IMPROVEMENT AND MAINTENANCE OF EXISTING WATER SUPPLY SYSTEMS

Report on a Government of India/
WHO Seminar
Kanpur, Uttar Pradesh
31 Oct. - 4 Nov. 1977

World Health Organization
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PART I - THE REPORT

1 INTRODUCTION

The Seminar was held at Kanpur, Uttar Pradesh, from 31 October to 4 November 1977 and was attended by 20 participants (see Annex) as well as a number of observers from the Uttar Pradesh Jal Nigam, the Kanpur Water Works and the Indian Institute of Technology, Kanpur.

It was organized jointly by the World Health Organization and the Central Public Health and Environmental Engineering Organization (CPHEEO) of the Ministry of Works and Housing, in collaboration with the Uttar Pradesh Jal Nigam and the Indian Institute of Technology, Kanpur.

The objective of the Seminar was to review managerial procedures and technological intervention in the operation and maintenance of existing water supply systems with a view to securing improved efficiency in the operation of the systems. This includes the use of new technologies to increase the throughput of water treatment processes and operations.

2 PROCEEDINGS

2.1 Opening Session

A speech of welcome was given by Mr T.G. Sankaran, Deputy Adviser (PHEE), CPHEEO, Ministry of Works and Housing, New Delhi.

Mr B.R. Vohra, Chairman of the U.P. Jal Nigam, served as Chairman for the Opening Session. On behalf of U.P. Jal Nigam, he welcomed the participants to Uttar Pradesh and expressed the best wishes of his organization for a successful and informative Seminar.

An introduction to the Seminar was given by Mr M.A. Acheson, Regional Adviser, Environmental Health, WHO/SEARO. Mr Acheson stressed that the role of the water supply engineer is not only to design and construct optimal systems but to operate and maintain them in such a way that maximum efficiency is obtained.

The inaugural address was given by Dr A. Bhattacharya, Director of the Indian Institute of Technology, Kanpur. Dr Bhattacharya pointed out an increased public awareness of the contamination of water sources and this was leading to an increased public demand for improved water supplies.

The vote of thanks was given by Mr K.N. Dwivedi, Managing Director of the U.P. Jal Nigam.

The keynote address, given by Dr R.C. Ballance, WHO Sanitary Engineer, Geneva, was on the topic "General Problems of Water Supply in Hot Climates". This address pointed out that seasonal or perennial water shortages are not unique to hot climates; neither are problems of raw water quality and the hot climate is not in itself an overwhelming constraint to water supply development. Nearly all of the developing countries could be described as having hot climates and the water supply problems they faced were essentially financial and economic rather than technical. It was concluded that in such circumstances it was incumbent on water supply professionals to manage, operate and maintain the systems in the most efficient manner possible, to maximize throughputs and to minimize loss and wastage so that the largest possible number of people could benefit from a safe and reliable water supply.

2.2 Summary of the Proceedings

31 October 1977: Technical Session I

Subject: Improvement of Community Water Supply Systems

Chairman: Mr M.A. Acheson

Author of the Working Paper on the subject: Mr S.T. Khare

The Paper was presented by Mr S.K. Tasgaonkar.

Increasing urban population was a major reason for the identified need to augment existing water supply capacities and this is further aggravated by an increase in the requirements of individual users as well as by rapid industrial expansion. The aspects of water treatment that are the most amenable to technological changes to permit higher production are sedimentation and filtration. Tube settlers or plate settlers permit increased surface loading in sedimentation while new filtration technology, particular dual (or multi) media filtration, will allow filtration rates of up to four times the allowable rate for conventional rapid sand filters. These techniques, furthermore, often result in the production of an improved quality of finished water. An interesting technological breakthrough that had been developed in India was the use of crushed coconut shell as the top layer of a dual-media filter. Excellent results using this material had been obtained at the Nasik Water Treatment plant.

The author points out that when these technologies are applied it is almost always necessary to make appropriate changes to the connecting hydraulic structures. Pipes, channels,underdrains and pumps have to be provided with increased capacities in order to accommodate the increased flow rate through the treatment process(es). Similarly chemical treatment units may have to be expanded to match the new capacities.

Two examples were provided of projects in Maharashtra State. One of these, at Nasik, is expected to permit a tripling of treatment capacity at an estimated cost which is 30% below the estimated cost that would result if conventional technology was used. The second proposal is for Pune where the addition of chemical coagulation and the modernizing of the hydraulic characteristics in sedimentation tanks and rapid sand filters - both of 19th century construction - will permit an increase of throughput from 72 mld to 88 mld. Estimated savings over what would be the cost of constructing new facilities are expected to be in the order of 80 per cent.

31 October 1977: Technical Session II

Subject: Pre-treatment

Chairman: Mr M.A. Acheson

Author of the Working Paper on the subject: Dr G.D. Agrawal

Pre-treatment, it was pointed out, means all water treatment steps prior to filtration. Thus processes such as aeration, algae control, pre-chlorination, chemical coagulation, softening, flocculation and clarification are all included. Since softening is rare in India and clarification was to be the topic of the Technical Session III, they were not discussed.

Pre-treatment processes involve chemical reaction or phase transfer. Unfortunately most of the engineers, chemists and managers concerned with water works operation are unfamiliar with process kinetics and the method of their control. Consequently, insufficient consideration is given to the possible advantages that pre-treatment can convey to further treatment steps. Many of the treatment systems that are constructed consist of an assembly of "off-the-shelf" products of equipment manufacturers.

Professor Agrawal provided factual data of four case studies in which pre-treatment processes, singly or in combination, were adopted and resulted in improved performance and/or reduced operational cost. These were the Kanpur City Water Works, the J.K. Rayon Water Works, the Obra Colony Water Works and the Obra Thermal Plant 'A' Water Treatment Plant.

1 November 1977: Technical Session III

Subject: Sedimentation and Filtration

Chairman: Dr R.C. Ballance

Authors of the Working Papers on the subject: Dr G.D. Agrawal and
Mr Y.D. Misra

Professor Agrawal described the sedimentation process, preferring, however, to use the term clarification. The presentation was a logical follow-up to technical session II and case study data were presented on the same four water treatment plants.

Clariflocculators were criticized for their complexity and difficult operation which seems to lead to the use of excess coagulant. Horizontal flow clarifiers - either circular or rectangular-produced better results according to the case study data presented. Modifications to horizontal flow clarifiers were described which have resulted in improved hydraulic characteristics and, in the data so far available, improved clarification.

Mr Misra reported on filtration experiments conducted at the Kanpur Water Works in which a slow sand filter was operated at a higher than normal rate. The increase in filtration rate was 33% at the expense of an 8% shorter filter run. Experiments using alum-treated feed waters on slow sand filters did not result in any deleterious effects either in the "Schmutzdecke" or in the body of the filter.

The conversion of one rapid sand filter to dual media was reported to have resulted in a four-fold increase in flow rate with no deterioration of effluent quality. Savings relative to providing the new capacity by the construction of conventional rapid sand filters were in excess of 90%. An important aspect of this study was the identification of an indigenous coal that could be used in place of the much more expensive anthracite used elsewhere.

Brief reference was made to multi-media filtration, and the difficulties of obtaining materials with higher specific gravity than sand or lower specific gravity than coal were noted. The use of crushed coconut shell as the top layer in dual media systems and up-flow filtration was also mentioned. Reference was made to a trial of the Multiple Inlet-Multiple Outlet, or MIMO, filter but no operating data are yet available.

1 November 1977: Technical Session IV

Subject: <u>A Plan for Preventive Maintenance of Rural Piped Water Supply Systems</u>

Chairman: Dr R.C. Ballance

Author of the Working Paper on the subject: Mr M.A. Acheson

Preventive maintenance is agreed to be essential for the satisfactory working of a water supply system and can be instrumental in extending the useful life of a system. Practical planning and scheduling of preventive maintenance procedures can optimize the utilization of skilled staff and avoid all the disadvantages of an *ad hoc* approach.

The paper deals with the approach to planning, the steps involved and the types of forms which can be used in compiling an inventory of the technical components and identifying specific maintenance procedures necessary for individual pieces of equipment. After appropriate operational rules are compiled, it is then possible to devise a reporting arrangement and to prepare the detailed maintenance schedule. A set of six blank forms are presented which can serve as models for establishing a maintenance programme.

The procedure is illustrated with a case study in which there are 15 piped rural water supply systems containing a wide variety of technical components.

2 November 1977: Technical Session V

Subject: Disinfection

Chairman: Dr R.C. Ballance

Author of the Working Paper on the subject: Dr S.R. Shukla

The need for disinfection of public water supplies was reviewed together with a consideration of the various ways in which disinfection can be accomplished. Chlorination, the most common disinfecting technique, was described in detail especially with regard to gas chlorination which is in use in most municipal waterworks throughout the country.

2 November 1977: Technical Session VI

Subject: Distribution

Chairman: Mr T.S. Swamy

Author of the Working Paper on the subject: Dr R. Pitchai

The cost of piping in water supply reticulation was recognized as being a substantial component of the cost of a water supply system. This justifies the exercise of great care in distribution designed to ensure that optimum pipe sizes are chosen, thus maximizing the utility of the system while minimizing cost. The Hardy Cross method and the electrical analogue method of analyzing pipe net works were reviewed, prior to the presentation of a series of mathematical models which are suitable for manipulation with a digital computer. This latter method permits the insertion of various constraining factors into a sequential random search procedure which in successive iterations yields a series of terminal designs. The least costly of these is the "optimal design".

It was pointed out that a wide choice of piping materials is available and wise choice should be based on careful evaluation of the merits and demerits of each available type. This coupled with the powerful computational capacity of digital computers permits engineers to design functional systems with minimum public investment.

4 November 1977: Technical Session VII

Subject: Waste Surveys and Leak Detection

Chairman: Dr R. Pitchai

Author of the Working Paper on the subject: Dr G. Bachmann

Dr Bachmann outlined the generalized procedures followed in waste surveys wherein comparisons are made between the quantities of water delivered to a zone and the consumption as determined by service meter readings. This indicates problem areas and zones of excessive loss.

For the location of leaks it is necessary to use specialized electro-acoustic equipment which detects and amplifies the sound of leaking water and permits a high degree of accuracy in pin-pointing the location of a leak.

The overall procedure was described by means of a case study in Malta where an extensive leak detection and correction programme was conducted in 1971.

2.3 Closing Session

The closing session took the form of a "brains trust". Mr T.S. Swamy, Adviser (PHEE), CPHEEO, Ministry of Works and Housing, was the Chairman, and the panel consisted of Dr Agrawal, Dr Bachmann, Dr Ballance, Dr Pitchai and Mr Vohra.

A number of questions were asked and comments were given during a lively discussion period. Major points that were brought out during this and other discussion periods included:

(1) In hot climates, the demand pattern may be higher and the availability of water lower than in cold climates.

(2) Apart from "sewage farming", the only known instance of intentional waste water reuse in India is at Bombay where some 2 mgd of waste water is reused for industrial cooling purposes.

(3) In many rural water supply projects - particularly when a group of villages is interconnected with a common system - the major problems are in administration and maintenance.

(4) Technical problems are not global but local and it is therefore necessary to seek an appropriate local solution. The sequence to follow in problem solving is:

(a) Intensive local exploration,
(b) List the possible technical alternatives,
(c) Analyze the economic investment, operation, maintenance,
(d) Analyze the socio-cultural and legal ramification aspects.

(5) New models of chlorination equipment are being made from PVC and this appears to overcome corrosion problems. Also, the vacuum feed system seems to present fewer problems of corrosion than other types.

(6) Increasing the water throughput at a treatment plant by, for example, tube settlers and dual media filtration requires that more than a linearly proportional increase in the chlorine dose be applied to the treated water because contact time has been reduced. (Unless, of course, the chlorine contact chamber is enlarged.)

(7) There are many problems with water meters; they are unreliable, their discs break, plastic bodies are fragile and aluminum bodies corrode. These problems need to be brought to the attention of the

manufacturers. Nevertheless, there should be metered connexions on piped systems wherever possible so that the quantity of water used will be known and control of the system will be possible.

(8) Leakage detection and correction is not a "one shot" activity; it should be part of a continuing maintenance programme. Sets of equipment and trained crews are currently available in India, and there should be inter-city co-operation on the use of such facilities. Any water authority purchasing leakage detection equipment should ensure that the purchase contract includes an adequate training programme for using the equipment.

(9) It is essential with any water supply system to have an appropriate maintenance programme. This requires that there are proper institutional arrangements within the responsible water supply agency and the most important aspect is proper management.

(10) Improved information transfer between and among engineers and water supply officials of the States is highly desirable. One mechanism for accomplishing this might be the inter-state exchange of staff.

On behalf of the participants, Mr D.M. Mohan, thanked the organizers of the Seminar, which he described as being useful, informative and instructive. On behalf of the organizers, Mr Swamy expressed his appreciation and satisfaction of the way in which the proceedings and discussions had been conducted and spoke of the importance that CPHEEO and WHO attributed to meetings of this nature. He also expressed his appreciation to the Indian Institute of Technology which had made its facilities available for the meeting and to the Jal Nigam which had made all the local arrangements for the Seminar.

The Seminar then concluded.

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PART II - WORKING PAPERS

Working Paper No.1

GENERAL PROBLEMS OF WATER SUPPLY IN HOT CLIMATES

by

R.C. BALLANCE*

1 INTRODUCTION

There are very few references which chronicle the problems of water supply in hot climates. Those that do, invariably relate to site specific problems and do not discuss the problem in general. A recent text** on a somewhat broader subject "Water Wastes and Health in Hot Climates" seemed to promise, at least by its title, some useful leads. This proved to be a disappointment since only one relatively short chapter on the "Microbiological Criteria for Tropical Water Quality" seemed to refer to the problems of water supply which may be unique to hot climates.

Perhaps the subject is misleading. Perhaps it would be useful to review just what is meant by "hot climates" in order to detect whether there are some consistent characteristics which will help to narrow down the scope of discussion. As a first step in this process, some attention needs to be given to climate and water occurrence as well as to demographic conditions in the hot climates.

2 CLIMATE

The hot countries are included in a strip about 35 degrees north and south of the equator. This would include all of Africa, the Middle East, the Asian subcontinent and nearby islands, most of South America except for southern Chile and Argentina, as well as most of Australia, Central America and the southern United States. Within this 70 degree wide strip the temperature is variable with altitude - some of the high places, for example in the Himalayas, the Andes and the Hindu Kush have cool climates, at least seasonally, but all have climates that are seasonally hot or very hot. The greatest climatic variation in these countries is rainfall. Many of these hot countries are perpetually dry such as the North African Sahara and Sahelian zones, the Middle East, the desert areas of the Asian subcontinent, Australia and the southern United States. Many are seasonally dry but, on the other hand, have monsoons and seasonal rainy periods that produce the highest recorded annual rainfalls in the world. It is clear that there is no uniform climatic condition that prevails in the hot climates other than high temperature.

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*Division of Environmental Health, WHO, Geneva

**Evison, L.M. and James, A. Microbiological Criteria for Tropical Water Quality, *Water, Wastes and Health in Hot Climates*, 30-51, Wiley, New York (1977).

3 HYDROLOGY AND HYDROGEOLOGY

Both surface run-off and groundwater occurrence are related to rainfall. Obviously where rainfall is very low, run-off will be correspondingly low. Similarly, we do not normally expect to find large groundwater reserves in areas of low rainfall although groundwater may travel many kilometers from the point where an aquifer is charged to the point where it is exploited. Wide variations in hydrological and hydrogeological conditions occur in our seventy degree strip of hot countries. Some areas are completely devoid of run-off and yet, in the hot countries, one finds the world's largest rivers - the Amazon, the Orinoco, the Nile, the Congo and the Ganga to name but a few. Similarly, some areas contain massive underground water resources while in others drilling for hundreds of meters yields nothing but an expensive dry hole. It seems that in terms of exploitable water resources there is no absolute and uniform relationship with the hot climate.

4 DEMOGRAPHIC CONDITIONS

The hot climate area contains half, or more, of the world's habitable land mass. The Arctic and Antarctic are excluded because of their inability to support significant populations. Within the seventy degree wide hot strip one finds slightly less than one-third of the world's population. Average population densities are, therefore, low and national densities are much lower than that of the Netherlands, for example, which has a population density of 411 persons per square kilometer (1050 per square mile). Within the hot strip there are presently 74 cities with populations in excess of one million. By the year 2000 it is expected that this number will increase to 276, largely due to high rates of overall population growth and an increase in rural-urban migration in conjunction with rapid industrialization.

5 ECONOMIC CONDITIONS

Virtually all of the less developed countries are located within the seventy degree wide hot climate strip. GNP's per capita range from \$230 in Asia to \$1480 in the Middle East as compared with a range of from \$2400 to \$6600 for the countries in the temperate zone. The major exceptions to this generalized picture are Australia (GNP per capita - \$5330), the southern United States (GNP per capita - \$6700) and Israel (GNP per capita - \$3460) as well as some of the oil rich countries of the Middle East and North Africa. Similar exceptions can be detected in the temperate zones, especially in North East Asia.

Along with depressed economic conditions, one also finds generally low levels of social services. In particular the average educational achievement level is low, and the range of health services - including water supply and sanitation - is limited, especially in rural areas.

6 WATER QUANTITY

This brief review indicates that the hot climate *per se* does not prevent the development of an efficient and reliable water supply system. We have the evidence of Israel, the southern United States and South Africa as well as a substantial number of very good systems in developing countries which all confirm that a hot climate is not an overwhelming constraint.

The climatic feature of inadequate rainfall or rainfall that is non-uniform over the year is a universal problem that is, no doubt, accentuated in the hot climates. But this is a problem that can be diminished provided that good management, engineering skills and adequate finance can be brought to bear on the problem. It is interesting to note, for example, the developments in water resources management that are taking place in the industrialized hot countries referred to above. In these countries water is so scarce that it must be reused and serious consideration is now being given to the renovation of sewage for eventual supplementation of the drinking water supply. Pilot projects in all three of these countries have proven the technical feasibility of waste water reuse. A large scale project in Orange County, California is now treating sewage effluent to produce a water which meets drinking water standards prior to injecting it into a series of aquifers from where it will eventually be withdrawn for municipal use.

Concurrent with renovation and reuse, the water supply authorities apply their managerial and engineering skills to reduce wastage and unnecessary usage to the minimum possible. Publicity campaigns are carried out to encourage users to save water. Leakage detection and correction is a continuing component of the programmes of maintenance. This is only natural since the reduction of waste is the least expensive alternative to the $1.00 per thousand US gallons cost of converting sewage into drinking water.

It is probably right to conclude that the major water supply problem in hot climates is the periodic shortage of water. And if this is correct then the major solution to the problem is to ensure optimum use of the available water by minimizing wastage. This, in due course, can be followed by allocating the highest quality resource to the highest category of use - drinking water supply - and to use lower quality waters and treated waste waters for lower categories of use.

7 WATER QUALITY

The high intensity of sunlight which is a prevailing condition in hot climates causes the temperature of surface waters in rivers, lakes and reservoirs to be consistently high - often up to 35°C. These high temperatures and the sunlight which causes them favour the production of large standing crops of algae. The well-known resulting problems to water supply are filter clogging in the treatment process and the production of unpleasant tastes and odours in the water.

In areas where the monsoon or rainy season brings annual heavy rainfalls, the resulting rapid run-off and flooding causes the rivers to carry heavy loads of silt. Additionally, the flood waters invariably have a different chemical composition to the "normal" water and these chemical changes - especially pH and alkalinity - will affect both chemical coagulation and chlorination processes during water treatment.

Frequently the bacteriological quality of surface waters is much below the desirable standards for raw waters. Where rivers, lakes and ponds are used for bathing, laundry, animal watering and so forth, it is inevitable that contamination will occur. Standard bacteriological testing of hot climate waters for indicator organisms of the coliform group may be misleading because coliform numbers can increase significantly in polluted waters warmer than 20°C. The concentration of the exclusively faecal *Eschericia coli* tends to be distorted by the presence of related organisms as well as by the regrowth phenomenon general to coliforms. There is no evidence, however to indicate that pathogens can grow and increase their numbers in natural waters.

8 CONSTRUCTION OF FACILITIES

Countries with hot climates have a distinct advantage over countries with temperate and seasonally cold climates insofar as construction is concerned. While certain work must be scheduled for times other than monsoon or heavy flood, much construction work can be carried out throughout the year with only minor protection against the elements. Cold climates on the other hand, present serious problems of excavation in frozen ground and in concrete placement during at least four months of the year. Furthermore, where freezing climates occur seasonally it is essential that all pipes and appurtenances in the reticulation be placed at a minimum depth of 1.5 meters in order to be below the depth of frost penetration. Since reticulation is the most expensive component of a water supply system it is obvious that the countries in hot climates have a distinct advantage.

The hot climate countries in the developing world have, on the other hand, serious construction disadvantages. These relate to economic conditions rather than to the climate and would include such things as inadequate communications, transportation and reserves of spare parts and materials.

9 OPERATION AND MAINTENANCE

A few general problems of operation and maintenance have been referred to under the section on water quality. To expand on these is difficult since most operation and maintenance problems are location specific rather than general. Some general observations are in order, however, in relation to most of the developing countries as opposed to industrialized countries.

Access to information is restricted. The engineer, designer or manager does not have sufficient exposure to technical information from any source. Salesmen and representatives of the manufacturers of equipment and supplies are not frequent visitors and thus do not provide the information contained in their catalogues and promotional literature. There are few, or no, technical journals available on a continuing basis and those that are available tend to describe esoteric research and development activities totally inappropriate to the pressing needs of developing countries. There are few, or no, professional associations and little opportunity for water supply professionals to meet and exchange knowledge and experience.

Manpower resources are weak. Some of the developing countries have an adequate cadre of educated professionals - many do not. Very few have a sufficient number of sub-professional technicians and it is impossible to operate and maintain systems without this category of personnel. Insufficient importance is given to this level of work - training is inadequate, pay scales are low and competent people are not attracted to this essential work. Given such constraints it is virtually impossible to develop the pride of workmanship and *esprit de corps* so necessary for the trouble-free operation of a water supply system.

Finances are inadequate. This is probably the most overwhelming and intractable problem for developing countries. Materials, supplies and equipment used in water supply systems are not much - if at all - cheaper in developing countries than they are in industrialized countries and there is very little prospect of reducing costs. It is impractical to expect people whose incomes are abysmally low to pay the full costs of constructing, operating and maintaining a water supply system. No simple solution to this problem is evident. It can only be fully overcome when there is a substantial improvement in national economies. Any form of charity can only be considered a stop-gap measure.

10 CONCLUSION

Problems of water supply in hot countries *do* exist but they can be overcome by the application of proper technologies. This is evidenced by the existence of efficient and adequate systems in industrialized hot countries.

The most difficult problems are related to the broader problem of economic under-development and these will not be easy of solution. In the meantime it is incumbent on water supply professionals to manage, operate and maintain the systems in the most efficient manner possible, to maximize throughputs and to minimize loss and wastage so that the largest possible number of people can benefit from a safe and reliable water supply.

Working Paper No.2

IMPROVEMENTS TO COMMUNITY WATER SUPPLY SYSTEMS

by

S.T. KHARE*

1 INTRODUCTION

In Maharashtra, most of the Municipal towns have now been covered with piped drinking water supply schemes. During the past one or two decades in most of the States in India there exists a similar condition. Many places have also been provided with conventional water treatment plants. However, due to the increase in the population of the urban areas and the increasing demands of water for domestic as well as for industrial purposes, there is a pressing demand for increased water supply. The concerned authorities are facing this problem at many of these towns. At many water works, the treatment plants are over-loaded to cope with the additional demands. However, there is a limit for such over-loading of the plants and the quality of the filtered water has, therefore, deteriorated at some of these plants. Due to the increase in demand of water all over the world, augmentation of the existing water treatment plants has become important and is likely to be faced at most of the existing municipal water treatment plants in the near future.

Due to the various demands of social and industrial developmental activities in the developing countries, it will be difficult to find adequate financial resources to provide such additional capacities of water treatment plants. However, with the new technological developments in the recent years in the field of water treatment, it is now possible to augment the existing conventional water treatment plant capacities by 100 to 300 per cent in a short period and also at a considerably reduced cost as compared to the construction of additional new facilities at such plants. The quality of treated water after application of such new technology can also be improved in addition to the increase in the quantity of supply.

The purpose of this paper is to discuss the new techniques of augmentation of the existing conventional water treatment plants and to give the important techniques of full-scale applications at two major water treatment plants which are under process of augmentation in the Environmental Engineering Organization of Maharashtra State.

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2 NEW TECHNIQUES OF AUGMENTATION

The augmentation of the existing treatment processes includes the expansion of both the pre-treatment and filtration capacities of the plants. The existing conventional treatment plants have been designed by specialized firms in this field and even though the basic design criteria are the same, there are many changes in the design of civil structures. Thus, the modification of the existing plants will require considerable ingenuity on the part of designer for the expansion of some of the old conventional plants.

2.1 Augmentation of Pre-treatment Works

The expansion of the pre-treatment works by new techniques includes the installation of tube settlers or plate settlers in the existing settling basins. This will also include consideration for the expansion of the mixing and flocculation arrangements and adequate sludge removal capacity as may be found necessary. The hydraulic capacity of the inlet and outlet pipes or channels has also to be considered along with the sizes of operating gates and valves.

2.2 Augmentation of Filter Units

In the case of conventional rapid sand filter units, if the existing beds are designed specially for effective back wash arrangements, they can be converted without much difficulty into high rate dual or multimedia filter beds so as to increase the plant capacity from 200 to 300 per cent. If the existing filter beds are not designed for effective back washing with expansion in the filter media from 10 to 50 per cent of the depth of the media, it will be necessary to make structural changes in the existing filter beds and some times even in the under-drain system.

The inlet and outlet pipe sizes may have to be increased to get the desired increased output. It may not be possible in some cases to change the sizes of influent pipe or channel without interrupting the plant operation and in such cases it may be necessary to install a separate pipe line outside the plant and to connect it to the existing influent pipe at several locations near the major take-off points. The backwash supply and waste lines need not generally be altered since the backwash rates are not significantly increased by conversion to dual or multimedia filter beds. In the case of filter rate controllers, it may become desirable to go for declining type rate controlling arrangements by suitable modifications, when a battery of filter beds is to be tackled. In case of individual units, modifications for higher flow rate controllers or manually controlled arrangements may have to be made.

2.3 Other Aspects

Among the other important considerations are the expansion in the raw and pure water pumping capacities, providing additional or increased size of mains and increasing the clear water pump capacity to cope with the increased demand; with all the modifications as suggested above it may be possible to increase the existing plant capacities by two to three times, without the need for any structural additions such as new clariflocculator and filter beds.

3 AUGMENTATION OF NASIK ROAD WATER WORKS

3.1 History

The Government Water Works at Nasik Road was originally constructed in the year 1922, with a small capacity of 2.7 mld. There were three m.s. circular tanks to give plain sedimentation followed by 3 m.s. circular slow sand filters. This plant was abandoned as it was of small capacity and further it was not possible to treat the turbid water from the Darna river source effectively for want of proper chemical mixing and flocculation arrangements. In order to meet the increased demand a new conventional treatment plant of 9 mld. capacity was constructed in 1955. This plant includes one clariflocculator and three numbers of rapid sand filter beds. Due to the further increase in the demand of water supply, the Government has now sanctioned a scheme for augmentation of the present plant capacity from 9 mld. to 27 mld. New pumping machinery and 500 mm dia additional raw water main has been provided to meet the increased demand. Distribution system has also been modified to give the increased supply. It has been decided to augment the existing works by application of the new technology as explained in detail hereafter.

3.2 Pre-treatment

Mixing: As the present mixing and flocculation arrangements are not adequate, a new chemical house with a wash water tank of adequate capacity is proposed to be constructed for washing the filter beds. In place of conventional mechanical chemical mixing units, only flash mixing arrangements with weir action is provided in the chemical house itself. Chemical solution tanks with compressed air mixing arrangements are proposed on the first floor while chemical storage is proposed at the ground floor.

Flocculator and Tube Settler: There are three m.s. circular tanks of 12.6 m dia and 4.85 m in height, with plain bottom. These tanks were fabricated in the year 1922, and were designed as plain sedimentation tanks as mentioned earlier. Even though these m.s. tanks were abandoned and not in use for a long time, their structural condition is satisfactory. In the new proposals for conversion of the existing plant, it is proposed to convert these m.s. tanks suitably to augment the additional capacity. In this conversion proposal, half of the central tank will be converted into mechanical flocculator while one tank will be converted into tube settler to give the additional capacity of 18 mld. The flocculator will have four compartments and it is designed on the basis of tapered flocculation system by keeping variable speed rotating baffles in these compartments. The flow in the flocculator will be in the up and downward direction in these compartments to give improved flocculation as per new concepts.

Details of Tube Settler: Existing verticle baffles in the m.s. tanks will be removed and masonary hoppers will be constructed at the bottom of the tank. A layer of tube settler consisting 50 x 50 mm square size rigid PVC tubes, 60 cm in height will be provided with its top one meter below the water level in the tank. The modules of the PVC tubes will be fabricated by fixing the tubes at 60° angle in the opposite directions and will be installed on suitable supporting structure of m.s. angles and channels. The inlet pipes from the flocculator will introduce the flocculated water through the perforated PVC pipes radially just above the top of hopper level. The water after passing through these tubes will be collected through the PVC perforated pipes provided in the radial direction at the top water level in the tank to collect the water in the collection well. With these outlet and inlet arrangements the loading through the tubes will be adjusted uniformly throughout the surface area of the tank. The surface loading on the tube settler considering 75% effective open area of tubes will be about 140 lit/sqm/min. This is within acceptable range as per on plant data, and also found satisfactory on the basis of pilot plant study carried out at the site. The action in the tube settler is to accelerate the flocculation which gives quick settlement in this process. This is now well accepted theory and detailed theoretical aspects are, therefore, not explained in this paper. Thus, by conversion of the existing discarded m.s. tanks into flocculator and tube settler, it will be possible to get the required additional capacity of 18 mld. for the proposed augmentation purpose. The work is proposed to be undertaken during 1977-1978.

3.3 Conversion of Filter Beds

For increasing the filtration capacity the existing three numbers of conventional rapid sand filters as constructed in 1955 are proposed to be converted into dual media filter beds. There are three filter beds with surface area of 41.30 sqm. each. The present rate of filtration is 3000 lit/sqm hr. Considering the proposed total augmentation capacity of 27 mld. the rate of filtration from these three filter beds after conversion into dual media filter beds will be about 8800 lit/sqm/hr.

Details of Conversion: In the existing conventional filter beds the back washing arrangements are not effective and hence structural modifications in the filter beds have been proposed. The existing filter beds are provided with false bottom arrangements and the sand media is directly placed on the same. In the proposed modifications of these filter beds, in one of these beds, the false bottom arrangements have been removed and the bed has been converted into a standard conventional filter box with central manifold and side PVC 75 mm dia. perforated laterals at the bottom. In the existing beds there was one central wash water gutter with top about 10 cm above the top of the sand bed. In order to give effective back wash with 30 to 50% media expansion, the central gutter has been raised by 60 cms and side wash water gutters are provided to collect the wash water. The new filter media consists of supporting gravel bed 45 cm thick, and dual media consisting fine sand bed 50 cm thick (e.s. 0.5 mm & u.c. 1.5) over which a top layer of crushed coconut shell media 40 cm thick is provided. The size of this new media at the top is between 1 to 2 mm sieve opening and has a specific gravity of 1.4

in the wet condition. The first filter bed was converted in 1974 and it gives the increased filtration rate of 9000 lit/sqm/Hr. with the effluent turbidity well below one unit, during the filter run. It is pointed out that this new filter media has been used for the first time in the world for the augmentation of the existing filter beds. The new media which was first tried at Ramtek filter in 1973 shows very satisfactory results so far.

The remaining two filter beds are presently under modifications. However, in these beds the existing false bottom arrangement is being adapted with the wash water gutter arrangement modified similar to the filter bed No.1. In place of supporting gravel bed only a 10 cm. thick layer of fine gravel, of size 3 to 5 mm, has been provided below the fine sand, over the false bottom slab. The actual observations of these two beds are yet to be taken for the increased rate of filtration.

Inlet and Outlet Arrangements: The inlet and the outlet pipes have been increased and new and simple head loss arrangements are provided for each bed. The existing automatic rate controlling arrangements were not functioning properly. It is proposed to provide declining type of rate controlling system for these filter beds.

3.4 Cost Aspects

The probable cost of construction for a new conventional filter plant of 18 mld. capacity may be about Rs 20 lakhs at the present day cost. The cost of the proposed modifications for flocculator, tube settler and conversion of the filter beds will be about Rs 7 lakhs. In addition to this the cost of the new wash water tank of increased capacity of 300 000 lit. along with the chemical house will be about Rs 4 lakhs. The existing wash water tank capacity is 90 000 lit. which is found to be inadequate for giving an effective back wash, and hence a new wash water tank is proposed in the modification proposals.

It can thus be seen that the modifications can be done within 2/3rd of the cost of a new conventional plant for additional capacity. If the wash water tank would have been of adequate capacity, and the filter beds of conventional design for effective back washing, the cost of the modifications would have been only 1/3rd of the cost of a new conventional plant of additional capacity of 18 mld. Thus, the saving in the cost of modifications of the existing conventional plant of standard design can be between 1/3rd and 2/3rd as compared to the provision of additional new plant for augmentation. The period of modifications can also be reduced to a considerable extent. The actual plant performance data after full modifications will be published in the near future.

4 AUGUMENTATION OF POONA CANTONMENT WATER WORKS

4.1 Present Works

The existing capacity of this water works is 72 mld. and is falling short of the growing demand of the town. The source of water supply is from Mutha right bank canal. The old plain sedimentation tanks with four numbers of slow sand filters were first constructed in the year 1890. Further, to cope with the turbid water during rainy season and increased demand, 10 numbers of rapid sand filters of size 5.5 x 3.0 m were constructed in 1915 and four numbers of rapid sand filters of size 9.75 x 6.47 m were constructed in 1930. Some chemical mixing arrangements with baffled channel were made before the old sedimentation tanks. Thus, the capacity of the water works was 31.5 mld. upto 1960. Further augmentation was done by providing new rapid sand filters of 13.5 mld. and 29 mld. in the years 1960 and 1968 respectively to give the present plant capacity of 72 mld. To meet the additional demand, it has been decided to increase the capacity of the old filter beds from 31.5 mld. to about 47.5 mld. along with the improvement in the pre-treatment as described below.

4.2 Improvements in Pre-treatment

As there are no proper mixing and flocculation arrangements in the existing plant it has been decided to provide new mixing and flocculation arrangements ahead of the existing settling tank. There are five numbers of big rectangular settling tanks and two of small sizes with the total volumetric capacity of 30 mega litres, with the present average flow of about 3 lakh litres per hour, the average detention period in these settling tanks is about 10 hours. Thus there is scope for 100% expansion of the present plant capacity. At present, the five numbers of settling tanks act in series. It is now proposed to provide separate inlet and outlet R.C.C. channels in all these settling tanks. Thus, all the settling tanks will act in parallel. The present manual sludge cleaning practice will be followed in future also by segregating individual tanks. The work of this expansion is likely to be undertaken for execution in the next year.

4.3 Augmentation of Filtration Capacity

In the proposed modifications it has been decided to augment the capacity of the old 14 numbers of filter beds from 31.5 mld. to 47.5 mld. The old filter beds have been designed for the rate of filtration of 3300 lit/sqm/Hr. From the actual observations, it is seen that the under-drainage system is not effective to give adequate back wash to bring the filter beds to clean conditions. It is, therefore, proposed to improve the back wash system by providing additional perforations in the A.C. pipe underdrain laterals in 4 numbers of big filter beds and to provide new P.V.C. 50 mm dia perforated underdrain laterals where the existing M.S. underdrain laterals have been heavily corroded and have inadequate perforations. The existing M.S. perforated air wash distribution pipes will be replaced by new pipes, where necessary. The supporting gravel media will be properly laid and new fine sand of e.s. 0.5 mm and u.c. 1.5 will be provided for 60 cms over the gravel bed. For effective collection of wash water the depths of the existing gutters will be increased and there will be a clear gap of 0.6 m between the top of the gutters and the top of the sand media to allow expansion of 30 to 50% during back wash.

These modifications will be able to give an increased rate of filtration of about 5000 lit/sqm/Hr., so as to give the increased output of 16 mld. Two existing old filter beds have already been modified, which have shown the possibility of more than 50% increase in the filtration rate, so as to augment the existing capacity of the old filter beds from 31.5 mld. to 47.5 mld. The existing rate controlling arrangements for all these old filter beds have been out of order for a long time and the rate of filtration is controlled by operating the valves manually which is not satisfactory for a large number of filter beds. It is, therefore, proposed to provide declining type rate controlling system for all these beds, so as to get increased output from the old filter beds with better effluent quality.

4.4 Future Scope

In the present proposals, it has not been proposed to modify the new rapid sand filter beds of 40.5 mld. capacity. As these beds have been provided with effective back washing arrangements, it will be possible to convert these filter beds into dual media filter beds to give 100% increase in the output from these new filter beds constructed after 1960. With the possibility of 100% increase of pre-treatment capacity as stated earlier, it will be possible to increase the total plant capacity by 100% in the future. As there is little scope for providing additional new treatment plant for want of adequate space in the existing compound, the expansion of the plant by 100% as stated above is likely to be undertaken in stages in the near future considering the ever increasing demand of water in the town.

4.5 Cost Aspects

For augmentation of additional capacity of 16 mld. of supply the cost of a new conventional plant may be about Rs 16 lakhs including additional wash water tank. As per modifications suggested above, the cost of modification of the old filter beds will be about 2.50 lakhs only. The cost of modifications of the pre-treatment works is not considered in this cost as it will give the expansion capacity of 100% as discussed above. The work of conversion of the old filter beds is under execution, while the work of modifications of pre-treatment works is to be undertaken shortly. The particular advantage is that the plant capacity can be augmented immediately after modifications of the old filter beds and before the modifications to the pre-treatment facilities.

5 CONCLUSIONS

From both the cases of augmentation of existing water treatment plants, it will be seen that there can be a considerable saving in the capital cost for such expansion by application of new and suitable technology. In addition to this, the augmentation work can be done in a short period and also in stages as per the availability of the funds and actual requirements of increased demands. It will also be possible to improve the quality of the effluent water by application of these new technologies in addition to the increased output which seems to be an important problem before the municipal water treatment plants in the developing countries.

Working Paper No.3

SOME ASPECTS OF PRE-TREATMENT

by

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1 INTRODUCTION

The term pre-treatment in water supply practice is conventionally taken to include all water treatment steps prior to filtration. It could thus include all or some of the following unit operations: (i) aeration, (ii) algae control measures, (iii) pre-chlorination, (iv) chemical coagulation, (v) chemical softening and other chemical treatment, (vi) mechanical flocculation, and (vii) clarification. Of these chemical softening and/or other special chemical treatment is rarely necessary or practicable in water supplies in India, while clarification is to be discussed in the next session, as such these types of treatment would not be considered in this presentation.

A look at the unit processes included for consideration in this paper, as listed above, would reveal that all of them involve some chemical reactions or phase transfers. The rates of such processes are hence going to very significantly depend on the chemical composition of the raw water, environmental conditions like temperature, dosages of chemicals and the operating conditions like extent of mixing, turbulence, etc. It is thus obvious that the performance of these unit operations, like any other chemical unit operations, is less a function of design of plant and more a function of proper operational control that gives full regard to the raw water characteristics and environmental conditions. The role of plant design for these processes is primarily to provide utmost flexibility and facility to the operational staff to be able to quickly assess any changes in raw water quality and environmental conditions and make suitable adjustments in the controls to maintain the desirable rates of transformation.

Unfortunately the professionals involved in water treatment are either engineers for whom head losses, power consumption, torque, stresses and stability are more familiar than rates of phase-transfer and chemical interactions, or chemists who also do not have any background of kinetics of the processes involved and methods of engineering control over such rates. Often it is seen that even top management in water-supply organizations give similar importance to design and installation of equipment for 'pre-treatment' of water as one would give in a building, and consider operation to be little more than occasionally

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adjusting the alum dose; much as one would change the ventilator opening with change in season. This paper will attempt to expose the fallacy in such a notion and to show that with an intelligent operation an existing plant can successfully handle problems and pollutional loads that were never even thought of at the time of design and also affect significant economies in cost of treatment. Another main theme of this paper is to indicate the difficulties and poor performance often resulting from adopting conventional designs and equipment from specialized vendors which severely limit the flexibility and freedom available to the operator.

As very specifically directed by the WHO, theory and textbook material will be avoided except when necessary to explain and interpret some first-hand field experience of the author with (i) Kanpur City Water Works - raw water sources Ganges Canal and R. Ganga Upstream of Kanpur, (ii) J.K. Rayon Water Works - raw water source R. Ganga downstream of Kanpur, (iii) Obra Colony Water Works - raw water source percolation wells in the bed of R. Rihand and (iv) Obra Thermal Plant 'A' Water Treatment Plant - raw water source Obra Reservoir on R. Rihand. The presentation would concentrate on the process, the problems faced and control measures and not on site details of the plants.

2 AERATION

Aeration as a unit process in water treatment is not common in India and the author has not seen a single water treatment plant in this country having aerators. As such aerators could have been skipped in this presentation. But the author strongly feels that inclusion of aerators in the flow-sheet could handle many pre-treatment problems in an effective and economic manner. Aeration effectively contributes to (i) raising pH of waters having significant CO_2 in dissolved form, (ii) precipitating iron, manganese and heavy metals, (iii) oxidising organic matter, (iv) producing a denser floc with smaller dose of chemicals especially when iron, manganese or organic matter are present and (v) tackling problems of taste, odour and colour. Of the above the author gives more importance to (iv). Many Indian raw waters contain significant amounts of organic matter that makes the floc produced fragile, light and watery. Even short periods of aeration could help convert this matter to being more amenable to flocculation. Such a situation was observed at the J.K. Rayon Water Works where the raw water had a COD as high as 80 to 90 mg/l and colour 18-30 units primarily due to the tannery wastes joining the river upstream of the intake. Flocculation was poor and the filtered water still had significant colour (10-15 units) and COD (40-50 mg/l). There being no provision for aeration, the problem was tackled by introducing prechlorination at the intake as described under the chlorination. However, laboratory experiments showed that aeration for 5 seconds could give as good results as the 12 mg/l optimal dose of chlorine at much cheaper cost.

The same was the feeling of the author at Obra where absence of aerators makes the removal of the 0.5 to 1.5 mg/l iron present in the water difficult and costly. The author hence has included aerators as essential pre-treatment units in the new water treatment plant to be built at Obra. It is strongly recommended that aerators should be invariably included in the flow sheet whenever the raw water is likely to contain significantly amounts of Fe as in the eastern and central region of the country or significant amounts of organic matter as is the case, or is going to be the case, in all rivers passing through densely populated regions.

Calculations for Obra gave the overall cost of aeration at Rs 0.02 per thousand gallons (Rs 0.0045 per thousand litres) in a total cost of treatment of Rs 0.18 per thousand gallons (Rs 0.04 per thousand litres) and a total cost of Rs 0.83 per thousand gallons (Rs 0.185 per thousand litres) as delivered to the consumer. It is likely to result in savings in the cost of chemical larger than the expenditure on aeration. A bonus will be the improved aesthetic looks of the plant and a greater freshness in the taste of the water.

3 PRECHLORINATION

Prechlorination is the second best course available for tackling problems due to taste, odour, colour, organic matter, iron and manganese. Besides it is effective in situations having excessive bacterial contamination, nematodes and other worms and algae problems. Since prechlorination does not require much equipment, and facilities and knowledge required are always available at the water works, it is often taken to be a panacea for most problems and can in fact be so for many with a proper understanding and control.

The experience of the author is on the effectiveness of chlorine in the following situations:

(1) Removal of organic matter and colour at the J.K. Rayon Plant.

(2) Control on the growth of slimy deposits in the system and in the viscose plant of J.K. Rayon.

(3) Control on algae problems at Kanpur Water Works.

(4) Control of algae growth in clarifiers at Obra.

The points that emerged from this experience, which are well supported by theory and research findings but are frequently forgotten by field staff are (i) the role of chlorine in prechlorination is *not* the *complete* oxidation or destruction of the reduced matter, whether mineral or organic, or the killing of algae, bacteria, nematodes and other worms but only to make them amenable to flocculation by *partial oxidation* of the reduced matter and "*knocking off*" of the organisms and (ii) the action of chlorine is a time dependent process and even the above partial action needs a significant contact time. Thus to be most effective there should be significant time gap between prechlorination and addition of coagulants, as shown by the following experience.

It was observed at J.K. Rayon plant that prechlorination just ahead of the plant even with doses as high as 40 mg/l gave very little removal of colour and organic matter. Thinking that it was because of the small time of contact available it was suggested that prechlorination be done at the intake so that approximately 20 minutes time of contact becomes available in the rising main. J.K. Rayon were not enthusiastic about this because of two reasons, (i) the difficulty of introducing chlorine under pressure at the intake and control over dosing, and (ii) the fear of corrosion in the rising main. To convince them a laboratory study was carried out the results of which are shown in Table 1 along with the raw water characteristics.

The data shows a prechlorination dose of 12 mg/l is optimal and that introduction of chlorine 20 mins before the alum addition point is much more effective than adding it just ahead of alum. This convinced the client and arrangement was made to introduce the chlorine by a pressure pump in the rising main at the intake. This immediately brought the colour and COD in the plant effluent to 2 APHA units and 6.5 mg/l which were found acceptable. Prechlorination as above also solved partly the problem of slimy growth in all pipe lines and plant including clear water storage tanks and the viscose plant units. Laboratory investigations had shown the growths to be primarily due to the fungus *A. Niger*. For complete control of the problem, post-chlorination residuals had also to be increased to 1.5 mg/l.

At Kanpur water works prechlorination was thought of when addition of copper sulphate solution at the Bhairon Ghat intake was found to be ineffective in tackling the problem caused by excessive algae in the river water during summers resulting in clogging of both slow and rapid sand filters. At one time the problem had become so serious that the author saw slow sand filters overflowing, making the whole area a pool of water and the water works staff was hard put to manage maintaining the supply to the city. Prechlorination at Bhairon Ghat intake with a dose as small as 1 - 1.5 mg/l was able to convert the algae to be amenable to flocculation so that even if they pass on to the filter, it is only as part of a dense floc, so that filter clogging does not result.

Due to the difficulty of maintaining chlorine supplies at another site and of control over the dosing etc., the Water Works in 1977 resorted to prechlorination at the Water Works just ahead of the channel leading to the coagulation - clarifier system. The results in this case were not as disappointing as at the J.K. Rayon plant as shown by the removals of algae in Table 2. This may at least in part be due to the fact that the water does get a long time of contact in the various channels and the so-called pre-setting tank before alum is added and coagulation starts.

At Obra raw water turbidities in summers were so low (less than 10 NTU) that significant algal growth started taking place in the clarifiers. Introducing prechlorination by 0.5 mg/l added just before alum feed stopped such growth, though it did not have any other noticeable beneficial effect.

An intelligent handling of prechlorination can thus tackle a large variety of water works problems.

4 CHEMICAL COAGULATION

The chemical used in Indian practice is generally ferric alum (or alumina ferric) with or without lime dosing. At Kanpur - both at the City Water Works and at the J.K. Rayon plant, lime dosing is not practiced since the pH is always higher than 7.5 and there is no Fe or Mn problem. At Obra because of the lower pH of raw water and the presence of Fe, lime dosing is resorted to. At Obra lime dose is regulated manually to maintain the pH in the clarifier in the range 7.4 to 7.6 as tested every hour by a pH paper. Lime dosages vary from nil to 80 mg/l. The operative staff has a tendency to overdose lime since this gives the clarifier a cleaner appearance although the filtrate turbidity may not improve.

4.1 Fixing and Controlling Coagulant Dose

Alum (which is almost the universal water coagulant used) dose is determined by jar tests. But the fact that neither the J.K. Rayon plant nor the Obra plants owned a Jar-test apparatus - several other large water works also are known to be doing without a good jar test apparatus in working order - and even Kanpur Water Works till recently had only a hand-operated jar test apparatus shows that actual field dose is currently settled by only the experience of the operating staff. The result in most cases is severe overdosing, resulting in waste of the chemical (which incidentally is in short supply) and also poorer performance. At the water treatment plant of Obra Thermal Power Station 110 mg/l of alum were being added in place of the 25-30 mg/l needed. This meant not only a wastage of alum but also a higher dose of lime to restore pH, thus increasing TDS in water which later had to be removed in demineralisers for the boiler feed preparation. Determination of a proper dose of alum with a fluctuating quality of water and later administering it in a controlled way is probably the most crucial problem of water works in India. Part of it originates from inadequate provision for measuring and controlling the concentration of alum in the feed solution, the rate of feeding the solution and the rate of raw water inflow and part of it is due to an inadequately trained and non-committed staff. At the four plants under reference, a few hints to the few committed members among the staff were able to significantly improve the situation. The theme was to discard the instruments and automatic controls which were not working any way, simplify the system and provide encouragements and incentives for commitment and efficiency.

4.2 Alum Feeding Systems

The best way to feed alum is to prepare alum feed solution in an agitator tank and to feed regulated quantities through suitable piping, and all modern water treatment plants have such devices. Yet there are still many water treatment plants in the country where there is no such arrangement and alum has to be fed through solid 12-16 kg cakes. Either the whole cake as one piece is hung in the inlet channel in a suitable crib or it is broken into pieces and thrown in the channel. Even where solution making and dosing systems are provided, they occasionally go out of order due to high corrosivity of alum solution and once again one is forced to some sort of solid feeding. The main problems in solid feeding observed at Kanpur Water Works were (a) the alum cake may not dissolve rapidly enough, and (b) difficulty of maintaining a uniform dose due to intermittent manual feeding of cakes. The first problem was solved by introducing V shaped cribs that hold alum cakes at points of high velocity of flow just ahead of the flash mixing hydraulic jump. For a large plant like Kanpur and with a committed staff, well trained and supervised, experience has shown that maintenance of uniform dosing was no worse than that achieved in the solution feeding systems at Obra.

4.3 Flash Mixing

Mechanical flash mixers existed at the J.K. Rayon plant as also the Obra Thermal Water Treatment Plant but were almost never functioning due to one reason or another as may probably be their fate elsewhere in the country. It was also observed that most water treatment plant staff do not fully appreciate the importance of flash mixing and are not bothered if the flash mixers are not operating. That this is a false notion is adequately proved by the Kanpur Water Works data given in Table 3. It may be seen that flash mixing of alum caused by properly designed hydraulic jumps introduced in 1977 resulted in 30% or greater savings in alum consumed. The total monthly saving of Tonnes for Kanpur is impressive by any standards.

Although hydraulic jumps consume more energy (head loss at Kanpur is 2-3 feet) and are less efficient as flash mixing device compared to mechanical agitators, in the opinion of the author they are much more desirable as they can neither go out of order nor be switched off and the value of good flash mixing in obtaining good floc can hardly be overstressed.

5 MECHANICAL FLOCCULATION

The author's experience at the four plants as also his observation at several others has made him entirely against the flocculators and clariflocculators designed and supplied in India by Dorr-Oliver, Patterson, Bird & Co. and other firms. They are too prone to go out of order. The flocculator mechanism has been out of order for years at Kanpur and the one at the Obra Thermal Water Treatment Plant also breaks down pretty often. In such circumstances the growth of floc has to depend entirely on the velocity gradients generated by normal flow in channels and tanks. And miraculous it may seem but it is a fact that growth of floc is not too bad even without the mechanical flocculators running, as may be seen at Kanpur Water Works and also the two plants at Obra. This may be in part the reason of why there is not much effort to put them right once they go out of order.

Our studies on two mechanical flocculators in the field - the J.K. Rayon plant treating the high organic material Ganga water and the Obra Thermal Plant treating the high iron content Rihand water show that the flocculator speed and hence the 'G' values provided are too large for these raw waters. The result is that floc formed with the flocculator running is finer and harder to settle than when the flocculator is not running. Working back from power consumption, flocculator RPM, plant dimensions and flow, the J.K. Rayon plant was found to be working at an effective value of G=109 Sec^{-1}, and at G.t of 1.8×10^5. Similarly the Obra plant was having a G of 84 Sec^{-1} and G.t. of 1.15×10^5. Laboratory studies indicated that the best value of G for both these waters was in the range of 20 to 25 Sec^{-1} corresponding to a G.t of 2.5 to 4.0×10^4 being adequate and satisfactory. When it was recommended by us to reduce the flocculator speed from the existing 2 RPM to 1 RPM in case of J.K. Rayon and 1.2 RPM in case of Obra plant it was found that there is no way to modify these speeds without major changes in either the motor or transmission. Short trials with a lower speed motor confirmed the desirability of the speed reduction. Unfortunately this motor was of too large a capacity and could not be installed permanently on the flocculators.

On the basis of above experience the author favours adopting lower rotorspeed viz lower value of G=20 to 25 Sec^{-1} for Indian waters. In any case, the transmission should provide for easy change of speeds with a minimum of three speeds (say those giving G values of 50, 35 and 25) being possible. It may also be worthwhile to consider baffled channel flocculators which would not be liable to failure and would be more efficient than the present no flocculator conditions.

6 CONCLUSIONS

With commitment, initiative, ingeniousness and understanding on the part of the operating staff and with desirably some support and backing from specialized professionals, even poorly designed existing plants can be made to take additional loads and give highly satisfactory performance with very cheap and simple modifications. The maximum problems and loss of efficiency is often due to the more complicated and costly (though designed on conventional basis adopted from foreign practice and having not much relation to Indian conditions) pieces of equipment.

7 ACKNOWLEDGEMENTS

All the facts and figures quoted are from the four water treatment plants named earlier and the author has to express his heartiest gratitude to the authorities and the staff of the Kanpur Water Works, Messrs J.K. Rayon,Kanpur, and the Obra Thermal Power Station, UPSEB, Obra for providing the author the opportunity to gain the above valuable experience. The author also has to acknowledge the help and co-operation from a number of post-graduate students who participated in the studies.

Table 1 - J.K. Rayon Kanpur, Water Treatment Plant

A Raw Water Characteristics (Period April-June)

pH 7.7 to 8.0

Turbidity 14.5 to 17.5 NTU

COD

Soluble 25 to 32 mg/l

Total 80 - 90 mg/l

Colour

Soluble 1 - 2 APHA Units

Total 18 - 30 APHA Units

B Results of Prechlorination Studies

Parameter	Unit	Raw Water	Flocculated & Settled Water						
			No Pre-chlorination	Clorine added just before alum			Chlorine added 20 mins before alum		
				6 mg/l	12 mg/l	18 mg/l	6 mg/l	12 mg/l	18 mg/l
Turbidity	NTU	16	7	9	7	9	6	3	4
COD	mg/l	83	31	38	18	33	9	3	7
Colour	APHA Units	22	8	9	5	8	3	0	0

Alum dose kept constant at 40 mg/l which was found to be optimal. Flash mixed for 30 secs., flocculated for 15 mins. at 40 RPM, Settled for 30 mins.

Table 2 - Effect of Prechlorination on Different Algal Species

	Algal Species	Cell count per ml in the sample from			
		Raw Ganga Water	Water Works	Clarified Water	Filtered Water
1.	Anabaena	2 220	800	160	-
2.	Anabaenopsis	1 160	1 080	80	-
3.	Merimopedia	840	800	240	-
4.	Oocysts	960	640	360	-
5.	Ankistrodesmus	840	560	200	-
6.	Nitzoschia	480	490	480	-
7.	Navicula	1 480	1 320	640	-
8.	Others	1 250	830	410	-

Prechlorination Dose: 0.4 mg/l at Intake

1.0 mg/l ahead of clarifier

Contact time: 25 Mins : Intake to Water Works

180 Mins : Prechlorination to alum dosing

155 Mins : In the clarifier

20 Mins : After clarifier to filtration

Table 3 - Saving in Alum by Introduction of Flash Mixing
(Kanpur Water Works - Ganga Water)

Period	Turbidities, FTU			Alum Dose mg/1	Total Water Treated M G	Total Alum consumed Tonnes
	Raw Water	Pre-settled water	Clarified Water			
Before Introducing Flash Mixing						
July 1976	756	565	18	48	904	196
August 1976	813	615	20	62	1 028	292
September 1976	499	476	20	57	773	199
				Total	2 705	687
After Introducing Flash Mixing						
July 1977	671	570	18	30	831	112
August 1977	726	618	18	31	1 016	138
September 1977	524	472	18	41	823	154
				Total	2 670	404

Saving in Alum over 3 months = 283 Tonnes

Per cent Saving in Alum = 40%

Working Paper No. 4

CLARIFICATION AND SETTLING

by

G.D. AGRAWAL, Ph.D.*

1 INTRODUCTION

Settling, also termed clarification, in water treatment practice is intended not only to remove the larger fraction of the turbidity to reduce the loads on and the clogging of the filters that follow but also to remove a large bulk of organic matter, iron, bacteria, viruses and many other impurities in themselves not settleable but removed in an adsorbed, entrained or enmeshed form in the floc. Some authors described such removal process as a sort of sweeping of the water by the settling floc. The modern trend is to have lower and lower clarified water turbidities to produce a final effluent that is not only sparkling clear but is almost entirely free of bacteria, virus and other undesirable impurities. Filter influent at Erie, Penna has thus an average turbidity of 3.5 FTU while that at San Francisco is always lower than 2 FTU. Compared to this the settled water turbidities of 15 to 30 FTU in most of the Indian Water Works presents a sad picture. Improvements in chemical treatment, flocculation and clarification to cause maximum removals of all types of impurities desired to be removed in this part of the treatment is going to be the crux of obtaining a better quality of water at economical costs.

As in the case of the earlier presentation on "Pre-treatment", this paper is going to be primarily based on the experience of the author on 4 water treatment plants: (i) Kanpur City Water Works, (ii) J.K. Rayons,Kanpur Water Treatment Plant, (iii) Obra Colony Water Treatment Plant, and (iv) Obra Thermal Power Station 'A' Water Treatment Plant.

There are three main types of settling basins in use in India: (i) circular clariflocculators, (ii) circular clarifiers, and (iii) rectangular settling basins. Fortunately the four plants, with which the author was associated, cover all these types. Both the plants at Obra have clariflocculators, the J.K. Rayon Plant has a circular clarifier of a rather strange design and the Kanpur Water Works operates with mainly rectangular settling basins. To the best knowledge of the author there are no tube-settler basins in India working at the moment. The author was associated with a project to instal one at Kanpur but the project is in cold storage. Recently we have designed tube-settlers for the new plant at Obra and expect these to be in operation by June 1978.

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The experience of the author on the various types of clarifiers is briefly presented below.

2 CLARIFLOCCULATORS

This is the type of clarifier chosen for most of the new installations today. In the author's opinion this is less due to better performance or any economy in either cost or space and more due to its elegant and compact looks and aggressive salesmanship by the many firms involved in this business. The performance of the clariflocculators at the two plants at Obra under typical operational conditions are given in Table 1. It can be seen that with the Obra Lake water the clariflocculator could not produce turbidities below 17 NTU even with near optimal operation with the author and the executive engineer incharge personally supervising the operation for 20 hours. The major problem was 'floc-boiling' in some segments of the flocculator. Thus two diagonally opposite segments would suddenly start appearing brownish near the outer walls and the brownishness would slowly spread to the total segment and then to the nearby segments. Complete flushing of the sludge or continuous drainage of large quantities of a rather watery sludge were seen to suppress the 'floc-boiling', the former for a few hours and the latter for as long as the bottom drainage was maintained at a high enough rate. The latter course involved significant amounts (up to 25%) of the water and essentially meant a lower effective loading rate.

Looking at it theoretically 'floc-boiling' in an upflow clarifier such as the clariflocculator is bound to take place if the sludge blanket filter effect is not operative. Theoretically a clarifier with a surface loading rate of 30m/day is expected to remove 100% of particles with settling velocities equal to or greater than 30m/day and any small particles in the ratio of their settling velocity/30m per day loading rate. In an upflow clarifier however the upward flow velocity of 30m/day will carry all particles of settling velocities even slightly less than the upward 30m/day velocity upwards to end in the effluent. These could only be retained if the sludge blanket was acting as an effective filter. It appears that the nature and volume of floc produced in these waters was not suitable to create an effective sludge blanket. This may also be one major reason of a tendency on the part of the plant staff to overdose alum. In most clariflocculators there is a provision for sludge recirculation and this may in some cases solve the problem of floc-boiling. But this is bound to make operation and control even more complex. Hence the author is of the opinion that upflow clarifier should be considered inferior to simple horizontal-flow rectangular clarifiers for water treatment plants.

As for the performance of clariflocculators with percolation well water, it is equally unsatisfactory for similar reasons. But no floc-boiling of a visible nature occurred. It may also be seen that overloading the system by over four times did not cause any significant deterioration in quality.

3 HORIZONTAL FLOW CIRCULAR CLARIFIERS

These do not seem to be very popular and no new plants are planning to employ them to the best knowledge of the author, though in his opinion they are far superior to the clariflocculators being aggressively sold and readily bought. There is little if any, upward velocity and hence no question of 'floc-boiling' problems.

The author had opportunity to be associated with two such clarifiers one giving good overall performance and the other rather poor. The one giving good performance is at TURRA near Renukoot, District Mirzapur designed to handle 20 000 gph of Rihand Lake water at a design surface loading of 650 gpd/ft^2 and a designed detention time of 130 minutes. The performance when operating at a flow of 22 000 gph, viz., a surface loading of 715 gpd/ft^2 and detention time of around two hours is given in Table 2. Although this plant was also a case of overdosing of alum (60 mg/l against the Jar-test optimal dose of 18 mg/l), the performance was in no way unsatisfactory and in any case far better than clariflocculators with best efforts and a similar water.

The other horizontal flow circular clarifier was really a queer case. In place of flow being from centre to the periphery it was essentially from one part of the periphery to another. The 24 m (80 feet) diameter clarifier is designed to handle 3 MGD of flocculated Ganga water at the J.K. Rayon Plant, with a design detention time of 2 hours. The outlet weir is only 41 feet in length covering only about 16% of the periphery and exceeding the usually permissible weir loadings. The inlet covers even less length, viz., 11 feet and can hardly be expected to distribute the flow evenly. Further the inlet and outlet are not even located diagonally opposite to each other but at an angle of 130°. All this results in intense short circuiting in the clarifier. Tracer studies gave the flow through time for the clarifier basin to be 43 mins against the designed detention time of 120 minutes. The performance hence was bound to be poor as is shown by data in Table 3. Since the plant cannot be shut down for long periods it is not easy to modify the system. The minimum changes proposed were to modify the inlet so that it will distribute the inflow over 60 feet of the periphery and to increase the outlet weir length also to 60 feet. Besides distributing the flow more evenly and reducing turbulence at the outlet this would make the angle between the inlet and outlet centre lines about 170°. It is expected that the change would significantly reduce short-circuiting and increase the flow-through time to at least 90 mins thus improving performance of the clarifier. The modifications, expected to cost around Rs 20 000 are yet to be implemented.

In the overall opinion of the author, radial flow circular clarifiers are positively far better from the performance and control points of view than clariflocculators but the only advantage they may claim over rectangular basins is in being able to continuously remove the sludge. Since the walling costs in case of a circular tank are going to be larger, this much extra investment on a sludge removal system for a rectangular basin would bring them on par from this point of view.

4 RECTANGULAR CLARIFIERS

In this category only the performance data and the results of the studies on the large rectangular clarifier basins of Kanpur Water Works would be presented.

Statistical analysis of raw water and clarified water turbidity data for 6 non-monsoon months - October 1975 to March 1976 gave the following values:

Raw Water Turbidities - Range 22 FTU to 118 FTU; Mean 57 FTU; Mode 40 FTU; Standard Deviation 7.2 FTU.

Clarified Water Turbidities - Range 16 FTU to 32 FTU; Mean 23 FTU; Mode 23 FTU; Standard Deviation 3.7 FTU.

The clarifier basins as operated during this period had a surface loading rate of 13 m/day or 270 gpd/ft^2 and should have theoretically removed 85% of the turbidity against the 43% observed. The clarified water in practice contained significant amounts of settleable flocs. The size distribution of floc in clarified water as observed under microscope ranged from 1 to 30 microns, with a mean floc size of 10 microns and a standard deviation of 5 microns of the turbidity present in the clarified water as much as 45-50% settled out within 10 minutes detention in a jar. The above data showed that the performance was very poor and probably there was significant short circuiting in the basin. As such it was decided to carry detailed field and simulated model studies on one of the clarifier Basins at the water works - called Settling Tank No.2 by the water works staff.

The Settling Tank studied has a length of 112.30 meters and a width of 78.6 meters at the top with a depth of 5.87 meters at the deepest point. During use over the past several decades it has silted up heavily and unevenly. A hydrographic survey of the bed contours gave the average depth of the tank to be 1.6 meters with a deep channel diagonally across the tank from the major inlet to the major outlet and two large plateaus near the two opposite corners. The water rushed through this deep channel from the inlet to the outlet with most part of the tank being dead pockets. While theoretically the capacity available in the tank even after such heavy silting is adequate to provide a detention time of 184 minutes for a flow 114MLD(25MGD) and 152 minutes even at the time of overloading to 140MLD(31MGD), tracer studies gave a mean flow through time of only 23.5 minutes with the modal and the median flow through times being only 10 minutes and 17.5 minutes respectively, for a flow of 114 MLD. Confirming the physical observation that most of the water rushes through the major outlet at the left, termed outlet No.2, tracer studies showed the outflow to be 15%, 75%, 7% and 3% respectively through outlet No.1, 2, 3 and 4, showing an extremely uneven distribution.

Applying Rehbun's method (Rehbun, M. and Argaman, Y. 1965, Evaluation of Hydraulic Efficiency of Sedimentation Basins, J. of Sanitary Engg. Division, ASCE, SAS, P37) of analysis to the tracer flow-through curve, it was observed that only 34.5% of the tank volume is effectively available for sedimentation, the remaining 65.5% being mere dead space; of the 34.5% available only one eighth (or 4.3% of the total volume) can be assumed to behave as a plug flow system and the remaining acts as completely mixed. These calculations as also the low Median to Mean and Modal to Mean flow through time ratios (0.744 and 0.425) respectively were proof of intensive short-circuiting.

A distorted hydraulic model to a scale 1/50 in the horizontal dimensions and 1/10 in the vertical direction was constructed and trials conducted to determine the best modifications that would reduce short circuiting and improve performance with the minimum changes in terms of effort and cost. After a number of trials it was concluded that baffles would not help much. The optimal arrangement was found to be one in which there would be two symmetrically placed slotted inlets each 5.0 meters in length, located at 17.5 meters from the long walls and two outlet weirs, one 7.6 meters long at the right corner and the other 5.0 meters long at 15.0 meters from the left wall. Results on the model indicated that such a modification would increase the ratio of the median to mean flow through time to 0.98 and that of modal to mean flow through time to 0.58, which would be significant improvements. Besides when analysed by Rehbun's method the modified system showed almost the entire tank volume to be effective for settling with dead spaces reduced to less than 1%. The equivalent plug flow fraction in the modified system came to 26.4% which is rather close to Rehbun's recommendation of keeping it at 30% in a good practical settling basin. Thus simple modifications costing less than Rs 5000 were expected to very significantly improve the performance.

The modifications have been carried out recently in the field and hopefully some data on their performance will be available for presentation at the Seminar.

5 CONCLUSIONS

With proper inlet and outlet arrangements to distribute the flow evenly and minimize short circuiting, simple horizontal flow circular or rectangular clarifier basins are better performers than any more complicated arrangements. Simple modification can help in improving and maintaining the performance of clarifiers.

6 ACKNOWLEDGEMENTS

The author expresses his heartiest gratitude to the authorities and staff of the Kanpur Water Works, Messrs J.K. Rayon, Kanpur and Obra Thermal and Hydel Power Administration, UPSEB, Obra for providing the opportunity for the above valuable experience. The author has also to acknowledge the help and co-operation of PG students who participated in the various studies.

Table 1 - Performance of Clariflocculators at Obra

Details	Plant No.1			Plant No.2		
Raw Water Source	Obra Reservoir on R.Rihand			Percolation Wells		
Raw Water - pH	7.3	7.4	6.7	7.4	7.4	6.6
Iron, mg/l	1.1	1.0	0.6	0.6	0.6	0.3
Turbidity, NTU	142	144	236	13	12	11
Chemical Dose - Lime, mg/l	80	-	75	-	-	40
Alum, mg/l	122	30	101	15	15	15
Flocculator Part -						
Det time, mins.	32	28	29	36	8.5	8.5
Clarifier Part -						
Det. time, mins.	125	115	116	140	31	31
Surface Loading gpm/sf.	805	881	872	770	3 200	3 200
Settled Water - pH	7.5	7.3	7.2	7.4	7.4	7.3
Iron, mg/l	0.7	0.9	0.5	0.5	0.2	0.05
Turbidity, NTU	26	17	22	7	8	8

Table 2 - Performance of Circular Clarifier at Turra

	Parameter	Raw Water	Clarified Water
(1)	pH	7.6	7.4
(2)	Turbidity, NTU	148	12
(3)	Iron, mg/l	1.2	0.5
(4)	Coliform MPN/100 ml	134	16

Table 3 - Performance of Circular Clarifier at J.K. Rayon

	Parameter	Raw Water	Clarified Water
(1)	pH	7.7	7.6
(2)	Turbidity, FTU	17	11
(3)	Colour, APHA Units	26	21
(4)	COD, mg/l	88	57

Working Paper No.5

FIELD STUDIES ON FILTRATION

by

Y.D. MISRA*

1 INTRODUCTION

The subject of filtration is so vast that it is difficult to take it up in a comprehensive way. I understand that the aim of this Seminar is to converse amongst ourselves our experience on filtration with special emphasis on improvement and up-grading of existing filters.

For better appreciation of problems, the subject needs to be divided under various heads:

2 SLOW SAND FILTERS

In my opinion, it is unfortunate that this old and efficient process of treatment has not received proper attention by those engaged on research and experimentation on filtration. The process has been forgotten to quite some extent under the impression of being old fashioned and slow. It is, however, encouraging to know that some studies have been done in India and in greater details by WHO in some foreign countries. Practically, in all the urban towns of the world trend for augmentation of existing treatment plants had been construction of rapid sand filters. However, some studies conducted at Kanpur on slow sand filters have created interest in them. In developing countries, where technical skill, instrumentation and maintenance facilities are scarce, construction of slow sand filters should be given thought.

3 RATE OF FILTRATION

Though the rates of slow sand filtration differ particularly on the depth of the sand but in many places in India, the normal rate is 50 gpd/sft. The main objection to these filters had been their very slow rate. There had been a fixed notion also that while higher rates of filtration were possible in rapid sand filters, nothing significant could be done in case of slow filters.

In this aspect some studies were conducted at Kanpur during the year 1970-1971 for a period of little over one year. Two slow sand filters were selected for study, filter No.7 and 8 each measuring 200' x 100'. One was run at the conventional rate while higher yield was taken from the other. Complete details of study were published in a paper "Over-

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loading of Slow Sand Filters" presented in Symposium on Water Treatment, Distribution and Management in February 1972 in Nagpur. The abstract of the study showed the following results:

Filter No.	Scheduled rate of filtration gpd/sft.	Actual average rate of filtration gpd/sft.	No. of filter runs	Average number of days of filter run	Quality of water filtered gpd/sft. filter run	% of water treated gpd/sft. filter run
7	65	51.08	10	32.4	1655	100
8	80	67.95	13	29.8	2025	122.3

The above study really indicates 33% increase in rate of filtration decreased filter run by 8% and increased water treated per filter run by 22%. The important inference that could be had from here is 22% extra quantity of water obtained per filter run did not cause a corresponding decrease in length of filter run. Against 22% increase of water, filter run decreased by 8% only. This is a positive advantage towards over-loading of slow sand filters. Detailed study on water samples taken on both the filter Nos. 7 and 8 did not show any loss of efficiency of bacterial or turbidity removal.

These slow filters have 25.4 cms. graded gravel below 15 cms. layer of coarse sand of 0.5 mm size overlaid by 60 to 90 cms. of fine sand of effective size 0.2 mm and uniformity co-efficient 2.5

Still higher rates of filtration were tried on Pilot Plant Filters (91.5 cm x 18.3 cm) constructed especially for experimentation. This study yielded following results:

S.No.	Actual rate of filtration gal/sq.ft/day	Length of run days	Quantity of water filtered per filter run gal/sq.ft	No. of days lost in scraping and recoupment per run
1.	104.7	40	4 238	3.5
2.	197.8	15	29 850	3.5
3.	296.6	10	29 850	3.5

These pilot scale filters had 90 cms. sand of effective size 0.35 mm and uniformity co-efficient 2.7. 25 cms. of graded gravel was provided below sand.

4 USE OF ALUM COAGULATED WATER

These high rates of filtration could be possible with the use of alum coagulated water as influent to filters. Huisman (1) and Ives (2) caution against use of alum coagulated water as influents to slow sand filters. According to Ives, alum flocs seal sand surface with quick rise in head loss and occlude the schmutzdecke. These in turn deteriorate biological purification of water through slow filters. Ives in another paper (3) points that under certain circumstances, due to low pH being caused consequent to alum treatment, aluminium hydroxide could precipitate in lower layers of filters which is undesirable from the point of view of cleaning the filter.

Studies conducted at Kanpur clearly show that these dangers did not occur at least in the circumstances prevailing in Kanpur. An exhaustive paper by I.C. Agrawal and G.D. Agrawal (4) on operating slow sand filters with alum coagulated water in Kanpur does not indicate excess concentration of precipitated alumina in deeper sand depths. Observations made by the authors have been given in Table 1 enclosed. Another paper by I.C. Agrawal ; G.D. Agrawal and Y.D. Misra (5) showed that in certain circumstances, even in minimum sized towns in India, provision of slow sand filters instead of rapid sand filters could be economical.

5 RAPID SAND FILTERS

There is hardly any Journal in Public Health Engineering which does not have frequent articles on rapid sand filters. Conventional rapid sand filters have been constructed and operated at an optimum rate of 1.5 gpm/sq.ft. But, there had been a race in USA and elsewhere to operate filters at very high rates of upto 6 to 7 gpm/sq.ft. In my opinion, this race must stop and a compromise must be made to ensure quality. One 1.8 mgd filter bed at Kanpur Water Works was converted into a dual media filter designed to yield 7.2 mgd. An increase of 5.4 mgd. capacity being added at a nominal expenditure of Rs 15 000/-. It would have otherwise cost about Rs 7 lacs.

The present trend is to design and operate filters at the rate of 2 gpm/sq.ft. Higher filter rates in routine are not common in India. The work done on conversion of rapid sand filter into dual media filters at Kanpur led to award of Ph.D. degree to Shri S.V. Ranaday, a lecturer in Engineering College, Sangli, Maharashtra. Some work has also been done in NEERI, Nagpur, on column Filters which give encouraging results regarding possible use of bituminous coal in place of anthracite. An important part of the study at Kanpur was to find out varieties of bituminous coal which may prove a substitute to anthracite whose procurement involves foreign exchange. For selecting the right bituminous coal it has to be seen that technical requirement are fulfilled and the broad parameters are the same as that of anthracite. For this purpose, coals of various collieries were tested. Finally, it was found that selected 'A' quality coal of Raniganj coal fields could provide a good substitute to anthracite. Its parameters compared as under:

S.No.	Parameters	Anthracite	Selected 'A' Coal
1.	Ash and Moisture content	18%	17.5%
2.	Specific Gravity	1.3 to 1.5	1.4
3.	Hardness Mho's scale	3.75	3.4
4.	Solubility in dilute HCL.	Less than 2%	1.4%
5.	Chloroform extract	Less than 0.5%	0.6%
6.	Durability	Less than 3% by weight	2.8
7.	Phenol Leaching	Not detectable	Not detectable

Results obtained in this study were of interest of Public Health Engineers in USA as well. In USA, too, anthracite is expensive. In case bituminous coals in USA also could substitute anthracite, it will be an achievement there also to get a cheaper second media filtration in two layer filters.

I am of the opinion that upgrading of rapid sand filters should be done in a more vigorous way by converting many more conventional filters into dual media filters at various places in the country. Even if we are able to achieve 3 gpm/sq.ft rate by minor changes in the under-drainage system, the work will give important data and at many places, construction of new filters could probably be dispensed with. The procedure of conversion is simple, Head loss occuring at Inlet, each place of the under-drainage system and exist system should be calculated and pipes and fittings which obstructed in increased rate of filtration should be changed/enlarged. For deciding specifications of coal and sand, it is preferable to conduct studies on column filters. Generally, 12" layer of coal of effective size 0.9 mm over sand of same thickness of effective size 0.4 mm should give good results. However, in dual media filters strict control of pre-treatment is pre-requisite. Turbidity of inlet water should not exceed 20 ppm. The dual media filter at Kanpur was run for several months at 3 gpm/sq.ft and no deterioration in quality of water was observed. Of course, filter run was reduced to about 40% at the above rate of filtration. Laboratory studies conducted at IIT, Kanpur have shown that for same rate of filtration dual media filters are more effective in removal of both turbidity and bacteria/virus. Regarding turbidity removal comparative data can be seen in Table 2.

6 MULTI-LAYER FILTRATION

Multi-layer filtration can be defined as rapid gravity filter consisting of several granular media both size and density graded, so that coarsest and least dense media is at the top. Work reported at University College, London by Shyam S. Mohanka (6) showed use of a 5 layer experimental filter consisting of the following:

Parameters	Polystyrene	Anthracite	Crushed Sand	Garnet	Magnetite
Thickness	14 cm	14 cm	14 cm	14 cm	9 cm
Size mm.	2.5	1.5	0.8	0.65	0.45
Density	1.04	1.4	2.65	3.83	4.9

Experiments with above filters showed that, especially with low turbidity influents, multi-layer filters afford much higher efficiency. But, this arrangement provided two difficulties:

(1) In back-washing, and

(2) large expansion and hydrophobic nature of Polystyrene.

In my personal opinion, if we are successful in developing metal seeded hydrophilic plastic granules with specific gravity 1.2 or so, triple media filter with these plastic granules, bituminous coal and sand may give some interesting results. The problem was, in fact, referred to Dr G.N. Mathur, Head of the Department of Plastic Technology, HBTI, Kanpur to develop such granules but it could not be followed up. Dual media filter with crushed coconut shell as a lighter media over sand is in use in Ramtek and is reported to be giving satisfactory results.

7 NEW TYPE OF FILTERS UNDER TRIAL

Quite a few type of filters have been designed by Indian Engineers and they are still in trial stage.

7.1 Mimo Filters

Multi Inlet Multi Outlet filter suggested and designed by Professor S.V. Patwardhan of Roorki University has been tried in a model scale in Delhi Water Works in July 1977. Though Professor Patwardhan has designed it as a compact package plant for coagulation, flocculation, filtration and disinfection, but in this paper only the filtration part is being discussed. This filter admits and distributes settled water in the sand media of the filter bed through a net-work of perforated pipe line at different levels of the sand depth. After water has travelled through a definite depth, it is collected in another net-work of pipe line and brought to the outlet chamber. Thus, there are two or more inlets and outlets at various depths of the sand depth. The designer has taken advantage of the fact that even in rapid sand filter design the filter depth that acts as filtering media is generally half or 2/3rd. As such, by giving multi inlets and multi outlets at different depths, rate of filtration could be increased and because such filters are likely to consume less floor area, they shall also prove economical in initial construction in certain cases. The model given by Professor Patwardhan to Delhi Water Works will be operated for about six months more and then it could be possible for us to give detailed data about its efficiency in removal of bacteria and turbidity.

7.2 Upflow Filters

These filters provide an ideal process of filtration, i.e., from coarse to fine media. Lot of model studies and, probably, studies on pilot plants of upflow filters have been done in various laboratories and elsewhere. Advantages claimed are high rate of filtration and low head loss. Mr S.M. Dhabadgaonkar of Regional Engineering College, Nagpur has introduced a new concept of declining rate upflow filter in which the area of cross-section goes on increasing with the height. This provides lower rate of filtration in denser media. Consequently, better quality of water is also expected from this.

I would like to end this paper with a note of John R. Baylis, Oscar Gullans and H.E. Hudson, Jr. "THERE IS, HOWEVER, NO DOUBT AT ALL THAT THE DUAL MEDIUM SYSTEM IS SUPERIOR TO ANY SINGLE MEDIUM BED IN CAPABILITY TO PRODUCE LONGER FILTER RUNS AND, PERHAPS, HIGHER QUALITY WATER."

REFERENCES

1 Huisman, L. "Biological or Slow Sand Filters", Background paper, WHO/CWS/RD/70.1

2 Ives, K.J.,"Filtration of Water and Waste Water" CRG Critical Reviews in Environmental Control, 293, August 1971.

3 Ives, K.J., "Algae and Water Supplies",4, Physical Removal of Algae "Water and Water Engineering",61, 432 (1957)

4 Agrawal, I.C. & Agrawal,G.D., "Operation of Slow Sand Filters for Alum Coagulated Water".

5 Agrawal, I.C., Agrawal, G.D. and Misra, Y.D., "Cost Analysis of Slow and Rapid Sand Filters" send to Journal of Indian Water Works Association.

6 Mohanka, S.S., "Multi-layer Filtration" published in Journal of American Water Works Association, October 1962.

Table 1 - Variation of Aluminium Concentration Along Depth

S.No.	Depth from the top of bed cms.	Total Aluminium conc., (mg/l)	Soluble Aluminium conc.(mg/l)	Estimated conc. of precipitated Aluminium (mg/l)
1.	Above sand bed	0.21	0.14	0.07
2.	6	0.16	0.13	0.03
3.	12	0.15	0.13	0.02
4.	52	0.15	0.13	0.02
5.	110	0.15	0.13	0.02

Table 2 - Comparison of Dual Media Filter with High Rate Rapid Sand Filter

S.No.	RAPID SAND FILTER		DUAL MEDIA FILTER	
	Rate of filtration in LPS/sq.m.	Effluent turbidity in FTU	Rate of Filtration in LPS/sq.m.	Effluent turbidity in FTU
1.	98	0.2	98	0.1
2.	196	0.2	196	0.1
3.	294	0.5	294	0.5
4.	294	0.2	294	0.3
5.	392	0.25	392	0.3

Working Paper No.6

DISINFECTION

by

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1 INTRODUCTION

Measures to treat the water by methods such as storage, coagulation, sedimentation and filtration would render the water chemically and aesthetically acceptable with some reduction in the bacterial content also. However, these cannot be relied on to provide a safe water and is necessary to 'disinfect' the water to destroy all the disease producing organisms. As the raw water sources are becoming increasingly prone to pollution by municipal and industrial wastes, the need for disinfection cannot be over-emphasized, to ensure the safety of the water supply.

Boiling of water or use of copper and silver vessels for storing water which effect some measure of disinfection have been in vogue for a long time in this country. Chemicals like chlorine and its compounds, bromine, iodine, potassium permanganate, ozone, etc., have been used as effective disinfectants. Physical methods like ultraviolet rays, thermal treatment and ultrasonic waves have also been used on a limited scale. A few public water supplies are disinfected with ozone in some of the advanced countries, but the only disinfectant in general use for public water supplies is chlorine.

Water disinfections process, as now ordinarily considered, involved specified treatment for the destruction of disease producing or pathogenic bacteria. Disinfection does not necessarily imply complete destruction of all living organisms, which can be accomplished only by sterilization.

Disinfection is not a substitute for other forms of water treatment. Disinfection usually requires dosing devices that need the attention of skilled operators to avoid breakdown and incorrect dosage. The dangers from the interruption of service are so great that they must be avoided by every possible means.

2 CRITERIA FOR A GOOD DISINFECTANT

For a chemical or an agent to be potentially useful as a disinfectant in water supplies, it has to satisfy the following criteria:

(1) be capable of destroying the pathogenic organisms present, within the contact time available and not unduly influenced by the range of

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physical and chemical properties of water encountered particularly temperature, pH and mineral constituents;

(2) should not have products of reaction which render the water toxic or impart colour or otherwise make it impotable;

(3) have ready and dependable availability at reasonable cost permitting convenient, safe and accurate application to water;

(4) possess the property of leaving residual concentration to deal with small possible recontamination; and

(5) be amenable to detection by practical; rapid and simple analytical techniques in the small concentration ranges to permit the control of the efficiency of the disinfection process.

3 FACTORS AFFECTING EFFICIENCY OF DISINFECTION

The disinfection efficiency is influenced by the following important parameters:

(1) type, condition and concentration of organisms to be destroyed;

(2) type and concentration of disinfectant;

(3) contact time and concentration of disinfectants in water, and

(4) chemical and physical characteristics of water to be treated particularly temperature, pH and mineral constituents.

4 DISINFECTION METHODS OTHER THAN CHLORINATION

(1) Heat or Thermal Treatment: Water can be disinfected subjecting it to vigorous boiling for 15 to 20 minutes. The method is applicable to individual use in times of emergency. Continuous flow water pasteurisers with flow rates of 1000 lph are also available.

(2) Ultraviolet Rays: This method of disinfection involves the exposure of a film of water, upto about 120 mm thick, to one or several quartz mercury vapour are lamps emitting ultraviolet radiation at a wave length in the range of 200 to 250 mm. Applications are limited to individual or institutional systems. The water should be free from turbidity and suspended or colloidal constituents for efficient disinfection.

The advantages are that the exposure is for short periods, no foreign matter introduced and no taste or odour produced. Over exposure does not result in any harmful effects. The disadvantages are that no residual effect is available and there is a lack of rapid field test for assessing the treatment efficiency; moreover, the apparatus needed is expensive.

(3) Ozone: It is a faintly blue gas of pungent odour. Being unstable, it breaks down to normal oxygen and nascent oxygen. This nascent oxygen is a powerful oxidising and germicidal agent. Ozone is produced by

passing high voltage electricity through dry atmospheric air between stationary electrodes. It is usually injected into the water in a baffled mixing chamber.

The efficiency of disinfection by ozone is more than that of chlorine and it is also unaffected by the pH or temperature of the water over a wide range. There is no residual disinfecting action and energy requirements are high.

(4) Oligodynamic Action of Metals: Certain metals, particularly silver when immersed in water have been observed to extent an inhibiting action on bacterial life. In this process silver with or without "activators", such as palladium or gold, is deposited on the surface of sand, porcelain, or filter candles. Water is than passed through the filter or allowed to remain in contact with the silvered surface for a certain time. Although great claims have been made for the advantages of the process, there have been no application, where practical experience might support these claims.

(5) Excess Lime: The "excess lime" process involves the application of sufficient lime for softening and clarification and, in addition, sufficient lime to exert a bactericidal effect. The necessary dose is between 10 and 20 ppm excess lime. After the bactericidal effect has been executed the excess lime must be removed, preferably by any one of the methods of recarbonation.

(6) Iodine and Bromine: The disinfection of water with iodine and with bromine is suitable for small water supplies, swimming pools, and for individual water supplies, for example canteen water for troops and water for travellers where properly disinfected water is unavailable.

(7) Potassium Permanganate: It is powerful oxidant, but its application as disinfectant is rare. To attain a satisfactory result a concentration of 2 mg/l and a 24 hour contact period is recommended.

5 CHLORINE AS A DISINFECTANT

Chlorine has been practically synonymous with disinfection in India. For the last century or so chlorine has been used in water treatment plants. The strong oxiding and bleaching properties have been useful to us in our treatment.

Chlorine and its Properties: Chlorine is an element, having the symbol Cl with an atomic weight of 35.45, melting point - 101.5°C and boiling point - 34.5°C. Gaseous chlorine is greenish yellow in colour and is approximately 2.5 time heavier than air. Under pressure, it is liquid with an amber colour and oily nature approximately 1.5 times as heavy as water. Liquefaction of chlorine gas is accomplished by drying, cleaning and compressing the gas to 35 kg/cm^2. During this process, the resultant liquid chlorine is separated from non-condensable gases. The temperature of the liquid chlorine in the container influences the internal pressure of the chlorine gas and hence its flow from the container. Liquid chlorine must be vaporised in order to be withdrawn as gas and this tends to reduce its temperature and thereby its vapour pressure. At too high discharge rates, the liquid will be cooled excessively resulting in the formation of frost on the out side of the containers. Fatal concentrations are generally avoided as its irritant nature is recognizable at much lower concentrations, the odour threshold being 3.5 ppm by volume. The safety limit for a working environment permits the maximum allowable concentration of chlorine in air of 1 ppm by volume for an exposure period of 8 hours.

6 EFFICIENCY OF CHLORINATION

It is an established fact that chlorine is used practically as a universal disinfectant for water as well as waste water. It has also been successfully used in the control of algae as well as in the destruction of iron-fixing and slime-forming bacteria in pipe lines.

A knowledge of the efficiency of chlorine disinfection and the factors associated with it is therefore very essential for all those engaged in the process of disinfection by chlorine.

Factors governing chemical disinfection efficiency of chlorine include: (1) hydrogen-ion concentration or pH; (2) temperature; (3) contact time; (4) alkalinity or acidity; (5) oxidizable substances; (6) nitrogonous compounds especially ammonia and amines; (7) characteristics of organisms present; (8) forms of available chlorine, and (9) concentration of chlorine. All these factors are inter-related so the effect of any one is affected by any of the others. Some of the above factors are discussed below:

(1) Hydrogen-ion concentration or pH; Chlorination is most effective at a low pH. When chlorine is added to chemically-pure water, a mixture of hypochlorous acid (HOCl) and hydrochloric acid (HCl) is formed:

$$Cl_2 + H_2O = HOCl + H^+ + Cl^-$$

At the pH level of most water supplies (above 4.0) the reaction goes to the right, leaving little chlorine in solution. Hypochlorous acid is weakly dissociated at pH values below 6.0. Above this level it dissociates almost completely as follows:

$$HOCl = H^+ + OCl^-$$

A modern theory (which has some substantiation) of the action of chlorine in destroying bacteria is that there is a chemical reaction between the HOCl produced and an enzyme system. (The reactions are presumed to involve irreversible HOCl oxidation of sulphydryl containing enzymes.) For equal concentration of chlorine added, other conditions being equal, 150 times as much chlorine must be added at a pH of 10 than at a pH of 5 to produce the same killing effect. At the pH range normal to most public water supplies a reduction of pH increases the bactericidal effect of chlorine. The pH of water to be disinfected is of controlling importance in chlorination of water. This has already been discussed earlier. However, it would be worthwhile emphasizing here than HOCl is about 80-100 times more powerful as disinfectant that OCl and as such it would be profitable for the engineer to keep the pH of water on the low side in order to have as much of the chlorine as possible in the HOCl form.

Incidently the addition of chlorine generally tends to decrease the pH values whereas the addition of hypochlorites tends to increase the pH value.

(2) Temperature: The effect of temperature on the efficiency of disinfection is another important factor not generally given its due. Disinfection is a combination of physical and chemical processes, involving diffusion and penetration through cell wall and reaction with enzymes. It is, like all such process, greatly affected by temperature. Generally, the higher the temperature, the more rapid the kill which can be expressed by the following expression for any given percentage of kill:

$$\frac{\text{Time required at temp. } T^{\circ}\text{ F}}{\text{Time required at temp. } (T+10)^{\circ}\text{F}} = 1.5 \text{ to } 3.5$$

This signifies that a decrease of 10°F in water temperature would necessitate almost doubling or trebling the contact time if the same percentage of kill is desired in both cases. If contact time cannot be increased the obvious alternative is to increase the dosage of chlorine.

(3) The Time of Contact: The period available for the interaction between chlorine and constituents of the water is one of the most important aspect of chlorination practice. The longer the time, the greater is the opportunity for destruction.

The time-rate of kill by a disinfectant under idealized conditions of constant concentrations of the disinfectant follows Chick's law which state that Y, the number of organisms destroyed in unit time is proportional to N, the number of organisms remaining, the initial being N_o. Stated differently, this means:

$$dy/dt = K\ (N_o - Y)$$

Where K is a proportionality constant which reflects the rate or rapidity of kill of disinfectant and therefore its efficiency. By integration within the limits, Y = o at t = o and Y = Y at t = t

$$\log_e \left(\frac{N_o - Y}{N_o} \right) = -\ Kt$$

$$\text{or } \log_e \left(\frac{N}{N_o} \right) = -\ Kt$$

For changing concentrations of disinfectant, the observed disinfecting efficiency is generally approximated by the relationship.

$$C^n.t = \text{Constant}$$

Where C is the concentration of the disinfectant, t, the time required to effect a constant percentage of kill, n is an exponent (called co-efficient of dilution) varying from 0.75 to 2.0 depending on the nature of the disinfectant, the species of organisms involved and the temperature. Values of n greater than 1 signify that efficiency decreases rapidly (as it is less diluted), whereas value of n less than 1.0 signifies that changes in concentration are less readily reflected in the efficiency of kill. This latter case could be stated in other words viz., the time factor is more important than dosage in its effect on efficiency; where n = 1, dosage and time of contact are of equal weight. In fact, the most commonly used value of n is 1.0.

It has been reported that the concentration of disinfectant for a constant percentage of kill depends also on the rate of flow of water, besides other factors mentioned in the previous paragraphs. The amount of disinfectant added should be proportional to the square of the rate of flow (unless the detection time provided is too long to be of consequence in any case) and not to the first power of Q as generally practised. Adding the disinfectant in proportion to only the first power of Q as is done in most proportionate feed mechanisms available in the market, only helps to keep the dose constant but does not take into account the fluctuating contact times produced by the variations in Q. This becomes of a special consequence where the distance of the first water consumer is very short from the point of disinfection, so that time is of material consequence. Under these conditions, it is very obvious that the engineer should arrange for sufficient capacity of the feeding equipment to be able to feed in proportion to the square of the rate of flow of water.

The importance of contact time can also be illustrated in the case of swimming pools, wherever heavier residuals of chlorine than generally obtained in drinking water have not proved invaluable in arresting the spread of infection, the simple reason being that the time available to the disinfectant for action is often only a fraction of a minute in passage of the infecting organisms from one swimmer to the next.

(4) Alkalinity and Acidity: These are complementary characteristics because as one increases the other decreases. Alkalinity decreases and acidity increases the effectiveness of chlorination.

(5) Oxidizable Substances: Chlorine is primarily an oxidizing agent. Hence oxidizable substances reduce chlorine and weaken its bactericidal effect. Oxidizable substances include iron, manganese, amines, hydrogen sulphide, nitrogenous compounds, and other organic matter.

(6) Nitrogenous Compounds: Ammonia and amines are the principal nitrogenous compounds affecting chlorination. Compounds of chlorine with ammonia, known as chloramines, may be formed somewhat as follows:

$$HOCl + NH_3 = NH_2Cl + H_2O$$

Chloramines have more power to penetrate bacterial cells and cysts than chlorine alone. The compounds are more effective, for example, in killing *E. histolytica*. Some chloramines affecting disinfection are mono-chloramine (NH_2Cl), dichloramine ($NHCl_2$), nitrogen trichloride (NCl_3), halazone and succinchloramide. The last two compounds include sulphur. Under reducing conditions nitrites and nitrates may react to form amines and to affect chlorine and chlorine residuals materially.

(7) Organisms Present: Some pathogenic organisms are more resistant to chlorination than others. The selective characteristic of chlorine as a disinfecting agent is important in controlling its use and application, e.g., *E. histolytica* is so highly resistant to chlorine that killing it by chlorination of a public water supply is impracticable.

(8) Form of Available Chlorine: Chlorine may be applied as gas, as a liquid, or as a solid. It is probably most effective when applied as a gas and least effective as a solid, although when expressed in terms of concentration of effective chlorine, there is no difference in bactericidal effect.

(9) Concentration of Chlorine: Bactericidal effect increases with concentration of available chlorine and at an increasing ratio.

7 AMOUNT OF CHLORINE REQUIRED AND CHLORINE DEMAND

The amount of chlorine required to kill pathogenic bacteria is dependent on the conditions affecting chlorination, as discussed above. It is customary, in practice, to apply chlorine until the amount found in the treated water is between a trace and about 0.05 to 0.10 or 0.20 ppm. The amount of chlorine required to produce such residual depends on the chlorine demand of the water, and it may reach as much as 5 ppm or more. Such high dosing will probably result in chlorinous tastes and odours. The period of contact should be 15 to 20 minutes before the water reaches the consumer.

The chlorine demand of a water is the difference between the amount of chlorine added and the amount of chlorine, free available and combined available, remaining in the water at the end of a specified contact period at a given temperature. It is to be noted that chlorine demand is a function of the amount of chlorine added, the time of contact, and the temperature. It is an expression of chemical relation, and it is not the amount of chlorine required for disinfection.

8 FORMS OF CHLORINE AND OF CHLORINATION

Available Chlorine is a measure of the total oxidizing power of a chemical containing chlorine and is determined by titrating the iodine released from an acidic solution of potassium iodine by the chemical. Thus the available chlorine in chlorinated lime is 35 per cent, in calcium hypochlorite it is 49.2 per cent, in hypochlorous acid (HOCl) it is 135.6 per cent and in chlorine dioxide it is 263 per cent.

Free available Chlorine is that existing in hypochlorous acid and the hypochlorite ion. It is not possible with the standard orthotolidine test, to distinguish quantitatively between free available chlorine and combined available chlorine because the orthotolidine test must be performed at a low pH to produce the desired greenish-yellow colour.

The addition of an acid reagent splits some chlorine from its combination with ammonia and organic nitrogenous compounds- This chlorine reacts with the orthotolidine in the same manner as other free chlorine. The flash test gives a qualitative measure of free chlorine, but the orthotolidine - arsenite modification gives a quantitative measures of the free chlorine indicated by the flash test.

Combined available Chlorine is that existing in water in chemical combination with ammonia or organic nitrogen.

Residual Chlorine is that amount of either free or combined available chlorine that exists in a sample at the time of analysis.

Free residual Chlorination produces a free available chlorine residual and maintains that residual through part or all of a water treatment plant or distribution system.

Combined residual Chlorination produces, with the natural or added ammonia in the water, a combined available chlorine residual and maintains that residual through part or all of a water treatment plant or distribution system.

The term chlorine residual is vague. It is necessary to distinguish between a free available chlorine and a combined available chlorine residual when considering the bactericidal effect of chlorination.

9 CHLORINATION PRACTICE

The type of available chlorine residual required and the characteristics of the water being treated determine the process of disinfection to be employed or maintained. All chlorination practice, irrespective of the point of application may be classified as free available residual chlorination (i.e., breakpoint or super-chlorination) or combined residual chlorination, depending on the nature of the chlorine residual formed. From practical view point both are not equally applicable to all water sources for disinfection or to improve the quality.

9.1 Plain or Simple Chlorination

This involves the application of chlorine to water as the only type of treatment to afford the necessary public health protection. Plain chlorination can be carried out in situations where:

(1) turbidity and colour of the raw water is low, not exceeding 5 to 10 JTU;

(2) raw water is drawn from relatively unpolluted sources;

(3) water contains little organic matter and iron and manganese do not exceed 0.3 mg/l; or

(4) sufficient contact period between the point of chlorination and the consumer is available.

9.2 Super Chlorination

This is adopted in case of an emergency like a breakdown or in case of waters which are heavily polluted or fluctuate rapidly in quality. It can give excellent results in waters where:

(1) plain chlorination produces taste and odour;

(2) the water is coloured; or

(3) iron and manganese have to be oxidized.

It may be resorted to on special occasions, when available contact time is limited, at the prechlorination stage. Super-chlorination can effectively destroy the relatively resistant organisms such as viruses and amoebic cysts. The dose of chlorine may be as high as 10 to 15 mg/l with contact periods of 10 to 30 minutes. Excess chlorine will have to be dechlorinated.

9.3 Breakpoint Chlorination

As already explained, the addition of chlorine to ammonia in water produces chloramines which do not have the same efficiency as free chlorine. If the chlorine dose in this water is increased, a reduction in the residual chlorine occurs, due to the destruction of chloramine added chlorine. A few possible schemes are as below:

$$2\ NH_2Cl + HOCl = N_2 + 3HCl + H_2O$$

$$2\ NH_2Cl + 2NHCl_2 + HOCl = N_2O + 7HCl$$

The end products do not represent any residual chlorine. This fall in residual chlorine will continue with further increase of chlorine begins to increase in proportion to the added dose of chlorine. The point at which the free residual chlorine appears and when all combined chlorines have been completely destroyed is the breakpoint and corresponding dosage is the breakpoint dosage. Breakpoint chlorination achieves the same results as super-chlorination in a rational manner and can therefore be construed as controlled super-chlorination. Figure 1 shows reactions of chlorine in water.

9.4 Dechlorination

When super-chlorination is employed the water usually contains excess of free available chlorine which must be removed before it become acceptable to consumers. Dechlorination is partial or complete reduction of undesirable excess chlorine in water by any chemical or physical treatment.

Prolonged storage and absorption on charcoal, granulated carbon and activated carbon are effective. Also, reducing compounds like sulphur-dioxide, sodium thio-sulphate and sodium-bisulphite are frequently used as dechlorinating agents. Dechlorination by sulphur-dioxide, and its derivatives is feasible, rapid and precise. About one part of SO_2 (by weight) is required for each part of chlorine to be removed, the reaction yielding HCl and H_2SO_4.

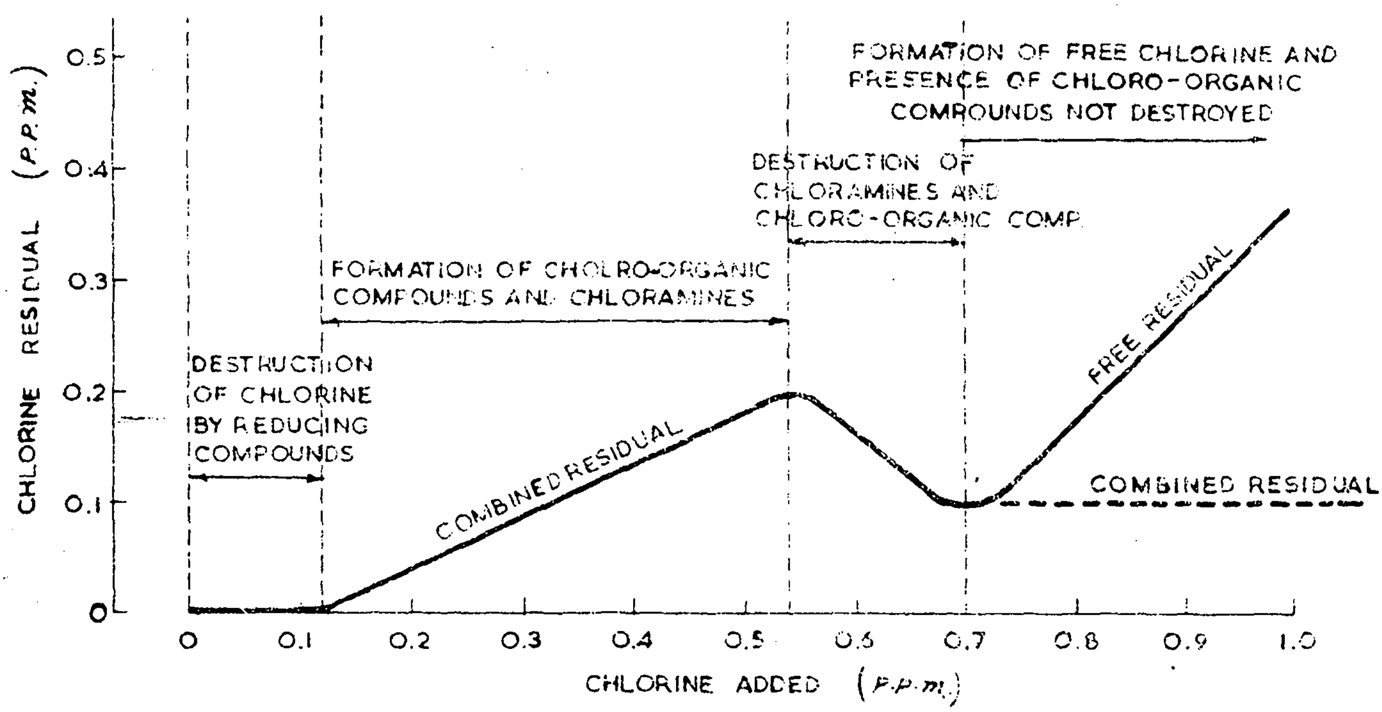

FIG. I REACTIONS OF CHLORINE IN WATER

10 POINTS OF CHLORINATION

The use of chlorine at various stages of water supply system right from raw water collection to the distribution network is a common practice and terms like pre, post and re-chlorination have come into common usage depending upon the points at which chlorine is applied.

10.1 Prechlorination

Prechlorination is the application of chlorine to water prior to any unit treatment process. The point of application as well as the dosage will be determined by the objectives, viz., control of biological growths in raw water conduits, promotion of improved coagulation, prevention of mud ball and slime formation in filters, reduction of taste, odour and colour and minimizing the post chlorination dosage when dealing with heavily polluted water.

10.2 Post-chlorination

Post-chlorination is the application of chlorine to water before it enters the distribution system to maintain the required amount of free chlorine.

10.3 Re-chlorination

When the distribution system is long and complex, it may be difficult to maintain the minimum chlorine residual of 0.2 mg/l at the farthest end unless a very high dosage is applied at the post-chlorination stage, which would, apart from being costly make the water unpalatable, at the reaches close to the point of chlorination. The maintenance of required residual in such cases can be accomplished by a stagewise application of chlorine in the distribution system which is called re-chlorination. Re-chlorination is carried out in service reservoirs, booster pumping stations or at points where the mains supply to distribution zones.

11 CHLORINATION AND WATERBORNE PATHOGENIC ORGANISMS

The causative organisms associated with typhoid fever, dysentery and various gastrointestinal disorders belong to the genus *Salmonella* or *Shigella*.

Cysts of *E. histolytica*, the causative organisms of ameobic dysentry are not destroyed by the normal concentration employed in the water works practice.

Nematodes (some being non-pathogenic) may be present in surface water supplies and the embryo of guineaworm can be sheltered by cyclops. These exhibit a higher resistance to chlorine than enterobacteria and enteroviruses.

Some of the enteroviruses (infectious hepatitis is a known disease of viral origin) are considerably more resistant to chlorine than coliforms. The conventional bactericidal chlorine treatment measures are not enough to kill viruses. A dose of 0.5 mg/l of free chlorination with a contact period of one hour is necessary to inactivate virus.

12 CHLORINE RESIDUAL

Satisfactory disinfection is obtained by pre-chlorination to maintain 0.3 to 0.4 mg/l free available residual throughout treatment, or 0.2 to 0.3 mg/l free available residual in the plant effluent at normal pH values. At higher pH of 8 to 9, at least 0.4 mg/l is required for complete bacterial kill with 10 minutes contact time and for 30 minutes contact time the dosage reduces to 0.2 to 0.3 mg/l.

Some workers consider that the survival of any human enteric viruses after such treatment is remote, especially when satisfactory flocculation is employed in the treatment regime. Complete data are unavailable upon which to base recommendations of residual chlorine requirements to ensure destruction of water borne viruses. However, in practice, 0.5 mg/l of free chlorine for one hour is sufficient to inactivate virus, even in water that was originally polluted and hence this may be adopted to make the water safe.

Where water supply is infested with nematodes, the supply should be pre-chlorinated for 6 hours to maintain a free available residual of 0.4 to 0.5 mg/l. This treatment attenuates most nematodes and renders them immobile which can be removed by settling processes.

13 APPLICATION OF CHLORINE

Chlorine can be applied to water by three methods:

(1) by the addition of a weak solution prepared from bleaching powder, HTH, etc., for disinfecting small quantities of water;

(2) by the addition of a weak solution of chlorine prepared by electrolyzing a solution of brine, where electricity is cheap; or

(3) by the addition of chlorine, either in gaseous form or in the form of a solution made by dissolving gaseous chlorine in a small auxiliary flow of water, the chlorine being obtained from cylinders containing the gas under pressure.

The third method is the common practice in public water supplies.

13.1 Chlorine Storage

Usually, 30, 45 and 67 kg. cylinders as well as half and one tonne containers are employed, the choice depending on the chlorine requirements. All cylinders or containers are intended for immediate use, intermediate storage being provided very rarely.

Liquid chlorine is usually stored in vertical cylinders made of solid forged (seamless) mild steel or in welded steel horizontal 'tanks';

nearly 80% of the content is in the liquid form, the rest being in the gaseous form. Containers are tested at pressures of 57 kg/cm^2 for cylinders and 35 kg/cm^2 for tanks. Cylinders upto 67 kg capacity should be stored vertically, so that a leaking container if found can be removed with the least possible handling of others. Tonne containers should be stored on their sides at all times. They should rest securely on cradles or on a level rack equipped with adequate safety blocks, to prevent rolling and be slightly elevated from the floor to keep them dry. It is preferable to provide for separate storage of full and empty cylinders. Care should be taken to prevent them from falling over or from being hit by moving materials. Dropping or bumping of containers is very dangerous. Containers should be stored in a cool ventilated area protected against external sources of heat like steam electric heaters or away from inflammable goods.

13.2 Chlorinator

A chlorinator is a device designed for feeding chlorine to a water supply. Its functions are: (a) to regulate the flow of gas from the chlorine container at the desired rate of flow, (b) to indicate the flow rate of gas feeding, and (c) to provide means of properly mixing the gas either with an auxiliary supply of water or with the main body of the liquid to be disinfected.

The usual fittings and parts of a chlorinator are:

(1) chlorine cylinder or tank supplied with its own main valve and filled with liquid and gaseous chlorine, under pressure;

(2) fusible plug, a safety device provided over all cylinders and containers designed to melt or soften between 70°C to preclude a buildup of hydrostatic pressure resulting from thermal expansion due to fire and other hazardous conditions;

(3) reducing valve to bring the pressure of the gas down to 0.70 to 0.3 kg/cm^2;

(4) pressure gauges, one for indicating the cylinder pressure and the other delivery pressure;

(5) measurement device, consisting of an orifice to measure upstream or downstream pressure of gas, with manometer containing an indicating liquid of carbon tetrachloride or concentrated sulphuric acid or by a rotameter, and

(6) a 'desiccator valve' or 'non-return valve' containing concentrated sulphuric acid or calcium chloride through which the chlorine must pass to free it from moisture so that the corrosive action of moist chlorine on the fittings is prevented.

13.3 Piping System

Moist chlorine unlike dry gas or liquid chlorine is highly corrosive. Pipelines, valves and other fittings through which dry chlorine passes should be tightly closed when not in use to prevent absorption of moisture from the air.

Dry chlorine gas or liquid chlorine under pressure should be conveyed through extra heavy wrought iron or steel pipe or flexible annealed copper tubing tested for 35 kg/cm^2 working pressure. The discharge line from the chlorine container should be flexible and sloping upwards, especially when chlorine is discharging in the liquid state. Long pipe lines should be avoided. Hard rubber, silver or platinum tubing is necessary for conveyance of moist chlorine gas or aqueous chlorine solutions at low pressure.

To prevent condensation of gas, piping systems and control equipment should be at the same or a higher temperature than that of the chlorine container. Chlorine gas lines are preferably located overhead rather than along the floor, to take advantage of the warmer ambient temperatures. For liquid chlorine piping systems, conditions which contribute to vaporisation should be avoided.

For pipes 35 mm diameter and smaller, connexions may be either screwed, welded or flanged. If flanged, facing should be small tongue or groove. Gaskets should be made of antimony lead (with 2 to 3% antimony) or asbestos sheet. Rubber gaskets are not suitable. Screwed fittings should be of forged steel construction.

Pressure indicators in the system have Teflon diaphragms or silver foil protectors. Pressure reducing valves may be of bronze or silver plated body with silver diaphragm or of Monel metal with a Teflon diaphragm.

13.4 <u>Number of Cylinders or Containers</u>

Normal chlorine dosages required to disinfect water supplies not subject to significant pollution would not exceed 2 mg/l. The actual chlorine dosage has to be determined on the basis of chlorine demand tests. The chlorine feed rate is then computed by dividing the expected maximum dosage of chlorine by maximum flow rate.

Total daily chlorine requirements can be estimated from the daily average consumption in a maximum day. The peak and the minimum rate requirements should be taken into consideration when designing a chlorine supply and feeder system and not merely the total daily requirements of chlorine.

When chlorine gas is withdrawn from a cylinder containing the liquefied gas, the pressure drops and the liquid 'boils' liberating more gas till the pressure is restored. This boiling absorbs heat continuously, thus producing a cooling effect in the liquid region. If the withdrawal is continued, the liquid may freeze and no more gas will be evolved. It is, therefore, essential to keep the atmosphere, round the containers in service warm and to ensure that there is not an abnormal rate of withdrawal from a single container with heavy demand of gas.

The recommended discharge rates are approximately 6.5 to 7.5 kg/hr. from a one tonne container and 0.8 kg/hr. from cylinders. Equipment should have sufficient capacity to exceed highest expected demand at any time and to provide continuous effective discharge under all prevailing hydraulic conditions. It is good practice to provide for duplicate equipment since disinfection process cannot be stopped at any time.

When the gas discharge rate from a single container will not meet the requirements, two or more can be connected to a manifold and discharged simultaneously. It is advisable not to couple more than four containers to a manifold. When discharging through a manifold, care must be taken that all the containers are at the same temperature, particularly when connecting a new cylinder to the manifold. Where more than 3 or 4 cylinders are used, the connexions should be arranged in groups so that one complete group can be changed at a time. Storage of chlorine lasting a month or two should be provided. It is advisable to keep the full cylinders in the same room as the cylinders in service.

13.5 Chlorine Housing

The chlorine cylinders and feeders should be housed in a fire-resistant building or an isolated room, easily accessible, close to the point of application and convenient for truck loading and safe container handling. The floor should be at least 15 cm above the surrounding ground and drainage should be adequate. Each such separate room or building should have at least two exists with doors opening outwards. Natural ventilation and means for cross ventilation that allows an approximate air change in 10 minutes is desirable. For small installations, provision of ventilator openings at the bottom, one opposite the other is adequate.

Separate and reasonably gas-tight enclosures opening to the outdoors should be provided for housing the chlorine feeding equipment in large installations and in buildings occupied by persons. These enclosures should be vented to the upper atmosphere and equipped with positive means of exhaust (near the floor level, at the centre of the room or opposite to the entrance) capable of a complete air change within 2 to 4 minutes in an emergency. A satisfactory ventilation scheme involves a combination of fresh air and exhaust system, consisting of fans that force the fresh air into the enclosure through openings near the ceiling with exhaust fans to clear away any chlorine contaminated air near floor level. The design of the exhaust system should not include the natural ventilation that may be available.

13.6 Chlorine Vaporizers

When the required gas discharge cannot be obtained without complex or potentially hazardous manifold system, artificial vaporization of chlorine may be necessary. In general where the chlorine requirements exceed 900 kg in a period of 24 hours, the use of vaporizer is indicated.

Chlorine vaporizer includes a sealed chlorine pressure chamber immersed in hot water baths, thermostatically controlled, to maintain constant temperature and to ensure superheated chlorine gas supply. These baths are vented to the atmosphere. It is standard practice to provide a manual or automatic pressure reducing valve in the chlorine gas outlet line adjacent to the evaporator; this prevents liquid chlorine from passing through the evaporator into the chlorine gas supply line and reduces gas pressure, thus helping to prevent chlorine liquefaction.

13.7 Chlorine Feeders

Chlorine feeders are used for control or measurement of chlorine in the gaseous state and to apply chlorine as a gas or as an aqueous chlorine solution. They are of two types - Direct feed equipment which involves metering of dry chlorine and conducting it under necessary pressure conditions to the water to be chlorinated, and Solution feed equipment involving metering of dry chlorine gas under pressure or vacuum and dissolving it in a small amount of water to form a strong solution prior to its application.

The principle of operation of these equipments, depends on the regulation of flow by establishing a pressure relationship between the upstream and downstream sides of either a constant or a variable orifice in the chlorine flow gas line. Control of the feed rate is effected either by varying the pressure differential across a fixed orifice (variable differential unit) or by varying the size of orifice (constant differential unit).

13.7.1 Direct feed equipment

The gas is applied directly through some type of diffuser into the water to be treated. Gas passes through a pressure compensating valve and then to a back pressure regulating and check valve designed to maintain a constant downstream pressure and to prevent liquid from entering the chlorine line in the event of shut down or the development of excessive back pressure at the point of application.

Since chlorine gas is only slightly soluble in water ($0.35m^3$ is required to dissolve one kg of gas at 20°C), direct feed of chlorine gas under pressure into the water system to be treated limits the rate at which gas can be fed. Maximum pressure at the point of application approximates 1.5 kg/cm^2. Hence the direct feed equipment is used only as an emergency equipment or for small installations. The point of application should be within six meters of the feeder and also ensure adequate mixing time ahead of pumps, valves and other fittings. Such feeders have a maximum individual capacity of 16.5 kg in 24 hours when applied to an open tank.

13.7.2 Solution feeders

They are preferred to direct feed equipments because the presence of undissolved chlorine gas might create corrosive conditions with moisture and the chlorine gas may escape while using direct feed equipments.

For these feeders to operate efficiently a continuous supply of 600 litres of water per kg of chlorine applied with a minimum flow of 5 lpm is needed.

These feeders are of two types, viz., (1) Pressure type and (2) Vacuum type.

(1) Pressure type

This system depends on the fact that if a stream of water is made to pass through a narrow constriction, there is a pressure difference between upstream and downstream of the constriction. The quantity of water passing is proportional to the square root of the difference of these two pressures. A diaphragm, in the tank, separates the incoming water from the solution tank but transmits the pressure of incoming feed. This device automatically stops or starts with the main flow. The diaphragm has a life of upto 12 months.

A diaphragm pump or plunger pump can be used for feeding chlorine solution against pressure. The pump can be driven by either an electric motor or hydraulic motor at constant speed or from a differential head of venturi tube or orifice plate (Figure 2).

(2) Vacuum type

Chlorine gas is maintained under vacuum throughout the metering apparatus. The opening of the chlorine inlet valve is governed directly or indirectly by the vacuum induced in the system (Figure 3).

The vacuum type consists of:

(a) a differential type of variable area flow meter;

(b) a compensating vacuum regulating valve to maintain a constant upstream pressure;

(c) a back pressure valve to maintain constant downstream pressure;

(d) an injector or educator to create the necessary vacuum in which gas also may be mixed with a small quantity of water prior to the point of application, and

(e) a vacuum breaker to prevent the possibility of water being drawn into the apparatus.

13.8 Ancillary Equipments

Weighing scales are necessary to record the weight of chlorine used in 24 hours which would serve as a check of the daily consumption and also enable the cylinders to be changed when they get empty. Chlorine gas can be absorbed in caustic soda; 500 gms of which, in 2 litres of water, will absorb 400 gms of chlorine. A jar containing about 5 litres or more of this solution should be kept handy in every chlorinator room. In case of emergency, provision should also be

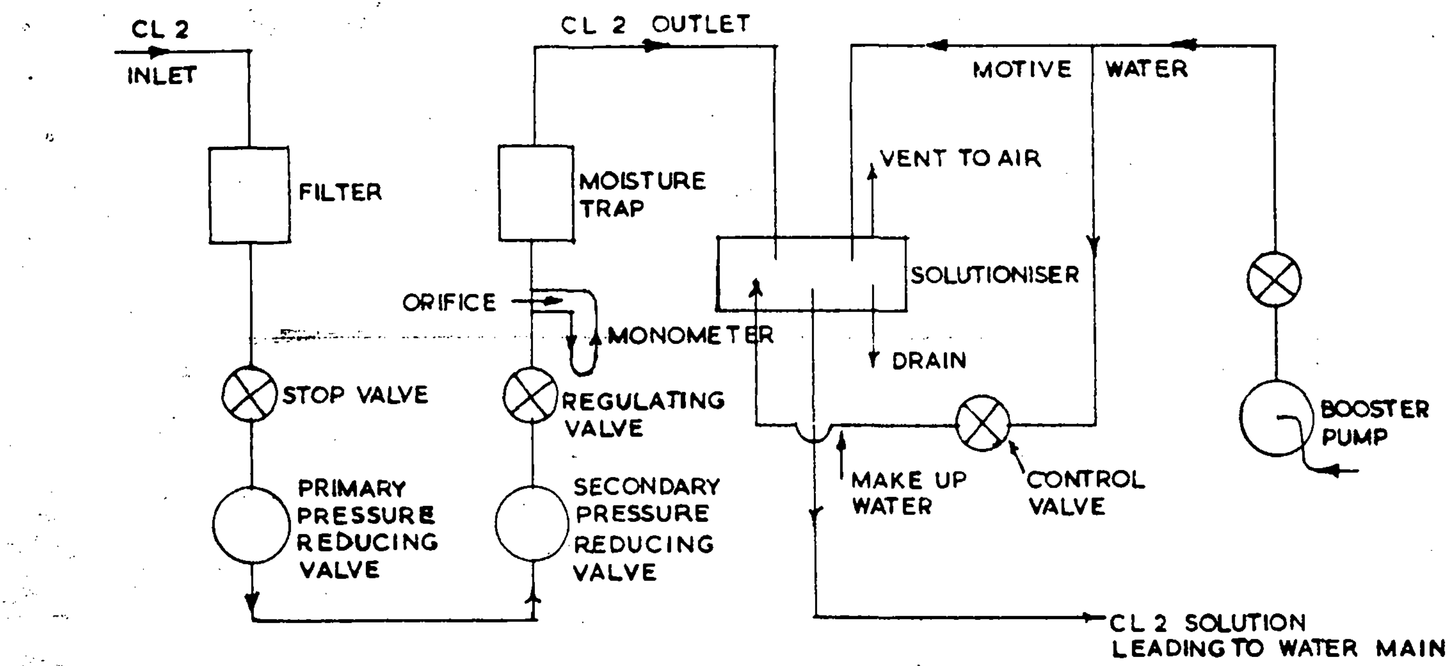

FIG. 2 PRESSURE TYPE CHLORINATOR

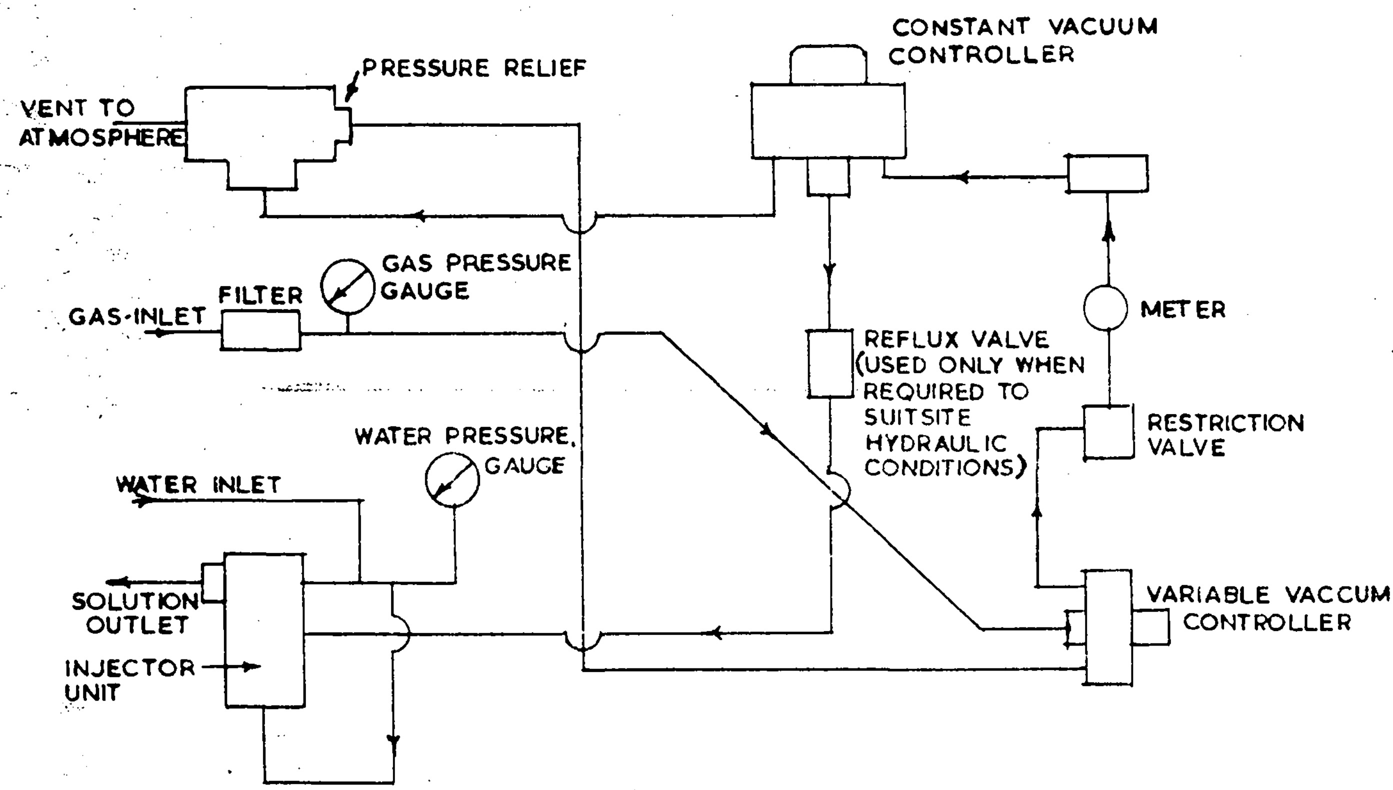

FIG. 3 VACUUM TYPE CHLORINATOR

available for passing chlorine into the solution through an iron pipe or through a hose properly weighted to hold it under the surface.

13.9 Safety Considerations

(1) Only trained personnel should be permitted to handle chlorine cylinders and chlorinating equipment. They should be made aware of the hazards involved, the precautions to be observed and first aid to be rendered in emergencies. Rubber gloves, aprons and suitable gas masks should be provided. These should be housed in an easily accessible (unlocked) cupboard placed outside the chlorinator room. It is very important that the operating personnel are trained in the proper use of gas masks. A faulty gas mask is worse than none at all. Hence it is very important that these are tested frequently and the canisters are changed at proper intervals.

(2) When a chlorine leak occurs, the mechanical ventilation system should be opened immediately before any person enters the chlorine room. It must be made a point that chlorine container valves are closed first before any investigation is started.

(3) Cylinders containing chlorine should be handled gently. They should not be bumped, dropped or rolled on the ground and no object should be allowed to strike them with force. The protective hoods over the valve should always be kept in place except when the cylinders are in use. Flames should never be applied to chlorine cylinders or their valves.

(4) Cylinders should not be stored in the open or in damp places. Empty cylinders should be stored away from full cylinders so that they do not get mixed up. It would be desirable to tag the empties as an additional precaution. Incidentally, this will ensure prompt return of used cylinders.

(5) In case the valve is found to be stuck, the cylinder should be immediately returned to the supplier. No attempt should be made to ease a stuck valve by hammering as this is very dangerous.

(6) Only the spanners prescribed for use should be used as it is important not to put too much leverage on the valves.

(7) Cylinders as well as the chlorinator should be treated at every shift period, for leaks, first by trying to detect the sharp irritating smell of chlorine, then by passing over each cylinder and round each valve and pipe connexions a rod, with a small cotton-wool swab tied on the end, dipped in an aqueous solution of ammonia. If Chlorine is present in the air, the swab will appear to 'smoke', due to the formation of white clouds of ammonium chloride. If the leak appears to be heavy, all persons not directly concerned should leave the area and the operator should put on his mask and make a thorough search for the leak. In tracing a leak, always work 'downstream', i.e., start at the cylinder and work down along the line of flow until the leak is

found. It will save many valuable minutes over the practice of starting in the middle of the chlorinator and searching vaguely back and forth over the whole equipment.

(8) Water should never be applied to a chlorine leak to stop it, as it will only make it worse. If the leak is in the chlorinator, the cylinder should be immediately shut off until the pressure has reduced. The joint or gasket should be repaired, replacing with new packing, if necessary. If the leak cannot be stopped, the cylinder should be taken out of doors to a safe place downwind of the water works and nearby habitation and the valve should be kept open till the container is empty.

(9) Solvents such as petroleum, hydrocarbons or alcohols should not be used for cleaning parts which come in contact with chlorine. The safe solvents are chloroform and carbon tetrachloride. Grease should never be used where it can come in contact with chlorine as it forms a voluminous frothy substance on reaction with chlorine. Only special cements recommended by manufacturers should be used.

(10) No direct flame should be applied to a chlorine cylinder when heating becomes necessary as this is hazardous. A water bath controlled not to exceed 27°C should be used.

(11) Before disconnecting the flexible leads from containers to gas headers, the cylinder valves should be closed first and then the gas under pressure should be drawn from the header and flexible leads before the header valve is closed. The exhaust system should be turned on and operated while the cylinders are being disconnected or repairs being made.

14 CONCLUSIONS

The efficiency of chlorination in preventing the known water borne diseases is open to no doubt, but it is essential to obey certain rules in its application, otherwise failures occur. The haphazard, uncontrolled addition of chlorine to water is not chlorination, but is dangerous in creating a false sense of security and it is responsible for many of the complaints and criticisms of the process. It is essential that the water to be treated should be in suitable condition, that the chlorine should be properly applied in respect of method and dosage, that the contact time should be adequate, and the control conscientious and efficient.

Expansion of the chlorination process has been rapid and it is now employed in all parts of the world. There are few public water supplies of any magnitude in which chlorination is not normally employed in one form or another. It has, without doubt, been one of the greatest advances in water purification and has proved of inestimable value to water authorities in permitting the use of many supplies which otherwise would not have attained the required standard of purity.

REFERENCE

1 "Manual on Water Supply and Treatment (Second Edition), Ministry of Works and Housing, Government of India Publication, 1976.

2 "Care and Use of Chlorinators" Refresher Course held at V.J.T.I., Bombay, February 1973.

3 Holden, W.S., "Water Treatment and Examination", J & A Churchill, 1970.

4 Cox, C.R., "Operation and Control of Water Treatment Processes", World Health Organization, Monograph Series No.49, 1964.

5 Babitt, H.E., Doland, J.J., Cleasby, J.L., "Water Supply Engineering", McGraw Hill Book Company, 1955.

6 "Safety Practice for Water Utilities", AWWA Manual, July 1959.

Working Paper No.7

WATER DISTRIBUTION - PROBLEMS AND A DISCUSSION

by

R. PITCHAI*

1 INTRODUCTION

The problem of determining economic pipe sizes for water distributions systems is quite familiar to engineers concerned with water supply. The pipe network portion accounts for a major share of the investment in distribution systems, which in itself represents from approximately 90% of total water works investment for small communities supplies from wells to about 40% for large cities(1). In such a context, the need to examine closely the decision-making process or the design of distribution systems is evident.

Several attempts have been made in the past four decades to find a satisfactory method of determining least cost designs of distribution networks. An optimal design has usually been defined as a system of pipes and other components that meets certain specifications relating to the distribution of flows and pressures and which requires a smaller investment than any alternative system capable of meeting the same hydraulic specifications. Trial and error methods, hydraulic and electrical models, and more recently, electronic digital computers have been employed for the solution of this problem. Before the advent of computers, a reasonably satisfactory solution had to be based on only a few trials because of the extensive labour and time involved in the computation. Even for moderately sized networks, the hydraulic balancing with a chosen set of pipe sizes was itself a laborious task and attention had to be paid to devise ways and means of speeding up this operation. With recent advances in mathematical programming techniques and the introduction of the high speed digital computer, emphasis is generally being shifted from the balancing part for which efficient algorithms are already available, to the economic network design part where powerful and rigorous techniques are being developed.

The total problem of optimizing the design of an entire distribution system of a large city with multiple supplies, impounding and service reservoirs, inlet and booster pumping stations, hydrants and several types of transmission devices like open channels and pressure pipes is quite a complex one which is rendered even further complicated by the variations to which the demand pattern is subject. It is not the scope of the present discussion to cope with this problem in its entirety; the objective here is to review certain analysis and design problems of significance and present a method which may be used to solve the important problem of minimizing the cost of the pipe network.

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2 REVIEW OF LITERATURE

A very brief survey of some of the pertinent literature of water distribution networks is presented in the following paragraphs. Information on certain planning factors such as per capita quantities and key design criteria for distribution systems such as recommended minimum residual pressures can be found in the Manual on water supply and treatment(1).

The review is in two parts. The first deals with hydraulic balancing when pipe sizes are given and the second with attempts at cost minimization.

2.1 Balancing the Network with a Given Design

The hydraulic problem is one of determining flows and head losses in a network of pipes where a set of inlet and draw-off flows or pressures are specified. The solution must be consistent with Kirchoff node and loop laws. The problem is usually treated as two distinct sub-problems: (i) to find the magnitude and direction of flows in the pipes for a given set of inflows and outflows at the boundaries, and (ii) to determine the headlosses in the pipes for a set of water elevations specified at certain junctions of the system. Notable among the several methods are those developed by Hardy Cross(2), the electrical analogy developed by McIlroy(3), and more recently, iterative procedures using digital computers. These methods have virtually replaced other earlier ones, largely on account of their accuracy and efficiency.

2.1.1 Hardy Cross method

The Hardy Cross method is a relaxation technique which, through successive iterations, applies a series of linearly approximated corrections to either assumed flows ($Q_{i,j}$) or head losses ($H_{i,j}$) in all the pipes of the system. When inflows are known, a distribution of flows in the pipes is first assumed, then a flow correction ΔQ_i is calculated as shown below and applied to all the j pipes in any loop i:

$$\Delta Q_i = - \frac{\Sigma_j H_{i,j}}{n \, \Sigma_j \frac{H_{i,j}}{Q_{i,j}}} \qquad \text{Eqn. (1)}$$

where n is the exponent of Q in the formula for head loss in a pipeline, viz.,

$$H_{i,j} = K_{i,j} \; Q^n_{i,j} \qquad \text{Eqn. (2)}$$

The subscript i refers to the loop number and the paired subscript (i,j) refers to the j^{th} pipe in loop i. Similarly, when water elevations at a few junctions in the network are specified, the head losses in the pipes are first assumed, i.e., trial values of pressure are assumed at the remaining junctions. Then an approximate head loss correction ΔH_i to be applied to all pipes meeting at any one junction i is computed as follows:

$$\Delta H_i = - \frac{n \Sigma_j Q_{i,j}}{\Sigma_j \frac{Q_{i,j}}{H_{i,j}}} \qquad \text{Eqn. (3)}$$

The paired subscript (i,j) here refers to the j^{th} pipe meeting at junction i. When a pipeline serves in more than one loop or joins more than one junction, it receives corrections from both loops or junctions. Upon iteration, when the flow or headloss corrections to be applied become smaller than a small predetermined limit, the process is stopped and the network flows and head losses are in (approximate) balance.

Although the Hardy Cross method has been quite popular with engineers and can be used to solve network problems to any desired degree of accuracy, its convergence may be quite slow for large and complicated networks. This is due in part to the fact that the flow correction from any loop is estimated on the basis of the Kirchoff loop law equation applicable for that particular loop only. But the common pipes are subject to correction from other loops too and since such common loop corrections are not considered in estimating flow correction of any one loop in the Hardy Cross method, the first degree approximations to corrections contained in Equation (1) tend to vanish rather slowly. A balancing algorithm based on the Newton-Raphson technique that simultaneously solves the Kirchoff law equation for all the loops has been described in the literature[(4),(5)]. The convergence of this algorithm is usually quite rapid and can be mathematically demonstrated.

2.1.2 Electrical network analyzer

The analogy between flow of current in an electrical circuit due to electrical potential and the flow of fluid in a pipe network due to head or potential has led to electrical models of the hydraulic network to solve complicated problems. The current flowing in a conductor is related to the voltage drop causing it according to Ohm's law as

$$V = RI \qquad \text{Eqn. (4)}$$

where V is the voltage drop, I is the current in amperes and R is the resistance in ohms. The electrical form of the Kirchoff laws are:

$$\Sigma_j \; I_{i,j} = 0 \qquad \text{Eqn. (5)}$$

and

$$\Sigma_j \; V_{i,j} = 0 \qquad \text{Eqn. (6)}$$

where $\Sigma_j I_{i,j}$ represents the algebraic sum of the currents $I_{i,j}$ over all lines j connected at junction i, and $\Sigma_j V_{i,j}$ stands for the algebraic sum of the voltage drop $V_{i,j}$ in all lines j around a closed circuit or loop i.

A compication which arises in such a representation is that the head loss in a pipeline varies (depending on roughness and the Reynold's Number)as the 1.75th to the 2nd power of Q, whereas the voltage drop in an ordinary resistor varies as the first power of I. Hence, special type of resistors that have the characteristic

$$V = KI^{1.75 \text{ to } 2} \qquad \text{Eqn. (7)}$$

have to be found to simulate pipe friction losses. Here, k is a 'resistor coefficient' analogous to the head loss coefficient, k_p.

Camp and Hazen(6) first exploited the analogy between electrical networks and hydraulic networks and made investigations with an electrical 'analogue' of a water distribution pipe network using a linear electrical circuit. After several investigations, McIlroy(3) perfected non-linear resistance elements ('Fluistors') that have electrical resistance characteristics satisfying equation(7), thus enabling the construction of a more exact electrical model of the hydraulic network.

Among the systems available for large network analysis, the electrical analyzer represents a valuable contribution. When carefully assembled and operated, it permits assessment and modification of sections of the prototype hydraulic network as the analysis proceeds. Direct converting and reading instruments eliminate the need for making computations. However, the physical assembling of the system is time consuming and special resistors are indispensable. Moreover, the resistors are available only in certain sizes and a source of error (or not more than 2.5% for individual resistors) is introduced in choosing the one with the nearest available coefficient k.

3 ECONOMIC DESIGN OF PIPE NETWORKS

The economic design of a pipe network is considered in the programming context. The optimum combination of pipe sizes is sought that minimizes the total cost of the system while meeting all the constraints set on it. The constraints include the hydraulic laws and operational ones such as the minimum permissible sizes, restriction to commercially available sizes, and usually, minimum residual pressure requirements at critical nodes. The cost of the network is generally assumed to include the first cost of the pipes, pumps and other components and the present value of maintenance and operating costs.

In 1969, Tong,O'Connor, Stearns and Lynch(7) published an 'equivalent length method' of balancing hydraulic networks and indicated that an approximate solution to the problem of economic pipe sizing can be simultaneously obtained therefrom. Using Hazen-William's formula for pipe flow, a new term L_e was introduced which was

$$L_e = \ell\,(100/C)^{1.85}\ (0.667/D)^{4.86} \qquad \text{Eqn. (8)}$$

where L_e is the length of a pipe of standard diameter (8") and standard Hazen-Williams C-value of 100. This pipe is hydraulically equivalent to a pipe whose actual length is ℓ, diameter is D and Hazen-Williams coefficient is C. Instead of applying the Kirchoff loop law to the sum of the head losses ΣH in the loops, the equivalent length method distributes the available head loss to the several pipes directly meeting the requirement $\Sigma H = 0$, and attempts to balance the relative pipe resistance in the form of equivalent lengths, L_e in all the loops of the network; i.e.,

$$\Sigma L_e = 0 \text{ for all the loops.} \qquad \text{Eqn. (9)}$$

An iterative procedure similar to the Hardy Cross method has been used for balancing L_e in this study. Assumed flows in all the pipes of the network are successively adjusted to balance the relative pipe resistances. It is claimed that such a balance leads to a minimum possible total of all the equivalent lengths and thus to 'least amount of pipe' in a network of equal sized pipes. Also, the imposition of the above condition, $\Sigma L_e = 0$, is reported to establish a general "evenness" of flow throughout the system, and "optimum design for any set of fixed conditions of topography, pressure requirements, source of supply, draft, and geometric pattern of distribution network"[7]. The elimination of the trial-and-error feature of Hardy Cross method was cited as an advantage of this algorithm. Two examples were presented to illustrate the application of the method.

In the search for better methods of water distribution system design, the "balancing of equivalent lengths" technique would appear to have merit particularly in initial studies preliminary to a comprehensive systems analysis. However, in networks with multiple sources and pump-type boundary conditions, the flow pattern may not be so obvious and problems of convergence could arise. Raman[8] has recently provided a comprehensive review.

4 PIPE NETWORK COST MINIMIZATION PROBLEM

It can be shown that the problem of minimum-cost design of a distribution pipe network subject to:

(1) the provision of required domestic and fire flows at specified draw-off junctions, and

(2) the maintenance of minimum residual pressure at critical junctions

can be cast as one of non-linear, integer programming[9]. Such a model and an engineering approach to its solution are briefly discussed.

More detailed exposition and reference to earlier works in the topic can also be found in literature[9].

4.1 Formulation of the Objective Function

The principal part of the total cost function of a distribution pipe network is the cost of pipes. The installed first costs of pipes can be related to their diameter by an empirical exponential function of the form.

$$C' = \alpha \ell D^n \qquad \text{Eqn. (10)}$$

where C' is in Rupees, ℓ is the length of pipe and D is the diameter. α and n are parameters to be determined locally. Then, the total installed cost of all the pipes in the networks is

$$C_p = \sum_{\text{all } (i,j)} \alpha \, l_{i,j} \, D^n_{i,j} \qquad \text{Eqn. (11)}$$

where the paired subscript (i,j) denotes the j^{th} pipe in loop i.

In addition to pipe cost, the cost of friction losses in the pipe network constitutes another important component of the total cost. In pumped systems, it represents the cost of energy required to overcome pipe friction; in gravity systems, the same is an indirect cost on the system if we consider that higher pressures are desirable at the draw-off points. As such, the energy cost of pipe friction losses can be incorporated in the objective function for all supplies. Relating this cost to motive power prices (here assumed as electricity), the present value of costs associated with pipe friction losses in the system can be computed and incorporated in the objective function to be minimized. Such a total cost function is

$$C_T = \alpha \sum_{(i,j)} l_{i,j} \, D^n_{i,j} + \beta' \left(\frac{P_v wbE}{e} \right) \sum_{(i,j)} Q_{i,j} H_{i,j}$$

Eqn. (12)

where $Q_{i,j}$ and $H_{i,j}$ stand for the flow and head loss in pipe (i,j), P_v is the present value of an annuity of 1 Rupee discounted at rate r over the economic time horizon T, w is the unit weight of water, b is a load building factor, E is the unit cost of electricity and e is the wire-to water efficiency of pumping.

4.2 Formulation of the Constraints

The diameters, flows and head losses in the pipe network must meet certain constraints in the form of hydraulic flow formulae, Kirchoff laws for nodes and loops, and certain operational constraints regarding minimum pipe sizes, commercially available pipe sizes and minimum permissible residual pressures. Such constraints can be represented by the following set:

(a) $H_{i,j} - \left[84.1 \frac{1}{C_{i,j}^{1.85}} \, l_{i,j} \, D_{i,j}^{-4.87} \left| Q_{i,j} \right|^{0.85} \right] Q_{i,j} = 0$ for all pipes Eqn. (13)

(b) Some $\Sigma_{(i,j)} \; Q_{i,j} + q_m = 0$, for all nodes Eqn. (14)

(c) $(\Sigma_j \; H_{i,j})_i + S_i = 0$ for all loops Eqn. (15)

(d) $D_{i,j} \geqslant D_{min,}$ for all pipes

(e) $D_{i,j} \; \Sigma \left\{ D_A \right\} \doteq \left\{ D_1 \; . \; . \; . \; . \; D_a \right\}$ for all pipes

(f) $(_{\text{Some}} \; (^{\Sigma}i,j) \;\; H_{i,j})_k \leqslant h_k$, over all specified paths

(g) g_p (relevant q_m, S_i) = 0 for all pumps, if any Eqn. (16)

In the constraints set, (a) is a version of Hazen-Williams formula for flow in pipes; (b) and (c) are Kirchoffs' node and loop laws respectively; (d) assures that all pipes are not smaller than prescribed minimum size D_{min}; (e) specified that the sizes shall correspond to commercially available ones D_1, D_2. . . . D_a; (f) is the equivalent of maintaining minimum permissible residual pressures at draw-off nodes, by requiring that along specified pathways in the network, the sum of headlosses shall not exceed present magnitudes; and (g) guarantees that the inflow and pressure at pump nodes shall correspond to the specified characteristic curves of pumps. The quantities q_m, S_i and h_k stand for inflow (or outflow) at node m, unbalanced head at loop i, and maximum pressure difference permissible over path k, respectively.

4.3 Analysis

This mathematical model for cost minimization of pipe networks assumes that the layout and lengths of pipes are known, and for the moment, that only one demand pattern is considered. The problem can now be seen to be one of non-linear, constrained minimization in numerous variables. The constraint set (e) restricts the domain of feasible diameters to a few specific values thereby discretizing the objective function and the set of feasible diameters. In this analysis, it is assumed that α, η, P_v, b, E, e and C are known, non-negative parameters and $l_{i,j}$, q_m, S_i, D_{min}, D_A, and h_k are given input vectors.

The three sets of variables $D_{i,j}$, $Q_{i,j}$ and $H_{i,j}$ are treated as decision variables; i.e., the solution seeks that set of feasible $D_{i,j}$, $Q_{i,j}$ and $H_{i,j}$ which minimizes the total cost of the pipe network. For this non-linear, integer programming problem, an iterative, sequential search procedure has been developed[9] and the same is briefly outlined below.

4.4 Constructing a Starting Solution

The most direct way of meeting constraint sets (d) and (e) is to choose diameters as the variables to be set for a trial, and derive other decision variables (Q, and H), therefrom. Then, while selecting the diameters, only those feasible with respect to (d) and (e) may be chosen. Such diameter selection is a significant step which eliminates the trial and error procedures that would be otherwise required. The setting of such a diameter vector ($D_{i,j}$) leaves the flows and headlosses to be determined. Solving for $Q_{i,j}$ and $H_{i,j}$ with given $D_{i,j}$ from constraint sets (a), (b) and (c) is the familiar problem of hydraulic network balancing.

4.5 Constructing a Penalty Function

If the constraints system is now examined,the method of starting with a feasible diameter vector and balancing the network to obtain feasible flows and headlosses has given rise to a solution feasible with respect to all constraints except set (f). The resulting head losses may either satisfy or fail to satisfy set (f); i.e., head losses summed over specified paths may or may not be less than the permissible limits set. A rational approach to the treatment of these constraints is to weight them and blend them into the objective function in such a form that the violation of these constraints will penalize the causative design while ranking alternative designs. Such is the penalty function approach. This penalty function can be related to the extent of violation of the (f) type constraints.

4.6 Sequential Random Search Procedure

Having established the model and formulated a function to rank alternative designs, a sequential random search was conducted starting with a trial design (set of diameters) and improving it in successive iterations until a terminal design with very low probability of improvement resulted. The rationale of this method differs from that of classical optimization in that it does not attempt to identify the global optimum with complete certainty; rather, it provides a statistical estimator of the best design. The technique can be summarized as follows:

(1) Select a starting design from a specified population of starting designs.

(2) Proceed sequentially from the starting design to an improved Terminal Design (T.D.) according to a set of rules involving random sampling (sweetening).

(3) Repeat the above steps until several T.D's are obtained. This provides a sample of, say, in Terminal Designs.

(4) Identify the least costly of the n Terminal Design as the current estimator of the global optimum (we will call it the "Optimal design' hereafter). Steps (1) to (4) constitute a 'Search'.

(5) Examine the optimal design in detail and ascertain whether it is satisfactory from engineering experience. If so, the search is terminated. Otherwise, either the procedure is repeated from step (1), with a new set of rules for sampling or slight modifications which would satisfy engineering practice are made to the terminal design.

The algorithm described is a practical, heuristic tool for a mathematically complex and computationally laborious problem. It has been successfully applied to practical problems[10].

5 CHOICE OF THE PIPE MATERIAL

The common types of pipes encountered in water distribution are of cast iron, reinforced cement concrete, prestressed reinforced concrete, steel, asbestos cement, rigid polyvinyl chloride (PVC) and galvanized iron. In a given situation, the choice of pipe-material is influenced by several factors including the pressure to be withstood, operating conditions, such as the characteristics of the soil, traffic overload, quality of water to be carried, etc., maximum permissible diameters, the cost and availability of pipes. Special conditions of laying such as crowded thoroughfares, proximity to sewer lines, ground water levels, etc., also can influence the choice of pipe material.

For large diameter transmission mains (greater than 900 mm) subject to high pressures, steel mains have been advantageously used. For a variety of situations in the medium size range, cast iron has been used inspite of the initial heavy investment since it has a fairly long useful life and requires infrequent maintenance. Reinforced or prestressed concrete pipes have high carrying capacity and can maintain the carrying capacity for longer periods of time than pipes subject to tuberculation and consequent restriction of areas. In Tamil Nadu, for sizes up to 600 mm, R.C.C. pipes are generally used: for higher sizes, prestressed concrete pipes are used. Working pressures of 3 Kg/sq.cm. and smaller can be withstood by R.C.C. pipes whereas cast iron and steep pipes can withstand working pressures of as much as 12 kg/sq.cm. In the intermediate pressure range, prestressed concrete pipes may be found more suitable.

It is also common in current practice to use asbestos cement pipes for small diameter submains in the distribution system. They are light and easy to handle and generally corrosion resistant. PVC pipes are flexible, easy to join and are very effective in resisting corrosion. They have also high carrying capacities and can be manufactured to withstand the desired pressures. Their relative cost and non-availability in larger sizes needs to be taken into consideration in selecting such pipes.

In summary, it may be said that the choice of pipe material for a given water distribution situation should be based on a careful evaluation of all factors mentioned, assigning relative weights to them. The following table is provided as a ready reference on the available diameters and pressures for various types of pipes to help in such selection.

Table - Useful Data for Pipe Selection

Material	Class	Test Pressure, m of water	Working pressure* m of water	Average C Value	Available size (D) range, mm	Available lengths, m
Cast Iron	A	120	60		80 - 900	3.6,4,5.5
	A	180	90	100	80 - 900	-do-
	B	240	120		80 - 900	-do-
Steel	1	150	75		200 - 2000	3 to 5
	2	200	100	95	-do-	-do-
	3	250	125		-do-	-do-
Prestressed concrete	1	60	30			2,2.5,3
	2	120	60	120	80 - 1800	
	3	180	90			
RCC	P1	20	13		80 - 600	2,2.5,3
	P2	40	20	120	80 - 600	-do-
	P3	60	30		80 - 450	-do-
AC	5	50	25		80 - 300	3, 4
	10	100	50	130	-do-	-do-
	15	150	75		-do-	-do-
PVC	2.5kg/cm^2	50	25	140	90 - 315	3, 5, 6
	4.0	80	40		50 - 315	-do-
	6.0	120	60		40 - 315	-do-
	10.0	200	100		16 - 125	-do-

*Pumping mains. For gravity mains use two-thirds of test pressure.

6 EPILOGUE

Water distribution and the design of pipe networks are not new to environmental or hydraulic engineers. Over the past four decades, considerable progress has been made in the design and analysis of such systems employing the advances made in quantitative optimization techniques and computer technology. Even complicated systems including a variety of boundary conditions such as multiple reservoirs and pumps and multi-load conditions such as those occasioned by shifting demand pattern in the network can now be handled without resorting to empiricism. Similarly, the range of choice in materials, laying and jointing techniques, and maintenance procedures has widened giving an opportunity for the engineers in charge of the water distribution systems to provide ample returns for the public investment in such systems. With a progressive outlook and willingness to apply tested and proved techniques, he can creditably fulfil his obligations in this direction.

REFERENCES

1 Government of India, Ministry of Works and Housing, CPHEEO, 'Manual on Water Supply and Treatment', New Delhi, 1976.

2 Cross, Hardy, 'Analysis of Flow in Networks of Conduits or Conductors', Bulletin No.286, University of Illinois Engg Expt. Station, Urbana, Illinois, U.S.A., Nov. 1936.

3 McIlroy, M.S., 'Direct Reading Electrical Analyzer for Pipeline Networks', Jour. AWWA, 42, April 1950, p.347.

4 Warga, J., 'Determination of Steady-State Flows and Currents in a Network', Paper No.54-43-4, Proc. of the Instruments,Society of America, 9, part 5, 1954.

5 Martin, D.W. and Peters, G., 'The Application of Newton's Method to Network Analysis by Digital Computer', Jour. Inst. of Water Engineers, 17, 2, March 1963, p.115.

6 Camp, T.R., and Hazen H.L., 'Hydraulic Analysis of Water Distribution by means of an Electric Network Analyser', Journal NEWWA, 48, 383, Dec 1934.

7 Tong, A.L., O'Connor, F.T., Stearns, D.E., and Lynch, W.O., 'Analysis of Distribution Networks by Balancing Equipment Pipe Lengths', Jour. AWWA, 2, Feb 1961, p.192.

8 Raman, V., 'Developments in Water System Network Design', Journal Sanitary Engineering Division, Proc. ASCE, Oct 1970.

9 Pitchai, R., 'A Model for Designing Water Distribution Pipe Networks', Ph.D., Thesis and Technical Report, Harvard University, Cambridge, Mass., U.S.A., 1966.

10 Pitchai, R., 'A Computer-based Design of the Water Distribution Network of Jullundur City, Punjab', Report prepared for the Punjab Water Supply and Sewerage Board, Chandigarh, Oct 1977.

Working Paper No.8

WASTE SURVEYS AND LEAK DETECTION

by

G. BACHMANN*

1 INTRODUCTION

There is no need to explain in detail the importance of controlling wastage of drinking water and making surveys in order to detect leaks in water supply systems. The awareness of the problem has, during recent years, become worldwide and Governments, as well as the water agencies, water companies and municipalities, fully realize the implications that arise from uneconomic water use, loss of water produced and the complex problem of unaccounted-for water.

2 OUTLINE OF CASE STUDY

In the years of 1968-1973 a pre-investment study was carried out under a UNDP/WHO project on the request of the Government of Malta. The objectives of the project related to practically the whole spectrum of environmental health:

(1) Water supply, including review, modernization and augmentation of existing facilities;

(2) Sewage and waste water collection and disposal;

(3) Solid wastes collection and disposal.

Within the water supply component a variety of problems arose, particularly for the reason that natural water resources on the islands (there are two islands - Malta and Gozo) are limited by the existence of a rather flat fresh water lens, restricting the daily and yearly amount of abstraction. A sea-water desalination plant provided some additional drinking water, but it suffered from operational problems.

In view of the insular character of the supply system, careful use had to be made of the existing supply facilities, e.g., water extraction, treatment, storage, pumping and distribution. For more detailed studies a short description of the Malta system is given.

The network in various urban districts including the appurtenant installations which were found to be inaccessible, incomplete, deficient

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in operation, etc., was 60 to 80 years old. For these reasons a flow analysis of the two insular systems had to be made.

It was well known to the authorities that water losses were extremely high, though at different percentages in the various supply districts of between 20 and 50%. Therefore a leak survey, followed by a programme to detect, identify and finally to repair leaks and other defective spots within the network, had to be carried out.

3 PREPARATORY WORK AND INTERIM RESULTS

The analysis of the existing water supply system included the following actions:

	Activity	Information
1.	Completion and checking of survey sheets	Size and siting of mains, sluice valves, master meters, non-return valves, etc.
2.	Site investigation, including consultation with foremen and fitters	Position of sluice valves (open, closed, throttled), disused mains, work in hand, difficult layouts, working condition of meters and specials.
3.	Mastermeter readings	Timing of flow tests (Peak hour), special flow conditions.
4.	Leak detection	Physical condition of pipes and specials.

With reference to activity No.1 as shown above, many water supply systems are not furnished with up-to-date record plans; no accurate "as-built" drawings have been made of the pipe network including all accessories with proper description of technical data. An example of a complete survey sheet is shown in Fig.1 Numerous man/hours by draughtmen, technicians and responsible engineers, had to be spent in order to update these plans at scales of 1:5000 and, in a few areas, 1:2500.

A special examination was necessary to check the existing metering equipment, particularly master meters within the distribution network, as a pre-requisite for the leak detection programme. Again, serious efforts were necessary to locate, make accessible and bring into operational function these meters, particularly as they were required for selective flow and pressure tests.

Another survey was necessary in order to identify heavy consumers, e.g., industrial or commercial (hotels), military and other Government establishments.

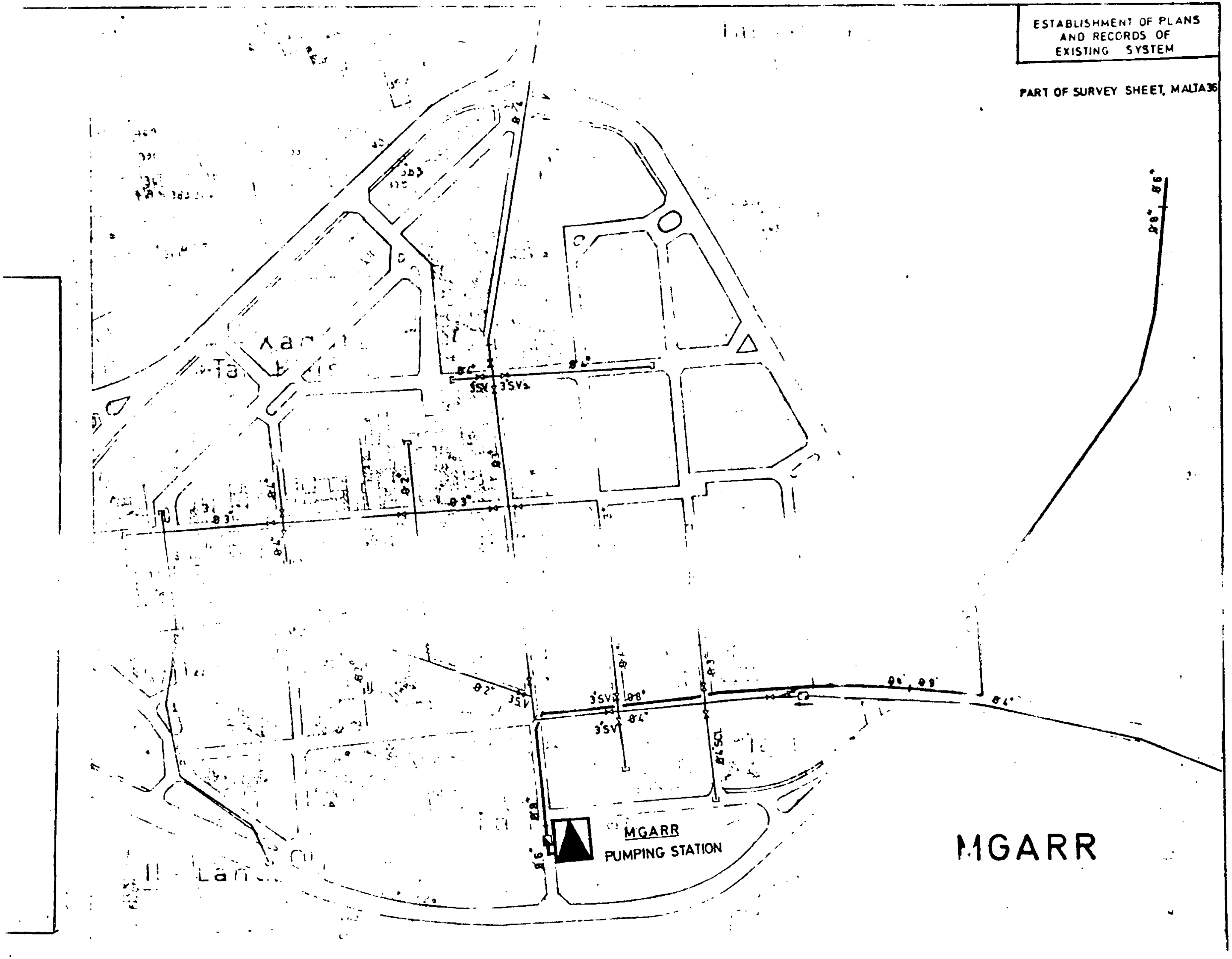
ESTABLISHMENT OF PLANS AND RECORDS OF EXISTING SYSTEM
PART OF SURVEY SHEET, MALTA36
3"SV
3"SV
3"SV
MGARR
PUMPING STATION
MGARR

Figure 1

LEGEND

Pipeline ø 5" and above

Pipeline ø 4" and below

ø 2" — ø in inches for pipes of English standard

ø 100 — ø in mm for pipes of metric standard

Sluice valve or stopcock, ø as pipeline or as indicated

Mastermeter, notch or venturi

Flap valve with direction of flow

Pressure reducing valve

Fire hydrant

Air valve

Reservoir with Name
Capacity: Imp. g. (cu.m)
Overflow level: ft. (m) AOD
Bottom level: ft. (m) AOD

Pumping station

Booster station

Pumping borehole

Gauging borehole

Gallery with shaft

Stone channel

PIPE MATERIALS:

2" and below: Wrought iron

2 1/2" and above: Cast iron, otherwise indicated as follows

AC	Asbestos cement
SI	Spun iron
SCL	Spun iron concrete lined
DBL	Ductile iron bitumen lined

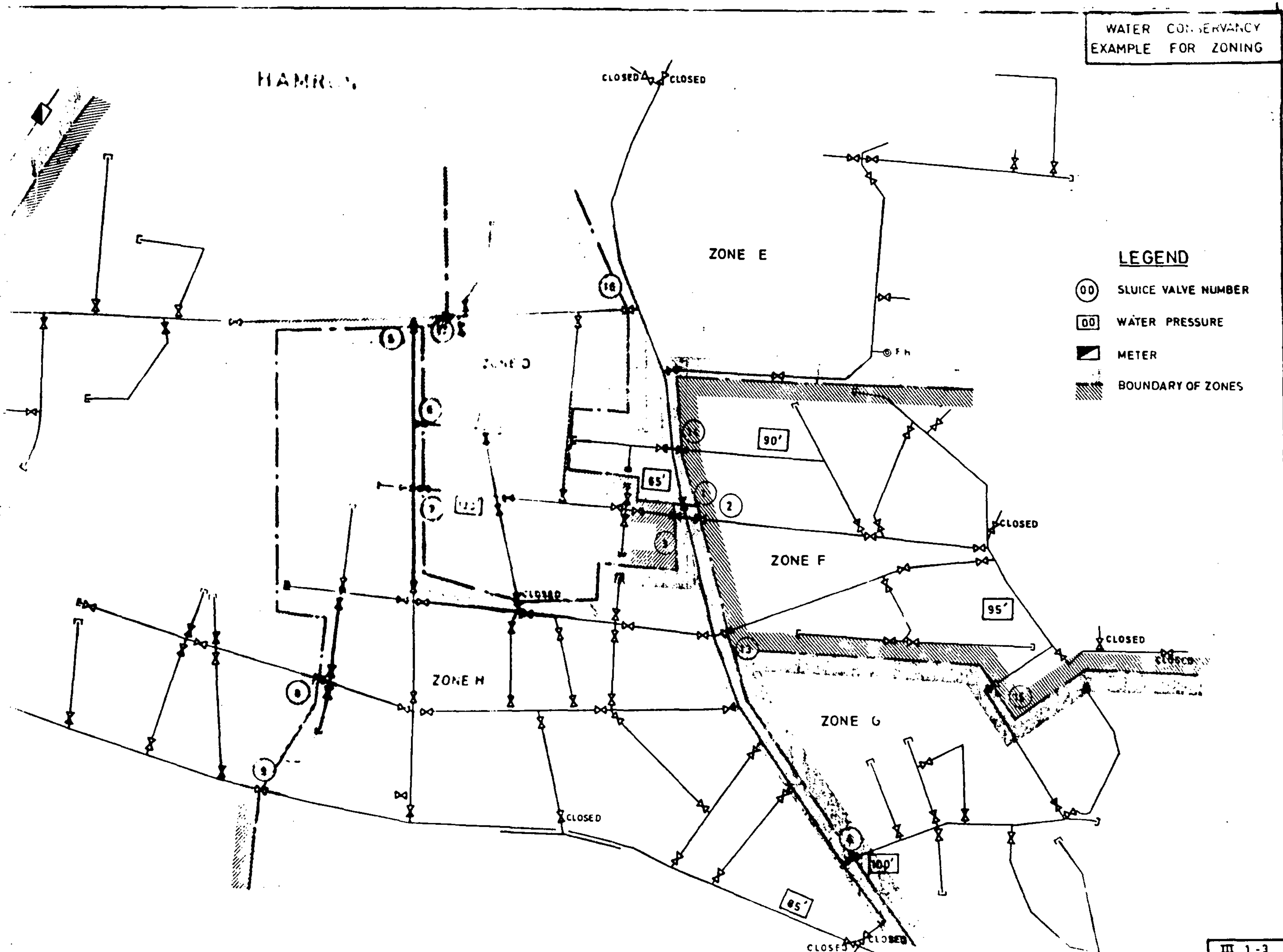
WATER CONSERVANCY
EXAMPLE FOR ZONING
LEGEND
SLUICE VALVE NUMBER
WATER PRESSURE
METER
BOUNDARY OF ZONES
ZONE E
ZONE F
ZONE G
ZONE H
CLOSED
90'
85'
95'
100'
85'
III 1-3

Figure 2

The next step necessary was the division of the network. This was decided according to practical circumstances, such as existence of pressure recorders, geographic figuration of the area and the necessity of distributing the survey over six days of the week so that for each working day a group of distribution areas could be surveyed. It should be mentioned that pressure readings are particularly necessary in order to identify or to re-calculate the roughness factor in the hydraulic calculation. An example for "zoning" the system is shown in Fig.2.

Finally, certain levelling work had to be carried out in order to fix the precise elevation above datum of each pressure recorder and manometer, pumping station, reservoir, etc.

Based on all these preparatory activities, a detailed programme and timetable was worked out, adjusted to the available manpower, transport, equipment, tools and necessary interchanges of water distribution and production.

The evaluation of pressure conditions is a handy instrument to reveal high rates of leakage in sections of the system. Low pressure, for instance, during the measuring hour, indicates deficiencies during the peak consumption hours.

Flow velocities, both to the high and the low extreme, indicate that the pumping operation may be inappropriate. The occurrence of low velocities indicated a tendency to corrosion of pipes.

The re-calculation of the friction co-efficient shows remarkable regional differences, indicating sometimes excessive encrustation of pipes.

4 LEAK DETECTION PROGRAMME

From the survey it was found that, on the island of Malta, only 1700 million gallons (equal to 47%) of the total annual production of 3600 million gallons, were recorded as paid for. An amount of about 11% of the total annual consumption has been estimated to be due to defective metering. In re-calculating total consumption figures, the gross per capita consumptions of the population served (about 310 000) have been broken down as follows:

		Gal/c.d.	L/c.d.	%
Gross per capital consumption of the population served (about 310 000) consisting of:	..	33.8	153	100
Domestic	..	7.6	34	22
Commercial	..	2.1	10	6
Industrial	..	2.2	10	6
Farming	..	0.8	4	2
Government	..	3.0	14	9
Unaccounted for water,losses,etc	..	18.1	81	55

The amount of unaccounted-for water and losses by wastage, etc., is therefore excessively high. The reasons for this have been assumed to relate to:

(1) incorrect production figures;

(2) incorrect metering of service connexions;

(3) estimates instead of readings;

(4) illicit consumption of water not paid for;

(5) consumption by the Waterworks Department;

(6) leakages within the network.

In order to obtain more information on the amount of water losses and the specific reasons, the leak detection programme was designed. As noted before, the reduction of leakages to a reasonable level is a pre-requisite for any meaningful flow analysis of the system.

5 METHODS AND EQUIPMENT FOR LEAK SURVEY

In principle, electro-acoustic leak detection uses the technique of the acoustic method, taking advantage of electronic amplification. By making use of transistors it has been possible to design light and handy equipment with little background noise, which with an increased magnification factor, makes it possible to determine leak sound from a great distance.

Furthermore, the combination of acoustic and optical signs (headphone and instrument) made it easier to locate the defect more accurately, by the simple comparison of readings.

The rough location of a defect was ascertained by means of a test rod, and the exact pin-pointing was effected by means of a ground microphone. Taking into account the normal transmission of sound through the soil, a special microphone was used for this purpose which registers exclusively the optimum range of frequency of a leak. The exact location of a defect is ascertained by moving the ground microphone backwards and forwards and comparing the intensity of the sounds.

If pipe-lines or covered valves or valve boxes must be located, certain other instruments and methods are available.

The team was equipped with the following instruments:

- electronic pipe locator with transmitter, receiver, search coil, earphone, etc.;

- electro-acoustic water leak detector with receiver, microphones, earphones, geophones, etc.;

- magnetic valve box locator;

- electronic metal and box locator.

6 EXECUTION OF LEAK DETECTION

The leak detection had to be carried out by a special team brought into the country. The Waterworks Department assigned several staff to the special team in order to make it familiar with the local conditions. Vice versa, this water works staff was trained in practical leak detection work as the job was carried out.

In order to facilitate local work only day-shifts were carried out in the first phase. Later, according to the programme, the detection work was carried out on a night-shift basis, especially in built-up areas.

The team used special forms to report leaks detected or other observations of water losses. At the end of each shift, the Waterworks Department was informed of the location and suspected nature of the leaks in order to initiate action for repairs.

Most of the leaks reported were repaired immediately by the gangs from the Waterworks Department Districts concerned. Where many leaks were found in one district, the leak detection team withdrew and left for another district, so that the gang would repair all these leaks before the survey was continued by the team. Certain pipelines in Malta were rechecked before the leak detection survey was completed.

While searching for leaks in the distribution pipelines, most of the house service connexions were checked too. In 1968 the total number of service connexions in Malta and Gozo was approximately 100 000.

The original team covered an average of between 6 and 7 km of pipelines, per shift of 8 hours.

7 TRAINING OF PERSONNEL IN LEAK DETECTION

Upon selection of Waterworks Department personnel to be trained, a leak survey specialist gave lessons and practical instruction in leak detection and pipe location. Following this, the trainees were taken in turn to join the leak detection teams during their shifts.

By the end of the leak detection programme, two local teams were in a position to continue leak detection as a matter of routine. They were called upon when, in the course of the leak survey, new leaks became apparent.

8 RESULTS AND CONCLUSIONS

The leak detection reports were summarized and evaluated. Wherever uncovered leaks were inspected by the project staff, the outflowing water was measured or estimated.

The total leaks suspected numbered 253 in Malta and Gozo, while 248 leaks were actually discovered. The 253 suspected leaks included 11 cases where misinterpretation of noises such as filling of water tanks during the night or night irrigation gave rise to erroneous reports. The same figure (253) also included 6 cases where a single leak was reported, while 2 leaks of varying nature, very close to each other, were discovered and repaired.

Included in the total of leakages are 3 cases of water thefts which were reported to the Waterworks Department for further action.

Where the project staff inspected uncovered leaks on trunk mains, broken pipes were responsible for the loss of water in most cases, while the water loss on sluice valves was caused mainly by defective packing. It was observed that, in many cases the pipes were surrounded by stones only, without any beeding of fine grained fill material. It is obvious that such conditions cause dangerous stresses on the pipes, especially under heavy traffic conditions.

The analysis of the leaks inspected on service connexions shows that, besides those leaks on the service pipes themselves, major leaks were observed on the tapping ferrules which connect the service pipes to the main.

The weak point of the tapping ferrules is apparently the washer. In most cases inspected by the project staff, leaking washer was the reason for the water loss. Some very heavy leaks were caused by loose tapping ferrules on old service connexions or by improper sealing of service connexions which were no longer in use.

As was discovered on trunk mains, service pipes were damaged by mechanical action caused by the improper bedding used for those pipes.

Only a very small number of water losses were caused by badly tightened stop-cocks and defective meters.

According to calculations, more than 50% of the water loss was caused by leakages in trunk mains although the number of such leakages amounted to only 22% of all water leaks detected, excluding water thefts.

Estimated total water loss from discovered leaks in 1000 gallon/day:

	Service Connexion	Trunk Mains	Sluice Valves	Total
Malta	628	920	12	1 560
Gozo	15	0.4	-	15.4
Total	643	920.4	12	1 575.4
% of Total	41	58	1	100
Total number of leaks detected in Malta and Gozo	160	54	31	245
% of Total	65	22	13	100

These results show that leaks of about 1.6 million gallons out of an average production of 9.1 million gallons per day have been detected which is 17% of the total average daily production.

It shows that there was still 36% of unaccounted-for water.

The production curve of the individual months in 1967 and 1968 shows that there was a considerable decrease in consumption in Malta for the month of April 1968, which was the first month of the intensive leak detection.

9 GENERAL REMARKS ON LEAK SURVEY

The prevailing system of leak detection achieved considerable success in reducing water waste. It was not, however, and could never be completely effective. In the absence of a comprehensive metering system the percentage of leakage and the areas requiring leak detection in preference to other areas were unknown. Zones of supply were inspected by the leakage gangs on a roster basis since the leakage rate in the zones, the amount of water saved by the repair of the detected leakages, and the effectiveness of leak inspection were unknown.

If leak detection was to become more effective and if a measure was to be obtained of its performance it had to be combined with a comre-hensive metering system. From the minimum night flow the leakage rate in the area could be assessed and from the weekly meter readings, it could be established whether the supply to the area had remained stable or whether new leakages had developed. By judicious use of the meter records, leak detection could be concentrated in areas where it was most likely to prove most beneficial.

10 DETAILS ON LEAK SURVEYS

Before systematic leak detection combined with a comprehensive metering systems control could commence, a pilot test was performed in three different supply areas. Each of these areas was such that the number of service connexions, as well as the consumption per service during one week, could be ascertained. Master meter readings established the daily flow and flow recorders registered the minimum night flow. Hence, the probable magnitude of leakage in the area could be assessed. night-shift inspections, in combination with location and repairs of leakages, revealed the differences in flow as a result of leak repairs.

The main results of the pilot tests can be summarized as follows:

The functioning service meters in one area registered a total flow of 352 m^3 in one week, and if, proportionately, it is assumed that 26 m^3 of water passed through the non-functioning meters, the total water consumed in one week was 378 m^3. The flow registered by the master meter during the same period was 673 m^3 and the minimum night flow of 1.8 m^3/h. Assuming that this night flow is due to leakages, the leakage rate can be then estimated at 305 m^3 per week, which is equivalent to 45.3% of total flow.

Following leak detection tests, night soundings, location and repairs of leaks, the daily supply to the area decreased to an average 54.5 m^3/d while the minimum night flow was reduced to practically zero, i.e., all the leakages in the area were eliminated and the master meter reading tallied with the total of the service meters.

In the other 2 pilot areas, the ratio between loss before and after repairs, found to be:

	Pilot area 2	Pilot area 3
Loss before repair	27.7%	60%
Loss after repair	10.3%	34%

As a summary combining the results from the three pilot areas, the average amount of leakage was 50% before the repairs and 24% after the repairs.

11 CONCLUSIONS

If the above results were taken to be indicative of the position for the whole of Malta, then out of an average daily production of about 9.5 mgd (in 1969), 4.5 mgd was being lost through leakages. If, as a result of intensive systematic leak detection and correction, the leakage rate could be reduced by 20%, savings of 1.9 mgd would be made.

Parallel to the reduction of leakages, the prevention of water misuse had to be taken into consideration also, and the following measures were suggested accordingly:

(1) Service meters installed where high consumption is likely, e.g., factories, farmhouses, hotels, houses with gardens, etc., should be given priority in changing when they are out of order.

(2) Meters for high consumption water services should be of the correct size for the flow they are intended to register.

(3) Meters for consumers should always be installed without a bypass. Experience has shown how difficult it is to keep a proper check on these by-passes especially if there are many.

(4) All service meters should be checked on a routine basis, at least every 5 years. The card index system, which has already been introduced, should be evaluated in this way and more personnel will, be needed to perform this task.

(5) The piping material used for service connexions should be changed to PE or PVC.

12 PROGRAMME FOR COMPREHENSIVE LEAK SURVEY

The situation may be different from system to system as regards the practicability of leak surveys. In Malta, the three pilot test exercises gave a sound platform for a comprehensive leak survey. A full programme has been established using all observations from the preliminary surveys.

It is not necessary to explain the procedure for the full, country-wide survey in Malta. However, as an example, the Annex describes the programme as it was developed, adopted and extended. The example also contains a summary of costs involved, which of course should be used with caution and would require up-dating since the costing dates back to 1968/1970. Still for comparative purposes, it might be useful as an example.

13 IMPLEMENTATION OF THE PROGRAMME

The comprehensive leak detection programme was carried out according to the schedule, although serious delays occurred due to unavailability of master meters as required, including an additional time lag for their proper installation.

The results of the leak detection activities have been compiled in a list as shown in Fig.3 and are indicative of the situation in the particular supply areas. In the first approach, 36 different areas were metered with an average loss rate of 47%, which by systematic leak detection, could be reduced to 33%. Note that recurrent leakages raised this percentage again to 36%, i.e., the net gain at the end of the exercise was 11% which was equivalent to 2800 cu m per day (612 000 gallons per day).

The subsequent detection exercises were of variable effectiveness and speed of action.

As a general observation the subsequent detection work which was supposed to be working under a routine schedule by the Waterworks Department staff, fell slightly short of expectations for various reasons. In re-evaluating the results, several factors would appear to compensate for and explain the shortcomings. These include:

- Much leak detection work was carried out in unmetered areas which is not included in the cost analysis;

- Heavier expenditure was incurred in the first year than in succeeding years due to the considerable number of new master meters which had to be installed;

- Greater savings in water leakage were to be registered in succeeding years than in the first year. This would be even greater if the service connexions had been changed to plastic pipes.

14 CONTINUATION OF THE PROGRAMME

As the metering system was to be extended over a larger area the problem of recurrent leakages would develop, as the aim was not just

Figure 3

Area No.	AREA	AS ORIGINALLY FOUND (A)				LOWEST ACHIEVED RESULT (B)					AS EXISTING ON 31.12.70 (C)					Area No.
		Date	Total Flow gls/day	Estimated Leak gls/day	% Leakage	Date	Total Flow gls/day	Estimated Leak gls/day	% Leakage	Saving gls/day	Date	Total Flow gls/day	Estimated Leak gls/day	% Leakage	Saving gls/day	
1	Floriana	3/70	118 100	48 000	41	3/70	25 090	28 800	30	19 200	22/12/70	106 600	43 200	41	4 800	1
7	Marsaskala	28/1/70	50 900	36 000	71	6/3/70	36 600	19 200	53	16 800	4/10/70	43 000	24 000	56	12 000	7
25	Mellieha		93 800	38 400	41	8/6/70	104 940	nil	0	38 400	20/8/70	135 080	nil	0	38 400	25
31	Marsaxlokk		19 500	8 400	43	31/8/70	21 100	2 400	11	6 000	26/11/70	14 100	nil	0	8 400	31
51	Msierah/Sliema		200 000	120 000	60		122 000	42 000	34	78 000	25/1/71	127 600	48 000	38	72 000	51
32	Kirkop		21 400	9 600	45	24/6/70	12 800	nil	0	9 600	26/1/71	10 700	nil	0	9 600	32
10	Marsa/Valletta		84 000	23 000	28		70 000	7 000	10	16 000		70 000	7 000	10	16 000	10
45	Mosta/Main Str.	25/3/70	69 000	12 000	17	10/7/70	78 200	12 000	15	nil	10/7/70	78 200	12 000	15	-	45
42	Mosta/Fort Str.	25/2/70	26 500	4 800	18	21/7/70	36 000	3 120	9	1 680	21/7/70	36 000	3 120	9	1 680	42
43	Mosta/Hope Str. "A"	6/70	6 000	-	-	-	-	-	-	-	-	-	-	-	-	43
43	Mosta/Hope Str. "B"	6/70	7 000	-	-	12/70	4 400	-	-	-	-	-	-	-	-	43
44	Mosta/Speranza	7/70	30 000	-	-	-	-	-	-	-	-	-	-	-	-	44
77	Msida/Ta'Xbiex	8/6/70	201 740	105 600	52	22/6/70	177 980	95 040	53	10 560	2/11/70	190 080	105 600	56	-	77
54	Qrendi	4/70	50 000	10 000	20	1/7/70	31 400	2 400	8	7 600	1/7/70	31 400	2 400	8	7 600	54
16	Tal-Virtu	6/70	28 210	nil	0	-	-	-	-	-	25/10/70	27 400	2 400	9	2 400	16
17	Siggiewi	25/3/70	99 900	52 800	52	15/1/71	70 000	24 000	34	28 800	15/1/71	70 000	24 000	34	28 800	17
8	Zejtun	28/3/70	161 000	91 200	56	2/6/70	156 000	72 000	46	19 200	16/11/70	160 000	72 000	45	19 200	8
6	Kalkara	10/6/70	57 100	33 600	59	9/7/70	32 000	240	1	33 360	9/7/70	32 000	240	1	33 360	6
57	Safi	25/6/70	10 100	nil	0	12/9/70	17 100	nil	0	-	18/1/71	7 600	nil	0	-	57
72	St.Andrews/Swieqi	5/6/70	27 830	nil	0	2/10/70	16 720	nil	0	-	2/10/70	16 720	nil	0	-	72
38	Ganu Street	2/7/70	4 080	nil	0	25/9/70	4 200	nil	0	-	25/9/70	4 200	nil	0	-	38
34	Lija	13/3/70	19 900	9 600	48	7/4/70	22 900	7 200	31	2 400	24/1/71	27 100	10 800	40	1 200	34
93	Malta Drydocks	21/6/70	239 580	132 000	55	25/7/70	154 440	79 200	51	52 800	25/7/70	154 440	79 200	51	60 000	93
13	Qormi Roundabout	20/7/70	63 700	38 400	60	10/8/70	27 560	2 160	8	36 240	10/8/70	27 560	2 160	8	36 240	13
11	Qormi Garden Str.	18/7/70	246 400	100 320	41	24/8/70	211 860	58 080	27	42 240	24/8/70	211 860	58 080	27	42 240	11
100	Paceville	14/7/70	298 100	121 440	41	6/8/70	310 200	121 440	39	-	6/8/70	310 200	121 440	39	-	100
5	Vittoriosa	22/7/70	100 500	55 200	55	10/12/70	44 500	1 200	3	54 000	10/12/70	44 500	1 200	3	54 000	5
29	Bidnija	1/9/70	6 750	1 920	28	17/9/70	6 170	1 920	31	-	7/9/70	6 170	1 920	31	-	29
67	Dingli	-	-	-	-	-	-	-	-	-	-	-	-	-	-	67
85	Delimara	1/7/70	16 000	4 800	30	3/10/70	12 440	nil	0	4 800	3/10/70	12 440	nil	0	4 800	85
19	Zebbug	13/7/70	261 500	148 800	57	7/8/70	245 000	108 000	44	40 800	7/8/70	245 000	108 000	44	40 800	19
59	Gudja	9/11/70	29 500	10 800	37	11/1/71	18 700	nil	0	10 800	11/1/71	18 700	nil	0	10 800	59
78	Gzira	11/9/70	474 900	240 000	47	26/11/70	407 000	192 000	51	48 000	26/11/70	407 000	192 000	51	48 000	78
58	Mqabba	2/12/70	38 800	nil	0	2/12/70	38 800	nil	0	-	2/12/70	38 800	nil	0	-	58
23	Mdina	25/10/70	9 450	nil	0	25/10/70	9 450	nil	0	-	25/10/70	9 450	nil	0	-	23
2	Valletta	6/11/70	438 000	240 000	55	7/1/71	326 000	168 000	51	72 000	7/1/71	326 000	168 000	51	72 000	2
			3 609 240 (16 406 m^3/d)	1 696 680 (7 712 m^3/d)	47		2 921 550 (13 280 m^3/d)	1 047 400 (4 761 m^3/d)	36	649 280 (2 951 m^3/d)		2 999 900 (13 636 m^3/d)	1 086 760 (4 940 m^3/d)	36	617 120 (2 805 m^3/d)	

to bring down, but also to maintain a low leakage rate in the distribution system. During the first year a gross saving of 3860 m^3/day (850 000 gal/day) was effected, but this was reduced by recurrent leakages totalling 1090 m^3/d (240 000 gal/day).

The magnitude of recurrent leakages is bound to rise as the metered area is extended further and in the future more effort will have to be spent in maintaining a low leakage rate, i.e., in overcoming recurrent leakages, than in bringing down the leakage rate. It would be interesting to know the rate of recurrent leakages over the whole distribution system so that an assessment could be made of the personnel required to keep it under control. Results obtained during the first year, however, have not been considered sufficiently conclusive to provide this information. The periodic testing of master meters is suggested as another measure to control leakages.

15 ASSESSMENT OF LOSSES BY LEAKAGES

The evaluation of the leak detection exercises shows some interesting figures. For instance, the leaks occurred at different places, mainly however, within house connexions, but also in sluice valves and water mains as well as tapping ferrules.

By repairing the defective spots, average savings per repair could be obtained, of between 7.5 m^3/day and 19.7 m^3/day. The summary of occurrence of leaks was as follows:

119 leaks in house service connexions	56%
23 leaks in sluice valves	11%
31 leaks in mains	14%
40 leaks in tapping ferrules	19%
213 leaks in total	

The repairs of these leaks saved 3290 m^3/day (714 000 gal/d). Extending the findings to the total water production of 46 000 m^3/day (10 250 000 gal/d), the 47% of leakage is equivalent to 21 900 m^3/d (4 820 000 gal/d). This was estimated to consist of:

$$\frac{119 \times 21\,900}{3290} = 804 \text{ leaks in house services}$$

$$\frac{23 \times 21\,900}{3290} = 155 \text{ leaks in sluice valves}$$

$$\frac{23 \times 21\,900}{3290} = 209 \text{ leaks in mains}$$

$$\frac{40 \times 21\,900}{3290} = 270 \text{ leaks in tapping ferrules}$$

i.e., a leakage of 21 900 m^3/day (4 820 000 gal/day) may have been caused by 1438 or say 1500 leaks.

There were 107 000 metered services in Malta, and there could be assumed to be 1074 leaks in the metered service connexions if leaks occurring in the tapping ferrules were also included. In other words 1 in 100 of the metered services were leaking.

1319 leaks were detected by the Waterworks Department in one year and repaired, i.e., the output of the Waterworks Department in one year was equal to the number of leaks existing at one time in all the island. If for every leak repaired, another leak were to recur after one year, the performance by the Waterworks Department was just sufficient to keep the existing leakage rate constant.

The above assumptions showed that if the leakage rate was to be reduced the present output by the Waterworks Department (i.e., the number of detected leaks per year) would have to be about doubled. This could be achieved either by:

(1) increasing personnel;

(2) more intensive efforts by the personnel concerned;

(3) systematic leak detection.

Annex

EXTRACT FROM REPORT ON "UNDP/WHO WASTES DISPOSAL AND WATER SUPPLY PROJECT, MALTA" PROPOSED PROGRAMME FOR COMPREHENSIVE LEAK DETECTION

Objectives of the Programme

The section of the Water Works Department charged with the implementation of this programme had to accomplish two main objectives:

- Installation of master meters

- Systematic leak surveys

Installation of master meters

The first objective "Installation of Master Meters" involves the following activities:

A. Select supervisory and clerical staff. These personnel will also be engaged on the second objective. The staff will consist of the following:

(i) Engineer Assistant i/c of Section

(ii) Foreman to help and substitute Engineer Assistant

(iii) One assistant to read master meters and install flow recorders

(iv) One assistant to read high consumption meters

(v) Four assistants to help Engineer Assistant and Foreman in preparing for and conducting leak detection tests

(vi) One clerk typist for keeping records

(vii) Four cars with drivers.

B. Select personnel for construction gang, meter installation gang, pre-cast concrete gang and night sounding gangs. These gangs will be formed as follows:

(i) Construction gang: mason, stone dresser, skilled labourer, truck with driver. This gang will be responsible for constructing the meter chamber.

(ii) Meter installation gang: pipe fitter, 2 miners, labourer, truck with driver. This gang will be responsible for installing the meter complete with sluice valves and by-pass.

(iii) Pre-cast concrete gang: skilled labourer and labourer. This gang will be responsible for preparing all pre-cast concrete slabs required for the meter chambers.

(iv) Night sounding gangs: there will be three night sounding gangs each equipped with leak detection instrument. Each gang will be formed of three men, and one van with driver will be with the three gangs. The need for the three gangs will be explained later.

C. Prepare store for meters, sluice valves, pipes and specials, tools, concrete aggregate and building stone etc.

D. Prepare moulds for pre-cast concrete slabs.

E. Prepare programme of meter installations in liaison with District Foreman taking into consideration substitution and availability of meters.

F. Provision of stone, aggregate, sand, cement and reinforcement steel.

G. Provision of meters, dirt boxes, special castings, meter covers, sluice valves, etc.

H. Prepare tenders for the provision of new meters, dirt boxes, meter covers, etc., which are not available from General Store.

I. Issue of tender for the above material.

J. Selection of tender and placing of order.

K. Receipt of material from abroad.

L. Installation of meters and construction of meter chambers. 100 meters are estimated to be required. Twenty meter installations will be completed in 11 weeks.

M. Completion of 40 meter installations.

N. Completion of 60 meter installations.

O. Completion of 80 meter installations.

P. Completion of 100 meter installations.

Systematic leak surveys

The second objective "Systematic Leak Survey" involves the following activities:

A. Select supervisory and clerical staff. Similar to 'A' of the first objective.

Q. Dividing distribution system into metered zones estimated to total about 100 eventually.

R. The testing of each zone involves the following preparations and procedures:

(i) Installation of meter registering all supply to the zone

(ii) Install flow recorder and assess total leakage from minimum night flow; check with Sewerage Section where water was infiltrating into sewers

(iii) Assess consumption of the zone from service meter readings exclusive of leakage and check with flow chart readings

(iv) Select and test sluice valves which will have to be closed during the test and prepare plan of distribution system of the zone

(v) Conduct leak detection test

(vi) Night sounding to locate exact position of leaks

(vii) Repairs of leakages

(viii) Install flow recorder to observe effect of the leakages repaired

(ix) Repeat leak test or night sounding until leakage reduced to a satisfactory low level.

It is anticipated that the Engineer Assistant and his foreman, working independently will each be able to complete one zone per week. Therefore 100 zones will be completed in about 60 weeks, allowing for some extra time, and the first twenty zones in 12 weeks.

As each zone is completed its condition is monitored by weekly readings of the master meter. If leakages recur in zones which are already completed a third group carries out leak detection tests as explained above, so that the normal progress of completing two new zones per week will not be disturbed. The three night sounding gangs were proposed with this in mind, i.e., two gangs will be continuously engaged on new zones while the third gang will be mostly engaged on completed zones.

S. Completion of 40 zones

T. Completion of 60 zones

U. Completion of 80 zones

V. Completion of 100 zones

According to the network plan in App.III.1-4 this target will be reached within a period of 60 weeks. Over the whole period therefore there will be an average saving of about 8600 m^3/d resulting from reduction of leaks from 36% to 15%.

Costs involved

The following expenditure will be incurred, the capital cost of which is included in the investment cost of the existing system for the first phase (Section VI.6), and the personnel cost in the total running cost of the existing system. The calculation shows the rate of the water saved by leak survey and the cost of water lost thus represents an additional cost saving. However the main benefit arises when the additional expenditure on water conservancy is compared with the cost of producing new water.

				£
(a) Investment cost				£
	(i)	Cost of supplies:		
		(1)	100 sets of meters and dirt boxes of all sizes @£40	4 000
		(2)	100 sets of meter fittings, each comprising 3 sluice valves, 2 angle branches, 2 bends, 2 flanged ends, 1 flange adaptor, 1 meter cover etc. @£10	1 000
		(3)	Material for 100 meter chambers, stones, concrete, reinforcing steel for slabs @£20	2 000
		(4)	Electronic equipment, l.s.	5 000
			Total supplies	12 000
	(ii)	Costs of civil works:		
		(1)	Personnel: 5 men at £2 per day for 700 men-days	7 000
		(2)	Trucks and cars with drivers, at £8 per day, for 350 days	2 800
			Total civil works	9 800
	(iii)	Total capital works		21 800
	(iv)	Material depreciated over 7 years at 8% interest = 19.2% of 15 900	=	3 100/a
	(v)	Civil works depreciated over 20 years: 10.2% of 5900	=	600/a

(b)	Maintenance cost	£
	- 3% of investment cost	600/a
	- Repair of leaking pipes	4 000/a
(c)	Personnel cost	
	- 27 men for leak survey @ £2 per day	19 700/a
(d)	Total cost	28 000/a

A saving of about 8600 m^3/d = 3.14×10^6 m^3/d (690 mga) will be obtained at a cost of 0.007 £/m^3 = 6.4 d/1000 gal. = 22 000/a

Working Paper No.9

A PLAN FOR PREVENTIVE MAINTENANCE OF RURAL PIPED WATER SUPPLY SYSTEMS

by

M.A. ACHESON*

1 INTRODUCTION

There is general recognition of the need for routine servicing of installations and equipment in water sypply as in other engineering systems. There is also clear evidence that the economic benefits of preventive maintenance, as opposed to repair and replacement on breakdown, are considerable. Nevertheless it is all too often found that, while lip-service is paid to preventive maintenance, practical planning is neglected and the advantages of optimizing the work of a team to service several schemes are overlooked in favour of an *ad hoc* approach to each individual system, no matter how minor.

The case study presented here using a real situation but with disguised names and slightly simplified installations, is based on a proposal made by a WHO short-term consultant, Mr A. Jacome, in 1974. It must be admitted that the plan was never implemented for a number of reasons not connected with its merits or demerits. However, the logic and simplicity of the proposal remain valid and the purpose of presenting here in some detail, a plan for ensuring the preventive maintenance of fifteen piped rural water supply systems is to show how, with advance planning, a feasible programme can be designed.

The paper is therefore divided into two sections. The first deals with the approach to planning, the steps involved and the types of form which can be used. Because of the fact that it is often not very meaningful to study blank forms, the second, and longer, section of this document deals with the case study already referred to.

In order to evolve a plan to fit a particular set of circumstances, several steps require to be taken. The objective in this case is to prepare a programme so that a team located at a central point (regional or district) is able to cope with the preventive maintenance of a number of water supply systems, the alternative being that each system should operate its maintenance schedule independently. It is not proposed to dwell on the advantages, economic or other, of centralized services or standardization. To some extent these are taken for granted. What is attempted is to propose, given a certain number of systems which might correspond to those existing in a particular administrative unit, how a programme may be organized.

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The steps to be undertaken are as follows:

(1) Technical Inventory: This involves the recording of all technical data with regard to each scheme. An individual form is proposed for each item (see Form No.1), particularly relevant for items such as pumps, motors and chlorinators.

(2) Compilation and classification: This is now carried out for the systems and tabulated on Form No.2, according to groups, which are allocated letters (A, B, C, ... etc.).

(3) Identification: Individual items of equipment may then be identified and allocated numbers within their particular groups (e.g., A4, H5, N3). Form No.3 can be used.

(4) Establishment of rules: This involves the determination for each group of the type and frequency of maintenance required, the composition of the service team, and the time that it will take. Rules must be drawn up as to what service must be carried out at weekly, monthly, 3-monthly, 6-monthly and annual intervals in order to satisfy the requirements for preventive maintenance. These rules can be recorded on a table such as Form No.4.

(5) Reporting: Arrangements must be made for records to be kept of visits, tasks performed, spares and supplies consumed etc. Form No.5 may be used.

(6) Preparation of Plan: Based on the data recorded on Forms 2, 3 and 4, the plan of visits must be devised taking into account the location of the schemes, the travel time required, the tasks to be performed, the composition of the teams and the time factors involved. The plan must allow for the regular visiting of the team to the systems at the required intervals of time and generally in the same order. A form for recording the plan is given as Form No.6

Samples of these six blank forms which will be described further in dealing with the case study, are attached, and constitute Annex 1.

2 CASE STUDY

Form 2 in Annex 2 is completed to show the installation and equipment in the 15 piped rural water supply systems included in the scheme, including type of source, pump and motor, chlorination, storage and distribution. The design population served is not given but ranges from 1000 to 3800 with an average of about 2100. Identification numbers are then allocated on Form 3. Form 4 is completed to show the rules adopted, and the planned programme is given on Form 6. These completed forms constitute Annex 2.

As general assumptions, the following should be noted:

(1) the fifteen schemes vary in size and complexity, but none is large enough to warrant a full-time staff other than an operator with basic training, who can report directly to a higher level in the case of any malfunction.

(2) The fifteen schemes are the total number of such schemes existing in the administrative area considered. If additional schemes were to be added, the plan would obviously have to be amended to include them. There may, of course, be larger manicipal schemes in the same district being operated independently under the same water supply authority.

(3) Distances between schemes are not excessive, do not require more than one or two hours road travel, and communications are good.

(4) The size of the team is undefined and would have to relate to local conditions. In this case, it is assumed that the team would be basically a maintenance technician with a helper, driving their own transport (Jeep-type), supplemented by specialist personnel as shown in the rules.

(5) This team is under the supervision of an engineer (or senior inspector) who is present at some time during each annual inspection to a scheme as a minimum requirement, i.e., for at least one or two days per month.

(6) A specialist pump and motor mechanic will supplement the team on 3-monthly visits to other than gravity schemes. This will give him a variable involvement, discussed later.

(7) Daily and weekly preventive maintenance, while identified by the rules, is carried out by the operator alone and is not covered by the plan prepared for the regional maintenance team.

(8) Monthly preventive maintenance is carried out in one working day, three and six monthly service in two days, and annual maintenance in three days. Exceptions have been made in regard to three-monthly service of stream intake galleries where only one day was found to be required, and at village Alpha, which is supplied from a direct pipe connexion to an existing main, only two days are required for the annual maintenance visit.

(9) Where additional labour is required, it is assumed to be engaged locally on a daily basis.

(10) A special team is called in for annual service of bored-well systems (only one in this group).

(11) Maintenance and repair of water meters is carried out under a separate programme involving a regional workshop.

(12) The programme for preventive maintenance is worked out on a basis of 20 working days in the field per month, allowing flexibility for urgent repair work, holidays, etc.

3 DISCUSSION

It is hoped that the methodology will be clear to anyone working through Annex 2. It is evident that a mobile team such as that described can, with careful advance planning, reasonably take care of the preventive maintenance of a group of fifteen rural piped water supply schemes of the size and complexity of those considered here.

The following observations may be made:

(1) In practice, the tasks to be performed may be many more than those enumerated here which are given more or less as examples. They should in fact be elaborated in as much detail as possible in an actual situation.

(2) The plan as given shows the team working for 19 days in four months of the year, 20 days in five months and 21 days in three months. This flexibility should be feasible.

(3) Although the plan shows the visits made each month in exactly the same order, in practice it may be necessary to modify this to accommodate visits of more than one day depending on week-ends and holidays.

(4) Geographical and logistical considerations will dictate to what extent the team returns to its base during a cycle of visits, and no guidance can be given on this aspect.

(5) In the proposed plan, it is noted that the specialist pump mechanic will spend from 0 to 9 days in any month with the regional team, with a median number of 7 working days.

It is never possible, when describing a proposal rather than reporting results, to escape a degree of artificiality, and it is probable that in practice several adjustments would have to be made to ensure workability. Nevertheless, it is felt that this study has demonstrated a method of planning which may be adapted to fit a variety of similar schemes.

Annex 1

Form No.1

PROJECT IDENTIFICATION

WATER SUPPLY SYSTEMS - TECHNICAL INVENTORY

DISTRICT________________ LOCAL AUTHORITY________________

SCHEME________________

NAME OF THE INSTALLATION OR EQUIPMENT________________

LOCATION________________

____

IDENTIFICATION NUMBER________________

POSTAL ADDRESS________________

____

NAME PLATE & ADDITIONAL DATA

	Attached	Not Available	
Information from the manufacturer			NOTE: If the information from the Manufacturer and/or the Manuals are not available they should be requested.
Description & Assembly Manual			
Operational Manual			
Maintenance Manual			
Spare Parts List			

Form No. 2

PROJECT IDENTIFICATION

COMPILATION & CLASSIFICATION OF INSTALLATIONS & EQUIPMENT

Type of installation of equipment → / ↓ Location																									
GROUP →																									

Form No.3

PROJECT IDENTIFICATION

IDENTIFICATION NUMBERS FOR INSTALLATION & EQUIPMENT

Type of installation of equipment → / ↓ Location																									
GROUP →																									

Form No.4

PROJECT IDENTIFICATION

PREVENTIVE MAINTENANCE RULES FOR INSTALLATIONS & EQUIPMENT

Group	Description	Maintenance period	Responsibility		Task to be performed	Materials, Spare Parts, Lubricants required
			Level	Crew		

Form No.5

PROJECT IDENTIFICATION			
REPORT FORM FOR PREVENTIVE MAINTENANCE OR REPAIR			
DISTRICT________	LOCAL AUTHORITY________		
SCHEME________			
Date	Identification Number	Tasks Performed	Materials, Spare parts, Lubricants used

Comments:

Form No.6

PROJECT IDENTIFICATION														
PREVENTIVE MAINTENANCE CALENDAR FOR INSTALLATIONS & EQUIPMENT														
MONTH:			MONTH:			MONTH:			MONTH:			MONTH:		
Working* Day No.	Location	Task to be performed	Working* Day No.	Location	Task to be performed	Working* Day No.	Location	Task to be performed	Working* Day No.	Location	Task to be performed	Working* Day No.	Location	Task to be performed

*It is assumed that there are 20 working days per month. Actual dates should be used when known.

Form No.2

PROJECT IDENTIFICATION

COMPILATION & CLASSIFICATION OF INSTALLATIONS & EQUIPMENT

Type of installations or equipment → / ↓ Location	INTAKES: Well: Hand Dug: Plain	INTAKES: Well: Hand Dug: With filter	INTAKES: Well: Bored	INTAKES: Stream: Perforated pipe	INTAKES: Stream: Direct	INTAKES: Spring	INTAKES: Pipe connexion to main	Pump house	Centr. pumps H.S.	Diesel engines	Elect. Motors	CHLORI-NATION: Gravity feeder	CHLORI-NATION: Test kit	TANKS: Storage	TANKS: Break pressure	Stand posts	MAIN & DISTRIBUTION SYSTEM PIPELINES: Pipe length in meters (Dia. mm): 150	100	75	63	50	38	25	WATER METERS: Main	WATER METERS: Domestic
GROUP →	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q							R	S
ALPHA							1							1		15		4501	1275	351					124
BETA			1					1	2	2		1	1	1		9					4210	1067	381	1	80
GAMMA	2							1	2	2		1	1	1		13			311	1088	253	850		1	13
DELTA	3							1	2	2		3	2			8					3752	680	259	1	8
EPSILON	1							1	2	2		1	1	1		11			1362		788	930	91	1	11
ZETA		1						1	2	2		1	1	1		10	140	1131	777		204	341		1	45
ETA	1							1	2	2		1	1	1		11			1214	371	1067	1043	262	1	61
THETA	1							1	2	2		1	1			3					2454			1	3
IOTA				1								1	1	1	1	16			593	1096	1020	680			49
KAPPA				1								1	1	1		25		279	948	750	900	2402	190	1	112
LAMBDA				1								1	1	1	2	15		1045	1219	305	396	731			49
MU				1								1	1	1		15		6663	6379	2073		1236		1	124
NU					1							1	1	1	1	21	305	1346	803	457	2003	1542		1	136
XI						1						1	1	1		7			2077	1829	567	1714		1	25
OMICRON		1						1	2		2	1	1	1		8	274	488	174		326			1	53
TOTAL	8	2	1	4	1	1	1	8	16	14	2	16	15	13	4	187	719	15453	17332	8320	17940	13217	1183	12	893

Annex 2

Form No.3

PROJECT IDENTIFICATION

IDENTIFICATION NUMBERS FOR INSTALLATIONS & EQUIPMENT

Type of installation or equipment → / ↓ Location	INTAKES											CHLORINATION		TANKS			MAIN & DISTRIBUTION SYSTEM PIPELINES							WATER METERS	
	Well			Stream													Pipe length in meters (dia.mm)								
	Hand Dug																								
	Plain	With filter	Bored	Perforated pipe	Direct	Spring	Pipe connexion to main	Pump house	Centr. pumps H.S.	Diesel engines	Elect. motors	Gravity feeder	Test kit	Storage	Break pressure	Stand posts	150	100	75	63	50	38	25	Main	Domestic
GROUP →	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q							R	S
ALPHA							G1							N1		P1-15				Q1					S1-124
BETA			C1					H1	I1,I2	J1,J2		L1	M1	N2		P16-24				Q2				R1	S125-204
GAMMA	A1,A2							H2	I3,I4	J3,J4		L2	M2	N3		P25-37				Q3				R2	S205-217
DELTA	A3-A5							H3	I5,I6	J5,J6		L3-L5	M3,M4			P38-45				Q4				R3	S218-225
EPSILON	A6							H4	I7,I8	J7,J8		L6	M5	N4		P46-56				Q5				R4	S226-236
ZETA		B1						H5	I9,I10	J9,J10		L7	M6	N5		P57-66				Q6				R5	S237-281
ETA	A7							H6	I11,I12	J11,J12		L8	M7	N6		P67-77				Q7				R6	S282-342
THETA	A8							H7	I13,I14	J13,J14		L9	M8			P78-80				Q8				R7	S343-345
IOTA				D1								L10	M9	N7	O1	P81-96				Q9					S346-394
KAPPA				D2								L11	M10	N8		P97-121				Q10				R8	S395-506
LAMBDA				D3								L12	M11	N9	O2,O3	P122-136				Q11					S507-555
MU				D4								L13	M12	N10		P137-151				Q12				R9	S556-679
NU					E1							L14	M13	N11	O4	P152-172				Q13				R10	S680-815
XI						F1						L15	M14	N12		P173-179				Q14				R11	S816-840
OMICRON		B2						H8	I15,I16		K1,K2	L16	M15	N13		P180-187				Q15				R12	S841-893

Form No.4

PROJECT IDENTIFICATION

PREVENTIVE MAINTENANCE RULES FOR INSTALLATIONS AND EQUIPMENT

Group	Description	Maintenance period	Responsibility		Task to be performed	Materials, Spare Parts, lubricants required
			Level	Crew		
A	Hand dug well intake	3 months 1 year	R R	m,o,a e,m,o	- Clean intake and adjacent area - Disinfect well with chlorine - Inspect sanitary condition - Inspect structures	
B	Hand dug well intake with sand filter	3 months 1 year	R R	m,o,a e,m, o,a	- Clean intake and adjacent area - Disinfect well with chlorine - Remove sand filter, wash sand, reassemble to level - Inspect sanitary conditions - Inspect structures	
C	Bored well intake	3 months 1 year	R N,R	m,o,a e,w, m,o,a	- Clean adjacent area - Clean casing and screens with appropriate tools and chemicals - Measure well output before and after cleaning, recording total flow and specific capacity	
D	Stream intake with perforated pipe	3 months 1 year	R R	m,o,a e,m,o	- Clean intake and adjacent area - Clean sand over perforated pipe - Inspect sanitary condition - Inspect structures	
E	Stream intake direct to collection chamber	3 months 1 year	R R	m,o,a e,m,o	- Clean intake and adjacent area - Clean intake channel - Drain collection chambers and disinfect with chlorine - Inspect sanitary condition - Inspect structures	
F	Spring intake	3 months 1 year	R R	m,o,a e,m,o	- Clean intake and adjacent area - Disinfect with chlorine - Inspect sanitary condition - Inspect structures	
G	Intake with pipe connexion to treated water main	1 year	R	e,m,o	- Inspect connexion, check for leaks and repair if necessary	
H	Pump house	1 day 3 months 1 year	L R R	o m,o,a e,m, o,a	- Clean and sweep floor, wipe and clean walls and piping - Clean and sweep exterior site, tend lawn and garden - Check and repair any leaks in supply and drainage lines - Check and repair any leaks in roof - Paint inside and outside walls and piping	
I	Centrifugal pumps, horizontal shaft	1 day 6 months 1 year	L R R	o m,p,o e,m, p,o	- Record readings of pressure head and suction gauges - Check packing glands to leak slightly during operation - Check tightness of nuts and bolts - Check lubrication of bearings - Replace grease/oil in bearings without disassembly - Check alignment - Replace packing in glands - Total overhaul: disassemble, clean and replace all defective parts	

KEY: Level: L = Local R = Regional N = National

Crew : e = engineer m = maintenance team o = operator a = additional labour as required
w = special team for servicing deep wells p = pump and motor mechanic s = meter service team

Group	Description	Maintenance period	Responsibility		Task to be performed	Materials, Spare parts, Lubricants required
			Level	Crew		
J	Diesel engines	1 day 1 week 1 month 3 months 1 year 2 years	L L R R R R	o o m,o m,p,o e,m, p,o m,p,o	- Check oil level and top up if necessary - Check diesel supply, clear fuel tank filter cap vent hole - Lubricate all lubrication points - Record oil pressure, temperature, current, rpm, battery charge - Record working hours and total since last oil change - Clean outside parts, check nuts and bolts for tightness - Wash and clean air filter - Check battery level and terminals - Drain and wash fuel tank, wash and clean filter,adjust belt tension - Remove deposit from exhaust system - Dismantle injector and test spray - Clean and inspect injector and valves - Check oil filters - Check clutch system - Fit new fuel filter element - Check starting system - Check and regrind valves - Clean deposits from cylinders and pistons - Replace piston rings, if necessary - Disassemble and check clutch system - Overhaul: complete disassembly, cleaning and replacement of defective parts	- Change engine oil every 250 hours or in accordance with manufacturer's instructions
K	Electric Motors	1 day 6 months 1 year	L R R	o m,p,o m,p,o	- Record voltage, current, gauge readings - Check lubrication - Change lubrication oil/grease without disassembly - Check starters - Overhaul: complete disassembly, cleaning and replacement of defective parts	
L	Gravity chlorine solution feeders	1 day 1 month 1 year	L R R	o m,o m,o	- Prepare correct strength solution from calcium hypochlorite - Clean outside of feeder solution tanks and valves - Disassemble unit and clean all parts - Overhaul: complete disassembly, cleaning and replacement of defective parts	
M	Portable chlorine test kit	1 week	L	o	- Wash and clean all components	
N	Storage tanks	1 month 1 year 3 years	R R R	m,o m,o, a,e m,o,a	- Check float valve and water level - Drain tanks, wash and clean inside and outside - Disinfect floor and walls with chlorine solution - Check all valves, open and close, and repair, if necessary - Paint exterior pipelines - Paint all metal surfaces	
O	Break-pressure tanks	1 month 1 year 3 years	R R R	m,o e,m, o,a m,o,a	- Check float valve and water level - Drain tanks, wash and clean inside and outside - Disinfect floor and walls with chlorine solution - Check all valves, open and close, and repair, if necessary - Paint exterior pipelines - Paint all metal surfaces	
P	Stand posts	1 week 1 month 1 year	L L R	o o e,m, o,a	- Check valves for leaks and repair, if necessary - Check float valve and water level, if applicable - Drain storage tank, wash and clean inside and outside - Disinfect walls and floor with chlorine solution - Check all valves, open and close, and repair, if necessary - Paint exterior piping and metal surfaces	
Q	Main and distribution pipelines and including valves and fitting	1 year	R	e,m, o,a	- Check and repair leaks in pipelines - Check all sluice, scour, air and pressure-reducing valves in mains and distribution systems; open and close valves and repair, if necessary - Drain water lines through scour valves and refill	
R	Main water meters	2 years	N	s	- Overhaul: complete disassembly, cleaning and replacement of defective parts	This work will be performed in the central water meter workshop.
S	Domestic water meters	2 years	N	s	- Overhaul: complete disassembly, cleaning and replacement of defective parts	

Form No.6

PROJECT IDENTIFICATION

PREVENTIVE MAINTENANCE CALENDAR FOR INSTALLATIONS & EQUIPMENT

MONTH: JANUARY

Working* Day No.	Location	Task to be performed
1,2	ALPHA	G1-1y N1-1y P1-15-1y Q1-1y
3	ZETA	J9-1m J10-1m L7-1m N5-1m
4,5	BETA	C1-3m H1-3m J1-3m J2-3m L1-1m N2-1m
6	ETA	J11-1m J12-1m L8-1m N6-1m
7	MU	L13-1m N10-1m
8	NU	L14-1m N11-1m O4-1m
9	THETA	J13-1m J14-1m L9-1m
10,11	GAMMA	A1-3m A2-3m H2-3m I3-6m I4-6m J3-3m J4-3m L2-1m N3-1m
12	IOTA	L10-1m N7-1m O1-1m
13	XI	L15-1m N12-1m
14,15	DELTA	A3-3m A4-3m A5-3m H3-3m J5-3m J6-3m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m N13-1m
17,18,19	EPSILON	A6-1y H4-1y I7-1y I8-1y J7-1y J8-1y L6-1y N4-1y P46-56-1y Q5-1y
20	KAPPA	L11-1m N8-1m
21	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: FEBRUARY

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2,3	ZETA	B1-3m H5-3m J9-3m J10-3m L7-1m N5-1m
4	BETA	J1-1m J2-1m L1-1m N2-1m
5,6	ETA	A7-3m H6-3m I11-6m I12-6m J11-3m J12-3m L8-1m N6-1m
7	MU	L13-1m N10-1m
8	NU	L14-1m N11-1m O4-1m
9,10	THETA	A8-3m H7-3m J13-3m J14-3m L9-1m
11	GAMMA	J3-1m J4-1m L2-1m N3-1m
12,13,14	IOTA	D1-1y L10-1y N7-1y O1-1y P81-96-1y Q9-1y
15	XI	L15-1m N12-1m
16	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
17	OMICRON	L16-1m N13-1m
18	EPSILON	J7-1m J8-1m L6-1m N4-1m
19	KAPPA	D2-3m L11-1m N8-1m
20	LAMBDA	D3-3m L12-1m N9-1m O2-1m O3-1m

MONTH: MARCH

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3	BETA	J1-1m J2-1m L1-1m N2-1m
4	ETA	J11-1m J12-1m L8-1m N6-1m
5	MU	D4-3m L13-1m N10-1m
6,7,8	NU	E1-1y L14-1y N11-1y O4-1y P152-172-1y Q13-1y
9	THETA	J13-1m J14-1m L9-1m
10	GAMMA	J3-1m J4-1m L2-1m N3-1m
11	IOTA	L10-1m N7-1m O1-1m
12,13	XI	F1-3m L15-1m N12-1m
14	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
15,16	OMICRON	B2-3m H8-3m I15-6m I16-6m K1-6m K2-6m L16-1m N13-1m
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18	KAPPA	L11-1m N8-1m
19	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: APRIL

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3,4,5	BETA	C1-1y H1-1y I1-1y I2-1y J1-1y J2-1y L1-1y L2-1y P16-24-1y Q2-1y
6	ETA	J11-1m J12-1m L8-1m N6-1m
7	MU	L13-1m N10-1m
8	NU	L14-1m N11-1m O4-1m
9	THETA	J13-1m J14-1m L9-1m
10,11	GAMMA	A1-3m A2-3m H2-3m J3-3m J4-3m L2-1m N3-1m
12	IOTA	L10-1m N7-1m O1-1m
13	XI	L15-1m N12-1m
14,15	DELTA	A3-3m A4-3m A5-3m H3-3m I5-6m I6-6m J5-3m J6-3m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m N13-1m
17,18	EPSILON	A6-3m H4-3m J7-3m J8-3m L6-1m N4-1m
19	KAPPA	L11-1m N8-1m
20	LAMBDA	L12-1m N9-1m O2-1m O3-1m

*It is assumed that there are 20 working days per month. Actual dates should be used when known.

MONTH: MAY		
Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2,3,4	ZETA	B1-1y H5-1y I9-1y I10-1y J9-1y J10-1y L7-1y N5-1y P57-66-1y Q6-1y
5	BETA	J1-1m J2-1m L1-1m N2-1m
6,7	ETA	A7-3m H6-3m J11-3m J12-3m L8-1m N6-1m
8	MU	L13-1m N10-1m
9	NU	L14-1m N11-1m O4-1m
10,11	THETA	A8-3m H7-3m I13-6m I14-6m J13-3m J14-3m L9-1m
12	GAMMA	J3-1m J4-1m L2-1m N3-1m
13	IOTA	D1-3m L10-1m N7-1m O1-1m
14	XI	L15-1m N12-1m
15	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m N13-1m
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18,19,20	KAPPA	D2-1y L11-1y N8-1y P97-121-1y Q10-1y
21	LAMBDA	D3-3m L12-1m N9-1m O2-1m O3-1m

MONTH: JUNE		
Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3	BETA	J1-1m J2-1m L1-1m N2-1m
4	ETA	J11-1m J12-1m L8-1m N6-1m
5	MU	D4-3m L13-1m N10-1m
6,7	NU	E1-3m L14-1m N11-1m O4-1m
8	THETA	J13-1m J14-1m L9-1m
9	GAMMA	J3-1m J4-1m L2-1m N3-1m
10	IOTA	L10-1m N7-1m O1-1m
11,12,13	XI	F1-1y L15-1y N12-1y P173-179-1y Q14-1y
14	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
15,16	OMICRON	B2-3m H8-3m L16-1m N13-1m
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18	KAPPA	L11-1m N8-1m
19	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: JULY		
Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3,4	BETA	C1-3m H1-3m J1-3m J2-3m L1-1m N2-1m
5	ETA	J11-1m J12-1m L8-1m N6-1m
6	MU	L13-1m N10-1m
7	NU	L14-1m N11-1m O4-1m
8	THETA	J13-1m J14-1m L9-1m
9,10,11	GAMMA	A1-1y A2-1y H2-1y I3-1y I4-1y J3-1y J4-1y L2-1y N3-1y P25-37-1y Q3-1y
12	IOTA	L10-1m N7-1m O1-1m
13	XI	L15-1m N12-1m
14,15	DELTA	A3-3m A4-3m A5-3m H3-3m J5-3m J6-3m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m N13-1m
17,18	EPSILON	A6-3m H3-3m I7-6m I8-6m J7-3m J8-3m L6-1m N4-1m
19	KAPPA	L11-1m N8-1m
20	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: AUGUST		
Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2,3	ZETA	B1-3m H5-3m J9-3m J10-3m L7-1m N5-1m
4	BETA	J1-1m J2-1m L1-1m N2-1m
5,6,7	ETA	A7-1y H6-1y I11-1y I12-1y J11-1y J12-1y L8-1y N6-1y P67-77-1y Q7-1y
8	MU	L13-1m N10-1m
9	NU	L14-1m N11-1m O4-1m
10,11	THETA	A8-3m H7-3m J13-3m J14-3m L9-1m
12	GAMMA	J3-1m J4-1m L2-1m N3-1m
13	IOTA	D1-3m L10-1m N7-1m O1-1m
14	XI	L15-1m N12-1m
15	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m N13-1m
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18	KAPPA	D2-3m L11-1m N8-1m
19,20,21	LAMBDA	D3-1y L12-1y N9-1y O2-1y O3-1y P122-136-1y Q11-1y

*It is assumed that there are 20 working days per month. Actual dates should be used when known.

MONTH: SEPTEMBER

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3	BETA	J1-1m J2-1m L1-1m N2-1m
4	ETA	J11-1m J12-1m L8-1m N6-1m
5	MU	D4-3m L13-1m N10-1m
6,7	NU	E1-3m L14-1m N11-1m O4-1m
8	THETA	J13-1m J14-1m L9-1m
9	GAMMA	J3-1m J4-1m L2-1m N3-1m
10	IOTA	L10-1m N7-1m O1-1m
11,12	XI	F1-3m L15-1m N12-1m
13	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
14,15,16	OMICRON	B2-1y H8-1y I15-1y I16-1y K1-1y K2-1y L16-1y N13-1y P180-187/1y Q15-1y
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18	KAPPA	L11-1m N8-1m
19	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: OCTOBER

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3,4	BETA	C1-3m H1-3m I1-6m I2-6m J1-3m J2-3m L1-1m N2-1m
5	ETA	J11-1m J12-1m L8-1m N6-1m
6	MU	L13-1m N10-1m
7	NU	L14-1m N11-1m O4-1m
8	THETA	J13-1m J14-1m L9-1m
9,10	GAMMA	A1-3m A2-3m H2-3m J3-3m J4-3m L2-1m N3-1m
11	IOTA	L10-1m N7-1m O1-1m
12	XI	L15-1m N12-1m
13,14,15	DELTA	A3-1y A4-1y A5-1y H3-1y I5-1y I6-1y J5-1y J6-1y L3-1y L4-1y L5-1y P38-45-1y Q4-1y
16	OMICRON	L16-1m N13-1m
17,18	EPSILON	A6-3m H4-3m J7-3m J8-3m L6-1m N4-1m
19	KAPPA	L11-1m N8-1m
20	LAMBDA	L12-1m N9-1m O2-1m O3-1m

MONTH: NOVEMBER

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2,3	ZETA	B1-3m H5-3m I9-6m I10-6m J9-3m J10-3m L7-1m N5-1m
4	BETA	J1-1m J2-1m L1-1m N2-1m
5,6	ETA	A7-3m H6-3m J11-3m J12-3m L8-1m N6-1m
7	MU	L13-1m N10-1m
8	NU	L14-1m N11-1m O4-1m
9,10,11	THETA	A8-1y H7-1y I13-1y I14-1y J13-1y J14-1y L9-1y P78-80-1y Q8-1y
12	GAMMA	J3-1m J4-1m L2-1m N3-1m
13	IOTA	D1-3m L10-1m N7-1m O1-1m
14	XI	L15-1m N12-1m
15	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
16	OMICRON	L16-1m L13-1m
17	EPSILON	J7-1m J8-1m L6-1m N4-1m
18	KAPPA	D2-3m L11-1m N8-1m
19	LAMBDA	D3-3m L12-1m N9-1m O2-1m O3-1m

MONTH: DECEMBER

Working* Day No.	Location	Task to be performed
1	ALPHA	N1-1m
2	ZETA	J9-1m J10-1m L7-1m N5-1m
3	BETA	J1-1m J2-1m L1-1m N2-1m
4	ETA	J11-1m J12-1m L8-1m N6-1m
5,6,7	MU	D4-1y L13-1y N10-1y P137-151-1y Q12-1y
8,9	NU	E1-3m L14-1m N11-1m O4-1m
10	THETA	J13-1m J14-1m L9-1m
11	GAMMA	J3-1m J4-1m L2-1m N3-1m
12	IOTA	L10-1m N7-1m O1-1m
13,14	XI	F1-3m L15-1m N12-1m
15	DELTA	J5-1m J6-1m L3-1m L4-1m L5-1m
16,17	OMICRON	B2-3m H8-3m L16-1m N13-1m
18	EPSILON	J7-1m J8-1m L6-1m N4-1m
19	KAPPA	L11-1m N8-1m
20	LAMBDA	L12-1m N9-1m O2-1m O3-1m

*It is assumed that there are 20 working days per month. Actual dates should be used when known.