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Euroconsult
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Yemen Arab Republic

Rada' water supply and sanitation project

Yemen Arab Republic
Ministry of Municipalities and Housing
Ministry of Electricity and Water
National Water and Sewerage Authority

Kingdom of the Netherlands
Ministry of Foreign Affairs
Directorate General of
Development Cooperation

Rada Water Supply and Sanitation Project

FINAL DESIGN REPORT

VOLUME I - MAIN REPORT

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Euroconsult
Arnhem
The Netherlands

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SUMMARY OF MAIN TECHNICAL FEATURES

Basic criteria		Phase I 1995	Phase II 2010
Population estimate		50.000	75.000
Population served (95%/ 100%)		47,500	75,000
Residential consumption rate av/ max l/c/d		50/ 60	70/ 84
Large consumers consum rate av/ max l/c/d 10%		5/ 6	7/ 8.4
Total consumption demand rate av/ max l/c/d		55/ 66	77/ 92.4
Total consumption av/ max - m^3/ d		2613/3135	5775/6930
Unaccounted for water	m^3/ d	627	1386
(20% of consumption on max. day			
Max. production capacity	m^3/ d	3762	8316

Scope of work (phase I)

<u>Water Supply</u> (NWSA)

- No. of wells to be drilled and constructed — 6
- Reservoir with necessary components (750 m^3) — 1
- Transmission/ collector main - km - DCI (150 - 200 - 300) — 5.54
- Distribution system with necessary components:
  - Primary - km - (200 - 300) DCI — 5.39
  - Secondary - km - 90 - HDPE — 27.27
  - Tertiairy - km - - (50 - 32 - 25) HDPE — 98.41
  - No. of house connections — 5000
- O&M building complex with necessary components m^2. — 774

<u>Sewerage</u> (NWSA)

  - Primary - km - (200 - 250 - 300 - 400) PVC — 8.97
  - Secondary - km (160) PVC — 44.00
  - Tertiairy - km (110) PVC — 48.00
  - No. of house connections — 4500
  - Transmission sewer main - km (400) PVC — 5.00
- Waste water treatment plant unit consisting of approx.
  - Anaerobic pond (9280) m^2 — 4 no.
  - Facultative pond (54,100 - 25,350 - 12,600) m^2 — 3 no.
  - Infiltration pond (7524 m^2):
- Supply of transport vehicles for counterpart staff.

<u>Drainage and Solid Waste Disposal:</u> (MMH)

  - Road surface drainage - km (asphalting + kerbstones) — 13.86
  - Tertiary drainage - m^2 (5 cm concrete layer) — 34000
  - Supply of additional heavy equipment to enable Rada Baladiya to level and prepare the roads and to execute solid waste programmes.
  - Supply of transport vehicles for personnel staff.
  - Continuation of implementation of the EHE programme.

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LIST OF ABBREVIATIONS AND ACRONYMS

AWWA	American Water Works Association
BOD	biochemical oxygen demand
COD	chemical oxygen demand
DGIS	Directorate General for International Cooperation, Netherlands Ministry for Development Cooperation
DIN	Deutsche Industrie Norm (German Industrial Standard)
DIP	Direct Improvement Programme
d.s.	dry solids
EHE	environmental health education
ISO	International Standards Organization
LCCD	Local Council for Community Development
MLSS	mixed liquor suspended solids
MMH	Ministry of Municipalities and Housing
MSL	mean sea level
mwc	metre(s) water column = metre(s) head (water pressure)
N_{kj}	Kjeldahl nitrogen
NWSA	National Water and Sewerage Authority
OC	oxygenation capacity
p.e.	population equivalent
RIRDP	Rada Integrated Rural Development Project
RUA	Rada Urban Area
RUDP	Rada Urban Development Project
RWSSP	Rada Water Supply and Sanitation Project
SS	suspended solids
SVI	sludge volume index
UASB	upflow anaerobic sludge blanket
WHO	World Health Organization
YAR	Yemen Arab Republic
YGEC	Yemen General Electricity Company
YR	Yemen Riyal

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1. INTRODUCTION

1.1. Project objectives

The objective of the project is to improve the public health situation through a well-functioning water supply, sewerage, solid waste disposal and drainage system in the Rada Urban Area (RUA).

The project, therefore, includes the implementation of water supply, sewerage, drainage and solid waste disposal systems, together with a public health education and institutional strengthening programme.

In anticipation of the construction of full-fledged systems, a number of immediate improvements have been made during the so-called Direct Improvement Programme.

1.2. Project area

1.2.1. General location

The project area is situated in the Al Bayda Province of the Yemen Arab Republic (YAR) (see Fig. 1.1.). It comprises the Rada Urban Area, consisting of the communities of Rada and Musalla, and the urbanizing area As Safiyah, located in between these two. It is located at a distance of 55 km from Dhamar, 160 km from Sana'a and 115 km from Al Bayda, on the national highway.

1.2.2. Project area

The project area is the area within and along the proposed ring road, as shown in the master plan of Rada. It includes the communities of Rada, As Safiya and Musalla (see Fig. 1.2).

Topographic information on the project area is based on the information gathered during the Rada Urban Development Project (1980-1982), with additional information obtained from topographical surveys carried out by Consultants in 1988/1989 and aerial photographs of the area made in 1989.

1.2.3. Description of Rada

The Rada Urban Area (RUA) is the largest settlement in the Al Bayda Province, where Al Bayda is the second largest town and seat of the provincial government. The main function of the RUA is that of an administrative and commercial centre in the almost exclusively agricultural Rada District.

The RUA is one of Yemen's so-called secondary towns. It is situated at an altitude of 2100 m, on löss-like soil, along the Wadi Qarn Attah, surrounded by volcanic outcrops.

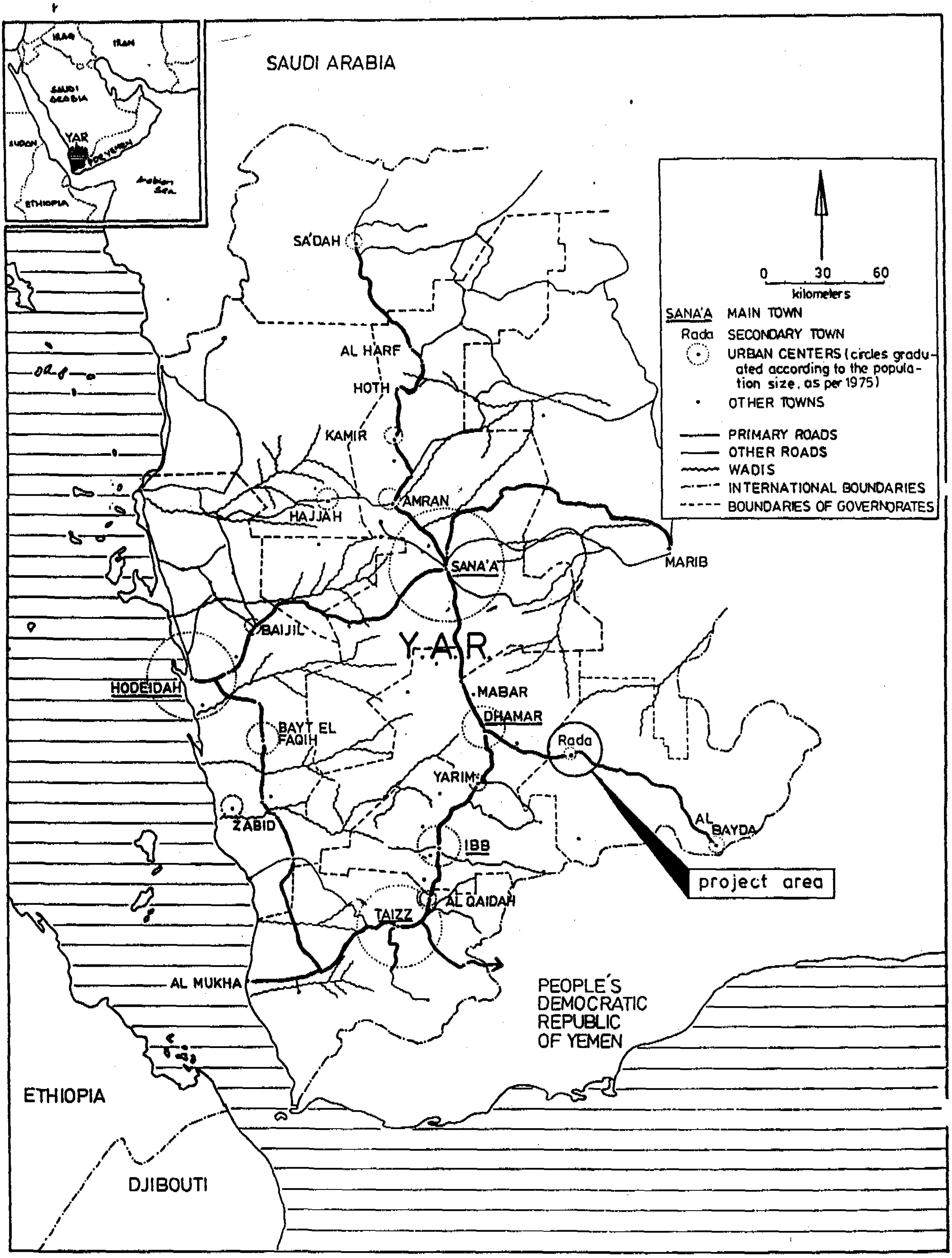


Fig. 1.1. Reference map of the Yemen Arab Republic

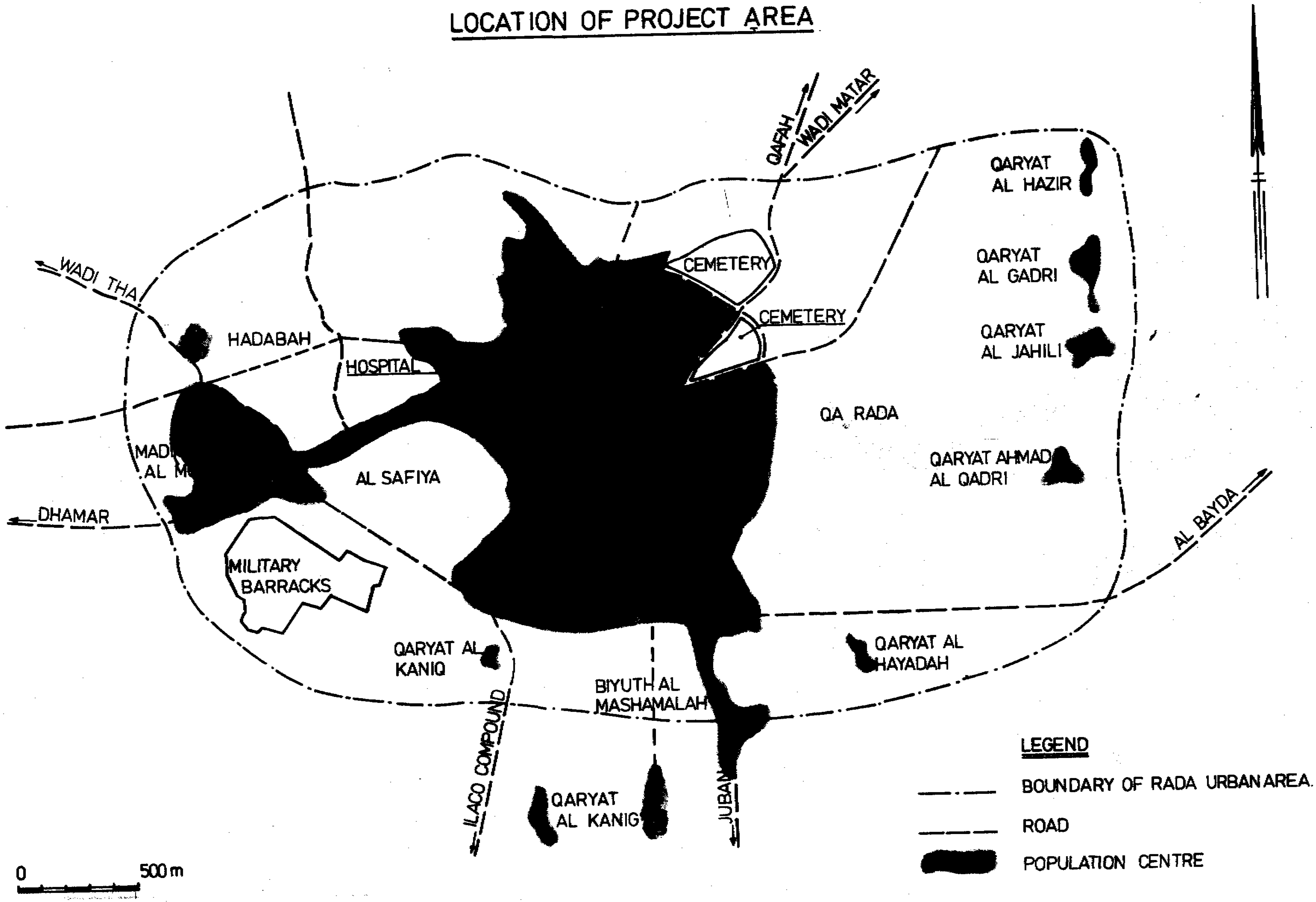


Fig. 1.2. Location of project area

Rada and Musalla are old settlements with many traditional Yemeni houses in a typical Yemeni lay-out. Situated on old trading routes Rada has functioned as a regional centre for a long time. The present connections to the region are formed by the Dhamar-Al Bayda road (since 1981), some regional (improved) feeder roads and many tracks.

The climate is semi-desert, temperate, with much sunshine. Annual rainfall amounts to 225 mm, with temperatures ranging from 2 °C to 22 °C in the coolest, and 15 °C to 30 °C in the hottest months. Usually there is a lot of dust. Local drainage conditions are poor, resulting in many puddles, mud and flooding as a result of rainstorms. The plain's main drainage pattern is through the wadi to the north. There are no perennial streams in the area, and groundwater for agriculture and drinking is drawn from the alluvial deposits, and more especially from the sandstones and volcanic rocks underlying the Rada plain.

The recent development pattern within the RUA is mainly characterized by the growing together of Rada and Musalla in As Safiyah, where many modern one-storey houses are built, and by the growth of the built-up area towards the Dhamar-Al Bayda road.

All housing in the RUA is privately developed. South of the fort the old commercial functions can be found in the old suq, which leads from the social centre in the RUA, the square in front of the Al Amiriyah mosque, to the new suq, where most newer commercial functions can be found, including the bank, and qat and meat shops. There also are the many cattle markets and a daily wood and qat plant market. Around Bab Al Mahjari is a concentration of woodworking and building materials shops. A number of simple industries, mainly manufacturing building materials, can be found outside the centres in the newer developing areas, mostly close to or in residential areas. There too are several schools, some new public buildings (such as the hospital, mother and child clinic, police station), petrol stations, garages, etc.

Many commercial functions have sprung up along the Dhamar-Al Bayda road, forming another commercial centre just south-east of Musalla. Small neighbourhood shops and clothes shops can be found all over the built-up area.

There are no parks or playgrounds, and only few trees. Qat is grown mainly west of, but also in Rada. Fruit trees have almost gone from the private gardens. East of Rada and east of Musalla there are cemeteries.

Several government offices, including a Governor's office, are situated in Rada. The former road camp south of Musalla is now in use with the military. There is no slaughterhouse in Rada; only a semi-official killing place exists north of the city.

Relatively good accessibility to all these functions is provided by the roads and rights of way of the internal road network of the RUA.
A number of roads has been technically improved. Yet, the over-all poor technical state of the road network results in a low level of comfort for all traffic, but also in low speeds that make the area relatively safe for pedestrians and playing children. As elsewhere in the YAR quite often speed bumps have been constructed by the inhabitants themselves.
Most buildings in the RUA have piped water. At present most of the network is above ground.

There is no waste water disposal system, with only part of the houses having soak pits. Most of the waste water, however, still ends up in the streets.

Until recently, solid waste used to be collected only partially and then dumped in the wadi. In addition, many waste dumps could be found all over town. The situation has improved markedly, however, with the implementation of the Direct Improvement Programme, when 2,000 tonnes of solid waste were removed from the city, 95 containers were distributed over the town, and a compactor truck was put in operation for collecting and disposing of solid waste.

Almost all houses and other buildings are supplied with electricity from the YGEC power plant in Rada. The network of 11 kV lines runs overhead, as do the connections from the transformers to the buildings. Electricity supply is erratic, however. Next to the old, practically defunct, street lighting system that was restricted to a few streets, in 1982 some modern street lighting was installed in a number of main streets of Rada. It is expected that around 1992/1993 Rada will be hooked up to the national power grid by means of a 33 KV overhead connection to Dhamar. At that time also the 11 kV grid in Rada itself will be rehabilitated.

1.3. Population forecast

Based on the data of the 1986 population census, the 1988 population of the RUA has been estimated at about 35,000. Compared with earlier population data this would mean a (calculated) average annual population growth rate of over 10%, which is far beyond the national and regional growth rates. For that reason, historic population data has been used with caution, the more so as wide variations in calculated rates for urban and population growth are encountered, depending on the source of data and the delineation of the area assumed.

Taking into account an annual growth rate (at national level) of about 2.5%, the growth rate of the RUA has been estimated at 5% per annum initially, resulting in a total population of approximately 50,000 by the year 1995. Thereafter, the net population growth is expected to slow down somewhat, to reach a total of 75,000 inhabitants by the year 2010. This is equivalent to an average annual population growth of about 3% between 1995 and 2010.

For the technical design of the water supply, waste water disposal and drainage systems, the project area has been divided into 18 districts (see Fig. 1.3.). A small survey on numbers of houses and population densities, carried out by Consultants in May/June 1988, was used to determine the population of each individual district, taking into account also the data obtained during earlier social surveys.

Population forecasts for the sub-areas, taking into account present and future land use, present population densities, etc., are given in Table 1.1. for the design horizons 1995 and 2010. This table shows that the total project area is about 610 ha, 20 ha of which is presently used as cemeteries. The average population density is expected to increase from about 57 persons/ha at present to 123 persons/ha in the year 2010.

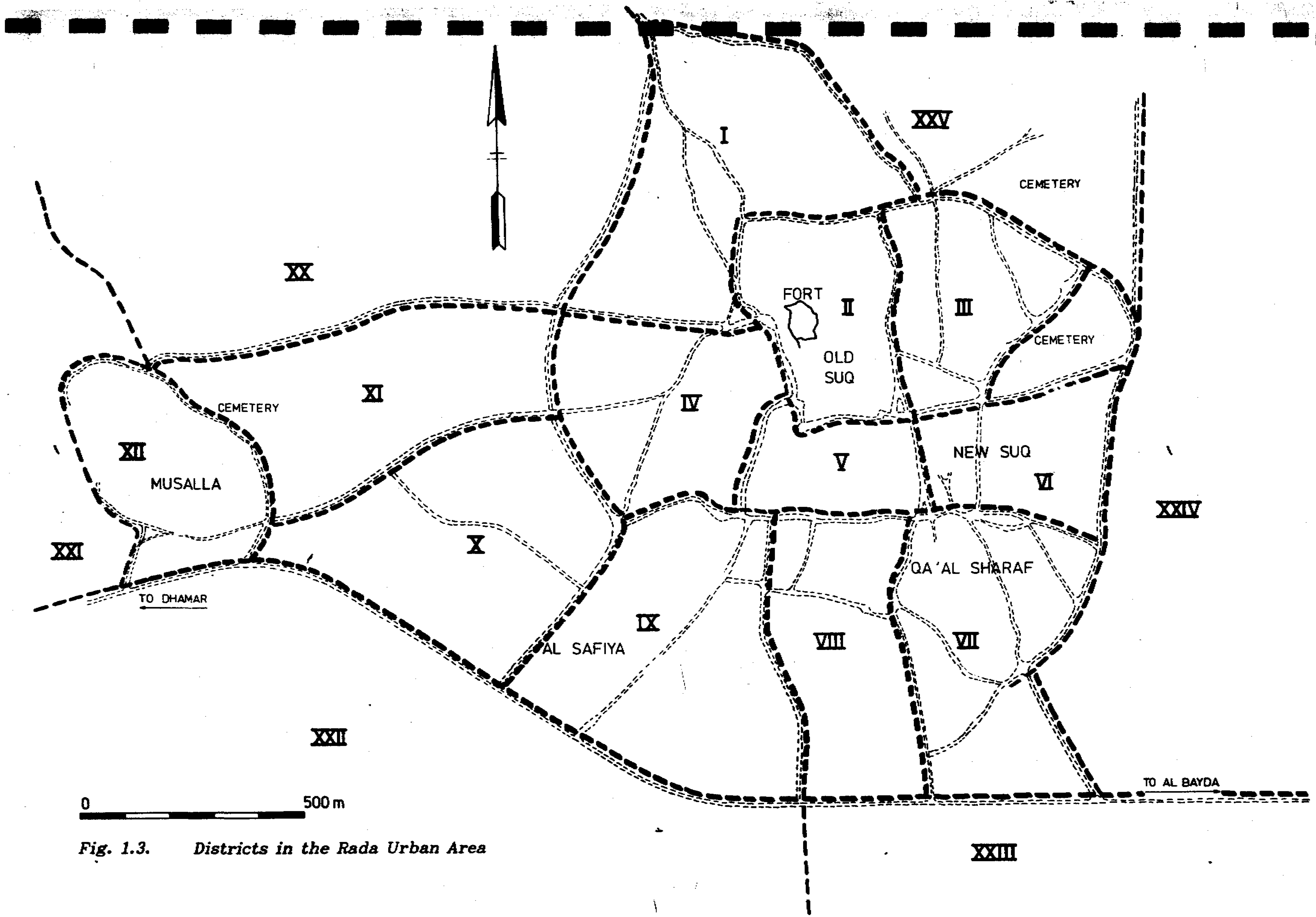


Fig. 1.3. Districts in the Rada Urban Area

Table 1.1. Population forecast per district

District No.	Surface [ha]	---1988--- Density [c/ha[1]]	Population	Growth [[2]]	---1995--- Density [c/ha]	Population	Growth	-------2010-------- Density [c/ha]	Population	Rounded
1	25.0	160	4,000	4.0%	211	5,264	1.5%	263	6,581	6,500
2	13.5	239	3,225	2.0%	274	3,705	0.5%	296	3,992	4,000
3	13.8	199	2,750	2.0%	229	3,159	0.5%	247	3,404	3,400
Cem.[3]	4.5	-	-	-	-	-	-	-	-	-
4	19.8	146	2,900	2.0%	168	3,331	0.5%	181	3,590	3,600
5	8.5	400	3,400	0.5%	414	3,521	0.2%	427	3,628	3,650
6	12.5	100	1,250	2.0%	115	1,436	0.5%	124	1,547	1,550
7	20.9	148	3,100	2.0%	170	3,561	0.5%	184	3,838	3,850
8	18.4	125	2,300	2.0%	144	2,642	1.0%	167	3,067	3,050
9	30.2	96	2,900	3.5%	122	3,690	2.0%	164	4,966	5,000
10	27.6	60	1,656	16.0%	170	4,680	1.0%	197	5,434	5,450
11	18.7	53	1,000	16.0%	151	2,826	2.5%	219	4,093	4,100
12	17.8	154	2,750	2.0%	177	3,159	0.5%	191	3,404	3,400
Cem.	7.6	-	-	-	-	-	-	-	-	-
20	102.3	10	1,000	12.0%	22	2,211	8.0%	69	7,013	7,000
21	13.3	11	150	12.0%	25	332	8.0%	79	1,052	1,050
22	86.5	9	800	12.0%	20	1,769	8.0%	65	5,610	5,600
23	44.2	11	500	12.0%	25	1,105	8.0%	79	3,506	3,600
24	102.0	12	1,250	12.0%	27	2,763	8.0%	86	8,766	8,800
25	16.4	12	200	12.0%	27	442	8.0%	86	1,403	1,400
Cem.	7.9	-	-	-	-	-	-	-	-	-
TOTAL	611.4		35,131			49,594			74,894	75,000
Average:		57			81			123		
Over entire project area:				5.0% (1988-1995)		49,433	3.5% (1988-2010)		74,882	

[1] c/ha = capita per hectare

[2] Population growth in percent per annum

[3] cemeteries

2. WATER SUPPLY

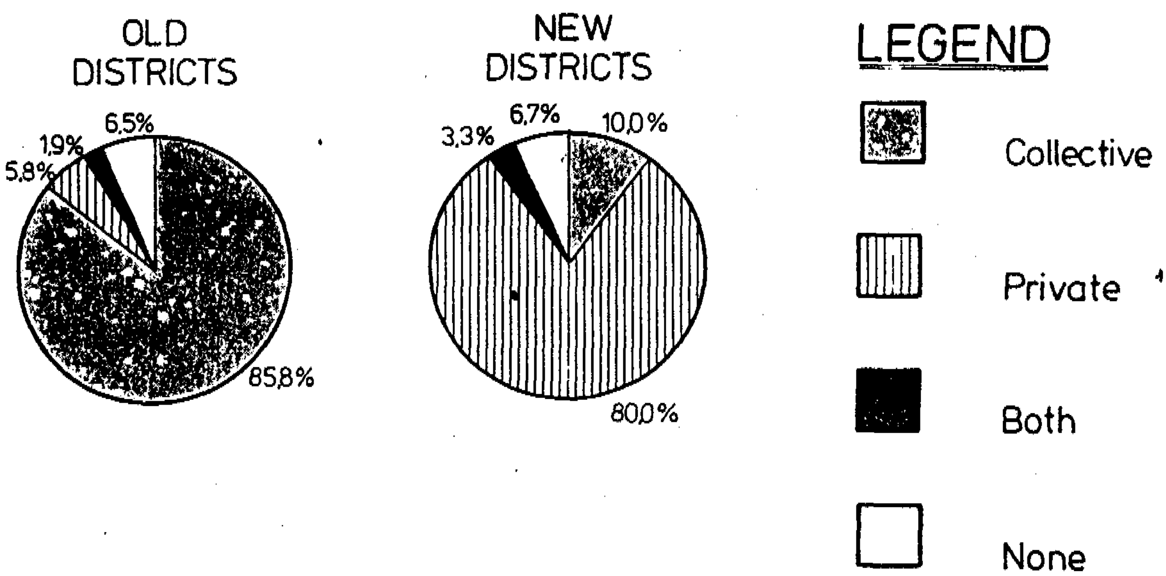
2.1. Present situation

2.1.1. General

Within the Rada Urban Area three larger and a few smaller water supply systems exist, each serving a section of the urban area. Together these systems serve about 93% of the houses in the town, by means of un-metered house connections. The largest system is cooperatively owned; the others are private.

There is a remarkable difference between the older[1] and the newer districts[2]: while in the older districts 86% of the houses have a connection to the existing collective water supply system, in the newer districts 80% of the houses obtain their water from private sources, generally boreholes. It is known that from these sources usually a larger and more regular supply can be obtained, though at higher cost. In older as well as newer districts there are houses with both sources of supply and a limited number with no connection at all. In the last case water is usually obtained from a neighbour or relative close by.

Details of the types of connections are given in Fig. 2.1.



[1] i.e. with most of the houses built before 1980, viz. districts 1 - 8 /, 12.

[2] i.e. where the majority of the houses was built after 1980, viz. districts 9 - 11.

Nearly all consumers[3] have a storage tank on the roof of the house, because the supply is intermittent due to the limited capacity of the systems (there are no storage facilities in these systems, so the pressure would fall during peak hours). In this respect no difference exists between the older and newer districts.

The water consumption is estimated to be around 45 - 50 l/person/day. There is a clear tendency, showing also in the number of taps and modern flush toilets in the newer districts as compared to the older ones, to use more and more water, particularly if it is available or may be expected to become so in the near future.

The consumer is charged a connection fee and a flat rate on a monthly basis. The rates are different for the various systems, the cooperative system charging a flat rate of YR 35 per month.

The systems are not mapped; neither are operational data such as quantity pumped or running hours of pumps recorded.

2.1.2. Water resources

The water for the systems is pumped from boreholes yielding up to 8 l/s each. The depth of the boreholes varies from 100 to 200 m, with static water levels being about 35 m below ground level. The boreholes are located within the RUA. Pumping equipment typically consists of a turbine pump driven by a diesel engine. Within a 3 km radius from the centre of the RUA the abstraction of groundwater for water supply purposes amounts to no more than 10% of the total groundwater abstraction, the bulk being used for irrigation.

Most water is drawn from volcanic rocks and sandstones, which form moderate to good aquifers in which water flows through cracks and fissures. Concentrated abstraction for agricultural purposes locally causes an ongoing lowering of the groundwater table, but on a regional basis recharge still exceeds abstraction, though the latter is increasing rapidly. Water samples from three wells in the RUA which were taken in July 1986 show an increase in conductivity from 1500 µS/cm in 1982 to 2400 µS/cm in 1986. Samples taken from the distribution system showed extremely poor bacteriological quality.

2.1.3. Distribution systems

Each supply system has its own distribution network, usually without reservoir. Larger pipes are generally buried, but connections are all above ground. The pipes are of galvanized iron, with diameters of 2" (50 mm) or 3" (75 mm) for the transportation and distribution mains, and ½" (13 mm) for the connections. Pipe materials used are limited to straight lengths, tees, elbows and gate valves.

The length of a house connection may be considerable, as it is common practice to serve a number of houses from one central point, with ½" connection pipes running to the individual houses.

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[3] 98% of all houses have a storage tank larger than 1 m^3, as shown by the survey carried out in February/March 1989 (see paragraph 2.1.4.)

The systems are operated for a limited number of hours per day, and supply to the various sub-areas within a system is on a rotation basis. This means that the mains are mostly without pressure.

Leakage is observed very regularly at present and the chances of contaminated water entering the pipelines are very high as the pipes often pass through pools of stagnant waste water.

2.1.4. Present water quality and use

In February/March 1989 a survey was carried out, covering 7.5% of the Rada population, to obtain detailed information on the present situation regarding the actual water supply conditions, water use and sanitation.

As was mentioned before, almost every house is equipped with its own storage tank. It can be concluded that the reliability of the collective supply system in particular is judged as quite poor. This is also the reason for many households to obtain additional water from other sources.

Table 2.1. gives the percentages of houses where the taste of the water is judged good (sweet) or poor (saline), combined with the percentages of houses where extra water is obtained from outside sources (usually water trucks), as well as the reasons given: too little or irregular supply. The results are also illustrated in Fig. 2.2.

Table 2.1. Perceived water quality and reasons for extra water (answers given by indicated % of interviewed persons)

District	Taste		Reasons for wanting extra water			Total %
	saline	sweet	quantity	quality	both	of district
1	90.7	9.3	34.3	65.7	----	81.4
2	63.6	36.4	37.9	48.3	13.8	82.9
3	15.4	84.6	85.7	----	14.3	72.4
4	56.7	43.3	40.0	12.0	48.0	80.6
5	30.3	69.7	88.5	----	11.5	74.3
6	30.0	70.0	54.5	9.1	36.5	84.6
7	55.2	44.8	22.2	44.4	33.3	54.5
8	13.6	86.4	64.3	21.4	14.3	56.0
9	30.0	70.0	40.0	33.3	26.7	48.4
10	13.3	86.7	50.0	16.7	33.3	33.3
11	20.0	80.0	60.0	20.0	20.0	45.5
12	----	100.0	95.2	----	4.8	72.4
Total	37.8	55.9	55.3	26.1	18.6	67.9
no opinion	6.3					32.1 no extra water
	100%			100%		100%

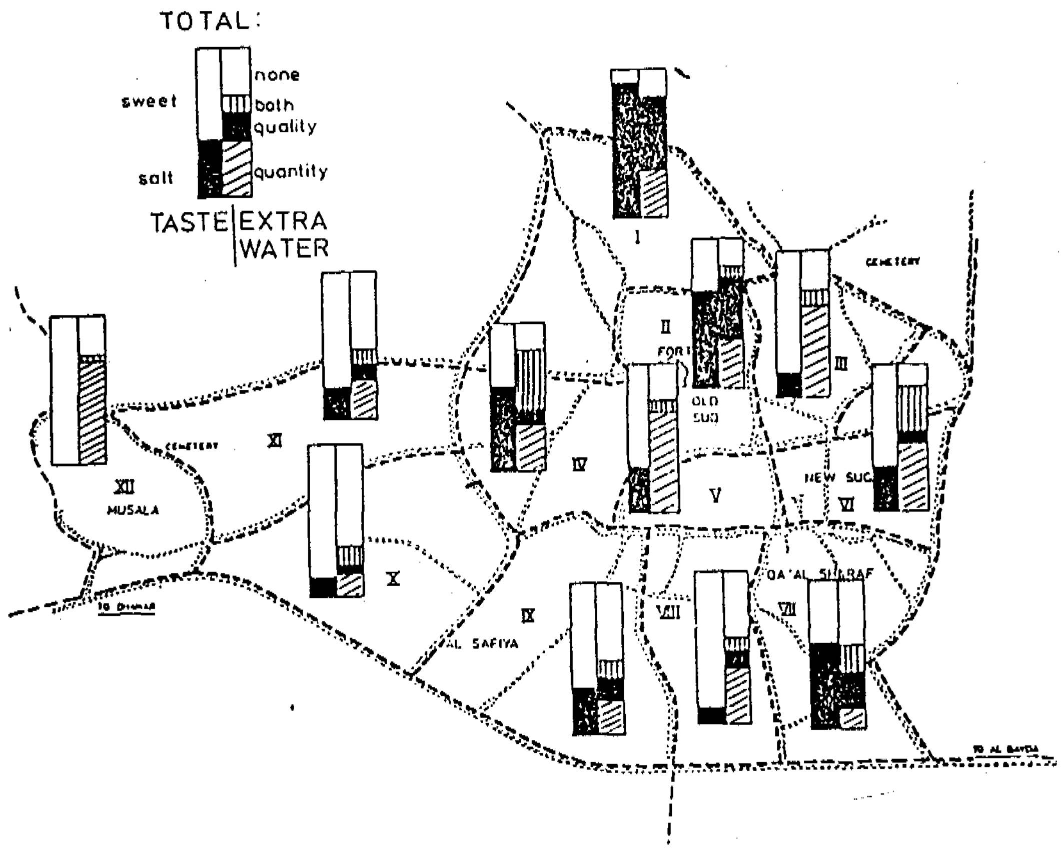

Fig. 2.2. Taste of water and extra supply, per district

It can be seen that the old districts 1, 2, 4 and 7, where most houses are connected to the collective system, score high on poor quality and give this as a reason for obtaining extra water (district 4 gives a low score on quality as sole reason but scores high on "both" as reason). Table 2.1 (right-most column) indicates that almost 70% of Rada's households want extra water from outside the existing supply systems, about 55% being because of shortages in the supply. Approximately 40% of the population find the taste of the water bad, i.e. too saline.

Table 2.2. gives an indication of the number of taps and washing machines reported during the survey. Indicated are the percentages of respondents that own the indicated numbers of taps/washing machines.

Table 2.2. Numbers of taps and washing machines in use

	0	1	2	3	4	>4	0-1	≥2
TAPS:								
Older districts:	16.9%	33.1%	21.2%	14.6%	5.8%	8.8%	50.0%	50.0%
Newer districts:	18.3%	16.4%	10.0%	15.0%	13.3%	26.7%	35.0%	65.0%
WASHING MACHINES:	15.0%	85.0%						

The difference between the older and newer districts in total number of houses connected by taps, though not very big, does show that in the newer areas the tendency towards more taps, thus in-house distribution systems, is clear. In the newer districts almost 80% of the toilets is of the mixed type (of which again about 2/3 of the cistern flush type), as compared with only 28% in the old districts, of which more than half is then of the pour flush type. All this indicates clearly the tendency towards higher water use in the newer houses and neighbourhoods. In the future this tendency will most certainly continue.

The percentage of households owning a washing machine appears to have gone up from approximately 40% in 1982[4] to 85% at present! As was the case then, hardly any difference presently exists between the old and the new houses. This too points towards increased water use in the future, when a new system will make this available.

2.2. Design criteria

2.2.1. General

The design of the new water supply system is based on criteria developed by NWSA[5], in line with the national policy on the economical use of scarce water resources.

Where no NWSA criteria exist, internationally accepted engineering criteria and standards are used, e.g. WHO's International Guidelines for Drinking Water, and ISO, AWWA, DIN and NEN standards.

2.2.2. Design horizon

The year 1995 has been taken as design horizon for the technical design of the water supply system. For such aspects as the availability of water sources and the capacity of the primary distribution network, which are important for further development of the proposed system after 1995, the year 2010 has been taken as the second design horizon.

2.2.3. System requirements

Because of the present intermittent supply, virtually all consumers have their own storage tank (ground tank and/or roof tank), the capacity of which generally is several times the daily consumption. Because of the risk of contamination of the water in these tanks, it is recommended that their use be discontinued once the new water supply system has become operational. Consequently, the new distribution system must be based on a 24 hours supply.

Reservoirs being very expensive components of any water supply system, a continuous 24 hours supply from the water sources into the distribution system has been selected. A -- relatively small -- reservoir is still required to level off the variations in demand over the day (see also para. 2.3.4).

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[4] RUDP Social and Economic Survey, Final Report, vol. 2.

[5] National Water and Sewerage Authority, Yemen Arab Republic

The net production capacity of the system will be based on the average demand during the maximum day ("maximum day demand"), with the required storage capacity determined by the demand fluctuations during that maximum day.

2.2.4. Pressures

For the design of the distribution network the following criteria have been taken into account regarding the required pressure:

Elevation of the tap above street level (4-storey building):	13.5 m
Minimum required pressure on tap, during peak hour:	5.0 m
Losses in house connection and in-house:	1.5 m
Minimum pressure in reticulation system:	20.0 m[6]

Maximum pressures will occur during the night when the demand is at a minimum. In the main part of the distribution system this maximum pressure should not exceed 5 bar[7] to prevent too much leakage. In that way also leaking taps and under-registration by water meters can be minimized.

2.2.5. Levels

Levels have been taken from the available topographical maps and corroborated by extensive field surveys. All levels are indicated relative to the 2100 m contour line. The most important levels are indicated in Table 2.3.

Table 2.3. Surface levels in project area

Location	Level [m][8]
Top of Musalla	59
Surroundings of Musalla	38
Town centre	28
Fort reservoir	65
Eastern part of project area	24
Highest point of Qu Ash Sharaf	30
Highest point of Harat Al Qana	50

Table 2.3. shows that within the project area the supply levels vary between 24 and 59 m, contributing to a difference in pressure of 35 m!

[6] above street level

[7] or: about 50 m water column

[8] relative to contour line of 2100 m + MSL

2.2.6. Pipe materials

The selection of materials for the distribution system has been based on a long lifetime, long-term experience with the material, and on the standards mentioned in paragraph 2.2.1.

For the primary distribution network (diameters 200 - 300 mm) ductile iron has been chosen because it is a sturdy material with a very long service life. No service connections will be made to these pipes.

For the reticulation system (diameters 25 - 90 mm) a sturdy but flexible material is required due to the erratic alignment and often unknown obstacles (rocky outcrops). House connections have to be made while the system is under pressure. High-density polyethylene (HDPE) is a suitable material for this purpose, and used all over the world. Construction is flexible and the mechanical joints are easy to install.

2.3. Water requirements

2.3.1. Population supplied

As is mentioned in paragraph 1.3. the population within the RUA is expected to grow from an estimated 35,000 in 1988 to approximately 50,000 in 1995 (phase 1) and 75,000 in the year 2010 (phase 2).

The new system will be laid out to supply all inhabitants with water. Initially, however, it may not yet be possible to connect all inhabitants. It has been assumed, therefore, that in 1995 95% of the inhabitants (or: 47,500 people) will be connected, whereas in the year 2010 all 75,000 inhabitants are expected to be supplied from the new system.

2.3.2. Water demand

On the basis of the results of previous studies, and in line with the national policy on the economical use of scarce water resources, NWSA has set the following water demand criteria for secondary towns in the highlands:

Year	Water demand [l/c/d[9]]
1990	60
1995	72
2005	86
2010	100

These figures represent per capita water demands on the so-called maximum day, inclusive of un-allocated water (see paragraph 2.3.3.) and minor non-residential demands (shops, small restaurants, etc.).

[9] litres per capita and per day

For large non-domestic consumers, however, 10% will be added to the demand thus calculated; this additional demand (10%) includes also a 20% unaccounted for (see Table 2.5.).

2.3.3. Unaccounted for water

The new water supply system will not include any treatment other than a safety chlorination. Consequently no treatment losses will occur.

Leakage from the transmission and distribution mains and house connections will be the main source of unaccounted for water (UFW). Additional losses will occur by under-registration of water meters, non-registered connections, flushing of the distribution network, fire fighting, etc.

Similar to other secondary towns in the YAR a provision equal to 20% of the maximum day demand has been made for unaccounted for water.

The volume of unaccounted for water is more or less independent of actual water consumption (although at moments of high demand the use of water through non-registered connections increases, leakage goes down, as the water pressure falls). The allowance for unaccounted for water, both under conditions of average daily demand and peak hour demand, is thus taken equal to that for maximum day demand (see Table 2.5.).

2.3.4. Fluctuations in demand

a. Maximum day consumption

The total daily consumption depends on several factors, such as the day of the week, temperature, special events, etc. The actual maximum day consumption under present water supply conditions in Rada cannot be determined because the supply does not satisfy the current demand.

For the secondary towns a coefficient of 1.2 has been adopted to calculate the maximum day consumption.

b. Peak consumption

Due to fluctuations during the day peaks in consumption will occur. Generally a coefficient of 1.5 to 2.5 times the average hourly consumption is applied to calculate the maximum hour consumption.
For Rada the assumed fluctuation of the consumption during the day, compared with the average hourly consumption, is given in Table 2.4. and illustrated by Fig. 2.3.

The figure shows that the consumption on the maximum hour is two times the average hourly consumption.

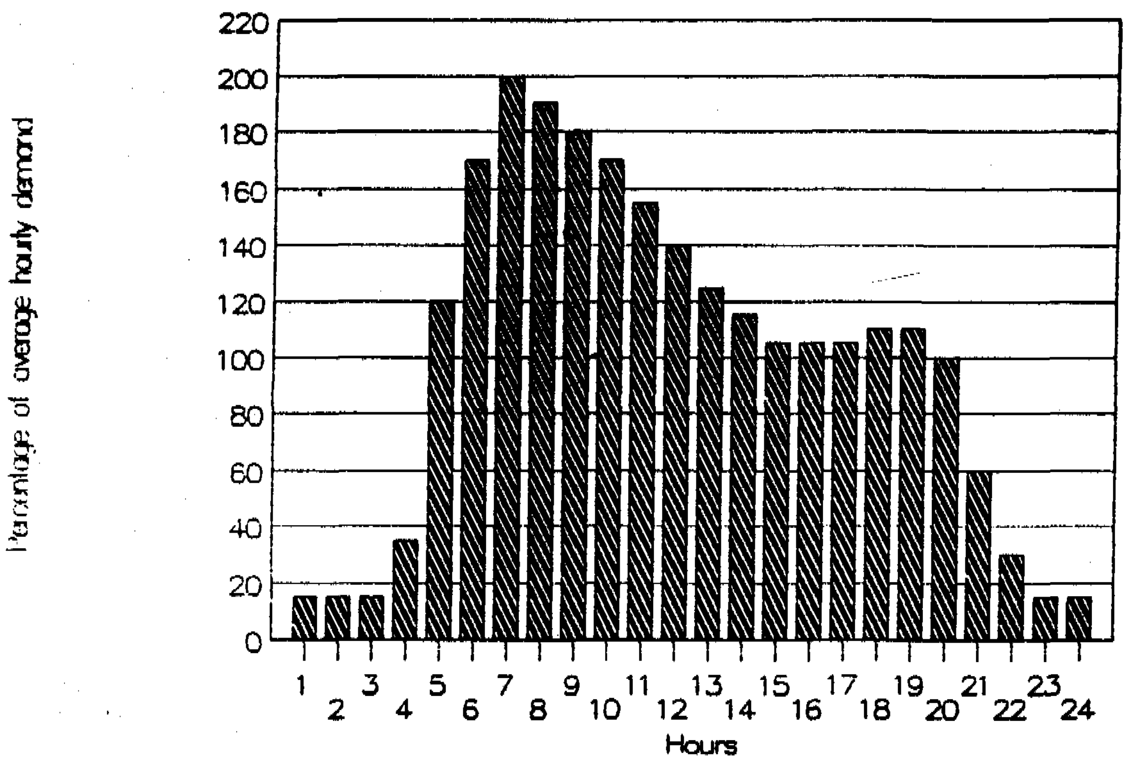

Fig. 2.3. **Hourly demand fluctuation**

Table 2.4. Hourly demand fluctuations
(in % of average hour consumption)

Hour	Consumption	Storage Outflow	Storage Inflow
1	15		85
2	15		85
3	15		85
4	35		65
5	120	20	
6	170	70	
7	200	100	
8	190	90	
9	180	80	
10	170	70	
11	155	55	
12	140	40	
13	125	25	
14	115	15	
15	105	5	
16	105	5	
17	105	5	
18	110	10	
19	110	10	
20	100		0
21	60		40
22	30		70
23	15		85
24	15		85
	100.0	25.0	25.0

2.3.5. Recapitulation

The water demands for the first and second phases (50,000 and 75,000 inhabitants, respectively, corresponding with the years 1995 and 2010) are summarized in Table 2.5., whereas water demands per district (situation 2010) are shown in Table 2.6. Since storage will be used to level out variations in demand during the day, net water production will be based on the maximum day demand, i.e.:

(36.3 + 0.2 x 36.3) = 43.6 l/s for the first phase (year 1995)
(80.2 + 0.2 x 80.2) = 96.3 l/s for the second phase (year 2010)

Table 2.5. Summary of water demand

Year:	1995			2010		
Demands in l/c/day	Average demand	Maximum day	Maximum hour	Average demand	Maximum day	Maximum hour
Residential consumption	50.0	60.0	100.0	70.0	84.0	140.0
Large consumers (10%)	5.0	6.0	10.0	7.0	8.4	14.0
Subtotal	55.0	66.0	110.0	77.0	92.4	154.0
Unaccounted for water = 20% of max. day	13.2	13.2	13.2	18.5	18.5	18.5
Total demand	68.2	79.2	123.2	95.5	112.9	172.5
Population		50,000			75,000	
Percentage served		95%			100%	
Population served		47,500			75,000	

	[m³/day]	[m³/h]	[l/s]	[m³/day]	[m³/h]	[l/s]
Average total consumption	2,613	109	30.2	5,775	241	66.8
Max. day consumption	3,135	131	36.3	6,930	289	80.2
Unaccounted for water (20% of consumption on maximum day)	627	26	7.3	1,386	58	16.0
Max. day demand (= production capacity)	3,762	157	43.6	8,316	347	96.3
Max. hour (= distribution capacity)		244	67.7		539	150.0
Storage capacity [m³] (25% of consumption on max. day)	784			1,733		

The water demand for the various districts of the project area is given in Table 2.6:

Table 2.6. Water demand per district in the year 2010

District No.	[ha]	Population	Average consumption [m³/h]	Consumption max.day [m³/h]	Leakage [m³/h]	Demand max. day [m³/h]	Peak demand [m³/h]
1	25.0	6,500	20.9	25.0	5.0	30.0	46.7
2	13.5	4,000	12.8	15.4	3.1	18.5	28.7
3	13.8	3,400	10.9	13.1	2.6	15.7	24.4
4	19.8	3,600	11.6	13.9	2.8	16.6	25.9
5	8.5	3,650	11.7	14.1	2.8	16.9	26.2
6	12.5	1,550	5.0	6.0	1.2	7.2	11.1
7	20.9	3,850	12.4	14.8	3.0	17.8	27.7
8	18.4	3,050	9.8	11.7	2.3	14.1	21.9
9	30.2	5,000	16.0	19.3	3.9	23.1	35.9
10	27.6	5,450	17.5	21.0	4.2	25.2	39.2
11	18.7	4,100	13.2	15.8	3.2	18.9	29.5
12	17.8	3,400	10.9	13.1	2.6	15.7	24.4
20	102.3	7,000	22.5	27.0	5.4	32.3	50.3
21	13.3	1,050	3.4	4.0	0.8	4.9	7.5
22	86.5	5,600	18.0	21.6	4.3	25.9	40.2
23	44.2	3,600	11.6	13.9	2.8	16.6	25.9
24	102.0	8,800	28.2	33.9	6.8	40.7	63.2
25	16.4	1,400	4.5	5.4	1.1	6.5	10.1
Cem.[10]	20.0	-	-	-	-	-	-
Total:	611.4	75,000	240.6	288.8	57.8	346.5	539.0
				ditto, in [l/s]:		96	150

2.4. Water resources

2.4.1. Introduction

Rada town is situated approximately in the middle of a set of interconnected plains surrounded by hills and mountains, many of which are of volcanic origin. The plains are underlain by alluvium up to 20 m thick. Cretaceous sandstones and Tertiary and Quaternary volcanic rocks form the surrounding hills and mountains and the hard rock floor of the plains underneath the alluvium. Tectonically the Rada plains are situated near the eastern border of the Red Sea Graben characterized by a number of large step faults running northwest-southeast, which greatly influence the geology of the area.

The main aquifers are formed by the Cretaceous Tawilah sandstone in the east and north of the Rada catchment and the overlying Tertiary and Quaternary volcanic rocks in the south and west of the area. Groundwater in the Rada plains catchment is abstracted at a rate of approximately 20 million m^3/year from about 600 dug wells and more than 200 boreholes. Wells are especially concentrated west and south of the town. More than 90% of the water is used for irrigation.

[10] cemeteries

The still increasing groundwater abstraction from wells has caused a decline of the water level in many places in the Rada plains. Groundwater recharge occurs via a northwest-southeast groundwater flow, and via the infiltration of rain and flood water in the plains after the occasional heavy rainstorm.

Springs can be found at several locations in the surroundings of Rada. The spring water originates from the tertiary and quaternary volcanic series. Most of the springs are not perennial, however, though some permanent springs can be found.

Surface water is seldom found on the Arabian peninsula. Perennial flows mostly originate from small springs. In the vicinity of Rada no feasible streams are found.

For the Rada water supply system medium to deep groundwater is the only realistic option.

2.4.2. Selection of well field location

Due to the poor quality of the water and the declining water table caused by heavy pumping for irrigation a new area had to be found and new wells drilled for the abstraction of more and better quality groundwater for the new drinking water supply system of the Rada Urban Area.

Two potential locations for well fields can be indicated near Rada, north and south of the town, respectively. On the basis of a groundwater study carried out by RIRDP[11] in 1983/84 the northern site has been selected. Representatives of the LCCD[12] indicated, moreover, that this location is available for the exploitation of potable water. Detailed background information on the selection of the northern location is given elsewhere[13].

The area has a relatively good recharge of the groundwater due to groundwater inflow from the large slopes of Jebel Isbil where recharge from rainfall is high and groundwater quality is good. The Tawilah sandstone formation is the main aquifer in this area. Permeability of the sandstone itself is low but high when fractured. Faults and intrusive dikes are the main fracture zones forming highly permeable conduits, while the sandstone forms the groundwater reservoir.
Wells in the area show moderate to high yields, especially from fracture zones.

The site selection survey contained an aerial photographic study, followed by an electro-magnetic survey. The anomalies in the electric conductivity of the sub-soil measured by this method give a good indication as well as the exact position of the water-bearing fracture zones related to faults or intrusive dikes. During the electro-magnetic survey 18 suitable sites on fracture zones and dikes have been indicated.

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[11] Rada Integrated Rural Development Project

[12] Local Council for Community Development

[13] Report on the site selection for drilling boreholes for the water supply of Rada Urban Area, Rada' Water Supply and Sanitation Project, November 1988.

Geo-electrical sounding indicates only a moderate permeability of the sandstone. Due to the required spacing between the wells these sites are sufficient for 10 production wells.

2.4.3. Groundwater quality

Water samples from wells of the villages Ghawl Adh Dhra, An Nazim, As Salil Al Ala and Qusayr, all situated nearby the projected well field, have been analyzed between April 1986 and 1988. The lower and upper limits of a number of chemical substances as found there are shown in Table 2.7.

The chemical values presented in Table 2.7 show that the quality of the groundwater found near the projected well sites is usually within the standards named 'acceptable' by the WHO. For that reason it is assumed that no treatment of the water other than a safety chlorination will be required. The electric conductivity (salinity) of the groundwater, as shown in Fig. 2.4. also shows that the selected well field site is favourable from a point of water quality (relatively low salinity).

Table 2.7. Chemical standards set by the WHO and concentrations in groundwater nearby the projected well field[14]

Substances	lower limit of results mg/l	upper limit of results mg/l	WHO max. acceptable level mg/l	WHO max. allowable level mg/l
Fluoride (F)	0.5	1.82	1.5	10
Nitrate (NO_3)	9.5	13.5	45	100
Iron (Fe)	0.02	0.03	0.3	0.5
Magnesium (Mg)[15]	1	31	50	150
Manganese (Mn)	0	0.25	0.1	0.5
Calcium (Ca)[15]	60	92	75	200
Tot Hardn. ($CaCO_3$)	154	328		500
Sulphate (SO_4)	24	57	400	1000
Chloride (Cl)	95	165	250	500
Sodium (Na)	44	104	200	500
pH range	7.5	8.9	6.5-8.5	

[14] The standards for most of the chemical substances listed above are for the taste (or preservation of building materials) and not for health. Only the standards for fluoride and nitrate are for health.

[15] Since the last few years the WHO no longer gives standards for calcium and magnesium.

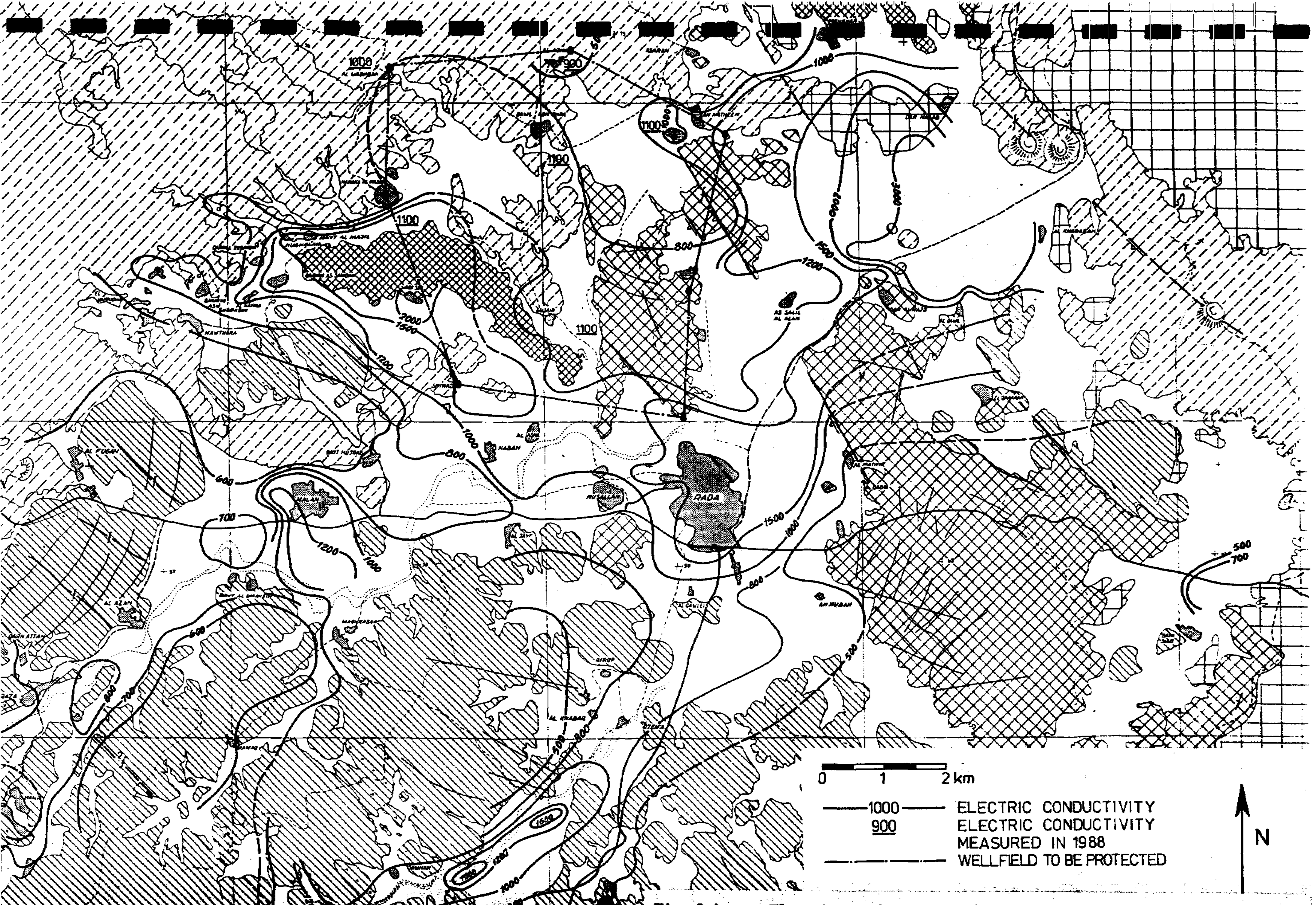


Fig. 2.4. Electric conductivity of the groundwater in the Rada Basin

Part of chemicals dissolved in the water originate from:
- volcanic rock;
- uncontrolled waste dumping polluting the water;
- increasing use of pesticides and fertilizer (especially for qat) endangering its suitability for human consumption.

Although water found might first appear to agree to standards set, it is possible that the quality deteriorates in time as water from deep aquifers (expected to contain a higher concentration of salt) is extracted. For this reason it may be necessary to limit the abstraction from one area and to look for well sites in other areas in the future.

Based on the information contained in Table 2.7. groundwater from the projected well field seems unlikely to be aggressive. However, free CO_2, of great importance in this respect, has never been analyzed. This should be done on water samples collected during the drilling operations.

2.5. Description of the new water supply system

The general lay-out of the new water supply system for the Rada Urban Area is shown on the background of the geohydrological map; see Fig. 2.5.

Water from a series of deep wells, situated in the well field between An Nazim, Qawl Adh Dhra and Al Qusayr (about 5 km north of Rada town) is pumped, through a collector-cum-transmission main, to a reservoir located approximately 800 m north of Qaryat Al Habadah. Originally a reservoir site 750 m north of the wadi crossing near Harat Al Qana had been foreseen[16], but for a number of reasons a new site has been selected[17].

The reservoir will be situated on a compound that also may house the generator house, chlorination building and operation and maintenance building. Another location for this compound, nearer to the wellfield has however NWSA's preference. As was mentioned before, the groundwater quality is expected to be such that no treatment other than a safety chlorination will be required.

[16] see Inception Report, July 1988, page 3 - 9

[17] Reasons for shifting the reservoir location are:
- the accessibility of the original site is very poor, requiring extensive improvement and road construction works, and increasing the unit cost of the construction works to be carried out there;
- the site elevation is higher than necessary, increasing pumping cost as well as the danger of leakage in the distribution system;
- room for the other buildings on the compound would not be available at the reservoir site, but only at a lower level, in agricultural land. This would imply protracted land acquisition.

The new location, - if compared with the original one as mentioned in the Inception Report - does mean a more eccentrical feeding of the distribution network, however, as well as extending the collector/transmission main by about 400 m.

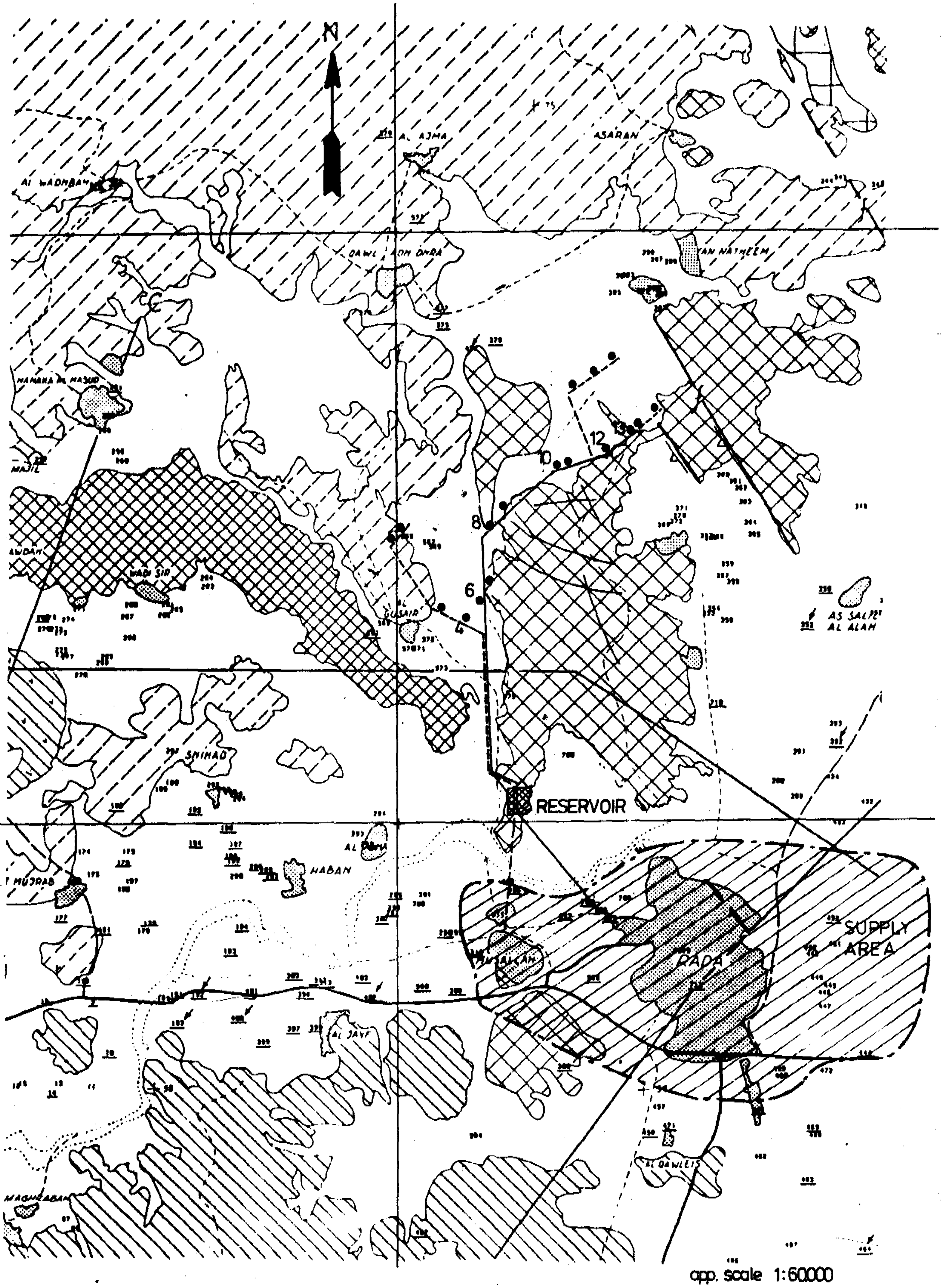


Fig. 2.5 General lay-out of water supply system on background of geohydrological map

Disinfection of the water will take place by adding gaseous chlorine before the water enters the reservoir, thus providing the required detention time in the reservoir itself.

From the reservoir the water will flow to the town by gravity. In the town it is distributed by means of a primary distribution network that will eventually be looped[18], but initially constructed as a branched system, to limit investments during the first phase. All secondary systems - except in the outlying districts (XX thru XXV) and in districts I, II and XI - will be looped, however.

All connections will be provided with house water meters. For fire fighting and flushing purposes, fire hydrants will be provided.

2.6. Well field

2.6.1. Lay-out

The well field is located approximately 800 m north of Qaryat Al Habadah, the northwestern-most part of the Rada Urban Area, as shown in Fig. 2.5. A diagrammatic representation of the well field is given in Fig. 2.6.

The required source capacity is 44 l/s for the first phase (corresponding with the year 1995) and 96 l/s for the second phase. With an expected average long-term capacity of 10 l/s and some spare units, six deepwells will be required in phase 1 and a minimum of eleven in phase 2.
For the first phase wells 4, 6, 8, 10, 12 and 13 will be developed. For the second phase wells 1, 2, 3, 15, 16 and 18 can be added to the system.

2.6.2. Deepwell details

Typical deepwell details are given in Figs. 2.7. and 2.8. The depth of the finished deepwells is assumed to be around 150 m, with borehole drilling expected to continue up to 200 - 250 m, depending on the actual conditions met. The diameter of the deepwell is 16" (ø 400 mm) over the first 18 m, the remainder having a diameter of 12" (ø 300 mm). As shown in Fig. 2.7. the top of the deepwell will be provided with a 14" (ø 350 mm) conductor casing over approximately 15 m, whereas the actual casing pipe/well screen will have a diameter of 8" (ø 200 mm). The annular openings between the conductor casing and the soil, and between the conductor casing and the regular casing will be filled with cement grouting. A gravel pack will be applied up to 10 m above groundwater level, and covered by a sand plug of 2 m and a clay plug of 5 m thickness. Each deepwell will be provided with a piezometer guiding pipe.
In Fig. 2.8 a possible solution for the housing of the deepwells is shown. However preference is given to built the well and pump just outside the pumphouse building, - although properly protected -, because of facilitating repairs and regular operation and maintenance. Pumphouses will be provided with proper walkways.

[18] in the 2010 situation (phase 2)

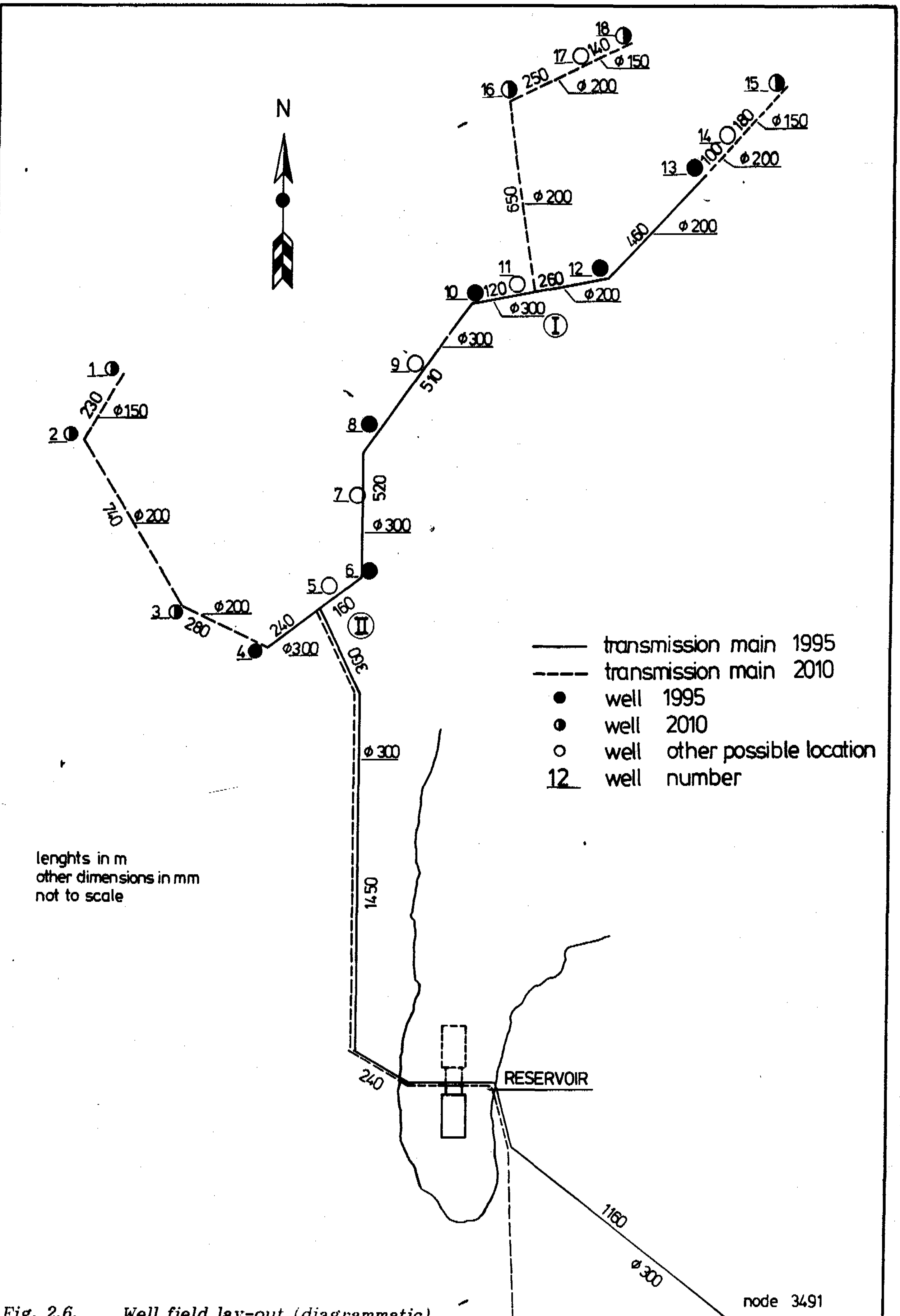


Fig. 2.6. Well field lay-out (diagrammatic)

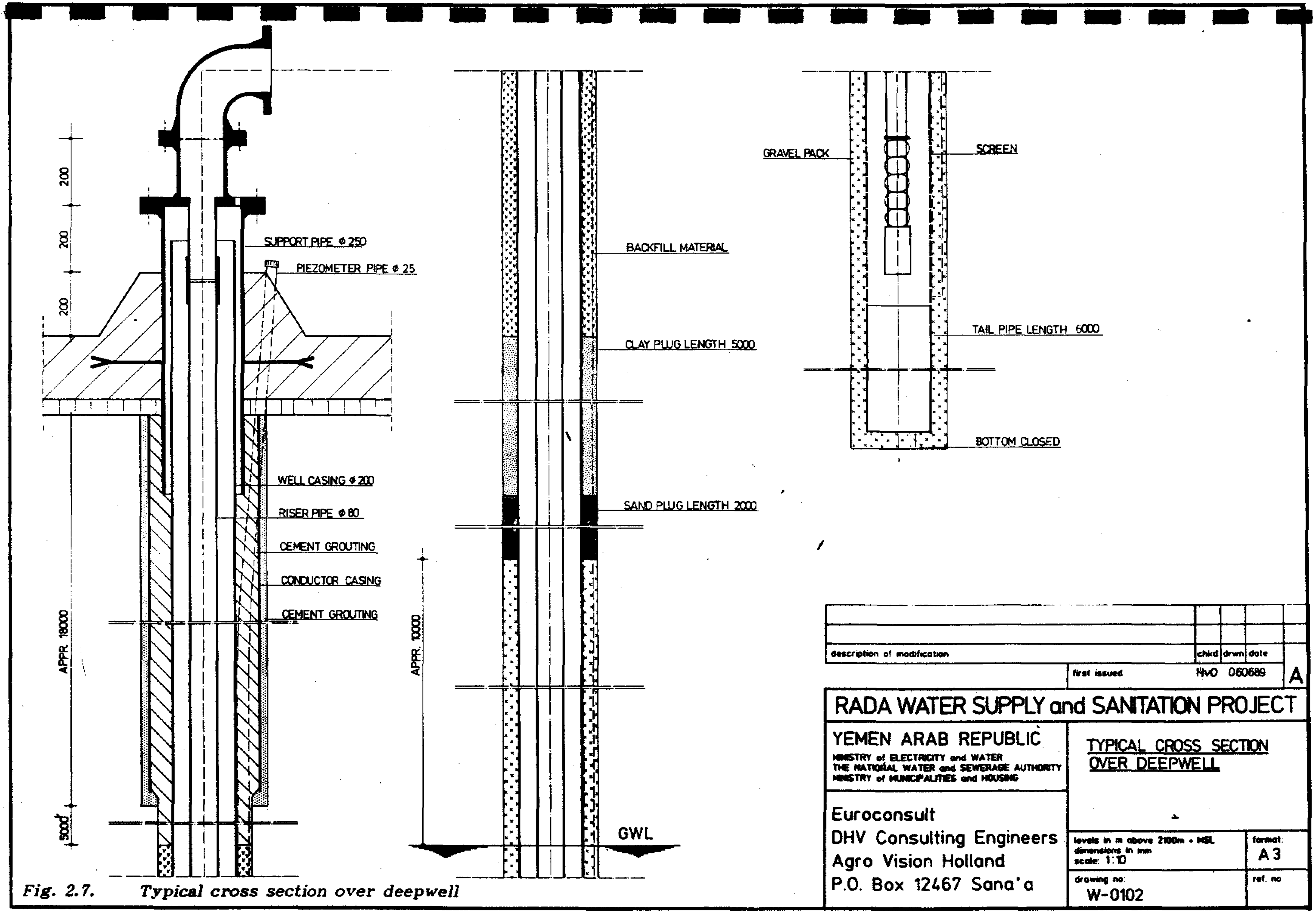


Fig. 2.7. Typical cross section over deepwell

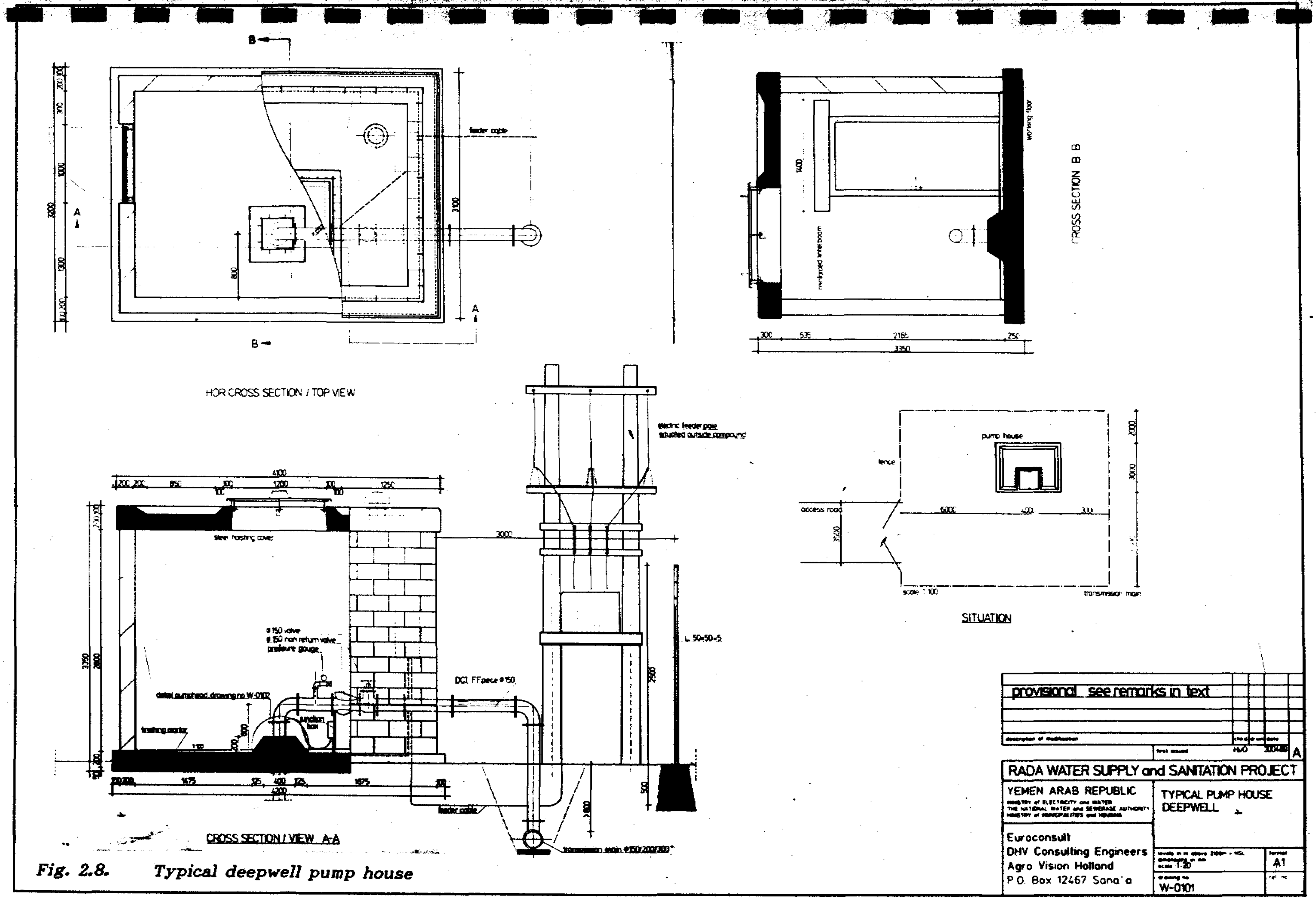


Fig. 2.8. Typical deepwell pump house

Each deepwell is provided with an approximately 18.5 kW multi-stage submersible pump of 150 mm diameter, with a capacity of 10 l/s at a head of between 103 and 116 m, as indicated in Table 2.8.[19]

Table 2.8. Calculation of deepwell pumps (provisional)

Design horizon:	1995	2010
No. of pumps (effective)	5	10
Total capacity in l/s (m^3/h)	50 (180)	100 (360)
Velocity in transmission main [m/s]	0.4	0.8
Losses [mwc/100 m]	0.05	0.2
[mwc/1500 m]	0.75	3.0
Elevation reservoir level (maximum)	2173.50	2173.50
Estimated static water level in boreholes	2080.00	2080.00
Difference [mwc]	93.50	93.50
Estimated draw-down [mwc] when pumps are in operation	8.50	10.50
Total static head [mwc]	102.00	104.00
Dynamic losses (range, [mwc])	1 - 3	4 - 12
(average, [mwc])	2	8
Total head (range)	103 - 105	108 - 116
Total head (average)	104	112

Each deepwell is provided with a pressure indicator (manometer), non-return valve, gate valve, wash-out and sampling point. In addition, there is a hose connection which can be used for local water supply for cleaning purposes, sampling or air release. Near deepwell 4 a hydrant will be installed, to allow flushing of the branch transmission main. Details of the possible deepwell pump houses are given in Fig. 2.8.(see also remarks earlier in this paragraph).

2.6.3. Operation and power supply

The deepwell pumps are operated manually, on the water level in the storage reservoir at the compound site. To protect the pumps against dry running a thermal overload protection (thermistor) is applied for each pump. Details are given on the P&ID[20] for the well field and reservoir compound in Fig. 2.11.

[19] As neither the static groundwater level nor the exact draw-down at an abstraction of 10 l/s are known at this stage, the pumping heads given are approximate only.

[20] piping and instrumentation diagram

Power will be obtained from the utilities compound by means of a 11 kV overhead power line (see paragraph 2.11). Taking into account the distance between the boreholes (with a minimum of 500 m), each deepwell will be provided with a pole-mounted step-down transformer (11 kV/0.4 kV), power capacity about 50 kVA. Because of its low weight, aluminium wire is preferred for the overhead line. The minimum available size is 50 mm^2, which gives no limitations to the number of deepwell pumps that can be operated simultaneously.

2.7. Transmission mains system

2.7.1. Lay-out

Parallel to the line of deepwells a collector-cum-transmission main will be constructed to collect water from the deepwells, as shown in Fig. 2.6. For the first phase (1995 situation) wells 4, 6, 8, 10, 12 and 13 will be developed. For the second phase (year 2010) the wells 1, 2, 3, 15, 16 and 18 can be added to the system.

The first part of the transmission main to be constructed for the first phase will be a main from deepwell 13 to the reservoir, with a short branch from deepwell No. 4. For the second phase it is envisaged to extend the branch to deepwell No. 4, to connect deepwells Nos. 3, 2 and 1, while extending the main line to deepwell No. 15. In addition, a new branch would have to be constructed to deepwells Nos. 16 and 18.

Based on the experience gained during the first phase, decisions will be made as to how many wells (and at which locations) will actually be developed in the future. Phasing the construction of the transmission main system would thus be advantageous, as it provides more flexibility regarding future extensions of the well field and the corresponding transmission main capacity (see paragraph 2.7.2.).

In the connection main (ø 150 mm) between the deepwells and the transmission main non-return valves and gate valves will be installed. To reduce the investment cost, no water meters will be installed there, however. If necessary, flow measuring can be done by means of a portable flow meter.

In the transmission main 4 non-return valves will be installed, at the location of junctions I and II, to prevent back-flow in case of a pipe burst. They can also be used for maintenance of a downstream pipe section. Typical details of these junctions are shown in Annex A, Fig. A.20.

2.7.2. Hydraulic calculations

Hydraulic calculations for the transmission main system are given in Annex A1. For the ultimate situation (phase 2) the diameter of the transmission main between junction II and the reservoir (distance 1500 m) should be 400 mm. For the 1995 situation, however, a diameter of 300 mm would be sufficient. Rather than directly installing a ø 400 mm main, sufficient for the ultimate situation, it is thus possible to lay a ø 300 mm main directly, to add another ø 300 mm main for the second phase. Both alternatives have been worked out in Annex A1. The average heads on the deepwell pumps, and consequently the energy costs, would be the same in either case.

In case of phasing the transmission main between junction II and the reservoir the total investment cost would be more than installing one ø 400 mm main directly, but would allow half of the pipe procurement costs to be postponed, a solution that is preferred by NWSA as well.

Because phased implementation also gives a greater flexibility regarding the extension of the well field, while limiting the maximum pipe size for both the transmission system and the primary distribution system to 300 mm, the detailed engineering design of the transmission main is based on a phased approach: one ø 300 mm main between junction II and the reservoir for the first phase, and another one for the second phase.

The collector/transmission main system will be made of ductile iron, of the following lengths and diameters:

Diameter [mm]	Length [m] laid for Phase 1	Phase 2	Total [m]
150	60	580	640
200	720	2020	2740
300	4760	1700	6460
Total:	5540	4300	9840

2.8. Reservoir compound

2.8.1. Lay-out

The lay-out of the possible compound at the reservoir site is shown in Fig. 2.9. In addition to the storage reservoir, the site also contains an operation and maintenance building, generator house, chlorination building and storage shed, while providing room for pipe storage. The operation and maintenance building lay-out should be as simple as possible. Final details on lay-out and materials will be dealt with in the tender drawing, to be approved by NWSA.

The chlorination building, storage reservoir and generator house will be described in more detail in the following paragraphs.

The proposed operation and maintenance building is shown in Fig. 2.10. NWSA have a preference to built the utility and operation and maintenance building, i.e. all buildings except the reservoir, on groundlevel on a compound nearer to the well sites. There is no objection, neither technical nor financial, to do so, if the land can be acquired. It has the following main components:

- operation room
- office
- drawing office
- engineering office
- water laboratory
- main store
- meter workshop
- valve workshop
- welding workshop
- car workshop

All workshops will be equipped with the relevant equipment and tools.

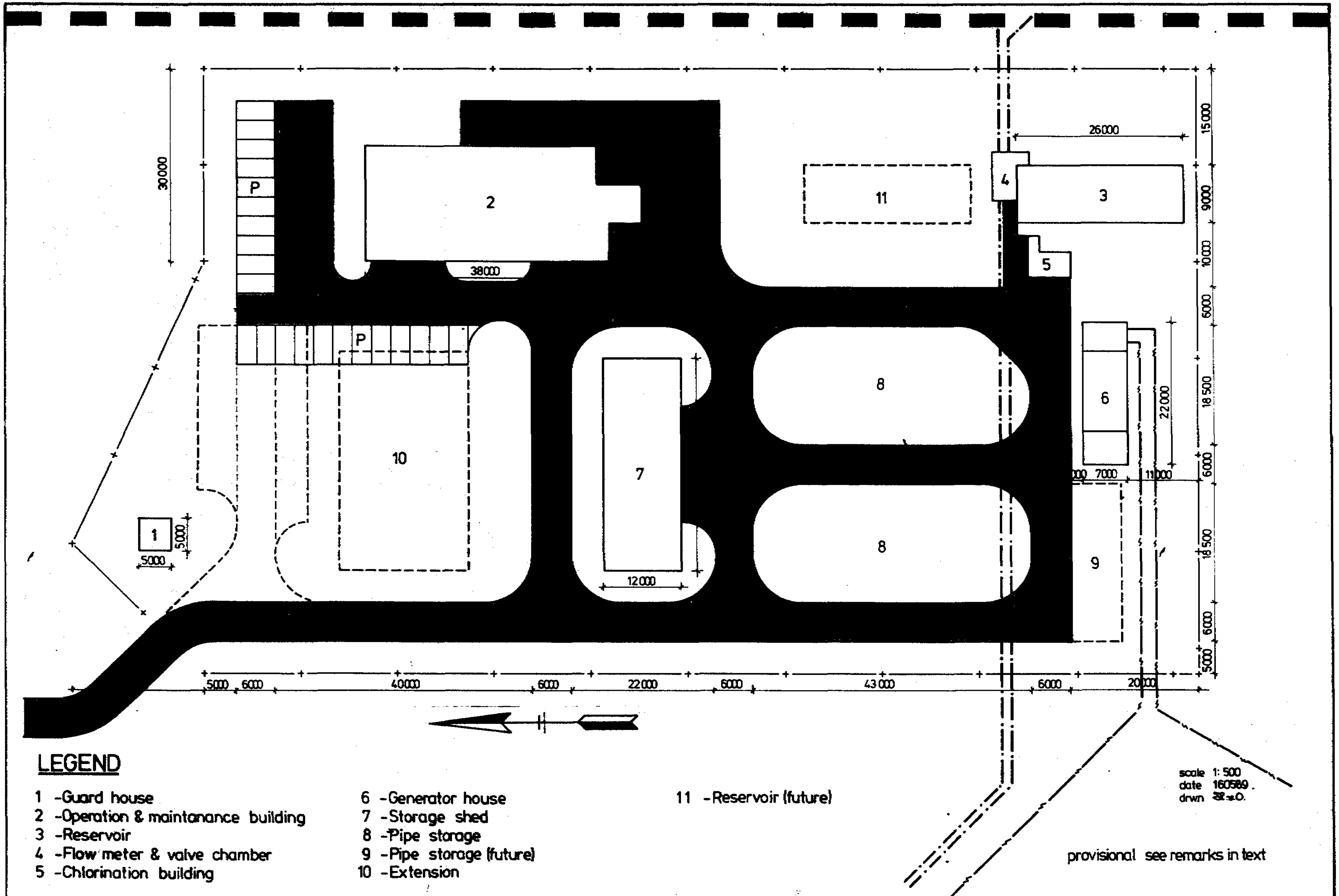


Fig. 2.9. *Lay-out of reservoir site*

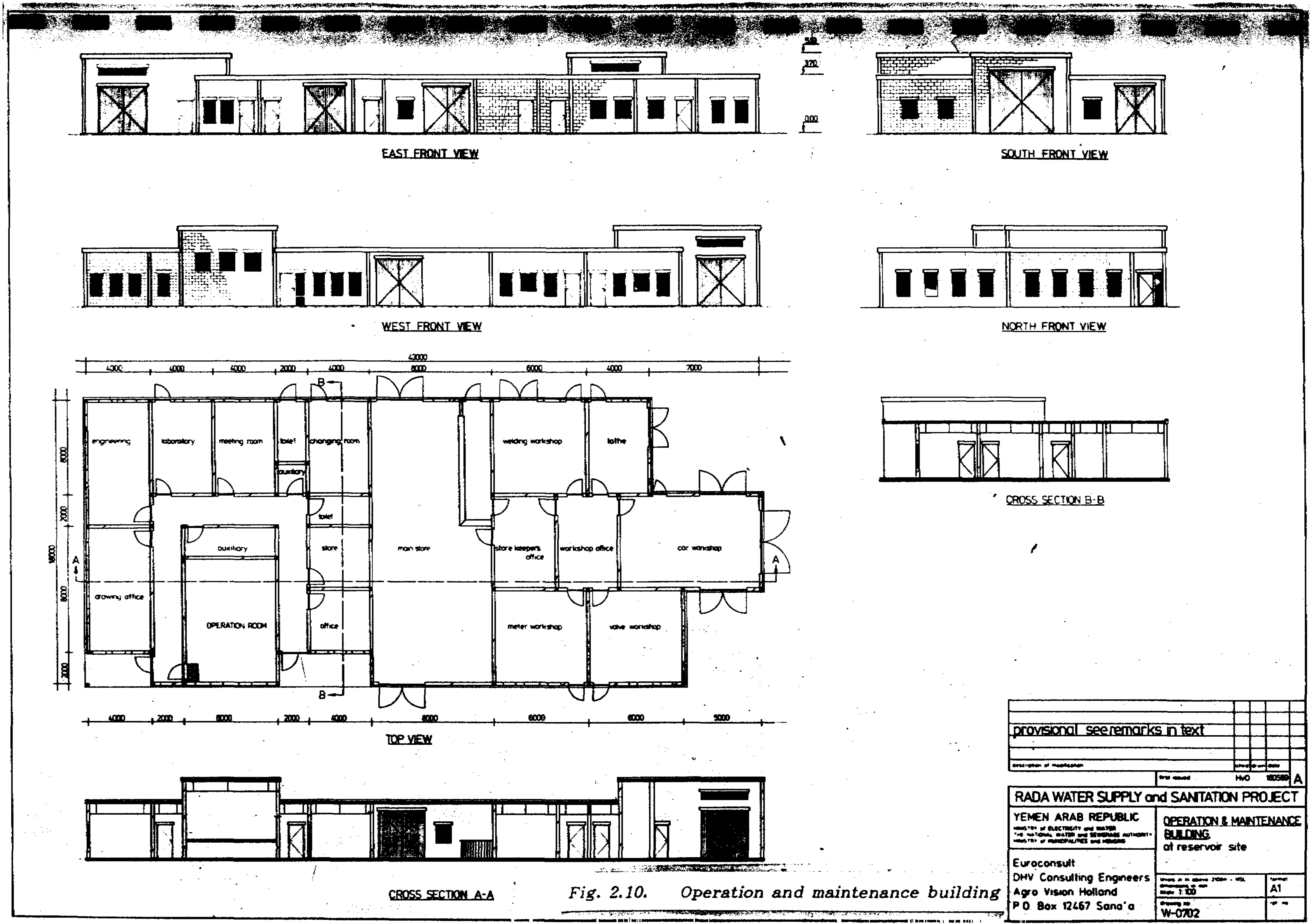


Fig. 2.10. Operation and maintenance building

The entire reservoir site and utilities compound will be provided with safety equipment such as fire fighting equipment, etc. In addition, terrain lighting, where necessary, will be installed.

2.8.2. Operation and control

Operation of the deepwells, chlorination installation and storage reservoir will be coordinated on the utility building site. Details are given in the P&ID (Fig. 2.11.). In this figure, the deepwells are shown to the left (for the first phase only), the reservoir to the right and the chlorination installation at the top. Water for local use at the compound is pumped from the storage reservoir's outlet into a 2 m^3 reservoir (top right of Fig. 2.11.). This reservoir also provides water for the emergency and eye shower at the chlorination building. In case the utilitybuilding compound is built on groundlevel, above describe set-up has to be adapted accordingly, which requires in fact no real major changes.

2.9. Chlorination

2.9.1. System selection

The treatment of the deepwell water is restricted to a safety chlorination. The equipment used for that purpose will also be used for the initial disinfection of the water supply system at the time of commissioning. The required capacity of the chlorination equipment is based on a residual chlorine level of 0.1 - 0.2 mg/l at the periphery of the distribution system. The metering capacity before the reservoir is estimated at 2 mg/l. The corresponding Cl_2 consumption is shown in Table 2.9., for both chlorine gas and calcium hypochlorite.

Table 2.9. Chlorine consumption at reservoir site

		1995	2010
max. day demand (= prod.cap.)	[m^3/day]	3762	8316
max. hour production	[m^3/h]	50	100
min. hour production	[m^3/h]	10	10
estimated Cl_2 consumption	[kg/day]	7.5	16.6
estimated hypochlorite consumption (70% Cl_2)	[kg/day]	10.8	23.8

A cost comparison between the two options, with calcium hypochlorite and gaseous chlorine, respectively, shows that the use of calcium hypochlorite would be marginally cheaper, at a daily cost of YR 345 versus YR 375, for the 1995 situation. Calcium hypochlorite is not locally available, however, whereas gaseous chlorine is. As the available experience is limited to gaseous chlorine as well, it has been decided to base the disinfection of the water for the new water supply system of Rada on gaseous chlorine.

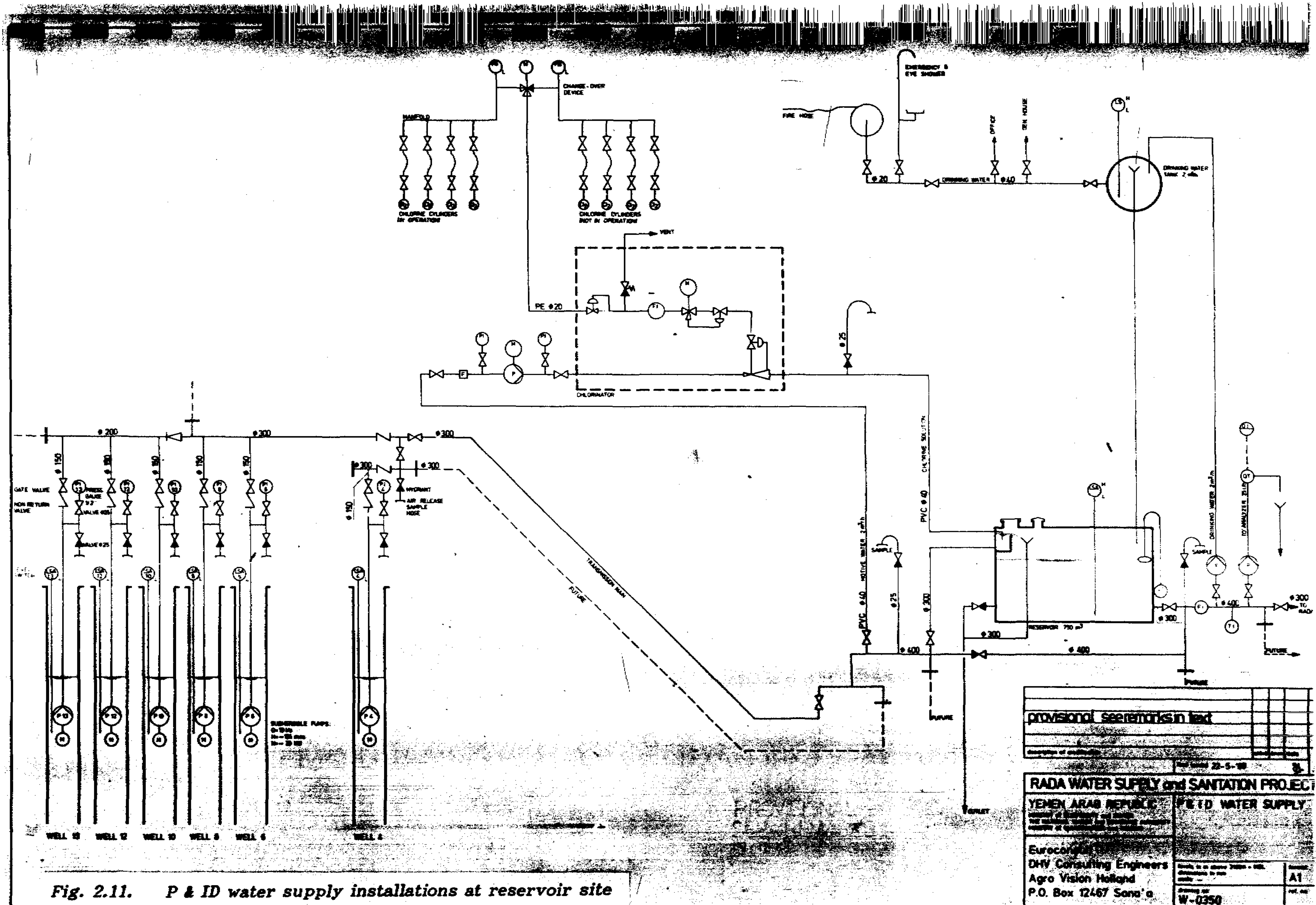


Fig. 2.11. P & ID water supply installations at reservoir site

Operational parameters for disinfection with gaseous chlorine are presented in Table 2.10:

Table 2.10. Operational parameters, chlorination

		1995	2010
max. day demand (= prod. cap.)	[m^3/day]	3762	8316
Cl_2 consumption at 2 mg/l dosing	[kg/day]	7.5	16.6
20 cylinders @ 50 kg suffice for	[days]	133	60
max. production capacity (5/10 wells)	[m^3/h]	180	360
maximum dosing at 2 mg/l	[kg/h]	0.36	0.72
minimum dosing (1 pump operational)	[kg/h]	0.072	0.072
dosing range	[g/h]	72 - 720	
time covered with 1 cylinder	[days]	6.65	3.0
ditto, with set of 4 cylinders	[days]	26.6	12.0

2.9.2. Chlorination building

The chlorination building is shown in Fig. 2.12. It consists of two compartments, one with the chlorinator and motive water pump and the other with two sets of 4 chlorine cylinders each, as well as two half-open storage areas with a capacity of 16 chlorine cylinders each. One of the storage areas is for full cylinders, the other for empty ones. Two sets of 4 cylinders each are connected to the manifold, with an automatic change-over device switching over from the empty to the full chlorine gas cylinders. Fire fighting facilities, an emergency and eye shower and compressed air masks and safety clothing will be provided.

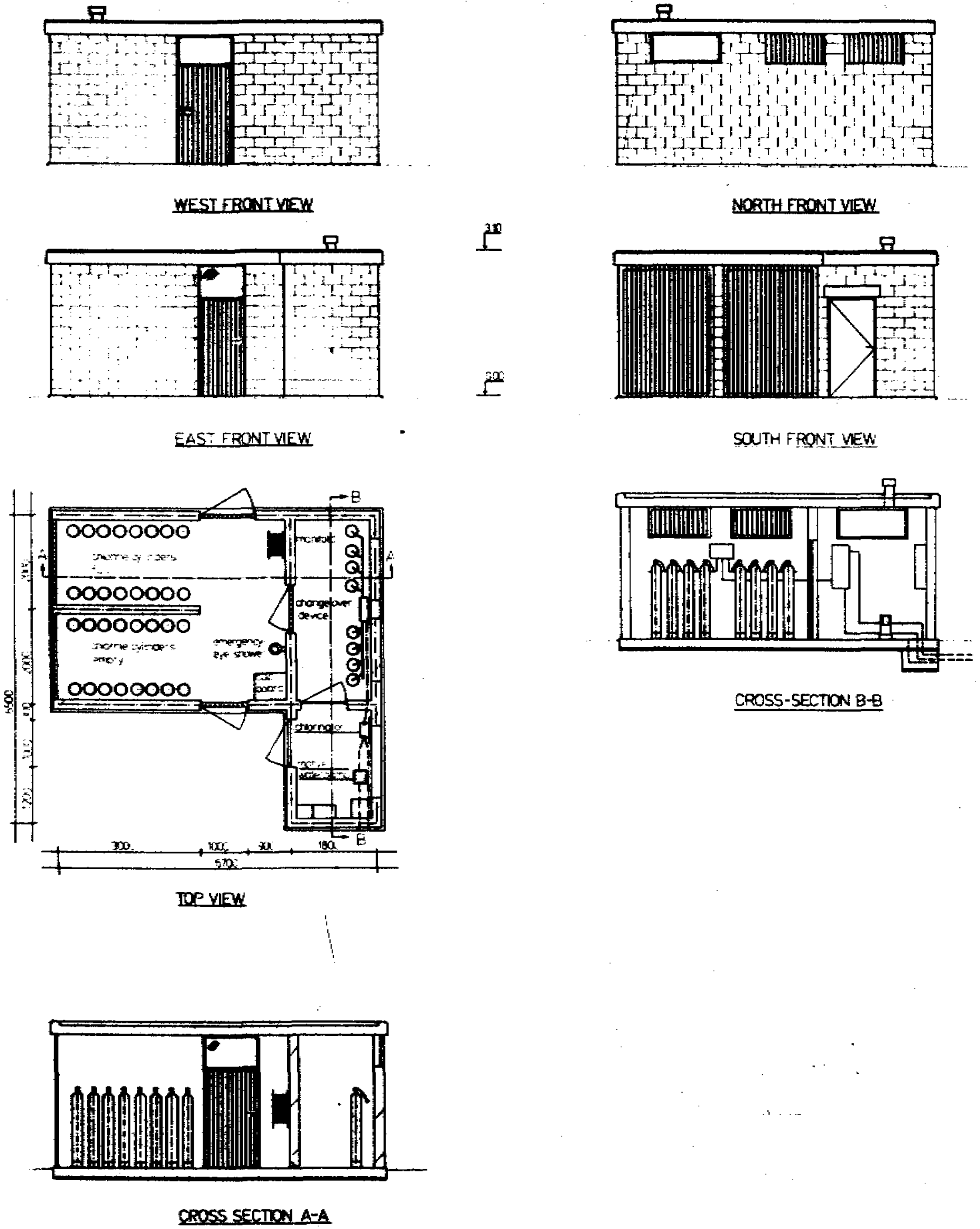
2.9.3. Operation

The chlorinator is fed with gas from one of the chlorine cylinder batteries and with water from the motive water pump (2 m^3/h at 3 bar, 0.55 kW pump). The proposed chlorinator is a Wallace & Tiernan Chlorator V.741.GST with additive-rate actuation by switching deepwell pumps on or off (see also Fig. 2.11.). The chlorine solution is dosed at the inflow weir(s) of the storage reservoir, or directly in line before the storage reservoir.

2.10. Storage reservoir

For the second phase a total storage capacity of 1500 m^3 will be required. To postpone as much as possible those investments that are not strictly required for the first phase, it is planned to construct the reservoir in two equal chambers of 750 m^3 each. The first chamber will be constructed directly, whereas the second chamber would be required for phase 2 only. Details of the reservoir are shown in Fig. 2.13.

Each reservoir chamber is 8 m wide and 25 m long, with a net water layer of 3.75 m thickness at an internal reservoir height of 4.15 m. Each chamber is provided with a baffle in the shape of a longitudinal wall over 21 m, to prevent short-circuiting of water between inlet and outlet points. All piping: inflow, outflow, overflow and drain pipes, have the same diameter: 300 mm.



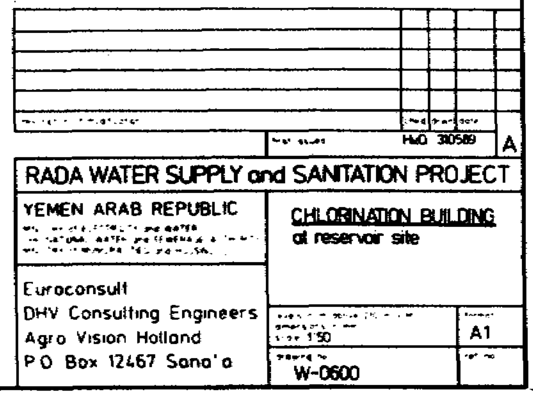


Fig. 2.12. Chlorination building

Because of the quality of the reduced original, this drawing is attached on full size (A1) to this report in the backflap.

Fig. 2.13. *Water reservoir*

The ground elevation at the reservoir site is approximately 2170 m + MSL, resulting in a maximum water level in the reservoir of approximately 2174 m + MSL. The inflow of water into the reservoir is located above the maximum water level, so back-flow from the reservoir cannot occur.

A by-pass valve will be installed between the transmission main and the outgoing distribution main, for maintenance purposes.

All piping is concentrated in a pipe gallery located between the two reservoir chambers. In that gallery also the following additional provisions are located:
- motive water take-off and pump, for feeding the chlorinator(*)
- bulk water meter
- pump for local water supply (*)
- water sampling point

* Will eventually be relocated to utility compound site on other location than reservoir site.

Details are shown in Fig. 2.13. and on the P&ID, Fig. 2.11. The bulk water meter has a diameter of 150 mm and is equipped with a mechanical counter as well as a pulse unit. The sampling point is provided with a residual chlorine analyzer and transmitter. The system is meant to monitor rather than directly control the chlorination.

2.11. Power supply

2.11.1. Existing and planned public power supply

The present capacity of the power supply system in Rada is not sufficient to supply the required power to the compound and well field. The planning of the national electricity company YGEC, however, indicates a future connection of Rada to the 33 kV line from Dhamar, about fifty kilometres west of Rada. Information obtained from Lahmeyer International, the German consultant charged with the design of the medium voltage line from Dhamar to Rada, is as follows:

The electricity network in Rada will be an 11 kV system, supplied by means of one step-down transformer 33/11 kV (power capacity 5000 kVA) installed near the existing power plant. A part of the existing overhead lines will be maintained, whereas new lines will be added.

The medium voltage single line diagram of the design shows a so-called "rural reserve" connection with a power capacity of 350 kVA, equivalent to approximately 280 kW (see Fig. 2.14.). This reserve connection could well be used for the compound and the well field. To that effect a formal request has been submitted to YGEC about electricity connection to the water supply plant, claiming the "rural reserve" connection as mentioned in the power supply design, for the reservoir compound and well field. The necessary power capacity is about 220 kW (275 kVA), thus well within the limits set for the "rural reserve".

The time schedule for the erection of the power connection between Dhamar and Rada is such that in the most optimistic scenario that connection could be realized somewhere in 1992. Most probably it will take until the year 1993, however, for the new power supply system to be commissioned.

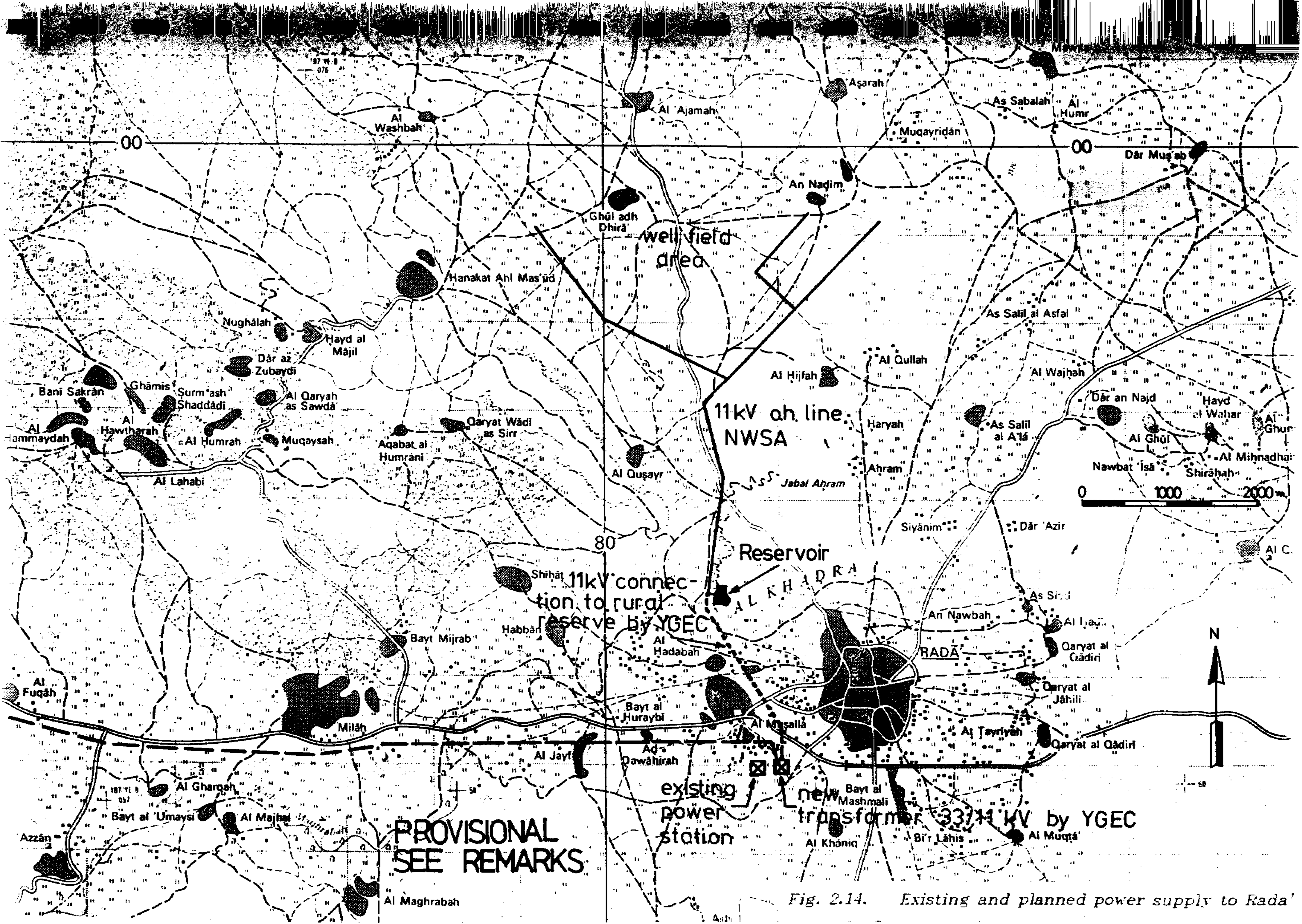


Fig. 2.14. Existing and planned power supply to Rada'

2.11.2. Power supply system for reservoir compound and well field

The power required during maximum load conditions is illustrated in Table 2.11:

Table 2.11. Power required at reservoir site/well field

Well field:

- installed: 6 pumps; max. 5 in operation
- data pump motor: 18.5 kW (= 21.3 kVA), power factor = 0.87, 2900 rpm
  I_{nom} = 38 A, I_{start} = 5.5 * I_{nom}
- power during starting: 5.5 * 38 * 0.66 = 138 kVA

  Note: 0.66 is factor to convert Amperes to kVA

Compound:

- chlorinator:	1 kW
- motive water pump, local water supply pump	1 kW
- workshop	10 kW
- lighting	4 kW
- contingencies	4 kW
	20 kW (25 kVA)

Maximum load conditions:

- max. starting conditions phase 1 (compound, 4 deepwells operational, 5^{th} deepwell pump being started):
  25 + 4 * 21.3 + 138 = 248 kVA
- ditto, for phase 2 (compound, 9 deepwells operational, 10^{th} deepwell pump being started):
  25 + 9 * 21.3 + 138 = 355 kVA
- maximum operating conditions phase 1: 25 + 5 * 21.3 = 132 kVA
  average power consumption during operation: 90 %, or: ≈ 100 kW
- maximum operating conditions phase 2: 25 + 10 * 21.3 = 238 kVA

The extension of the power supply system in Rada will not be in time for the commissioning of the new water supply system, while the current power supply capacity is far from sufficient even without the added load of well field and reservoir site. Therefore the generation of electricity independently of YGEC is recommended, if only for the first phase.

Assuming a maximum overload capacity of 50% during 120 seconds, the capacity required during maximum starting conditions (see Table 2.11.) would be 248/1.5 = 165 kVA. It is recommended to install three diesel generator sets, two with a capacity of 85 kVA and one with a capacity of about 165 kVA (continuous power). These generators will be the sole source of power until the "rural reserve" connection will have been realized. From that time onwards, the generator sets will act as emergency stand-by only.

For the supply of power to the well field there are three possibilities:

a. To generate 0.4 kV and to transport 0.4 kV to the well field. This solution is not acceptable because the voltage is too low in relation to the distance to the deepwell pumps. The cable would be too heavy. Moreover, there would not be a possibility to connect the 11 kV power main in the future.

b. To generate 11 kV and to transport 11 kV to the well field. Two important disadvantages are: 11 kV generator sets with a power capacity below 200 to 300 kW are not current and are 50% more expensive than the usual 400 V types.
For financial reasons this solution is thus not recommended.

c. To generate 0.4 kV and to increase the voltage to 11 kV by means of a step-up transformer.
The advantages of this solution are:

- 0.4 kV generator sets in the range of 60 kW and over are very common;
- a 11 kV connection between compound and well field fits very well in the future power supply set-up as indicated in the YGEC planning;
- this equipment requires the lowest investment costs.

Based on the above it is recommended to install three diesel generator units, voltage 0.4 kV, power capacity: two of 85 kVA each and one of abt. 165 kVA, and a step-up transformer to increase the voltage to 11 kV for supplying the well field. Taking into account the distance between the boreholes, each deepwell pump will be provided with a pole-mounted step-down transformer (11 kV/0.4 kV), power capacity about 20 kW.

The proposed power supply set-up is visualized in Fig. 2.15. As shown there, initially only the compound will be fed directly, at 400 V, from the generator sets, with the well field supplied via 11 kV overhead line and step-down transformers at each well head. After realization of the planned extension of the Rada public power supply system, the direct connection between the generator sets and the compound will be used only during emergency power conditions. Normally, however, both the compound and the well field would be fed from the 11 kV overhead line, all being supplied through step-down transformers.

2.11.3. Generator house

For reasons of control, maintenance and diesel oil supply, the generation equipment is located in a building proposed at present at the reservoir compound, as shown in Fig. 2.9., but possibly to be relocated.

The generator sets are installed in a generator house, as shown in Fig. 2.16., with a floor area of the generator room of about 70 m^2. Next to it an oil storage is provided, with two horizontal storage tanks of 10 m^3 each. The oil consumption of the diesel engines is about 0.28 l/kWh, or for average operation conditions during 20 h/day, 300-600 l/day.

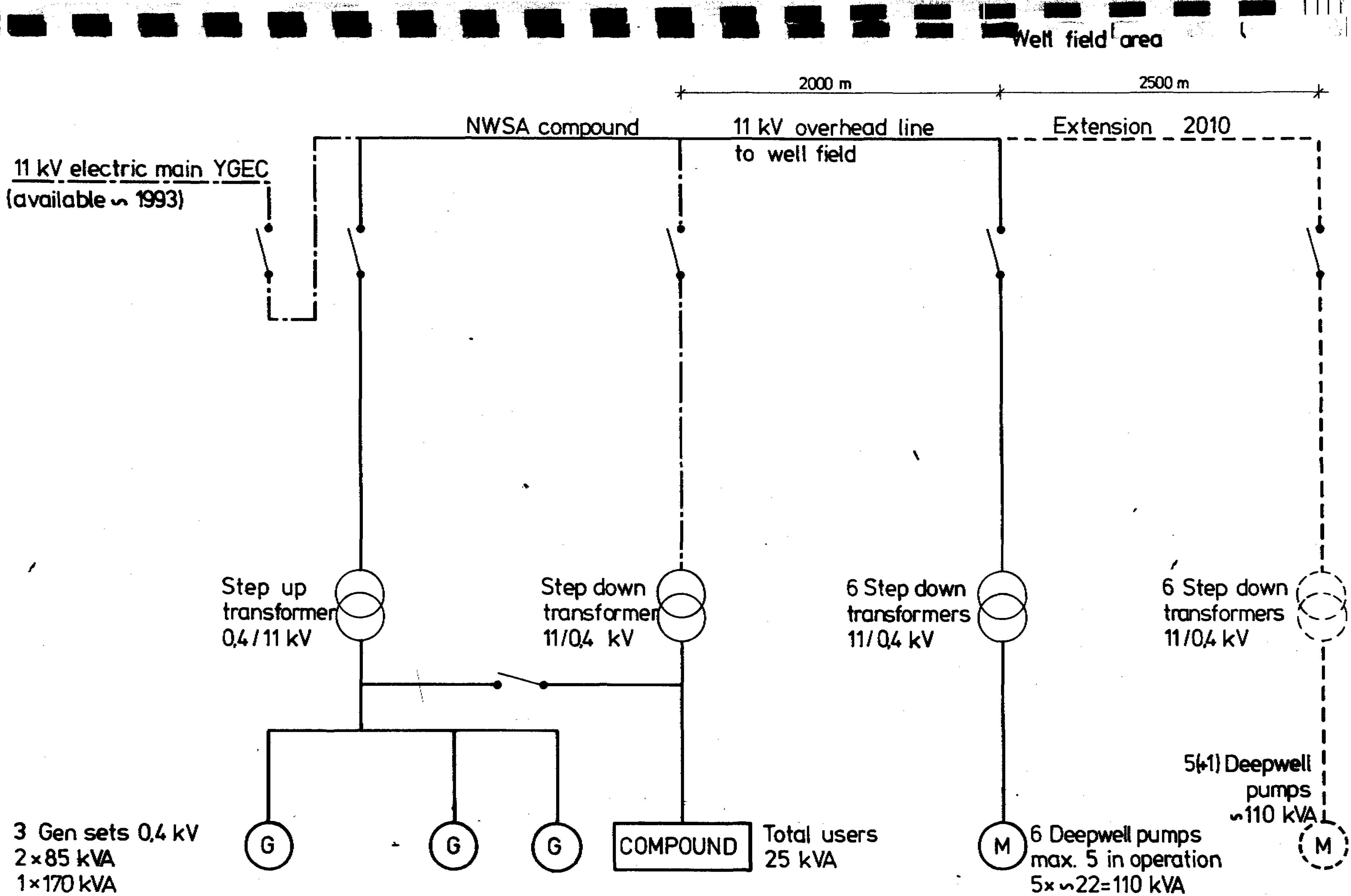
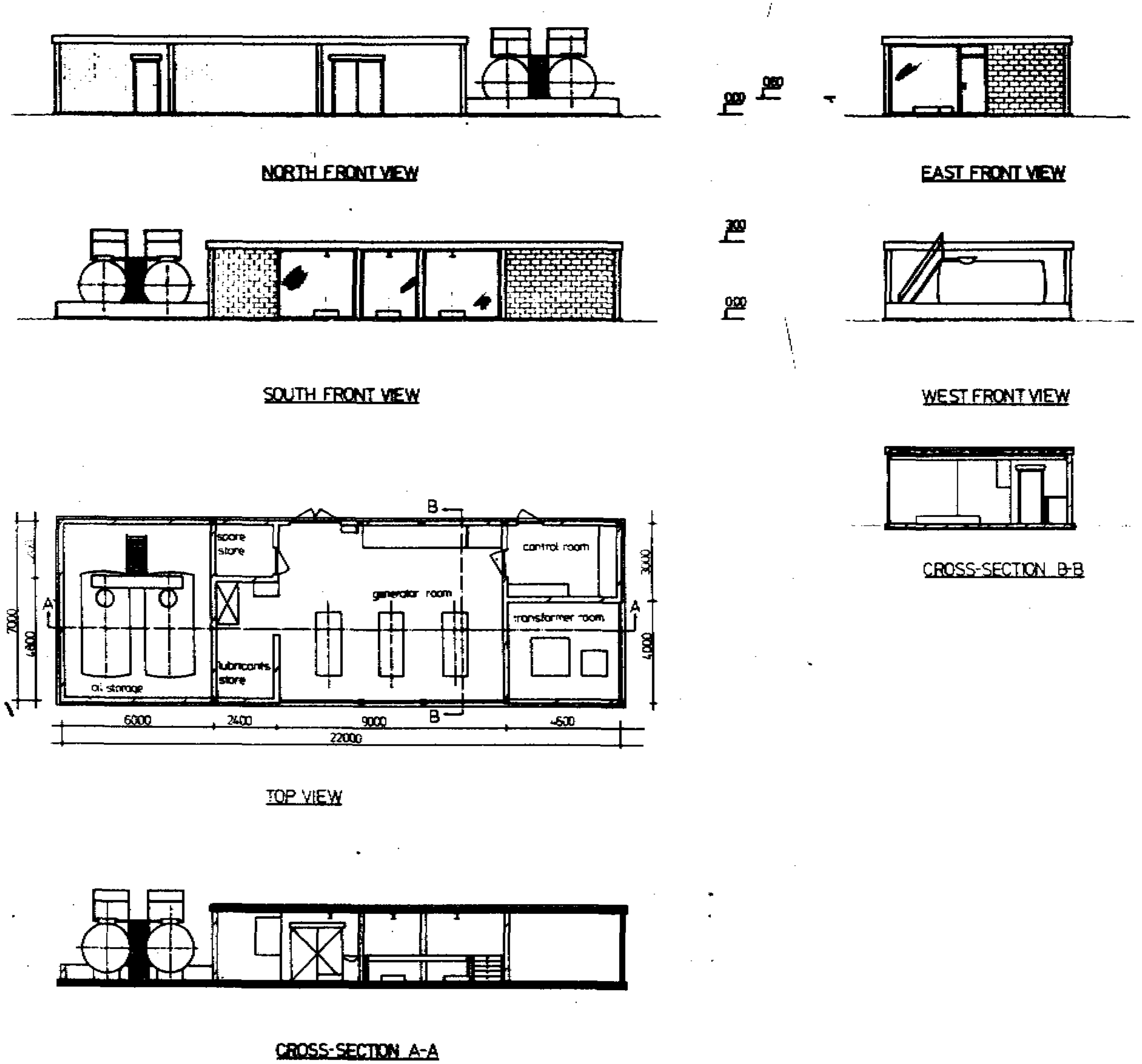


Fig. 2.15 MV power supply diagram, reservoir compound



RADA WATER SUPPLY and SANITATION PROJECT

YEMEN ARAB REPUBLIC

GENERATOR HOUSE
reservoir site

Euroconsult
DHV Consulting Engineers
Agro Vision Holland
P.O. Box 12467 Sana'a

scale 1:100 | A1

W-0501

Fig. 2.16 Generator house at compound

2.12. Distribution system

2.12.1. Introduction

a. General approach

In any distribution system, primary, secondary and tertiary distribution pipes can be distinguished.

The purpose of the primary network is mainly the transportation of water from the reservoir(s) to the various parts of the town. The distribution systems, and especially the primary networks that are designed nowadays, are usually so-called looped systems. In the Inception Report such a looped system was proposed[21]. To reduce the investment costs for phase 1 of the project (design horizon 1995), the primary distribution system has been designed as a branched system only. It is recommended that during the second phase (design horizon 2010) this system be extended into a looped system. For the districts III thru X and XII the secondary distribution system will be a looped one directly from the beginning, however. For the districts I, II and XI and the outlying districts XX-XXV the secondary system will be branched as well, until those areas will have fully developed.

The primary distribution network will be kept under pressure from a central point, viz. the storage reservoir. The primary system is divided in sections by means of valves, thus allowing the disconnection of individual sections for repair, extension, etc. Individual houses must not be connected to the main system but to the so-called reticulation system.

With regard to the design of the primary distribution network the following aspects have been taken into account:
- the primary network is a looped one in the ultimate situation (2010), but initially a branched one;
- the water will enter the primary system near Qaryat Al Habadah;
- for the pipe alignments existing roads will be used as much as possible;
- the primary system must be able to supply also future districts just outside the presently built-up area.

b. Leakage control

The supply area will be subdivided into several supply districts, as indicated in Fig. 1.3. Within the presently built-up area these districts cover an area of 8.5 - 30.5 ha each.

They are connected to the primary network at one or two locations. At such connections a district water meter, gate valve and non-return valve will be installed. In that way water can only enter a district at these connections, and not leave it, so that the water entering the district can be accurately measured. This set-up has been selected to control leakage and prevent spreading of potentially contaminated water.

---

[21] Inception Report, Rada Water Supply and Sanitation Project, Volume I - Main Report, July 1988, page 3-14.

With the aid of these district water meters and the water meters at the individual house connections the amount of leakage can be calculated. Leakage is then defined as the difference between the volume of water passing the district water meter(s) and the totalized registered consumption of the house water meters in the concerned area. Also the readings of the night flow may give an indication of possible leakage within that district.

The secondary system will be divided into sections by means of gate valves.

2.12.2. Primary system

a. Lay-out

Fig. 2.17. shows the selected aligament for the primary distribution network. The main primary distribution pipe is between the storage reservoir and the centre of the project area (node 4394[22]).

The primary pipes that are located in built-up areas are projected along existing roads to avoid delays during construction resulting from planning and expropriation procedures.

b. Hydraulic calculation

The hydraulic calculation of the distribution system has been carried out with the computer program WANACA.

The water demand for the various districts of the project area has been given in Table 2.6. The peak demand as mentioned there has been taken into account for the hydraulic calculations.

The diagrammatical lay-out of the primary and secondary distribution systems is indicated in Fig. 2. 18.

For the calculation of the primary network the demand of the supply districts is assumed to be concentrated in the "centre of gravity" of the relevant district. This point is then joined to the connecting location(s) between primary and secondary system by means of two dummy pipes. In this way the abstraction from the primary system by the reticulation systems of the districts is simulated.

The minimum pipe diameter for the primary system has been fixed at 200 mm. Similar to the transmission main between the well field and the storage reservoir, the diameter of the first part of the primary main could be 400 mm, to cater for the ultimate situation. This would imply an over-capacity of 100% for the 1995 situation, however, while making the entire supply dependent on a single feeder main only.

[22] the node numbers are indicated in Fig. 2.18. and Fig. A.2., Annex A.

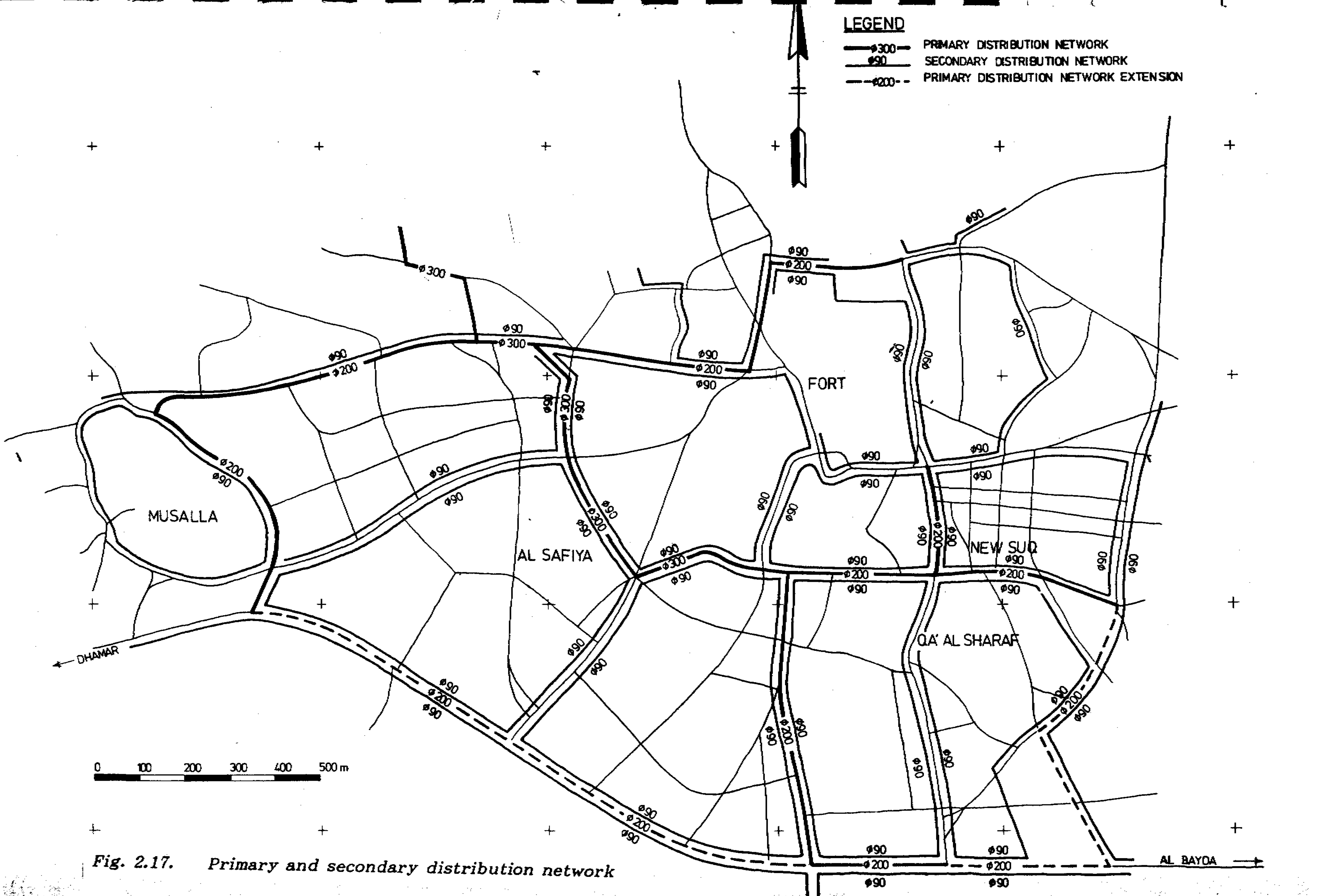


Fig. 2.17. Primary and secondary distribution network

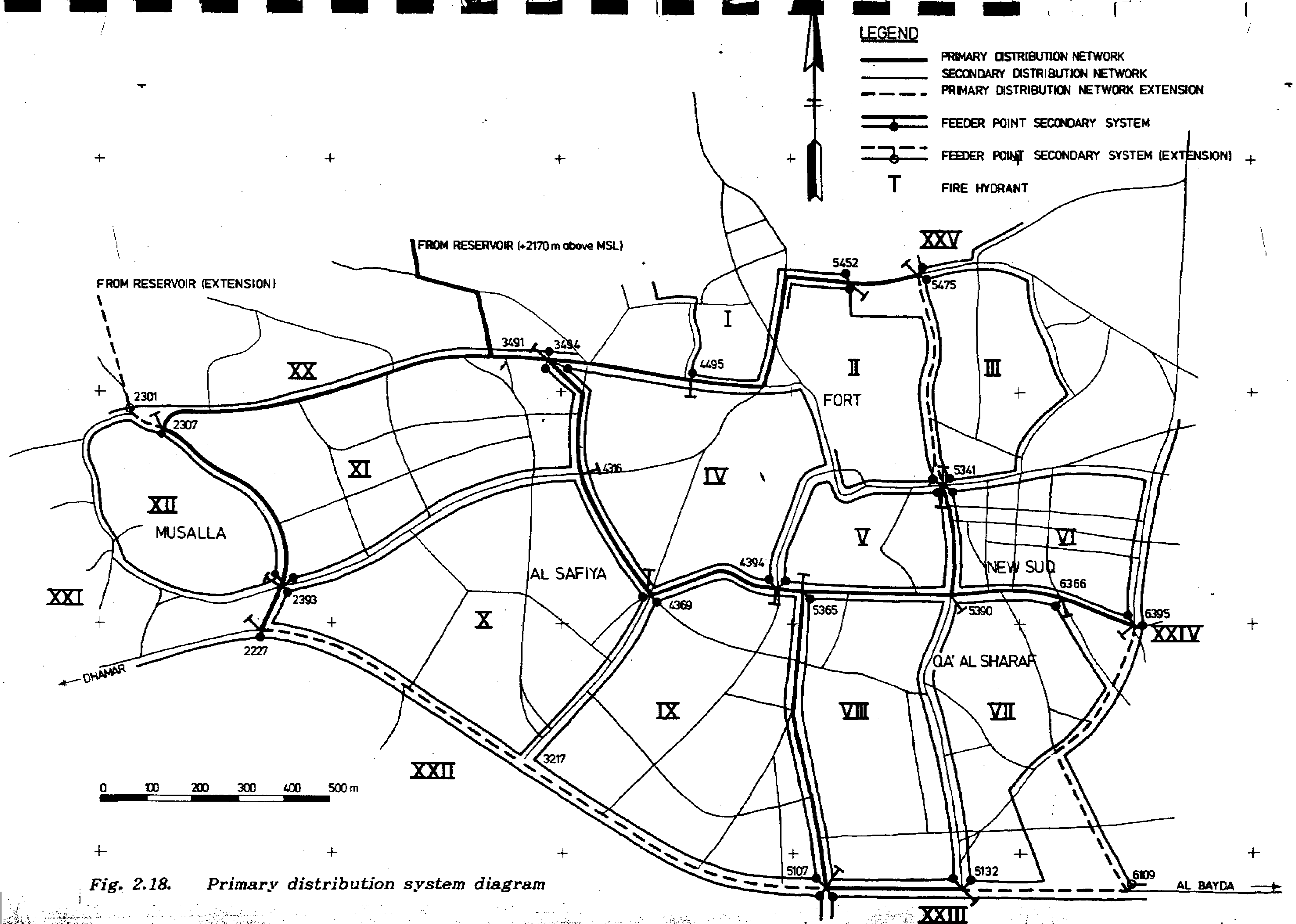


Fig. 2.18. Primary distribution system diagram

For that reason it is proposed to phase the first part of the main distribution pipe in the same way as the transmission main. For the first phase a ø 300 mm main will be laid up to node 4394, passing nodes 3491, 3494 and 4369. For phase 2 a second main with the same diameter will be laid between the reservoir and node 2307 near Musalla.

The results of the hydraulic calculation of the primary distribution network are shown in Annex A.2, for the demand and configuration in 1995 and 2010 (peak flow).

In the proposed set-up of the water supply system the effective pressure at the location of the reservoir will be at least 70 mwc. This results in maximum pressures at night time of about 45 mwc in the eastern part of Rada. This pressure is in principle, according to set design criteria, not sufficient, however, for supplying Musalla (residual pressures down to 6 mwc only). Solving this problem by raising the reservoir level over 16 m would result in maximum pressures of over 60 m in the remaining part of the distribution network. This is not a desirable situation and consequently a booster station should have been provided at Musalla to ensure a sufficient pressure there. However, no boosterstation will be provided, amongst others due to complex operation and maintenance (see also paragraph 2.13.3).

A similar but minor problem occurs in a small part of the Harat Al Qana area (district 11). During peak hours the pressure there may drop to about 12 m, against 22 m as mentioned in the design criteria (paragraph 2.2.).

Annex A.2. also presents the results of the hydraulic calculation for the 2010 situation under fire-fighting conditions. From two adjacent fire hydrants (nodes 5390 and 6255) 10 l/s each is abstracted during those conditions.

The required lengths for the primary distribution network (phase 1) are:

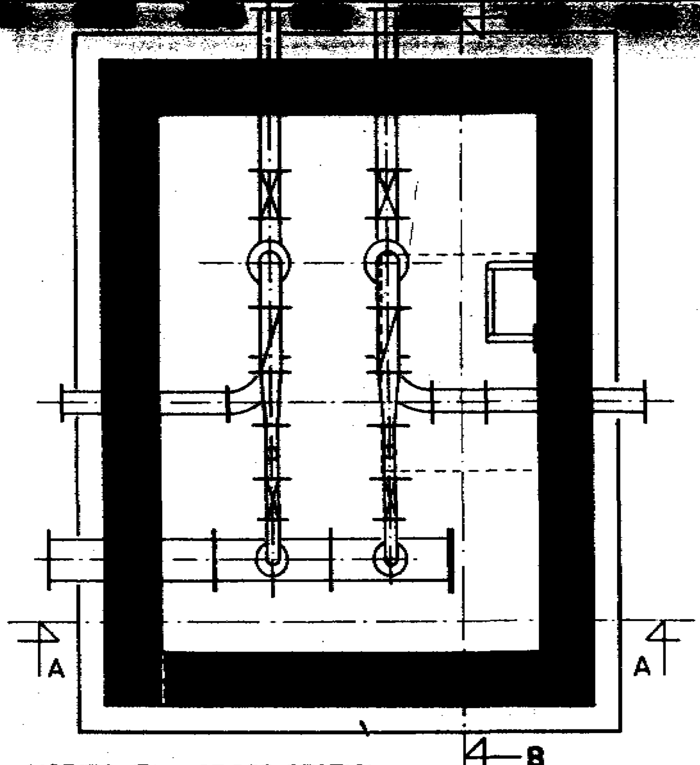
ø 200 mm: 4410 m
ø 300 mm: 980 m

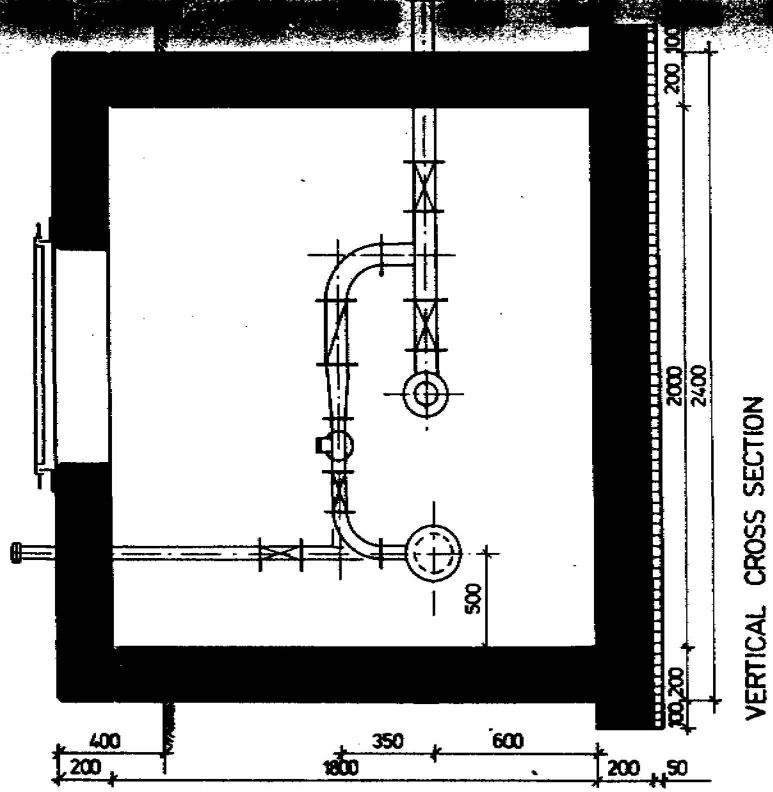
c. District water meters

The connections between the primary and secondary distribution systems are concentrated in so-called district water meter chambers. An example of a district water meter chamber to which two districts are connected, is given in Fig. 2.19., with similar details for water meter chambers feeding one and three districts, respectively, in Figs. A.21 and A.22, in Annex A.
The district water meter chambers are of the underground type. The main fittings in these chambers are:

- a valve in the main (ø 50 mm) connecting to the primary system
- a non-return valve in the same main, to prevent flow through one district to another
- a bulk water meter
- (if required at that location) a branch-off to the fire hydrant
- a Tee with two valves in the outgoing secondary distribution pipes

The district water meter chambers will be constructed of reinforced concrete and have a steel hinged cover.





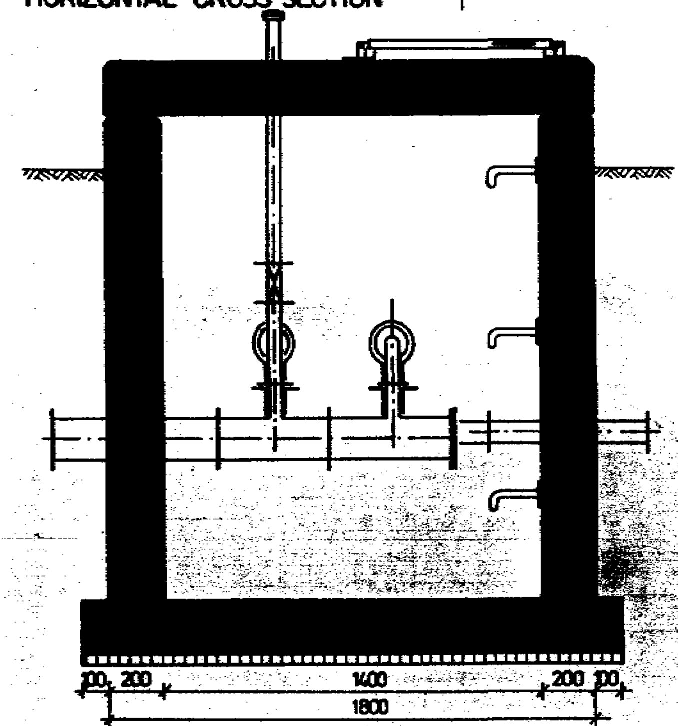


Fig. 2.19. Typical district water meter chamber (feeding 2 districts)

description of modification	chkd	drwn	date

first issued 25-5-'89 A

RADA WATER SUPPLY and SANITATION PROJECT

YEMEN ARAB REPUBLIC
MINISTRY of ELECTRICITY and WATER
THE NATIONAL WATER and SEWERAGE AUTHORITY
MINISTRY of MUNICIPALITIES and HOUSING

TYPICAL DISTRICT WATER METER CHAMBER, FEEDING 2 DISTRICTS

Euroconsult
DHV Consulting Engineers
Agro Vision Holland
P.O. Box 12467 Sana'a

levels in m above 2100m + MSL
dimensions in mm
scale 1:20

format: A3

drawing no: W-0906

ref. no:

2.12.3. Booster pumping station at Musalla

As mentioned in the previous paragraph, in principle - according to design criteria - a booster station is required to guarantee sufficient pressures at Musalla during peak hours. For that reason a booster station as shown in Fig. 2.20. was proposed to be included in the primary water distribution system.

Without booster station, the pressure in the nodes near Musallah will vary at maximum demand (peak hour) between 7.5 and 25 m.w.c. and at night between 12.5 and 30 m.w.c. Therefore, the complete distribution network will always completely be filled. If the design criterium of 20 m.w.c. is lowered to 10 m.w.c., only 3% of the houses will not have enough pressure in accordance with this definition at peak hours, but still pressure enough to the third floor of the houses on the top of the hill in Musallah is available. It was therefore decided by NWSA to delete this pumpstation.

2.12.4. Secondary and tertiary systems

a. Lay-out

The supply districts are shown in Fig. 2.18. In general the districts are connected to the primary distribution network at 2 locations each. For the outlying districts (XX-XXV) this will often be possible only in the future, e.g. after the fully looped primary mains system will have been established.

The design of the secondary and tertiary systems is based on using a standard pipe diameter of 90/73.6 mm[23] for the secondary mains and of 50/40.8 mm for the tertiary system. The secondary main encircles the whole district or the main part of it.
The tertiary pipes are connected to the secondary pipes. Valves are installed in such a way that parts of the system can be taken out of operation (see typical details in. Fig. A.23., Annex A.2.). The lay-out of the distribution system in the districts I - XII is given in Annex A.2., drawings DW-001 through DW-012. These districts comprise the bulk of the presently built-up area. For the outlying districts, numbered XX and higher, sufficiently detailed information is not always available at this stage. Details on those districts will be given as and when they become available.

[23] outer/inner diameter

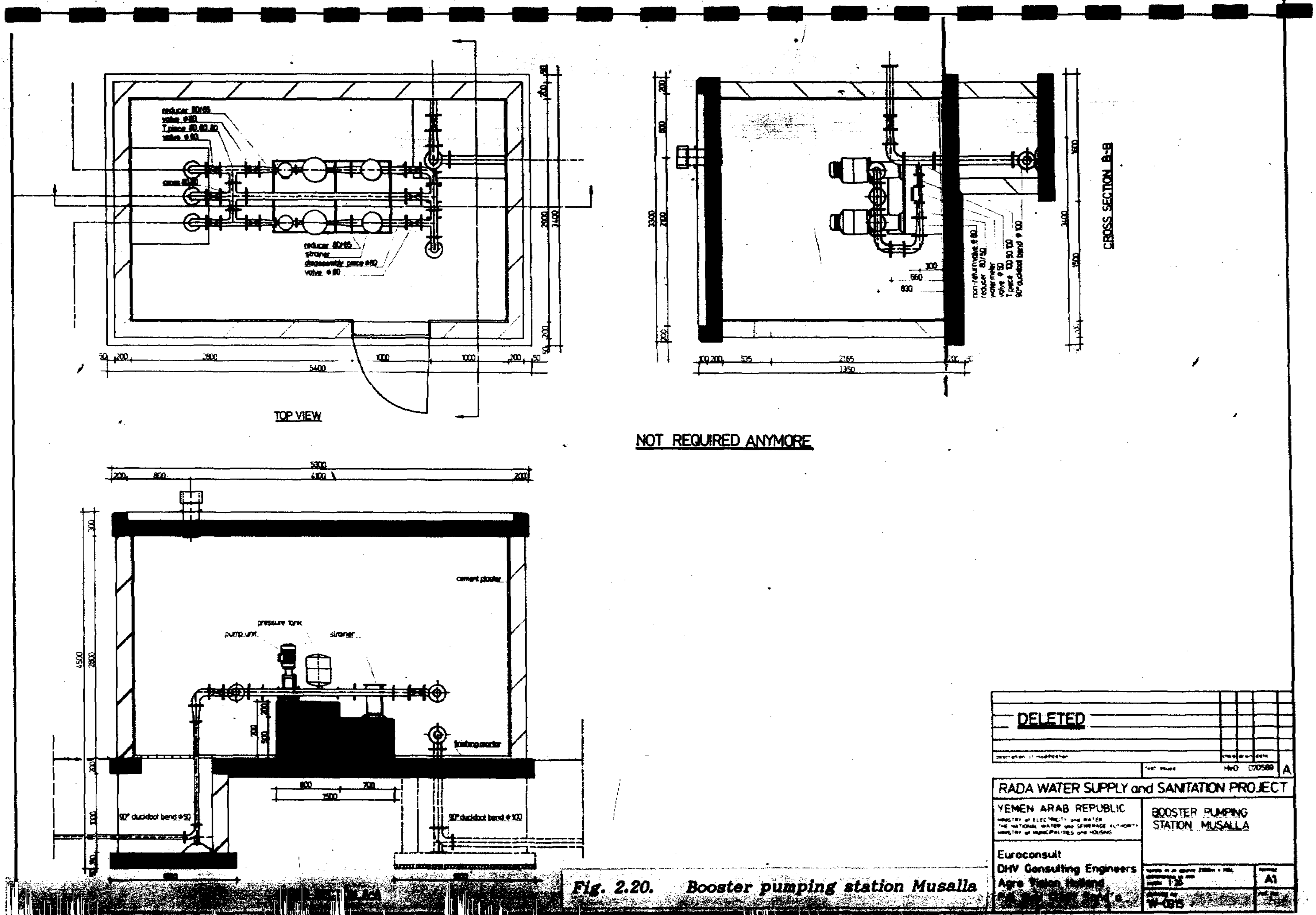


Fig. 2.20. Booster pumping station Musalla

b. Hydraulic calculations

The hydraulic calculation of the distribution districts has also been carried out with the computer program WANACA. For calculation purposes the water demand has been taken as concentrated in the individual nodes of the distribution network. Details of the hydraulic calculations for the presently urbanized districts (Nos. I - XII) are given in Annex A.2., for peak flow conditions, year 2010.

A break-down of the pipe lengths required for the secondary and tertiary systems is given in Table 2.12:

Table 2.12. Breakdown of pipe lengths for secondary/tertiary system

Diameter [mm]: District No.	90 mm	50 mm in calc.	50 mm extension	50 mm total	25 mm total	No. of connections
I	1220	4370	160	4530	5850	542
II	1160	2790	220	3010	4720	437
III	1375	1990	380	2370	4030	373
IV	1840	2045	205	2250	4240	393
V	1615	1285	400	1685	4990	462
VI	1395	1536	204	1740	3360	311
VII	1840	2790	210	3000	4550	421
VIII	1860	2140	200	2340	3380	313
IX	2295	3795	205	4000	4240	393
X	2190	2965	445	3410	2430	225
XI	1465	1680	445	2125	1480	136
XII	1285	3325	220	3545	4720	437
Sub Total:	19540	30711	3294	34005	47990	4443
XX	1700			2100	1540	143
XXI	-			1000	230	21
XXII	2125			2000	1230	114
XXIII	1700			1900	770	71
XXIV	2200			2400	1930	179
XXV	-			1000	310	29
Total:	27265			44405	54000	5000

2.12.5. Service connections

Service connections will be made only to the (HDPE) reticulation system, and never to the (ductile iron) primary mains. The service connection pipes will be ø 25 mm HDPE. Up to 4 houses can be accommodated by one connection to the reticulation system, as indicated in Annex A.2., Fig. A.24.

Each house connection will be provided with a water meter and non-return valve and installed -where possible - just inside the boundary of the premises, as shown in Fig. 2.21.

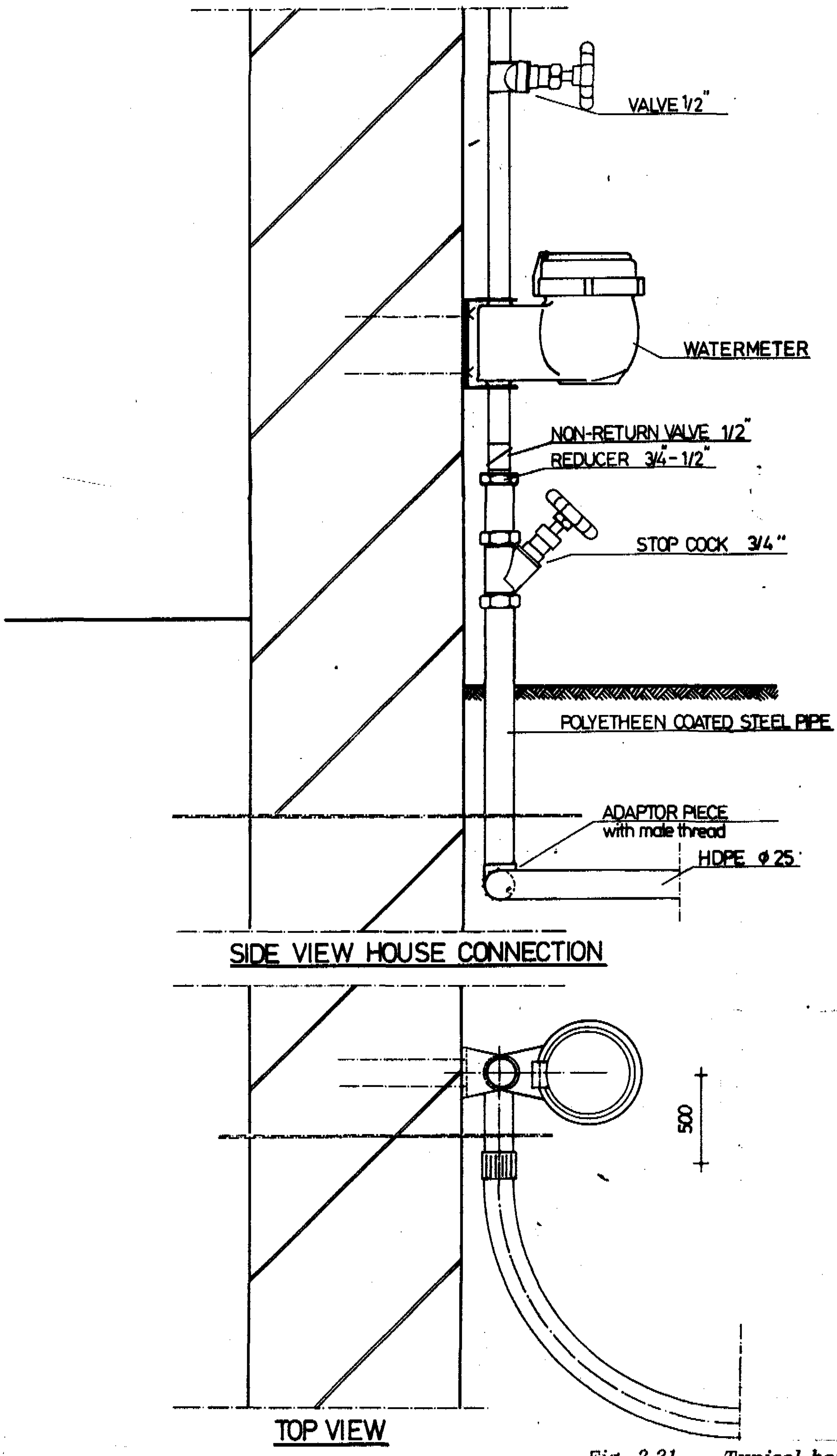


Fig. 2.21. Typical house connection

A total of 47,500 persons are expected to be connected to the distribution system by the year 1995. Based on an average household size of 7, 6785 house connections would have to be constructed. Adding approximately 10% to these for connections other than for households, the total number of service connections will become 7500.

Within the framework of the project 5000 connections will be constructed. The fees obtained for the connection of these are for instance to be used for the supply and installation of the remaining 2500 connections.

A break-down of the projected connections by category and size is given in Table 2.13:

Table 2.13. Breakdown of connections by category and size

Connections	Number Total	Size 25 mm	32 mm	50 mm
Non-commercial:				
- households	4,500	4,500	-	-
- schools, mosques	100	-	50	50
Commercial:				
- shops/restaurants	300	200	100	-
- workshops, factories, hospital, etc.	100	25	50	25
Total:	5,000	4,725	200	75

2.12.6. Fire fighting and flushing

In general the houses in the YAR are not highly inflammable. Conflagrations do occur, however. Also because of the growing use of modern materials for building and decorating, as well as the still expanding electricity network, fire hazards may increase considerably, however, especially in and near suqs, industries, petrol stations, etc.

For this reason it is proposed to place fire hydrants. These can also be used to vent and flush the primary system, e.g. after extensive repair or extension work. Flushing of the system must be done at night time, when it is the most effective and does not result in inconvenience for the users.

The fire hydrants will be connected to the primary distribution system. In this way the required supply (10 l/s) can be provided without disturbing the water balance in the supply districts. The fire hydrants are of the above-ground type, with 2 Storz couplings of ND 75.

The hydrants are to be placed on the following nodes:

2227 *[24]
2307
2393 *
3494 *
4316
4369 *
4394 *
4495 *
5107 *
5132 *
5341 *
5365 *
5390
5452 *
5475 *
6366 *
6395 *

2.12.7. Standardization

Standardization has been introduced of details of the primary and secondary distribution system, to simplify the preparation of bills of quantities and the administrative work during the construction period as much as possible. A computer program (FIT) has been prepared, which is based on standard details with standard fittings, and can totalize them, either per district or for the entire water supply component of the project.

2.13 Transport

In order to execute the tasks and duties under RWSSP's obligations properly some additional transport is required.
It was therefore in principle agreed to purchase in 1990 the following cars

- one Toyota, saloon type, long-base, 4WD, petrol
- one Toyota hardtop, 4WD, diesel

[24] hydrants indicated with * can be combined with a district water meter

3. WASTE WATER DISPOSAL

3.1. Present situation

3.1.1. General

The rapid development in Yemen since 1962 has resulted in many changes in the social/cultural habits of the Yemeni people. Because of the increasing water consumption also the amount of and problems caused by waste water have grown. Since groundwater exploitation started in 1972 the average water consumption has risen from 10 - 15 l/c/day to about 50 l/c/day. Moreover, especially flushing toilets have become rather common.

In the past waste water was poured away by bucket and thus spread over a large area. Also excreta were discharged to public areas. After the introduction of piped water supply the waste water situation has caused severe problems, especially in the presently densely populated areas. At locations where waste water cannot be drained or infiltrated into the sub-soil, the water becomes stagnant and clearly forms a considerable hazard to public health. This problem is compounded by the solid waste situation. The topsoil is clogged and waste water starts to run off through small gutters, dug in the sandy streets.

In the whole of Rada more than half of the waste water is disposed of in soakaway pits. Only now and then waste water is used for the garden. If human solid waste is disposed of separately this is always done in special facilities. On the other hand, where modern, cistern flush toilets are used, waste water is always disposed of in either a soakaway or, sometimes, a sewer system. Nevertheless: about one third of the waste water in Rada is still ending up in the streets, posing health hazards.

3.1.2. Source and disposal of waste water

Table 3.1. shows (in percentages) the different ways of disposing of the waste water.

From this table it is clear that more than half of the waste water is disposed of in soakaway pits (in case of the mixed toilets even 100%) while solid waste from the traditional toilets always goes to special facilities. Only in a few cases waste water is used for the garden.
This is graphically illustrated in Fig. 3.1.

Table 3.1. Ways of waste water disposal by source[1]

	Garden	Street	Soakaway	Sewer	Special Room	Total
Kitchen	5.4%	37.5%	55.6%	1.5%	-	100%
Washing machine	8.5%	37.5%	54.0%	-	-	100%
Toilet separated:						
- liquid waste	5.5%	44.3%	50.2%	-	-	100%
- solid waste	-	-	-	-	100.0%	100%
Toilet mixed	-	-	100.0%	-	-	100%

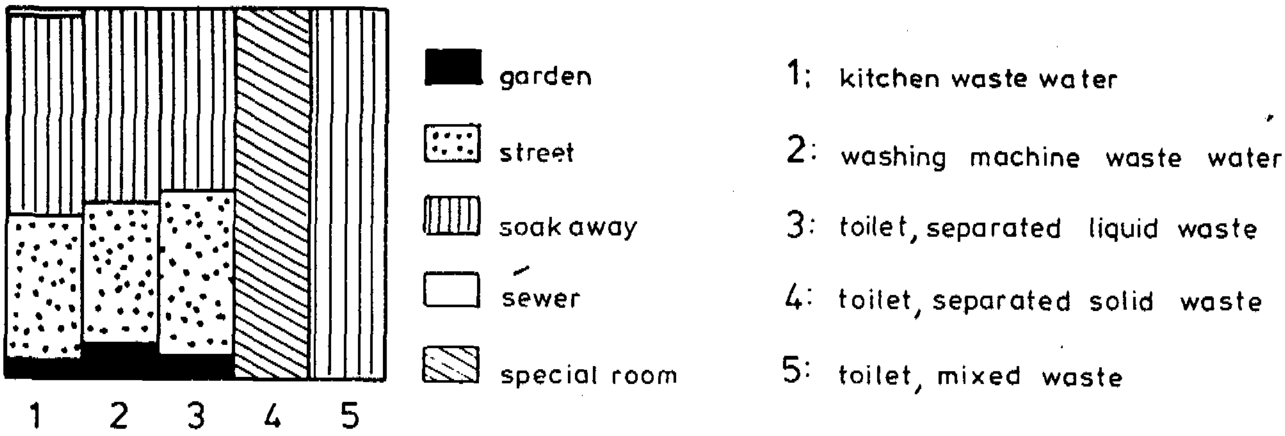


Fig. 3.1. Different ways of waste water disposal

3.1.3. Type of toilets

The distribution of the different types of toilets per district is given in Table 3.2. A distinction has been made between the following types:

- traditional, separated system: urine and stools are separately disposed of;
- mixed system/pour flushed: one disposal point, little water used;
- mixed system/cistern flushed: one disposal point, 8-12 litres at a time.

As shown in Fig. 3.2., all over Rada there are still approximately 60% traditional toilets, while of the remaining 40% about 1/3 is of the pour flush type. There is, however, a marked difference between the old and the new districts, as shown. Almost 80% of the toilets in the new districts are of the mixed type (of which almost 3/4 of the cistern flush type) against less than 30% in the old districts (of which only 2/5 of the cistern flush type). The tendency towards more modern types of toilets and thus higher waste water production is clear.

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[1] Present Water Supply and Sanitation Situation in Rada, Results of a sample survey in 333 houses, Rada Water Supply and Sanitation Project, 30 March 1989.

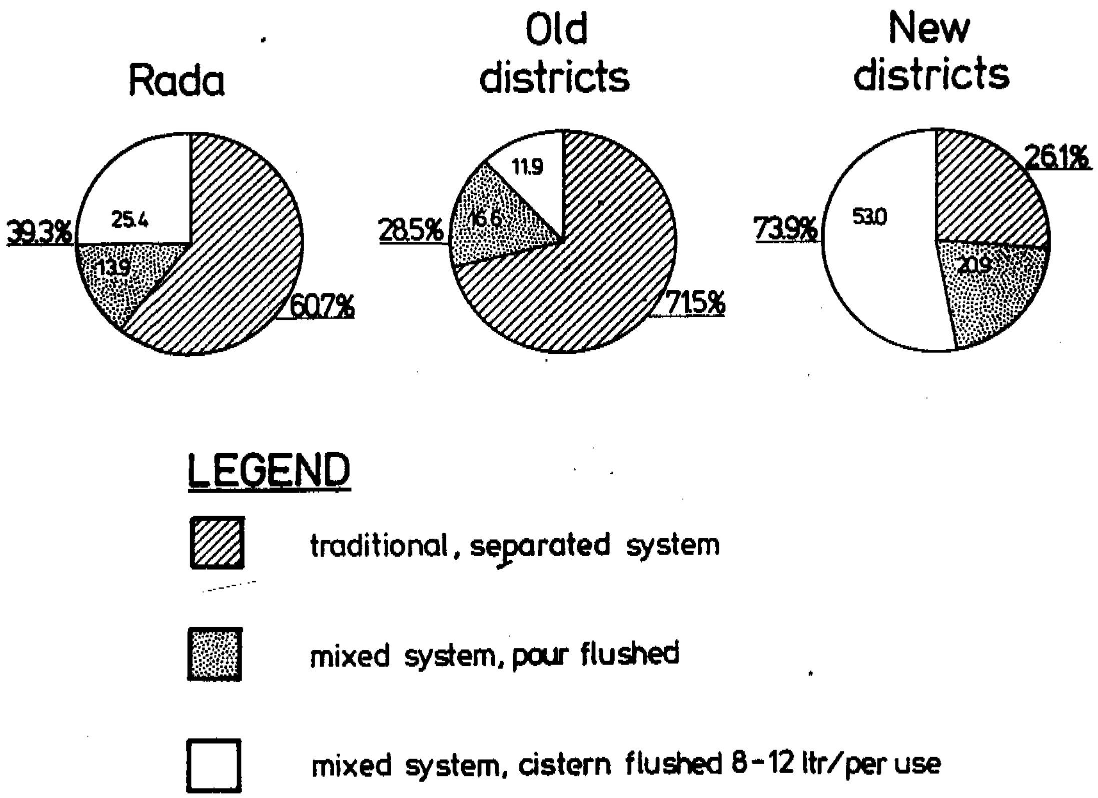

Fig. 3.2. Types of toilets in old and new districts

3.2. Design criteria

3.2.1. Waste water production

The volume of waste water produced is directly related to the number of inhabitants (paragraph 1.3.) and the water consumption as mentioned in paragraph 2.3.

The waste water production is based on the assumption that 80% of the consumed water will be discharged as waste water.

Table 3.2. Types of toilet (per district)

District	System: Separated	System: Mixed	Mixed system: Pour flush	Mixed system: Cistern flush
1	76.7%	23.3%	50.0%	50.0%
2	85.7%	14.3%	40.0%	60.0%
3	62.1%	37.9%	45.5%	54.5%
4	22.6%	77.4%	45.8%	54.2%
5	82.9%	17.1%	33.3%	66.7%
6	23.1%	76.9%	30.0%	70.0%
7	75.8%	24.2%	12.5%	87.5%
8	68.0%	32.0%	62.5%	37.5%
9	12.9%	87.1%	22.2%	77.8%
10	29.4%	70.6%	41.7%	58.3%
11	30.0%	70.0%	14.3%	85.7%
12	93.1%	6.9%	0.0%	100.0%
Total	60.7%	39.3%	35.4%	64.6%

3.2.2. Peak factor

Because of the characteristics of waste water discharge, the maximum hourly discharge cannot be derived directly from the water supply parameters. To calculate the maximum hourly discharge from the residential and commercial areas, a peak flow factor P is used, for which the following formula applies:

$$P = 1.5 + 2.5/\sqrt{q}$$

in which q = average hourly flow of waste water, expressed in l/s. (For some more details about the application of this formula see Annex B2).

As can be seen from this formula and from its application in Table 3.5., the value of P decreases with increasing waste water flow, which is in accordance with investigations. The formula is widely applied and gives reliable results.

For small-diameter pipes with less than about 2000 persons connected, the above formula is replaced by the relation presented in Fig. 3.3. The minimum flow taken into account is 2 l/s, being the discharge capacity of a flush toilet. In the sewer system the flow will decrease due to storage in the pipes. On the other hand the discharge of a toilet will be combined with the flow originating from other parts of the sanitary system, thus more or less compensating for this decrease in flow.

3.2.3. Minimum diameters of sewers

In sewer systems different types of sewers can be distinguished, viz. primary, secondary and tertiary sewers.

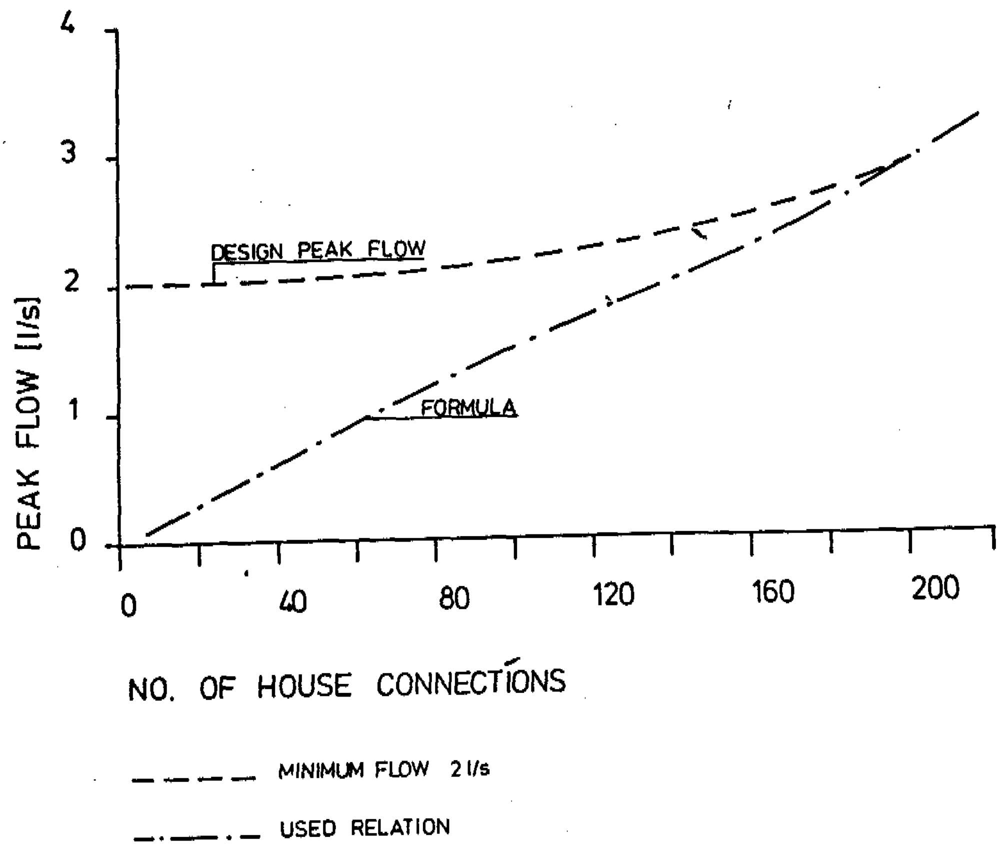

Fig. 3.3. Sewerage peak flow vs. number of house connections

Tertiary sewers are mainly used for house connections. They collect the sewage and then transport it to the secondary sewers. These, in turn, transport the sewage from a small area to the primary sewer system, which carries it to the sewage treatment plant.

In general the tertiary sewers of a waste water collection system have a diameter of 110 mm (see also paragraph 3.5.1.). To prevent blocking, the primary sewers must have a minimum diameter of 200 mm.

For reasons of traffic loads and sufficient slope for secondary sewers, the depth of primary sewers is based on a minimum soil cover of 1 m.

3.2.4. Hydraulic calculation

For the calculation of the capacity of a sewer, it is common practice to assume uniform flow conditions. The flow can then be calculated with the Chézy formula:

$$Q = A * C * \sqrt{(R * I)}$$

in which:

Q = discharge $[m^3/s]$
C = flow coefficient $[m^{0.5}/s]$
R = hydraulic radius [m]
A = area of the wet cross section $[m^2]$
I = slope of the energy line

The flow conditions in rather rough sewer conduits are of the turbulent type. Consequently, the flow coefficient C equals:

$$C = 18 * \log (12 * R/k)$$

in which k = wall roughness [m].

For sewer systems generally a wall roughness of 1.5 mm is used, which takes into account the hydraulic head losses by manholes.

By using these equations the discharge Q in a completely filled sewer can easily be calculated. In case of partly filled conduits the discharge depends on the depth of flow. In such circumstances uniform flow implies that the slope of the water level is the same as that of the sewer itself.

The calculation of the primary system will be based on the peak flow in such a way that the peak flow does not exceed 60% of the full hydraulic capacity of the pipe. To establish the minimum diameters required, the ratio between peak flow and the full capacity of the pipe has been calculated, depending on the number of houses/persons connected, and the slope of the pipe. The results of that calculation are shown in Table 3.3.

3.2.5. Slope of sewers

A certain slope of the sewers is required to create sufficient hydraulic capacity. Waste water contains a large amount of solid matter that has to be transported. To create an adequate transportation capacity (self-cleansing conditions) the following equation can be applied for non-cohesive material:

$$tau = rho * g * R * I$$

in which:

tau	= shear stress	$[N/m^2]$
rho	= density of the sewage	$[kg/m^3]$
g	= gravitational acceleration	$[m/s^2]$
R	= hydraulic radius	[m]
I	= slope of the energy line	[m/m]

For waste water sewers of a separate sewer system[2] the critical shear stress is about 1 N/m^2.

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[2] i.e. separate discharge of storm water by other means

Table 3.3. Capacity of secondary sewers

Persons	500	700	1000	1500	2000	2500	3000	4000	5000
Houses	75	100	150	220	295	370	440	590	735
Peak flow [l/s]	2	2.2	2.4	3.2	4.3	5.3	6.4	8.3	10.1

Slope 1: Q_{peak}/Q_{full} in % Diameter 110 mm

50	28	30	33	44					
75	34	37	40	54					
100	39	43	47	62					
150	48	52	57	76					
200	55	61	66	88					
250	62	68	74	98					

Slope 1: Q_{peak}/Q_{full} in % Diameter 160 mm

50	5	10	11	14	19	24	29	38	45
75	6	12	13	18	23	29	35	47	55
100	7	14	15	20	27	34	41	54	64
150	8	17	19	15	33	41	50	66	78
200	10	19	21	29	38	48	57	76	90
250	11	21	24	32	43	54	64	85	-
350	13	25	30	38	51	63	76	-	-
500	15	30	34	45	61	76	91	-	-

Slope 1: Q_{peak}/Q_{full} in % Diameter 200 mm

50	3	5	6	8	11	13	16	21	25
100	4	7	8	11	15	19	22	30	35
150	5	9	10	14	18	23	27	36	43
200	5	11	12	16	21	26	32	42	50
250	6	12	13	18	24	29	35	47	56
300	6	13	15	19	26	32	39	51	61
400	7	15	17	22	30	37	45	59	70
500	8	17	19	25	33	42	50	66	78
750	10	20	23	31	41	51	61	81	96
1000	12	24	26	35	47	59	71	94	-

As mentioned earlier, the sewers are designed for peak flow conditions. Therefore the minimum design flow of 2 l/s for 110 mm pipes is grossly exaggerating for the actual to be expected flows. The storage capacity of a 110 mm line is 3/4 l/m; i.e. even in cases two toilets are flushed in neighbouring houses at the same moment, no problems will be envisaged in the sewerage collection network. Peak flow is also used to determine the slope of the sewer that should produce the required shear stress.
Especially in case of small volumes of water (thus small diameters and a small hydraulic radius) the slope of the sewers must be steep, which may cause deep excavations and consequently high investment costs.
In practice peak flows occur only occasionally, which implies that in other periods the critical shear stress is not reached. During the periods of lower flows siltation may occur. Removal of these particles requires a much higher shear stress than that needed to prevent siltation. This implies that also in sewers that are designed to be self-cleansing, deposits will occur. Fortunately these deposits are set into motion again by small hydraulic disturbances caused by individual discharges of small duration (e.g. toilet flushing). Test executed for a study on behalf of the Netherlands Government, showed clearly that in 110 mm lines - due to hydraulic forces - solid wastes are transported on a considerable larger distance, than was the case for 125 mm or 160 mm lines. Pipelines of 125 mm or 160 mm for house connections will have no lesser maintenance problems as 110 mm lines.
It was also concluded from these tests that the slope for house connections should be between 1:50 and 1:200.

3.2.6. Manholes

At each intersection and change of direction manholes are required for construction and maintenance purposes. In addition, manholes are required every 50 - 60 m for inspection and maintenance of the sewers. In narrow streets (max. 75 meter length) no inspection chambers are required in the middle of the street, except for a big manhole at the entrance and eventually one half-way of such a street.

It is very important that the profile of the sewer is continued in the manholes, to reduce hydraulic losses and prevent clogging. The minimum size of the manholes must be the pipe diameter plus 0,5 m with a minimum of 0.90 m. To increase the corrosion resistance of the concrete manholes a coating of epoxy tar will be applied.

3.2.7. Materials

For the construction of a waste water disposal system concrete, PVC or glazed stoneware pipes are normally applied. The cheapest and most corrosion resistant material is PVC. This material has therefore been used in the design and cost estimates. The manholes are constructed in reinforced concrete; their covers are of ductile iron.

3.2.8. Pollution strength of the waste water

It is assumed that 1 p.e.[3] in Yemen equals 50 g BOD per day, 11.7 g Nitrogen (N_{Kj}) per day and 50 g suspended solids (SS) per day. The pollution strength of the waste water can thus be calculated as follows:

Table 3.4. Pollution strength of waste water

		1995	2010
BOD	[mg/l]	1136	812
N_{Kj}	[mg/l]	266	190
SS	[mg/l]	1136	812

3.2.9. Waste water temperature

For the design of the sewage treatment plant an average daily waste water temperature has been taken into account of 15 °C in winter, and 25 °C for the summer period.

3.3. Waste water production

The waste water production for Rada is shown in Table 3.5. (over-all figures) and Table 3.6. (waste water production per district). The districts are the same as shown in Fig. 1.3.

[3] population equivalent, i.e. the pollution load produced by one person under average conditions

Table 3.5. Waste water production (over-all)

Year		1995			2010	
Total population		50,000			75,000	
	[m^3/day]	[m^3/h]	[l/s]	[m^3/day]	[m^3/h]	[l/s]
WATER CONSUMPTION:						
Average consumption	2,613	109	30.2	5,775	241	66.8
Max. day consumption	3,135	131	36.3	6,930	289	80.2
WASTE WATER PRODUCTION:						
Production = 80% of water consumption Percentage served = 90%						
Average production	1,881	78	21.8	4,158	173	48.1
Peak factor		2.04	2.04		1.86	1.86
Peak production		160	44.3		322	89.5

Table 3.6. Waste water production, per district

District No.	Surface	Population 1995	Perc. served	Sewage production[5]	Population 2010	Perc. served	Sewage production[4]	average	peak
	1995				2010				
	[ha]		[%]	[m^3/h]		[%]	[m^3/h]	[l/s/ha]	[l/s/ha]
1	25.0	5,264	100	9.2	6,500	100	16.7	0.19	0.34
2	13.5	3,705	100	6.5	4,000	100	10.3	0.21	0.39
3	13.8	3,159	100	5.5	3,400	100	8.7	0.18	0.33
4	19.8	3,331	100	5.8	3,600	100	9.2	0.13	0.24
5	8.5	3,521	100	6.1	3,650	100	9.4	0.31	0.57
6	12.5	1,436	100	2.5	1,550	100	4.0	0.09	0.16
7	20.9	3,561	100	6.2	3,850	100	9.9	0.13	0.24
8	18.4	2,642	100	4.6	3,050	100	7.8	0.12	0.22
9	30.2	3,690	100	6.4	5,000	100	12.8	0.12	0.22
10	27.6	4,680	100	8.2	5,450	100	14.0	0.14	0.26
11	18.7	2,826	100	4.9	4,100	100	10.5	0.16	0.29
12	17.8	3,159	100	5.5	3,400	100	8.7	0.14	0.25
20	102.3	2,211	40	1.5	7,000	75	13.5	0.04	0.07
21	13.3	332	40	0.2	1,050	85	2.3	0.05	0.09
22	86.5	1,769	40	1.2	5,600	75	10.8	0.03	0.06
23	44.2	1,105	40	0.8	3,600	75	6.9	0.04	0.08
24	102.0	2,763	40	1.9	8,800	65	14.7	0.04	0.07
25	16.4	442	40	0.3	1,400	85	3.1	0.05	0.10
Cem.[6]	20.0	-	-	-	-	-	-	-	-
TOTAL:	611.4	49,594	90	78.0	75,000	90	173.0	0.08	0.24

	1995		2010	
Average waste water production:	78	m^3/h	173.3	m^3/h
	1881	m^3/day	4158	m^3/day
Peak factor:	2.04		1.86	
Maximum waste water production:	160	m^3/h	322	m^3/h
	44.3	l/s	89.5	l/s

[4] based on a water consumption of 70 x 1.1 l/c/d, 80% waste water production, and 100% of the population served by the water supply system.

[5] based on a water consumption of 50 x 1.1 l/c/day, 80% waste water production and 95% of the population served by the water supply system.

[6] cemeteries

3.4. Description of the new waste water disposal system

3.4.1. General lay-out

The lay-out of the new waste water disposal system for Rada is given in Fig. 3.4. It shows the area covered by the sewerage system as well as the transmission sewer to the waste water treatment plant and the treatment plant itself.

Due to the general west-east gradient of the surface, three primary sewers, running in that direction, collect the water from the tertiary and secondary systems. The sewerage system is primarily a branched gravity system, with alignments and levels based on the results of the topographical measurements and designs made for the drainage works (see chapter 4).

The three primary sewers join in the north-eastern part of Rada, from where a separate transmission sewer transports the sewage under gravity to the waste water treatment plant. This plant is located about 5 km from the periphery of the sewerage area, as shown in Fig. 3.4.

3.4.2. Selection of treatment process

In the Inception Report[7] a comparison has been made between the following options for treating the waste water of Rada:

a. facultative ponds
b. anaerobic ponds, in series with facultative ponds
c. upflow anaerobic sludge blanket (UASB) reactors, in series with facultative ponds
d. oxidation ditch
e. UASB in series with oxidation ditch
f. pre-clarification followed by activated sludge treatment
g. pre-clarification followed by trickling filters

Three more promising options (b, d and e) have been worked out in more detail[8]. Each of these systems was judged on a number of aspects, such as:
- costs
- process stability
- quality of effluent
- complexity of operation
- terrain utilization
- environmental aspects

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[7] Inception Report, Volume I - Main Report, Rada Water Supply and Sanitation Project, July 1988, paragraph 4.4.

[8] Inception Report, Volume 2 - Annexes, Rada Water Supply and Sanitation Project, July 1988, Annex B-5.

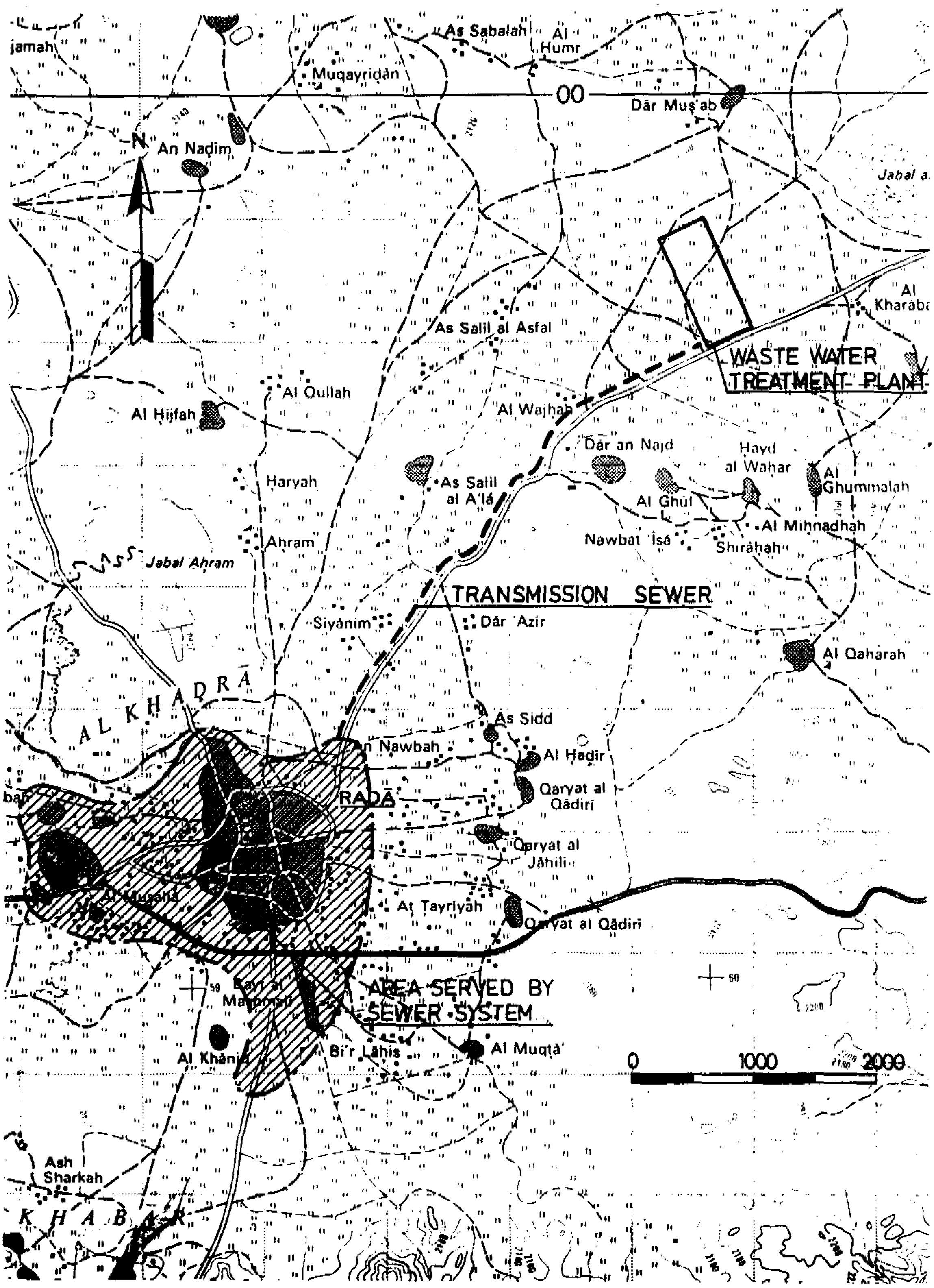

Fig. 3.4. Lay-out of waste water disposal system

it appeared that the investment costs of the anaerobic-facultative pond option (b) would be considerably higher than those of the oxidation ditch option (d). Annual costs of both systems would be about the same. On the basis of process stability and effluent quality, however, there is a preference for the oxidation ditch system. The UASB-oxidation ditch system (e) shows investment and annual costs that are about the same as those of the oxidation ditch option. The system was not selected, however, for reasons of lower process stability and complexity.

Costs of the pond systems are strongly influenced by the cost of sealing the pond bottom. Additional investigations carried out at the potential sites for the waste water treatment plant indicated that bottom sealing would indeed be necessary for the anaerobic-facultative pond option, corroborating the original assumption[9]. The influence of possible re-use of the effluent on system choice was investigated at the same time, again resulting in a preference for the oxidation ditch option, as the higher seepage from a pond system would result in reduced income from the sale of effluent. Moreover, the effluent quality, in terms of BOD, suspended solids and salt content, is better for the oxidation ditch than for the other reviewed systems.
However the yearly O&M costs for ponds are considerably lower as for oxidation ditches (abt. 25-30% for ponds vs. oxidation ditch); also operation is more simple for ponds and lower qualified operation personnel can be used.

The over-all conclusion is, therefore, that for the treatment of the waste water from Rada a lagoon system is the preferred choice.

3.5. Sewerage system

3.5.1. House connections

a. Diameter

The minimum diameter for house connections and tertiary systems constructed in PVC is 110 mm (ND 100). This diameter is internationally wide applied, and due to the limited volumes of waste water (0.8 * 70 l/c/d) in Rada, sufficient for house connections (see also Table 3.3.).

A larger pipe diameter for the house connections/tertiary system will increase the investment cost, not only because of the higher cost of the house connections and tertiary sewers themselves, but also because the minimum diameters of the secondary and primary sewers would have to go up. Good engineering practice in designing a sewerage system is based on the rule that the pipe size has to be increased to at least the next larger diameter when changing from tertiary to secondary, and from secondary to primary sewers. In this way blocking of a sewer (if at all) will occur only in the near vicinity of the connection that causes the blockage.

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[9] <u>System choice for waste water treatment</u>, Rada Water Supply and Sanitation project, March 1989.

Furthermore the maintenance of a 160 mm pipe will not be less than that required for a 110 mm pipe. Laboratory investigations carried out by Consultants show that the transport distance of sediments and discrete particles in a 110 mm pipe is 4 times larger than the distance covered in a 160 mm pipe, under conditions that are otherwise the same. Because of the smaller cross-section of a 110 mm pipe the effective hydraulic pressure as well as the floating depth are larger than in a larger-diameter pipe.

Blocking of house connections may occur by flushing garbage through the toilet. For this kind of eventuality the house connections will be provided with a connection/inspection chamber (see Fig. 3.5).

To minimize blocking and reduce maintenance it is of utmost importance that the alignments of the tertiary sewers are smooth and that smooth materials are used, without sharp angles, etc.

b. Connection of in-house plumbing to tertiary system

There are two types of in-house plumbing systems. One is the type with the outlet pipes connected to the outside of the house, based on the traditional free outflow of liquid waste. The other is the modern type with real in-house pipes, leaving the house below ground level.

The connection between the pipes at the inside (preferred) or the outside of the houses and the tertiary system will be a connection chamber as shown in Fig 3.5.
This connection chamber provides the junction between the vertical and horizontal pipes, which can be of different size and material. To simplify construction, the vertical pipe will be connected to the connection/inspection chamber by a bend. The chamber will be covered with a concrete slab, which can be lifted if maintenance is required.

The project will install the connection chamber, but its maintenance will be the duty of the house owner.

In some cases another type of connection (below surface level) has to be installed, for instance on such locations, where the house connection has to leave the premises/court-yard first.
The boundary between the private sewer line and the NWSA tertiary pipe is an inspection chamber just inside or outside the wall that usually encircles the premises. The private sewer must also be a 110 mm PVC pipe, so the inspection chamber can be as shown in Fig. 3.5. In exceptional cases - only after final approval by NWSA - other types of inspection/ connection chambers may be used (i.e. for instance in line prefab PVC units).

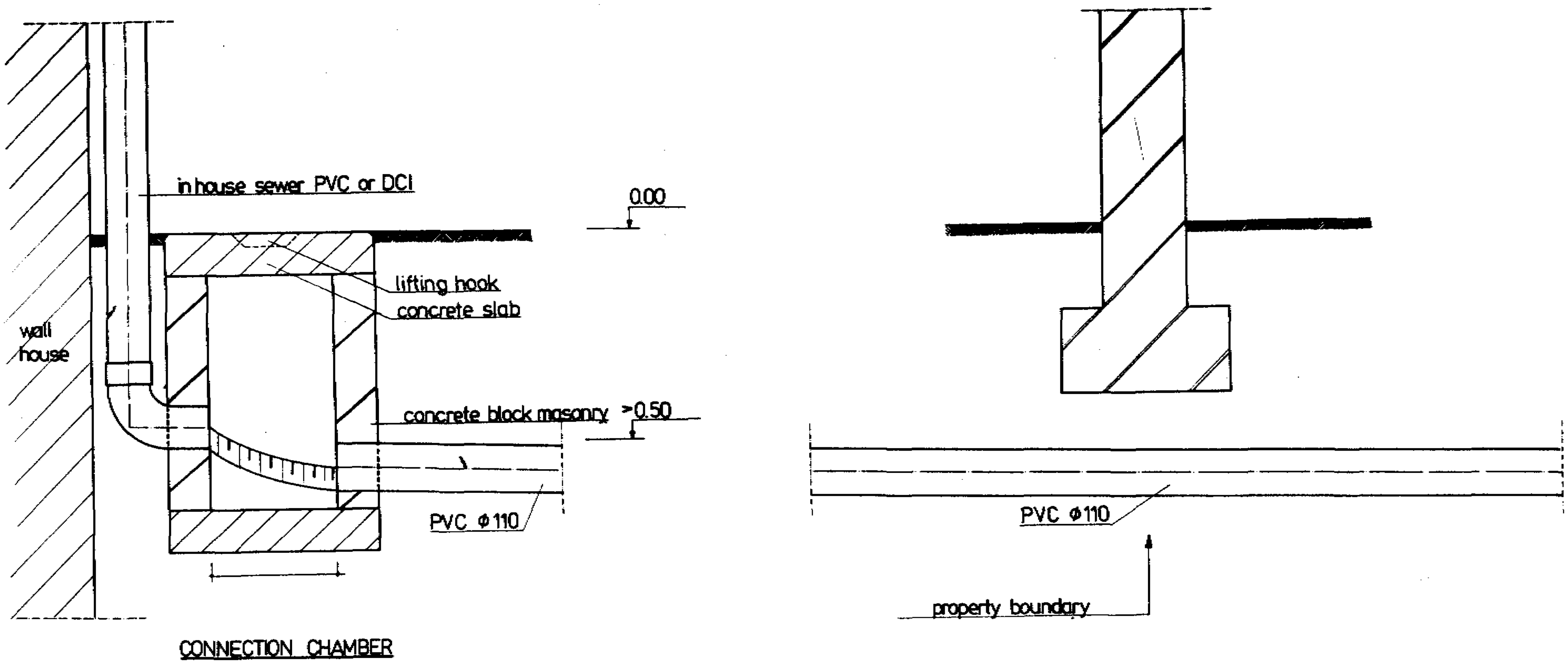


Fig. 3.5. Typical house connection to sewerage system

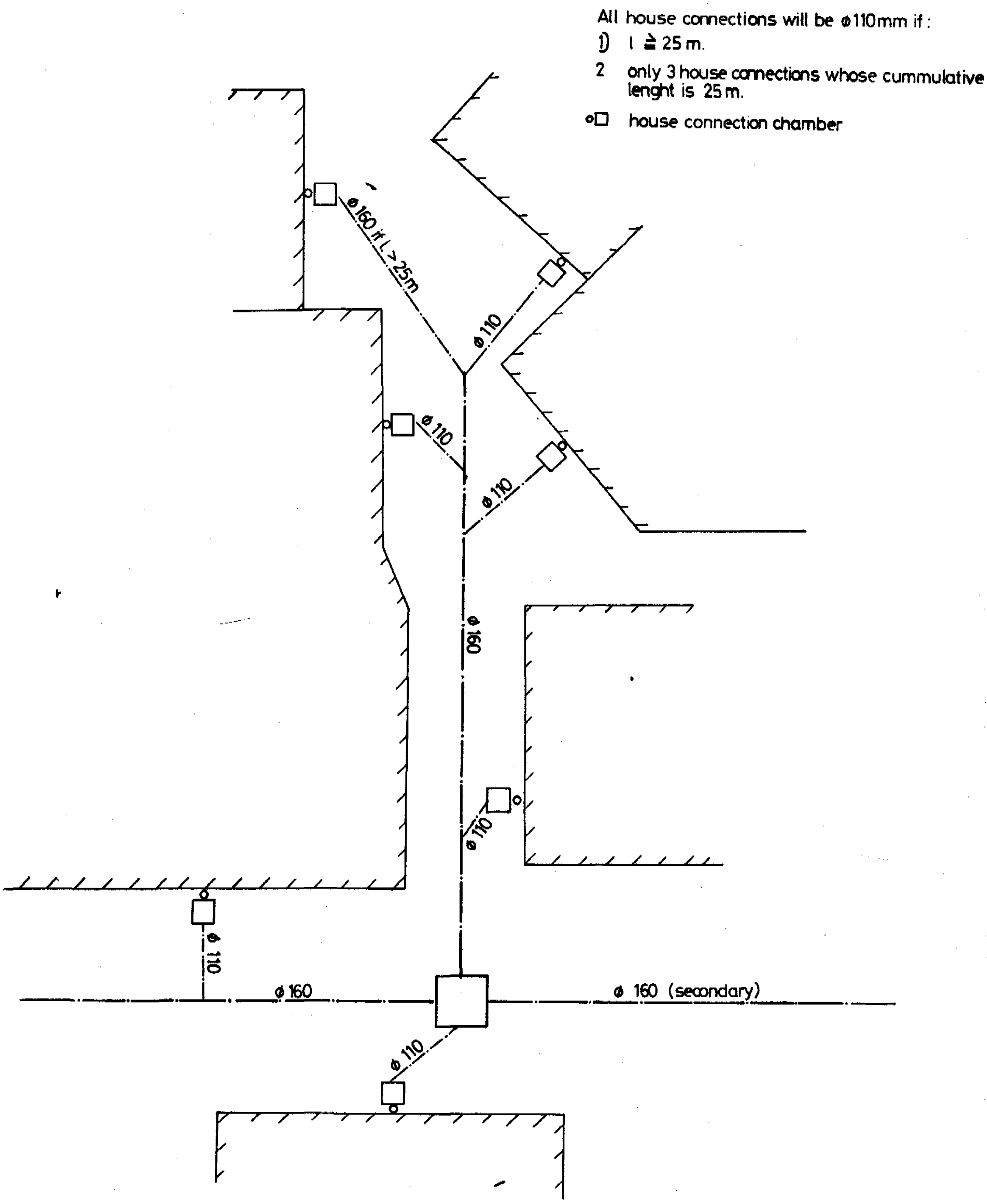


Fig. 36 Typical layout of tertiary system in narrow streets.

3.5.2. Tertiary system

The lay-out of the tertiary system depends on the infrastructure (especially roads) in front of and surrounding the location of the connection.

a. Small street/alleys where cars can hardly or not pass

In this situation the tertiary system will start at the connection chamber of the most remote house. The minimum cover on the tertiary pipe in this kind of streets should be 0.5 m in sandy grounds or a 0.1 m concrete cover in rocky areas. The house connection is made under a horizontal angle of 45^0 with the axis of the street/alley (see Fig. 3.6.).

The horizontal slope of the tertiary sewers is at least 1:50 to 1:200. The length of the tertiary sewer - if 110 mm - will not exceed 25 m.; in case the distance is longer than 25 meter automatically 160 mm diameter pipes will be used. No more than 3 house connections will be connected on one single point at a 160 mm drain, under the condition that the total length of the 3 connections of 110 mm lines will not exceed 25 meter in total. Each house will have its own individual connection/ inspection chamber.

The tertiary sewer is connected to the secondary (ø 160 mm) sewer by means of a manhole (in sandy soils), or with a Y-connection (in rocky soils)(see Fig. 3.7.).

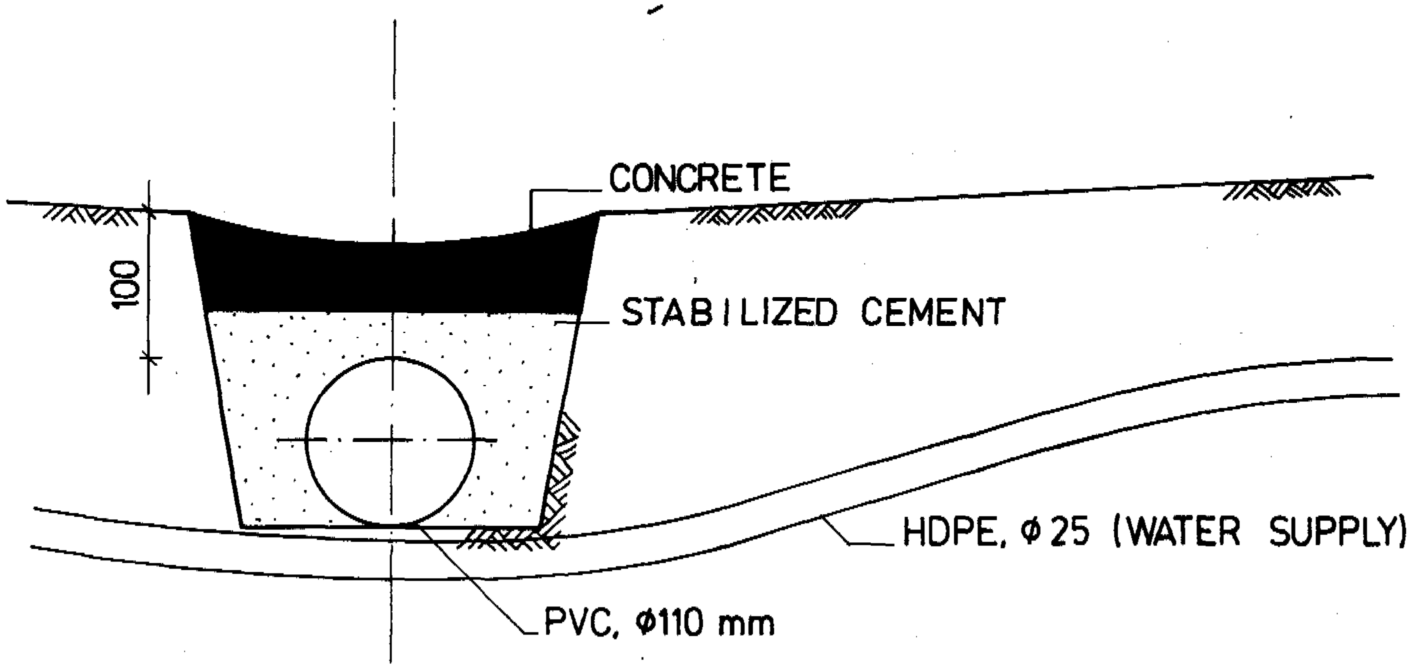
b. Roads suitable for traffic.

The house connection pipe will first leave the premises, with a slope of at least 1:150. Just inside or outside the boundary of the premises an inspection chamber will be installed, as already mentioned in (a). The tertiary pipe with a diameter of 110 mm will continue to the secondary sewer in the street. The cover on top of the tertiary sewer will be at least 0.7 m. The connection between the tertiary and the secondary sewers will be a top inlet or a manhole. When the angle between the axis of the street and the line from the manhole and the house connections is more than 45^0, the connection can be made directly to the manhole, as shown in Fig. 3.8. If not, the connection will consist of a Y-piece and a 45^0 bend. (see Fig. 3.9.).

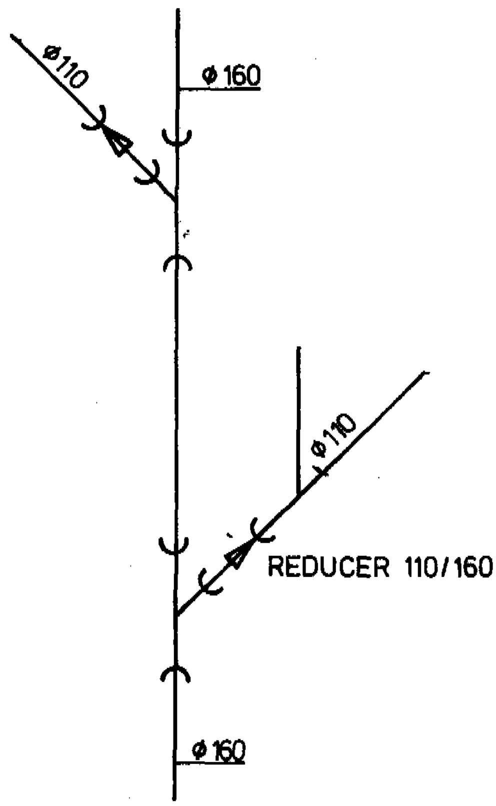
3.5.3. Secondary system

The secondary sewers will have a diameter of 160 mm. Due to the required depth of the house connections, the minimum cover on these sewers will be 0.9 m as shown in Fig. 3.9. The cover will be less in rocky areas, where the pipe will be protected by concrete. The sewers will be laid parallel to the surface. However, a minimum slope of 1:300 must be maintained.
The lay-outs for 4 typical districts are presented in Annex B (Figs. B.2. - B.5.)

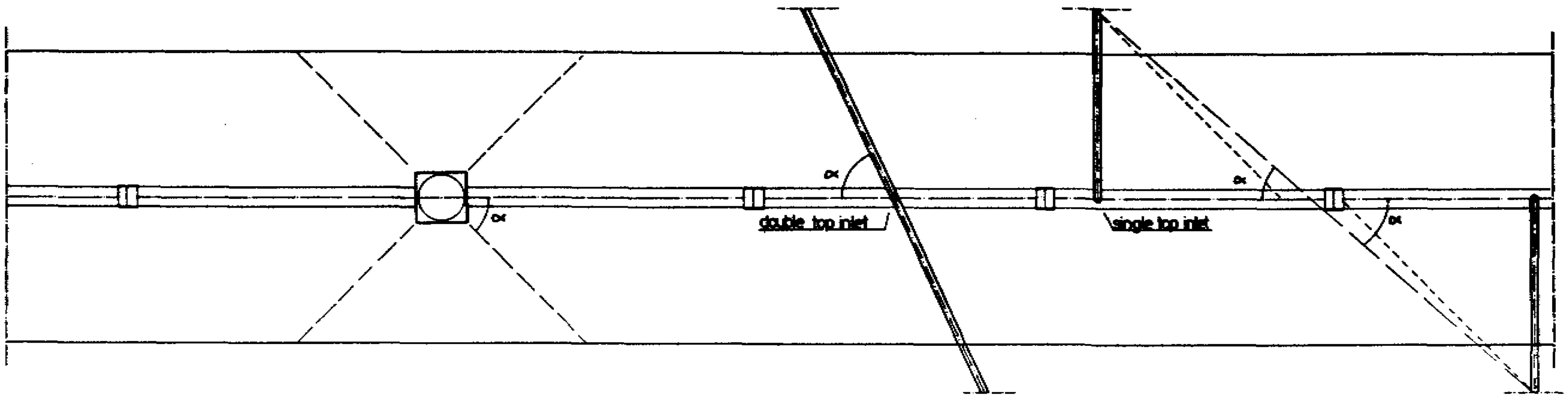
Fig. 3.7. Typical details of tertiary system in rocky soils

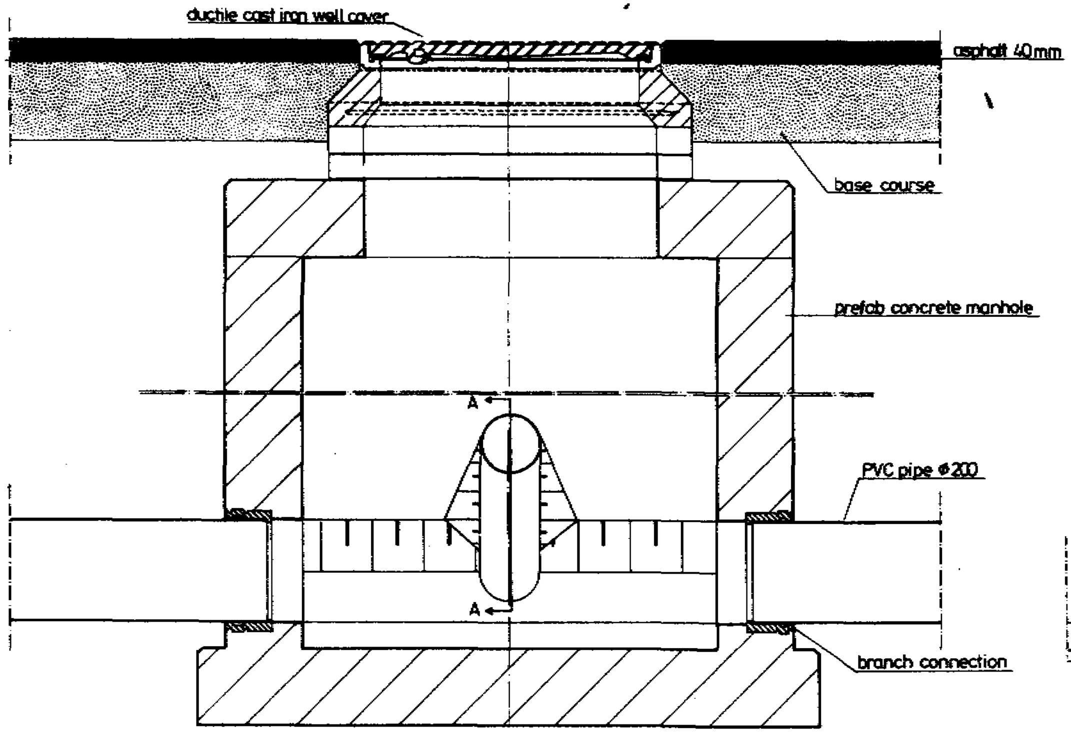


A. CROSS SECTION OVER TERTIARY SEWER

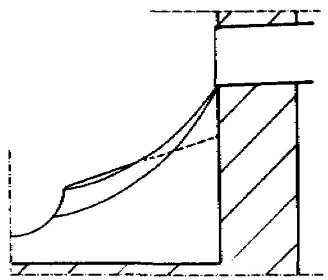


B. CONNECTION BETWEEN TERTIARY AND SECONDARY SEWERS





TYPICAL CROSS SECTION MANHOLE



cross section A-A

Fig. 3.8. *House connection to manhole or top inlet*

description of modification | chkd | drwn | date

first issued HvO 80689 A

RADA WATER SUPPLY and SANITATION PROJECT

YEMEN ARAB REPUBLIC
MINISTRY of ELECTRICITY and WATER
THE NATIONAL WATER and SEWERAGE AUTHORITY
MINISTRY of MUNICIPALITIES and HOUSING

HOUSE CONNECTION TO MANHOLE OR TOP INLET

Euroconsult
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levels in m above 2100m + MSL
dimensions in mm
scale

format A3

drawing no

ref no

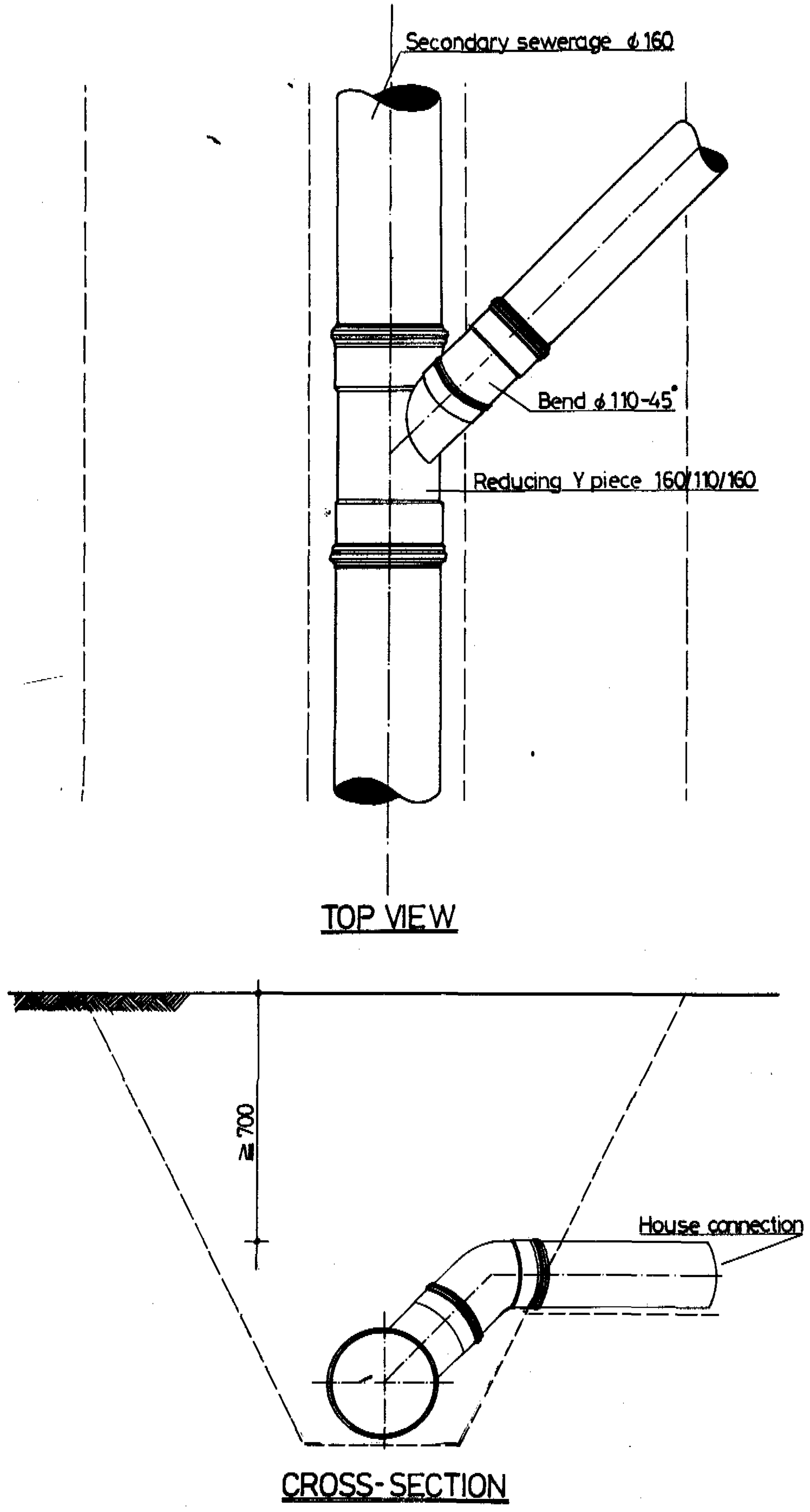


Fig. 3.9. Typical details of house connection on secondary sewer by top inlet

Fig. 3.10. Lay-out of primary sewer system

3.5.4. Primary system

The primary system consists of sewers with diameters of 200, 250, 315 or 400 mm. The lay-out of the primary system is shown in Fig. 3.10.

Because of the general west-east gradient of the surface, three primary sewers have been designed in this direction. The waste water is collected in a collector sewer in the north-eastern part of the project area. This collector sewer transports the sewage to the waste water treatment plant north-east of the town.

The minimum cover on the primary sewers will be 1.25 m, due to the required slope of the secondary sewers.

The calculations of the primary system are shown in Annex B. A diagram of the primary sewer system with node numbers is shown in Annex B, Fig.B.1.

For determining the levels of the sewers, manholes, etc. the street levels resulting from the surface drainage calculations have been taken into account (see Chapter 4).

3.5.5. Low-lying areas

a. General

Within the project area, waste water discharge by gravity is not possible from the northern part of the Harat Al Qana district. A sewage pumping station will be constructed there, to serve about 110 houses in the first phase (year 1995). The required pumping capacity is about 4.2 m^3/h and 9.2 m^3/h for the years 1995 and 2010, respectively.

Two other low-lying areas will be backfilled, so that sewage and surface run-off can no longer accumulate there.

b. Sewage pumping station Harat Al Qana

The situation of the sewage pumping station Harat Al Qana is shown in Fig. 3.11. Details of the pumping station itself are given in Fig. 3.12. the pumping station is equipped with 2 submersible sewage pumps (one operational, one stand-by), each with a capacity of about 9 m^3/h at 5.5 mwc and a power consumption of 0.8 kW. Power will be obtained from the public power grid. For safety reasons (to prevent back-flow of sewage to the houses) the wet pit of the pumping station is equipped with an overflow pipe (ø 160 mm). The sewage is pumped out through a ø 110 mm PVC pressure pipeline.

3.5.6. Breakdown of the required pipe lengths

The pipe lengths required for phase 1 (1995) are shown in Table 3.7.:

Table 3.7. Break-down of required pipe lengths

	Nominal sewer diameter [mm]						
System:	110	160	200	250	315	400	manholes
Primary	-	-	3,915	1,510	1,995	1,550	315
Secondary	-	44,000	-	-	-	-	930
Tertiary	48,000	-	-	-	-	-	5,200[10]
TOTAL	48,000	44,000	3,915	1,510	1,995	1,550	6,445

About 90% of the population served by the public water supply system, (or: 42,750 persons) are expected to be connected to the sewage system in the year 1995. Based on an average of 7 persons per household, about 6,100 private connections should be made by that time, while about 700 commercial and other non-residential connections (mosques, schools, etc.) can be expected.
By RWSSP 4500 house connections will be realized.

[10] inspection chambers

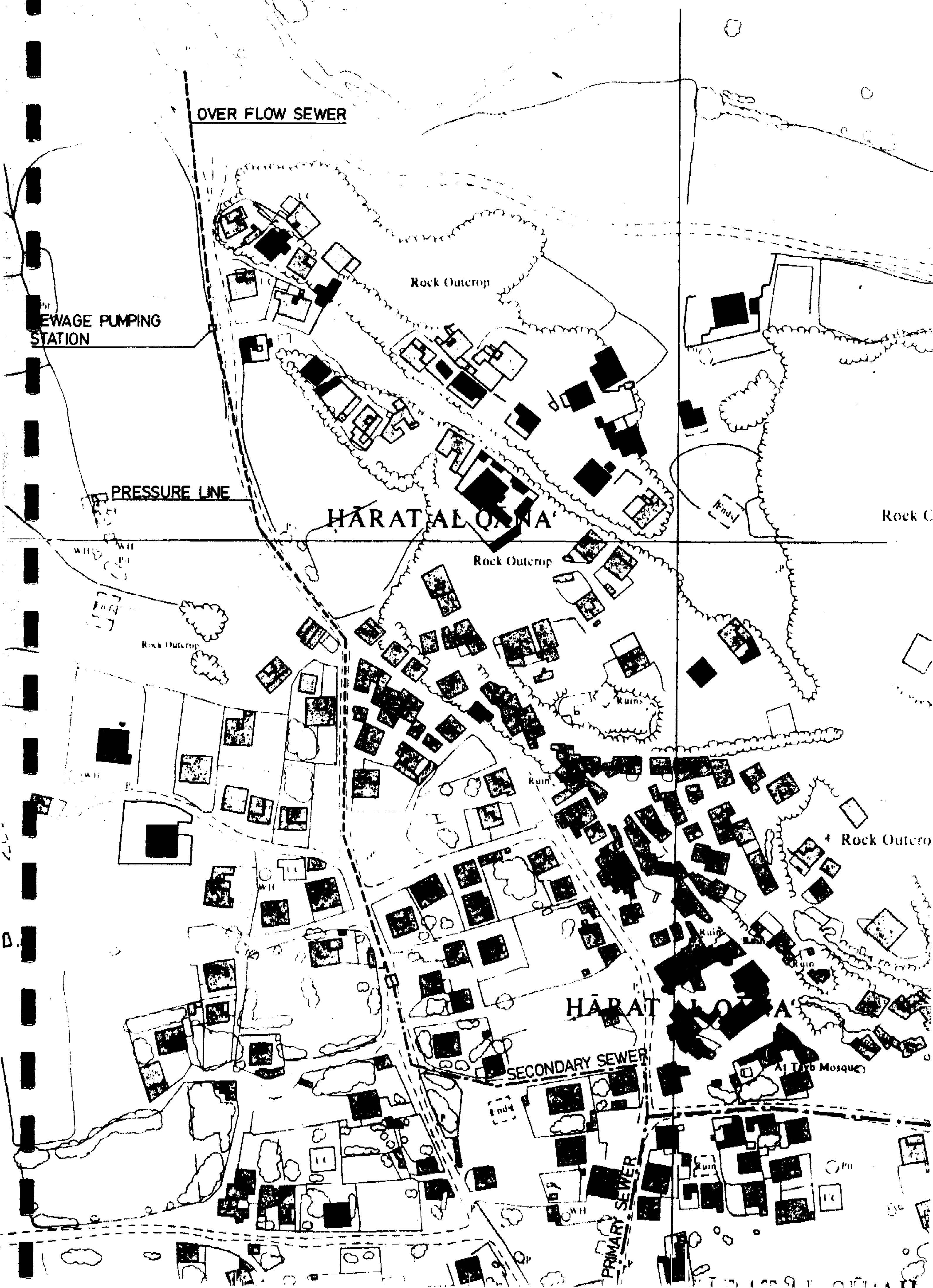
In order to give a more detailed picture on house connections and networks to be realized per district, under the project implementation as envisaged now, the following table is presented:

Table 3.7A <u>Breakdown of required pipe lengths, house connection and manholes for sewerage system per district</u>

BREAKDOWN SEWERAGE

Distr. No.	Area ha.	Connec- tions 1995	Number of manholes primary	Number of manholes secondary	Number of inspection chambers for h.c's + tert. system	Secondary pipes m' 160 mm	Tertiary pipes m' 110 mm	Remarks
1	25	542	23	106	606	3960	5751	Pressure pipe 110,
	--	---	--	---	---	200	350	length 350 m' for
2	13	437	10	94	486	3235	4637	sewer pumping station
3	14	373	10	103	407	4420	3958	Harat al Qana overflow
4	20	393	27	65	453	2545	4170	∅ 160 length 200 m'.
5	9	462	0	53	528	1690	4902	
6	12	311	25	44	341	2805	3300	
7	21	421	20	146	481	6465	4468	
8	18	313	14	52	392	3413	3322	
9	30	393	24	87	452	4605	4170	
10	28	225	31	50	300	2435	2300	
11	19	136	19	40	162	1970	1443	
12	18	437	18	90	522	4283	4637	
20	--	---						
thru	--	47	--	---	70	824	502	
25	--	---						
A	--	---	32	---	---	160	----	along cemetary, near to
district (3+25)								(500+160+500+170): 42
= 32 pc.								
B	--	---	62	---	---	----	----	For transmission sewer
4650 m':								75 = 62 pc.
C	--	---	--	---	---	990	----	Extra next to primary
syst. in								various districts 1 thru
12								
								(310+275+130+275) = 990
m'.								
Total		4500	315	930	5200	44000	48000	

Fig. 3.11. Lay-out sewage pumping station Harat Al Qana



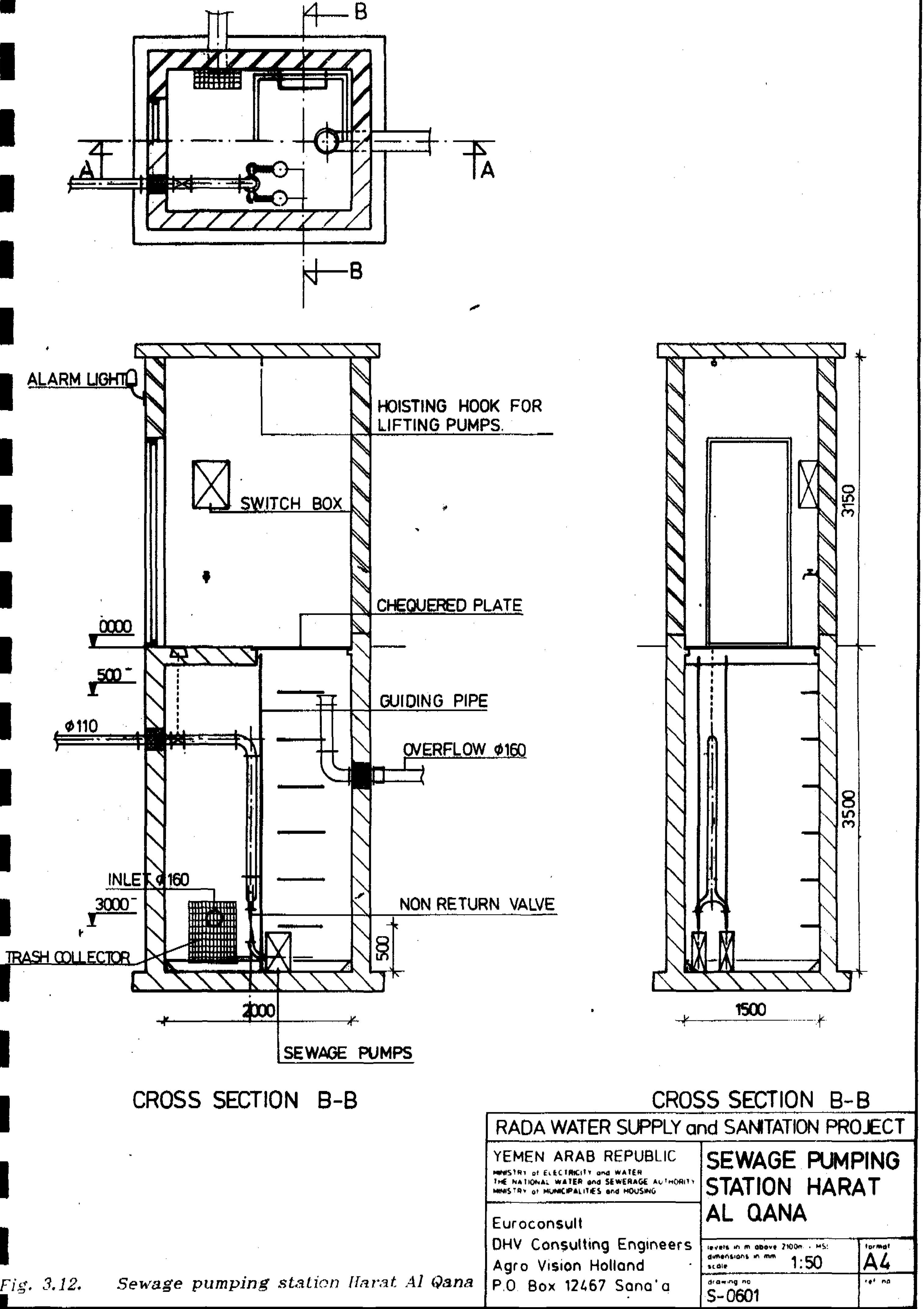


Fig. 3.12. Sewage pumping station Harat Al Qana

3.6. Transmission main to waste water treatment plant

The waste water treatment plant will be located about 5 km north-east of the soccer field, as shown in Fig. 3.4. The surface gradient is 1:250. Consequently the sewage can flow by gravity to the treatment plant. A 4650 m long pipe with a diameter of 400 mm is sufficient to discharge the peak flow of 89.5 l/s in 2010 (hydraulic capacity of pipe amounts to 127 l/s).

The pipe alignment and longitudinal section over the transmission sewer are shown in Fig. 3.13., with manhole details given in Fig. 3.14. The last section of the transmission sewer will be provided with bolted manhole covers, to prevent the covers lifting off in case of emergency overflow conditions at the waste water treatment plant.

3.7. Waste water treatment plant

3.7.1. Lay-out and operational set-up

a. Process parameters

The sewage load on the treatment plant is indicated in Table 3.6., and amounts to 1,881 m^3/day for the first phase, and 4,158 m^3/day for the second phase (or: 160 and 322 m^3/h, respectively). The corresponding BOD loads are 2138 kg/day and 3375 kg/day, N_{Kj} loads are 500 and 790 kg/day, and suspended solids loads again 2138 kg/day and 3375 kg/day, for phases 1 and 2, respectively.

Process parameters for the waste water treatment process are indicated for the design horizons 1995 and 2010, as shown in Table 3.8.[11]:

<u>Table 3.8. Process parameters, waste water treatment</u>

	1995	2010
population served	42,750	67,500
flow rate, average [m^3/d]	1,881	4,158
flow rate, maximum [m^3/h]	160	322
BOD [kg/d]	2,138	3,375
SS [kg/d]	2,138	3,375
N_{Kj} [kg/d]	500	790

[11] see also Inception Report, Volume 2 - Annexes, Annex B-5, Rada Water Supply and Sanitation Project, July 1988

b. Major system components

The waste water treatment plant has the following components:

- screens
- flow rate measurement
- anaerobic ponds
- facultative ponds

The plant will be operated under gravity, without the use of pumps.

The sludge from the anaerobic ponds is stored and dried in a fourth anaerobic pond.

c. Phasing

The waste water treatment plant, which is a combination of anaerobic and facultative ponds, will be constructed in a phased manner, the phases corresponding with the design horizons 1995 and 2010. A minimum set-up will be constructed straight away, to cater for the 1991 situation.

The anaerobic pond system will be constructed directly for the design horizon 2010, whereas the facultative ponds will be constructed in phases.

The lay-out of the waste water treatment plant is indicated in Fig. 3.15., with the planned extension shown in dotted lines.

A diagrammatic representation of the waste water treatment components, for phases 1 and 2, is given in Fig. 3.16 and 3.17, respectively.

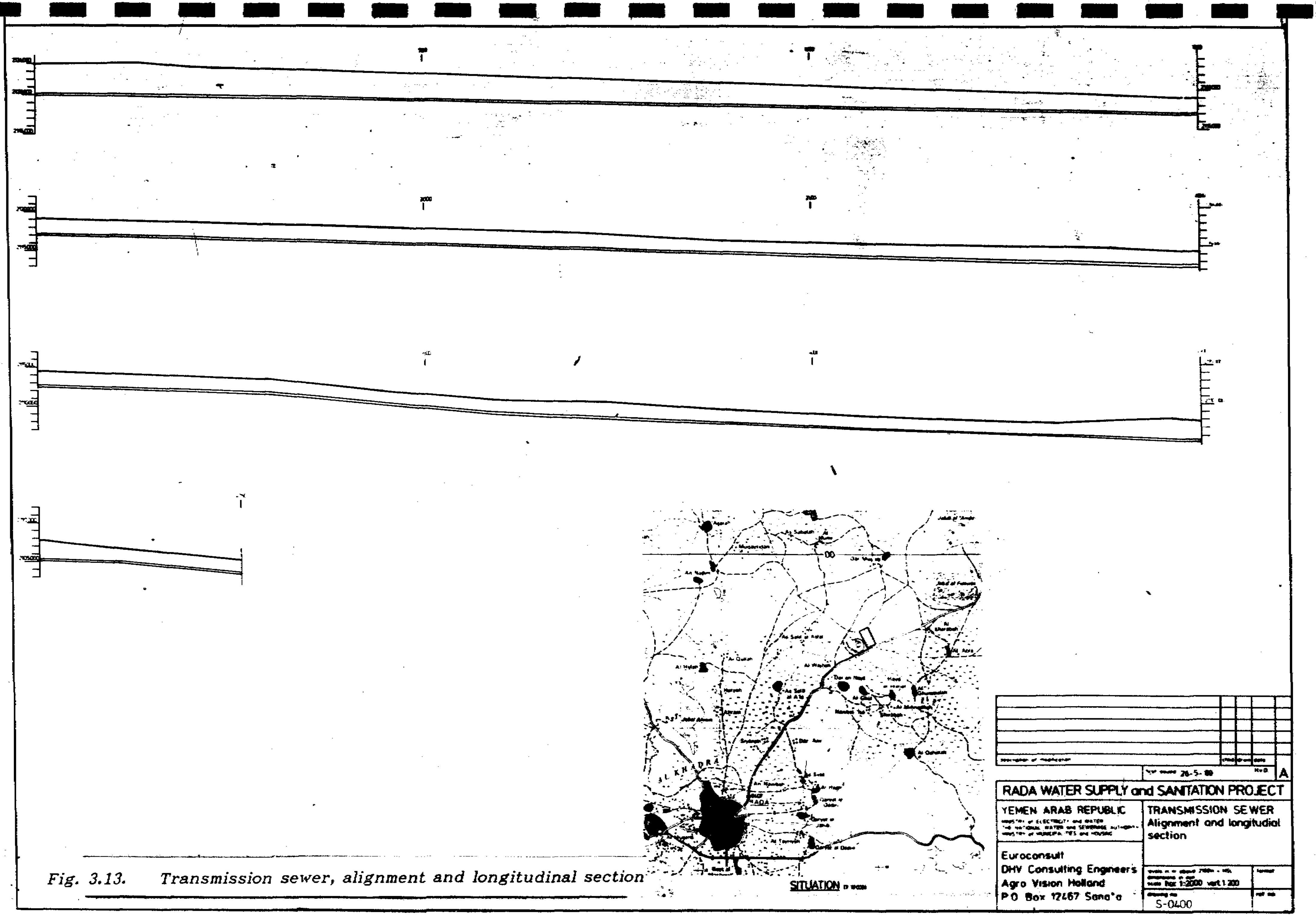
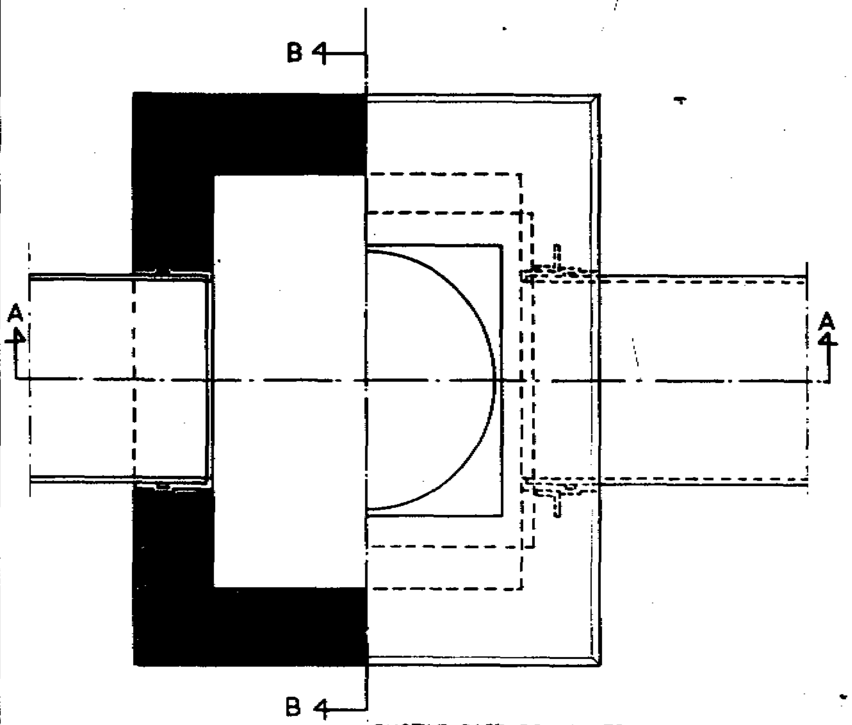
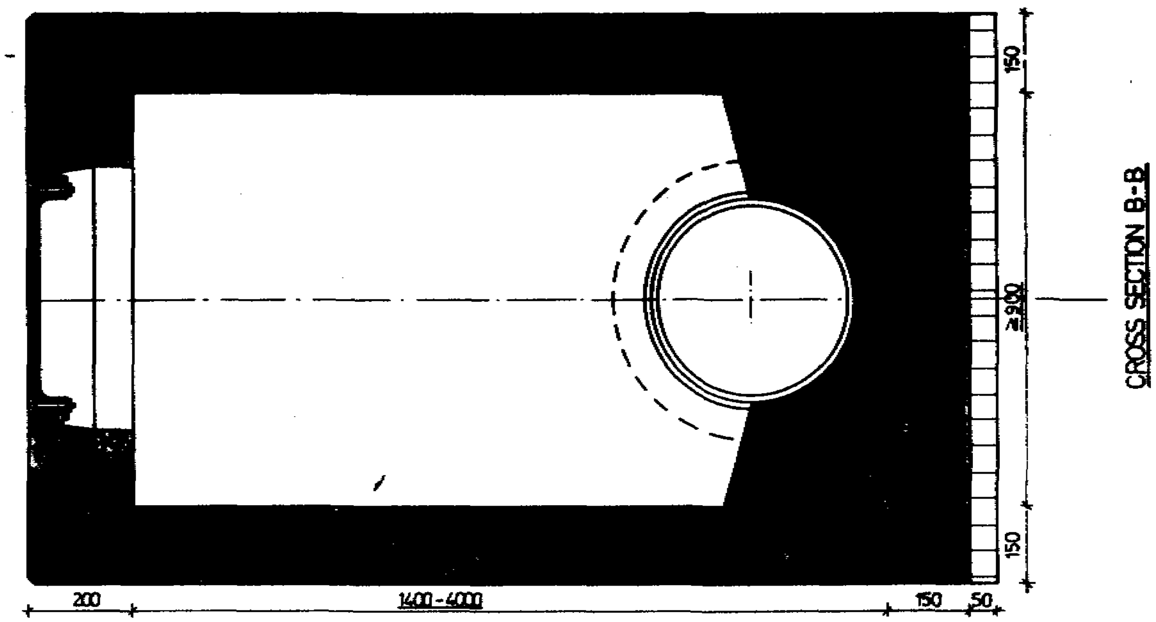
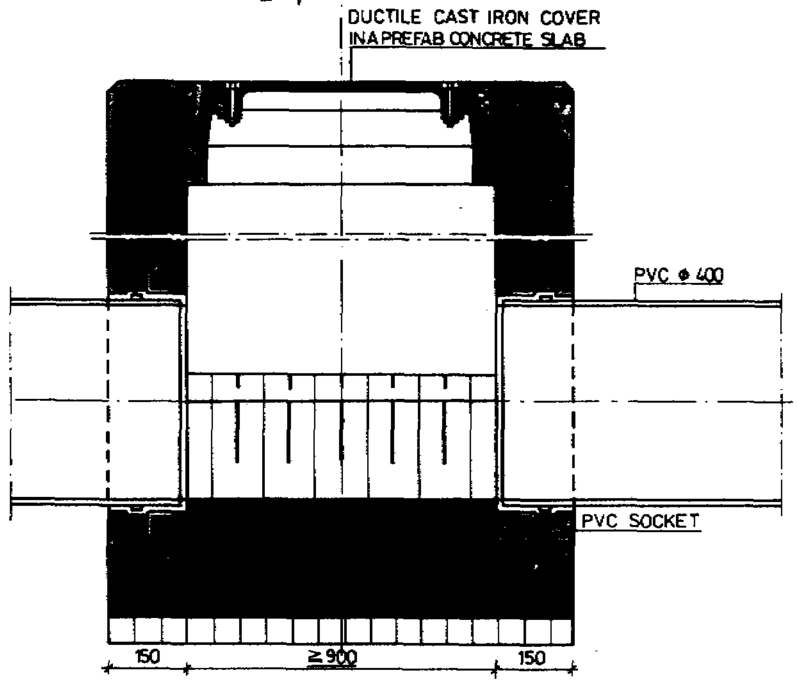


Fig. 3.13. Transmission sewer, alignment and longitudinal section







CROSS SECTION A-A

Fig. 3.14. Typical manhole in transmission sewer

description of modification	chkd	drwn	date	
Manhole cover		JLv0	11-12-89	B
first issued 21-5-'89		JL.		A

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MANHOLE TRANS-MISSION SEWER

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levels in m above 2100m - MSL
dimensions in mm
scale 1:10

format A3

drawing no: S-0408

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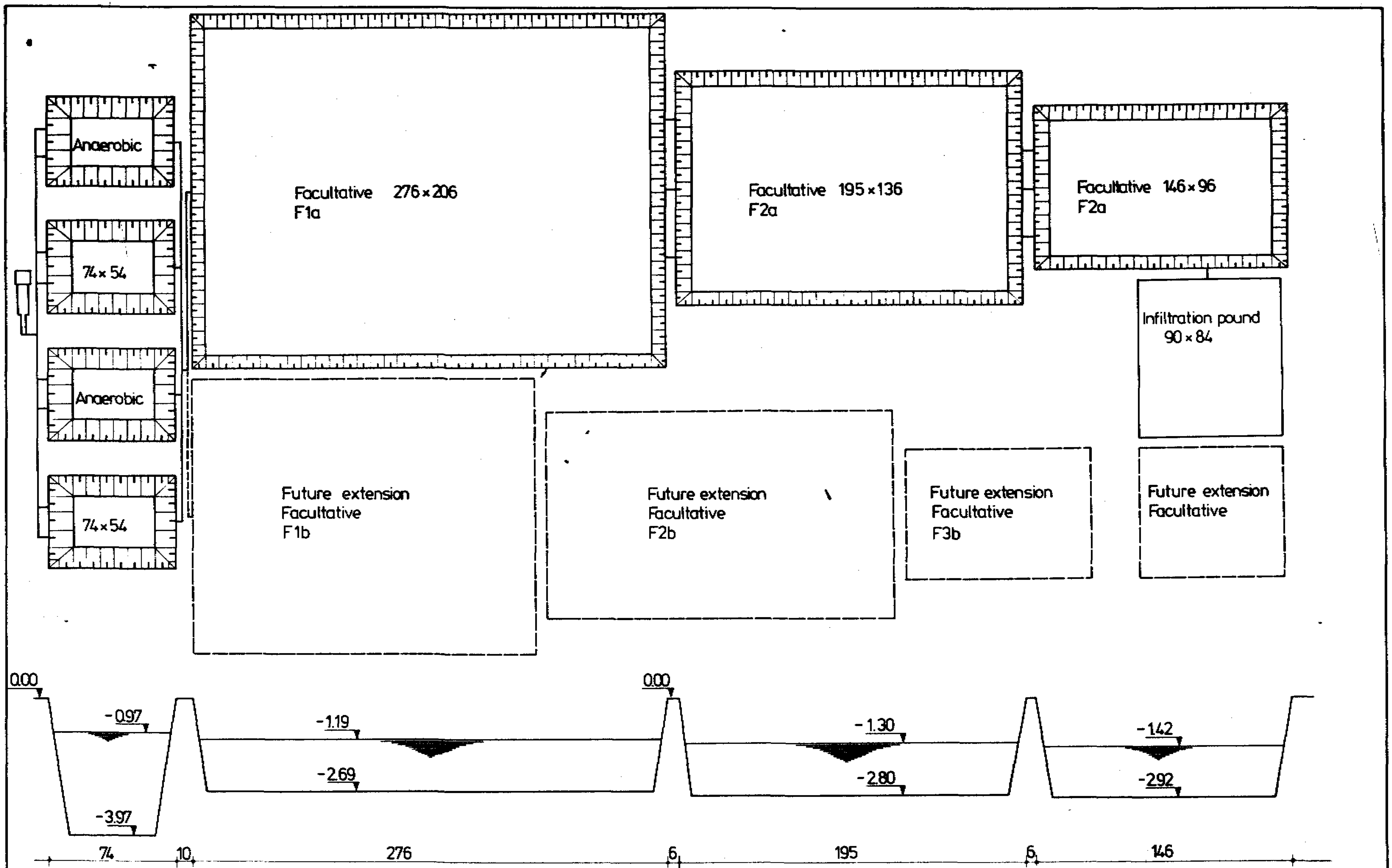

Fig. 3.15.

Waste Water Treatment plant : Layout and longitudinal profile

0 50 100 m.

dimension in meters
scale 1: 2000 / 1: 100
date 041289
drwn

3.7.2. Screens

To protect the piping in the plant from obstruction and to avoid large floating matter from entering into the ponds, a screen is required, with characteristics as given in table 3.9. Two parallel screens will be installed.

Table 3.9. Characteristics of the screen

clearance	[mm]	20
flow rate	[m3/h]	350

3.7.3. Flow rate measurement

For measuring the flow rate a Venturi channel is incorporated in the influent line, with a hydraulic capacity of 350 m^3/h.

3.7.4. Anaerobic ponds

a. Design

The design criteria applied for the anaerobic ponds are:

- hydraulic retention time	[days]	5
- depth	[m]	3
- BOD removal efficiency	[%]	50

The dimensions of the ponds are given in table 3.10.:

Table 3.10. Dimensions of the anaerobic ponds

total volume	[m^3]	20,800
number of units	[-]	3
volume per unit	[m^3]	6,930
lengt	[m]	58
width	[m]	40
depth	[m]	3

The ponds are laid out for the 2010 design horizon. This implies that they are underloaded in the 1995 situation. It is expected, however, that this underloading will not cause any problems.

To ascertain an equal flow, 2 inlets are needed per unit. Thus, a splitter box with 6 outlets is required, 2 for each pond. To take a pond out of operation for desludging it must be possible to close off the waste water supply to each pond. While one pond is being desludged, the other two remain in operation.

b. Sludge removal

To prevent having to use sludge pumps, for pumping sludge from the anaerobic ponds into a sludge lagoon, the solution has been selected whereby sludge is dried in the anaerobic pond itself.

In this case 4 ponds are required, each designed for 1/3 of the total load, as indicated in Table 3.10. Normally, three ponds will be in operation. When a ponds has become full of sludge, it is taken out of operation and the fourth pond is put into operation. The pond that is out of operation is drained and the sludge allowed to dry. After the sludge has dried, it is removed, and the pond is ready for operation again. All ponds will be provided with a drain system, to allow rapid draining of the ponds and dewatering of the sludge.

Because of the need for a fourth pond, the splitter box must be provided with 8 outlets (2 for each pond).

The ponds must be accessible to a shovel/dozer, for removing the sludge after drying.

c. Temporary operation mode

To cater for the 1991 situation, initially only one anaerobic pond and one facultative pond will be put into operation.

d. Structural aspects

Inlet

The inlet of the raw waste water into the anaerobic pond must be:
- equally distributed along the embankment, the distances between inlet - points not exceeding 30 m;
- at a depth of 1.50 m below the surface.

Bottom sealing

Percolation tests have shown the soil percolation to be very high, so that the use of a bottom sealing is required for both the anaerobic and the facultative ponds. Although some tests will still be carried out on the economic feasibility of sand/cement sealing, in principle HDPE foil, covered by a layer of 0.10 - 0.30 m of sand, and supported by a compacted soil, will be applied as bottom sealing material.

The lagoon dikes will be protected by concrete slabs.

Outlet

The outlet is provided with a scum guard that reaches to at least 0.50 m below the surface.

The velocity of the water flowing towards the overflow weir must not exceed 0.05 m/s.

On the basis of these criteria the overflow weir has been given the following dimensions:

- maximum water height at crest	[mm]	50
- width	[m]	2.50
- opening under scum guard	[m]	0.40

3.7.5. Facultative ponds

The facultative ponds are arranged in two series of 3 each, numbered F1, F2 and F3, whereby ponds indicated with 'a' cater for the 1995 situation and those indicated with 'b' have to be added to cater for the 2010 horizon. The 'a' and 'b' series of ponds are operated in parallel.

a. Design criteria.

The design is based on the formula of Mara:

$$BOD_{effluent} = \frac{BOD\ of\ influent}{1 + k_T * t}$$

in which:

k_T = $0.2 * 1.05^{T-20}$ in $days^{-1}$
T = (water) temperature in °C
t = retention time in days

The capacity of the facultative ponds is determined by the waste water temperature in the coldest month. for the city of Rada' this temperature has been estimated at 15°C.

Based on this formula, the total retention time in the system must be 52 days, in order to reach an effluent BOD of 30 g/m^3.

To avoid the risk of anaerobic conditions in the first pond, the organic load should not exceed 200 kg BOD/(ha./day) in the coldest month, as shown in Fig. 3.18[12]. Anaerobic conditions will lead to a reduction in purification efficiency.

The design criteria for the facultative ponds have been summarized in Table 3.11:

[12] McGary, M.G. and Pescod, M.B. (1970) "Stabilization Pond Design Criteria for Tropical Asia", Proceedings 2nd International Symposium Waste Treatment Lagoons, Kansas City, MO.

Table 3.11. Design criteria of the facultative ponds

Total retention time	[d]	52
Depth	[m]	1.50
Loading 1995	[kg BOD/d]	1,069
Loading 2010	[kg BOD/d]	1,690
Number of steps in series	[-]	3
Loading rate in the F1 pond	[kg BOD/(ha.d)]	200

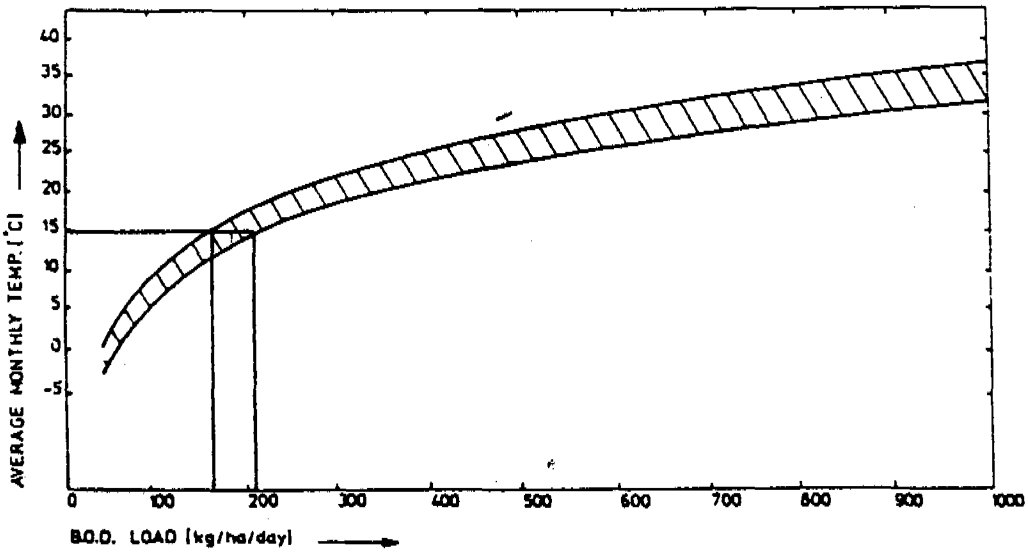

Fig. 3.18. Maximum admissible organic load of the first pond in relation to the waste water temperature

b. Design characteristics

The ponds have the following design characteristics:

volume of ponds	[m^3]	4,158 x 52 = 216,200
total surface of ponds	[ha]	216,200 / (1.5 x 10,000) = 14.41
overall loading rate	[kg BOD/(ha.d)]	1,690/14.41 = 117
surface F1 ponds	[ha]	1690/200 = 8.45
number of ponds	[-]	2
surface F1a pond	[ha]	1069/200 = 5.35
surface F1b pond	[ha]	8.45 - 5.35 = 3.10
length to width ratio	[-]	from 1 : 1 to 3 : 2
removal efficiency	[%]	70
inlet at shortest side		

Surface of other ponds (F2 and F3)	[ha]	14.41 - 8.45 = 5.96
2nd pond	[ha]	2/3 of 5.96 = 3.97
loading rate	[kg BOD/(ha.d)]	(1 - 0.7) * 1690 / 3.97 = 128
surface pond F2a	[ha]	1 - 0.7) * 1069/ 128 = 2.51
surface pond F2b	[ha]	3.97 - 2.51 = 1.46
length to width ratio	[-]	from 3 : 2 to 2 : 1
inlet at shortest side		
surface F3 ponds	[ha]	1/3 of 5.96 = 1.98
surface F3a	[ha]	1.25
surface F3b	[ha]	0.73

Ponds F3a and F3b may be combined to 1 pond F3 of 1.98 ha.

c. Dimensions:

pond F1a		
surface	[ha]	5.35
l : w ratio	[-]	1 : 1 to 3 : 2
length	[m]	230 to 270
width	[m]	230 to 200
depth	[m]	1.50
pond F1b		
surface	[ha]	3.10
l : w ratio	[-]	1 : 1 to 3 : 2
length	[m]	175 to 200
width	[m]	175 to 155
depth	[m]	1.50
ponds F2a		
surface	[ha]	2.51
l : w ratio	[-]	3 : 2 to 2 : 1
length	[m]	195 to 225
width	[m]	130 to 110
depth	[m]	1.50
pond F2b		
surface	[ha]	1.46
l : w ratio	[-]	3 : 2 to 2 : 1
length	[m]	150 to 170
width	[m]	100 to 85
depth	[m]	1.50

pond F3a		
surface	[ha]	1.25
l : w ratio	[-]	3 : 2 to 2 : 1
length	[m]	140 to 160
width	[m]	90 to 80
depth	[m]	1.50
pond F3b		
surface	[ha]	0.73
l : w ratio	[-]	3 : 2 to 2 : 1
length	[m]	100 to 120
width	[m]	70 to 60
depth	[m]	1.50
ponds F3a and F3b combined to F3:		
surface	[ha]	1.98
l : w ratio	[-]	3 : 2 to 2: 1
length	[m]	170 to 200
width	[m]	115 to 100

Scum baffles will be provided only in the last ponds (F3a and F3b). If scum occurs, it must be removed immediately.

The influent will be evenly distributed over the length of the embankment of the pond, the interval between inlet points not being larger than 50 m.

The velocity per inlet point must not be more than 1.0 m/s.

3.7.6. Operation and maintenance

a. Screens

The screens must be raked frequently. It is recommended to rake the screen 4 times per day, viz. every 6 hours.

In addition, the status of the screen must be checked frequently during the day and particularly in periods of heavy rainfall, e.g. once every two hours. Extra raking of the screens is required when they appear to be (partly) obstructed.

It must be noted that the screenings are highly infectious. The staff involved in cleaning the screens must therefore be provided with adequate protection.

The screenings must be stored in a container with a tight fitting lid, since the stored screenings produce a strong smell and will otherwise form a breeding place for insects. Stored screenings must be removed and brought to the garbage disposal site at least once per week.

b. Anaerobic ponds

Desludging must take place when a pond is filled with sludge to approximately half its height. Based on the figures for suspended solids in the incoming waste water (Table 3.8.), a sludge load of 2,138 kg/d for the 1991 horizon and 3,375 kg/d for the 2010 horizon is expected. Taking into account the assumptions mentioned in Table 3.12., the ponds must be desludged once every 2 to 3 years. As there are 3 ponds in operation at any one time, every 10 months one pond will have to be desludged.

Table 3.12. Calculation of the desludging period

design horizon		1995	2010
assumptions:			
SS removal efficiency	[%]	90	90
conversion of TSS	[%]	60	60
TS of sludge	[kg/m³]	100	100
maximum filling	[%]	50	50
calculations:			
solids accumulation	[1000 kg/yr]	281	444
sludge accumulation	[m³/yr]	2,810	4,440
accum. per pond	[m³/yr]	936	1,480
50% of volume of pond	[m³]	3,867	3,867
desludging period	[yr]	4	2.6

c. Facultative ponds

Banks

The banks must be kept clear of vegetation. This means that they must be inspected and the vegetation mowed if it becomes abundant. Control of vegetation by herbicides is not recommended, since these chemicals may affect the biological processes in the pond and may render the effluent of the ponds less suitable for irrigation.
Any damage to the banks must be repaired immediately.

Dikes

Dikes must be protected from rodents. Inspection must take place every month. Holes must be repaired immediately. Rodents can be controlled either by predators or by using poison.

Scum

Scum formation on the F1 and F2 ponds is to be avoided since no scum guard is foreseen for these ponds. If scum occurs, therefore, it must be removed as quickly as possible, since it would prevent light from reaching the algae in the pond, which would result in anaerobic conditions. Scum can best be removed at the lee side of the pond, by using scrapers to pull the scum onto the bank. After drying, the scum can be removed with a lorry.

The F3 ponds are equipped with a scum guard. Therefore, it is expected that scum from the F1 and F2 ponds will accumulate in the F3 ponds. Scum must be removed instantaneously, as described before.

Dredging

In the ponds some accumulation of solids will occur, caused by losses of sludge from the anaerobic ponds, dust bowls and dead biomass. The contribution of the latter two factors is estimated to be 1 to 2 cm per year, which implies that it would take 30 to 50 years to fill the ponds to half of their depth. The contribution of sludge losses from the anaerobic ponds, however, may be much more. For that reason it is recommended to check the depth of the F1 ponds near the inlet once every year. If the depth has become less than 0.75 m, the sediment must be dredged. This can be performed in the same way as for the anaerobic ponds, using a sludge pump on a float. The dredged sediment can be put into the fourth (sludge drying) anaerobic lagoon for drying.

3.7.7. Lagoon system design assessment and consequences

The above design for the lagoon system was checked by a senior expert with a lot of theoretical and practical experience in the design of lagoon systems for semi arid and (semi) tropical countries, both in the Middle - East and South - America. His recommendations, especially concerning the lay-out and size of the maturation ponds (F3) and detailed lay-out of the secondary facultative ponds will be incorporated in the final tender drawings to be approved by NWSA. For budget purposes the recommendations have no consequences.

3.7.8. Re-use of effluent

A pre-feasibility study on the possible re-use of effluent has been carried out, which came up with the following conclusions and recommendations[13]:

- Treated effluent from sewage treatment plants is potentially a valuable source of water. However, certain precautions and restrictions must be observed.
- In many places in the world it has been shown that re-use of effluent for irrigated agriculture can be successful. So far, there has been no experience with the controlled use of effluent for irrigated agriculture in the YAR, however.
- It is recommended that the effluent be used for irrigated agriculture. The seasonal excess of effluent could be stored in a deep storage reservoir for use later in the year.

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[13] Pre-feasibility study on reuse of effluent from waste water treatment plant, Rada Water Supply and Sanitation Project, March 1989.

- For waste water treatment preceding re-use, preference is given to an oxidation ditch over a facultative pond system, for the following reasons:
  - more water will be available for irrigation when using an oxidation ditch, in comparison with a facultative pond system;
  - the water quality from an oxidation ditch will be more suitable for irrigation, because BOD, N_{Kj} and SS levels will be considerably lower than with a pond system. Moreover, the salts content will also be lower.

However it was decided to implement for the waste water treatment of Rada a lagoon system. Therefore an additional study is required on prevention of algae bloom in ponds and re-use of effluent from this lagoon system for agricultural purposes. This study will be carried out by RIRDP (The Rada Integrated Rural Development Project).

- The effluent can still have a high pathogen content. Chlorination of effluent is not recommended as it is often not successful. Preference is therefore given to the use of a deep storage reservoir, having the dual purpose of storing excess effluent and further reducing pathogens.
- Preference is given to a re-use site some 5 km north-east of Rada town, as indicated in Fig. 3.27. (site 3):
  - the area is located sufficiently far from the populated area to reduce health risks associated with the use of effluent from a sewage treatment plant;
  - the area is presently not in use for agriculture, and the design of required irrigation and drainage facilities will not be hampered by existing plot boundaries and fragmentation of holdings;
  - the land is reportedly public property, which will make the development of an irrigation scheme much easier than would otherwise be the case.
- Soils in the proposed effluent re-use area have a high salts content and leaching will be needed to make these soils productive.
  The groundwater table in the area is presently more than 12 m deep and is declining at a rate of approximately 0.5 m /year. Resalination after reclamation is highly unlikely.
- Vegetables and qat will have to be excluded from the cropping pattern in the proposed effluent re-use and agricultural development area. fruit trees, cereals and fodder crops can be grown, however.
- Since controlled use of effluent from sewage treatment plants for irrigated agriculture will be new in Yemen and because land reclamation is required, it is recommended that the scheme be developed and initially managed by the Ministry of Agriculture as a research and demonstration farm. The scheme could be gradually expanded for smallholder farming, once the land has been reclaimed and the required infrastructure established.

Taking the above into account, however, the re-use of effluent, being of a predominantly agricultural nature, is considered to be beyond the scope of the Rada Water Supply and Sanitation Project itself.

To present a feasible solution irrespective of the final decision to be taken regarding re-use of effluent, the design incorporates an effluent infiltration pond. Whenever circumstances would warrant a revision of the way in which the effluent is disposed of, such revision can be incorporated in the design, however.

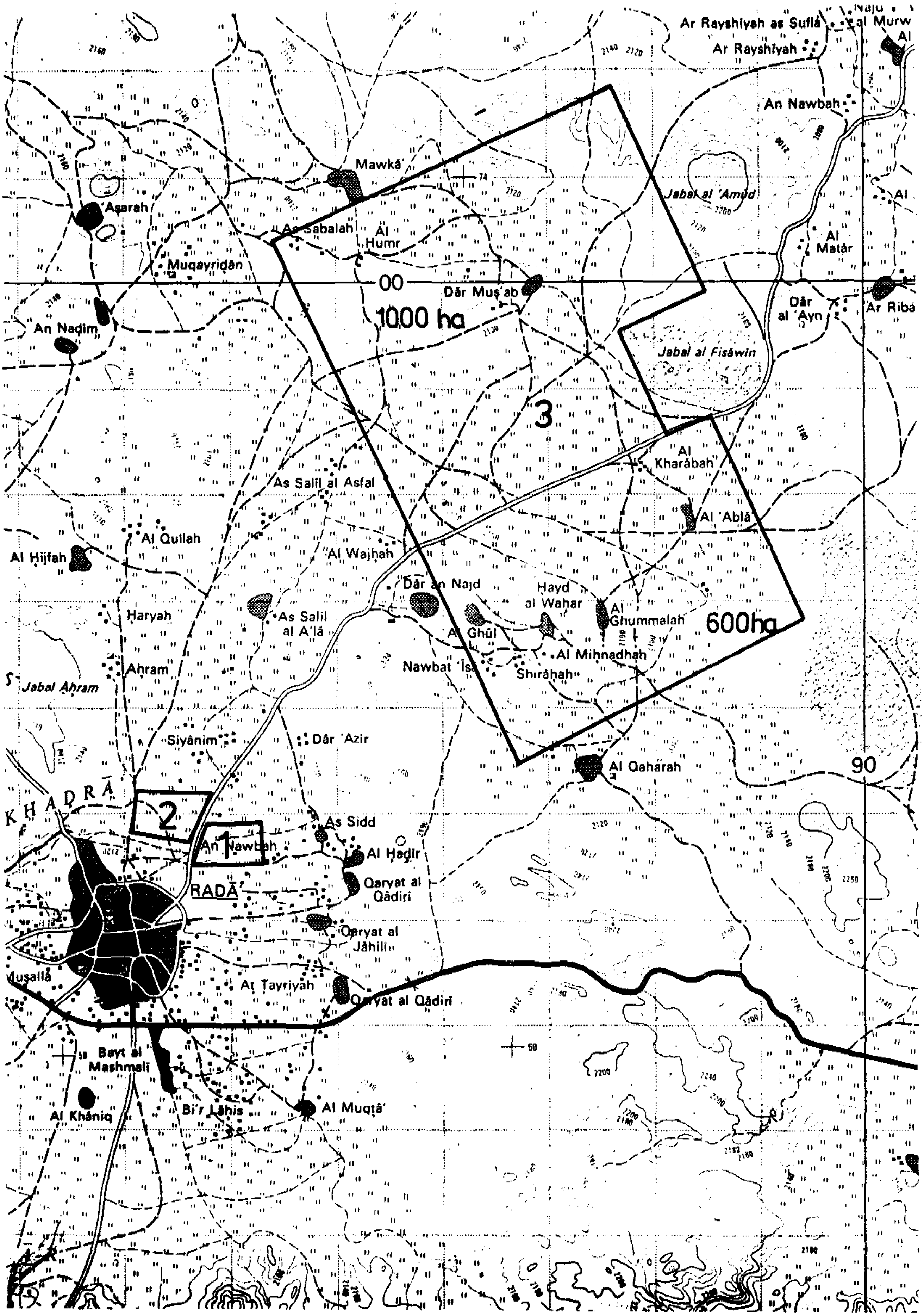

Fig. 3.19 *Possible sites for agricultural development plan*

4. RAINWATER DRAINAGE

4.1. Present rainwater drainage conditions

Rada is situated on a gently west-east sloping plain with some rocky outcrops. Its micro drainage patterns are shown in Fig. 4.1. This figure shows that the surroundings of Rada drain through two wadi courses: the Wadi Al Arsh, flowing in a west-east direction north of Rada, and another wadi, which flows from south to north. There is no clear alignment of the latter one north of the Dhamar - Al Bayda road.

In Rada no special rainwater drainage system exists. This is mainly because traditionally the people have settled at locations with high natural gradients, and partly due to the relatively few days of rain per year.

In densely populated hilly areas of the town rainwater that is not stored in micro depressions is discharged by runoff along footpaths and streets. This water is transported to the lower parts of town and collects at natural field depressions and on the plains at the outskirts, where it disappears due to infiltration.

In less densely populated areas the rainwater does not in general cause any runoff, but infiltrates into the subsoil, while a minor part evaporates. Only during exceptional rainstorms runoff takes place. Due to the west-east oriented gradient of the surface large volumes of runoff pass through the densely populated areas in the middle and southern parts of the old city, on their way to the plains east of the town.

The northern part of the town drains directly into the Wadi Al Arsh, which runs in a west-east direction directly north of it.

Although it is clear that rainwater drainage problems did exist in the past, the situation - especially in the densely populated areas - has become worse over the years. This has several reasons:

As a result of the fast urban growth in the last decade, many depressions that had traditionally been reserved for rainwater collection, have gradually been built up with houses and other buildings. This not only disturbs the old drainage pattern, but also increases the number of buildings exposed to flooding.

The road drainage is very poor. The increasingly motorized traffic, in combination with the absence of a road drainage system, ruins the existing unpaved roads more and more. Especially as a result of the water remaining in the holes, the top soil softens and the holes are deepened by the motorized traffic. Large holes with a depth of over half a metre are now a common result of this cumulative process.

In relation with the solid waste problem it has to be mentioned that through the increasing volume of solid waste and the absence of an effective garbage collection system until very recently, drainage courses were often blocked by solid waste. Field depressions which are not yet built up have proved to be an ideal spot for dumping refuse. Rainwater mixed with waste water is collected in these depressions.

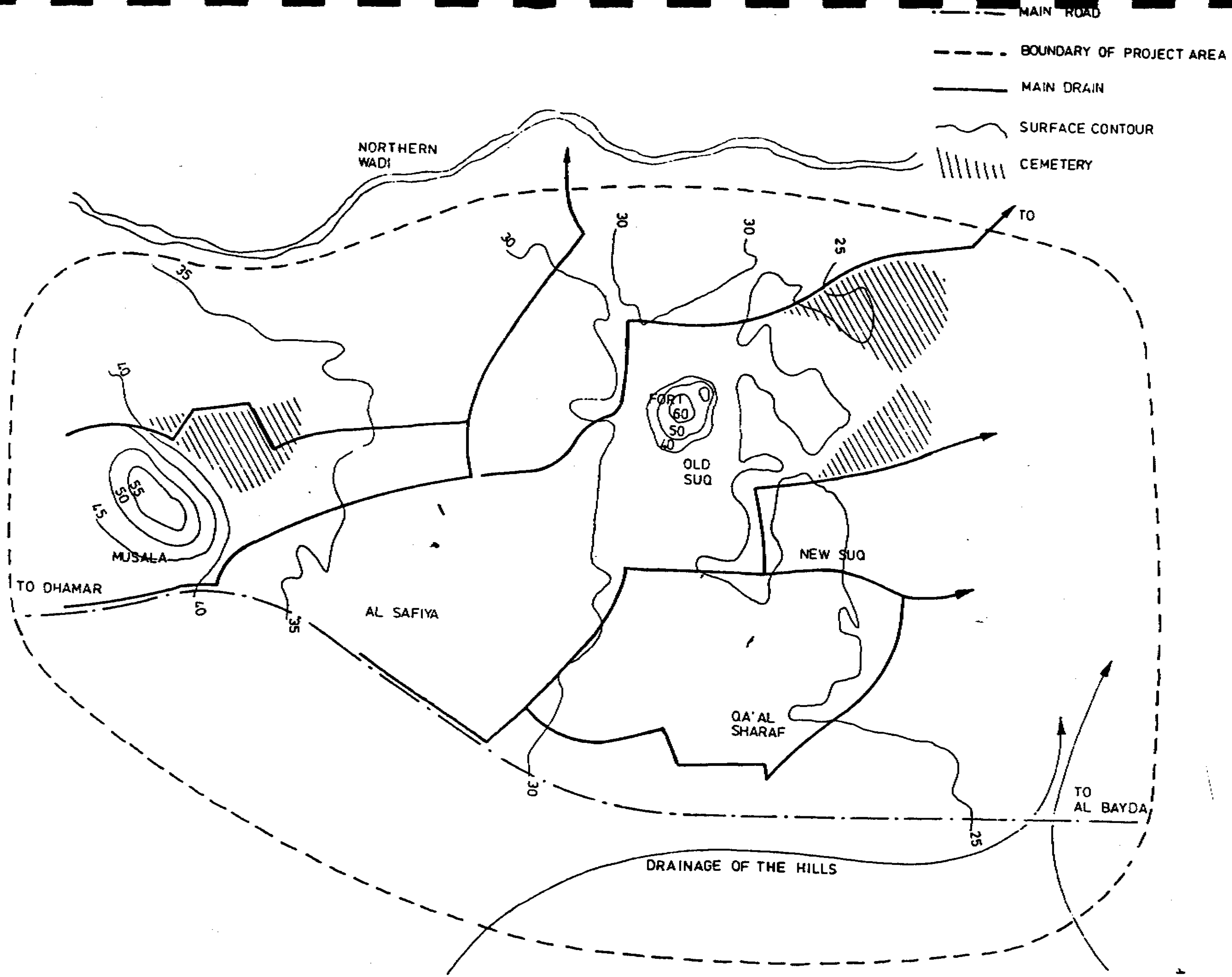

Fig. 4.1. Macro drainage pattern of the Rada area

The combination of waste water, solid waste and rainwater runoff creates situations that are a severe hazard to public health.

When the waste water and the solid waste will have been cleared from the streets, the most unhealthy aspects of this situation will have been removed. However, a proper rainwater drainage system will still be needed for reasons of public health, protection of buildings, road maintenance and comfort.

4.2. Design basis

4.2.1. Climatological and hydrological conditions

a. Climate

Rada is situated in that climatological area of the YAR that is known as the inland area. There, the effective rainfall seasons are confined to the periods February-May and July through early September. During these periods the conditions and humidity of the atmosphere are sufficient to cause heavy rains: the average annual precipitation in Rada is about 200 - 250 mm.

The maximum daily temperature is rarely below 20 ^{0}C in winter, or over 35 ^{0}C in summer. In dry periods the minimum relative humidity varies between 10% and 20%, with a maximum around 50%. In wet periods saturation (100%) can occur.

b. Hydrology

In the Rada region, large monthly rainfall totals are usually the sum of many moderate rains. The short period intensities are usually also moderate, the total duration and quantity being such that - due to infiltration - mostly only brief local runoff is generated. Occasionally, however, major storms do occur and from time to time these produce serious floods.

A drainage system should prevent most of this flooding in urban areas by channelling the rainwater away. It has to be taken into account that occasionally major storms can result in the discharge of rainwater from quite large areas. Flood control measures will have to prevent that large volumes of water from rural areas have to pass through the urban area.

To design a particular system, detailed rainfall data, in particular related to the critical storm and with short observation intervals (10-60 minutes) should be available. Daily figures alone are not sufficient.

Since 1977 rainfall has been recorded in the immediate vicinity of Rada, in the framework of the RIRDP project. The data of the Rada rainfall station have been used for the design of the drainage system.

On the basis of these daily rainfall observations over the period 1977-1987 (11 years), monthly averages have been calculated (see Fig. 4.2.). These clearly show the dry and wet periods. Information on incidental readings during storms or parts of these is available, but not complete, and too scanty for statistic and stochastic calculations. For that reason the calculation of the design storm had to be based on daily data only.

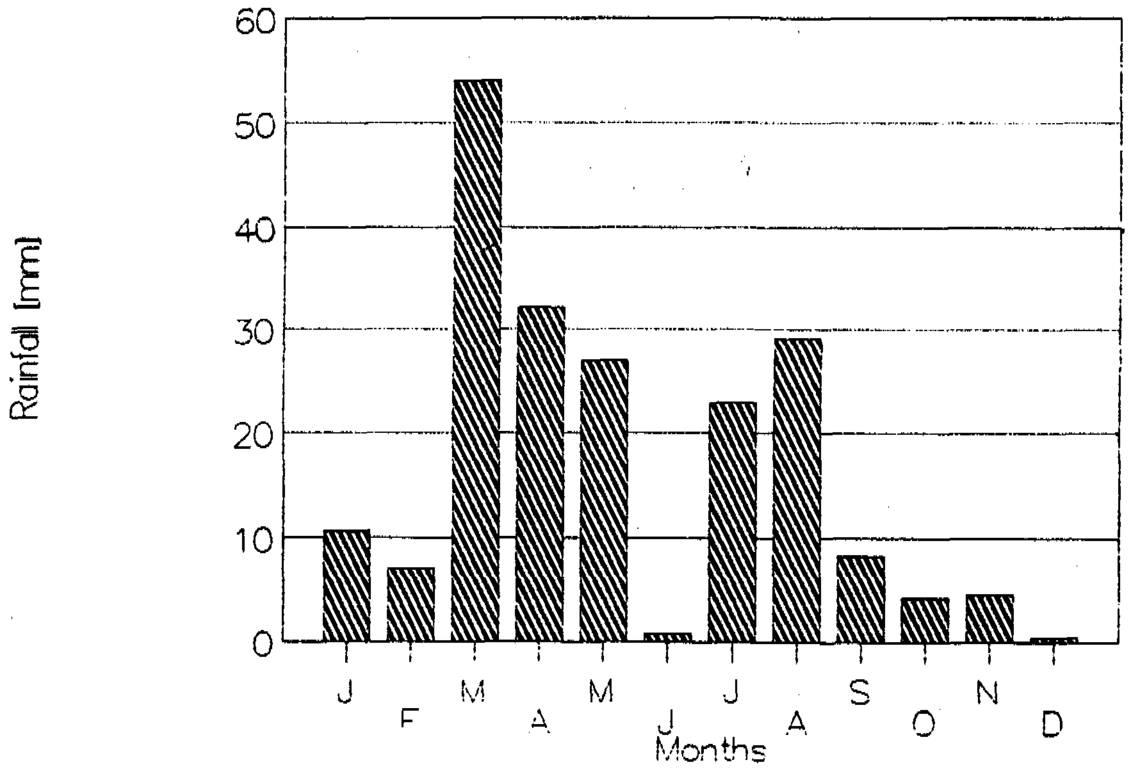

Fig. 4.2. Average monthly rainfall in Rada

For the extrapolation of total rainfall within 1, 2, 3 and 4 days the Gumbel theory has been used. The so-called k-days rainfall data and the calculation of their extreme values according to the Gumbel theory are shown in Table 4.1.

The design of a rainwater drainage system is based on a chosen acceptable rainstorm recurrence interval. The longer the recurrence period, the larger the rainwater drainage system and, consequently, the higher the investment and maintenance costs. For urban areas such as Rada a recurrence period of 2 years is usually chosen.

The extrapolation of the daily data to data for 15 minutes to a few hours, has to be based on an intensity-duration equation. The most commonly used equation is:

$$I = \frac{a}{(b+T)^n}$$

where: I = rainfall intensity [mm/h]
T = duration [h]
a, b, n are constants

Table 4.1. k-Days rainfall and extreme values

	YEAR	1 day	2 days	3 days	4 days
	1977	20.5	20.5	20.5	28.3
	1978	52.5	61.0	61.0	61.0
	1979	39.4	39.4	49.6	49.6
	1980	34.4	34.4	37.9	37.9
	1981	24.5	34.7	37.1	43.7
	1982	65.8	65.8	126.8	126.8
	1983	40.7	40.9	40.9	40.9
	1984	50.9	77.0	92.4	101.1
	1985	40.8	40.8	40.8	50.2
	1986	27.4	43.9	43.9	44.2
	1987	45.2	47.1	47.1	47.3
Average:		40.2	46.0	54.4	57.4
Standard deviation:		12.7	15.4	28.6	28.3
Extreme values:					
1 per 2 years		38.1	43.5	49.2	52.0
1 per 5 years		50.8	58.3	69.9	71.0
1 per 10 years		59.2	68.0	83.6	83.5

For Rada this equation becomes:

$$I = \frac{27}{(0.2 + T)^{0.88}}$$

The rainfall depth and intensity data for periods less than 1 day are shown in Table 4.2. and illustrated in Fig. 4.3.

Table 4.2. Rainfall data, recurrence period 2 years

Duration	Rainfall	Intensity	
	|mm|	|mm/h|	|l/s/ha|
15 min	11.6	46.4	129
30 min	16.6	33.2	92
45 min	19.6	26.1	73
1 hr	21.6	21.6	60
1.5 hrs	24.6	16.2	45
2 hrs	26.1	13.0	36
2.5 hrs	27.4	10.9	30
3 hrs	28.4	9.5	26
6 hrs	32.0	5.4	15
12 hrs	35.4	2.9	8
24 hrs	38.7	1.6	4

From these data a design rainstorm with a recurrence period of two years has been derived, using the method known as the USA Soil Conservation Procedure. According to this method the highest intensities are concentrated in the middle of the storm.

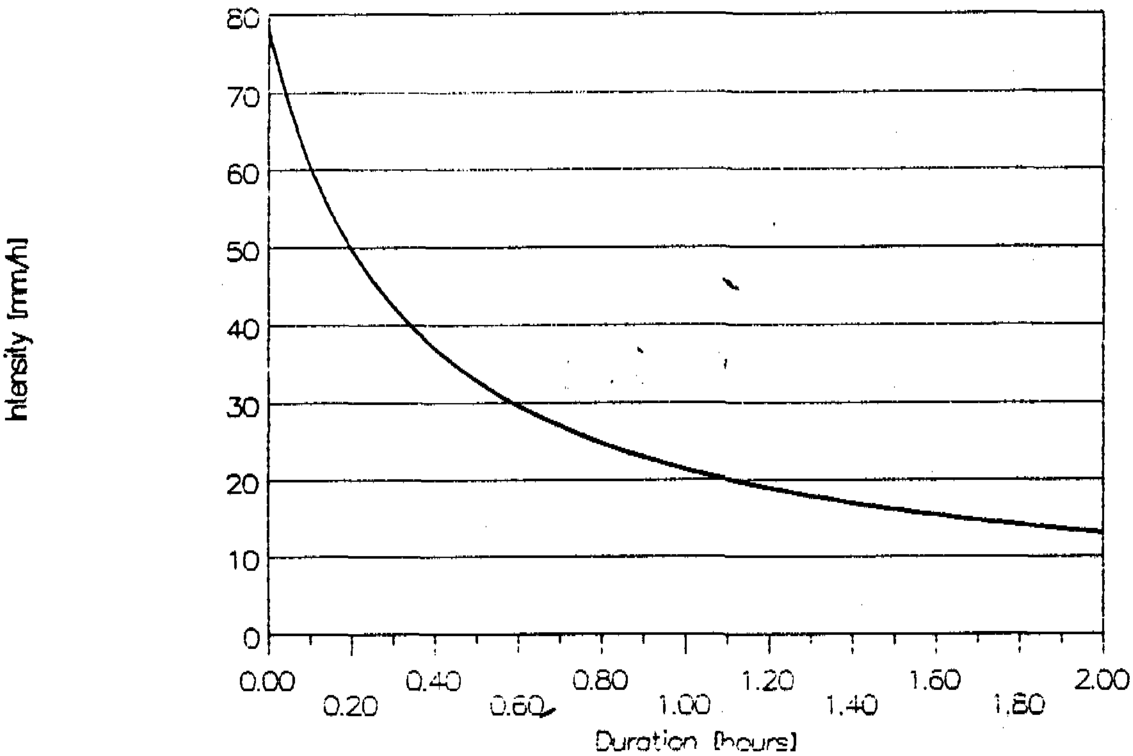

Fig. 4.3. Intensity - duration curve (freq. 1 in 2 years)

The intensities resulting from the rainfall - depth relation are arranged around the peak in decreasing intensities as shown in Table 4.3.

Table 4.3. The 1 in 2 years design storm

Period [min]	Duration [min]	Intensity [mm]	Intensity [mm/h]
0 - 30	30	1.3	2.6
30 - 60	30	2.7	5.4
60 - 75	15	3.0	12.0
75 - 90	15	11.6	46.4
90 - 105	15	5.0	20.0
105 - 120	15	2.7	8.0
120 - 150	30	1.8	3.6
150 - 180	30	1.0	2.0

The results are illustrated in Fig. 4.4.

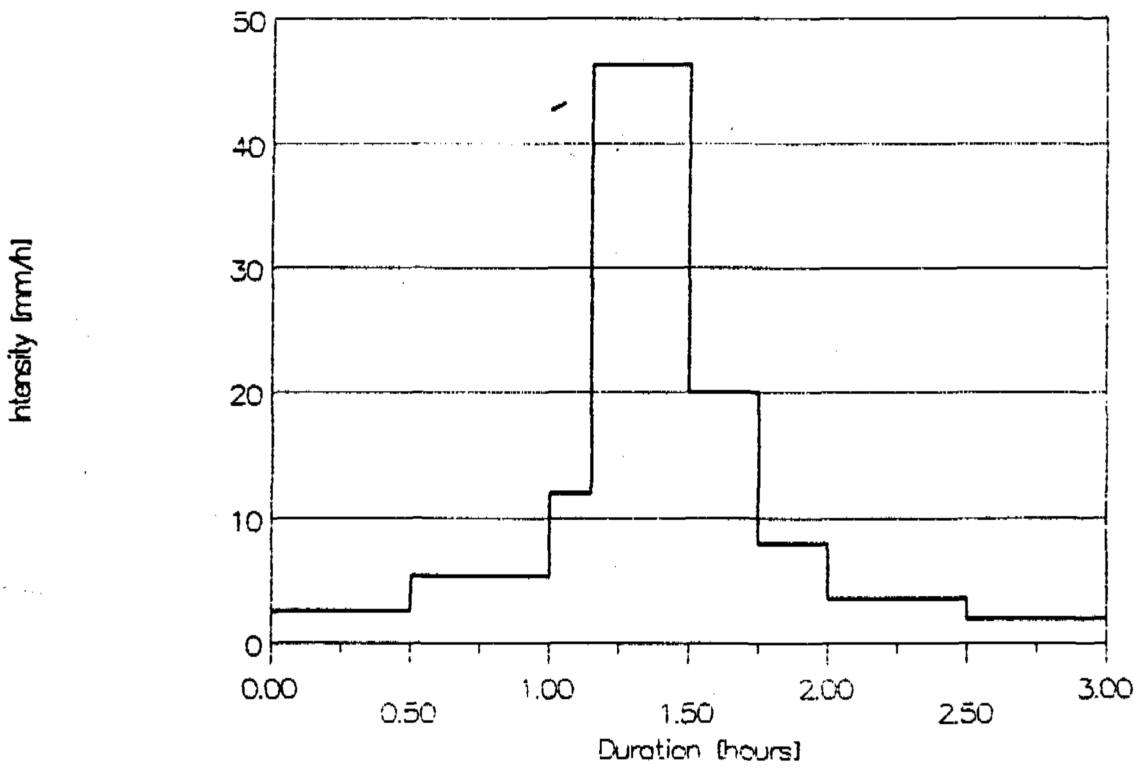

Fig. 4.4. Design storm

4.2.2. Runoff coefficient

The runoff coefficient is the ratio between the amount of rainfall on a certain area and the runoff from that area. Due to surface retention, infiltration and evaporation, this coefficient is less than 1. During rainstorms the retention is the main factor. The runoff coefficient should be based on ultimate catchment development and be weighted where more than one kind of land use exists or is likely to develop within the same catchment.

It has to be taken into account that especially in the new and recently built-up areas the premises are surrounded by brick walls. Due to these walls the area surrounded by them hardly contributes to the runoff of that area. On level areas, moreover, a considerable volume of water can be stored before runoff actually starts.

Taking into account the circumstances under rainwater conditions, occurring once per 2 years, the runoff coefficients used are shown in Table 4.4.

Table 4.4. Runoff conditions during heavy storms

Land use	Runoff coefficient
Residential area:	
- low density, modern set-up	0.1 - 0.3
- medium density	0.3 - 0.5
- high density (traditional built-up areas)	0.6 - 0.8
Commercial area (suq)	0.7 - 0.9
Industrial area	0.3 - 0.7
Paved areas (asphalt road)	1

4.2.3. System selection

a. Combined versus separate systems

For the discharge of waste water and rainwater two main types of sewerage and drainage systems can be distinguished:
- combined system for waste water and rainwater;
- separate system for waste water collection (sewers) and for the discharge of rainwater (drainage system).

Under the climatic conditions prevailing in Yemen, separate systems for the collection of waste water and rainwater are to be used. The high rainstorm intensities and the long dry season make this mandatory, as is explained below.

Due to the high rainstorm intensities (about 45 mm/h), a combined system would require pipes with diameters as large as 1500 mm. With a minimum cover of 1 m (due to the space required between the various pipelines for house- and gully hole connections), the construction depth would be at least 2.5 m. As a result, the cost of trench excavation and other investment costs would be extremely high, if compared with a separate system. The increasing occurrence of rock in trenches at greater depths would raise the construction costs of a combined system even further.

The ratio between waste water and rainwater flow in Rada is estimated to be between 1:100 and 1:300. An unacceptably high level of maintenance would result from the very low flow velocities and the deposition of materials during the dry season, if a combined system would be used.

Moreover, the waste water treatment plant of a combined system would have to be designed for the maximum waste water flow and most of the rainwater flow. Otherwise, excessive overflowing and indiscriminate discharge of polluted water would occur during each rain storm. The hydraulic capacity of the treatment plant would thus be at least three times the maximum dry-weather waste water flow. Consequently, the costs of construction as well as the costs of operation of the treatment plant would increase considerably.

Even then, several times a year amounts of rainwater polluted with waste water would still be discharged when the influent flow exceeds the treatment plant's hydraulic capacity.

Based on the above mentioned environmental, technical, financial and maintenance considerations, a separate system has been chosen for Rada.

b. Type of rainwater drainage systems

For the separate drainage of rainwater various types of drainage systems can be used:

- **closed drain structures, with the following characteristics:**
  - collector drain under the road surface, and:
  - closed culvert structure, either
    - in the axis of the road, or as
    - closed side culverts (at one or both sides of the road)
- **open culverts, usually as follows:**
  - open side culverts (at both sides of the road)
  - one side open culverts
- **road surface drainage:**
  - road cover in the form of a shallow V-shaped channel, or slightly concave channel

A comprehensive comparison of these systems under conditions as prevailing in Rada has been made elsewhere[1]. Each system has its own advantages and disadvantages, related to aspects such as flow capacity, required space, impairment of traffic, public safety, maintenance, construction of crossings, investment cost, options for extension and phasing during project implementation.

An assessment of the various advantages and disadvantages is given in Table 4.5. The scoring given there are in qualitative terms: favourable, neutral and unfavourable.

[1] Considerations and recommendations for storm water drainage, Rada Water Supply and Sanitation Project, March 1989

Table 4.5. Assessment of rainwater drainage systems

Drainage system / Aspect	Alt. 1 drain collector at axis of road	Alt. 2 closed culvert at axis of road	Alt. 3 open culvert both sides of road	Alt. 4 road surface drainage
Required space	+	+	-	+
Traffic impairment	+	+/-	-	+
Safety of public	+	+	-	+
Flow capacity	-	-	-	+/-
Likelihood of blockage	+/-	+/-	-	+
Accessibility of water	-	-	+/-	+
mains, sewer lines				
and cable crossings	+/-	+/-	-	+
Costs:				
Investment	-	-	+/-	+
Maintenance	-	-	-	+
Possibility for extension and phasing	-	-	+/-	+

+ = favourable
+/- = neutral
- = unfavourable

c. Recommendations

Open culverts have two major disadvantages: (1) there is not enough space available, and (2) serious maintenance problems must be expected due to deposits of sand, gravel and garbage in the drains. In addition, safety aspects should not be under-estimated: as a temporary solution in anticipation of the actual implementation of full-scale drainage improvement works, during the DIP[2] an open drainage ditch has been constructed between nodes 5392, 5346 and 6340, with the result that several cars, including the line bus, have got stuck in them. Closed culvert boxes at the road axis obviously don't have this drawback, but are only appropriate in combination with asphalted roads, to prevent excessive maintenance problems.

The investment cost of drainage by means of open culverts is up to four times the costs of road asphalting alone. The maintenance problems for closed side culverts are smaller because the cover slabs can be lifted. However, maintenance costs for closed drainage systems may still be 10 to 20 times higher than for road surface drainage.

Based on the overall assessment of the alternative drainage systems, it appears that road surface drainage has a number of important advantages, and the

[2] Direct Improvement Programme

highest total score on all aspects in combination. It has been decided, therefore, to adopt this type for rainwater drainage improvement in Rada.

4.3. Description of the new rainwater drainage system

The design of the new rainwater drainage system is based on two issues. The first is that it must be avoided that rainwater from outside the town will enter the town area. The second is that as fast as possible a discharge of the rainwater runoff from within the town direct to the wadi north of Rada or to the plains east of the town is desirable. Based on the topographical map with contour lines, the basic set-up of the drainage system has thus been designed (see Fig. 4.5.)

After a rain storm the drainage will function as follows: the tertiary and secondary drains (i.e.: shaped road surfaces) will discharge the rainwater on to the roads that constitute the primary drainage system. These, in turn, discharge on to the plains area east of Rada and to the Wadi Al Arsh, by means of a total of 12 outlets, 8-9 of which will be provided with special outlet structures.

There are four main drainage areas: north-west, north-east, middle and south.

The north-west area may be assumed to start in Musalla, from where the rainwater runs towards the collector over a relatively steeply sloping area. On its way, the N-W primary drain also collects drainage water from district XI[3], before it finally discharges in the agricultural area of district XX, at the boundary between districts XI and I. District XX at present is predominantly agricultural, with a large number of dikes and small dams to retain the water for irrigation. No drainage problems exist there.

The north-east area starts on the plain west of the Fort or in other words in the center of district IV. The primary drain collects rainwater from district IV, a rather open area, and from districts I, II and III. It follows the boundary between districts I and II and discharges into the plains east of Rada, near the soccer field.

The middle area comprises districts IV, V and VI, as well as parts of districts III, VII, VIII, IX and X. These include both rather densely populated (e.g. districts V, VI and VIII) as well as rather open areas. The middle area has two discharge points. The main discharge is into the plains east of Rada, just south of the cemetery; the other discharge is from the new suq, by means of a short drain, directly to the plains. The new suq is situated on a relatively high area, which is surrounded by the new drainage network.

The south area comprises the remaining part of the built-up area of Rada, between the Dhamar-Al Bayda road and the other drainage areas. Its discharge point is about 200 m north of the main road. The rainwater is discharged into the plains to the east, at a point approximately 200 m east of the projected ring road.

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[3] see district map (Fig. 1.3.)

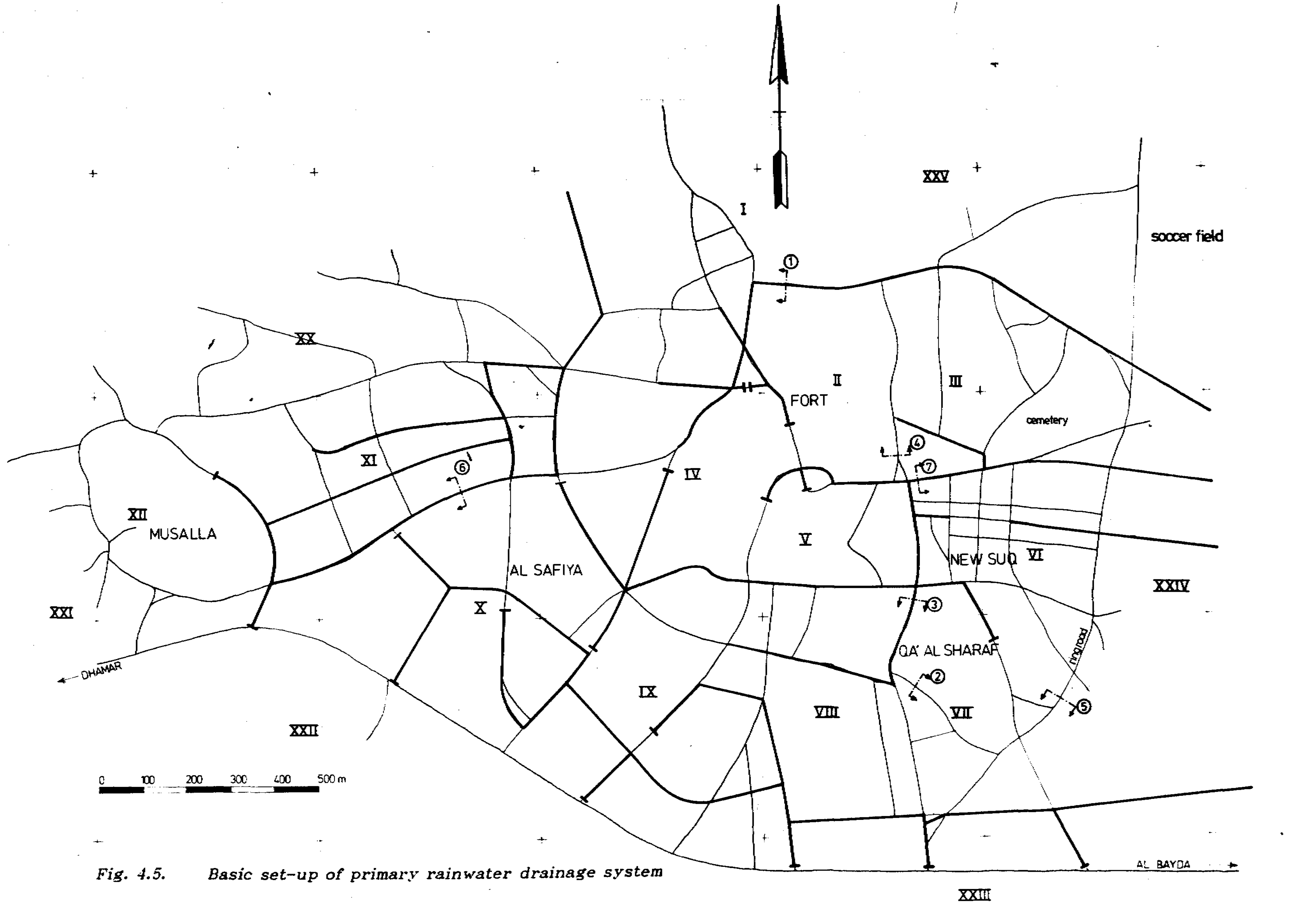


Fig. 4.5. Basic set-up of primary rainwater drainage system

4.4. Primary rainwater drainage system

4.4.1. General

On the basis of the information obtained during detailed topographical surveys a straight sloping alignment has been determined for the primary rainwater drainage system. For its computation the computer program CYCLONE has been used. This is a program that is especially suited to the computation of unsteady flow in open and/or closed water conduits for urban drainage systems. The CYCLONE program is suitable for the computation of time-dependent hydraulic processes involving a system of water courses with a certain storage capacity, using the so-called diffusion wave theory. The program applies the kinematic wave approach for the simulation of the surface runoff process. By this method the precipitation is transformed into an inflow hydrograph for the hydraulic system.
Because of the flexibility of the program, it takes little time and effort to examine a set of alternatives.

For the input of the program the design rainstorm as mentioned in paragraph 4.2.1. has been used. The computation is furthermore based on the division of the drainage system in 4 major catchment areas, as described in paragraph 4.3.

Both surface and invert levels are input data of the program. This then calculates the so-called channel depth and the maximum water levels and discharges during the rainstorm. The time in which these maxima are reached (relative to the beginning of the design storm) is also mentioned in the output data.

The results of the calculations show very clearly that - as far as possible - the rainwater has to be drained in accordance with the west-east inclination of the surface. Local excavations up to 0.5 m will thus have to be carried out to ensure straight longitudinal profiles. In Annex C.2. information is presented regarding the present and proposed surface levels and the required invert level of the drainage profile. The present surface elevation data are based on detailed topographical surveys. The proposed invert and surface levels are the results of the output of the computer program and the criterion of a straight alignment.

The results of the computer calculations for the individual rainwater drainage areas are shown in Annex C.1. These comprise the entire rainwater drainage system, including secondary and tertiary branches.

Cross sections over the primary system at the five points identified in Fig. 4.5. are shown in Fig. 4.6. Details of the curbstone design to be used for the primary system are shown in Fig. 4.7. The roads which have to be paved for drainage purposes are indicated in Fig. 4.8.

4.4.2. Depression areas

In Rada a number of natural depressions have traditionally been used for the storage of rainwater. The gradually increasing number of houses and other buildings not only disturb the old rainwater drainage pattern, but also increase the number of buildings exposed to flooding.

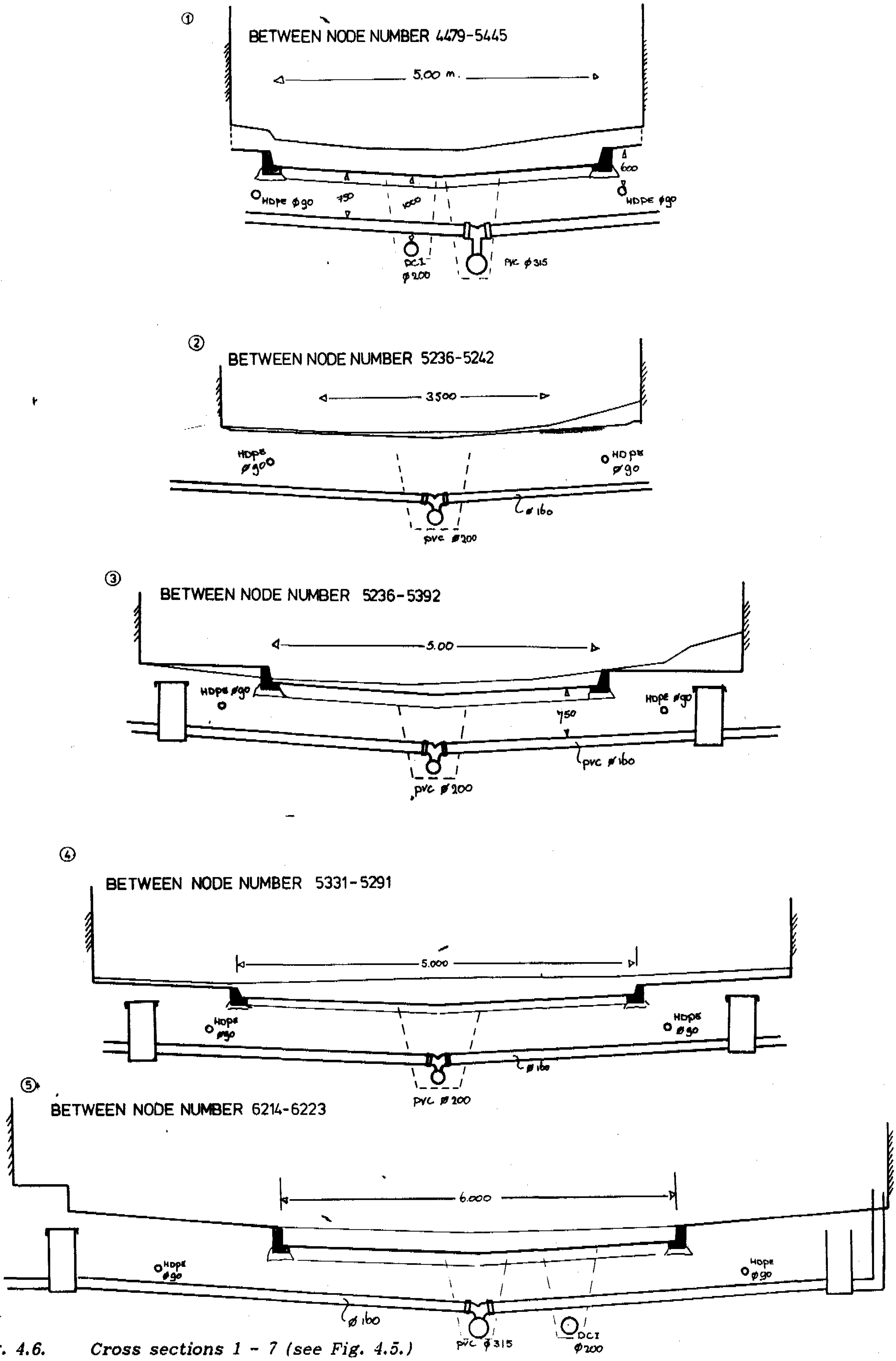
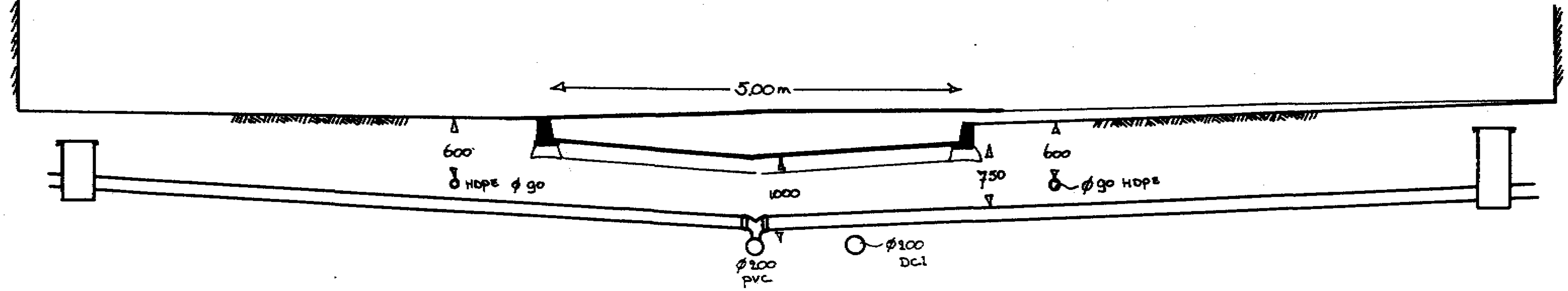


Fig. 4.6. Cross sections 1 - 7 (see Fig. 4.5.)

⑥

BETWEEN NODE NUMBERS 3354-3343



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BETWEEN NODE NUMBERS 5432-5343

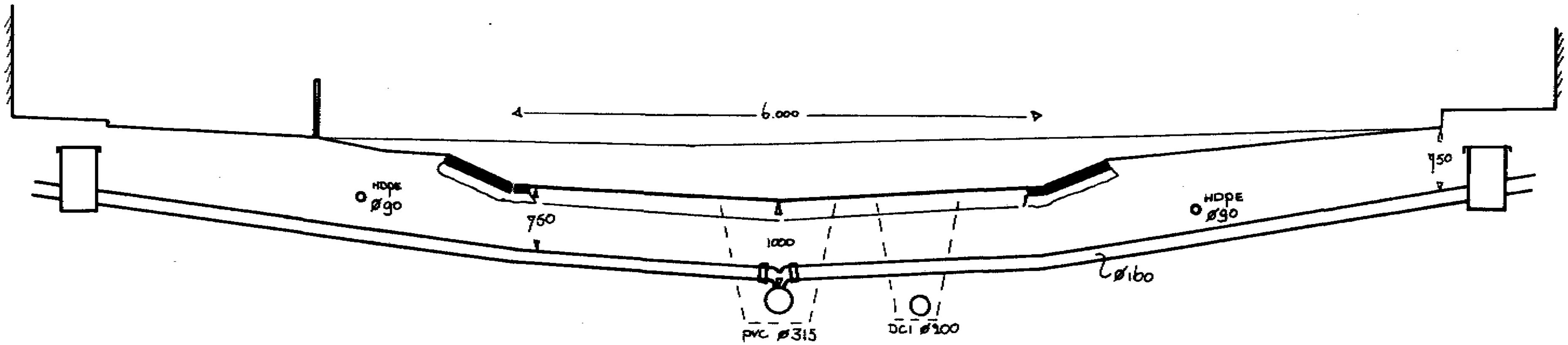
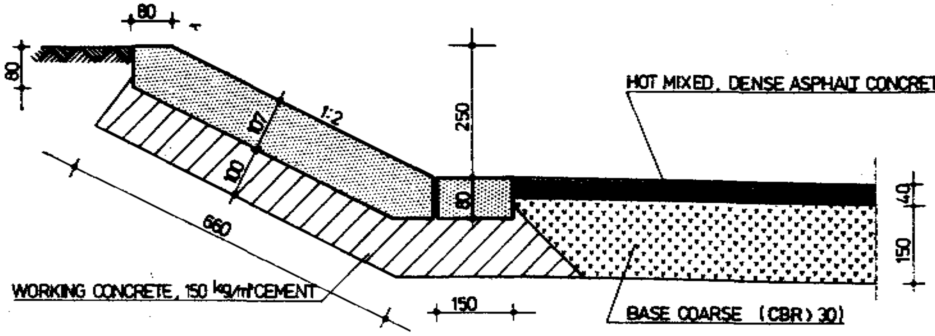
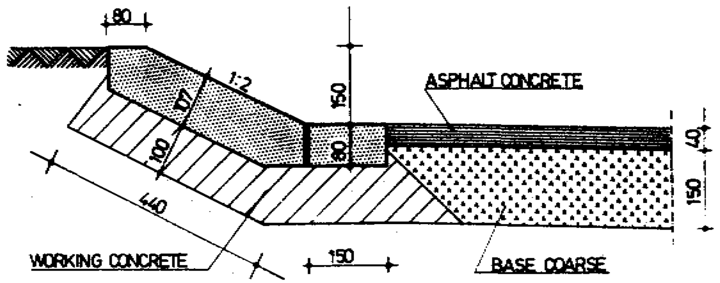


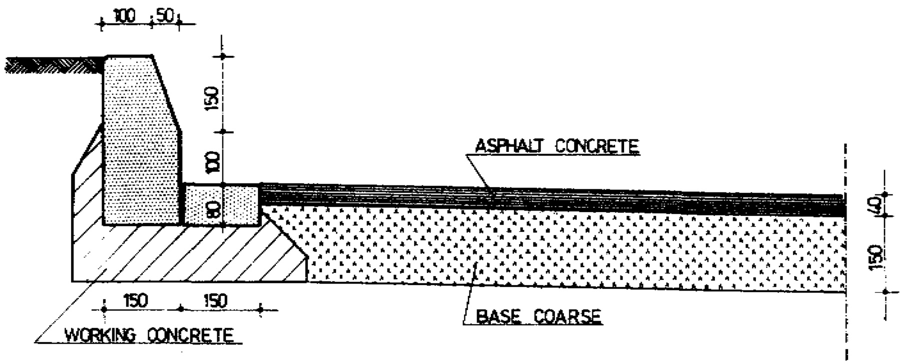
Fig. 4.6. Cross sections 1 - 7 (continued)



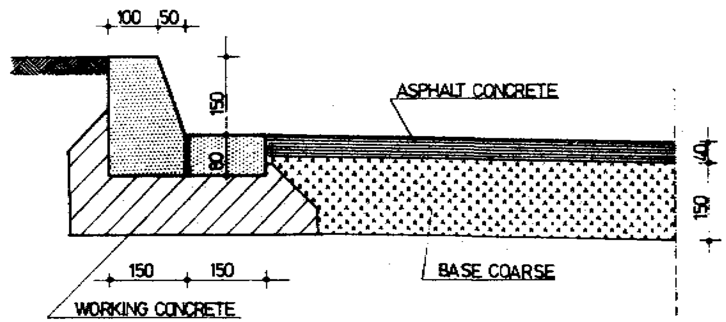
KERB STONE DETAIL in front of shops



KERBSTONE DETAIL in front of shops



GENERAL KERB STONE DETAIL



GENERAL KERB STONE DETAIL

Fig. 4.7. Curbstone details

The levelling of the streets that will be done in the framework of the primary drainage system will solve the problem of flooding.

In the Inception Report it was proposed to use a few of the natural depression areas for the storage of rainwater, and to empty them by pumping the water into the proposed sewerage system.
However, recent experience in these areas shows that under the pressure of increasing basic prices the depressions are filled up and that houses are constructed on the reclaimed land. Consequently the runoff of these areas has been taken into account in the design of the rainwater drainage system.

4.4.3. Roads to be paved

The proposed rainwater drainage is based on asphalt-lined shallow channels. The primary system will have to be lined. The total length of these sections is 13.21 km. To this have to be added 0.57 km for restoring already existing asphalt pavement. For a good traffic circulation it is desirable to add another 3.33 km, covering sections of the secondary drainage system alignment, and partly corresponding with the existing ring roads. Under the RWSSP however, 13,78 km of primary surface (rainwater) drains will be implemented only.

4.5. Secondary rainwater drainage system

The project will in principle construct the primary rainwater drainage system only, as it is felt that both the secondary and tertiary rainwater drainage system are the responsibility of the local government, since it would comprise keeping the existing street surface in the proper shape.

As has been mentioned before, the street and invert levels have been calculated for the entire rainwater drainage system, if only for establishing the correct sewer and manhole levels. Moreover, in those streets where either water supply pipes or sewers will be laid, the street surface will have to be restored by the contractor involved. While doing so, these streets should be shaped in the manner required for rainwater drainage. That this will be necessary may be seen from the fact that the height of the manholes is based on the ultimate street level as required for rainwater drainage purposes.

Consequently, whereas officially the secondary rainwater drainage system is not a part of the work to be implemented under the project, in practice most parts of the road surfaces will already be brought under the required levels and slopes. Contrary to the primary system, however, no asphalting of the top layer, for rainwater drainage purposes, will be carried out for the secondary (and tertiary) system.

4.6. Tertiary rainwater drainage system

As was mentioned in the previous paragraph, the secondary rainwater drainage system is not considered to be a part of the activities of the Rada Water Supply and Sanitation Project, but rather of the local government. Similarly, the tertiary rainwater drainage system is not considered to be included in the formal RWSSP construction activities either.

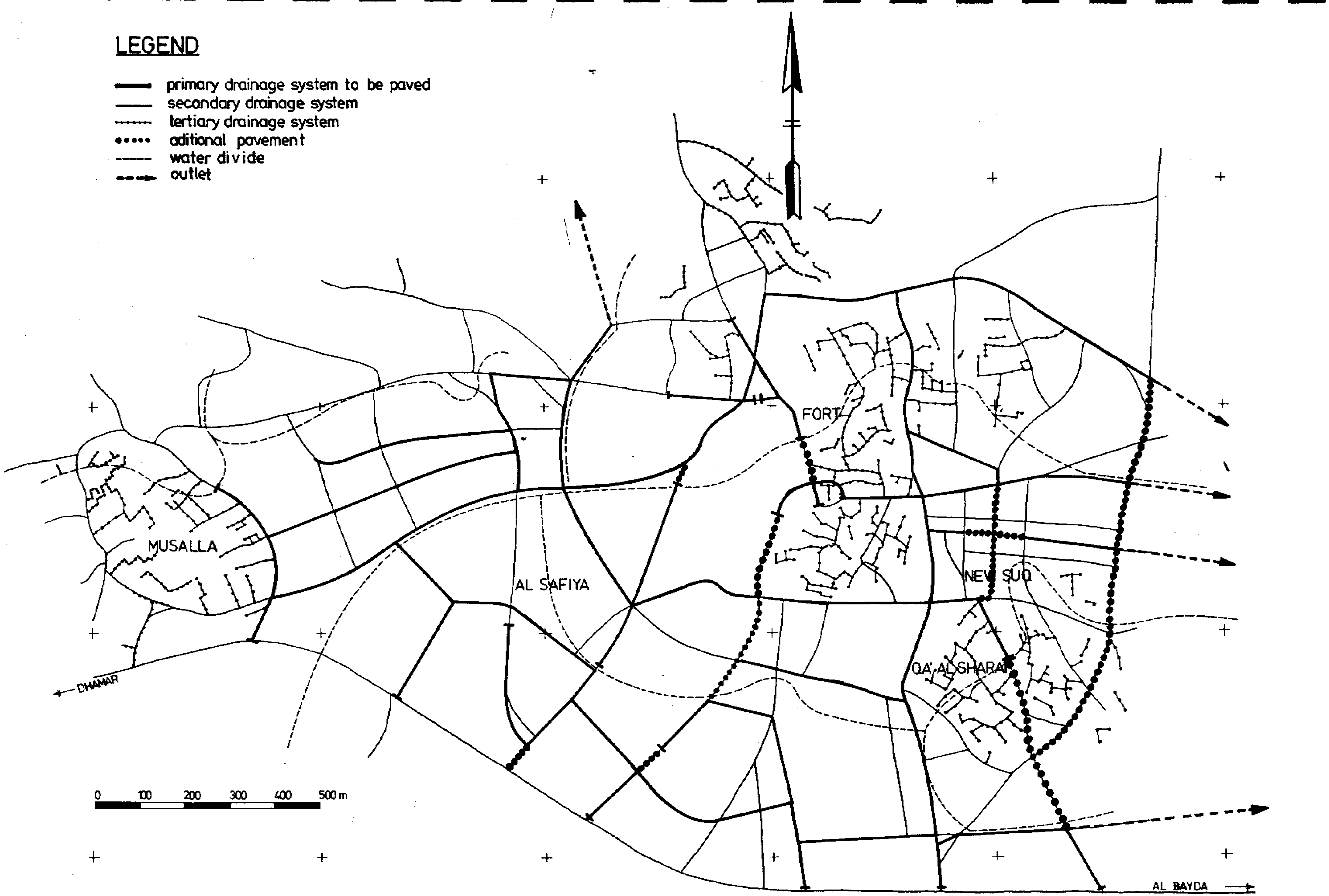


Fig. 4.8. Roads to be paved for rainwater drainage purposes

There is, however, a major difference with the secondary rainwater drainage system in that the relatively shallow depth at which tertiary water supply and sewer pipes may be laid, will often require that measures be taken to protect them from damage. Especially in rocky areas, narrow streets and alleys, where deep trenching is either impossible, dangerous for adjacent houses/walls/ fences, or too expensive, the pipeline will be at a shallow depth so that protection measures are required. This protection will generally be by means of a gulley-shaped concrete layer that follows the sewer lines or house connections and in that way takes care of rainwater drainage.

In principle only the pipe work will be covered, but partly the concrete layer will be extended to existing structures such as houses, walls or side walks.

In the layout for the rainwater drainage system presented in figure 4.8, those tertiary roads, which have to be paved (concreted) are indicated. In total about 13.6 km of tertiairy roads should receive a concrete decklayer; roads have an average width of approximately 2.5 m. width. The thickness of the concrete layer will be about 5 cm. Total surface to be covered is 34000 m^2 and 13600 * 0.05 m * 2.5 m = 1700 m^3 concrete will be used. In the following table 4.6. the tertiary roads, which will be paved for rainwater drainage are indicated, splitted over the districts:

Table 4.6. Tertiary roads to be paved, splitted over districts:

District no.	Tertiary drain (m')
1	2200
2	2000
3	1500
4	----
5	1800
6	100
7	3000
8	----
9	----
10	----
11	----
12	3000
20 thru 25	----
Total	13,600

Examples of tertiary rainwater drainage works are shown in Figs. 4.9. and 4,10.

Furthermore, as is the case for the secondary rainwater drainage system, all road restoration works after pipes have been laid, will be done in such a way that the correct levels and slopes of the road surface as required for drainage will be established. Therefore, even more than for the secondary rainwater drainage system, in practice considerable parts of the tertiary rainwater drainage system will be included, if not always with a top layer.

4.7. Outlet structures

From the topography, contour lines and the present runoff it is clear that the rainwater will flow to the wadi north of Musalla and Rada and to the plains east of the town. Because at present the areas around Rada are all used for agricultural purposes, discharge of the rainwater can benefit the farmers there.

Because the discharge of rainwater takes place in agricultural areas, the shape and the construction of the outlet structures will have to be adapted to the agricultural conditions. This means that farmers with their fields close to the outlet structures will have the opportunity to benefit from the discharged rainwater. Depending on the quantity of rainwater discharged, area available and cooperation of the farmers, a final design of the outlet structures can be made. A typical outlet structure is presented in Fig. 4.11.

At the end of the shallow primary rainwater drainage system the outlet structure starts, running parallel to it. This outlet structure consists of a trench or ditch with slope and bottom protection in the first part of the structure, to avoid erosion. Because natural rock is easily available, the protection can be made as rip-rap, stone jointing or with gabions.
The experiences with rip-rap or stone jointing are not encouraging, however (regular maintenance is essential). On many locations where Irish crossings were provided, gabion protection works for wadis are used successfully. Therefore a construction with gabions is recommended.

Table 4.7. indicates the maximum flow to be expected at 5 of the larger outlet structures.

<u>Table 4.7. Outflow from outlet structures</u>

Area	Outflow node No.	Max. outflow in m^3/s
North-west	4453	1.1
North-east	6495	1.8
Middle	6340	2.2
	6380	0.1
South	6286	1.5

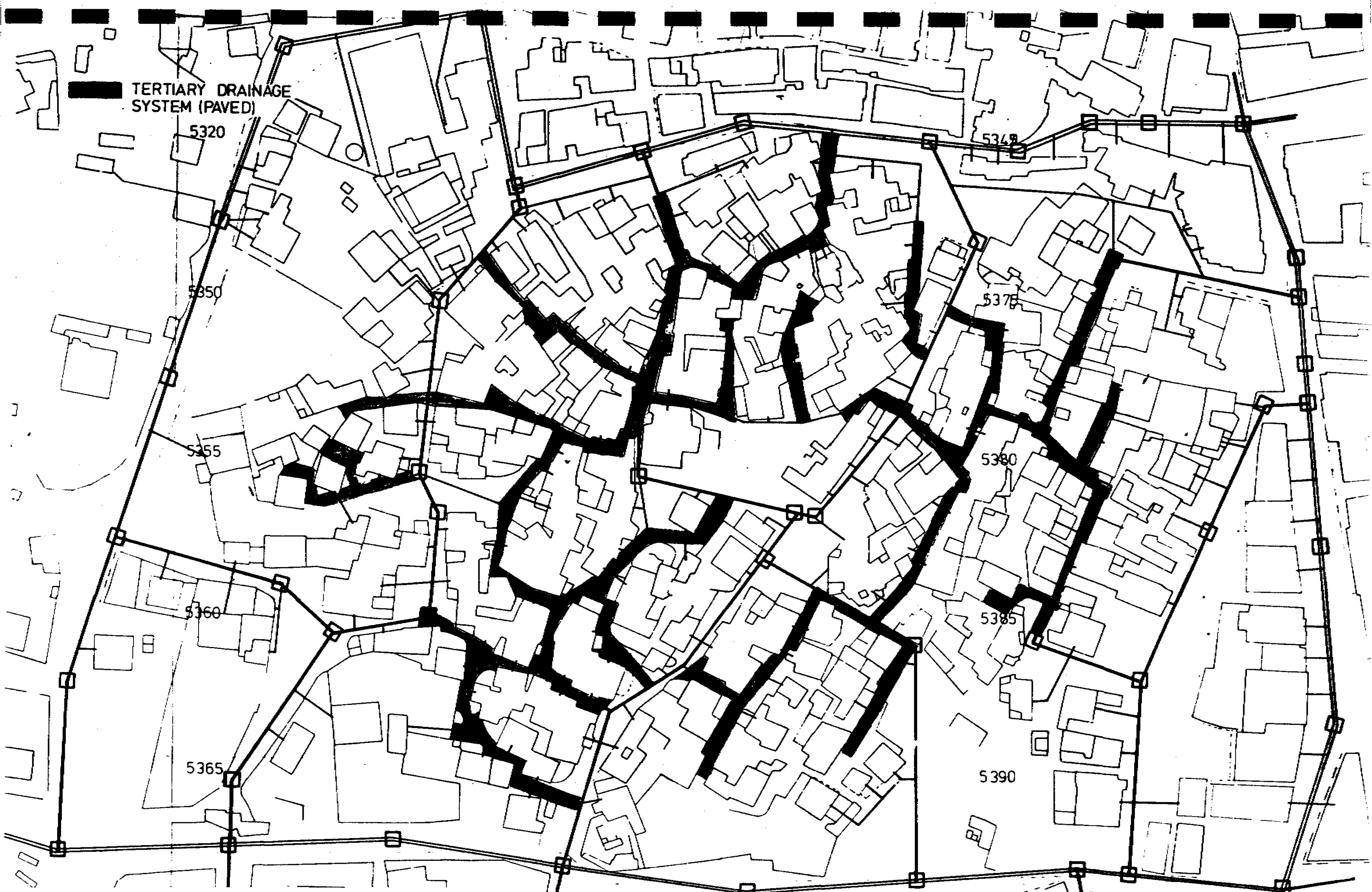
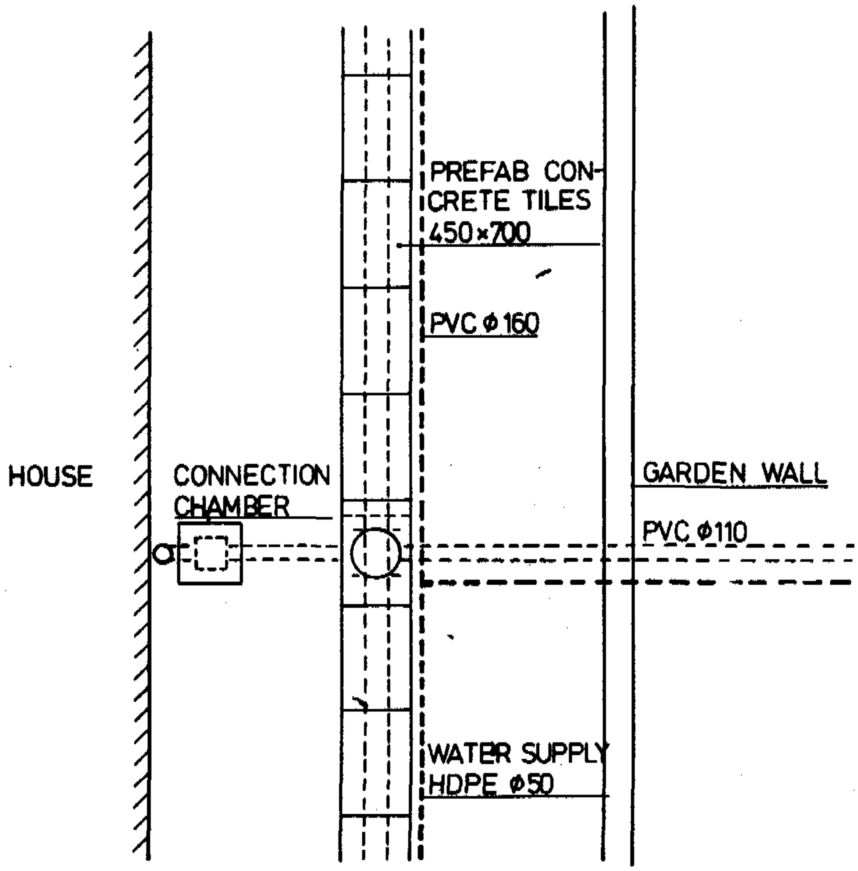
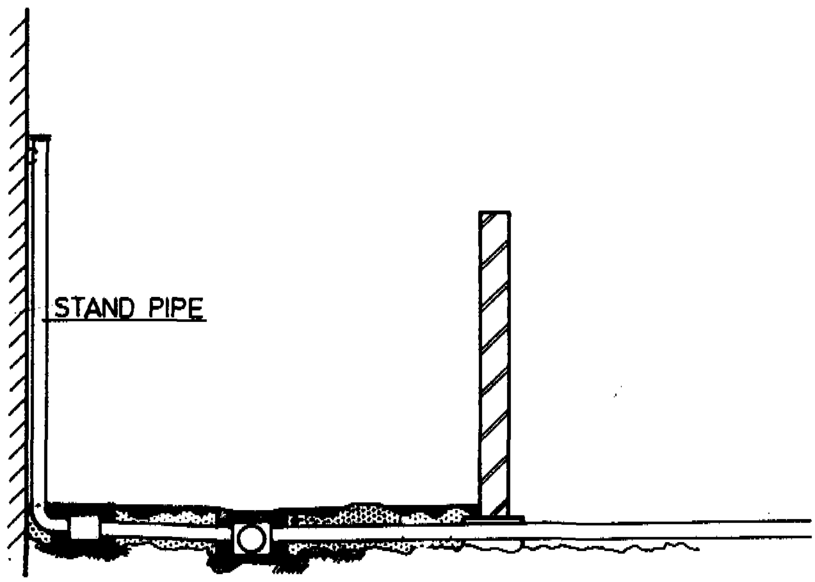


Fig. 4.9. Typical drainage lay-out for tertiary system



TOP VIEW



CROSS SECTION

Fig. 4.10. Typical cross sections, secondary/tertiary drainage system

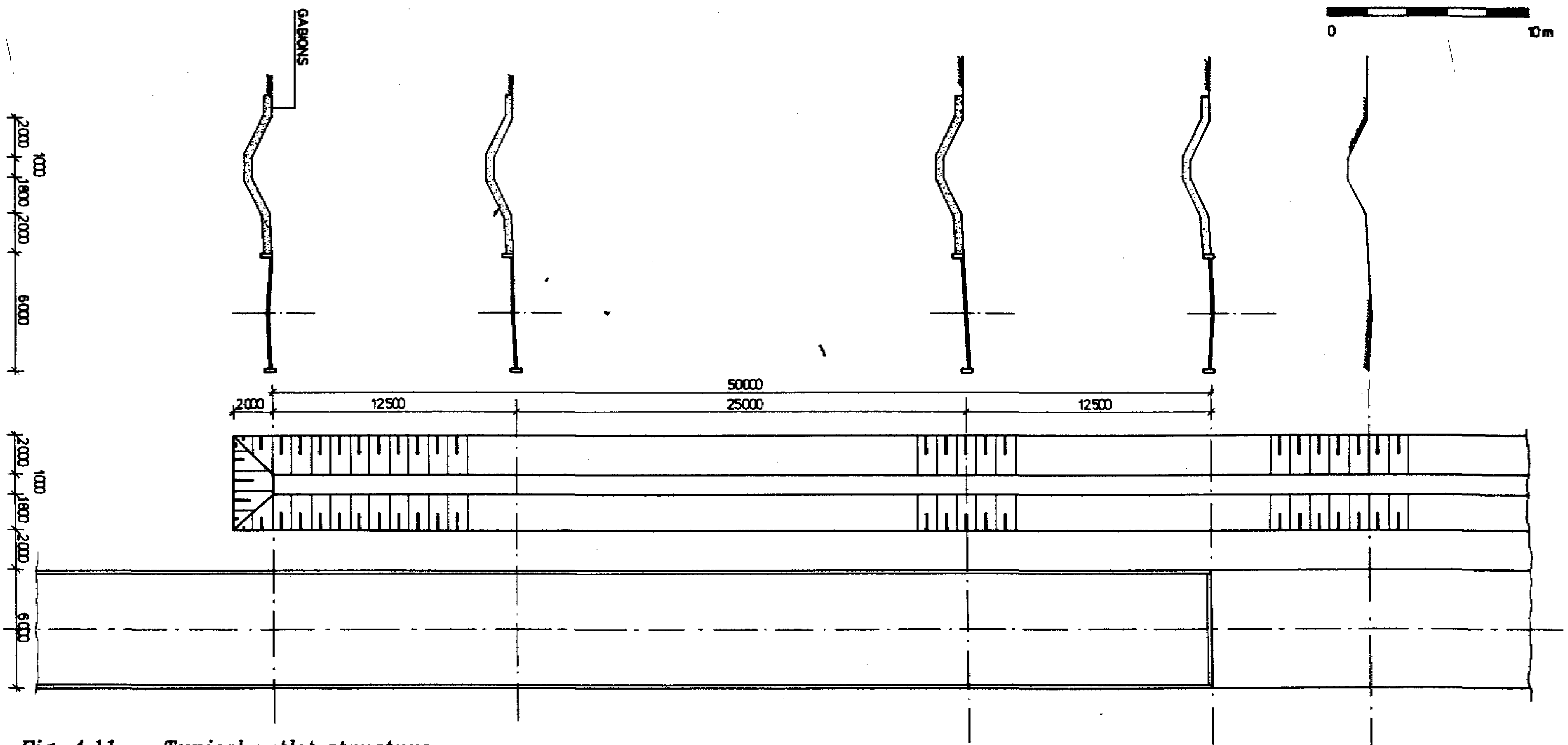


Fig. 4.11. *Typical outlet structure*

4.8. Equipment necessary to maintain the rainwater drainage system

It is the task of the pipelaying contractor for the water supply and sewerage to level all the roads in the secondary and parts of tertiairy streets, up to such an extent, that after completion of these works the complete roads are up to the designed level.
It is the task of the Municipality of Rada in future to maintain these roads, with their - partly already available - equipment.
In order to fulfil their tasks in future properly, it was decided to buy some extra equipment, which partly can be used also for the solid waste collection, to be financed under RWSSP obligations. The following equipment, proposed to be paid from Netherlands funds, has to be purchased.

1. one "Bobcat" (small shovel with small backhoe)
2. one "Poclain" backhoe (or equal)
3. one roller unit "Dynapac" (or equal)
4. one grader "Caterpillar" 160 G/ 16 ton
5. one bulldozer "Caterpillar" D6H/ 17.5 ton
6. two DAF FA 2305 heavy duty tipper trucks

Note: Item 1 scheduled for 1990; item 2 thru 6 for 1992

5. SOLID WASTE DISPOSAL

5.1. Situation at the start of the project

At the start of the Rada Water Supply and Sanitation Project the Rada Municipality had no appropriate organization with sufficient collection equipment and technical facilities to implement regular solid waste collection and disposal activities. Several years earlier a waste disposal system had been introduced by the Municipality, but, due to an impractical design of the containers and a lack of manpower, it did not achieve the envisaged results. Furthermore it is noted that the introduction of the system was not accompanied by a corresponding environmental health education programme. Consultants were informed by the Director of Rada Municipality that at the start of the project 28 persons were dealing with garbage collection activities from time to time, but without proper terms of reference.

At the start of the project, the Rada Municipality had 5 vehicles at its disposal:

- one Mitsubishi 3-ton tipper truck;
- two Nissan 5-ton tipper trucks;
- one water sprayer suction vehicle;
- one Komatsu W-70 wheel loader;
- one Komatsu grader.

Minor repairs were carried out at a private workshop in Rada town.

A number of sites around Rada were used as dumping places for garbage. A major site was along the Al Bayda road, at a distance of about 4 km from Rada.

5.2. New waste collection system

5.2.1. Introduction

Chapter 6 of the Inception Report of the Rada Water Supply and Sanitation Project gives a detailed description of the proposed new solid waste collection system. The following phases were recommended:

- implementation of a crash programme within the framework of the proposed Direct Improvement Programme;
- selection and training of local operational staff;
- establishing regular collection routings for compactor trucks;
- preparation of selected landfill and training of landfill staff;
- distribution of containers immediately after execution of crash programmes and in accordance with designed routing;
- investigations into an appropriate additional collection system for three districts in the centre of Rada town: Harat Al Hafrah, Harat Faqish and Harat Al Qana;
- making necessary road improvements;
- starting regular collection and disposal activities;
- monitoring of routing schedules and implementation of necessary adjustments.

On the basis of a detailed survey in Rada the approach in waste collection as detailed below was recommended.

5.2.2. Waste collection districts

For solid waste collection the town has been divided into 13 districts, most of which are shown in Fig. 5.1. (note: these districts do not coincide with those for water supply, sewerage and drainage):

I	New suq
II	Harat Al Hafra
III	Harat Faqish
IV	Harat Al Qana
V	Hazyaz
VI	Area east of new suq
VII	Qa Ash Sharaf
VIII	Area south of Harat Al Hafra
IX	Southern part of As Safiya
X	Northern part of As Safiya
XI	Madinat Al Musalla
XII	Area south of Dhamar - Al Bayda road
XIII	Qa Rada

5.2.3. Number of containers per district

The proposed numbers of containers are indicated in Table 5.1.:

Table 5.1. Numbers of containers, per district

No.	Area	No. of containers
I	New suq	22
II	Harat Al Hafra	30
III	Harat Faqish	30
IV	Harat Al Qana	15
V	Hazyaz	15
VI	East of the new suq	13
VII	Qua Ash Sharaf	25
VIII	South of Harat Al Hafra	30
IX	As Safiya south	40
X	As Safiya north	40
XI	Madinat Al Musalla	40
XII	South of Dhamar - Al Bayda road	10
XIII	Qa Rada	20
	Military camps	25
	Schools, hospital, government buildings	12
	Total:	367

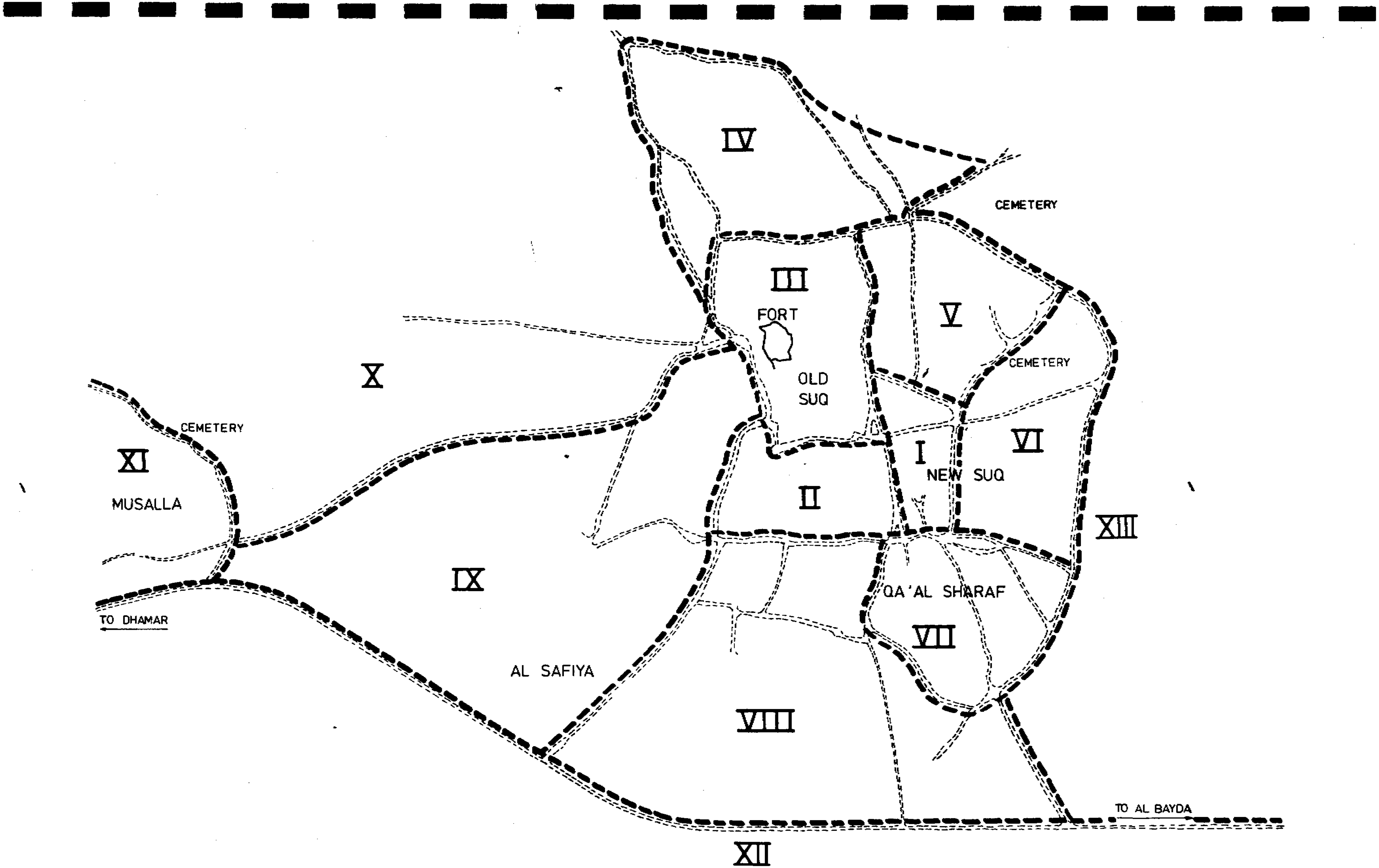


Fig. 5.1. Solid waste collection districts

Out of a total of 425, some 15 containers would be kept as spares to replace damaged containers that would be under repair in the workshop. Another 43 containers would be used for extension areas later on.

5.2.4. Frequency of emptying containers

The frequency with which the containers should be emptied, is indicated in Table 5.2.:

Table 5.2. Total number of containers to be emptied weekly

District No.	No. of containers	No. of collections per week	Total per week
II	30	3	90
III	30	3	90
IV	15	2	30
V	15	2	30
VI	13	2	26
VII	25	3	75
VIII	30	2	60
IX	40	2	80
X	40	2	80
XI	40	3	120
XII	10	2	20
XIII	20	2	40
Camps, schools, etc.	37	average: 3	111
80 containers around the suq and along the main roads, to be emptied during the early hours; daily, also on Fridays			560
		TOTAL:	1,412

5.2.5. Number of trips per week

Daily waste collection will have to start preferably early in the morning (5 a.m.) in order to collect the waste from the suq and from along the main roads in town before it gets crowded, and to prevent the compactor trucks from blocking the narrow streets.

Experience gained under similar circumstances in the YAR learned that the filling rate of containers is relatively high and that with 40 containers per trip during the early morning one truckload of 6.5 tons can easily be collected within two hours.

On Fridays the compactor trucks have to make only one trip to the suq area. The other days of the week, from 8 a.m. onward, the waste collection of each truck will have to be concentrated on two or three districts more.

The filling rates in suburbs and the speed of collection in densely populated areas with narrow and unpaved roads slow down the waste collection. Over 70 containers will have to be emptied to obtain one truckload under these conditions.

The time involved in carrying out this second trip will be three to four hours, including cleaning up some container sites by the four labourers assigned to each truck.

On an average each truck will have to cater for 111 containers per day, as follows:

total of containers to be emptied weekly	1,412
containers to be emptied on Friday	80
containers to be emptied during the week, excluding Fridays	1,332
containers to be emptied daily, excl. Fridays	222
containers per truck and per day	111

Details of the daily trips of each truck are given in Table 5.3.

Table 5.3. Details on daily trips, per compactor truck

Truck No.	Day of the week	No. of trips	No. of containers
1	Friday	1	40
2	Friday	1	40
1	Saturday	2	111
2	Saturday	2	111
1	Sunday	2	111
2	Sunday	2	111
1	Monday	2	111
2	Monday	2	111
1	Tuesday	2	111
2	Tuesday	2	111
1	Wednesday	2	111
2	Wednesday	2	111
1	Thursday	2	111
2	Thursday	2	111
	TOTAL:	26	1,412

5.2.6. Equipment to be supplied

Under the RWSSP Technical Assistance Programme the following equipment would be made available:

- initially two compactor trucks, including spare parts
- one skip-loader crane truck
- 225 containers with a capacity of 1.6 m^3 each

- 200 locally manufactured containers with a capacity of 1 m^3 each
- four Toyota HI-LUX pick-ups 4WD
- three motor bicycles 250 cc
- office/store room
- workshop, with tools and equipment
- audio-visual equipment

During the implementation of the waste collection programme the need to procure a third compactor truck has been considered, and in principle a third truck is required by 1990/1991.

5.2.7. Selection of landfill site

During the inception period Consultants carried out a two-week survey for selecting appropriate methods of solid waste disposal. On the basis of economic and technical factors it was concluded that in the foreseeable future the most suitable method would be to dispose of the solid waste at a properly managed sanitary landfill.

In close cooperation with the Director General of the Rada Municipality five potential sites were identified for further investigation. The sites were compared on the basis of a number of characteristics[1], which showed a clear preference for site No. 4.

This site, situated some nine kilometres east of Rada, is located in a basalt formation with low vertical permeability and consists of two suitable gorges. The most suitable of these has a length of 600 metres and has sufficient capacity for the next 15 to 20 years. Strong winds do not pose any problem. Road conditions will not be detrimental to the compactor truck material, covering material is available, and accessibility is good. The site is shown in Fig. 5.2.

5.3. Already implemented programme components

5.3.1. Introduction

As was indicated already in the Inception Report, during the so-called Direct Improvement Programme the emphasis would be on solid waste aspects, in combination with community information and environmental health education activities. The activities that have been carried out are described in the following paragraphs (see also list of planned activities in paragraph 5.2.1.).

5.3.2. Workshop

A workshop for the maintenance and repair of rolling stock has been established at a location south of the Dhamar - Al Bayda road, approximately 250 m south-west of the existing power plant, and adjacent to the compound of the Highway Authority. The lay-out is given in Fig. 5.3.

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[1] **See Inception report, Volume 1 - Main report, paragraph 6.3., Rada Water Supply and Sanitation Project, July 1988**

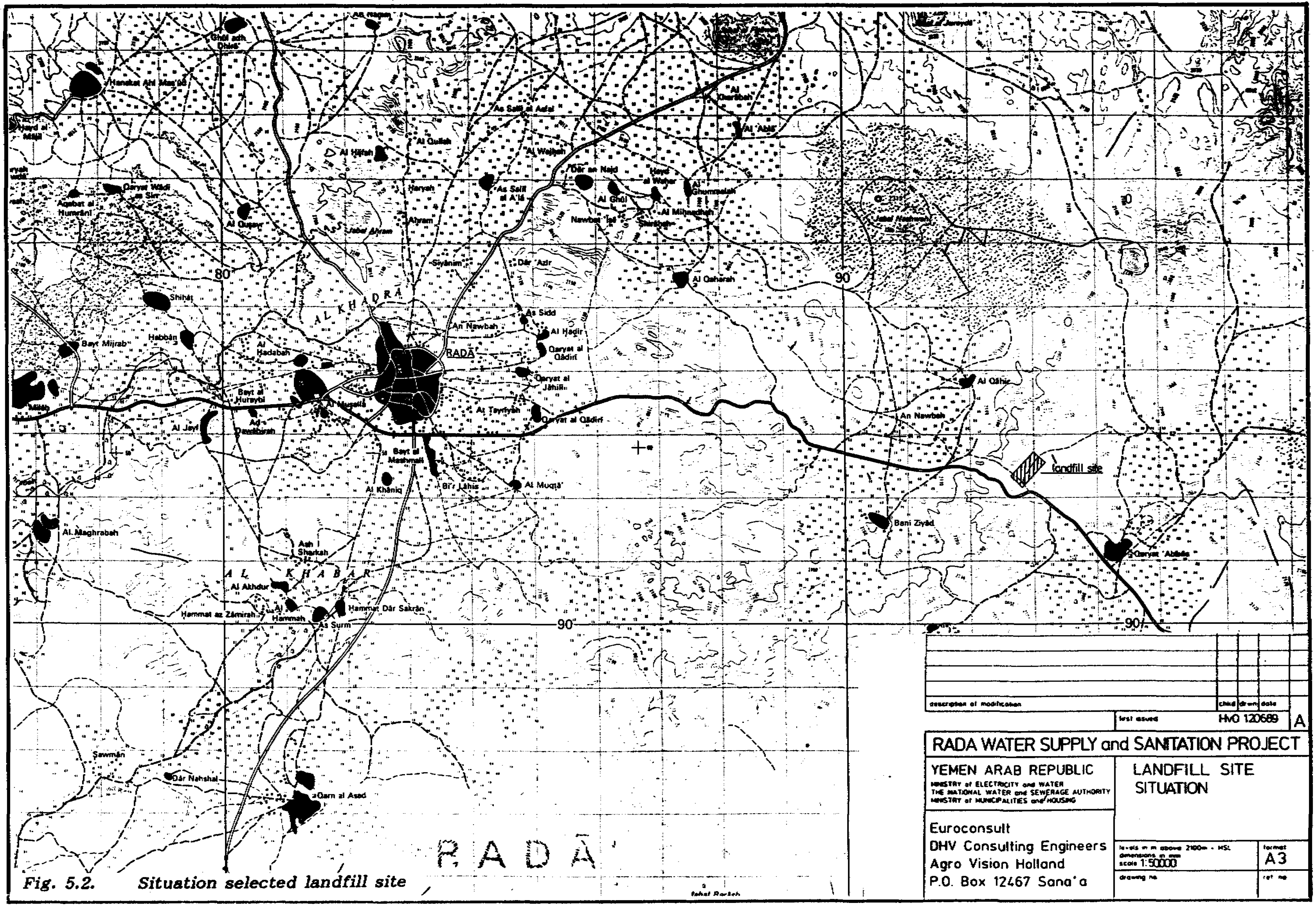

Fig. 5.2. *Situation selected landfill site*

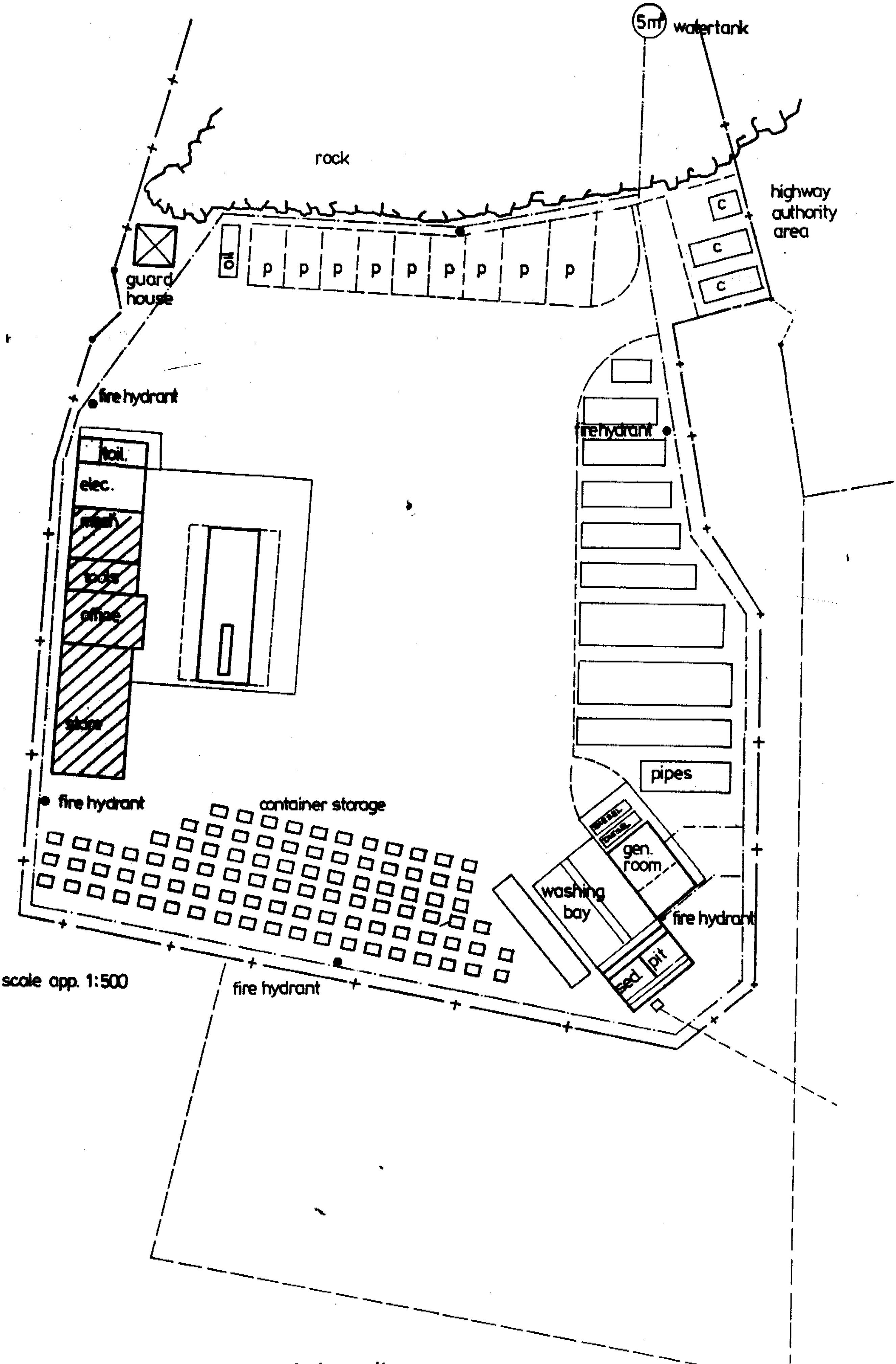

Fig. 5.3. *Lay-out of workshop site*

Construction of the workshop started in June 1988 and was finished by November 1988. The official opening was done by the Netherlands Minister for Development Cooperation, on January 16, 1989.

The compound has a number of components, the description and status of completion of which are described in Table 5.4.

Table 5.4. Workshop components and status of implementation

No.	Description	Percentage completed
1.	Guard house 4 x 4 m	100 %
2.	Main building - spare parts store 7 x 12 m	0 %
	- office 4.5 x 3 m	0 %
	- tools store 3 x 7 m	0 %
	- fitting workshop 5 x 7 m	0 %
	- electrical workshop 4 x 7 m	100 %
	- sanitary unit 2.5 x 4.5 m	100 %
	- compressor room 2.5 x 2 m	100 %
3.	2 units open maintenance shed 15 x 5 x 5 m	50 %
4.	vehicle washing place + sedimentation basins + water collector pit; total 14 x 7.5 m	100 %
5.	generator building with room for pressure cleaning device 8.5 x 6 x 3 m	100 %
6.	modified 40-ft container with office, store and spares store (imported)	100 %
7.	spare parts storage containers, 3 x 20 ft + 1 x 10 ft (not originally foreseen)	100 %
8.	housing accommodation for solid waste collection team; 2 x 10x18x3 m + 1 x 10x24x3 m	0 %
9.	sanitary facilities (toilets, showers, wash basins); 2 x 7.5x5x3 m	0 %
10.	fencing, entrance gates, lighting poles, water tank + pipelines	100 %

Operational staff has been selected and trained, and includes:
- workshop manager
- mechanic
- assistant mechanic
- electrician
- puncher
- storekeeper
- vehicle cleaner
- 2 guards

The workshop is equipped with a 25 kVA generator for safeguarding the power supply. Because the uncompleted parts of the workshop will not be executed with direct funding under RWSSP, it was proposed to complete the workshop with a budget from the so-called countervalue funds.

5.3.3. Clean-up programme

Within the framework of the Direct Improvement Programme and on the occasion of the celebrations of the 26^{th} of September, a clean-up campaign was held in the period September 19 - October 3, 1988. The campaign was organized by the EHE[1] division, in close cooperation with the local authorities and with active community participation. The cleaning campaign meant to be the start of the regular solid waste collection and disposal programme. With the aid of the equipment owned by the Rada Municipality/the project as well as hired trucks and wheel loaders, in total about 2000 tonnes (over 5000 m^3) were removed from the town and dumped at the presently used - temporary -dump site north of the Dhamar - Al Bayda road, about 4 km from the centre of Rada.

5.3.4. Procurement of equipment and rolling stock

Most of the equipment and rolling stock mentioned in paragraph 5.2.6. has been procured, as follows.

- Two compactor trucks have been ordered and supplied. Customs procedures have caused considerable delays. A deal could be made with the Municipality of Ta'izz, whereby one of the new compactor trucks available in Ta'izz was made available to Rada, with the understanding that it would be replaced by one of the trucks purchased for Rada as soon as that would have been cleared by customs.
- Rather than a tipper crane truck, a skip loader has been procured. The crane component is installed in November 1989 finally, after awaiting customs clearance, since October 1988.
- A total of 225 imported containers (1.6 m^3 each) and 200 locally made containers (1 m^3 each) have been procured. Up till July 1989 only the larger containers have been used, so as to derive the maximum effect of the single compactor truck available at that time.
- The four Toyota HI-LUX pick-ups 4WD and three motor bicycles have been procured.
- The audio-visual equipment to be used by the Environmental Health Engineering Section has been procured and is being used.

5.3.5. Solid waste collection

On November 3, 1988 a start was made with the distribution of solid waste containers over the town. As mentioned above, the fact that up till July 1989 only one compactor truck has been available, has been the reason to limit the use of containers to the larger-size containers (1.6 m^3) only, as the use of smaller containers would be less effective.

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[1] environmental health education

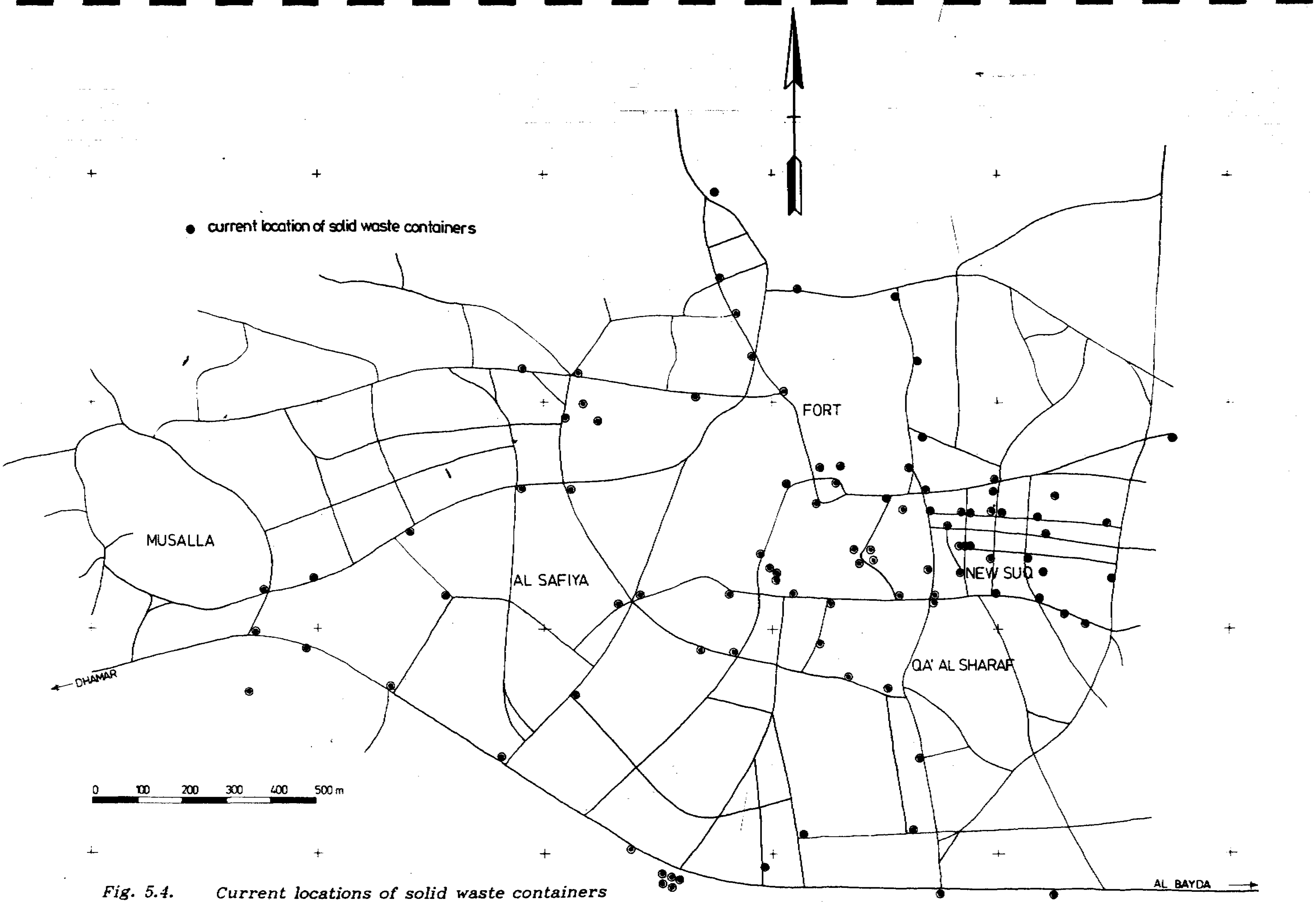

Fig. 5.4. *Current locations of solid waste containers*

Regular garbage disposal services started in the old suq area (15 containers) and Al Khabar/RIRDP (4 containers). Services were extended to the new suq area (11 containers) and hospital (2 containers) by November 23, 1988. In the last week of November another 32 containers were placed, along major roads in Rada. In March 1989 20 containers were placed in Harat Al Hafra, whereas during the first extension another 11 containers were used to increase the over-all container density in the eastern part of the town, bringing the total number of containers in use to 95 (situation early June 1989; see Fig. 5.4.). For the month of June 1989 a second and third extension were planned, covering Qua Ash Sharaf (16 containers) and Harat Faqish/Hazyaz (17 containers), respectively.

The total number of inhabitants with containers available for solid waste disposal rised to 13,725 by the end of June 1989, or about 40% of the current population. Coverage by containers is provisional only, however, as it is expected that in future additional containers may prove to be required. To identify the need for additional containers, the use and location of containers is reviewed every fortnight. The current locations of the containers is indicated on Fig. 5.4. (situation June 1989).

In the suq the containers are emptied daily, in other areas three times per week, except for Al Khabar, where they are emptied once per week. The protracted customs clearance procedure for the equipment, seriously hamper the solid waste collection and disposal activities. Measures are urgently needed to streamline customs clearance procedures.

Staff involved in the waste collection services includes:

- Head Operations, Container Supervisor, Sanitary Inspector, truck operators, compactor truck driver, grader operator, shovel operator, vacuum truck driver, labourers, sweepers, and workshop staff.

In the future, starting from July 1989 onward, the area covered with containers will be further extended:

- Harat Al Qana, Harat Al Rawdah, the area south of Harat Al Hafra and Madinat Al Musalla (first priority as soon as the second compactor truck will have become operational);
- As Safiya, Qa Rada, military camps, schools, etc., and the area south of the Dhamar - Al Bayda road (second priority).

These extensions will cover about 10,100 and 11,300 persons, or: some 29% and 32% of the present population, respectively.

5.3.6. Landfill site

Landfill site No. 4 as recommended in the Inception Report, has indeed been selected and approved by the local government. Countervalue funds up to YR 1.6 million have been earmarked for this purpose. Because the new dumpsite is not yet ready the training of landfill staff could not yet take place.

5.4. Future activities

5.4.1. Procurement

On the basis of the investigations into appropriate additional collection systems for the three densely populated districts in the centre of Rada: Harat Al Hafrah, Harat Faqish and Harat Al Qana, and Musalla, it was decided that additional equipment would be needed for solid waste collection there. In addition, replacement vehicles and additional equipment are expected to be required in the following years.

The following equipment is recommended to be procured:

- 2 Holder tractor units (already ordered)
- 1 compactor truck (scheduled for 1991)
- 1 back hoe (scheduled for 1992, see also par. 4.8 in chapter "Rainwater Drainage")
- 1 small front wheel loader ("bobcat"); scheduled 1990 (see also par. 4.8 in chapter "Rainwater Drainage").

The Holder tractor units will be especially designated to solid waste collection in the above mentioned four densely populated districts. Each unit is equipped with a lifting device, suitable for one container (either 1.6 m^3 or 1 m^3) and a front-mounted scraper blade. The intention is to use these units in narrow streets (especially in rocky areas) that are inaccessible to regular compactor trucks, so as to prevent the distances between individual containers from exceeding 50 - 75 m. They are also equipped with spraying installations, not only for cleaning containers, but also for spraying insecticides in fly infested areas or in stagnant pools, etc. The Holder tractor units have been tested in the Netherlands, in July 1989, and will - hopefully - be operational by the end of 1989.

A third compactor truck is expected to be required by 1991 at the latest, so that procurement should take place not later than in 1991.

For maintenance of solid waste container sites, other solid waste maintenance activities and for surface area maintenance, a back hoe and a bobcat will be required. The bobcat is a very small-size dozer/front wheel loader, to be used in otherwise inaccessible areas. Because other additional heavy equipment will be made available after 1990, it was concluded that the "Bobcat" unit should be provided with a small backhoe.

As a second priority it is envisaged that the existing 2 Nissan tipper trucks need to be replaced by 1991/1992. Likewise, a bulldozer will be required for the execution of a number of tasks, including maintenance of the dump site and grading of roads for drainage purposes. (see also par. 4.8 in chapter "Rainwater Drainage").

It was agreed that under the 1990 budget the following equipment should be purchased
- 5 - 10, five (5)m^3 containers for skip - loader truck
- additional workshop equipment for tyre repair, balancing and welding

For transport (1990):

- 1 Toyota hardtop, 4WD, diesel
- 1 Toyota pick-up, 4WD, double cabin, diesel
- 1 Toyota long-base, saloon type, 4WD, diesel

It was furtheron in principle agreed that following transport should be purchased in 1991-1992

- 2 Toyota hardtop, 4WD, diesel
- 1 Toyota pick-up, 4WD, double cabin diesel
- 1 Toyota bus, 25 persons, diesel

Because it was agreed that most of the (additional) heavy equipment and transport means as mentioned above will be only purchased in 1991-1992, it was proposed that for 1990 a small budget should be made available to rent heavy equipment (like "Poclain" Backhoe's), to be financed through Netherlands funds.

5.4.2. Workshop extension

As shown in table 5.4. not all components of the workshop have been realized already. An extension of the workshop is foreseen, including the construction of store rooms for mechanical/electrical equipment and accessories, a meeting/instruction room and a diesel/petrol station on site. This extension should be funded by so-called countervalue funds.