so $z = 0{,}67 \times 550 =$

$= 368{,}5\ mm^3$

$x = (d - z) = (550 - 368{,}5)$

$= 181{,}5\ mm$

$F_t = V \cot\beta = 881.76 \times 200/368.5$

$= 478.57\ kN.$

From strain triangle $\varepsilon_s = 181/368 \times 0{,}0038$

$= 0{,}00172 < 0{,}002.$

$f_s = 351\ N/mm^2.$

$A_s = (478.57 \times 10^3)/351 = 1363\ mm^2$

~~Provide~~ 3T25 A_s prov. $1473\ mm^2$

$100A_s/bd = 100 \times 1473/320 \times 550 = 0{,}84$

Allowable Shear Stress:

$= 0{,}6\ N/mm^2 \times 2 \times 550/200$

$= 3{,}3\ N/mm^2$

$v = 5{,}01\ N/mm^2$

So $b(v - v_c) = 320(5{,}01 - 3.3) = 547$

$f_{yv} = 460$, T16 @ 250 or $\boxed{\text{T12 @ 150}}$ → Provided over $550 \cdot 2/3$

74

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

Sheet No N1
Made By AP.
Checked By
Date
Ref.

JOB

SECTION NIB TO BRIDGE BEAM DESIGN.

Ultimate design load = (6+1,5)×1,4 + 4×1,6
= 10,5 + 6.4
= 16.9 kN/m²

Reaction each end = $\frac{2.3}{2}$ × 16.9
= 19.435 kN for each m run

Design bearing stress
= 0,4 × 25
= 10 N/mm²

Effective bearing length
= 100 + 1000/2
= 600mm

Net bearing width = 19.435×1000/600×10
= 3.2mm min. width of 40mm can be used.

Allowance for spalling
= 20mm

Allowance for inaccuracies
= 25mm

Total min bearing width = 85mm

Nominal length limit = 2.3 + 2×0.085
= 2.47mm

Nib projection = 85 + 40 + 13/2 = 131 mm say 135mm

Assume 40mm cover, line of action of the load = 135 − 15 = 120mm from face of beam.
Distance a = 120 + 40 + 12 = 172mm

75

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

Sheet No N2.
Made By KP.
Checked By
Date
Ref.

JOB
SECTION

M = 19.435 × 0.172 = 3.3 kNm/m run.

~~effective~~ Depth = 140 - 40 - 12/2
= 90mm

$M/bd^2 f_{cu} = 3.3 \times 10^6 / 1000 \times 94^2 \times 25 = 0.0149 < 0.156$

$A_s = 3.3 \times 10^6 / 0.87 \times 410 \times 0.95 \times 94 = 103 mm^2/m$.

Spacing not greater than
3 × 94 + Bar Diameter

Min = 0.13/100 × 1000 × 140
= 182mm²

Use min = T10 @ 250 c/c

$v = (19.435 \times 10^3)/1000 \times 94 = 0.206\ N/mm^2$

$100A_s/bd = 354 \times 100 / 1000 \times 94 = 0.376$

$v_c = 0.79 \times (0.376)^{1/3} (400/94)^{1/4} = 0.65\ N/mm^2$

Allowable Shear Stress = 0.657 × 2 × 94/172
= 0.72 N/mm²

$v_c/2 = 0.36 > 0.206$

So curtailment of bars is allowable.

ANNEX No. 3

TRANSFORMER TOWER

STRUCTURAL CALCULATION

Consulting Engineers and Project Managers

Telephone +263-4-750775/9 Telefax +263-4-750780

Bending Schedule

CHIMOIO WATER SUPPLY PROJECT

TRANSFORMER TOWER

Sheet Nº:	4007/94/C/102
Made by:	R.S.B.
Checked by:	A.P.
Date:	JULY 1994
Drawing Reference:	4007/94/C/2.12

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
BASES	01	T16	10	40	1025	38	215	670			
4#											
BEAM 1	02	T12	4	24	1900	38	865	230			
6#	03	T16	2	12	2000	20	2000				
	04	T12	2	12	2000	20	2000				
	05	R8	11	66	1000	60	230	180			
BEAM 2	02	T12	4	24	1900	38	865	230			
6#	05	R8	14	84	1000	60	230	180			
	06	T12	2	12	2500	20	2500				
	07	T16	2	12	2500	20	2500				
SLAB 1	11	T12	18	18	3025	33	2830				
1#	12	T12	15	15	4025	33	3830				
	13	T10	18	18	3025	33	2830				
	14	T10	15	15	4025	33	3830				
COLUMN 1	08	T16	4	16	6000	42	690	305	4885	65	
4#	09	T16	4	16	2250	51	265	2030			100
	10	R8	36	144	900	60	180	180			

All Bending Dimensions are in accordance with BS4466

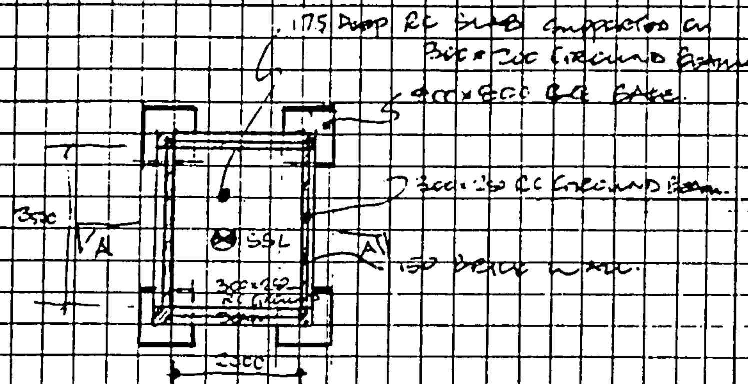
76

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

Sheet No ①
Made By
Checked By
JOB
Date
SECTION

Ref to DRG. 4007/94/C/2.1

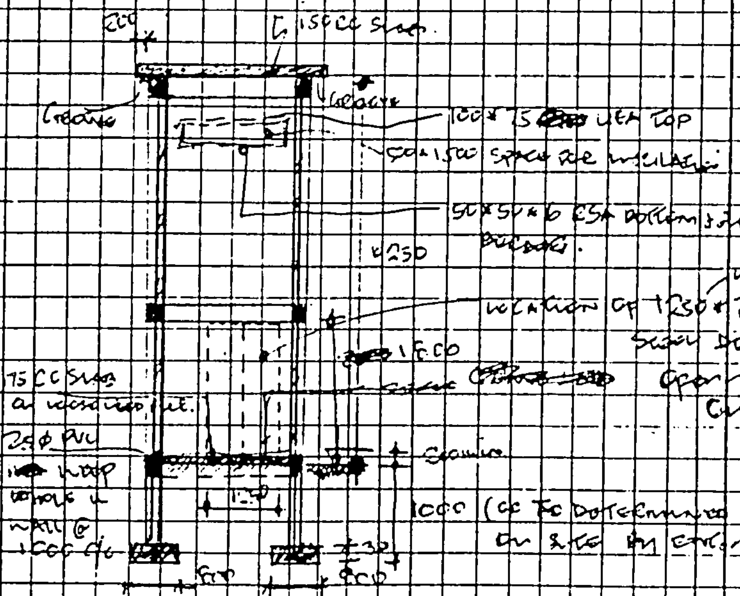


FOUNDATIONS AND PLAN AT GROUND LEVEL.

NOTE.
1. Founding level = 1000mm below natural ground level.
2. SSL = 150mm above NGL
3. Schedule of building frame members sizes.

Bases = 800 x 800 x 350 deep.
Columns = 250mm sq
Ground beams = 300 x 250
Ground slab = 175 deep.
Ring beams = 300 x 250
Roof slab = 150 deep.
Brickwalls = 150 wide. Plastered outside and internally & ext.

Similar for slab but see ventilation



SECTION

DATE 17/6/94

FAX: 00 263 4 750780

TO: MIRKO RISTIC, C/O LAYOUT JOHANNES FREY
FROM: STEPHEN HUGMAN, ADRA.

1) TRANSFORMER TOWER

PLEASE FIND ATTACHED DRAWINGS FOR THE TRANSFORMER TOWER.

WE ARE REQUIRED TO PROVIDE THE CIVIL WORKS; BUILDING, PLUS DOOR, VENTILATORS, WINDOWS, 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)

RADIO MAST MAY BE FIXED TO REAR OF TOWER.

RICARDO DO AMARAL ~~ED~~ OF AMALGAMATED POWER MAY PHONE YOU ABOUT PUMPS

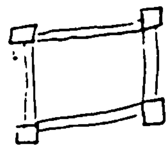
I WILL BE IN CHIMOIO SUNDAY ? AND

80

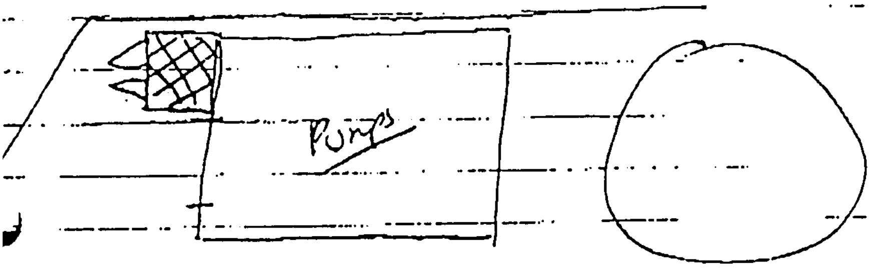
4) DO YOU HAVE DETAIL OF ~~[illegible]~~ RIVER CROSSINGS?

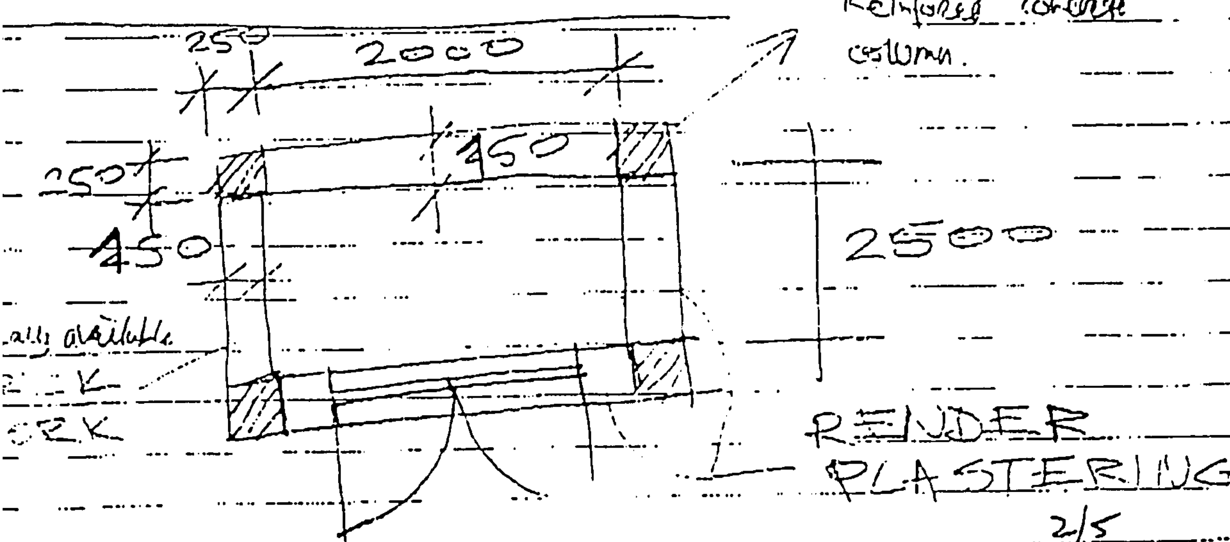
HURRY BACK ?

Steve

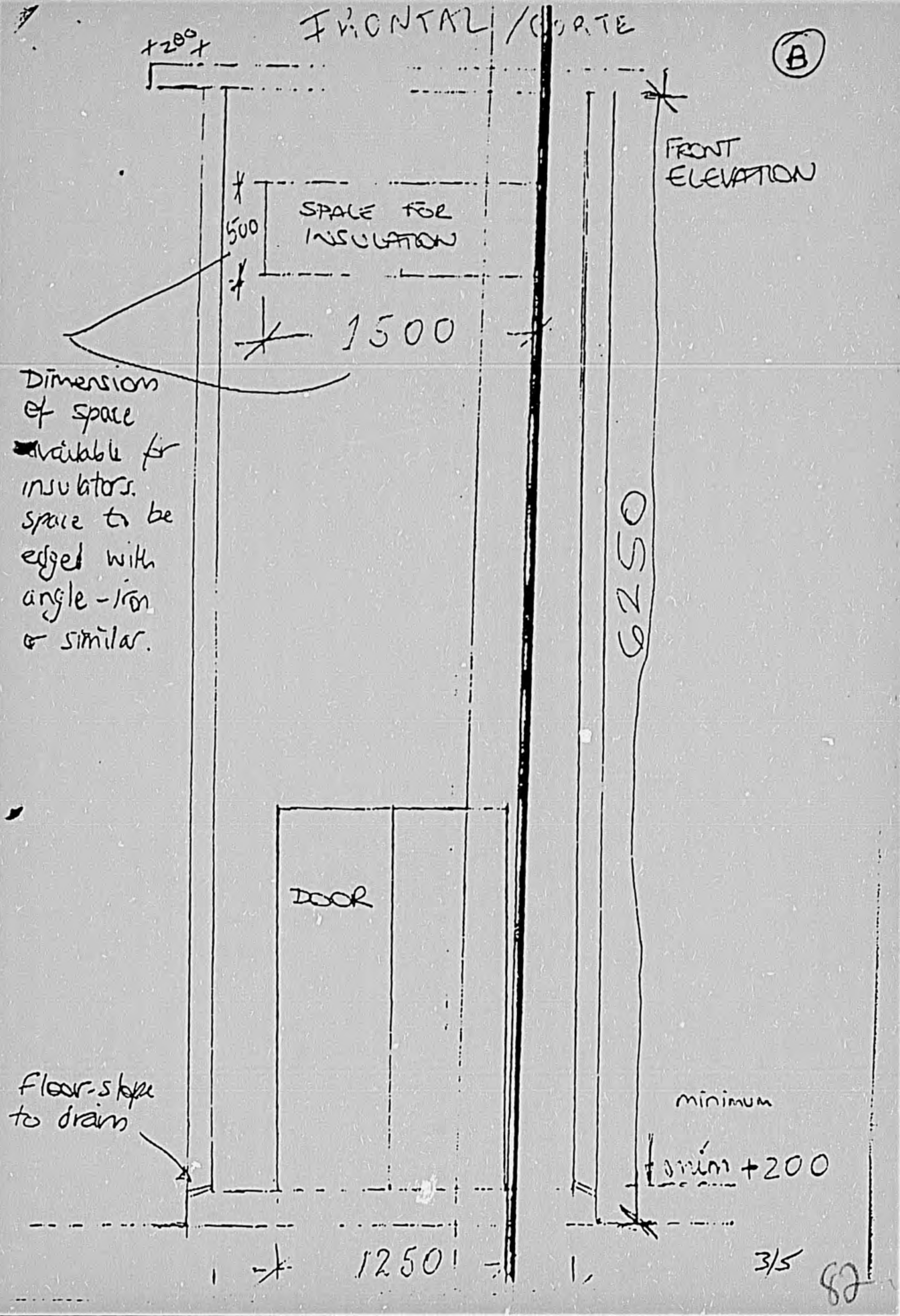


(A) ROUGH LAYOUT, TRANSFORMER TOWER

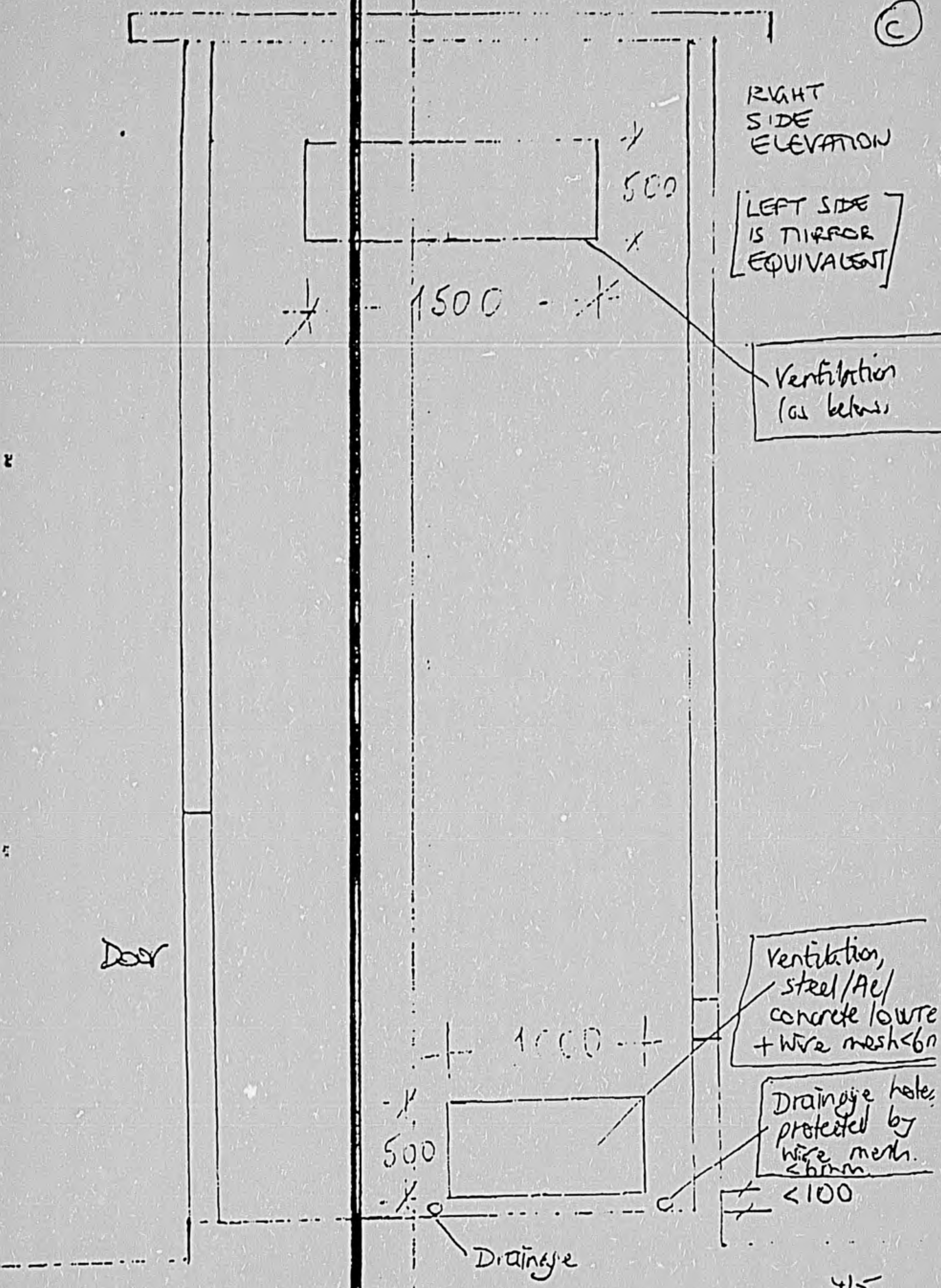


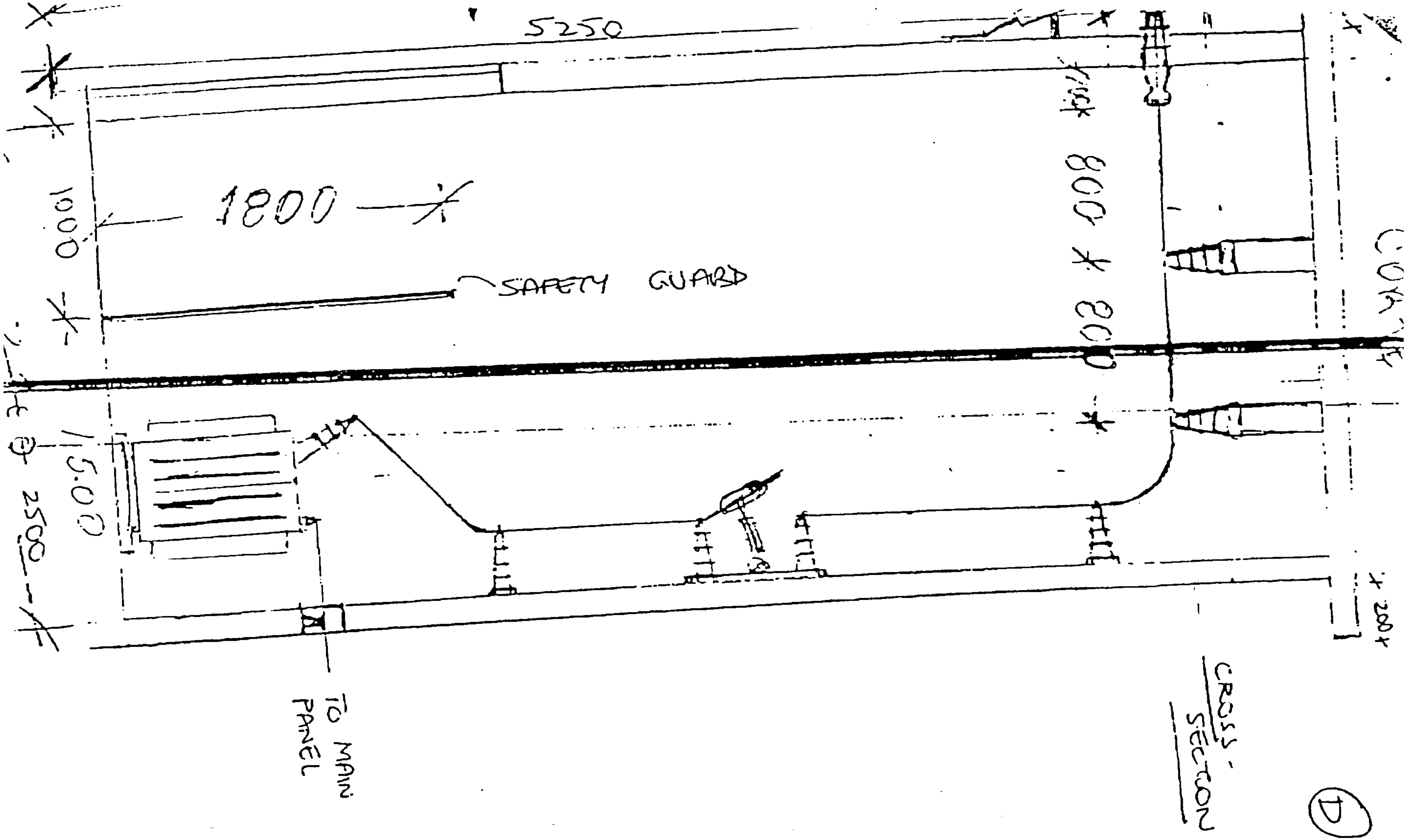


2/5


FRONTAL / CORTE
B
+200+
500
SPACE FOR INSULATION
1500
Dimension of space available for insulators. space to be edged with angle-iron or similar.
FRONT ELEVATION
6250
DOOR
Floor-slope to drain
minimum
drain +200
1250
3/5
82

LAT[illegible]L DI[illegible]/CONT[illegible]


C
RIGHT SIDE ELEVATION
LEFT SIDE IS MIRROR EQUIVALENT
500
1500
Ventilation (as below)
Door
Ventilation, steel/Al/ concrete louvre + wire mesh<6m
1000
500
Drainage holes protected by wire mesh <6mm
<100
Drainage


5250
1800
1000
SAFETY GUARD
800 x 800
1500
2500
TO MAIN PANEL
CROSS - SECTION
D

ANNEX No. 4

CALCULATION FOR SIZE OF TRANSFORMER

85

CHIMOIO WATER SUPPLY PROJECT

LOAD ESTIMATES.

Intake Tower Pump — load ~ 92kW.
At 0.8 p.f = 115kVA.

Irrigation Pump — load = 20kW
At 0.8 p.f = 25kVA.

1 ring socket — load = 10kW
At 0.8 pf = 12.5kVA.

Ancillary services (Lighting + Power) — load = 5kVA.

Total connected load = 157.5 kVA

Worst case loading:

Assume Intake Tower Pump starting

starting load = $\frac{5}{\sqrt{3}} \times 115kVA$

= 332 kVA

Irrigation pump = 25kVA

Other loads = 17.5kVA

Total starting load = 374.5kVA

Transformer size = 315kVA

20% overload for short period = 378kVA

∴ 315kVA Transformer is adequate.

ANNEX No. 5

HEADER RESERVOIR

STRUCTURAL CALCULATION

Consulting Engineers and Project Managers
Telephone +263-4-750775/9 Telefax +263-4-750780

Bending Schedule

CHIMOIO WATER PROJECT
RESERVOIR

Sheet Nº:	4007/94/C/101
Made by:	R.S.B.
Checked by:	A.P.
Date:	JULY 1994
Drawing Reference:	4007/94/C/4.2

Member/ Section	Bar Mark	Size	Nº in each	Total Nº	Length [mm]	Shape Code	A	B	C	D	E
RESERVOIR	01	T12	152	152	2400	39	1170	125			
	02	T12	307	307	2925	20	STR				
	03	T12	253	253	1525	51	440				
	04	T12	172	172	2525	52	950	375	950		
	05	T12	190	190	5675	65	5675				
	06	T12	16	16	3275	42	610	305	2240	60	
	07	T12	16	16	1425	37	1160				
	08	R8	88	88	775	61	165	165			
	09	T12	48	48	900	35	700				
	10	T12	96	96	1500	61	465	210			
	11	T12	15	15	2650	61	695	550			
	12	T12	15	15	2875	61	400	960			
	13	T12	14	14	3100	39	1775	150			
	14	T12	28	28	6975	20	STR				
	15	T12	96	96	1725	39	810	175			
	16	T12	96	96	2475	82	810	210			
	17	T12	16	16	2050	35	1850				
	18	T12	7	7	2700	38	445	1850			
	19	T12	15	15	3200	61	685	835			
	20	T12	16	16	2400	38	305	1850			
	21	T12	14	14	3700	55	1350	150	1910	150	
	22	T12	24	24	2150	33	1860				
	23	T12	190	190	2000	65	1980				3155
	24	T12	56	56	1650	39	810	75			
	25	T12	56	56	6600	20	STR				
	26	T12	156	156	1050	85	720	185	50		
	27	T12	20	20	1775	82	695	50			
	28	T10	4	4	1350	20	STR				
	29	T10	20	20	1150	39	545	95			
	30	T12	8	8	2300	20	STR				

All Bending Dimensions are in accordance with BS4466

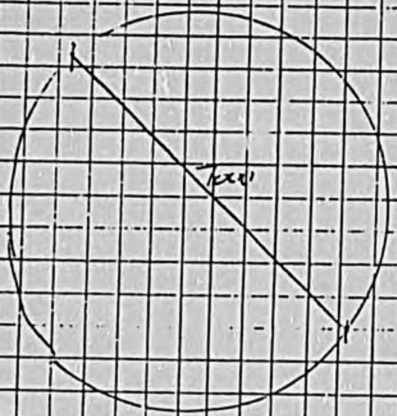
Nicholas O'Dwyer & Partners
Consulting Engineers
2 Robert Mugabe Road
P.O. Box UA 424, Union Avenue, Harare
Telephone 702531-5
Telefax 702536
Telex: 22345 CODWYR ZW

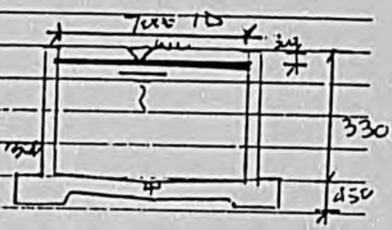
Sheet No 1
Made By
Checked By
Date
Ref. 1.3 Reservoir/

JOB
SECTION Water Tank/Reservoir.

Description

- Circular Reservoir 7000 mm internal Diameter
- Max Amendment of Depth Above (invert) level = 3000
- Total Depth of Reservoir 3.0(+0.3 for Base) = 3300





Section thru circular tank

- Assumption: Tank will be designed following these assumed steps

1. Serviceability Criteria to determine wall thickness
2. Design of vertical bending steel. Area to be determined from ULS for flexure. Spacing will be checked for the serviceability state of cracking, using formula in Appendix, BS 8007.
3. Hoop Reinforcement is designed in the service state at 1/4th points up the wall
4. Shear resistance will be checked to BS 8007 & Table 3.7 BS 8110
5. Base Design

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 COOWYR ZW

Sheet No (2)
Made By
Checked By
Date

JOB

SECTION Reservoir Determination of wall Thickness

Ref 103 Reservoir/

(i) Determination of wall thickness

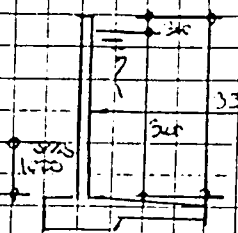


Fig 1

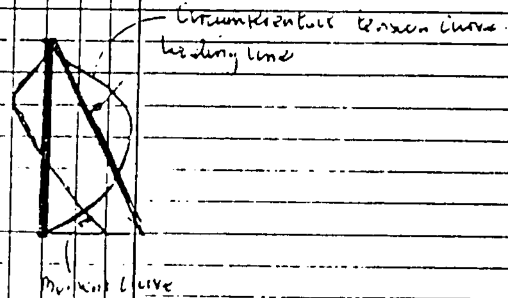


Fig 2

Fig 2 shows forces acting on a free top fixed base circular wall subjected to triangular loading. There are 2 critical positions to fix the wall thickness: max hoop tension approx halfway up the wall and max moment at the base.

Internal I.C & CA suggestions: taking value of 1.5 N/mm² max concrete tension.

Reynolds Westbrook Table A3: for Ring tension

$T(kN) = w \cdot H \cdot r \times$ coefficient $H^2/Dt \sim 33m^2/70m \times 0.25 \approx 6.22$

for $H^2/Dt \approx 6$, max hoop tension $= 0.514 \times w \times H \times r$.

If t is in metres, $f_{ct} = 1.5$ N/mm² then the tension capacity of the concrete /m height is $1.5 \times t \times 10^3$

Equating: $0.514 \times w \times H \times r = 1.5 \times t \times 10^3$

$\therefore t = 0.000342666 \times w \times H \times r$

But since $H^2/Dt = 6$, then $r = H^2/12t$

$\therefore t^2 = 0.0000285555\, w \times H^3$ (1)

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

Sheet No (3)
Made By
Checked By
Date

JOB
SECTION Reservoir (thickness)
Ref 103 Reservoir

For the bending criteria at the base, we will make an assumption of $M/bt^2 = 0.50\ N/mm^2$ for

$f_{cu} = 30\ N/mm^2$, $d/t = 0.75$

From the Table for moments (Appendix Table A5), Evans & Westbrook,

$$M\ (kN\,m/m) = w \cdot H^3 \times \text{coefficient}$$

for $H^2/Dt = 60$ $M = 0.0187 \times wH^3$ kN m/m

But the moment of resistance/m width

$$M_r = 0.50 \times 10^3 \times t^2$$

Equating: $0.5 \times 10^3 \times t^2 = 0.0187\, w\, H^3$

$$\therefore t^2 = 0.0000374\, w\, H^3 \quad \text{--- (2)}$$

Criteria (2) is obviously critical

$\therefore$ for $w = 9.8\ kN/m^3$, $H = 3.300\ m$,

$$t^2 = 0.0000374 \times 9.8 \times 3.3^3$$
$$= 0.0132$$
$$t = 0.115 \quad (2.25m)$$

t = 115mm
Assume $\underline{t = 200mm}$

For hoop tension/compression, R.C. Designers Handbook suggests:

$$t = \frac{wHD}{2}\left[\frac{1}{f_{ct}} - \frac{(m-1)}{f_{st}}\right]$$

where f_{ct} = tensile stress of concrete = 1.5 N/mm²
f_{st} = 130 N/mm²
m = modular ratio = 15

$$\therefore t = \frac{9.8 \times 3.3 \times 7.0}{2}\left[\frac{1}{1.5} - \frac{14}{130}\right]$$

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 ....

Sheet No (4)
Made By ....
Checked By ....
Date ....
Ref. 1031 Reservoir

(2) Limit State Design for Vertical Steel

0 Assumption: For the Vertical Steel (bending) factored loads will be used for the water pressure with base assume i) as pinned & ii) as fixed

Table A5 (Reynolds & Steedman) for fixed base and Table A6 for pinned base will be used. (Although the wall thus will fix to with the base slab, any rotation will throw the moment into the wall.)

Moment = coeff × w × H^3 = coefficient × ~~322.333~~ ~~7830~~ 352.5 kN.m

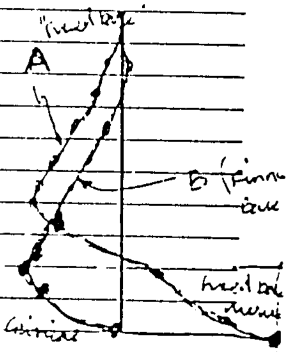
Coefficients are read for H^2/Dt = $3300^2 / 7200 \times 200$

= 7.7 say 8.

Earth pressure will be ignored as minimal

Table 1

Depth below top (m.)	Fixed base coefficients	Moments (c.w.H^3) (kN.m) (A)	Pinned base coefficients	Moments (c.w.H^3) (kN.m) "B"
0.1H	0.0	0	0	0
0.2H	+0.0001	+0.035	0	0
0.3H	+0.0002	+0.07	-0.0002	-0.071
0.4H	+0.0008	+0.282	0	0
0.5H	+0.0016	+0.564	+0.0007	+0.247
0.6H	+0.0028	+0.987	+0.002	+0.71
0.7H	+0.0038	+1.339	+0.0038	+1.339
0.8H	+0.0029	+1.022	+0.0057	+2.0
0.9H	+0.0022	-0.775	+0.0054	+1.9
1.0H	-0.0146	-5.15	0	0



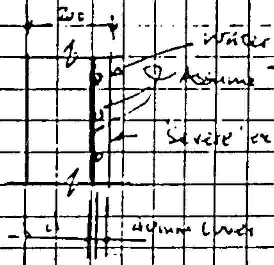
— max. (pos/moments.
• Critical moments.

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 ODWYR ZW

Sheet No (5)
Made By
Checked By
Date
Ref. 1031 / Reservoir /

JOB
SECTION Reservoir ULS Design



From Fig 3
$d = 200-40-16-8$
$d = 136$ mm

For water face M_c
$= 5.15$ kN.m
Table 1

Fig 3.

$\gamma_f = 1.4$; ULS Moment $= 5.15 \times 1.4$
$= 7.21$ kN.m

$\frac{M_u}{f_{cu} b d^2} = \frac{7.21 \times 10^6}{30 \times 1000 \times 136^2} = 0.013$

$z/d = 0.95$; $z = 0.95 \times 136$
$= 129.2$

$\therefore As = \frac{7.21 \times 10^6}{250 \times 129.2} = \cancel{221 \text{ mm}^2}$ 223 mm^2 — Assume min steel Reinforcement

$A_{min} = \frac{0.24}{100} \times 1000 \times 200 = (480 \text{ mm}^2/\text{m})$

$\therefore$ Use min steel $\Rightarrow$ R12 @ 150, As prov. $= (754 \text{ mm}^2/\text{m})$ (Note R12 @ 200 Also Adequate)

Check cracking

From Elastic Theory : $z/d = 1 - \frac{1}{3}\left(\frac{x}{d}\right)$ Assuming $x = \frac{1}{2}d$.

$z = 125$mm (120)

$f_s = \frac{M \text{ (service)}}{A_s \times z} = \frac{5.15 \times 10^6}{754 \times 125} = 54.6 \text{ N/mm}^2$ (52.75) (Computed)

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

Sheet No 6
Made By
Checked By
Date
Ref. 1031 | Reservoir/

JOB
SECTION Calculation of crack width.

$f_s \geq \frac{0.8 f_y}{E_s} \; (\text{or } 8000) = \frac{0.8 \times 250}{200 \times 10^3} = 0.0010$

$\varepsilon_s = \frac{54.6}{200 \times 10^3} = 0.000273 < 0.001$ ✓ OK.

0.000273

$x/d = -\alpha_e \rho + \sqrt{\alpha_e \rho (2 + \alpha_e \rho)}$

$\alpha_e = \frac{E_s}{E_c} = 15.4 \qquad \rho = \frac{754}{10^3 \times 136} = 0.00554.$

$\therefore \alpha_e \rho = 15.4 \times 0.00554 = 0.0854$

$x/d = -0.0854 + \sqrt{0.0854 (2 + 0.0854)}$

$= 0.336$

$\therefore x = 0.336 \times 136 = 45.8 \text{mm}$

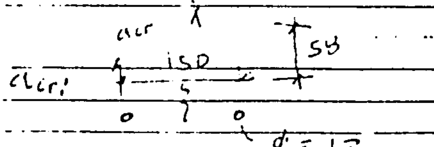
$\varepsilon_1 = \left(\frac{200 - 45.8}{136 - 45.8}\right) \times 0.000273 = 0.00047$

$\varepsilon_m = 0.00047 - \frac{10^3 (200 - 45.8)^2}{3 \times 200 \times 10^3 \times 754 (136 - 45.8)}$

$= -0.00058 + 0.00047$

$= -0.0001$

$\varepsilon_m < 0.0001$ mm



$a_{cr} = \sqrt{58^2 + 150^2} = 160.8$

$w_{cr} = \frac{3 \times 160.8 \times 0.0001}{1 + 2\left(\frac{160.8 - 53}{\ldots}\right)} = 0.012 \text{mm}$ satisfactory.

94

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

Sheet No 7
Made By
Checked By
Date
Ref 1031 Reservoir/

JOB
SECTION Reservoir : Hoop Tension Check

(3) Check for critical Hoop tension:

By inspection, the critical Hoop force
$= T = C . w . H . R$ for
a pinned base

$C = 0,443$

$\therefore T = C \times w \times H \times R$
$= 0,443 \times 9.81 \times 3.3 \times 3.5$
$= \underline{72.9 \text{ kN}}$

Assuming fs for steel 130 N/mm² (higher for MS)

$A_s = \frac{T}{f_s} = \frac{72.9 \times 10^3}{130}$

$= 560 \text{mm}^2 < 754 \text{mm}$ As prov.

$\therefore$ Hoop tension not critical.

prov. ~~so~~ R12 @ 150 CNRS A.B.

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 ....

Sheet No (8)
Made By ....
Checked By ....
Date ....
Ref. 1B 31 Rosegrove

ULS shear check on wall stem.

Design shear force at base — Table A7 Reger + Westbrook

$= 1{,}4 \times 0{,}174 \times w \times H^2$

$= 1{,}4 \times 0{,}174 \times 9{,}81 \times 3{,}3^2$

$= 85{,}8$ kN/m

From step 2 $A_s = 754\text{mm}^2$ (E12@150)

$100 A_s / bd = 0{,}554$; $v_c = 0{,}61$ $\left(\begin{array}{l} d = 136 \\ f_{cu} = 30 \end{array}\right)$

$\therefore V_c = 0{,}61 \times 10^3 \times 136 / 10^3$

$= 82{,}96$ — $\therefore$ SHEAR NOT OK!

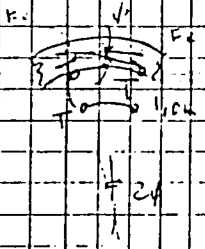
$\therefore$ INCREASE WIDTH OF WALL TO 225mm

$\therefore$ eff Depth $= 141\text{mm}$

and $V_c = 118{,}21$ kN/m $> 85{,}8$

The outside tension bars are to extend a given lap length of say 700mm above base. $\therefore$ check for shear at this point.

Shear at 0,9 H will be calculated under triangular load, ULS shear at the base and a parabolic approximation of the fixed base tension.



Resolving the ring tension T into a circumferential force, F_c & a radial force F_R

For a 1m width of wall, & radius to centre of wall $= 3{,}6175\text{m}$

$2\phi = \dfrac{360 \times 1}{\pi \times 7} = 16{,}5°$

$\phi = 8{,}20°$

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

Sheet No (9)

Made By

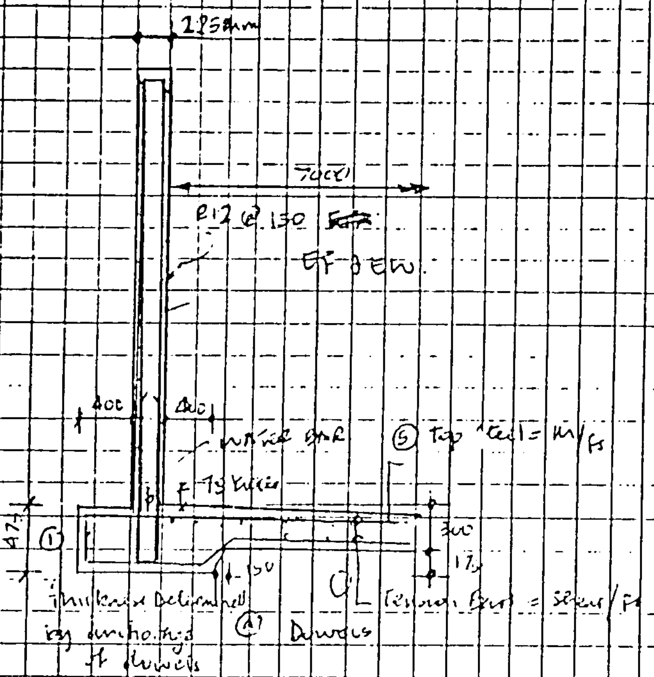
Checked By

Date

Ref.

JOB

SECTION Reservoir Section



Reinforcement.

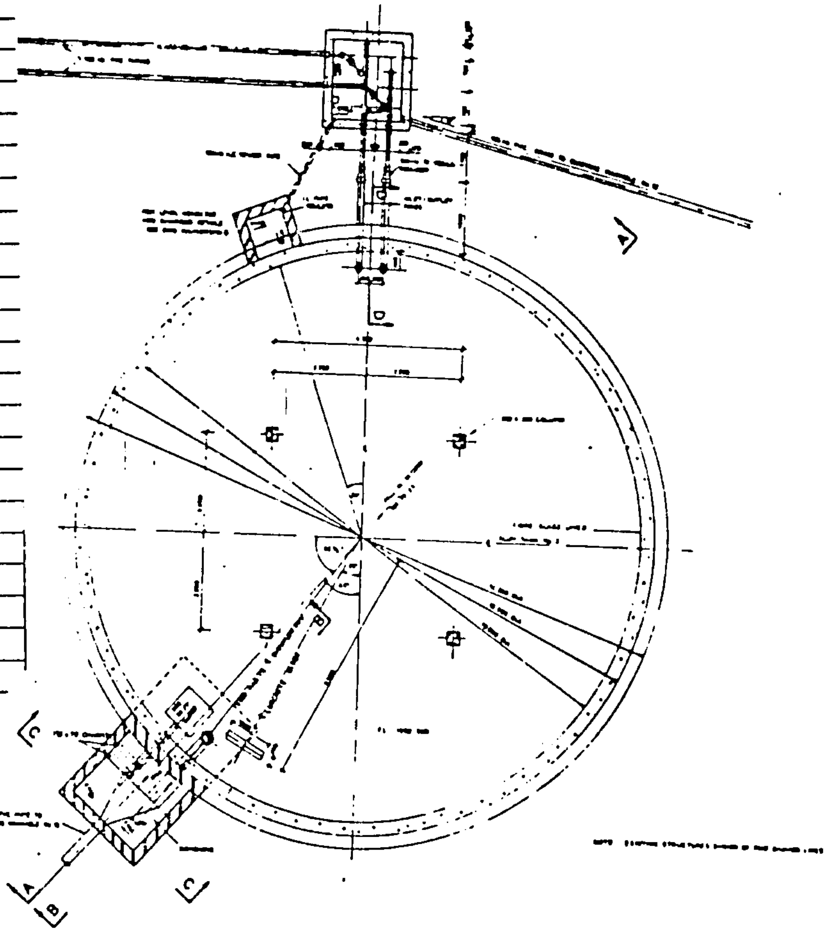
(1) R12 @ 150 E.F. EW. wall sec.
(2) R16 @ 200 Base Top & Bottom
(3) min R12 @ ~~150~~ Base Distribution
(4) R16 @ 200 Dowels.

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

Sheet No A1
Made By
Checked By
Date
Ref Fcs/1031

JOB
SECTION Appendix A



FLOOR PLAN

G6

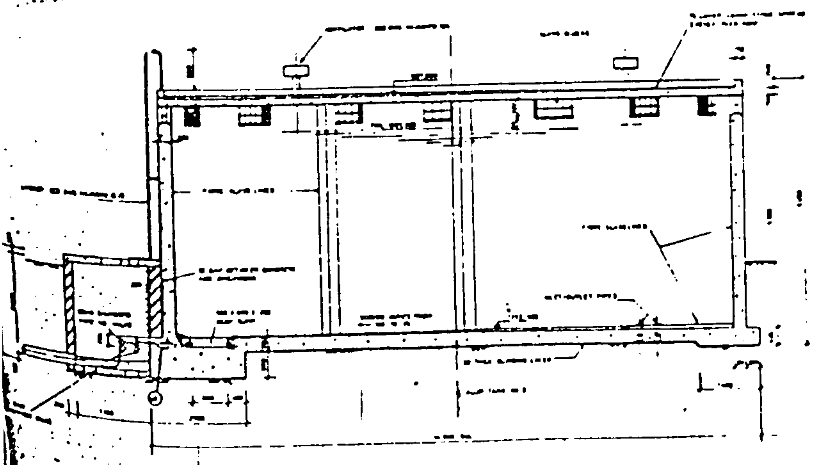
Nicholas O'Dwyer & Partners
Consulting Engineers

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

Sheet No A2
Made By
Checked By
Date
Ref

JOB

SECTION Appendix E



SECTION A–A

GG

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 Appendix C

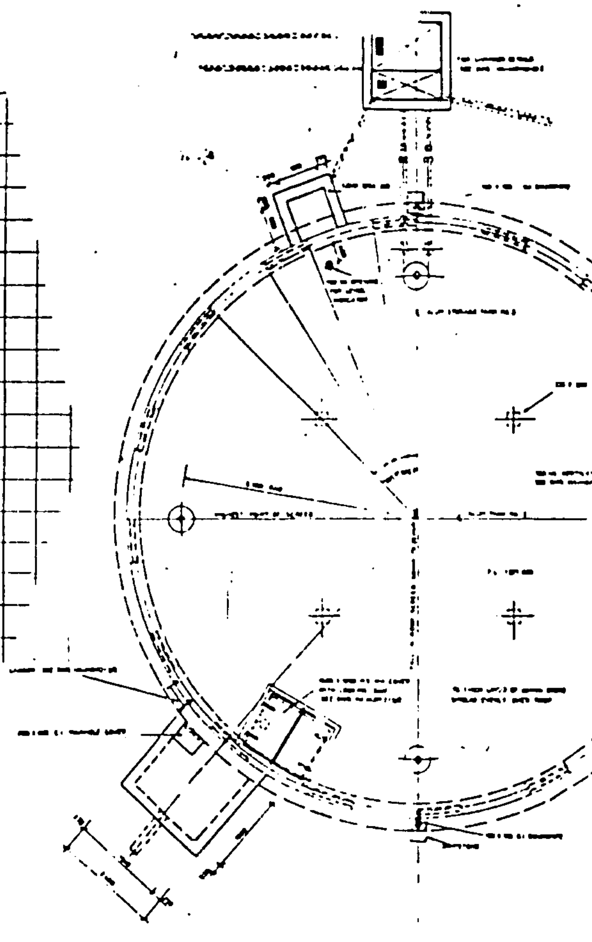
Sheet No A3

Made By

Checked By

Date

Ref 1:31 Res/



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 Appendix E

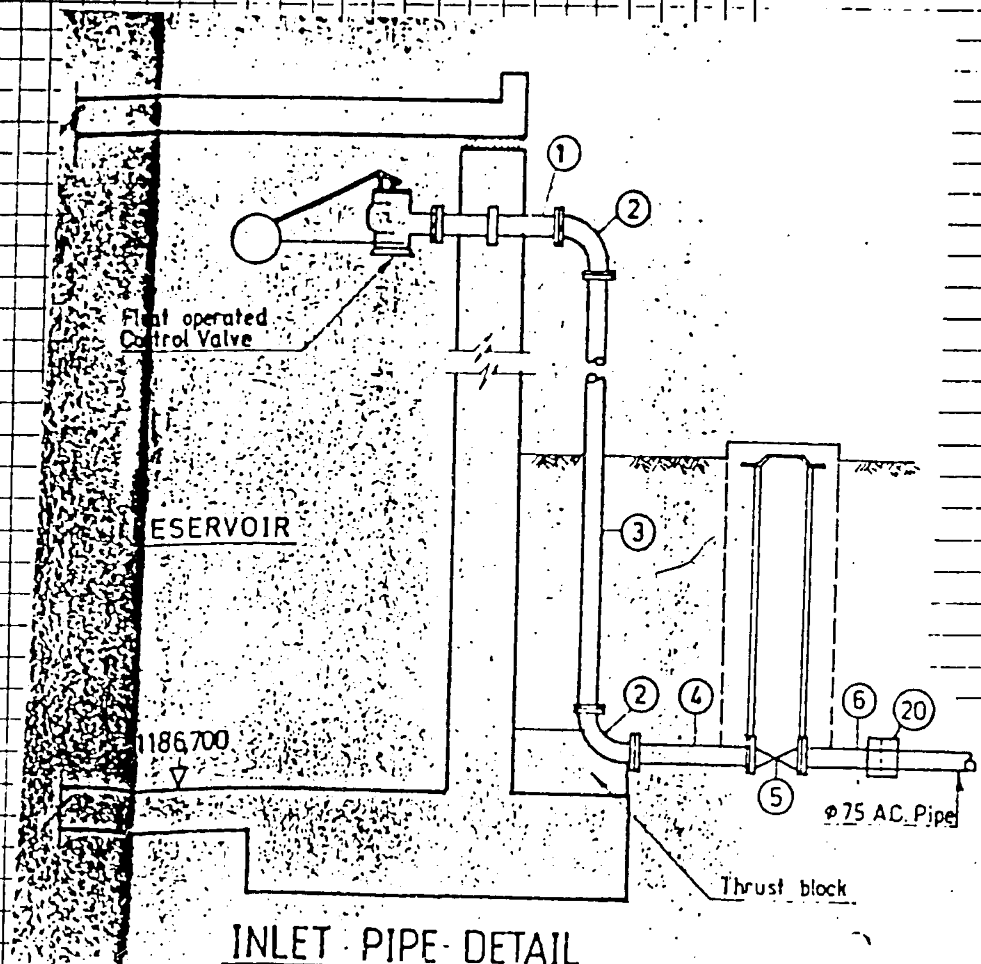
Sheet No 15

Made By

Checked By

Date

Ref



INLET PIPE DETAIL

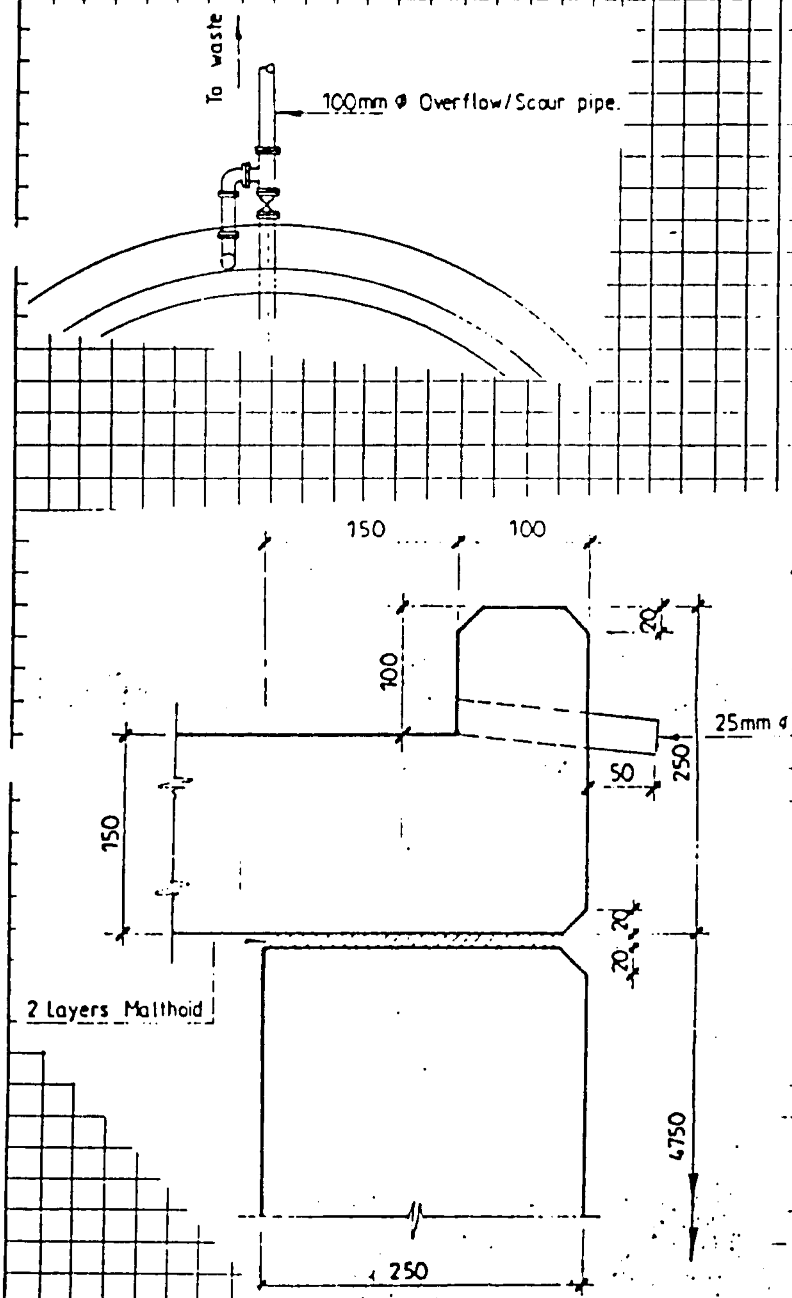
Scale 1:25

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

Sheet No A4
Made By
Checked By
Date
Ref Rcs/

JOB
SECTION Afinani D



DETAIL A
Scale 1:5

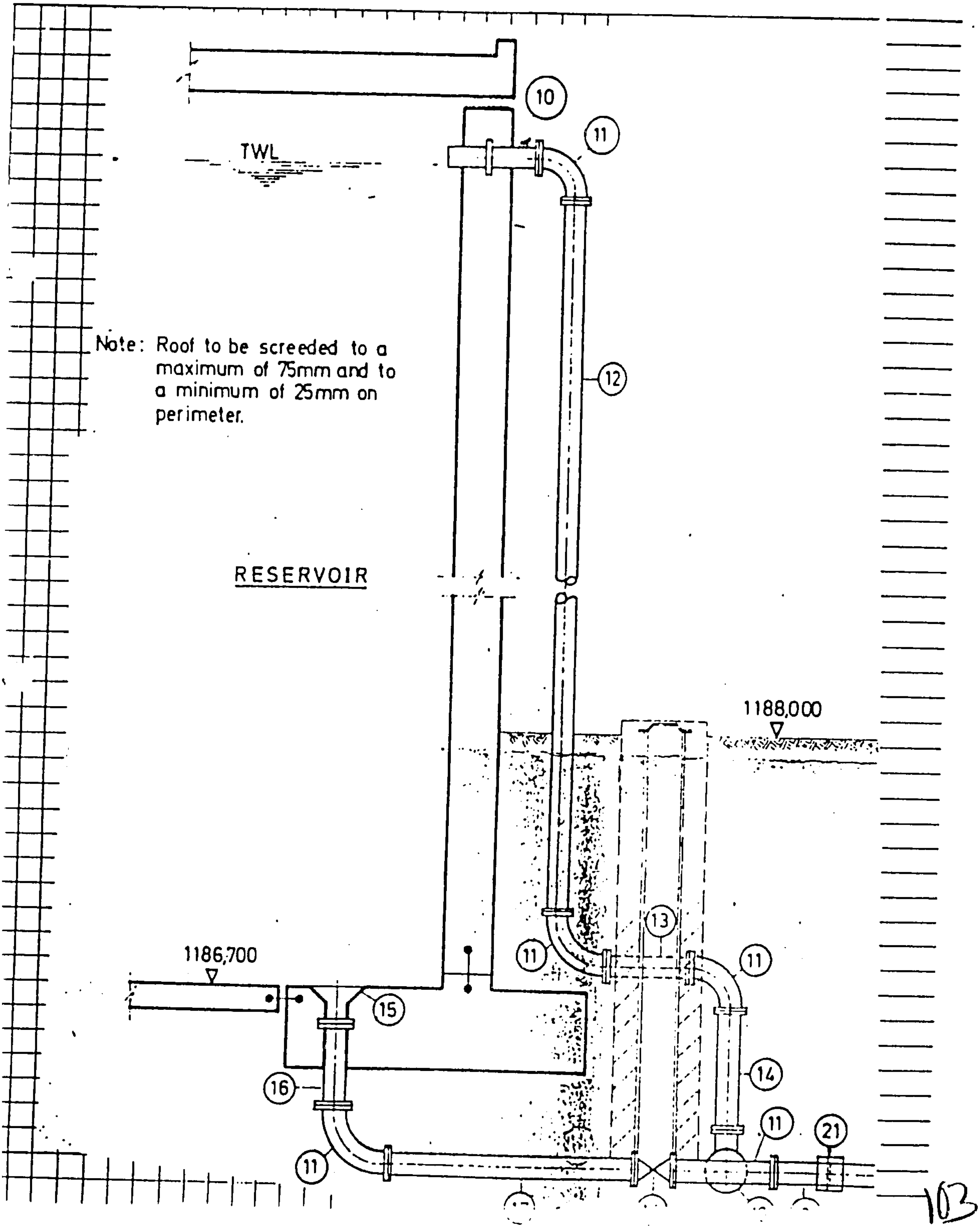
162

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

Sheet No A6
Made By
Checked By
Date
Ref

JOB
SECTION



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

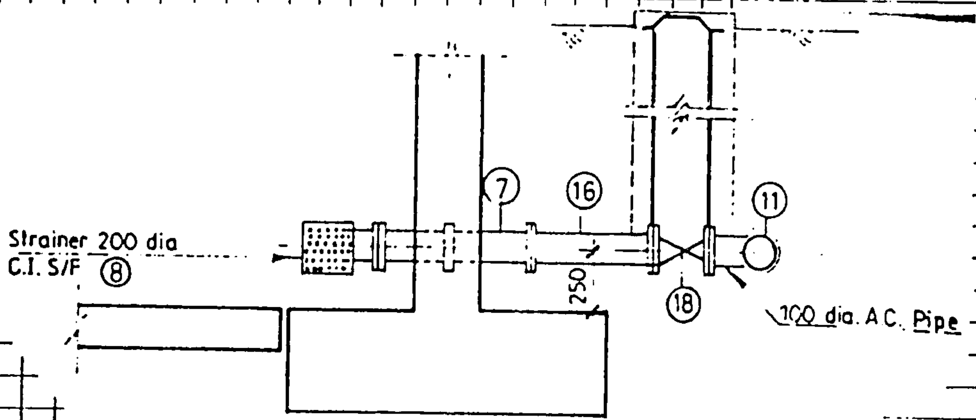
103

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 Appendix T

Sheet No A7
Made By
Checked By
Date
Ref



OUTLET PIPE DETAIL

Scale 1 : 25

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

Sheet No. 4 A8

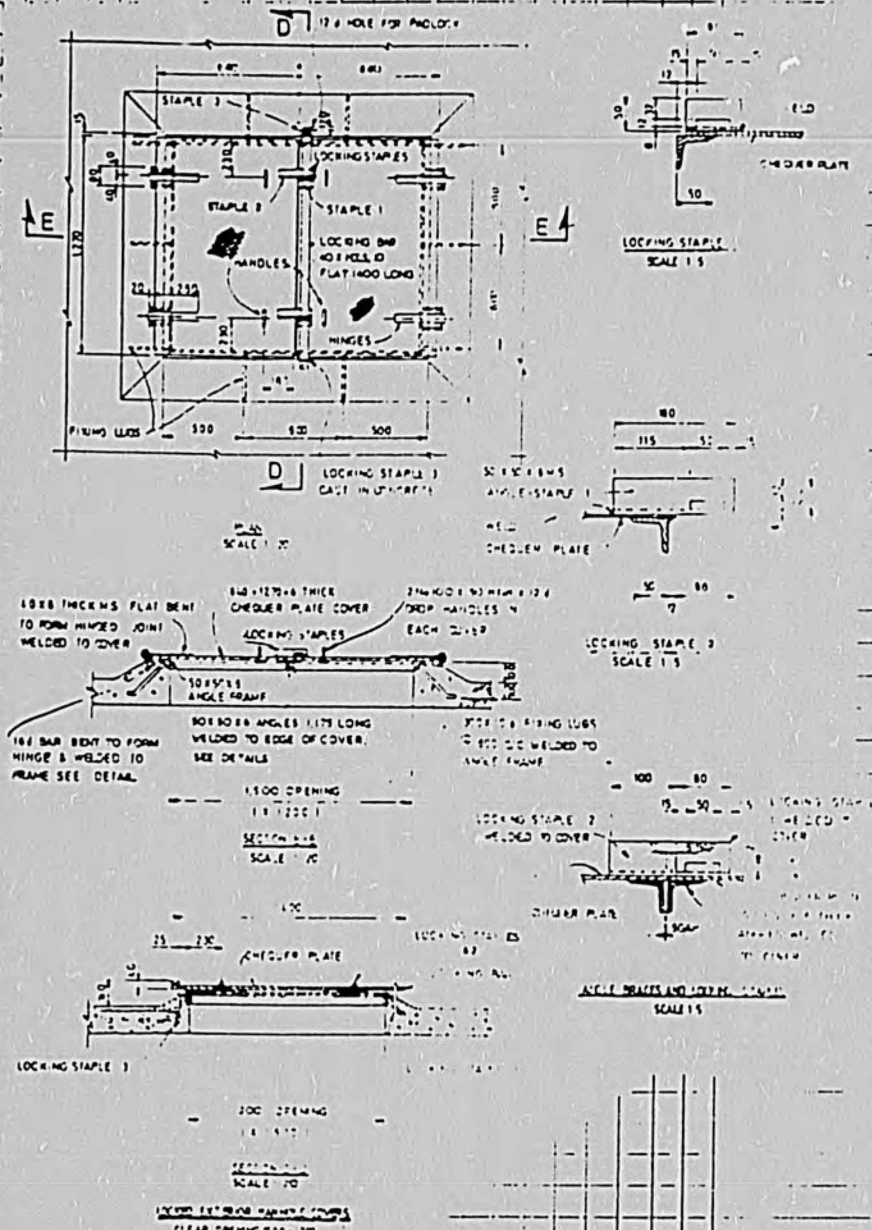
Made By

Checked By

JOB

Date

SECTION Annexure H

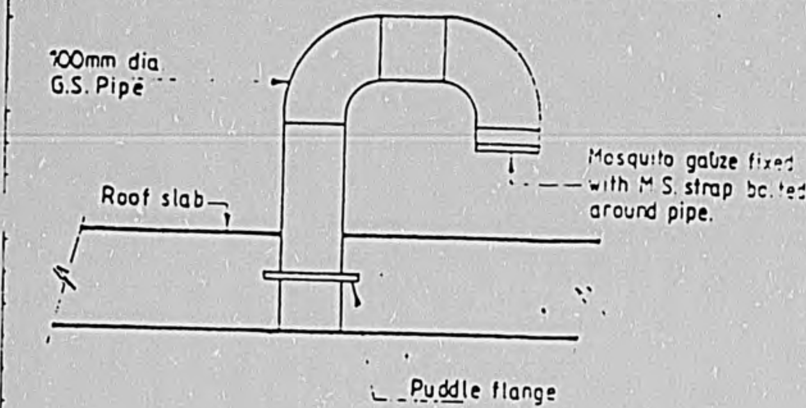


Nicholas O'Dwyer & Partners
Consulting Engineers

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

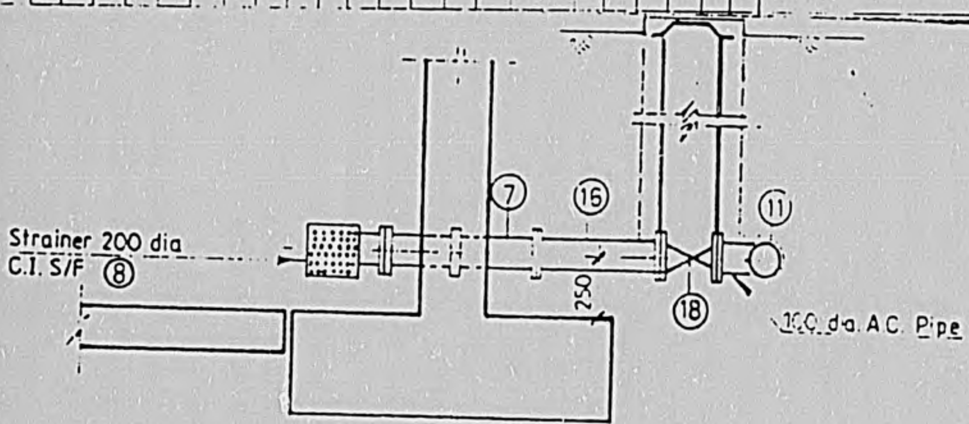
JOB ....

SECTION .... Appendix I



VENTILATOR DETAIL

Scale 1:10



OUTLET PIPE DETAIL

Scale 1:25

106

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

Sheet No A10

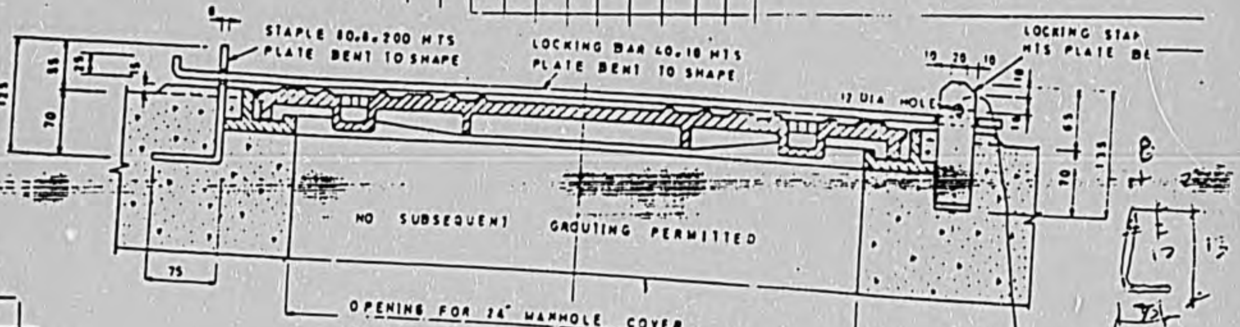
Made By

Checked By

Date

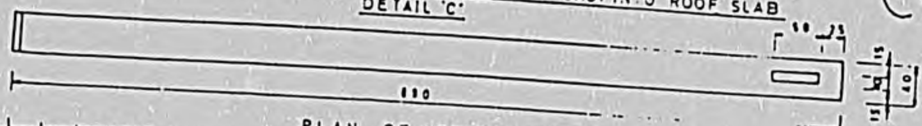
JOB

SECTION Appendix J

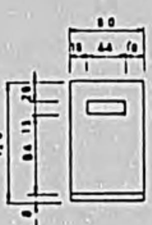


LIGHT DUTY RECTANGULAR COVER CAST INTO ROOF SLAB

DETAIL 'C'



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 ** 1875