PAPER 6

# The Development of Sewerage and Sewage Treatment in Dubai

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## INTRODUCTION

In the early 1960's, Dubai was a vigorous but still relatively small trading community, with an estimated population of about 60,000.

The provision of piped water was placing a burden of increasing sewage loads on cesspool and soakaway arrangements built in less ample times and there was a resulting urgent need to protect the population in the congested central areas from health risks. The Ruler prudently commissioned a report on sewage collection, treatment and disposal to take account also of the emerging modern commercial development of the creek waterfront.

The question in Dubai, as in so many similar towns in the Middle East, was at what rate and to what extent could the city be reasonably expected to grow over a period of 30 years or so?

By modern standards, life in Dubai, in that recent era before the oil revolution, before world monetary inflation, and before mass travel, was remarkably simple. Although the first intimations of development in the Middle East were apparent, the phenomenal rate at which it has since occurred was beyond the imagination of the planners and designers of the day. Indeed, planners were often reprimanded for exaggerating when they attempted prophetic judgements.

Contemporary thinking was that the population of Dubai might ultimately reach 100,000, dependent on whether or not oil was discovered.

Ambitious plans were therefore made, at the Ruler's direction, to design and construct sewerage for an equivalent population of 45,000 in the town centre, and a first quarter of a sewage-treatment plant suitable for the ultimate population. It was also recommended and agreed that land should be reserved at the treatment-works site for doubling capacity to cope with a population of 200,000 in the more distant future.

While construction of the first phase of the sewerage network was taking place betwen 1968 and 1971 to provide Dubai with the first modern system in the Arabian Gulf, it was already evident that the population would rise significantly above 100,000. Instructions were therefore given in 1970 for phase II of the sewerage scheme to proceed, to serve a potential population of 200,000 but for the works to be expanded to its originally planned size of 100,000 population.

This pattern was repeated periodically throughout the early 1970's so that, by 1976 when a major review of sewerage was undertaken, 'ultimate' populations of 500,000 to 800,000 were commonly quoted!

Events were usually a few steps ahead of implementation of plans and designs, so rapid was growth. The original plans had, therefore, to be modified and supplemented to meet changing circumstances. This paper relates the highlights of the development of the scheme, concentrating on the treatment plant where effluent quality was improved as interest in reuse quickened, in parallel with raising capacity. Innovative treatment changes had to be introduced to meet rapidly rising sewage loads and to make the best use of a restricted site.

## ORIGINAL SEWERAGE MASTER PLAN

The simple concept, shown in Figure 1, was to provide a series of drainage districts each served by a local pumping discharging through trunk sewers into one of five main pumping stations, three of which would act as staging posts along the route to the treatment works.

The township of Dubai is divided by the Creek, with the main centres of development close to the sea. Therefore, to provide remoteness from the development that existed at that time the sewage-treatment works site was selected some seven kilometres inland alongside the Creek.

The option of minimal preliminary treatment with discharge to sea had already been rejected on amenity and water-conservation grounds. While initial works design was for a'partially'-treated effluent before discharge to the substantial volume of the Creek, the expectation from the outset was that quality would eventually be improved for effluent reuse.

## PLAN DEVELOPMENT

The original master plan was still valid in 1970 when phase II was ordered, but growth and development aspirations two or three years afterwards precipitated a major reappraisal which was carried out in 1976.

Remarkably, the site of the recently-built treatment plant was expected to be swamped by advancing development. Even if growth did not materialise it would be put under pressure to give way to increasingly popular creekside leisure activity. Figure 2 shows the development expectations in 1976. Alternative plans were therefore described, allowing for the north-eastern and south-western extremities of the expanding towns to be served by new treatment works or for one very large, even more remote, but more centrally located works to serve all needs. In either plan the options of continuing to use the 'existing' works in its 100,000 or 200,000 population form, or abandoning it altogether, were included. The elements of this flexible approach are shown in Figure 3.

Two main plans were envisaged. Plan 1a proposed a single new treatment plant located south of the existing works, 4 km from the creek's south eastern shore. Plan 2a suggested two new works, one to serve Deira and central Dubai and the other for the fast-developing coastal strip from Al Jumeirah towards Jebal Ali. In both plans there was an option to retain or abandon the existing works.

The resultant equivalent populations to be served by the treatment plants are shown in Table I.

In each option, therefore, a substantial new plant was proposed, to be supplemented in three cases by smaller ones.

Although plans 1 were expected to be more expensive than their counterpart plans 2, and plan 1a was the most expensive, the differences were not substantial, and plan 1a was preferred on environmental grounds of relative remoteness from future development. There was an added advantage in the potential for recharging the aquifer at T'Awir with highly-treated effluent.

Because the estimated total population of 800,000 was, at that time, still an upper limit (the lower one being 500,000) a further modification of the preferred plan was proposed. It was suggested that a five-year approach be adopted to retain the existing plant for an equivalent population of 100,000 and to limit the size of the new plant to an equivalent 400,000. The position could be reviewed again in 1982 and a decision then be taken to double or otherwise multiply the capacity of the new plant.

## THE POSITION IN 1981

By 1978, the capacity of the existing sewage-treatment plant had been raised to its originally conceived limit of a design population of 100,000 treating an average daily flow of just under 16,000 $m^3$/day to produce an effluent quality of 50 mg/l SS and 50 mg/l BOD.

Before the extensions were started in 1976, the flow was beginning to outstrip the original plant and a temporary lagoon was quickly conceived to raise capacity to about 24,000 $m^3$/day. However, interest in effluent reuse was quickening and the lagoon design was modified to produce a lower and near-constant-volume higher-quality effluent (5,500 to 6,500 $m^3$/day to a 30: 20 SS : BOD standard). The capacity of the works was thus limited to an average daily flow of 21,500 $m^3$/day.

Flows were continuing to rise, partly from new sewerage connections and partly from increased per capita sewage yield, and the average daily flow early in 1981 had risen to 22,700 $m^3$/day, with a consequent deterioration in main works effluent quality.

A decision had been taken by then eventually to abandon the existing works, in favour of a large single works similar to that envisaged in the 1976 master plan.

Because the planning, design and construction of the new works would take several years, it was also decided to make best possible use of the existing plant, but at minimum cost. The rest of this account is concerned with those further modifications and with parallel conceptual and detailed design and construction of a treated-effluent storage and distribution system.

## SEWAGE VOLUME PREDICTIONS

While preliminary work was being done on the existing works uprating designs, flows continued to rise sharply and reached about 26,000 $m^3$/day by August of that year. Predictions for 1985, and beyond, foresaw flows increasing to anything between 35,000 and 55,000 $m^3$/day before the new works could be made available. The lower figure could only be achieved with severe control of house-connection work which would have been unpopular.

## TREATMENT PLANT UPRATING OPTIONS

Without substantial and unwelcome investment on the existing plant, the options for uprating were necessarily limited and had to incorporate innovative use of capacity to achieve the objectives of maximum volumetric and qualitative treatment.

Two main options were offered:

Uprate to a capacity of 27,000 $m^3$/day and a single effluent standard of 20 : 20, but with virtually no treatment unit additions.

Uprate to 32,000, 35,000 or 45,000 $m^3$/day, with increasing but still economic treatment-unit additions.

The effluent quality was raised because natural potable-water sources were becoming overtaxed and the cost of providing water was consequently rising. Considerable savings were to be had in diverting potable-water use away from irrigation by increased effluent reuse, and chlorination economics dictated the adoption of a fully nitrified 20 : 20 standard.

Figures 4 to 7 illustrate diagramatically the basic treatment plant and the uprating options.

The innovations common to all the uprating options were:-

By analysis of raw sewage pumping patterns it was possible to demonstrate that flows peaked lower than allowed for in the original design. Hydraulic capacity was therefore available to enable biological performance to be improved.

Further additional hydraulic capacity could be utilised in primary sedimentation tanks and biological filters by abandoning arrangements for effluent recirculation in the existing works.

A nitrifying filter deodorising tower had recently been built to scrub gases collected from the inlet works, but it has not yet been commissioned. If flows through it could be raised to its maximum hydraulic capacity an additional liquid-flows treatment facility was available as an adjunct to deodorisation.

Each of the options modified the use of the existing lagoon. It had been equipped with integral aeration, settling and aerobic digestion to operate in parallel with the biological filters. It was now proposed to use it as a nitrification stage after primary filtration, with denitrification of RAS carried out in an anoxic zone at the head of the lagoon into which a varying proportion of primary settled sewage according to total loading would be fed to provide substrate.

Accommodation of flows above 27,000 $m^3/d$ necessitated intermediate settling between filtration and aeration to reduce solids loading. This was to be achieved at 32,000 $m^3/d$ by high loading of the old primary sedimentation tanks to relieve two for intermediate settling.

If flows were raised to 35,000 $m^3/d$ it was possible, by the addition of a single intermediate settling tank, to produce 27,000 $m^3/d$ of 20 : 20 effluent with the balance being discharged to the Creek at a standard better than 50 : 50. However, the arrangement was particularly sensitive to upsetting because it used all units to the limit.

Two variations were therefore offered. One would divide flows after intermediate settling, part going to the aeration lagoon and part to an oxidation/maturation pond. The other would add a further line of biological filters and humus tanks to secure a 20 : 20 standard effluent for all flows.

At 45,000 $m^3/d$, because all the available primary settling capacity was needed, four new final settling tanks would have to be built so that the existing humus tanks could be used for intermediate settling.

All options incorporated thickening of surplus activated sludge before digestion. In the case of the 27,000 $m^3/d$ design one of the existing sedimentation tanks would be converted for thickening. A new thickening arrangement would be built for all other options.

## PREFERRED TREATMENT APPROACH

It was decided that because it would probably be several years before the reuse of effluent rose to 27,000 $m^3/d$ it was not necessary to adopt a design incorporating high-quality treatment for more than that volume.

It was recognised that sewage flows would continue to rise above that level before the new works was available, but the increased cost of additional capacity for a relatively short investment period was unwelcome.

A compromise between volume and cost was eventually selected, by adopting the restricted 35,000 $m^3/d$ option. A firm 27,000 $m^3/d$ would be treated to a 20 : 20 standard and the balance to about 30 : 50 for discharge to the Creek. Flows above 35,000 $m^3/d$ could be diluted with the balance of higher-quality effluent not immediately used for irrigation, or a decision could be taken to add further treatment units, if the flow increase became intolerable.

Figure 8 provides a layout of the treatment plant and indicates the recently constructed modifications and the possible future additions to raise the total treated flow to 45,000 $m^3/d$ and the higher quality effluent portion to 37,000 $m^3/d$.

## OPERATING EXPERIENCE SINCE WORKS COMMISSIONING

The uprated treatment plant was commissioned in November 1983. There was a risk of producing a filamentous culture if humus sludge was used as a 'seed' for the activated sludge process. A 'natural culture' of purifying micro-organisms was therefore developed by filling the aeration tank with settled sewage and aerating this for a few days until the COD and ammoniacal nitrogen ($NH_4N$) content had been reduced to a low level. At this stage settled sewage was fed at a continuous, low rate to the aeration tank.

The pH, D.O., MLSS and settled COD and $NH_4N$ levels were monitored daily and as soon as nitrification (oxidation of ammonia) was essentially complete the flows were gradually increased. Results obtained during this initial commissioning period are summarised in Table II.

### Nitrite problems

Generally the results obtained during this period were satisfactory. However, the final effluent had a high chlorine demand due to the presence of excessive concentrations of nitrite.

On checking it was found that sulphide oxidation was incomplete in the anoxic zone and sulphide residuals in the order of 2 mg/l were present. These levels had to be reduced as sulphide is toxic to protozoa and may inhibit the growth of nitrifying organisms, so the simple expedient of removing some of the wood baffles separating the anoxic and aerobic zones was tried. This had the dual effect of allowing highly-aerated liquor into the anoxic zone which promoted rapid oxidation of sulphides, as well as reducing the retention period in the zone. The modification did not significantly affect the denitrification (reduction of nitrate to nitrogen gas) rate in the zone, but immediately reduced the sulphide levels to less than 0.5 mg/l. Within a few days oxidation of nitrite to nitrate was essentially complete.

## COMMISSIONING THE AEROBIC DIGESTOR

As soon as the MLSS content in the aeration tank reached about 2600mg/l excess sludge was pumped to the aerobic digestor. In the digestor, sludge is thickened by switching-off the eductor-aeration system for a period before feeding with sludge, to allow the sludge to settle and consolidate, the feed then displaces supernatant liquor over a weir. Initially, the formation of a thick scum on the surface of the digestor reduced process efficiency, but after a few weeks little scum persisted and clear liquor was displaced.

## PROBLEMS WITH THE ACTIVATED-SLUDGE PROCESS

All proceeded well with the commissioning of the activated sludge process until mid-January when the process was treating 26,000 $m^3/d$ of settled sewage and filter effluent in a 1:1 ratio. There was a power cut and after this the ammonia levels in the effluent climbed to 15 mg/l despite reducing the flows to 21,000 $m^3/d$. However, after a few days the ammonia levels began to drop and it was possible to increase the flows to 26,000 $m^3/d$ again.

Unfortunately, almost immediately after this there was a blockage problem on the primary tank desludging lines and the tanks were not desludged for a few days. As a result considerable amounts of primary solids discharged into the aeration tanks and within a few days a thick scum formed on the final tanks. The scum contained large numbers of filamentous organisms.

Scum problems persisted for almost a month and then, to compound operational difficulties, for inexplicable reasons (possibly through the discharge of a toxic trade waste) nitrification was inhibited and the sludge started bulking. Although full nitrification was regained in the next few weeks the mixed liquor SVI climbed from 100 ml/g to 250 ml/g and then, almost as rapidly, began to fall again. However, the recovery was shortlived and within a week or so the SVI increased again.

At this time it was felt that drastic action would be needed to control the SVI and scum problems and chlorination of return sludge was therefore practised.

The effects of chlorination at 3.mg/l were quite dramatic. Within two days, filaments and protozoa reduced significantly and scum formation on the final tanks was virtually eliminated. SVI continued to rise, however and nitrification began to be inhibited, despite reducing chlorination to 1.5 mg/l.

Full nitrification was restored by stopping chlorination and feeding the aeration tank with settled filter effluent only but, in the absence of soluble carbonaceous substrate, the effluent became turbid due to the presence of lysed bacterial solids, and the scum problem returned. Then, contrary to all expectations, the SVI deteriorated rapidly to over 400 ml/g and scum production decreased.

It was decided that improvement and control of sludge quality might best be established by reverting to a settled sewage feed, chlorinating RAS at a sufficiently low rate in order not to affect nitrification, and by gradually building flow to design loadings as SVI improved. Scum control became secondary because it did not affect final effluent quality. Flow was therefore reduced to 10,000 $m^3/d$, MLSS to 2200 mg/l and RAS chlorination at a dose rate of 1.5 mg/l was reintroduced.

SVI dropped to 350 ml/g as settled sewage flows were increased to 15,000 $m^3/d$. Chlorination was reduced and then stopped at the end of April to preserve nitrification and SVI continued to improve to 200 ml/g as flows were increased to 20,000 $m^3/d$ by mid June. As this paper goes to print, the signs of process stabilisation are promising and further results will be discussed when the work is presented to the conference.

EFFLUENT REUSE CONCEPTS

Concurrent with the decision to uprate the existing sewage-treatment works, instructions were given to design effluent-storage and distribution facilities to satisfy rising demand for irrigation-quality treated effluent.

Irrigation practice had already established the use of:

Chlorination-treated effluent, transmitted by pipeline to Saffra Park, to the cement works, and for roadside irrigation enroute.

Brackish water from the Mazar wellfields to the south east of the town for use at Union Square, Al Nasr Square, and the new Dubai hospital.

The design objectives were to assess immediate and long-term demands, to match them with sources, to make proposals for treatment storage and distribution, and to ensure public health.

Table III summarises current (1982) use, immediate potential, and short-term future-demand predictions.

The Municipality has an '8% green' policy for long-term future irrigation of public areas, including road reservations and a projected use of 121,500 $m^3/d$ is possible ultimately, or 130,000 $m^3/d$ if a recreation area is developed around the Creek.

Early water availability comprises 27,000 $m^3/d$ from the uprated existing sewage-treatment plant, 5,000 $m^3/d$ to 6,000 $m^3/d$ from a temporary sewage-treatment plant on the east side of the Creek at Hor AlAnz, and 3,500 $m^3/d$ of brackish water from the Mazar wellfield, together totalling 35,500 $m^3/d$ to 36,500 $m^3/d$.

Effluent from the new sewage-treatment plant is expected to reach 55,000 $m^3/d$ soon after commissioning and 130,000 $m^3/d$ by 1994, by which time the other treatment-works sources should have been diverted to the new works. There should, therefore, be enough reclaimed water to meet estimated demands.

According to plant salinity tolerance, it was calculated that between 1.1 and 1.7 times crop water requirement is necessary to ensure the leaching of surplus salts. By averaging plant distribution and the use of species common in Dubai a maximum need of 15 mm/day can be calculated for grassed areas, and 20 mm/day for trees and scrubs.

The following quantities result:

| | | |
|---|---|---|
| Grass | 15 $l/m^2/day$ | |
| Shrubs | 15 l/day | ) averages |
| Trees | 45 l/day | ) |

| | |
|---|---|
| Fully landscaped road reservations | 180 $m^3$/km/day |
| Partially landscaped (single row trees/shrubs) | 15 $m^3$/km/day |
| Parks (predominantly grass) | 150 $m^3$/ha/day |

Highest mean demands occur between May and August. Lowest means prevail from December to February and are about one third of the maximum uses.

As has been described, the main potential outlet for effluents in Dubai is landscape irrigation. Some Gulf States have decided that only potable water will be used for areas where the general public have access. It has been recommended and agreed for Dubai that strictly-controlled effluent irrigation of public areas will be pursued. Stored effluent will be disinfected by chlorination and irrigation will take place largely at night.

World Health Organisation health-criteria guidelines suggest effective removal of bacteria plus some removal of viruses for non-potable Municipal re-use. State of California guidelines are more specific, in requiring a microbiological limit of 23 coliform organisms/100 ml for the irrigation of landscaped areas and this standard has been adopted for Dubai.

## EFFLUENT-SYSTEM PRINCIPLES

The pressure requirements of an effluent distribution system, as with any other water system, are met from strategically-sited high-level reservoirs or by local boosting of a low-pressure bulk supply. The third possibility of pumped pressurisation only at source, the treatment plant, was rejected because irrigation demands are large-volume short-period ones and unreasonably large pipelines would be required to satisfy them.

It was convenient to divide the town initially into three main distribution zones, one on each side of the creek, and one for the Jumeira coastal strip. In due course six zones will be required as shown in Figure 9.

Existing arrangements for irrigation are based predominantly on hoses and drip feed, but there are a few pop-up rotary impact sprinklers and pop-up spray heads. All of the existing methods encounter problems of blockage and, in the case of hoses, of non-uniform and often extravagant application, leading to drainage difficulties.

Different methods of irrigation are recommended according to the Municipality Parks Department's planting wishes. Low angle pop-up sprays are most suitable for grassed areas and for mixed road reservation planting; pop-up impact sprinklers for large grassed areas; bubblers or hose for small shrubs, especially those with lower salinity tolerance susceptible to foliar scorch; and bubblers, hose or drip irrigation for supplementary watering of trees and larger shrubs.

Investments in irrigation systems should be gradual to match development and to allow time for experimentation and experience building. Maximum flexibility of supply is therefore necessary to meet demand variations. Whereas automatic systems when installed will operate at night, point applications will continue in daylight hours.

It was therefore decided to mix supply arrangements, by relying initially on high-level zone storage and by gradually introducing local ground-level storage from which pressure boosting can be introduced, as necessary. In two of the initial three distribution areas, substantial elevated reservoirs have already been built for brackish-water supply, (1,800m³ at Al Mankhool and 900m³ at Burji Naharji) and a further 900m³ of tower storage will soon be provided at Jumeirah. The extent to which future zones will be furnished with elevated storage will depend on how irrigation practice evolves. In order to reduce head requirements at the treatment works, 1,000m³ ground-level storage and booster pumping is provided at each elevated tank.

The maximum head on the transmission mains is thereby maintained at 55 metres and pressure-reduction valves are placed on those take-off points drawing direct from the mains, rather than from zone distribution. The elevated tanks provide an operating head of 30 metres on each distribution system which offer a minimum outlet pressure of 15 metres. Each junction is provided with isolating valves.

The existing brackish-water system is to be retained to supply Union and Nasr squares, the new hospital and the road reservations along the routes to those locations but other links will be capped.

At the sewage-treatment plant, two storage lagoons are provided with a combined capacity of 10,000m³, offering a short reserve during periods of works breakdown. They are constructed as sand embankments lined with high density polyethylene water retaining membrane. They will normally operate in series but either can be isolated for repair or maintenance. Normal draindown will stop at 300 mm above tank flow to prevent hydrostatic uplift but complete draining is possible by the use of a pumped groundwater lowering installation.

Two chlorination points are provided, one upstream and one downstream of storage. The initial dose will be low at about 3 mg/l to remove most of the faecal coliforms. The second dose, of about 9 mg/l to obtain breakpoint conditions and provide a residual in the transmission mains, will enjoy a contact time of at least an hour compared to the 30-minute minimum required to ensure viral control. A control option is also available of administering breakpoint dosing before storage, and top-up dosing before transmission. The choice will depend on experience with effluent quality.

Future treated effluent from the new sewage-treatment plant, will serve zones 4 and 5 and be connected into the transmission system at Jumeirah and at Al Garhoud bridge to supplement supplies as irrigation provisions are extended.

## TABLE I

### EQUIVALENT POPULATION TREATMENT WORKS CAPACITIES BY MASTER PLAN OPTION

| Treatment Plants | Plans | | | |
|---|---|---|---|---|
| | 1a | 1b | 2a | 2b |
| Existing | - | 200,000 | - | 200,000 |
| New southern | 800,000 | 600,00 | - | - |
| New eastern | - | - | 600,000 | 500,000 |
| New western | - | - | 200,000 | 100,000 |

## TABLE II

## COMMISSIONING THE ACTIVATED-SLUDGE PROCESS - ANALYTICAL DATA

| Date | Feed Rate $m^3/d$ | MLSS mg/l | SVI ml/g | pH | Effluent Analysis (mg/l) SS | COD | BOD | $NH_4N$ |
|---|---|---|---|---|---|---|---|---|
| 1.11.83 | Aeration tank filled with settled sewage | | | | | | | |
| 7.11.83 | - | 189 | - | 7.5 | (77) | - | - | (10) |
| 10.11.83 | 4000 | 189 | - | 7.5 | (80) | - | - | (0.3) |
| 12.11.83 | 4000 | 530 | - | 7.2 | 23 | 175 | - | 21 |
| 15.11.83 | 8000 | 820 | - | 6.8 | 23 | 101 | 10 | - |
| 20.11.83 | 13000 | 1616 | 99 | 7.2 | 21 | - | 10 | 1 |
| 5.12.83 | 18000 | 2864 | 140 | 6.9 | 21 | - | 14 | 0.9 |
| 8.12.83 | 20000 | 3104 | 119 | 6.9 | 19 | - | 14 | - |
| 19.12.83 | 21000 | 2368 | 118 | 6.8 | 19 | - | - | 12 |
| 27.12.83 | 23000 | 2684 | 108 | 6.9 | 18 | - | 12 | 2 |
| 4.1.84 | 24000 | 2904 | 96 | 7.0 | 23 | - | 14 | 0.6 |
| 8.1.84 | 26000 | 2720 | 93 | 6.9 | 19 | - | 14 | 3 |
| 16.1.84 | 26000 | 2920 | 97 | 6.9 | 20 | - | 19 | 7 |

## TABLE III

## EFFLUENT DEMAND ESTIMATES

| Use | Estimate of Demand ($m^3/d$) Existing | Potential Immediate | Potential Short Term future |
|---|---|---|---|
| **Municipal** | | | |
| Road reservations | 1,200 | 5,500 | 14,000 |
| Jaffa Park | 2,300 | 9,000 | 9,000 |
| Other public areas and buildings | | 2,500 | 8,000 |
| **Private** | | | |
| Palace gardens and environs, and hotels | 2,500 | 4,500 | 13,000 |
| **Industrial** | | | |
| Cement works | 2,000 | 2,000 | 2,000 |
| Mushrif National Park | - | - | 18,500 |
| | 8,000 | 23,500 | 64,500 |
| + Brackish Supply to Deira | 3,500 | | |

Appendix I

## BASIC TECHNICAL DATA

| | | |
|---|---|---|
| Average sewage flow | | 35 000$m^3$/d |
| Peak sewage flow for biological treatment | | 70 000$m^3$/d |
| Crude sewage analysis | SS | 314mg/l |
| | BOD | 307mg/l |
| (Feb 84 results) | COD | 778mg/l |
| | $NH_4N$ | 47mg/l |
| | $H_2S$ | 16mg/l |

Treated effluent standards

| | |
|---|---|
| 1) 27 000 $m^3$/d | 20mg/l SS |
| | 20mg/l BOD |
| 2) 8 000 $m^3$/d | 50mg/l SS |
| | 50mg/l BOD |

| PROCESS | UNIT | |
|---|---|---|
| Preliminary Treatment | Screening/Disintegration | |
| | No. of Units | 2 |
| | Type. Mechanically raked, bar screens | |
| | mm bar spacing | |
| | Gorator disintegrators | |
| | Grit Removal | |
| | No. of Units | 2 |
| | Type: Aerated, spiral flows channels | |
| | Grit Dewatering/Cleaning | |
| | Type: Cylcone, reciprocating rake | |
| | Preaeration/Balancing | |
| | No. of Units | 4 |
| | Retention | |
| | Average air flow | 1.5$m^3$/$m^3$ sewage |
| Primary | Primary Sedimentation Tanks | |
| | No. of Tanks | 4 |
| | Type: Circular, radial flow | |
| | Tank Diameter | 19.8m |
| | Average retention | 2.6h |
| | Average upward flow velocity | 1.2m/h |
| 1st Stage | Biological Filters | |
| | No. of Filters | 6 |
| | Type: Circular, media 65mm rounded gravel | |
| | Average settled sewage flow | 22 000$m^3$/d |
| | Design BOD loading | 0.63kg BOD/$m^3$.d |
| | Intermediate Humus Tanks | |
| | No. of Units | 2 |
| | Type: Circular, radial flow | |
| | Tank diameter | 19.8m |
| | Average retention | 3.2h |
| | Average upward flow vecolity | 0.95m/h |

| PROCESS | UNIT | |
|---|---|---|
| 2nd Stage Biological Treatment | Aeration Tank/Anoxic Zone | |
| | No. of Units | 1 |
| | Overall tank dimensions | Length 112m<br>Width 20.8m<br>Liquid depth 4.3m |
| | Anoxic Zone Mixers | |
| | No. of Units | 2 |
| | Type of Mixer: Submersible, Flygt 7.5kW | |
| | Aerators | |
| | No. of Units | 8 |
| | Type of Aerator: Vertical shaft, floating, surface aerator | |
| | 2 No. | 45kW |
| | 6 No. | 30/20kW |
| | Average settled sewage flow | 13 000m³/d |
| | Average filter effluent flow | 14 000m³/d |
| | Retention period (overall) | 6.9h |
| | F:M ratio | 0.23kg BOD/kg MLSS.d |
| | Final Tanks | |
| | No. of Tanks | 4 |
| | Type: Circular, radial flow | |
| | Tank Diameter | 18.3m |
| | Average retention | 3.0h |
| | Average upward flow velocity | 1.2m/h |
| | Deodorising Tower | |
| | No. of Units | 1 |
| | Volume of Media | 1 455m³ |
| | Filter effluent flow | 8 000m³/d |
| | BOD loading | 0.9kg BOD/m³.d |
| | Tower Effluent Settlement Tank | |
| | No. of Tanks | 1 |
| | Type: Circular, radial flow | |
| | Tank diameter | 19.8m |
| | Retention | 2.8h |
| | Upward flow velocity | 1.1m/h |
| Sludge Treatment | Aerobic Storage/Thickening | |
| | No. of Tanks | |
| | Type: Circular, eductor aeration | |
| | Tank capacity | 950m³ |
| | Retention (thickened sludge) | 10 - 14 days |
| | Anaerobic Digestion | |
| | No. of Tanks | 2 |
| | Tank Diameter | 18.9m |
| | Retention | 23.8d |
| Disinfection | Chlorination | |
| | Type: Chlorine drums 2 No. vacuum chlorinators | |
| | Chlorinator capacity | 14kg/h each |
| | Average chlorine dose rate | 10mg/l |

Appendix II

Cumulative Sewerage System Installation Achievement and System and Treatment Cost from 1969 to 1983

| Year | House Connections<br>No. | Sewerage System Length<br>Km. | Pumping Stations<br>No. | Cost (Uncorrected to present day)<br>dhm millions |
|---|---|---|---|---|
| 1969 to 1977 | | | | 264 |
| 1978 | 11,900 | 275 | 53 | 366 |
| 1979 | 18,200 | 370 | 69 | 500 |
| 1980 | 20,700 | 410 | 87 | 607 |
| 1981 | 21,800 | 439 | 90 | 691 |
| 1982 | 24,100 | 480 | 102 | 752 |
| 1983 | 25,300 | 500 | 106 | 819 |

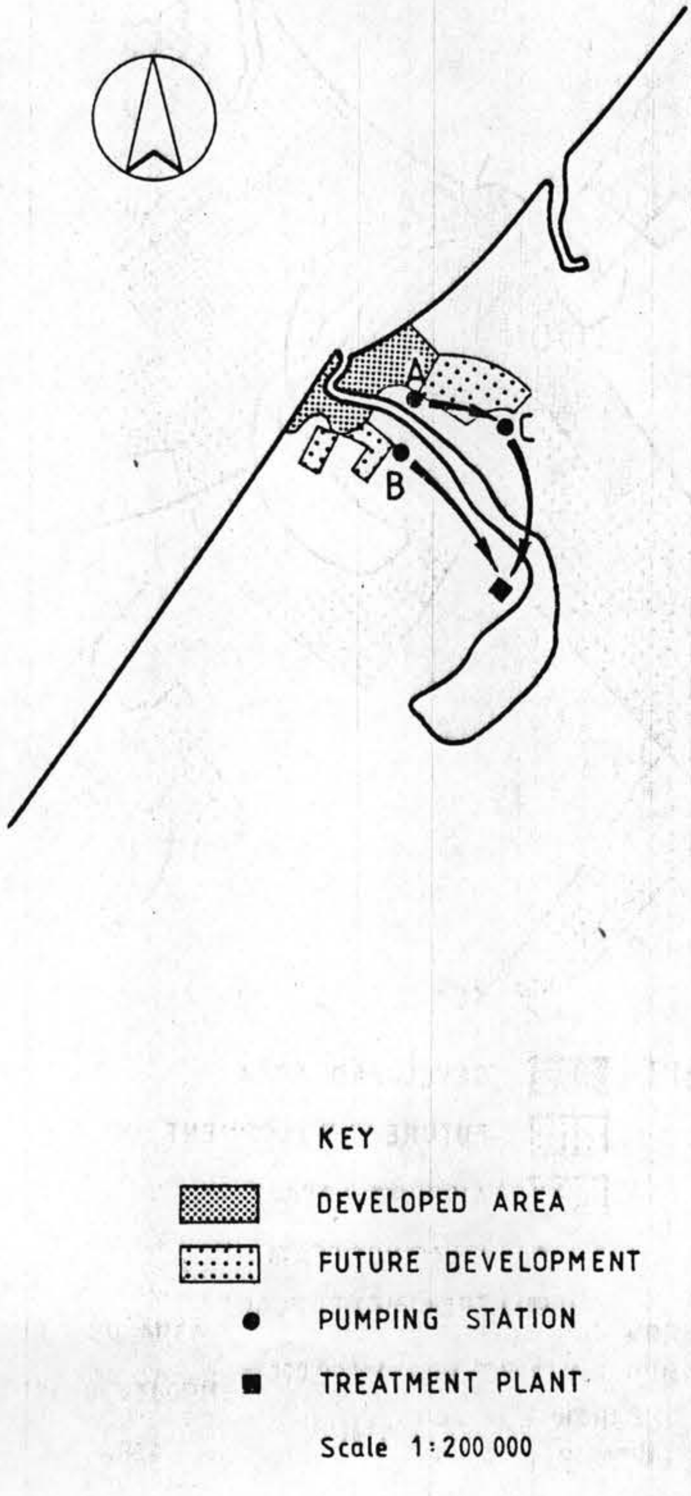



FIG. 1 THE 1965 MASTER PLAN ELEMENTS

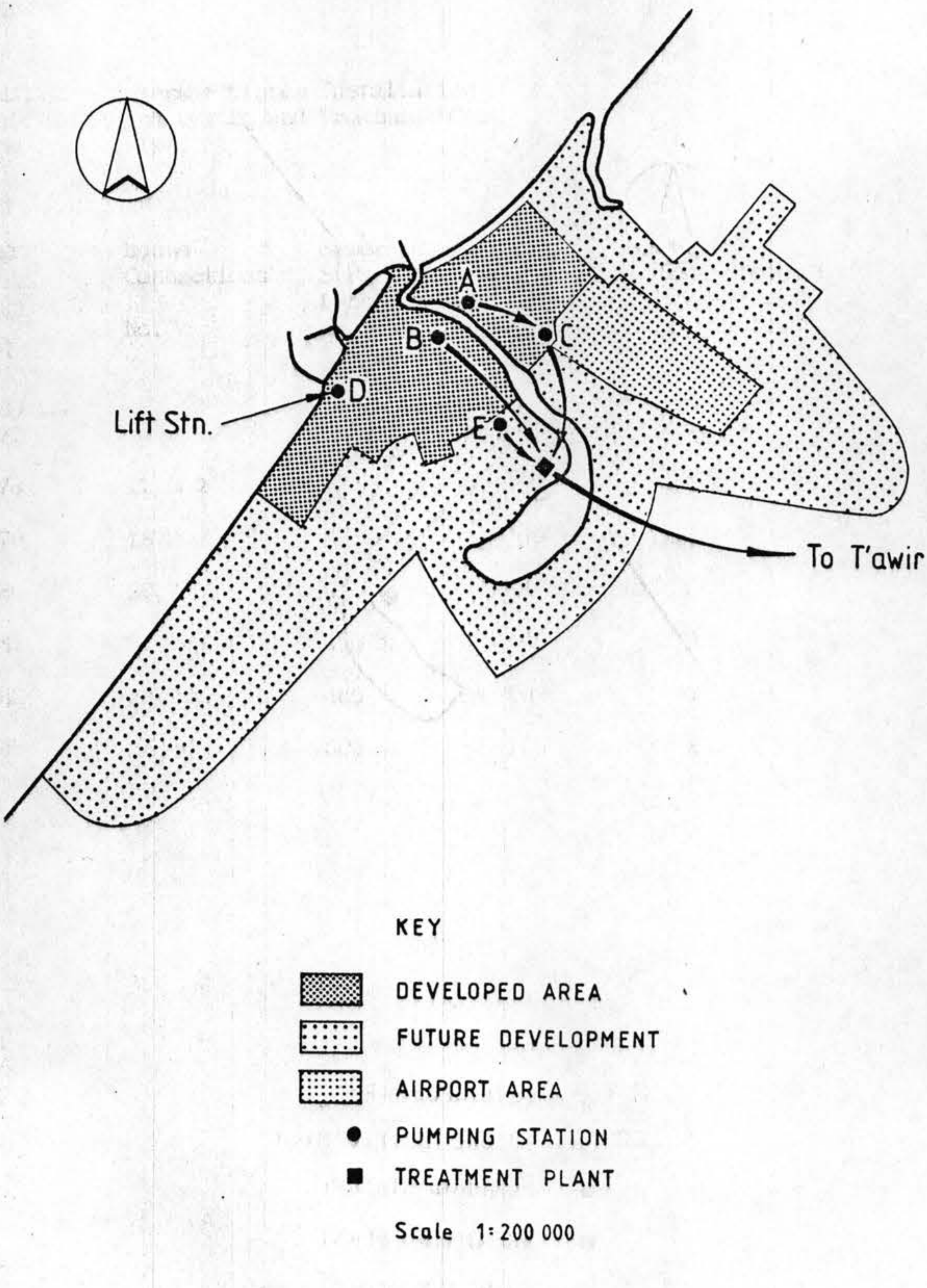
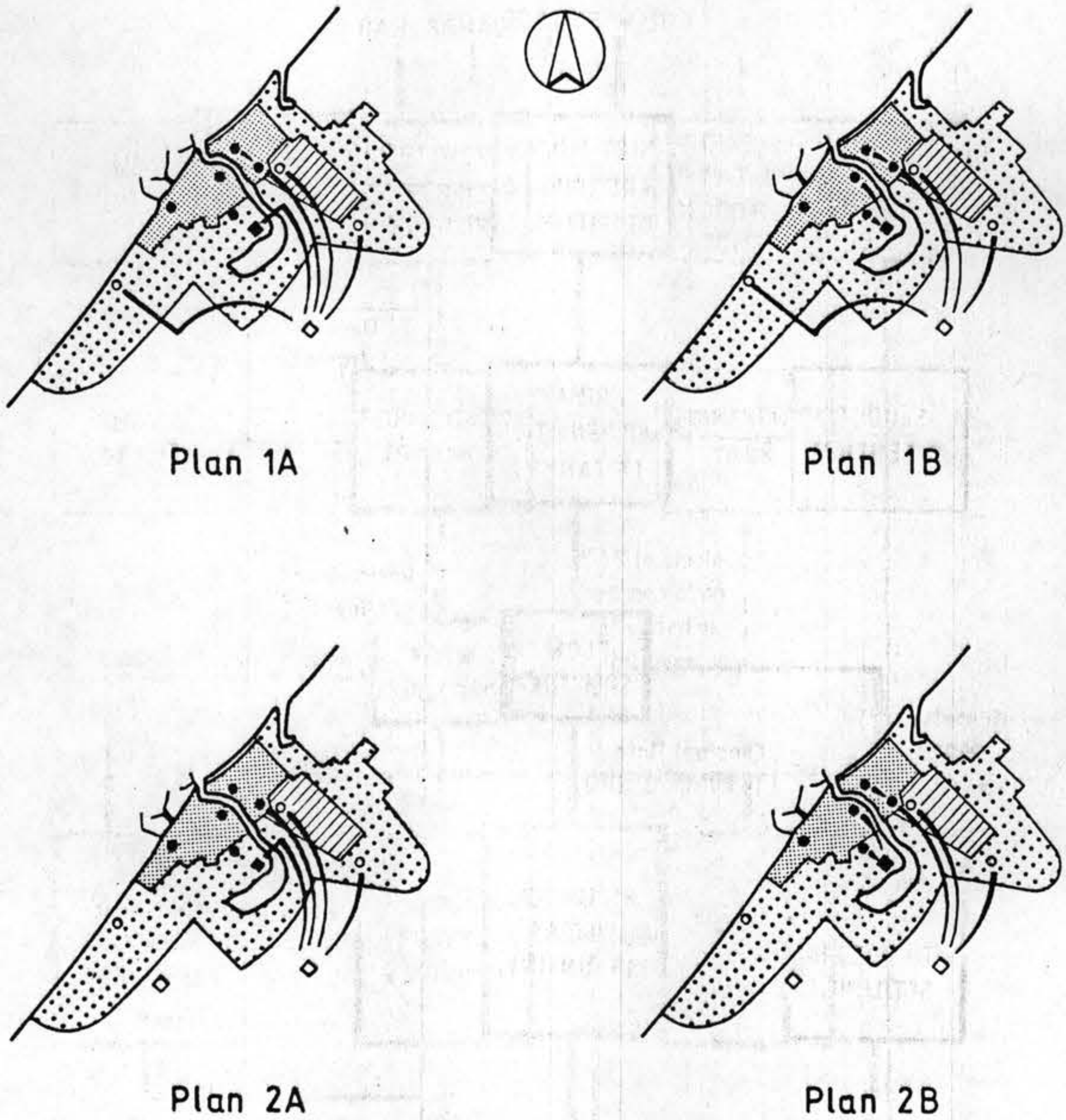



FIG. 2 GROWTH BY 1976 – RELEVANCE OF OLD PROPOSALS




KEY

DEVELOPED AREA
FUTURE DEVELOPMENT
AIRPORT AREA

■ EXISTING WORKS
• EXISTING PUMPING STATION
□ NEW WORKS
∘ NEW PUMPING STATION

Scale 1:500 000

FIG. 3 1976 MASTER PLAN OPTIONS

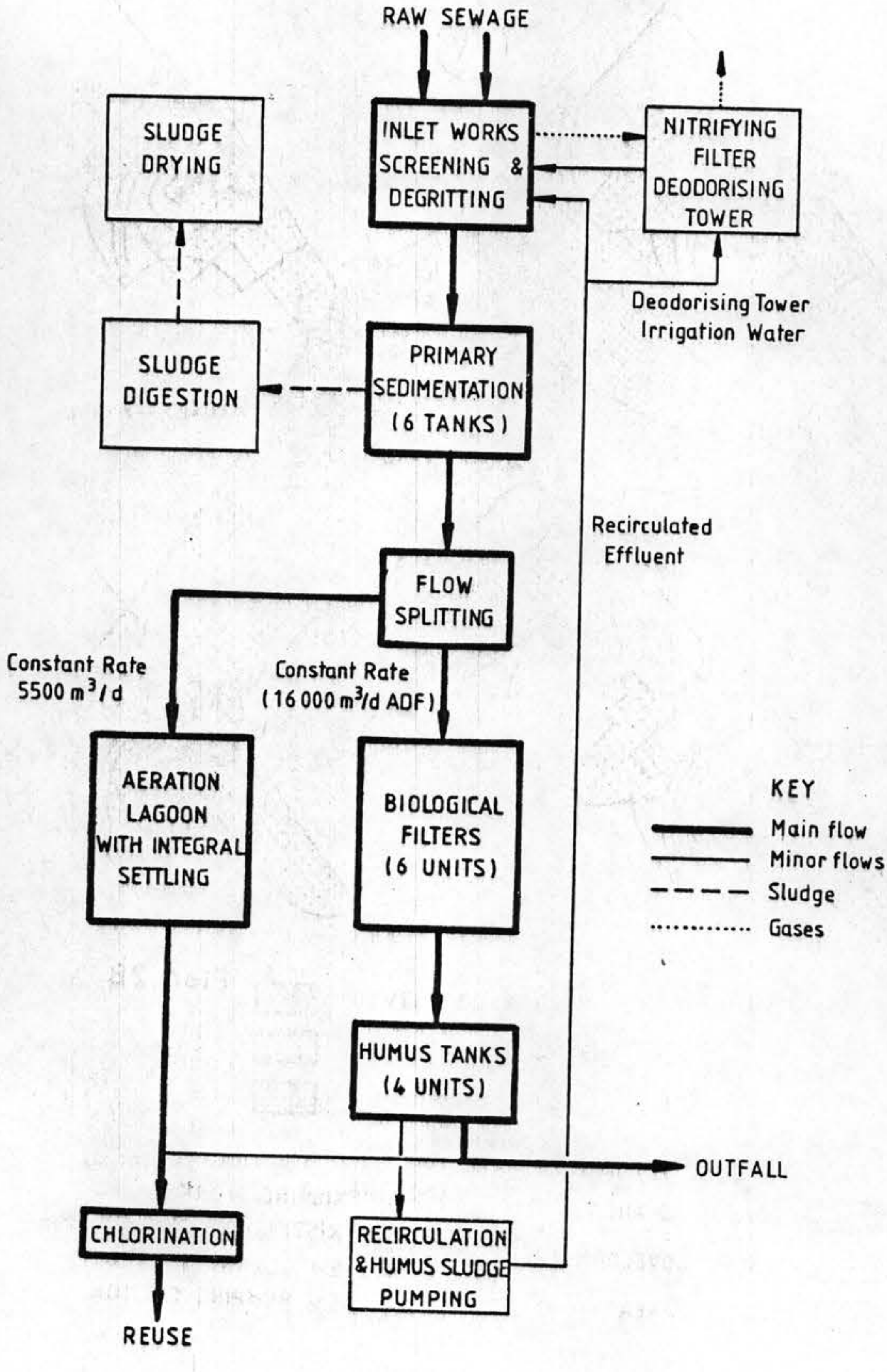



FIG. 4 BASIC PLANT $5\,500 m^3/d$ to 30:20 standard
$16\,000 m^3/d$ to 50:50 standard

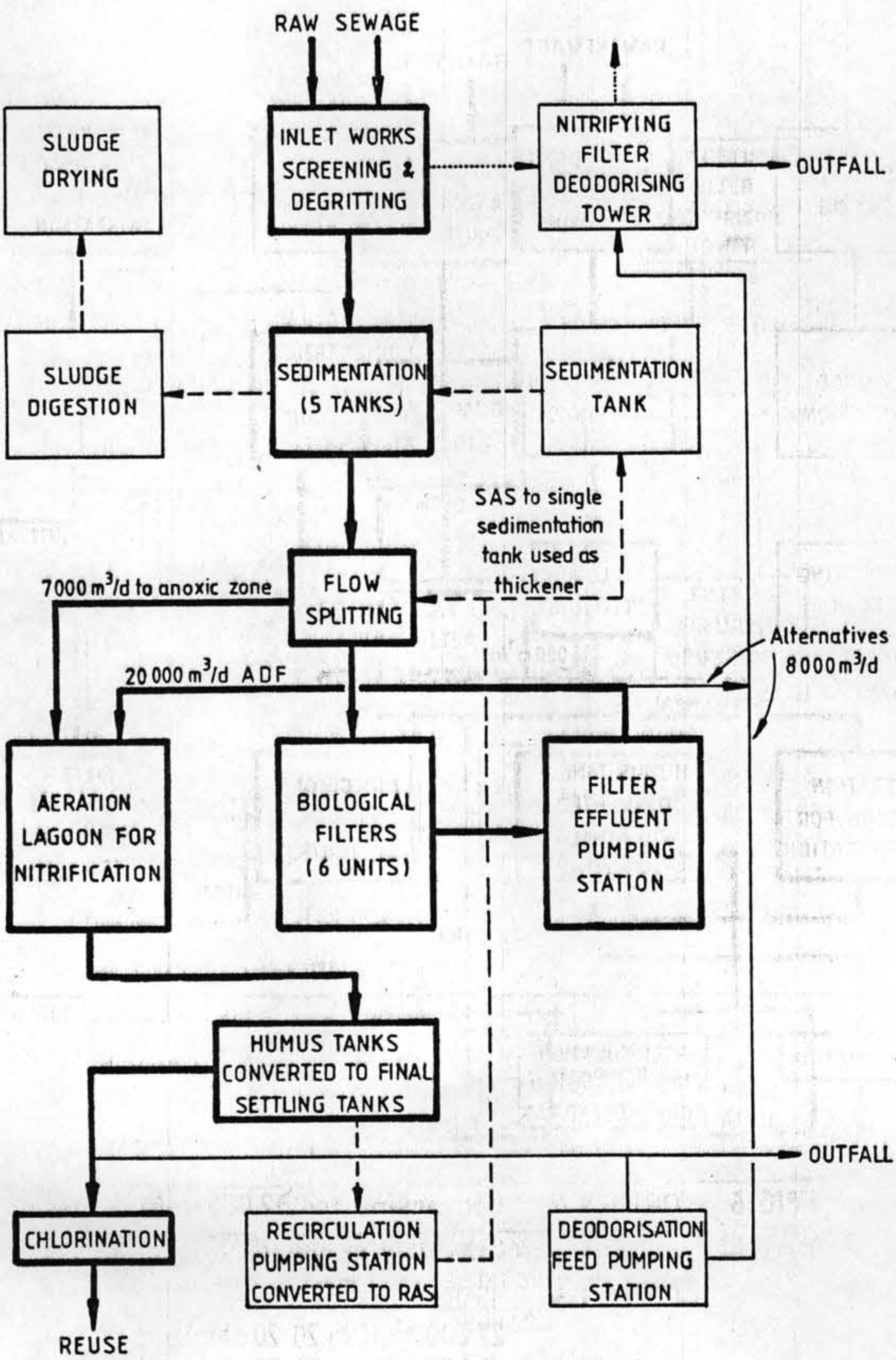



FIG. 5 OPTION 1 Conversion for 27 000 $m^3/d$ to 20:20 standard

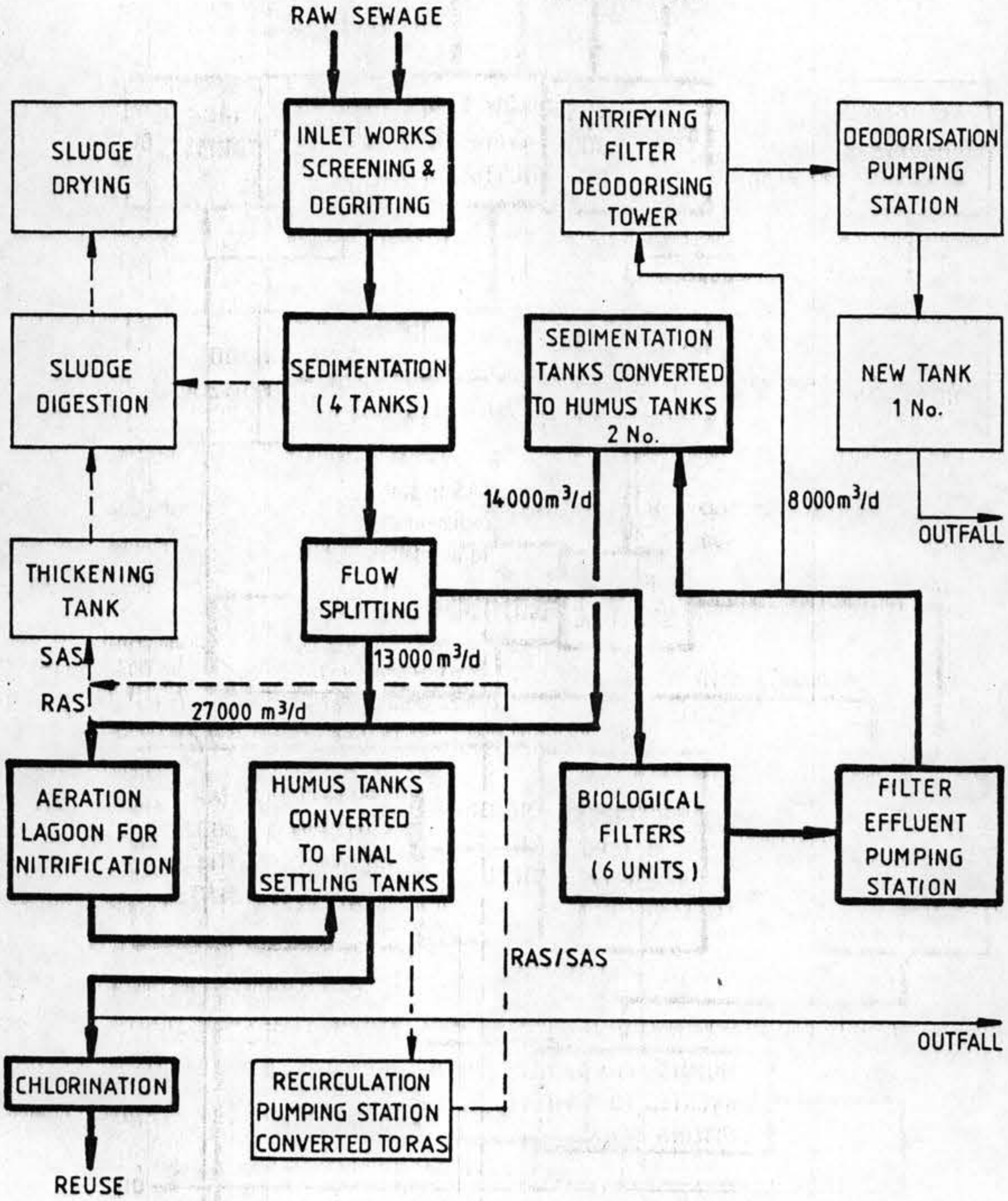



FIG. 6 OPTION 2 Conversion for 32 000 $m^3/d$ to 20:20 standard

OPTION 3 35 000 $m^3/d$ of which :-
27 000 $m^3/d$ to 20:20 standard
8 000 $m^3/d$ to 30:50 standard
Similar to Option 2 but further new intermediate settling tank added to converted existing sedimentation tanks.

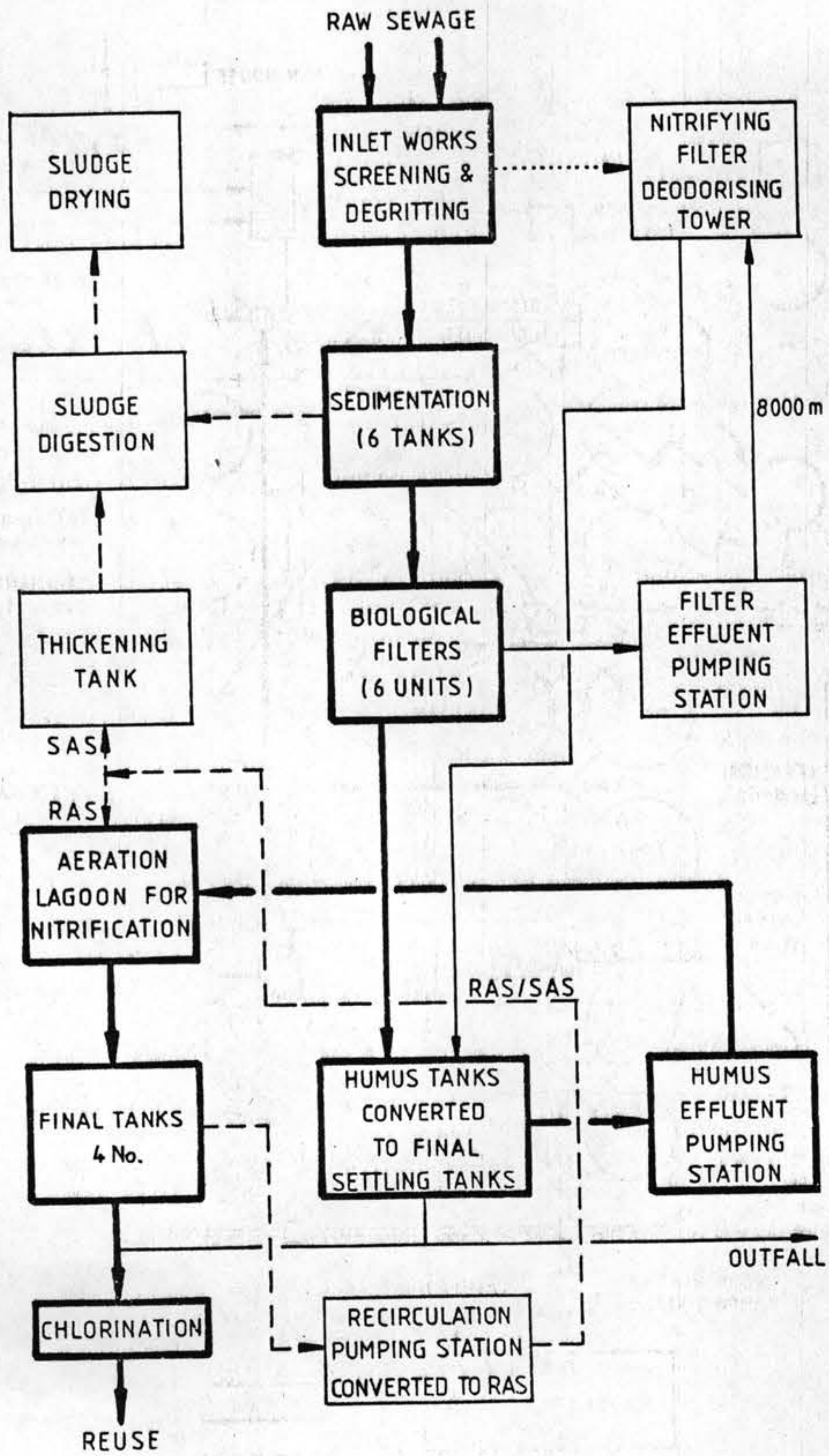



FIG. 7 OPTION 4 Conversion for 45 000 $m^3/d$ to 20:20 standard

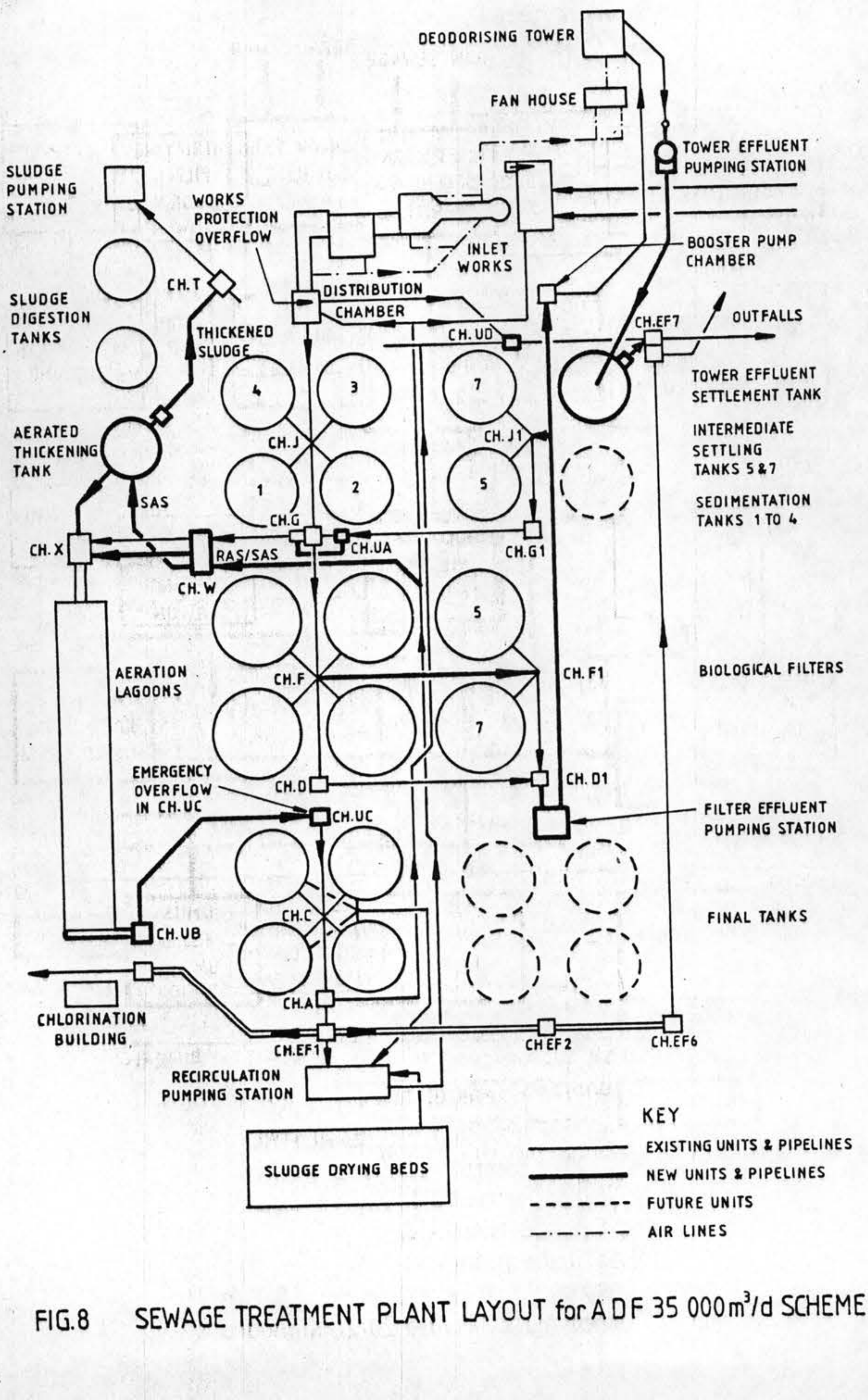
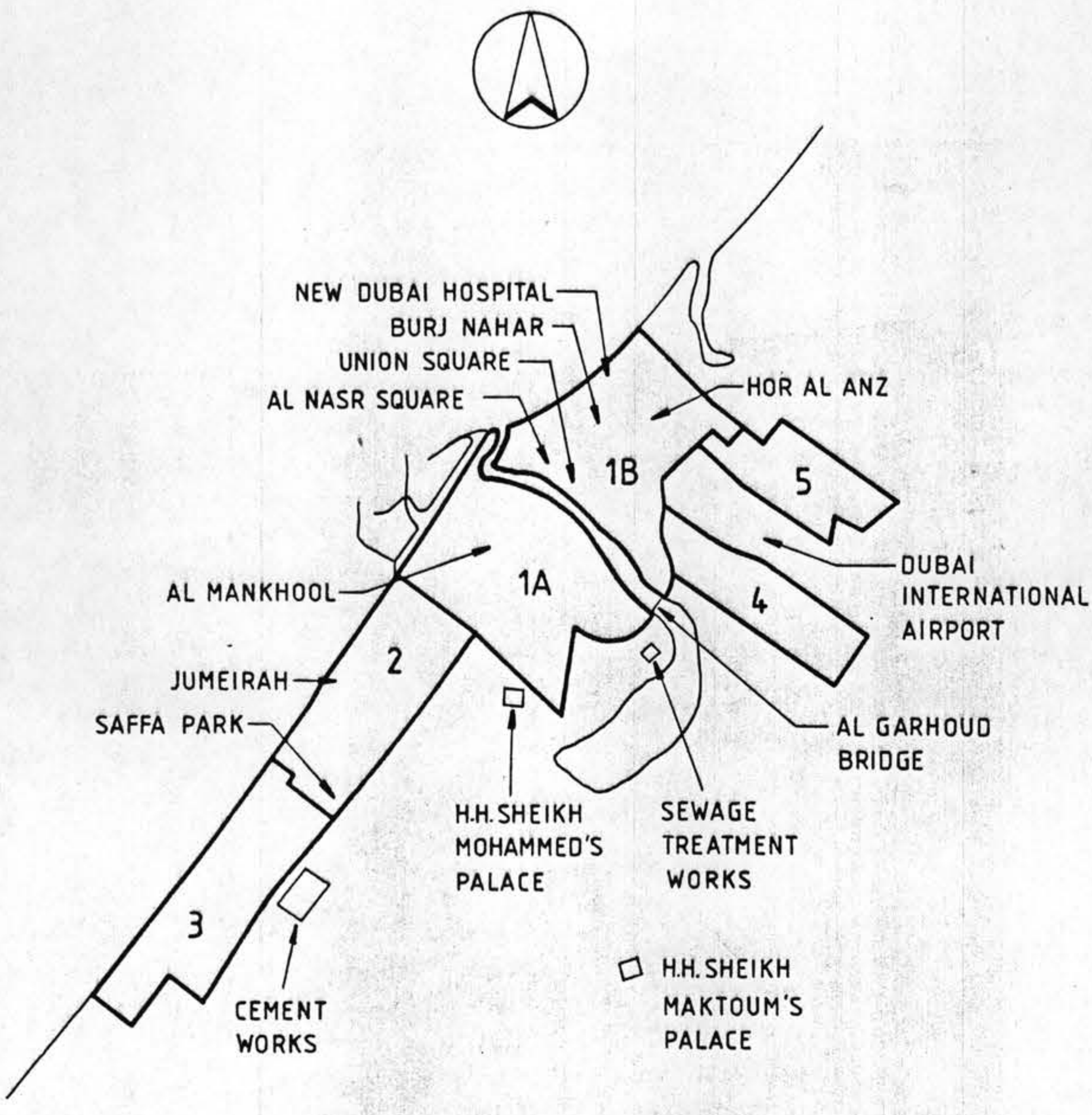



FIG.8 SEWAGE TREATMENT PLANT LAYOUT for ADF 35 000 $m^3/d$ SCHEME




DISTRIBUTION ZONES

1A DUBAI
1B DEIRA
2 JUMEIRAH
3 AL DIFA'A ROAD
4 AIRPORT
5 AL GHUSAIS

Scale 1:200 000

FIG. 9 EFFLUENT DISTRIBUTION ZONES

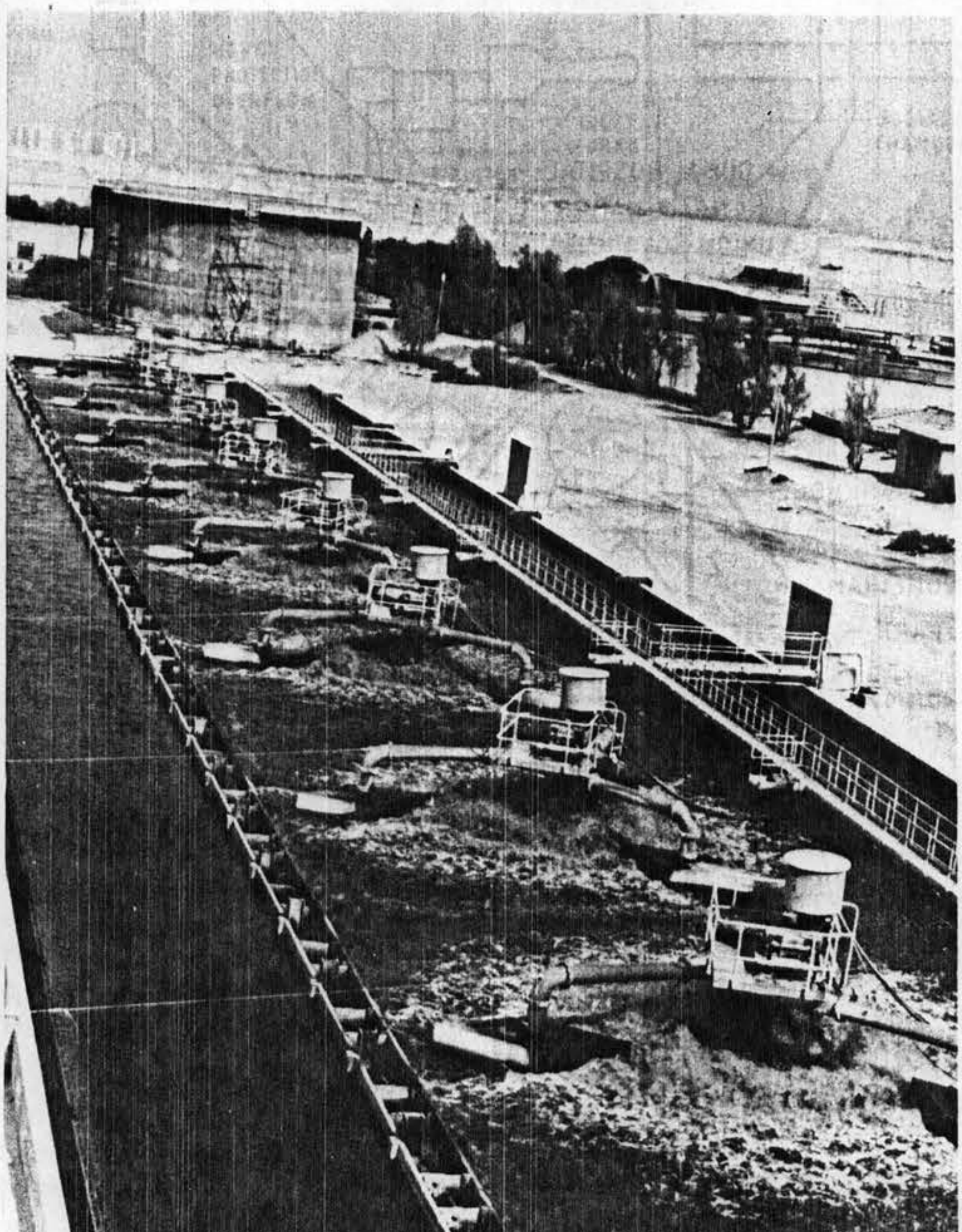


FIGURE 10 The aeration lagoon before modification

FIGURE 11 Road-reservation irrigation

FIGURE 12   Aerial view of sewage-treatment works