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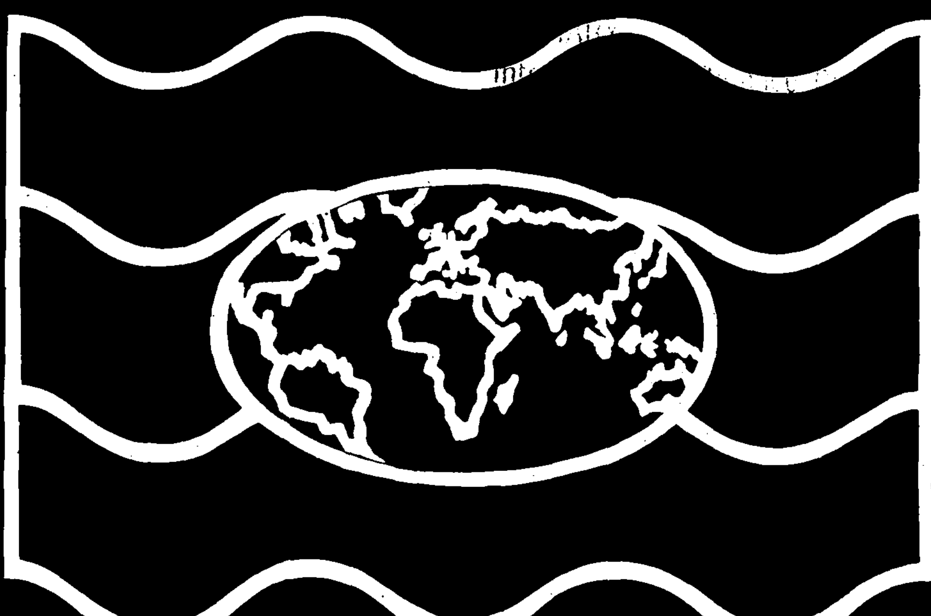

Opening Speech

Low Cost Technology for Public Water Supplies in Developing Countries

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1. INTERNATIONAL DRINKING WATER SUPPLY DECADE 1981-1990

Where people live, water is available because without water no life on earth is possible. In many instances, however, this water is polluted with faecal matter and contains pathogenic organisms responsible for the spread of water borne diseases such as helminthiasis, amoebe dysentery, typhoid and cholera, shigella and other diarrhoeal disorders, infectious hepatitis and gastro-enteritus. In other cases the water has to be carried over long distances, causing much hardship and severely reducing the amount of water used. The resulting lack of hygience gives rise to the so-called water washed diseases with skin and eye infections, bacillary dysentery and louse borne fever as most important representatives. Even an ample supply of water at a short distance from the community may endanger the health of the people concerned because it harbours organisms that cause water based diseases such as schistosomiasis and guinea worm or water related diseases such as malaria and yellow fever next to onchocerciasis and sleeping thickness. It is estimated that 80% of all illnesses in developing countries are in one way or another related to water.

According to the World Health Organization, the number of people without reasonable access to safe water amounts to 1500 million, increasing to 2000 million at the end of this decade. According to the World Bank, the cost of providing these people with some sort of public supply lies between 100 and 300 billion US dollar, say $20 billion per year to reach the goal of the U.N. decade of safe water for all by 1990. This amount of money is small compared to the world expenditure on cigarettes ($90 billion/year) and negligeable to the global arms bill ($500 billion/year), but the majority of the money has to come from the developing countries themselves. These are poor and have many other pressing needs, asking that the utmost economy is practised during construction and operation of public water supplies, that a low cost technology is applied (2, 5, 13, 14, 15)*.

The money mentioned above should not only be made available, it should also be spent in a competent way. Huge numbers of technical, administrative and managerial personnel are thus necessary for design and construction as well as for operation and maintenance and they are simply not available. On one hand this means that education and training should be given high priorities, while on the other hand less labour intensive ways should be sought. For design and construction this means a shift from projects to programs (10, 11), while for operation and maintenance the emphasis must be placed on an appropriate technology, to be used in the millions of villages by simple people after a short on the job training (3, 9, 16, 19, 25, 27).

* The numbers between brackets refer to the programmed papers

2. ANALYSIS OF A LOW COST TECHNOLOGY

The cost of any work can be subdivided in the cost of construction on one hand and the recurrent costs for operation and maintenance on the other. For public water supplies in developing countries mostly only a small part (say 20%) of the construction cost is borne by the local population, the bulk of the money coming from higher government agencies and from foreign aid. The cost for operation and maintenance is nearly always paid in full by the people themselves. In many cases this is a very heavy burden, consuming 5% or more of the family income (compared to 1% in developed countries). During design, all efforts should therefore be made to keep these costs down, by using a gravity supply instead of a pumped one, by applying biological treatment instead of a chemical one with the additional advantage that lack of money to buy oil and coagulating or desinfecting chemicals does not endanger the operation or reliability of the supply.

Both costs mentioned above may further be subdivided in the cost of labour and the cost of materials, while in developing countries with shortages of qualified personnel and foreign currency, the labour requirements needs to be split in skilled and unskilled labour and the necessary materials in those locally available next to the ones that have to be imported. In the countries under consideration, unskilled labour is in abundant supply and should be used as much as possible, even when mechanization and automation is economically (much) more attractive. In many countries expert personnel for design is also widely available and when not the case, hiring foreign experts offers little difficulties. When they are also used for on the job education of local technicians, the country will soon become independent in this respect. For operation and maintenance, however, the situation is completely different. The large majority of the supplies are small ones in rural areas, where the necessary skills are not available. Training during the construction period is the logical answer, but after gaining some experience, the operator is tempted to go to a larger town where life is more attractive. To prevent a subsequent collapse of the small village water supplies, some scheme of incentives is clearly needed, providing the operator with a better house, paying him a bonus after three years of uninterrupted service, etc. Part-time jobs for local artisans may also be an attractive proposition.

To improve the economic situation in the country concerned, locally obtainable materials should be used as much as possible with the added advantage that they are also available for repair purposes after the construction period has ended. In principle there is little to object against the use of materials and equipment from abroad, provided that they are simple, rugged and reliable so that operation, maintenance and repair can be carried out by local people. With regard to a future shortage of foreign currency an adequate store of spare parts should be bought at the same time. Differences in climatic conditions and in the way equipment is handled by the public should finally be given due consideration.

3. POSSIBILITIES FOR LOW COST TECHNOLOGY

According to the hydrologic cycle of fig. 1, water for domestic use may be abstracted
as roof drainage before the rainwater reaches the ground;
as ground catchment before the rainwater runs off or percolates downward;
as groundwater;
as springwater at the point of re-emergence to ground surface and
as surface water in rivers and lakes.

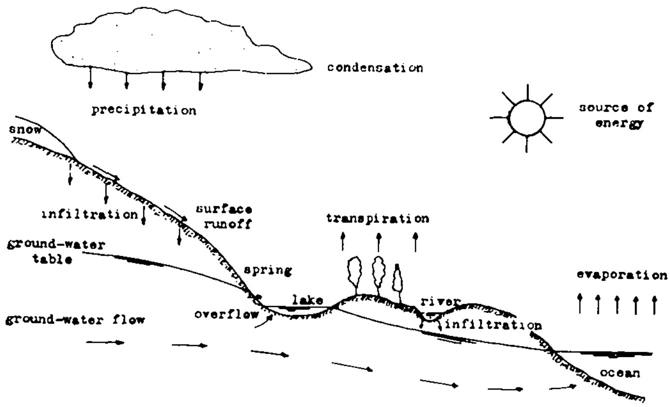

Fig. 1 Hydrologic cycle

From times immemorial, rainwater has been used as a source for domestic supplies all over the world and for rural areas in developing countries, in particular with a dispersed population, it is still an excellent source. Roof drainage has the advantage that the catchment area is already available and only a gutter and some storage vessel need to be provided (fig. 2). In dry climates the amounts that can be collected in this way are rather small, asking for a ground catchment of larger extent when greater quantities are needed. In both cases storage is needed to tide over dry periods. For roof catchments, ferro cement containers (20) are used on an increasing scale, while for the larger amounts of water from ground catchments cisterns have to be applied (6). The catchment area and thus the rainwater harvested may be contaminated with wind blown dust, bird droppings, etc. With roof catchments, the first amounts of water appearing after a dry period should therefore be discarded, while for ground catchments the cistern should be provided with a sand filter for instance as shown in fig. 3.

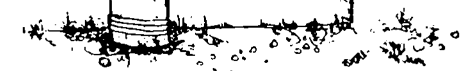

Fig. 2 Roof catchment

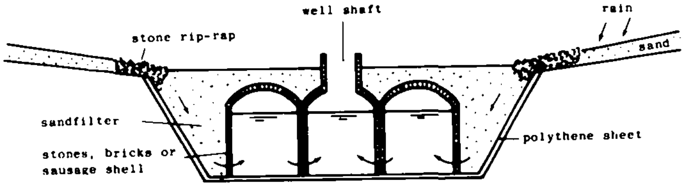

Fig. 3 Ground catchment with cistern

Groundwater does not contain pathogenic organisms responsible for the spread of water borne diseases as mentioned in section 1. Moreover it can be withdrawn at various sites in the community, doing away with the need for transport. As such it provides a cheap and reliable water supply in the rural areas of developing countries. When the groundwater is present at a short distance below ground surface, less than 5 to 10 m, it can be recovered with dug wells having an inside diameter of 1-1.5 m (fig. 4). For the abstraction of deep groundwater tube wells must be used, having for village use a diameter of 0.05-0.15 m and up to 0.6 or more for large capacities. Dug wells have the enormous advantage that they can be made by the villagers themselves using local skills and locally available material. The open top of fig. 4 allows the abstraction with bucket and rope, but exposes the water to contamination after which it becomes the source for water borne diseases. The well should therefore be provided with a water-tight cover, using a handpump for withdrawal, for instance as shown in fig. 5 (7).

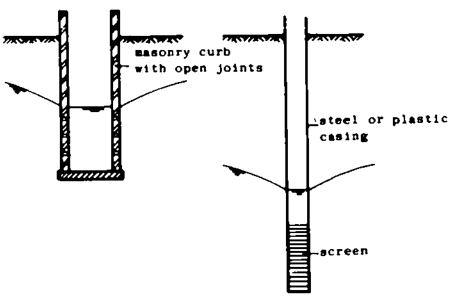

Fig. 4 Dug well and tube well

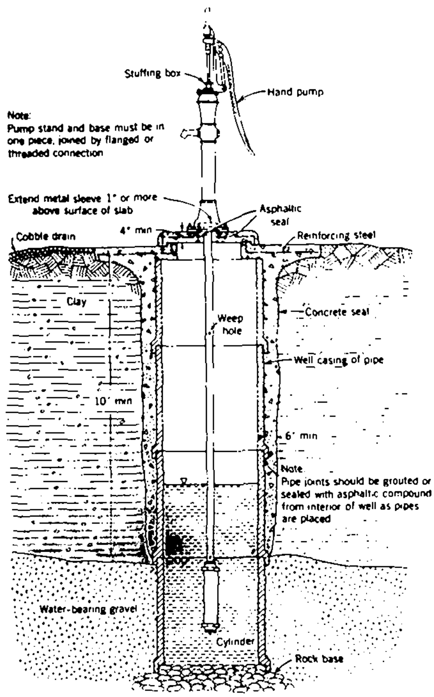

Fig. 5 Dug well sealed for sanitary protection
(after Joint Committee on Rural Sanitation)

Tube wells can be constructed in different ways, but specialized equipment and expert skills are always required. During the last 20 years many new methods of well drilling have been deviced to lower the cost by shortening the time of construction (8). With regard to the small diameter, the water has to be abstracted with pumps, hand-driven for small capacities and low lifts and motor driven when larger amounts have to be recovered at greater depths (8). When tube wells have to be used, a programma must be set up to keep the drilling crew and their equipment occupied for a longer period.

A spring is a place where a natural outflow of groundwater occurs, either locally (tubular springs) or spread out over some distance (line springs). At these springs groundwater can easily be recovered and when simple precautions are taken to prevent contamination, it will be safe in hygienic respect (fig. 6). As disadvantage must be mentioned that transport is nearly always necessary. In many cases, however, this can be accomplished by gravity, doing away with the need of pumping. In developed countries practically all springs attractive for public supplies have already been incorporated, but in developing countries many opportunities still exist.

Surface water in rivers and lakes is directly visible and can easily be abstracted (fig. 7), but it always needs treatment, in particular to remove turbidity and pathogenic organisms. For relatively clear waters as can be found in lakes, slow sand filtration (fig. 8) is the best solution, accomplishing both purposes at the same time. For more turbid waters, slow sand filtration can still be used, provided that the major part of the suspended matter is removed by pre-treatment, by settling or filtration with or without the addition of coagulating chemicals to save on the cost of construction (11, 12, 15).

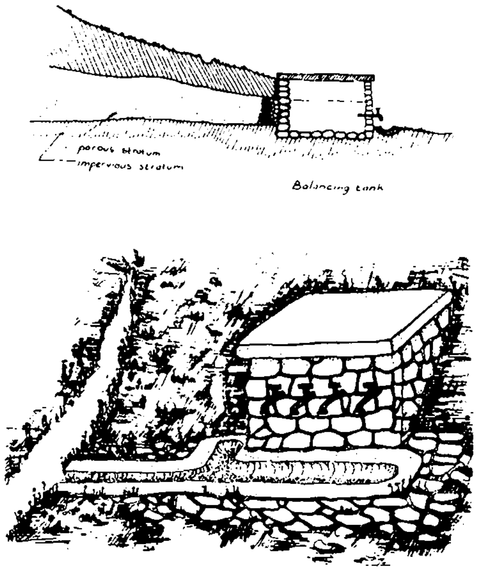

Fig. 6 Sanitary water intake of small capacity

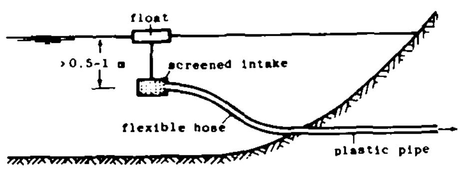

Fig. 7 Surface water intake of small capacity

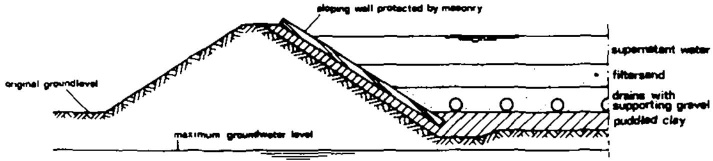

Fig. 8 Slow sand filtration with sloping wells

For river water of a high turbidity, chemical coagulation and flocculation should be used, followed by settling or upflow filtration (15) as pre-treatment and rapid filtration as final one. This is a fairly complicated and expensive process, but fortunately many simplifications are possible, such as the use of pebble bed instead of mechanical flocculators (10, 16), the use of plate settlers instead of plain sedimentation basins (10, 16), the use of multi-layer filters with a high silt storage capacity doing away with the need for settling tanks altogether and the possibility of back-washing a filtering unit with the effluent of other units still in operation. The effluent of a rapid filter, however, may still contain pathogenic organisms, asking for disinfection. This can be accomplished in the classical way by dosing chlorine or chlorine compounds (24), more reliable by filtration over a granular medium containing disinfectants (22) and more simple by ultra violet light (23).

Surface water in the meanwhile does not need to be used directly, it can also be applied to reinforce or to create groundwater sources, which in their turn will yield a clear water safe in hygienic respect. Fig. 9 shows one of the many possibilities for bank infiltration, also providing water when in rainless periods the river has dried up. The artificial recharge scheme of fig. 10 has only limited storage capabilities, but it is a cheap and excellent way to treat water. When accomplished in a suitable aquifer (fig. 11), both storage and treatment are obtained, while with detention times of several months it is an excellent way to transform reclaimed waste water into a potable drinking water.

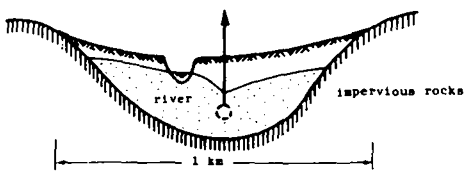

Fig. 9 Bank infiltration

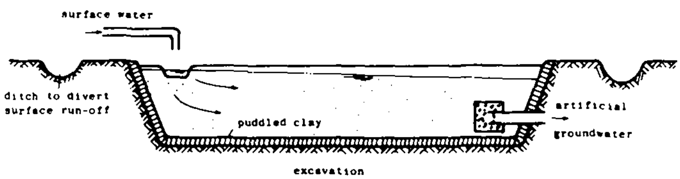

Fig. 10 Horizontal flow slow sand filtration

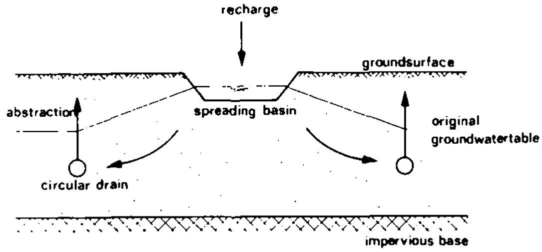

Fig. 11 Artificial recharge

In developed countries, transport and distribution of water are responsible for about 70% of the cost as delivered to the consumer. In developing countries with shortage of funds, these items should therefore be reduced as much as possible. With a dispersed abstraction of rain- and groundwater, there is little need for transport and distribution, but with point recovery of spring and surface water these items cannot be avoided altogether. The distribution system can be kept small by using a branched one with public taps at intervals of say 500 m, with the added advantage of reducing water losses by leakage. As disadvantages must be mentioned the reduction in consumption by the need to carry the water home (20 l per capita and day means 100 to 150 kg for the housewife to carry home) and the accompanying quality deterioration by contamination. A better proposition in these respects is the use of yard connections, that is one tap outside the house for a few houses close together. Paper 21 will discuss this possibility. When yard and house connections are used on a larger scale, a more extensive distribution system in looped form is required, bringing with it the need for an optimal design (18).

With the exception of spring water recovered some distance above the distribution area, pumping is always required. For small amounts and low lifts, hand pumps may be applied for which today reliable constructions are available. Larger amounts and/or higher lifts require the use of motor pumps. The pumps themselves are quite sturdy, as is also the case with electric motors. Internal combustion engines on the other hand require frequent and expert maintenance, which is not always available. Regular provision of fuel moreover is often not assured, giving a preference for renewable energy sources (27).

From the foregoing it will already be clear that a good management is a prerequisite for success. Next to this a sound administration is required to keep the undertaking viable on the long run.

Discours d'ouverture

Technologie à faible coût pour la distribution publique de l'eau dans les pays en voie de developpement

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1. LA DECENNIE INTERNATIONALE DE L'EAU POTABLE ET DE L'ASSAINISSEMENT 1981-1990

L'eau est généralement disponible là où habitent les gens car sans eau aucune vie n'est possible sur terre. Dans de nombreux cas cependant, cette eau est polluée par les matières fécales et contient des organismes pathogènes responsables de la diffusion par voie d'eau de maladies telles que l'helminthiase, la dysenterie amibienne, la typhoïde et le choléra, le shigella, et d'autres désordres diarrhéiques, l'hépatite virale et la gastro-entérite. Dans d'autres cas, il est nécessaire de porter l'eau sur de longues distances, ce qui est très pénible et réduit considérablement la quantité d'eau utilisée. Le manque d'hygiène qui en résulte est source des maladies dites de lavage à l'eau ; infections de la peau et des yeux, dysenterie bacillaire ou fièvre transportée par les poux qui sont les plus courants. Même une source abondante d'eau à proximité de la communauté peut mettre en danger la santé des personnes concernées puisqu'elle peut engendrer des maladies dues à l'eau telles que la schistosomiose et le ver de Guinée ou des maladies indirectement liées à l'eau telles que la malaria ou la fièvre jaune avec à côté l'onchocerccose et la maladie du sommeil. On estime que 80 % environ de toutes les maladies des pays en voie de développement proviennent de l'eau.

Selon l'Organisation Mondiale de Santé, le nombre de personnes ne bénéficiant pas d'un accès raisonnable à une eau potable est d'environ 1,5 milliard et atteindra 2 milliards à la fin de cette décennie. Selon la Banque Mondiale, organiser la distribution d'eau pour ces personnes coûterait 100 et 300 milliards, disons 20 milliards $ par an,pour atteindre l'objectif de la décennie ONU, l'eau potable pour tous en 1990. Le montant est modeste, comparé aux dépenses mondiales sur les cigarettes (90 milliards $ par an) et négligeable si on le compare à la facture globale de l'armement (500 milliards $ par an), mais les pays en voie de développement doivent fournir pour la plupart eux-mêmes le financement. Ils ne peuvent consacrer que peu à la construction et au fonctionnement de services d'eau et ont donc besoin de technologies à bas-prix (2,5,13,14,15)*.

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* Les chiffres entre parenthèses se réfèrent aux documents du programme

Non seulement les sommes citées ci-dessus doivent être rendues disponibles, elles doivent aussi être dépensées de façon compétente. Il est nécessaire d'avoir recours à du personnel technique, administratif et de gestion en quantité suffisante pour la conception, la mise en route, le fonctionnement et l'entretien des installations. Celui-ci n'est pas nécessairement disponible. D'un côté, ceci veut dire qu'il faut donner la priorité à l'éducation et à la formation tandis que de l'autre il faut trouver des moyens plus économiques du point de vue de la main d'oeuvre. Concernant les plans et la construction ceci implique un déplacement des projets en direction de programmes (10, 11), tandis qu'en ce qui concerne le fonctionnement et l'entretien il faut insister en vue d'une technologie appropriée, qui sera utilisée dans les millions de villages par des gens simples après une courte période de formation sur place (3, 9, 16, 19, 25, 27).

2. L'ANALYSE DES TECHNOLOGIES A FAIBLE COUT

Le coût des travaux peut toujours être subdivisé entre le coût de la construction d'une part et les coûts de fonctionnement et d'entretien qui reviennent périodiquement d'autre part. En ce qui concerne la distribution publique de l'eau dans les pays en voie de développement, en général, une petite part seulement (disons 20 %) des frais de construction revient à la population locale, la plus grande partie de l'argent étant fournie par des organismes gouvernementaux et par l'aide venue de l'étranger. Les frais de fonctionnement et d'entretien sont presque toujours payés en totalité par les personnes elles-mêmes. Dans de nombreux cas, ceci constitue une lourde charge, représentant 5 % ou plus des revenus de la famille (comparé à 1 % dans les pays développés). Au stade des plans, il faut donc réduire au maximum ces frais, avec une distribution par gravité plutôt que par pompe, en utilisant un traitement biologique et non un traitement chimique, avec l'avantage supplémentaire et paradoxal qu'un manque d'argent - pour acheter le pétrole, les produits chimiques, de coagulation ou de désinfection - ne mettra pas en danger le fonctionnement ou la sécurité de la distribution.

Les deux natures de coûts citées ci-dessus peuvent encore être subdivisés entre le prix de revient de la main d'oeuvre et le prix des matériaux. Dans les pays en voie de développement où manquent le personnel qualifié et les devises étrangères, les besoins en main d'oeuvre sont divisés entre la main d'oeuvre spécialisée et la main d'oeuvre non-spécialisée, et les matériaux nécessaires entre ceux qui sont disponibles sur place et ceux qui doivent être importés. Dans les pays que nous considérons il existe une abondante main d'oeuvre non-specialisée qu'il faut utiliser le plus possible, même quand la mécanisation et l'automation sont (beaucoup) plus attirantes économiquement. Dans beaucoup de pays un personnel expert est aussi disponible sans difficulté et quand ce n'est pas le cas il n'est pas trés difficile d'employer des experts venus de l'étranger. Quand ceux-ci sont également utilisés pour la formation sur place de techniciens indigènes, le pays pourra être indépendant à cet egard. Cependant en ce qui concerne le fonctionnement et l'entretien, la situation est differente. La plus grande partie des installations de distribution se trouve en zone rurale, ou les compétences nécessaires ne sont pas disponibles. La solution logique serait la formation pendant la période de construction, mais après avoir acquis une certaine expérience il est tentant pour le personnel spécialisé d'aller dans une ville plus grande où la vie est plus attrayante. Pour empêcher cette désafection, et l'arrêt de la distribution d'eau qui s'en suivrait, il convient d'offrir un système incitatif à ces personnels = une maison, une prime après 3 ans de service, etc. Des emplois à mi-temps pour les artisans du pays pourrait également constituer une proposition attirante.

Afin d'améliorer la situation économique du pays concerné, il faut utiliser le plus possible les matériaux disponibles sur place, qui permettront également les travaux de réparation. En principe, il n'y a guère d'objections à l'emploi de matériaux et d'équipement venus de l'étranger, si ceux-ci sont simples, solides et sûrs, de telle sorte que la population locale puisse assurer le fonctionnement, l'entretien et les réparations de l'installation. En vue d'un manque éventuel de devises étrangères il faut aussi acheter en même temps un nombre suffisant de pièces détachées pour constituer une réserve. Les différences dans les conditions climatiques, et dans la façon dont l'équipement sera utilisé par les gens doivent, également, être prises en considération.

3. LES POSSIBILITES D'UNE TECHNOLOGIE A FAIBLE COUT

Selon le cycle hydrologique de la figure 1., une extraction de l'eau à usage domestique est possible :

- captation des eaux de toitures avant que l'eau de pluie n'atteigne le sol ;
- captation en bassins versants avant l'écoulement ou l'infiltration des eaux de pluie ;
- eau souterraine ;
- eau de source au lieu de ré-apparition à la surface du sol et
- eau de surface dans les rivières ou les lacs.

Depuis des temps immémoriaux, l'eau de pluie a servi de source à l'approvisionnement domestique et dans les régions rurales des pays en voie de développement, plus particulièrement là où la population est dispersée, elle demeure une source de base. L'écoulement des toits a l'avantage d'utiliser une surface de captation déjà disponible et seuls une gouttière et un reservoir d'emmagasinage sont nécessaires (fig. 2). Dans les climats secs, les quantités susceptibles d'être ainsi recueillies sont plutôt faibles, demandant une surface de captation plus ample lorsque sont nécessaires des quantités plus importantes. Dans les deux cas, il faut emmagasiner en vue des périodes séches. Pour la captation sur toit, des réservoirs (20) en ferro-ciment sont utilisés de plus en plus, tandis que pour des quantités d'eau plus grandes obtenues par captation au sol, il faut utiliser des citernes (6). La surface de captation et donc l'eau de pluie peut être contaminée par de la poussière apportée par le vent, par la fiente des oiseaux, etc. Dans les captations sur toits, il faut rejeter les premières quantités d'eau obtenues après une période sèche, tandis que pour les captations au sol la citerne doit être munie d'un filtre à sable tel que le montre la fig. 3.

Comme indique dans la section 1., l'eau souterraine ne contient pas d'organismes pathogènes responsables de la diffusion des maladies. En plus, son extraction est possible en divers sites dans la communauté, éliminant la nécessité du transport. En tant que telle, elle offre une source économique et sure dans les régions rurales des pays en voie de développement. Lorsque l'eau souterraine est présente à peu de distance de la surface du sol (5 a 10 metres), elle peut être recuperée avec des puits creusés ayant un diamètre intérieur de 1-1,5 m. (fig. 4). L'extraction d'eau souterraine profonde demande des puits tubes, ayant un diamètre de 0,05-0,15 m pour utilisation en village et allant jusqu'à 0,6 m ou plus pour des capacités importantes. Les puits creusés présentent un avantage important, car ils peuvent être réalisés par les villageois eux-mêmes, à partir des matériaux disponibles sur place. Le toit ouvert de la fig. 4 permet l'extraction avec une corde et un seau, mais expose l'eau à la contamination qui en fait une source de maladies. Il faut donc donner au puits un couvercle hermétique et utiliser une pompe à main pour l'extraction, dont la fig. 5 donne un exemple (7).

Les puits tubés peuvent être construits de diverses manières, mais un équipement et des compétences spécialisés sont toujours nécessaires. Pendant les 20 dernières années plusieures nouvelles méthodes de forage de puits ont été inventées afin de réduire le prix de revient en raccourcissant le temps de construction (8). En ce qui concerne les petits diamètres, il faut extraire l'eau avec des pompes, à main pour les petites capacités et basses poussées, à moteur lorsqu'il s'agit de récupérer des quantités plus importantes à des profondeurs plus grandes (8). Dans le cas de puits tubés, un programme doit être établi afin que l'équipe de forage ainsi que leur équipement soit utilisé sur une période suffisante.

Une source est un lieu d'écoulement naturel d'eau souterraine, soit a proximite (sources tubulaires), soit repandu sur quelque distance (sources linéaires). Il est aisé de récuperer l'eau souterraine de ces sources et lorsque de simples précautions ont été prises en vue d'empêcher la contamination, elle sera pure du point de vue hygiènique (fig. 6). Un désavantage demeure le transport, nécessaire dans presque tous les cas. Il est cependant souvent possible d'utiliser la gravité pour cela, évitant l'emploi d'une pompe. Dans lespays développés, presque toutes les sources se prêtant aux distributions publiques ont été intégrées mais dans les pays en voie de développement il existe de nombreuses occasions possibles.

Dans les rivières et les lacs, l'eau de surface est directement visible et l'extraction en est facile (fig. 7), mais elle a toujours besoin d'être traitée, afin surtout d'en éliminer la turbidité et les organismes pathogènes. En ce qui concerne les eaux relativement claires comme c'est le cas pour les lacs, une filtration lente par sable (fig. 8) demeure la meilleure solution, remplissant les deux fonctions en même temps. En ce qui concerne des eaux plus troubles, une filtration lente par sable peut encore servir, si la plus grosse partie des substances en suspension est éliminée en pré-traitement, par décantation ou par filtration avec ou sans l'utilisation de coagulants chimiques afin de réduire les frais de construction (11, 12, 15).

En ce qui concerne les rivières de grande turbidité, il faut se servir de coagulation chimique et de flocculation, suivi de décantation ou de filtration comme pré-traitement avec, finalement, une filtration rapide. Ce procédé est assez compliqué et coûteux, mais heureusement plusieurs simplifications sont possibles, telles que l'utilisation d'un banc de galets à la place de floculateurs mécaniques (10, 16), des décanteurs à plaques à la place de bassins de sédimentation (10, 16). L'utilisation de filtres multi-couches ayant une capacité élevée de fixation de substrat supprime la nécessité de bassins de décantation et le rincage d'une unité de filtration est possible avec l'effluent d'autres unités toujours en opération. Cependant, l'effluent d'un filtre rapide peut encore contenir des organismes pathogènes et il faut désinfecter. Ceci peut se faire de la manière classique avec un dosage de chlore ou un des composés du chlore (24), plus surement par filtration sur milieu granulaire contenant des désinfectants (22) ou en utilisant des ultra-violets.

Il n'est pas toujours nécessaire d'utiliser une eau de surface directement. On peut également s'en servir pour renforcer ou pour créer des sources d'eau souterraine qui à leur tour donneront une eau potable pure du point de vue hygiénique. La fig. 9 montre une des nombreuses possibilités de berges filtrantes, susceptible de donner de l'eau pendant les périodes sans pluies lorsque la rivière est à sec. Le système artificiel de recharge, fig. 10, est une façon excellente et peu coûteuse de conserver l'eau malgré ses capacités limitées de stockage. Une formation aquifère adaptée constitue un mode de traitement et de stockage,(fig. 11), et lorsque les temps de rétention sont de plusieurs mois ceci est une méthode excellente de transformer une eau usée en eau potable.

Dans les pays développés, le transport et la distribution de l'eau représentent environ 70% du coût pour le consommateur. Les pays en voie de développement ayant peu de fonds, ceci doit donc être reduit le plus possible. Avec une extraction dispersée d'eau de pluie et d'eau souterraine, le transport et la distribution ne sont pas toujours nécessaires, mais avec la récuperation en points fixes d'eau de source et de surface, ces aspects ne peuvent être évités. Il est possible d'avoir un petit systeme de distribution en utilisant des branchements avec robinets tous les 500 metres, avec l'avantage d'une réduction de la perte d'eau par fuites. Comme désavantage, il faut mentionner la reduction de la consommation qui résulte de la nécessité de porter cette eau à domicile (20 litres par habitant par jour signifie que la ménagère doit ramener de 100 a 150 kg) et la perte de qualité qui s'ensuit par contamination. De ce point de vue, il vaut mieux se servir de branchements dans les cours, ce qui signifie un robinet hors de la maison desservant plusieurs maisons proches l'une de l'autre. Le document 21 examine cette possibilité. Lorsque des raccordements avec des maisons et des cours sont utilisés à plus grande échelle, un système de distribution plus important en forme de boucle est nécessaire, ce qui implique un dessin optimum (18).

A l'exception d'eau de source récupérée à quelque distance au-dessus de la région de distribution, le pompage est toujours nécessaire. Lorsqu'il s'agit de petites quantités et de poussées basses, il est possible de se servir de pompes à main dont il existe aujourd'hui des modèles plus fiables. Des quantités plus importantes et/ou des hauteurs plus élevées de poussée nécessitent des pompes à moteur. Les pompes elles-mêmes sont très sûres, ainsi que les moteurs électriques. Les moteurs à explosion par contre demandent un entretien régulier et complexe, lequel n'est pas toujours possible. En plus, l'approvisionnement régulier en carburant n'est pas toujours assuré, ce qui fait préférer les sources d'énergie renouvelable (27).

D'après ce qui précède, il doit deja être évident qu'une bonne gestion est essentielle pour réussir. A côte de cela, une administration solide est necessaire afin de garantir le succès des projets à long terme.

Contents

Paper		Page
	Opening Speech on Low cost technology for public water supplies Discours d'ouverture sur les technologies à faible coût pour la distribution publique de l'eau. (Prof. Ir. L. Huisman, Netherlands-Pays-Bas)	
1	**Low cost technology from the point of view of the members of the U.A.D.E.** Technologies à faible coût, le point de vue des membres de l'Union Africaine des distributions d'eau. (M. R. Dhaoui, Tunisia-Tunisie)	Paper awaited Articles en attente
2	**Low cost water supply—an appropriate technology for rural areas and urban fringes.** La distribution d'eau à faible coût—une technologie appropriée pour les zones rurales et les banlieues urbaines. (Dr. K. Erbel, Germany-Allemagne)	1
3	**Low cost streamgauging.** Le dosage des courants, à faible coût. (G.H. Mortimer, Great Britain—Grande Bretagne)	9
4	**Cistern based water supply in rural areas in low developed countries.** La distribution de l'eau à base de citernes dans des régions rurales de pays faiblement développés. (G. Schulze, Germany-Allemagne)	19
5	**Shallow wells and handpumps—a Tanzanian experience.** Puits peu profonds et pompes manuelles: une expérience en Tanzanie (R. Trietsch, Netherlands-Pays-Bas)	33
6	**Mechanically drilled wells and motor-pumps.** Puits à sondage mécanique et motopompes. (P.K. Cruse, Great Britain-Grande-Bretagne)	45
7	**A comprehensive water treatment station using no electrical or mechanical equipment.** Poste de traitement d'eau complet sans electricité ni mécanique. (J.M. Mignot, France).	68
8	**Design and fabrication of a low cost package water treatment plant for rural areas in India.** La conception et la fabrication d'une usine de traitement global de l'eau à faible coût pour les régions rurales en Inde. (Dr. A.G. Bhole, India-Inde).	82
9	**Compact waterworks—a low cost water treatment plant for drinking water supply for small countries.** Les centrales hydrauliques compactes—une usine de traitement de l'eau à faible coût pour la distribution de l'eau potable dans des petits pays. (W. Kretzschmar, Germany-Allemagne)	100
10	**Roughing filters as pre-treatment for slow sand filtration** Filtres de dégrossissement, comme pré-traitement d'une filtration lente sur sable. (M. Wegelin, Tanzania-Tanzanie)	110
11	**Various aspects of the optimisation of surface water treatment in tropical countries.** Divers aspects de l'optimisation du traitement des eaux de surface dans les pays tropicaux. (K. Voss & H. Gros, Switzerland-Suisse).	119
12	**Design of water treatment plants in developing countries** La conception d'une usine de traitement de l'eau dans les pays en voie de développement. (Dr. S. Kawamura, U.S.A.-Etats-Unis).	131
13	**The roles of upflow filtration and hydraulic flocculation in low cost water technology.** Les rôles de la filtration de surécoulement et de la floculation hydraulique dans la technologie du traitement de l'eau à faible coût. (R. Gregory, H.T. Mann & T.F. Zabel, Great Britain-Grande-Bretagne)	140

14 **Development of simple and economic filtration methods for rural water supplies.**
Développement de méthodes de filtration simples et économiques pour les distributions d'eau en secteur rural.
(J.N. Kardile, India-Inde). 157

15 **A training course in the optimisation of water treatment plant performance for operators.**
Cours de formation sur l'optimisation des performances des installations de traitement des e aux pour les techniciens.
(G. Wagner, U.S.A.-Etats-Unis). 171

16 **Optimal design of water distribution systems.**
Conception optimale d'un système de distribution d' eau.
(S. Solanki & D.K. Ghosh, India-Inde). 188

17 **Appropriate water pumping systems and their applicability in semi-arid regions in the Dodoma region in Tanzania.**
Systèmes appropriés de pompage de l'eau et leur applicabilité dans les régions semi-arides de la Dodoma en Tanzanie.
(W. Stiebens, Germany-Allemagne). 201

18 **Some aspects of employment of ferro cement in water storage construction**
Quelques aspects de l'emploi de ciment armé dans la construction de réservoirs d'eau.
(J. Gotze, Germany-Allemagne). Paper awaited Articles en attente

19 **Piped water supply to low income households.**
Distribution d'eau par conduites pour les ménages à faible revenu.
(S.F. Postma, Netherlands-Pays-Bas). 222

20 **House installations in urban developments—use of modern materials in an acceptable way.**
Installations domestiques dans l'aménagement urbain—utilisation de matériaux modernes d'une façon acceptable.
(J.T. van der Zwan, Netherlands-Pays-Bas). 227

21 **Studies on development and performance of fixed bed disinfector**
Etudes sur le développement et les performances des désinfectants à bain stable.
(C. Venkobachar & R.K. Jain, India-Inde). 239

22 **Scope and limitations of disinfection of water by U.V. Radiation.**
Etendue et limites de la désinfection de l'eau par rayons U.V.
(Dr. W.J. Masschelein, Belgium-Belgique). 252

23 **Water disinfection practice and its effects on public health standards in developing countries.**
Epuration de l'eau dans la pratique et ses effets sur le niveau de santé publique dans les pays en voie de développement.
(H.T. Mann, Great Britain-Grande-Bretagne). 281

24 **Demand, finance, payment for water supply in less developed countries.**
Financement et paiement de la distribution d'eau dans les pays les moins développés.
(R.T. Whiteley, Great Britain-Grande-Bretagne). 292

25 **The prerequisites for quality management at a low cost in developing countries.**
Les conditions d'une gestion de qualité à faible coût dans les pays en voie de développement.
(G. Arrestat, France). 297

26 **Renewable energy sources and water treatment.**
Sources énergetiques renouvelables et traitement de l'eau
(C.D. Ouwens & H. Beurskens, Netherlands-Pays-Bas). 303

Paper 2

Low cost water supply—an appropriate technology for rural areas and urban fringes?

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RESUME

Technologies à faible coût dans la distribution d'eau -
Technologies adaptées pour les zones rurales et suburbaines?

Au vu de l'énorme déficit d'approvisionnement d'une part, et des moyens budgétaires extrêmement limités d'autre part, il ne fait aucun doute que les objectifs de la décade internationale de la distribution d'eau ne pourront être atteints avec des méthodes et technologies traditionelles. La question qui se pose à nous peut donc être formulée ainsi: l'application de technologies à faible coût constitue-t-elle le moyen adéquat permettant de surmonter le décalage existant entre les ressources disponibles et les objectifs visés?

Il faut s'attendre à des réactions sceptiques et négatives provoquées dès l'abord par le seul emploi de l'expression "technologie à faible coût". Elles mettent en cause l'acceptation même de nos propositions par les Gouvernements et groupes-cibles des pays du Tiers Monde.

Cet exposé est donc consacré à l'étude des dangers, des avantages et des inconvénients éventuels, et par conséquent, à la problématique générale de l'application des technologies à faible coût. Les recommandations émises s'inspirent des expériences rassemblées au cours des dernières années dans le cadre de projets de la coopération technique allemande.

Même si l'on précise au préalable que l'utilisation de technologies à faible coût ne doit bien évidemment avoir pour seule finalité de restreindre les coûts d'investissement et même si l'on englobe sous ce terme le concept de "technologies adaptées", un certain nombre de questions critiques restent malgré tout en suspens, qui doivent faire l'objet d'un examen très sérieux et minutieux, avant que ne soient appliquées les technologies sélectionnées. Le premier complexe de questions à traiter se situe autour du problème de la disposition des administrations locales et des groupes-cibles à accepter les mesures proposées. Même si, par rapport à d'autres projets comparables de niveau international élevé, il ne s'agit "que de" technologies à faible coût, on ne peut espérer que la population sera fière de "son" approvisionnement en eau que si elle comprend pleinement le sens, le but et la réalisation de l'approvisionnement en eau prévu, participe au processus de décision et à la construction (p. ex. par des méthodes de construction faisant appel à de la main d'oeuvre nombreuse), et seulement si l'on tient compte des conditions socio-économiques, des us et coutumes règnant sur place. Le second groupe de problèmes important provient, lors de l'utilisation de technologies adaptées plus simples, des risques sanitaires possibles particulièrement élevés, surtout du danger de contamination secondaire de l'eau potable, si les utilisateurs ne sont pas informés sur les causes des maladies provoquées par l'eau. Il faut dans ce cas beaucoup de

patience et d'engagement personnel, et surtout, il faut savoir que l'installation d'un système de distribution d'eau n'est pas en mesure de résoudre, à elle seule, les problèmes d'hygiène.En troisième lieu, il convient de nommer les problèmes d'exploitation et de maintenance. De nombreux systèmes de distribution d'eau faisant appel à des technologies de faible coût sont certes, caractérisés, s'ils ont été bien choisis et introduits, par des frais d'exploitation peu élevés et un entretien facile. Mais si l'on n'a pas pris en considération une technique adaptée, on court le risque que l'un des éléments essentiels du nouveau système ne se détériore et que les utilisateurs déçus retournent à leur source traditionnelle d'alimentation en eau.

Les technologies à faible coût peuvent constituer la solution; cependant, une grande partie des sommes économisées doivent être réinvesties en "software", par exemple encadrement, formation et perfectionnement. Ce n'est pas une tâche facile pour les bailleurs de fonds, auxquels il est souvent reproché de ne pas comprendre entiérement les besoins propres fondamentaux des groupes-cibles pauvres; c'est par contre certainement la bonne stratégie pour les pays du Tiers Monde qui disposent de ce seul moyen pour promouvoir eux-même et de manière durable leur propre essor.

1. Introduction

The situation is desperate: nearly two thirds of the urban and rural populations of developing countries have no reasonable access to safe drinking water; more than half of all deseases in the third world are considered to be caused by inadequate water supply; 1000 to 2000 children die for this reason alone every hour. According to latest world-bank and WHO-estimates, some 200 to 300 billion US$ will be needed up to the year 1990 to reach the goals of the International Decade for Water supply and sanitation.

In view of these tremendous deficits in water supply services and the limited financial means of both developing and "donor"-countries it should be obvious to all concerned that conventional methods and approaches will not be able to close this needs-resource gap.

The simple and logical answer seems to be: make water supply projects cheaper, chose low cost technologies!

But is this really the practical solution to the problem? Or is it again one of these numerous optimistic and naive philosophies which are bound to fail because some essential limitations were not recognized?

And if so, what are those limitations?

To start with: The mere mentioning of the term "low cost technology" may cause representatives of both industrial and developing countries to shrug their shoulders and stop discussing the issue all together. Scepticism is particularly pronounced with those who argue to have - long ago - understood the "tricks" of development aid: isn't this selling cheap, second-hand stuff, "grandfather-technologies"? Isn't this the "basic needs"-line again? Isn't it charity for the poor who should be satisfied with whatever they get?

Are we proposing a two class system: advanced technologies for the priviledged, low cost for the others? Do people deserve a different, usually inferior water supply system just because they live in a certain region, e.g. in rural instead of urban areas? Are certain groups entitled to less safe drinking water than others? Is it fair that women or children have to walk miles to fetch minimal quantities of water whereas others, with piped water supply in their house, waste it? Are we even allowed to raise the problem of cost as a constraint in a matter of such vital importance as water?

These questions show that it has become difficult to argue about low cost technologies - even about those which have not yet been tried. But what can we do in cases where the alternative is: inadequate and unhygienic water or improved, low-cost water supply?

The following comments will concentrate on the questionmark in this paper's headline: are low cost technologies an appropriate way of reaching the goals of the Decade?

Leaving the proposal and discussion of specific technologies to other papers presented at this conference I shall limit myself to the more general aspects of the theme, taking into account some of the past years' experiences of German Technical Cooperation projects.

2. General limitations in the use of low cost technologies

The main general objections to using low cost technologies in water supply schemes of developing countries may be the following:

- scepticism or even suspicion on the part of governments, officials or other decision makers
- missing acceptance within the target groups themselves
- health-(e.g. contamination-)risks
- problems of durability and technical reliability in operation and maintenance

To streamline our argumentation we should probably set a few things straight before we dig deeper into the problem:

1) Low cost technology must never mean low investment costs only. All phases of a project have to be considered, from the planning stage to construction, operation, maintenance and repairs. Total costs during the lifetime of the scheme is the criterion to check. Similarly it must not be restricted to certain parts of a scheme: it should be applied to the whole cycle of the water, from the source to transport, storage, drainage and sewage disposal (sanitation).
2) Low cost has to mean more than just the total price of the scheme. It must distinguish between foreign currency and local currency costs. It must take into account whether or not the beneficiarieswill be able to pay for the provided services. It must also recognize the positive effects of creating jobs, for example in the construction phase.

3) Low cost technologies are not the answer to all water supply problems. For example they will have to be applied much more carefully in densely populated urban areas than in rural regions with decentralized services.

4) Low cost technologies, as a first solution in areas with rapidly growing or temporary populations, may well be followed by systems based on higher technical standards, as soon as population figures in the defined area become predictable and as soon as it has become clear that tariffs and other revenues are sufficient to pay for operation, maintenance, repair or even replacement.

5) Low cost by itself should never be regarded as the exclusive, first-priority parameter in deciding for or against a certain scheme. Instead of this onedimensional approach we have to look for a careful optimum mix of ingrediants which go far beyond merely technical matters. The magic word "appropriate technology" probably describes best what we have to aim for.

But even when taking these aspects into consideration there remain enough doubts, dangers and objections which have to be explained, avoided and overcome before we can embark on the low cost technology line.

Before we look at the most important constraints one by one, I would like to expand a bit on the last of the above mentioned points: What really is appropriate technology? What are its vital elements, applied to water supply?

In our opinion, four basic conditions have to be fulfilled: the chosen technology must be appropriate in socio-economic aspects, in regard to natural conditions, financial resources and in relation to potentials of material, know-how and man-power. This means taking into account not only technical and financial parameters but also cultural, ethnic or ecological conditions within the framework of the existing local situation. To formulate it very simply: appropriate technology means - more than anything else - acceptable technology. To become accepted we should carefully study, select, test and individually apply the chosen technology to make sure that it is simple, safe and sturdy enough to be understood and mastered by the target group.

Summing up, appropriate technology may be identical with low cost technology; usually it is much more.

3. Specific problems in using low cost technology

3.1 The acceptance-problem

As indicated above it has two components: a proposed water supply scheme must first of all be accepted or at least be tolerated by the "official level", be it a national or regional government, a city council or a local chief. Low cost technologies are particularly in danger of being rejected, because the public relation effect and domestic "political yield" are usually low. It would certainly be a mistake to disregard the opinion in certain decision making bodies. Sooner or later their negative attitude about a scheme would show unpleasant consequences. A lot of patience will be needed to explain the concept, the advantages of the chosen solution and the benefits to be drawn. Sure enough, in exceptional cases a scheme will have to be pushed even if the official level is not yet fully convinced of its future success. Even more important is the acceptance of the beneficiaries themselves: Examples are told of cases, where technical solutions in general and water supply schemes in particular are rejected by a target group, either right from the start or soon after implementation. Whereas the latter case will be dealt with in connection with the problems of operation and maintenance, I shall try to list some of the reasons for this initial very critical danger:

- the target group may not be convinced that they really need the new scheme; therefore they are neither interested nor motivated
- they are not informed about the plans; they don't understand what is going on
- their culture, religion, customs, superstitions are not sufficiently taken into account
- they don't get the chance to actively participate in the scheme
- they are afraid that the operation-costs for the new supply are too high.

What then can be done to ensure local participation and motivation? Some of the answers are:

- inform the target group as soon as possible; ask their views, let them take decisions, make sure they understand function and usefulness of the proposed scheme
- study the beliefs and habits of the people; find out what rôle certain members of a society play in collecting, transporting, storing or distributing the water (particularly women!)

- make the cost-aspect clear to the future water supply users. Explain investment cost alternatives, tariff-questions, necessary financial resources for repairs or replacement
- involve important members of the target group in the planning, construction and implementation phases (e.g. community heads like local chiefs, medical doctors, teachers, midwives)
- Encourage inventiveness at the local level, selfreliance and use of existant know-how; chose labor intensive methods, thus creating jobs. Employ local manpower, e.g. in the non-harvest season, particularly for unskilled services like trenching, masonry works and pipe-fitting. Use local materials and production capacities (e.g. for fittings, pipes).
- create motivation, a sense of ownership and pride so that they will protect the scheme and handle it carefully

3.2 The contamination problem

The use of low cost water supply technologies must be carefully checked against a possibly increased risk of health hazards. Success, acceptance and confidence will largely depend on whether or not contamination in connection with water-borne, water washed or water-based diseases can be avoided.

For instance:

- dug wells instead of drilled wells imply the danger of wastes or small animals falling into them
- if ropes and buckets are used instead of pumps, they themselves are very often the source of water-contamination
- if shallow aquifers are taken as source of water supply, infiltrating sewage e.g. from pit-latrines may deteriorate the quality of the groundwater
- if - instead of groundwater - surface water sources like rivers, streams, lakes or ponds are used for supply, there is an even higher risk of contamination by wastes or sewage
- if cheaper materials are utilized for pipes (bamboo is an extreme example) then leaks, bursts and disruptions of flow become more likely, again causing secondary contamination

First of all it is important to make the users of water supply understand how contamination is caused and how it can be avoided.

This means that education in basic hygiene should start before or at least during the implementation phase of a water supply scheme; also that measures of waste disposal, basic sanitation and control of sewage should accompany a water supply project and, if possible, housing and general living conditions should be improved simultaneously with water supply and sanitation.

In addition, the following specific examples can be given to minimize risks:

- Dug wells should always be covered; to lift the water hand pumps should be installed wherever possible. If the users insist on traditional lifting devices, then only one regularly checked and cleaned rope and bucket should be used. The area around the well should be paved and properey drained.The walls of a dug well should be raised to a height of at least half a meter above ground; animals will have to be kept away, troughs should be constructed for watering

- Special precautions have to be applied if roof catchments, springs, ponds and cisterns are used for collecting and storing drinking water. They have to be protected against animals and not be used by human bathers because of the particularly harmful pathogenic organisms which might thus enter the source of water supply
- If instead of being transported by pipes water is carried and stored in containers (calabashs, buckets, clay-pots), people should be told to cover them, clean them regularly and keep them out of the reach of children and animals in the house; leaves may be used to protect the water against dust and other airborne impurities, some seeds and barks are known to eliminate bacteria by coagulation and flocculation. Men and women should also be tought that water must not be drawn from these containers by hand, but that special (e.g. wodden) scoops should be used.

To summarize: The use of low cost water supply technologies implies increased efforts in basic hygiene education, if health hazards are to be avoided. A simple technology on the other hand facilitates the understanding of the users, how the contamination problem can be kept in check; in this way there is a good chance to more than compensate for certain accepted deficits in standards, reliability or safety.

3.3 Operation and maintenance

Statistics claim that, on average, three years after construction more than half of all public water supply schemes in rural and urban fringe areas of developing countries are no longer functioning. Will this "death-rate" be even higher in the case of using low cost technologies?

Quite likely, if the users do not take proper care of their scheme! As mentioned before the main danger of any technology being rejected consists in the loss of confidence, which occurs if a scheme fails soon after implementation. In the case of water this will usually mean that people go back to their old sources of supply, with the consequence of particularly high health-hazards.

Some low cost technologies imply special efforts in operation, maintenance and care, in other words: saved financial means later on call for increased human input. On the other hand, carefully chosen and correctly introduced simple technologies usually have the advantage of low operation - costs and easy repair. It all seems to depend again on the acceptance-problem: if the users understand their scheme and if they are able to handle it with local means then the use of low cost technologies will rather increase than decrease the life-time of a water supply.

One of the prerequisites of successful operation are tolerable operating costs, in the case of water supply mainly energy costs (for lifting the water, resp. to guarantee proper pressure) and costs of water-purification. Great efforts will be necessary to counteract the spiralling price of oil. Some alternative (renewable) sources of energy like wind are competitive and ready to be introduced (or re-introduced), others - like solar cells - need more research and development before they can fulfill the criterion of being "low cost".

And, of course, there is still the alternative of human hands or animal-power instead of diesel - or electric pumps, with the desirable side-effect of reducing the waste of water and, at the same time, being absolutely reliable.

Similar positive effects are connected to using simple means of water purification like slow sand filters or chlorination by bleaching powder: basic knowledge of how these methods work is usually sufficient to find solutions to all everyday problems occuring during operation.

Summarizing, the choice of low cost technologies must incorporate a careful check to find out if problems of operation and maintenance will be manageable. Those parts of a water supply scheme which are known to be particularly vulnerable (e.g. handles of hand-pumps) must be simple, sturdy, easy to repair and replace and - if possible - locally producable.

Facilitating o + m has to be part of an integrated package of project activities, namely information and instruction for the users, training of local craftsmen; standardizing parts, maintenance and repair-procedures; most new schemes will initially need the assistance of advisors or supervisors.

It is our impression that these activities and programmes have not received enough attention in the past and that, to reach the goals of the Decade, they might again be underestimated. The reason is possibly that courses in maintenance are so much less spectacular than constructing and publicly presenting a new water supply scheme. In this respect both donor- and local organisations will have to shift their priorities: less financial input into the "hardware", more attention to activating and motivating manpower-ressources.

4. Conclusion

In spite of all scepticism and possible problems: the use of low cost technologies in water supply seems to be the only promising way to reach the goals of the Decade - certainly for rural areas, occasionally in urban fringes, only exceptionally in the case of densely populated urban centres. But there is one condition: low cost technology must at the same time fulfill the criteria of "appropriate technology"; otherwise, although being a step in the right direction, it will in many cases prove insufficient. Risks ans problems are not restricted to the three fields mentioned in this paper, but in discussing acceptance, health-hazards and operation + maintenance, one general aspect of low cost technologies should have become obvious: they are a two-sided affair. On one hand their use implies increased dangers, on the other they have the advantage of making exactly, these problems more transparent and thus corrective measures easier.

Paper 3

Low Cost Streamgauging

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U.K.

INTRODUCTION

The number of times a water supply scheme fails in its lifetime is dependant on the reliability of the water source and the extent of the storage facilities. A realistic assessment of these two factors can be made only with long reliable records of appropriate data and often this is not available.

The estimation of reliable yields from surface water sources requires a long-term programme of flow measurement of streams and rivers. A wide range of measurement techniques is available and generally the more expensive the system the greater the accuracy, but is accuracy the only criterion?

PRINCIPLES OF RIVER GAUGING

The method most commonly used is to continuously measure the water depth and by a separate process find a relationship between depth and discharge. By placing a weir or flume in a river, the depth/discharge relationship is known (1), but the construction costs of this type of structure can be high. An alternative method is to measure the velocity of the river and its cross-sectional area on a number of occasions so that gradually a depth/discharge curve can be established.

Water depth can be measured continuously by a recorder or read manually at regular intervals from a staff placed in the river. Velocity can be measured by a current meter or floats.

DEPTH RECORDER VERSUS STAFF READING

A depth recorder produces a continuous record on a chart or tape which when it is operating correctly will yield data in a form suitable for analysis. A good quality instrument will cost more than $800, but the housing, which must be weatherproof and well ventilated, will cost much more than this. Both the instrument and the stilling well will require regular maintenance.

Measurement of water depth by reading a staff is dependant on a conscientious observer. Daily readings are often sufficient, but a weekly reading is acceptable if the staff is coupled with a maximum depth recorder. This can consist of a solid float with a hole through its centre. A threaded rod passes through the hole such that the float can slide up the rod but a simple ratchet stops the downward movement of the float. A different type (2) uses powdered cork in a basket set in a tube. As the water level rises in the tube the powdered cork floats on the surface. As the water level falls its maximum height is marked by powdered cork left on the tube sides.

CURRENT METER VERSUS FLOATS

Careful measurement of discharge using a current meter should produce an accuracy of ± 4% (2) when compared to the 'true' value. A team of two or three people are required to take these readings and at intervals the currentmeter has to be returned to the manufacturer to check its calibration. For wide or deep rivers, a boat or a cableway will be required to position the currentmeter.

Velocity estimation using floats was first suggested by Leonardo de Vinci in 1649 (3) and in 1763 floats were used to estimate the flow in the Rhine (4). Many different types of float have been proposed (5),(6) and a number of site techniques have been used successfully (7),(8).

Two main criteria are necessary for a good float; it must travel at approximately the same speed as the water and it must be easily seen. Many common objects can be used as floats, for example, wooden sticks, partially filled beer bottles and oranges. Velocity estimation using floats will not be as accurate as current metering and results from work carried out on the differences between the methods is shown below.

Work carried out by	$\frac{V_{cm} - V_f}{V_{cm}} \times 100$		
	mean	max	min
Griffith (9)	-3.1	+2.3	-12.5
Nemec (10)	within a range of 15%		
Loughborough University	-5	+4	- 10

where V_{cm} = Velocity by current meter

V_f = Velocity by floats

Generally the float method tended to under-estimate the flow by about 4% (as compared with the current meter) and the total range of errors is about 15%.

WHY USE THE FLOAT/STAFF METHOD

The two main advantages of this system are the low capital costs and the simplicity of the equipment used. The disadvantages of the method are loss in accuracy and frequent site visits if the gauges are to be read daily.

The floats and measuring staff can be obtained easily in most countries and mechanical breakdowns are unlikely whereas automatic depth recorders and currentmeters have to be imported into many countries and breakdowns and malfunctions can occur for a host of reasons. The simplicity of the former method may produce a longer and unbroken flow record.

An important feature of the float/staff method is that the velocity measurement can be carried out at very regular intervals by semi-skilled staff, so the depth/discharge curve can be continually updated. This may be important on many rivers particularly in the semi-arid areas where the changing river profile can introduce enormous errors in the measurement of discharge (11).

EFFECT OF DATA ACCURACRY ON FLOOD (OR DROUGHT) ESTIMATION

Every river and gauging site is unique so it is not possible to make any definitive rules to cover all situations. However Appendix A shows the details of an exercise carried out to assess the effect of accuracy (or the lack of it) on the estimation of large floods. The aim of this work was to discover if ther are cases where a longer, less accurate record could be of more use than a shorter accurate one. The results shown in Appendix A are best illustrated by an example. Looking at figure 3a in the Appendix, with current metering methods, 10 years of data would be required to produce the same confidence limits ($2 \times S_p$) as 14 years of data by floats, if $C_v = 1.0$. More than 4 years of additional data derived from velocity measurement by floats could produce narrowerr confidence limits and so a more reliable estimate of design floats. For a river with $C_v = 0.2$ the record lengths for the same confidence limits are 10 years for the current meter record and 17 years for the floats.

What values of C_v for annual maximum flows occur in practice; some examples are shown in Table 1.

Table 1 - C_v values for various rivers

River	Country	$\frac{\text{Standard deviation}}{\text{mean}}$	Ref
Mekong	Vietnam	0.13	(15)
Indus	India	0.28	(15)
Thames	England	0.38	(16)
Paraiba	Brazil	0.3	(17)
Blue River	U.S.A	0.74	(18)
57 Rivers	Britain	0.21-0.93	(19)
Zambezi	Africa	0.4	

Investigators have found difficulties relating C_v to catchment characteristics or climate (20). In Britain generally the larger flatter catchments have low C_v values.

ARE LOW COST RIVERGAUGING TECHNIQUES USEFUL?

It is dangerous to draw general conclusions since each river has a unique flow distribution. The problems associated with measuring these flows are different for each station and for each country.

There are two situations where low-cost rivergauging may have advantages.
1. If a fixed amount of money has been allocated for setting-up a number of gauging stations for future assessment of water resources in an area, then the money can be spent on a limited number of conventional stations or a larger number of low-cost gauges. The solution is not a simple one since the number of potential sources is also an important factor.

An area with few gauges often requires flow data to be 'transferred' from a gauged river to one with no record and this process of transference can involve substantial errors (a). If many low cost gauging sites were installed, the problem of lack of data on any one river would be reduced.

2. Where money is restricted in a water undertaking it should be possible to set-up a low cost gauge at an early stage of a regional water development plan since the cost would be only a fraction of the more common 'expensive' station.

On many rivers if this type of gauge can be set-up 4 or 5 years earlier than the more expensive system then there could be little difference in the accuracy of the estimation of flood and droughts.

FINAL COMMENTS

It is not my aim to suggest that low cost streamgauging should replace the commonly used currentmeter/depth recorder method, but there are many situations where it should be considered more seriously than at present. Indeed in some areas the floats/staff method could produce 'better results' in situations where money for gauging is restricted.

APPENDIX A - THE EFFECT OF ACCURACY OF DATA ON STATISTICAL EXTRAPOLATION

The aim of this appendix is to show that under certain conditions the same 90% confidence limits associated with the estimation of an extreme flood can be obtained either by extrapolating an 'accurate' record, say by (current metering) of T years or using a less accurate record by (floats) of a duration greater than T.

In this exercise the following assumptions were made:

1. the annual peak flows conform to a Gumbel Distribution (13)
2. the errors incurred in the measurement of the cross-sectional areas of the river are the same for both methods
3. the depth measurement can be read to the same degree of accuracy by both methods
4. the errors in the velocity measurements by floats lie within the range of ± 7% of the measurements made by currentmeter.

The method of extrapolation first proposed by Chow (14) is used to calculate a peak flow $Q_p(T_r)$ associated with a return period of T_r years and the confidence limits $\pm\, C(T_r)$ associated with the extrapolation. Two return periods were considered, 1 in 100 years and 1 in 30 years.

One factor affecting the estimation of $Q_p(T_r)$ is the length of the record L_r. This factor also governs the magnitude of the confidence limits.

In the extrapolation, L_r and T_r are used to calculate K the frequency factor; $Q_p(T_r)$ can then be calculated using the equation

$$Q_p(T_r) = \overline{Q}_p + KS_p$$

where $\overline{Q}_p$ = mean of the annual peak flows
S_p = standard deviation of the annual peak flows

Associated with the value of $Q_p(T_r)$ are the 90% confidence limits $\pm\, C(T_r)$ which are calculated in terms of S_p.

The flow values can be plotted on Gumbel Distribution graph paper and an example is shown by line AA in figure 1. Associated with line AA are the upper and lower 90% confidence limits BB and B'B'.

Assuming line AA is determined by measurements made by currentmeters, then the values calculated from float reading will be contained in a band of ± 7% of the section of line AA covering the measured values. The upper and lower limits of this band are shown by lines BB and BB ($\pm\, C(T_r)$).

For the method using floats the worst case in calculating the confidence limits would be to consider line CC as the true distribution and to calculate the upper 90% confidence limits for this case giving line DD.

If the lower 90% confidence limit is calculated for line $C'C'$ in the same way, producing line $D'D'$, lines DD and $D'D'$ represent the limits of approximate estimation of the errors involved both in extrapolation and data measurement.

The upper limit (CC) of the error band was calculated by assuming the largest measured annual flow had an error of + 7% and the error of the smallest annual flow in the record was - 7%. These two values were connected by a straight line which is used to define line CC beyond the range of measured flow values. For the lower limits the largest measured flood measured by floats was assumed to have an error of - 7% (compared with currentmetering) and the error of smallest flood was taken as + 7%, procedure used to obtain the extension of line CC was then followed for this case and this extension to line CC was drawn. See figure 2.

From graphs similar to figure 2 the errors associated with measurement were read off for different return periods. For example for record length of 5 years the errors are:

Return period (years)	approx % error due to measurement
20	± 10
30	± 12
50	± 13
100	± 14

Record lengths from 10 to 30 years were taken and the 90% confidence limits were calculated for the currentmeter values for 1 in 100 year and 1 in 30 year events. These limits are represented by the vertical distance between lines CC and $C'C'$ on figure 1 and are expressed in terms of S_p. The 90% confidence limits for the extrapolation of float values (and data errors) were calculated for the same record lengths as used previously. In this case the limits were represented by the vertical distance between lines DD and $D'D'$ but which incorporated an additional variable coefficient of variation C_v ($C_v = \frac{S_p}{\overline{Q_p}}$).

Both sets of results were then plotted on a graph and are shown in figures 3 and 4. For explanations of the use of these figures see the section entitled 'Effect of data accuracy on flood estimating'.

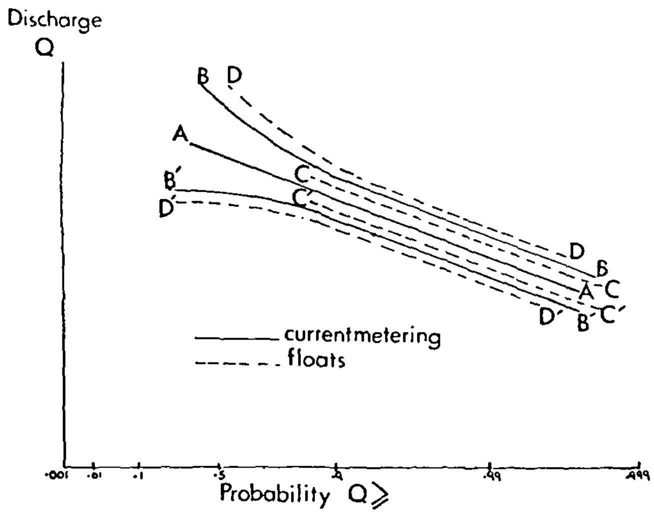

Figure 1 Confidence limits for floats and currentmeter

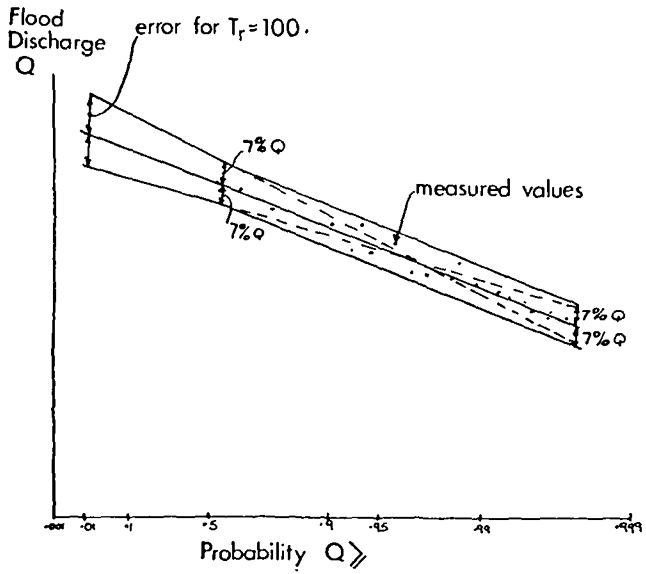

Figure 2 Method of calculating error bands for the float method

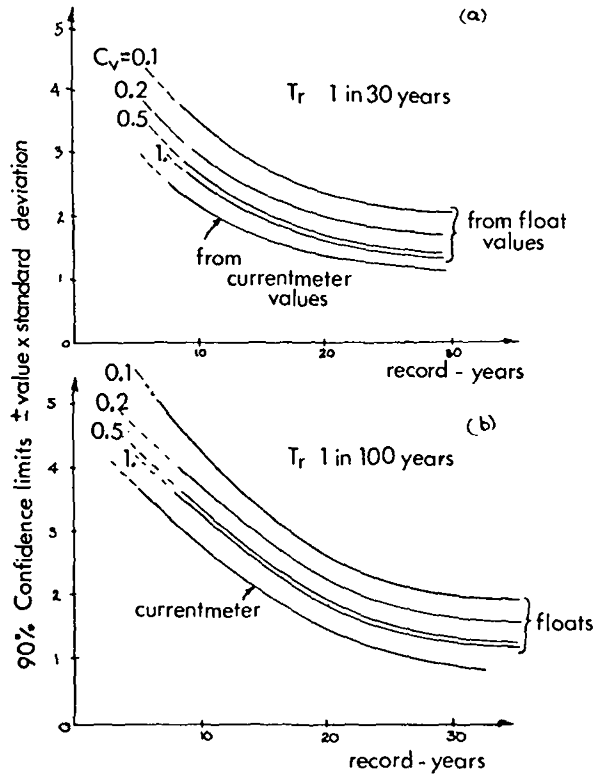

Figure 3 Relationship between confidence limits and record length for floats and currentmeter

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Jaugeage des Cours d'Eau à Faible Coût

RESUME

Une évaluation de la production et de la fiabilité d'une eau de surface nécessite un enregistrement exact et sur une longue durée des données relatives au courant ou au débit du fleuve. L'exactitude de ces données est importante mais un facteur tout aussi important est le nombre d'années pendant lesquelles cet enregistrement a été fait. Pour estimer les extrêmes hydrologiques (inondation et sécheresse), un enregistrement de longue durée mais "moins exact" peut, dans certains cas, fournir une meilleure estimation des courants exceptionnellement forts ou faibles qu'un enregistrement "plus exact" mais moins long. Dans ce cas, l'enregistrement "plus exact" est presumé avoir mesuré le courant du fleuve par compteur de courant et enregistreur de profondeur et l'enregistrement "moins exact" est obtenu en utilisant des flotteurs pour mesurer la vitesse et une perche pour mesurer la profondeur.

Le coût de l'installation d'un système avec flotteur et perche est évidemment moindre que le coût d'un enregistreur de niveau/compteur de courant ; donc si une méthode moins onéreuse peut être installée au premier stade du développement d'une ressource en eau ou si elle est plus sûre, elle pourrait avoir des avantages dans l'analyse hydrologique des données. Ou encore il pourrait y avoir une situation donnant à choisir entre un grand nombre de jauges bon marché et un petit nombre de jauges coûteuses. Un plus grand nombre de jauges pourrait reduire le besoin de transférer les données d'une prise d'eau à une autre - ce qui peut parfois entraîner des erreurs grossières.

Dans plusieurs pays, et depuis quelques années le jaugeage des courants utilise des techniques de plus en plus sophistiquées et de plus en plus onéreuses, mais dans de nombreux endroits le jaugeage des courants a bas prix peut être une solution avantageuse.

Paper 4

Cistern Based Water Supply in Rural Areas in Low Developed Countries

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PREFACE

This lecture treats the possibility of water supply based on rainwater; a simple, old, indigenous technology, which has a limited importance today but whose reactivation, so my pledge, is worth a discussion.

INTRODUCTION

Symptomatic for arid areas is an acute scarcity of water. The rate of evaporation is higher than the rate of precipitation which is very low as it is. Formation of surface water is, therefore, not possible. A natural storage of trickeled water is also impossible, then the low intensity and duration of precipitation, hastens the capillary rise of trickeled water and its evaporation from the surface. Is groundwater available inspite of these unfavourable conditions, its sources are to be found so deep, that its gaining is only possible with highly developed tools and techniques, whose acquisition and service is hardly to be realised without help from industialised nations.

In semi-arid areas one can clearly differentiate between wet and dry periods, whereby the rate of rainfall lies below 600 mm/year. Surface water can serve as a source of water supply only during the rainy season and the construction of long-distance pipelines is a possibility in certain cases. This is, however, a capital intensive solution for a developing nation. Thus, collection of rainwater and its storage in cisterns, can often be an only way out for arid and semi-arid areas.

In areas of the Mediterranean and the surrounding arid belt, the cistern construction is traditionally spread and has been preserved to the day. Stonemasons, masons and potters, who are needed for the construction of cisterns, are trades traditionally practised in these regions.

In coastal areas and islands, where groundwater is brackish and salty, the usage of rainwater can be a simple solution. Good results with this technique could be obtained in areas whose air pollution does not play much of a role. Thus rural areas are particularly suited for an application.

By improving water supply systems one should consider the social implications. In many countries women are responsible for the upkeep and the health of the family. The acquisition of vital water is part of this task. Water is often carried home over long distances.Women should not give up the task of acquiring water, rather should be integrated in water supply projects and be entrusted with the new technology. Considering this aspect the cistern based water supply seems to be appropiate as it can be easily be learned and understood. An aversion to traditional constructions, materials and trades as well as an overestimation of western industrial goods can only be overcome through convincing methods of water supply.

DEMANDS ON CISTERNS AND THEIR COLLECTING SURFACES

Detailed planning and estimation is necessary for the construction of a cistern. Knowledge of distribution, frequency and intensity of precipitation has to be gained. In certain developing countries, however, one can hardly expect to find statistics on this aspect over a long period of time. An estimation is therefore to be made. The determination of the rate of evaporation can be carried out by direct measurements for example with an evaporation pan. The material of the collecting surface should be smooth and weatherproof. Porous materials have to be avoided because these lead to absorption losses.

The daily requirements are a function of the number and habits of the persons to be supplied. The required capacity of the cistern can be calculated from the input summation curve and the consumption summation curve, whereby the input curve consists of the difference between the precipitation curve and the evaporation curve. Furtheron, it has to be found out how the water demand over the highest possible duration of a dry period can be covered. The collecting surfaces needed herefor have to be in an appropriate relation to the amount and frequency of precipitation as well as to the demand on water. The resulting size of houses (in case roof surfaces are used for collection) has to be within the financial possibilities of the inhabitants.

Location

Cisterns may be used for isolated houses and smaller village communities. An already existing stone or mud pit and a steep, impermeable surface, for example on a slope, is convenient for the construction of a central arrangement and it would reduce the construction costs (see fig 1). The pipelines leading to the houses have to be installed. In spread out villages the costs for the laying out of pipelines would rise. It has to be investigated whether a decentralised system were more convenient in such cases. The inhabitants of the single houses would then be responsible for construction, running and cleaning. In case of central cisterns, these activities were to be organised by the village community.

Fig. 1. Natural cistern in the Negev desert

The most convenient place for a cistern is the northern slope of a hill or the centre of a shady yard. The cistern has to be underground or at least be covered, in order to reduce not only evaporation and impurities but also to protect water from heat and sunlight which accelerate the formation of algal growth. The place of erection has always to be chosen in such a way that it is hygienically unobjectionable, namely far enough from sewage water disposal. The cistern has also to lie in a place unaffected by highwaters and floods.

Construction

Suited for collecting surfaces are roof surfaces, terraces or other suitably prepared surfaces like smooth rock, whose crevices have been previously filled to watertightness (see fig. 2). The material has to be such that it does not deteriorate the quality of water. It should consist of roof tiles or glaced clay pans. Asbest, roofing felt, reed, straw or cement pans are unsuitable however /1/. The collecting surfaces should have an adequate inclination to guarantee a rapid drain of rainwater. The roof gutters and collecting grooves should also have a continous inclination leading to the drainpipe. Puddles are to be avoides as these attract insects /2/. Hot climates are ideal breeding places for germs and germ carriers.

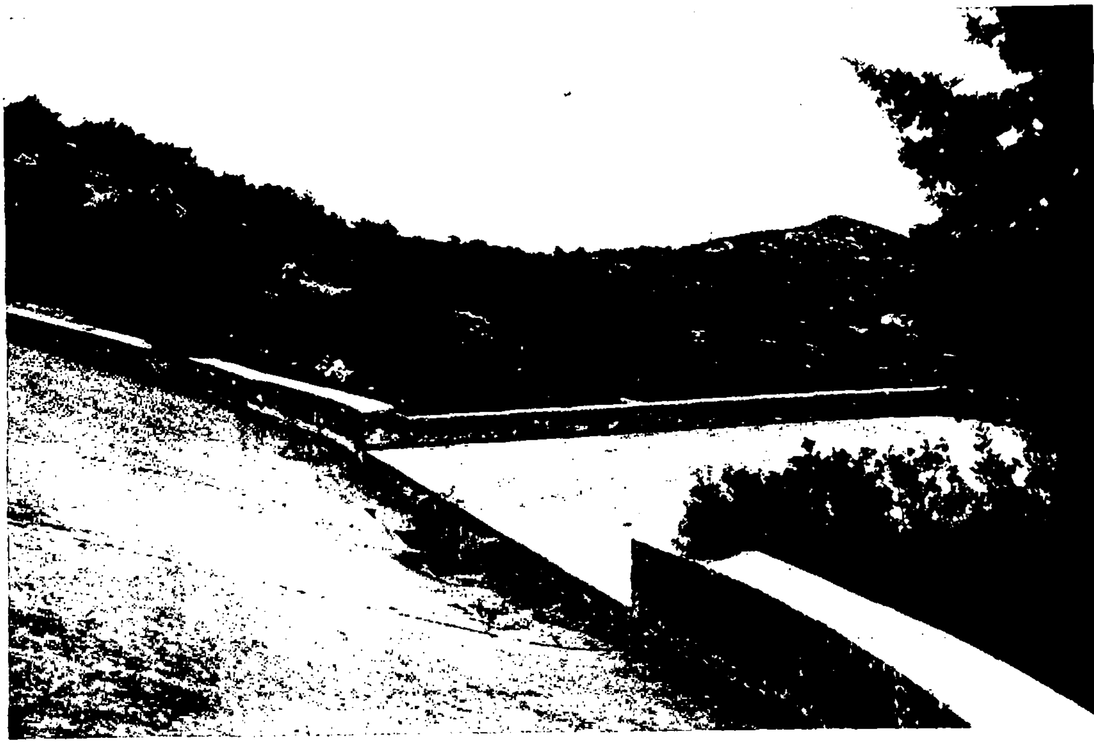

Fig. 2. Collecting surface and inlet of a cistern in Yugoslavia

The drainpipe has to be a closed pipe and not of wooden materials or lead /2/. The collecting pipeline has to be fitted with a sieve with a view to filter gros impurities like leaves, etc. The construction has to be such that a light and handy cleaning is possible. A switchable clap which allows a bypass of water around the cistern at the beginning at the precipitation and the leading of later rainfall to the cistern is of convenience /3/(see fig.3).

Cisterns are to be constructed possibly as underground storage reservoirs. The temperature of water is reduced due to the isolating effects of the walls and the surrounding earth. The form has to yield a maximum of capacity. On the other hand, the bottom and walls of the cistern in contact with water as well as the surface of water has to be at a minimum, so that loss of water due to trickling and evaporation is kept at limits.

The vault span of a masonary cistern has to be small, in order to reduce the danger of crack formation. If larger capacities are needed, then it is better to have a number of statically indepen dent chambers. Uneven consolidations and cracks have to be avoided, or else the watertightness of the container is reduced.

Fig. 3
Switchable clap

The base of the cistern should have an inclination. This leads to the collection of sedimented soil particles at a dead point, where the soil mineralises and suffers a considerable reduction of volume. The suction tupe of the pump should end at a height of 20 to 50 cms. from the bottom in order to avoid the transport of sediment (see fig. 4).

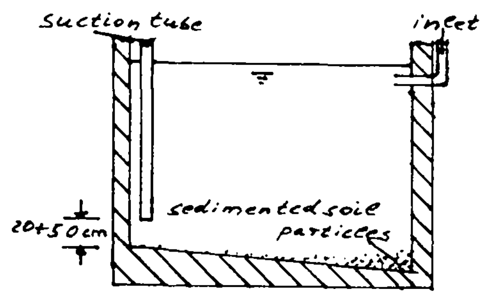

Fig. 4

The in- or outflows have to be far apart in order to guarantee renewal of water. The fitting of an overflow is absolutely necessary. It can serve as an overflow for the filter as well as for the container at the same time and is to be fitted with insect-proof metal gauze.

In case of two-chambered cisterns a manhole opening for cleaning purposes has to be built (see fig. 5).

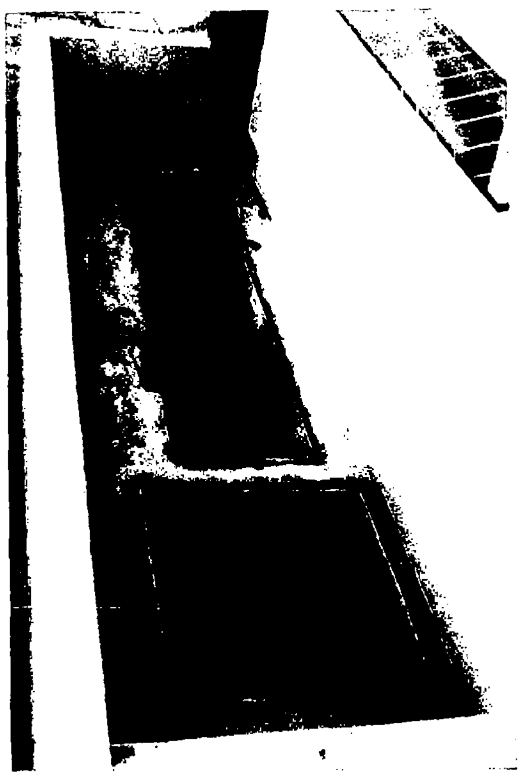

Fig. 5
House cistern
Helgoland
Filter and manhole

The lid should be of impermeable material and has to be fitted with a tightening to block an entry for vermin.

Function

Precipitation water is rich in oxygen and contains carbon dioxide, gases which have dissolved in the rainwater in the atmosphere. The pH is acidic. The additional carbon dioxide, a bye-product of biological processes remains above the water level in the cistern together with the former carbon dioxide. This improves the taste of the cistern water. Besides, carbon dioxide maintains the freshness of water for months together, thanks to its desinfecting properties.

The treatment measures used are sedimentation and filtration. The cistern are to be cleaned periodically for example once a year after a dry period. Cleaning is to be carried out the latest when the bad taste of water is noticed. A defective system will demand more frequent cleaning. This does not repair the defect, however.

On the other hand, examples are known, where cleaning has been proved unnecessary. The biological processes playing a role in a cistern have not been researched yet. A large one-chamber cistern with an outer filter and without an inlet shaft (for cleaning purposes) was opened after a running period of 30 years without interruptions for cleaning purposes or otherwise. A rigid, black 5cm-thick layer of sedimentation silt was found. Observations under the microscope showed just a sparse spread of bacterial and algal growth. The sedimentation silt obviously carries out important biological functions in the cistern and should not be therefore removed from an already running system without further considerations. Some users introduce the sedimentation silt after cleaning back into the cistern /4/.

A 2%-Soda solution can be used for cleaning. Suitable for the annexed desinfection are chlorine preparations. Before the revision of the cistern is carried out, it has to be made sure that there is enough oxygen in the cistern. Before the beginning of cleaning the carbon dioxide, a bye-product of biological processes, has to escape.

CONVENTIONAL CONSTRUCTION TYPES OF CISTERNS

Venetian Cisterns (see fig. 6)

This type was to be found in Venice before the introduction of running water system. The collecting rainwater flows into an inlet channel built around the cistern. It has to be layed out to possible heavy rainfall. Thereafter, the rainwater flows through a sand filter for purification. It can be then scooped or pumped off, out of a 4 m deep well-like shaft. The inlet openings are found at the lower third, so that the water describes a long as possible way.

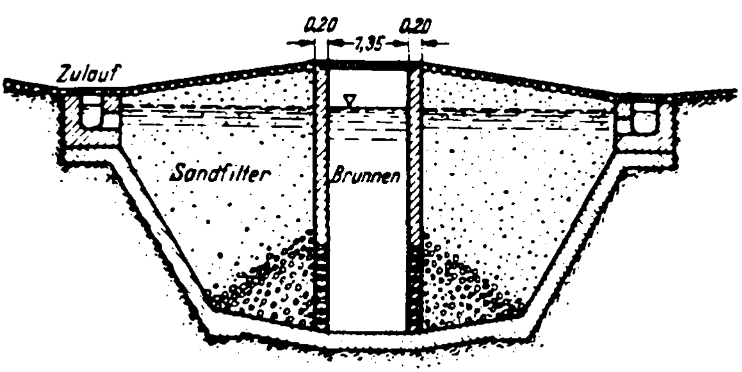

Fig. 6

The cleaning efficiency is thus increased. The effective volume of the cistern depends on the volume of the pores of the filter sand. The measurements are therefore relatively larger, as the pore volume amounts to around 15-20% of the total volume. Small trays and shallows, which have been previously sealed and filled with sand, are best suited. The trickling depth is however limited due to the seal. In this cistern the same phenomena take place as in a sand soil. The retention of the purified water in the well, can lead to contamination. A purification carried out just before consumption is recommendable. The well should be covered to avoid evaporation and impurification. One or more of these cisterns could find application in villages or smaller towns if proper sand and other construction material is available /5/.

"Purified Water Tank" Cistern (Siphon Cistern) (see fig. 7)

In this type, the filter and storage chambers are separated. Besides the mechanical filtration, a biological filtration is possible in the sand filter. However, the filter skin dies away in the dry periods, when a continous flow does not take place. The cleansing effect is than relatively low. Here too, a longer retention of the purified water can lead to contamination. A further disadvantage is the aeration of the purified water and the escape of carbon dioxide from the storage tank which can also lead to contamination./6/.

The construction of the vault with the filter load lying above, can be hardly carried out with the construction materials available in developing countries.

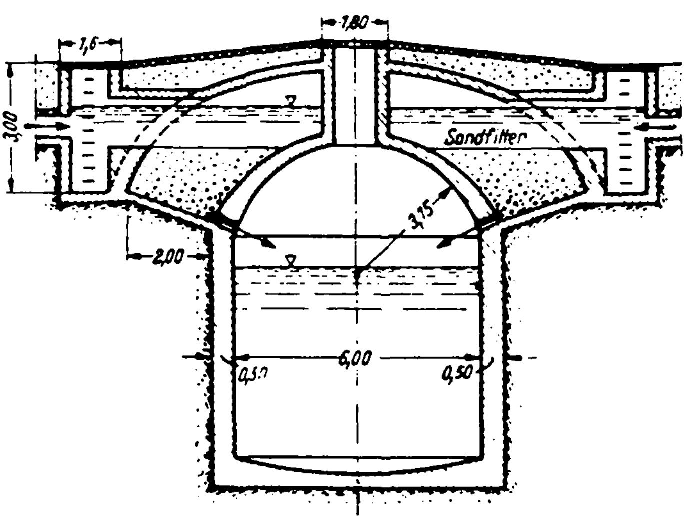

Fig. 7

American Cistern (see fig. 8)

The particularity of this cistern is the filter basket which stands free in the vessel. The water is filtered just before being drawn. The filter basket has a short filter length in comparison to the former types. The purification effect decreases with time. The filter can then be removed through the opening and the filter material either recharged or cleaned. The opening has therefore more than one function. It serves to draw water, for the removal of the filter basket and as an entrance to the vessel for cleaning and repairs /7/. This cistern can be placed inside or in front of a house. A periodical maintainance is however indispensable.

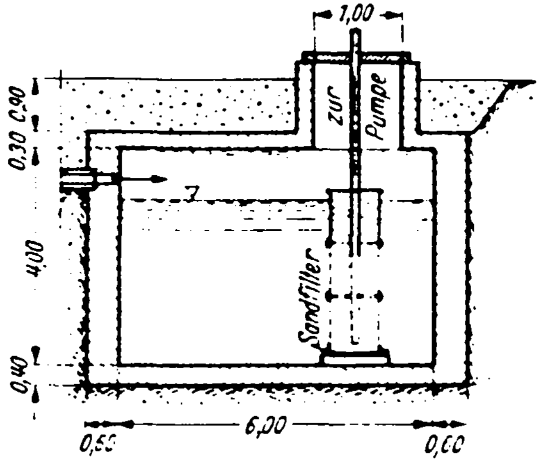

Fig. 8

German Cisterns

A striking property by these types is the two chamber principle. The collecting rainwater flows at first to the storage chamber. Sinkable particles found in the rainwater can undergo sedimentation in this vessel (see fig. 9).

The second chamber serves as a storage for the water which is needed for the immediate consumption. A continous purification takes place in the filter. Often a porous wall is used as a separation between the two chambers. Gravel and charcoal layers are used in Germany as filter material /8/ (see fig. 10).

A great disadvantage lies in the difficulty of exchanging of filter material. An exchange is only possible after emptying the cistern /9/ (see fig. 11). Cleaning is necessary after a running period of 4 to 6 years, as experience shows. The best time for thus operation is the end of a dry period.

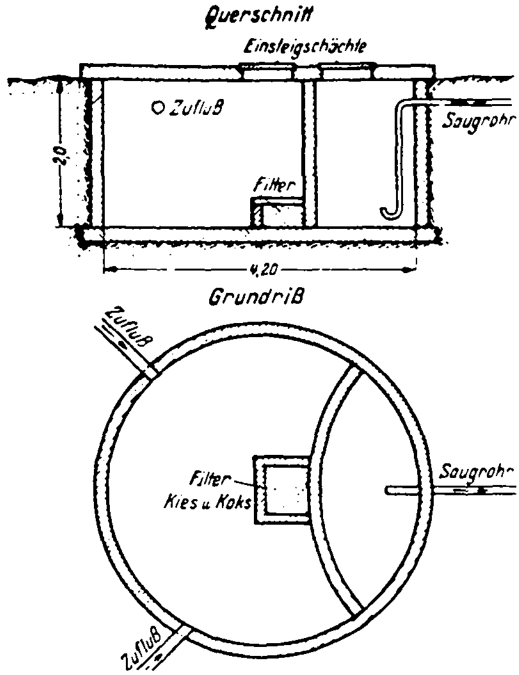

Fig. 9. German cistern form 1

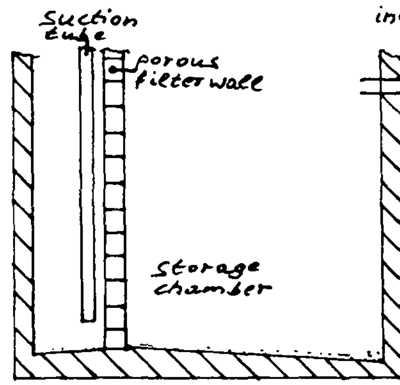

Fig. 10. Cistern with porous filter wall

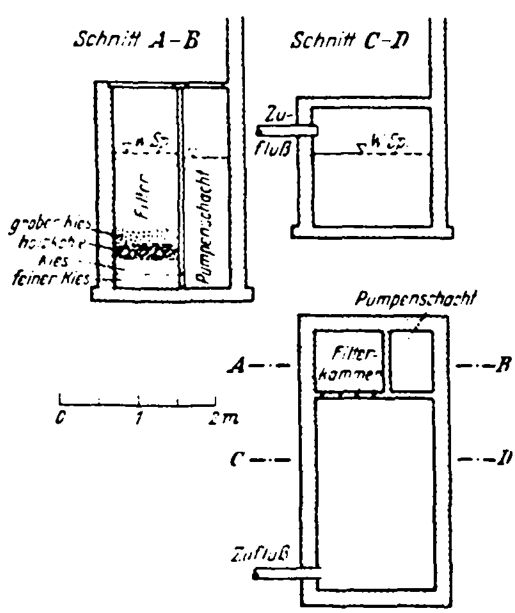

Fig. 11. German cistern form 2

SLOW SAND FILTRATION FOR RAINWATER TREATMENT

The crucial point by running cisterns is the hygienic point of view. It is difficult to keep the collecting surfaces clean, particularly in the dry periods and this is why they are a focus of germs. Contacting the collecting surfaces the precipitation water contaminates. Long duration of storage leads to formation of germs. Considering these points the purified water should be stored in a relatively small reservoir and be used as soon as possible. The capicity of the clear water reservoir should be such as to store the water consumption of one or two days. The raw water is stored in a large storage reservoir, where suspended particles sediment. The process of sedimentation is the more efficient the longer the water is retained in the reservoir. Having passed the storage reservoir the water is purified in a slow sand filter, which runs continously. The quantity of water to be treated in the filter corresponds to the daily consumption (see fig. 12).

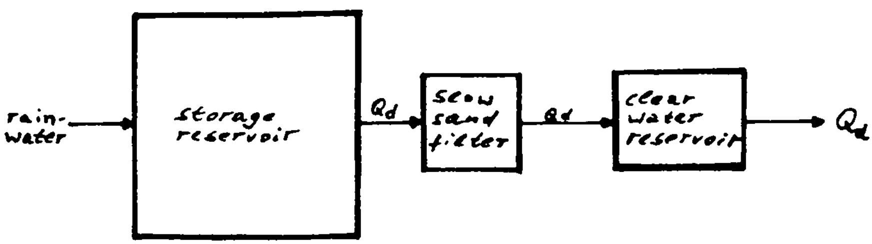

diagram of rainwater treatment

Fig. 12

The various processes that take place in a slow sand filter are physical, chemical and microbiological processes. An adsorpative film consisting of suspended particles and bacteria grows at the top of the filter sand. The raw water must pass this filter skin before reaching the filter medium itself /10/.

A slow sand filter may run for months without cleaning. As soon as the filter-bed resistance has increased to such an extent, that the filtration rate is reduced, it is time to clean the filter. The suspended particles which have been adsorped at the top of the filter-bed, can be removed by scraping off the surface layer to a depth of one or two centimeters /11/.

In slow sand filtration the rate of velocity should be kept below 0,2 m/h. The filter-bed, which is at least one meter thick, is conposed of washed quartz sand. The revised DIN 2001 (which will be published shortly) will recommend a grain size of one millimeter /12/.

LOCALLY AVAILABLE MATERIALS AND CONSTRUCTIONS

The following properties of traditional materials for the construction of cisterns must be investigated in low developed countries:

(1) hygienic point of view,
(2) impermeability towards water,
(3) heat insulation,
(4) solidity,
(5) bearing ability.

In areas, where clay is to be found, the wall material for cisterns can be hard baked tiles. Hydraulic mortar can be used as building material.

Stabilised soil is another usable material. Laterite with about 5 to 10% cement, is formed to bricks, which are then pressed and dried. They have a solidity comparable to that of lime-sandstone. The bricks are masoned with clay mortar containing 10 to 15% cement. This construction material, however, is not waterproof. This can be achieved through the careful laying of a cement plaster on the cleansed masonry. Additional protection on the outer wall and bottom can be offered by a 0,5 to 1 m thick layer of stamped mud.

In case of higher sulfate concentration in the soil, is a cement layer on the outer wall and a bitumen finishing recommendable to protect the masonry. If the bitumen finishing is not used, then lime-rich cement should not be used, as this has negative effects on the taste of the cistern water /13/.

Ferrocement, frequently used in India,is also suitable for the construction of cisterns. This will be, however, treated in a particular lecture.

FOOTNOTES

/ 1/ See revised DIN 2001 (which will be published shortly),5.5.4.6

/ 2/ See DIN 2001, "Leitsätze für die Einzel-Trinkwasserversorgung", 1959, p.5,4.325.

/ 3/ Joseph A. Salvato, "Environmental Engineering and Sanitation", 1972, p.150.

/ 4/ Richard Meyer, "Zur Trinkwasserversorgung aus Zisternen" in Archiv für Hygiene und Bakteriologie, 1953, p.473.

/ 5/ Otto Lueger, "Die Wasserversorgung der Städte", 1895, p.314.

/ 6/ Richard Meyer, op. cit., p.467.

/ 7/ Otto Lueger, op. cit., p.315.

/ 8/ Reinhard Schemel, "Wasserversorgung der deutschen Marschengebiete" in GWF, 1950, p.243.

/ 9/ Reinhard Schemel, op. cit., p.242.

/10/ L.Huisman & W.E.Wood, "Slow Sand Filtration", 1974, p.20.

/11/ L.Huisman & W.E.Wood, op. cit., p.78.

/12/ See revised DIN 2001 (which will be published shortly),5.5.4.6

/13/ Richard Meyer, op. cit., p.469.

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SUMMARY

This simple technology for storage and treatment of rainwater offers vast possibilities in rural areas with arid and semi-arid climates. The Mediterranean region and the surrounding arid belt, where the use of cisterns is traditionally spread, is particularly suited for the application of such a technology. The demands on collecting surfaces, location and construction of cisterns are described. Then the conventional construction types of cisterns like Venecian, American, German and siphon cisterns are explained. The collecting surfaces can be a permanent source of germs. The most tedious problem in this context is a hygienic one. A system is presented which contains a slow sand filter. In continous operation this biological filter yields the amount of clear water being used in one or two days. Finally building materials as stabilised soil, clay bricks and tiles and ferrocement are mentioned.

DISTRIBUTION DE L'EAU A BASE DE CITERNES DANS LES ZONES RURALES DES PAYS EN VOIE DE DEVELOPPEMENT

par Gabriele Schulze

Résumé :

Dans les pays en voie de développement des zones arides, les taux d'évaporation sont plus élevés que les taux de précipitation. D'autre part, la technologie simple de traitement et de stockage de l'eau par citernes fait appel aux matériaux et à l'artisanat local, surtout dans les régions méditerranéennes et aux alentours, qui disposent déjà d'une tradition utilisant pareille méthode.

Ce document propose les caractéristiques d'un certain nombre de citernes (surfaces et tailles nécessaires), en fonction des taux de précipitations et d'évaporation, dans le cadre d'une consommation individuelle ou collective. Le choix des sites et les modes de construction des différents modèles de citernes sont évoqués.

Les aspects concernant l'hygiène et l'entretien, les types de matériaux, les possibilités de filtration lente sont également analysés.

Les facteurs les plus importants dans ce type d'ouvrage restent l'hygiène, l'étanchéité, l'isolation thermique, la solidité et la stabilité.

Paper 5

Shallow Wells and Hand Pumps, a Tanzanian Experience

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ABSTRACT

At two recent conferences, held in Tanzania, the construction of shallow wells was promoted as one of the most promising low-cost methods of providing the country's rural population with a clean and dependable water supply. From 1974 onwards shallow well implementation programmes have been going on. Construction methods have changed, during that time, from hand- and machine-dug wells to primarily hand-bored wells, mainly because of lower cost and better opportunities for transferring the construction techniques to local agencies.
Hand pumps used are the "Shinyanga" pump (a version of the Craelius or "Kenya" pump), the "Kangaroo" pump and the "Nira" pump. All efforts are directed towards reducing the maintenance requirements of the hand pumps, as a result of which the Shinyanga pump will most probably no longer be used on a large scale.
Technical cooperation and coordination between the various projects and their donor agencies has begun and is expected to lead to standardization of the methods, equipment and hand pumps used.

INTRODUCTION

The experience gained in a number of technical cooperation projects in Tanzania's Regions Shinyanga, Morogoro, Mwanza, Mtwara, Lindi and Singida is used to illustrate the subject of shallow wells and hand pumps. Rural water supply projects in these regions have a large component of shallow well implementation and are co-financed by the Governments of Tanzania, the Netherlands, Finnland, Australia, the United Kingdom and the World Bank. Similar projects, financed by other sponsors are about to start in other regions.
The importance of constructing shallow wells as one of the most important low-cost solutions for reaching Tanzania's goal of "providing a source of clean and dependable water within reasonable distance of each village by 1991", was highlighted at the annual Regional Water Engineers' Conference in Tanga (May 1980) and at the "Morogoro Conference on Shallow Wells" (August 1980). The latter presented the state of the art of shallow well construction and maintenance in Tanzania to an audience composed of representatives of the donor agencies and officials of the Ministry of Water, Energy and Minerals, and has laid the basis for a coordinated approach, by donors and the Ministry, of the rural water supply problems in Tanzania (ref. 9).

COMPARISON OF RURAL WATER SUPPLY SYSTEMS

Depending on topographical, geological and other conditions there are several supply possibilities to choose from, for instance surface water or groundwater supplies, pumped supplies or gravity supplies, etc.
With few exceptions surface water is bacteriologically unsafe, may contain high sediment loads seasonally, and as a matter of standard practice will require more or less complicated treatment, thus resulting in high investment and operating costs.

Next to springs, which are mainly found in hilly or mountainous areas and even then in limited numbers, shallow wells and boreholes constitute the most reliable groundwater sources.
The construction of boreholes, relatively narrow-diameter wells of greater depth (more than 30 to 50 metres), requires machine-powered equipment. Even for the simplest type of drilling equipment, the percussion rig, high initial expenditure and skilled personnel are required, while its operation remains entirely dependent on the availability of fuel and spare parts.
Shallow wells do not have these disadvantages, as their construction requires a minimum of skill and no mechanized equipment. Furthermore, hand pumps* are generally sufficient for their exploitation.

In boreholes the water table is often so deep that hand pumps cannot be used.
Unless pumps can be powered by wind energy (requiring a very stable and constant wind pattern and/or huge water storage capacities) or solar energy (as yet requiring high capital outlays) a conventional pump, generally diesel-driven, will have to be used. Although such pumps can be very reliable, thier continued use presupposes the availability of skilled pump operators, skilled maintenance technicians and an adequate supply of fuel and spare parts.
As a rule not all these requirements will be met, expecially in rural areas of developing countries.
A comparison of development and recurrent costs for the various water supply options (table 1) shows the attractiveness of shallow wells from an economic point of view.
Experience in Tanzania shows that the actual development costs for shallow wells and boreholes or surface water supplies are even farther apart, with shallow well schemes costing about one-tenth of the alternative options.

* Following the momenclature used by the WHO Internatonal Reference Centre for Community Water Supply (ref. 7) the term "hand pumps" as used herein applies to any simple water lifting device, powered by human energy and used in rural village water supplies.

TABLE 1. COST ESTIMATES OF ALTERNATIVE RURAL WATER SUPPLY SYSTEMS IN TANZANIA (REF. 10)

	Development cost per person (TAS)	Operating cost per person (TAS)
Well without hand pump	65	3
Well with hand pump	100	3
Borehole; diesel pumped	250-275	8
Surface water supply; gravity; short transmission main	160	8
Surface water supply; gravity; long transmission main	320	8
Surface water supply; diesel pumped	365	12
Dam; diesel pumped	500	14

Note: US $ 1 = TAS 8.-

SURVEY FOR SHALLOW WELLS

The first and most important step in the construction of a shallow well is to find a suitable site, so that a successful shallow well can be made without having to rely on chance and run the risk of making expensive unsuccessful wells. A successful shallow well is defined as a well

- which gives water throughout the year, even during extremely dry periods
- with a yield sufficiently high to meet the daily water requirements of 250 to 300 people (in practice 3 to 6 m^3/day (ref. 11, 12))
- with a water quality that meets the water quality standards
- which is located within walking distance of the customers (less than 1 km)
- which is accessible on foot throughout the year
- which, by its location as well as its construction, is protected against any form of chemical or bacteriological contamination

Theoretically a comprehensive geohydrologic survey should give the best results, but the cost and the time available in large-scale well implementation projects render anything but the quickest and simplest methods impracticable.
Geoelectrical surveys were carried out at the beginning of the Shinyanga project, but the results were of hardly any value and the surveys were discontinued in favour of large-scale test drilling with small-diameter drilling equipment.

In Shinyanga light motor-drills and trailer-mounted motor-drills were used initially, but were abandoned in favour of hand-drilling equipment (ref. 1). Most of the other projects followed this example, with the exception of the Mtwara/Lindi project, where tractor-mounted machine-augers and hand-operated augers are used side-by-side (ref. 5, 9).
A typical hand-drilling set consists of Edelmann and riverside bits of 7 and 10 cm diameter, a screw auger, bailer and extension rods and handles, and costs approx. TAS 41 500 (ex-Morogoro), including 15 metres of casing pipe and a test pump.

The final adoption of a well site is based on the results of a water quality check and a simple pumping test with a hand pump. Experience in Shinyanga and Morogoro shows that a test yield of at least 200 to 500 litres per hour is required for a large-diameter (1-1.50 m dia.) well, whereas for a drilled or bored well (0.10-0.30 m dia.) the negligible storage capacity requires a minimum test yield of 500-1000 l/h.

SHALLOW WELL TYPES

Essentially, two main types of shallow wells can be distinguished: large- and small-diameter wells. Large-diameter wells are invariably dug wells, with diameters from 1 m upward, the vast majority of wells having a diameter of 1.2 to 1.5 m (ref. 3, 4, 6). Small-diameter wells can be augered or bored, driven, jetted, hydraulic or cable-tool percussion-drilled, bailed, hydraulic rotary-drilled, etc. and generally have diameters from 0.10 to 0.50 m.
Abundant literature on this subject is available (ref. 1, 2, 6).

The traditional way to make a shallow well is to dig it by hand, which is simple, requires very little investment and only unskilled labour. Yet, the possible dangers of digging, especially deeper, wells with inexperienced people should not be underestimated. Moreover, programmes that require a large number of wells to be finished within a fixed period impose their own restrictions.
Especially when the aquifer recharge is large, digging a well to the required depth may be impossible because of the inflowing water. Then two different methods can be followed: either the pit has to be dewatered during construction, or digging is continued as far as possible, after which the well is completed, to be deepened in the dry season, if necessary. The first option requires the use of pumps during construction, unless mechanical well-digging methods are used.
Suction pumps are simplest, but can be used only for water tables down to 7 m; for deeper wells lift pumps are required. Especially with larger inflows, machine-driven pumps are required, preferably electrical pumps, whereby great attention should be given to avoiding all risk of electrocution of the well diggers.
The problems encountered with the supply of spare parts and fuel have resulted in a re-assessment of the dug well concept in most of the on-going shallow well projects in Tanzania.

In a number of cases dewatering of the pit during construction resulted in washing out the sandy aquifer material from between horizontal clay layers; these subsequently collapsed and sealed off the aquifers. Therefore in several projects methods which did not require dewatering of the well were reverted to: hand- or percussion drilling.

Dug wells

The methods of making dug wells can be categorized in three groups (ref. 4, 6):

a. alternately deepening and lining the well shaft, in sections of about one metre
b. excavation of shaft to water table, then building the lining upwards
c. sinking of pre-formed cylindrical lining by undermining (caissoning)

In Shinyanga method b. was followed to the extent possible, after which either the danger of collapse of the well shaft or the presence of ground water necessitated the use of method c. The lining consisted of unreinforced concrete rings. These rings (1.25 m int.dia.; 1 m height) were of no-fines concrete at aquifer level and of plain concrete at other levels. The well rings were manufactured centrally and trucked to the site. There the rings were lowered into the well, using a simple tripod and winch, and digging continued as a caissoning process (ref. 1).
When the wells had been dug to the required depth and the concrete rings put into their final position, the space between the rings and the undisturbed soil was backfilled with coarse sand or gravel and a concrete or clay seal employed just above the aquifer level, in order to block seepage. On top of this seal the original soil material was backfilled up to the ground level, the well opening itself was covered with a concrete cover-cupump stand and the area around it with a concrete apron, cast in situ.
For wells with deep-lying aquifers the diameter of the concrete rings above groundwater level was sometimes reduced to save on cement and transport (fig. 1b).
The same procedure is followed in Mwanza and Singida Regions, whereas in Mtwara/Lindi a variation on this technique is used (ref. 5, 9). These relatively shallow wells (4 to 5 m deep) are dug by tractor-mounted excavators and lined with concrete rings of 0.5 m height and 1.0 m internal diameter. For deeper wells, digging beyond 5 m is by hand, using the caissoning process with 0.80 m diameter rings that telescope inside the first rings.

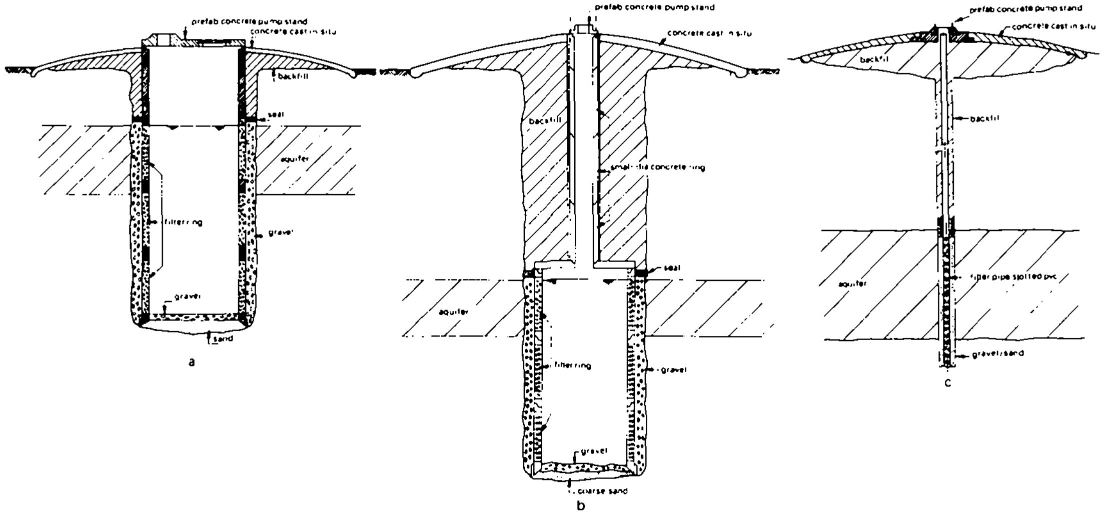

Fig. 1 Shallow well types

<u>Hand-bored wells</u>

In most cases, increasing the depth of a well is a more certain way of improving its yield than enlarging the diameter and - unless the presence of a poor aquifer necessitates the use of a large-diameter well because of its storage capacity - deeper bored or drilled wells may be preferred to large-diameter dug wells.
The problems caused by dewatering dug wells can be avoided in this way, while over-all costs per well can be reduced (fig. 2; ref. 1, 9).
Moreover, the construction time of a well (3 to 7 weeks for a dug well of 8 m deep) could be reduced to 1 week in the case of a hand-drilled well, as a result of which the output per construction group increases from 8 to 50 wells per year.
The requirement of higher aquifer transmissivity (yield during test pumping: 500-1000 l/h rather than 200-500 l/h as for dug wells) was met at the vast majority of the well sites. Thus for the Morogoro project the emphasis has been on hand-drilled wells (fig. 1c) from the start, while all other on-going shallow well projects in Tanzania are already using this method or will do so in the near future (ref. 1, 8, 9).

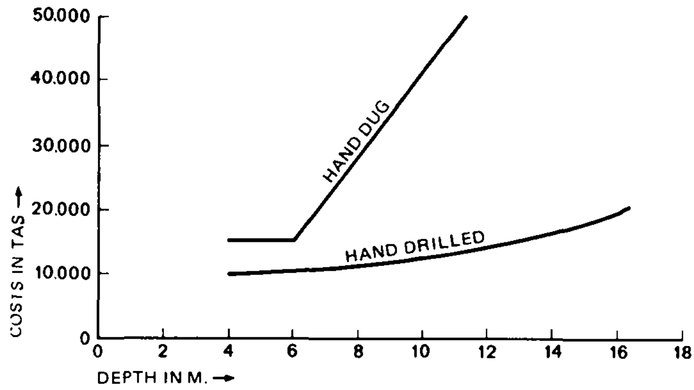

Fig. 2 Cost of shallow wells
Note: US $ 1.- = TAS 8.- Costs do not include overhead caused by expatriate staff and depreciation of vehicles and buildings.

Hand-drilling equipment consists essentially of a continuous-flight auger with various bits of two diameters (the smaller to be used inside a casing), a cross piece with four handles for turning the drill, bailers, casing pipe, etc. (fig. 3).
Sets in use at Morogoro have bit diameters of 30 and 23 cm, or 23 cm and 18 cm, and cost approx. TAS 100000 per set (ex-Morogoro).
At the beginning of the drilling operation the cross piece with handles, a continuous-flight auger and a bit are connected and hung from a cable that runs through a pulley block on top of a tripod.
A crew of 4 to 6 people, most of whom are employed on a semi-self-help basis, screw the drill down for approx. half a metre. The drilling assembly is then lifted, soil is removed from the auger, the assembly is lowered again, and this process continues, with or without a casing, until the required depth has been reached. Depending on the soil type a regular auger bit, conical auger bit, riverside bit or bailer is used.
When the hole is at the required depth a slotted pvc pipe is lowered into it, a gravel pack is put around the pipe, and the well is finished in essentially the same way as a dug well. The slotted pvc pipe is imported or manufactured locally from plain pvc pipe.

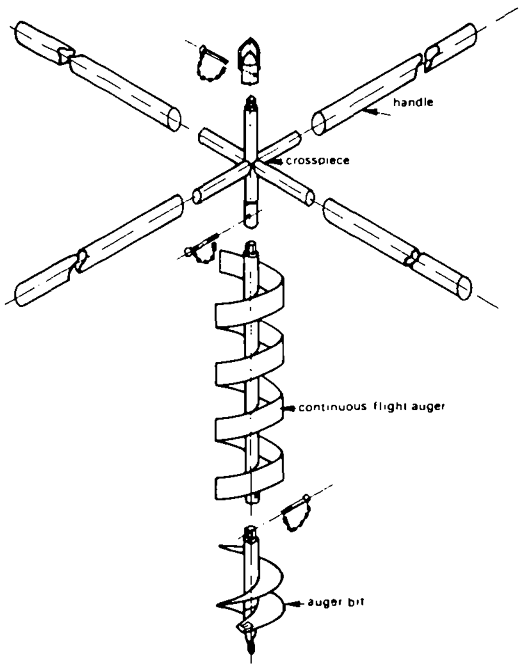

Fig. 3 Heavy hand-drill

HAND-PUMPS

Traditionally, bucket and rope are often used for drawing water in larger-diameter wells.
Not only is contamination of the ground water almost certain in these cases, but for smaller-diameter (drilled) wells the use of buckets is virtually impossible.
From the beginning of the Shinyanga Shallow Wells Project it was intended to construct only covered wells, equipped with hand pumps with the piston below groundwater level, to avoid the necessity of priming the pumps.
For a long time the "Shinyanga" pump was the only type used in this project. It is a modified version of the "Craelius", "Uganda" or "Kenya" pump used in East Africa and consisted as far as possible of standard pipe fittings, so that construction and maintenance could be easily carried out locally (fig. 4; ref. 1).

The cylinder assembly is completely different from the original versions, however, having a pvc or ABS cylinder, a commercially available manchet-type piston and brass piston and foot valves. In the course of time the assembly has undergone repeated modifications, aimed at improving the reliability and - especially - reducing the maintenance requirements of the pump. Details are given elsewhere (ref. 1, 7).

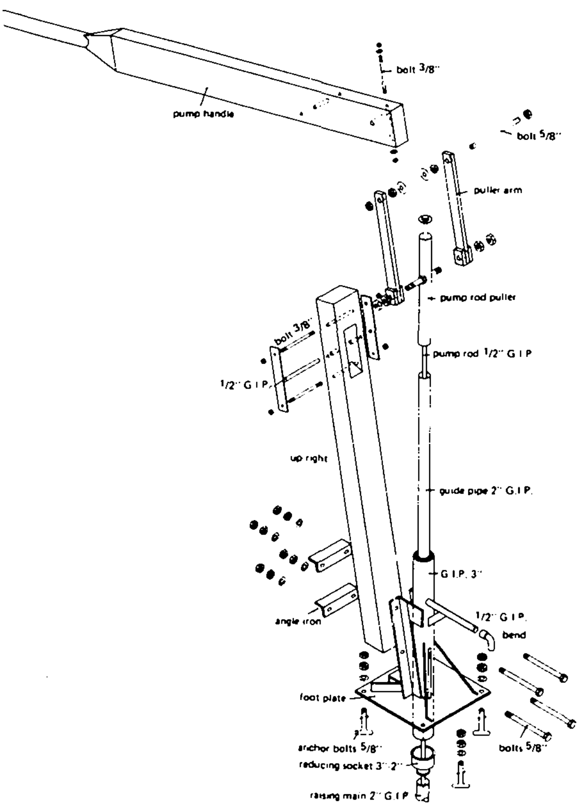

Fig. 4 Shinyanga pump

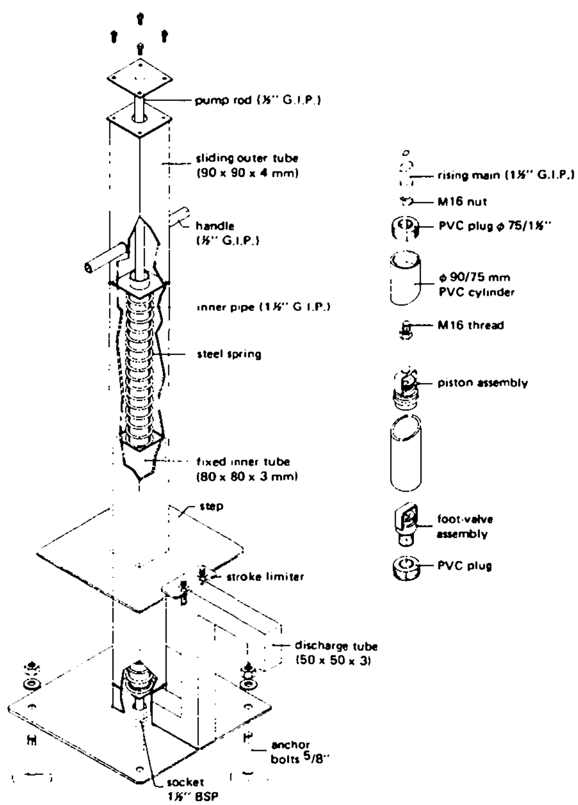

Fig. 5 Kangaroo pump

A number of considerations, one of these being that centralized maintenance of pumps and wells in an area as sparsely populated as rural Tanzania is extremely costly, have led to a shift in policy. The most important criterion is no longer that pumps should be built up from locally available materials, but that they should require as little maintenance for as long a period as possible.
This policy has led to the development of a new pump type: the Kangaroo pump. By constructing a pump head that acts in the same way as a pogo stick, the vertical movement of the pump piston is transferred directly to the pump head, without any levers or hinges that require lubrication (fig. 5).
Elimination of the wooden parts of the superstructure, which might be stolen for firewood, adds to the life expectancy of the pump.
At present the costs of the Shinyanga and Kangaroo pump heads are TAS 1500 and TAS 2200 respectively; the cost of a 3" cylinder assembly is TAS 1250.
Kangaroo pumps are used in all on-going Tanzanian shallow well projects, sometimes in combination with Shinyanga-type pumps, except in Mtwara/Lindi. There the Finnish Nira pump is used, which - in the same way as the Kangaroo - is constantly undergoing modification in order to further reduce maintenance requirements and to improve the pump quality. Coordination has been established between the projects in order to standardize and exchange pump components and possibly arrive at a common pump design.

MAINTENANCE

In rural Tanzania centralized maintenance of shallow wells would be extremely expensive, probably costing more than the entire water supply budget. With many piped supplies already out of order due to unavailability of spare parts or fuel, and lack of funds for operation and maintenance, reduction of the maintenance requirements has become of utmost importance.

Therefore, even though local maintenance can and must be improved, a high quality pump which requires a minimum of maintenance is preferable to a lower quality, locally made pump, even if the initial costs are somewhat higher and everything must be imported for the time being.
It should be noted, furthermore, that a considerable proportion of the locally available materials and parts used in pump- and shallow well construction have been imported in any case, so that the total net loss in added value to Tanzania is probably not significant.

In 1980 a National Shallow Wells Programme was started up; this will provide guidelines for nation-wide implementation of shallow wells in Tanzania, including educational and maintenance aspects.
It is envisaged that maintenance of shallow wells in Tanzania will be based on a 2 tier system, with the villagers (or their appointed pump attendants) carrying out simpler maintenance and District Maintenance Officers taking care of more complicated maintenance and repairs (ref. 9).
Training and education programmes aimed at increasing village participation in rural water supply are to be started shortly.

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Puits Phréatiques et Pompes à Main

RESUME

Lors de deux conférences récentes en Tanzanie, la construction de puits peu profonds fut promue comme une des méthodes les plus encourageantes pour fournir, a prix modique, la population rurale du pays en eau sanitairement acceptable.
A partir de 1974, des programmes pour créer des puits peu profonds se sont poursuivis. Les méthodes de construction ont evolué, des puits creusés a la main - ou avec machines - pour arriver aux puits forés à la main, en particulier à cause du coût moins elevé, de la facilité de transmission des techniques aux organisations locales.

Les pompes à main utilisees sont les pompes "Shinyanga" (basée sur la pompe Craelius ou "Kenya"), les pompes "Kangaroo" et les pompes "Nira". Tous les efforts visent a réduire l'entretien des pompes à main ; donc la pompe Shinyanga ne sera probablement plus utilisée sur une grande échelle.

La cooperation et la coordination techniques entre les divers projets et les organisations donatrices ont commencé et devraient conduire à une normalisation des méthodes, du matériel et des pompes à main utilisées.

Paper 6

Mechanically Drilled Wells and Motorpumps

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Summary

Ancient methods of water well drilling used a percussive system which is with us today as the familiar cable tool rig.

Rotary drilling rigs were introduced soon after engine power became available for mobile plant and, largely as a result of oil well drilling developments, have become highly sophisticated, costly and very efficient.

To apply modern water well technology at low cost and appropriate for the specific conditions of a developing country it is necessary to combine the reliability of modern power with the simplicity of well-tried methods together with fully trained personnel. This applies equally to the means of abstracting water from drilled wells.

This paper describes in outline the various drilling and pumping systems in use today and suggests methods appropriate for the developing country.

Introduction

Part I

CURRENT DRILLING METHODS - GENERAL DESCRIPTION

1.1 Cable Tool Drilling
1.2 Rotary - Direct Circulation - Mud
1.3 Rotary - Reverse Circulation - Water
1.4 Rotary - Air

A. Direct Circulation
B. Reverse Circulation
C. Down-the-hole hammer drilling
D. Foam Drilling

1.5 OTHER METHODS

A. Jetting
B. Hydraulic Tube Racking
C. Auger Drilling
D. Coring
E. Down-the-hole-motor (turbine)
F. Vibration Equipment

Part 2

LOW COST DRILLING

2.1 Lightweight cable tool/percussion drilling
2.2 Medium weight cable tool/percussion drilling
2.3 Chilled shot rotary drilling
2.4 Manual rotary jetting
2.5 Conclusion of Part 2

Part 3

MOTORPUMPS

3.1 Shallow Well Pumps

A. Centrifugal Pumps
B. Jet Pumps
C. Diaphragm Pumps
D. Chain Pumps

3.2 Deep Well Pumps

A. Reciprocating Pumps
B. Jet Pumps
C. Vertical Spindle Turbine Pumps
D. Electric Submersible Pumps
E. Helical Rotor Pumps

3.3 Conclusion of Part 3

Introduction

Although the history of water well drilling can be traced back some five thousand years it was not until the 18th century that truly mechanically driven drilling rigs appeared. Coincidentally the early iron masters were able to produce forms of casing sections and, later, tube lengths which enabled much of the current masonry and timberwork to be dispensed with.

The above combination of machinery and a lining which could be assembled on the surface meant that it was no longer necessary for men to enter the well - except perhaps to excavate the first few metres - and consequently no dewatering was required. This led to wells of reduced diameter which could now penetrate aquifers at depths far greater than hitherto possible, thereby exploiting the benefits of the usually increased reliability and higher quality of a deeper aquifer. This reduced diameter and depth increase generally ruled out the use of hand-held buckets for water abstraction and encouraged the development of the mechanical pump.

Modern drilling rigs for the 1980's are the result of evolution rather than any dramatic change of method. They are mostly built in Western Europe and the U.S.A. where competition between manufacturers is intense and where drilling contractors are acutely aware of the need to mobilise, drill, line the borehole, test and demobilise in the shortest possible time in order to remain competitive.

The above conditions have tended to result in very sophisticated drilling rigs, highly efficient and capable of rapid penetration rates. However, their very complexity demands a high standard of skill in operation, thorough maintenance, and a first class spares and service back up. It should also be noted that such rigs are generally rather specialised and as such, are limited to the narrow field for which they were designed - although some manufacturers quote an over optimistic versatility in their brochures.

Fortunately the on-site conditions in very many of the developing countries of the world are ideally suited to the application of well-tried but less exotic equipment and methods. These are described in Part 2.

Part I

CURRENT DRILLING METHODS - GENERAL DESCRIPTION

There are two main drilling methods: (a) Cable tool drilling (percussion) and (b) Rotary drilling. The latter method can be further sub-divided into different types There are also several other methods which do not exactly fall into the above categories and these will be mentioned.

It is stressed that the following descriptions are in outline only as space does not allow an in-depth coverage. The aim is to show what is available, its advantages and limitations.

1.1 Cable tool drilling

This is the oldest and one of the simplest mechanical methods used for well drilling. It has changed little during the last fifty years and indeed offers little room for improvement without losing some of its main advantages.

A steel chisel (bit) is raised and dropped by a reciprocating mechanism incorporated in the drilling rig. A heavy steel shaft is mounted above the bit to give weight and stability and the complete assembly is suspended on a wire line which is gradually lowered as depth is gained. (Figs. 1 and 2)

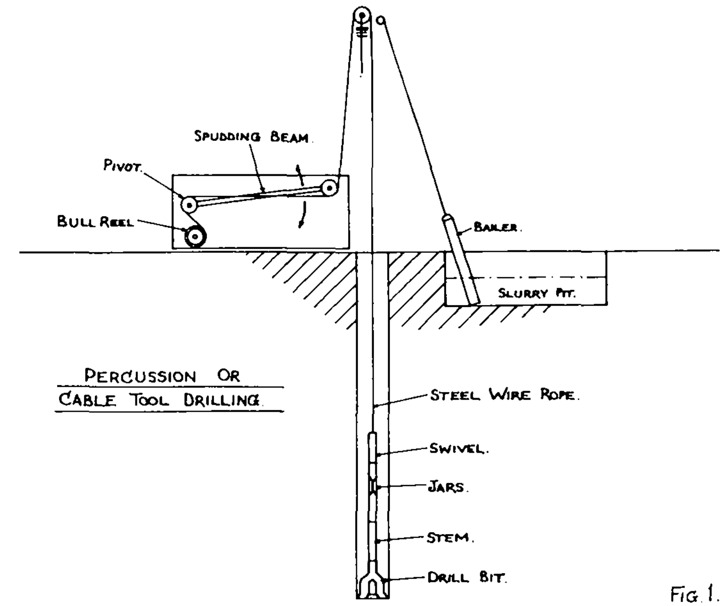

Spudding Beam.
Pivot.
Bull Reel
Bailer.
Slurry Pit.
Percussion or
Cable Tool Drilling.
Steel Wire Rope.
Swivel.
Jars.
Stem.
Drill Bit.

Fig. 1.

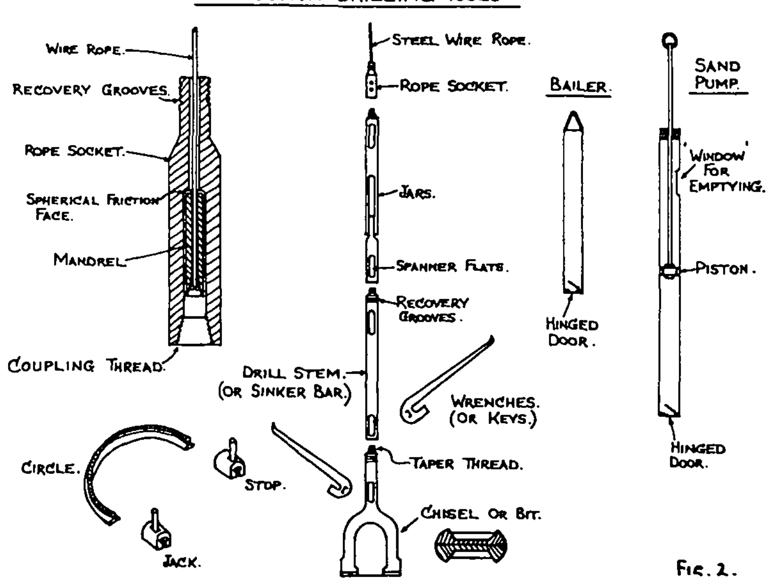

Percussion Drilling Tools
Wire Rope.
Recovery Grooves.
Rope Socket.
Spherical Friction Face.
Mandrel.
Coupling Thread.
Steel Wire Rope.
Rope Socket.
Jars.
Spanner Flats.
Recovery Grooves.
Drill Stem.
(or Sinker Bar.)
Taper Thread.
Chisel or Bit.
Wrenches.
(or Keys.)
Circle.
Stop.
Jack.
Bailer.
Hinged Door.
Sand Pump.
'Window' For Emptying.
Piston.
Hinged Door.

Fig. 2.

Water is added until the natural water table is reached, the purpose of which is to enable the pounding bit to crush and mix the ground into a slurry. Periodically the tools are removed and a tube with a hinged door at the bottom (a bailer) is run into the well in order to remove the slurry. The drilling tools are again lowered into the well and work recommenced. Although the bit is of H section a true and circular well is produced by the rotating action imparted by a combination of rope 'twist' and a friction swivel at the top of the tools. Although the tools are striking the bottom of the well they are only just doing so; throughout the drilling motion the driller adjusts the speed and feed to maintain the string of tools in tension during each stroke thereby ensuring a vertical hole.

Drilling continues until either the full depth is reached or until it is necessary to insert a column of temporary or permanent tubes.

Temporary tubes will be required when passing through loose sands, gravels and most non-cohesive formations as the swabbing action of the reciprocating tools would otherwise cause collapse of the well. Under the worst local conditions of instability a deep well may call for the temporary insertion of perhaps five reducing columns of tubes starting with say a 450 mm guide tube and telescoping down to 200 mm for the installation of permanent 150 mm casing and screens.

All the temporary tubes and associated tools have to be provided and handled. They do not form part of the final construction but have to be allowed for in the costing of the work. This is one of the factors against the use of cable-tool drilling in unconsolidated ground. On the other hand the many advantages inherent in this system may outweigh the disadvantages. Also it should be noted that some areas can be drilled without any fear of collapse, and that the only column of tubes to be inserted will be those comprising the permanent structure.

Advantages of Cable-tool/percussion drilling

(a) Simplicity The drilling rig is a conventional piece of mechanical plant, generally without hydraulics, and extremely robust.

(b) Low Capital Cost and Running Costs Due to the above simplicity the cost of the machine and tools is low compared with a rotary rig of similar capacity and the power requirement is modest. This leads to fuel and lubricant economy and any fitter/mechanic is able to service and repair the straightforward mechanism.

(c) Ability to make hole under almost all conditions A cable-tool rig is the only drilling machine which can be deployed on any location in the world with a fair chance of success. It will be very slow in hard solid rock and will have problems in boulders - although this latter comment can apply to other methods.

(d) Immediate facility for water sampling as drilling progresses It requires only a minute or two to remove the drilling tools and run a bailer to take a water sample. This is most important when working in an area where little prior knowledge of the hydrogeology is available.

(e) Low demand for drilling water Drilling water is only required until the natural water table is reached. This demand may be as low as two barrels for an 8 hour shift.

(f) World-wide tool availability The A.P.I. (American Petroleum Institute) standards are recognised in cable tool drilling and this standardisation leads to universal availability.

(g) Penetration of aquifer quickly apparent Due to the need to clean periodically the well as drilling proceeds the driller can tell when a water-bearing stratum has been reached. This again is an advantage, especially under the condition noted in (d) above.

Disadvantages

(a) Unconsolidated formations have to be supported by columns of temporary tubes

(b) Slow progress in hard rock

1.2 Rotary - Direct Circulation - Mud

In this system a fluid, generally a mud, is mixed in a pit or tank and delivered by a pump at high pressure through a flexible hose to the top of a rotating column of pipes called the drill string. It then flows through the tools to the bottom of the borehole and returns up the annular space to the surface and back into the mud pit. (Fig. 3)

A bit is located at the bottom of the drill string and is either of the roller cutter type or a bladed bit called a drag bit.

Mounted above the bit are one or more collars which consist of extra-heavy pipes and have the function of providing the weight essential to rotary drilling. They also provide directional stability.

As depth increases, further drill pipes are added to the tool string at the surface. The cutting produced by the bit are continuously washed away by the fluid and transported up to the mud pit where they settle out and the relatively solids free mud is again recirculated.

The mud also has other duties. It cools and lubricates the bit and tool string and, most important, supports unconsolidated formations by exerting a positive pressure against the unstable walls of the well.

This latter function is achieved by the plastering effect of the mud which produces a thin 'cake' on the wall.

Normally it is sufficient to insert a short length of conductor pipe into the top of the well to provide structural stability; the remainder of the hole may be drilled 'open' to receive the permanent column of plain and where necessary, screen tubes. The drilling fluid is then removed and any required development carried out before the pumping test.

Advantages

(a) Generally rapid penetration takes in most formation to great depth Hole clearing is carried out concurrently with drilling and progress is only interrupted for brief moments whilst adding a further drill pipe.

(b) Unconsolidated formation supported without inserting temporary tubes Unless severe caving and water loss occurs the mud column is sufficient to support the borehole walls.

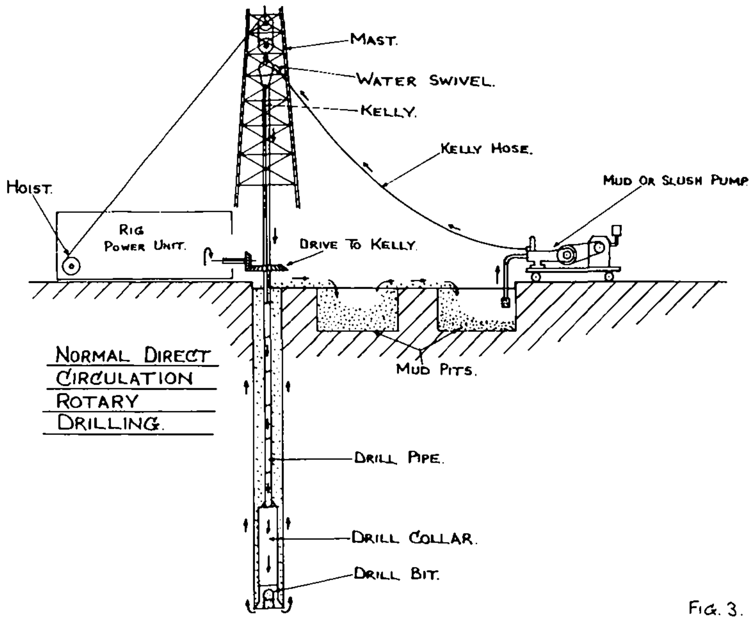

MAST.
WATER SWIVEL.
KELLY.
KELLY HOSE.
MUD OR SLUSH PUMP.
HOIST.
RIG POWER UNIT.
DRIVE TO KELLY.
MUD PITS.
NORMAL DIRECT CIRCULATION ROTARY DRILLING.
DRILL PIPE.
DRILL COLLAR.
DRILL BIT.

FIG. 3.

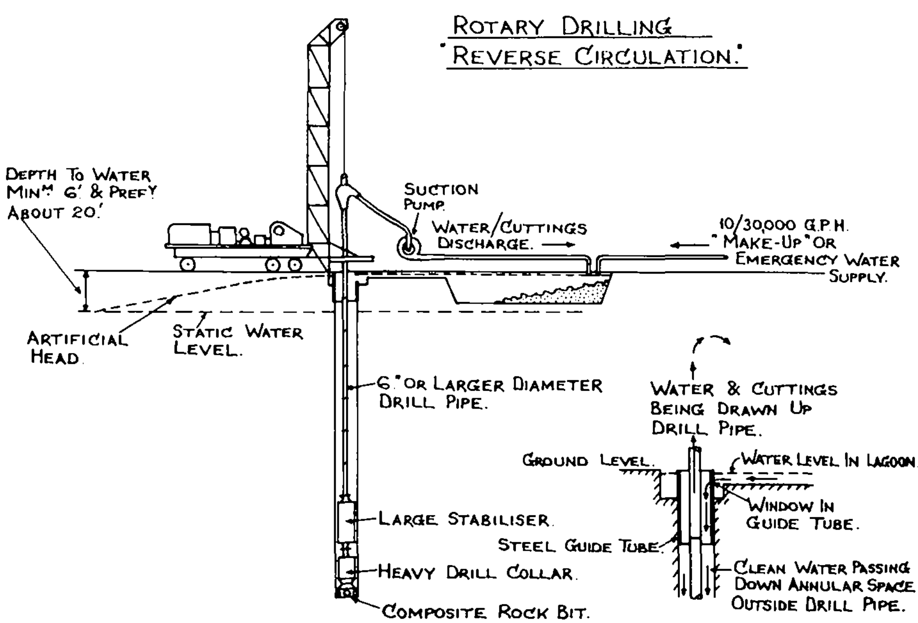

ROTARY DRILLING "REVERSE CIRCULATION."
DEPTH TO WATER MIN.M 6' & PREF.Y ABOUT 20'.
SUCTION PUMP.
WATER/CUTTINGS DISCHARGE.
10/30,000 G.P.H. "MAKE-UP" OR EMERGENCY WATER SUPPLY.
ARTIFICIAL HEAD.
STATIC WATER LEVEL.
6" OR LARGER DIAMETER DRILL PIPE.
WATER & CUTTINGS BEING DRAWN UP DRILL PIPE.
GROUND LEVEL.
WATER LEVEL IN LAGOON.
WINDOW IN GUIDE TUBE.
LARGE STABILISER.
STEEL GUIDE TUBE.
HEAVY DRILL COLLAR.
CLEAN WATER PASSING DOWN ANNULAR SPACE OUTSIDE DRILL PIPE.
COMPOSITE ROCK BIT.

FIG. 4.

Disadvantages

(a) Relatively high cost of equipment The added complication of the rotary rig - especially where this includes hydraulic equipment - raises the cost some 3 to 4 times that of a similar cable tool rig.

(b) High risk of mudding off aquifers Although the use of a self-destroying organic polymer mud is becoming more common, Bentonite clay is still in wide usage and it is often difficult to remove completely the clay particles - even when applying chemical dispersants.

(c) Possibility of losing all mud into fissures If this should occur it may not be possible to regain circulation without resort to 'lost circulation' additives or cementing dry fissures. Accordingly, an alternative method of drilling may have to be available.

(d) Considerable quantities of drilling water required In addition to the volume of the well and some losses of water into the formation it is necessary to have a mud pit capacity of not less than three times the well volume in order to provide sufficient 'dwell' for the solids to settle out of the circulating fluid.

(e) Difficulty in identifying an aquifer, especially when exploration drilling The fact that a column of fluid has been introduced into the well effectively masks the presence or otherwise of an aquifer (unless it is under strong artesian pressure) and it is difficult to determine the final depth and construction of the well unless prior knowledge of the hydrogeology is available. Electric logging is of valuable assistance in such cases.

1.3 Rotary-Reverse Circulation Water

This method of drilling is usually applied to the larger sizes of well in gravels, sands and some soft stable formations.

The equipment is similar to direct circulation machinery but differs in being larger and in particular, uses very large diameter drill pipe.

The drilling principle alters in that instead of fluid being forced down the drill pipe it is drawn upwards by suction pump or air lift at high velocity and carries the cuttings up from the drill bit to deposit them in a large settling lagoon on the surface. The solids-free water is then conducted via a channel back to the well where it flows at low velocity down the annular space to the rotating tools (Fig. 4)

As clean water is normally circulated there is little or no contamination of the aquifer. In the event of clays being drilled in the upper section of the hole the lagoon may be cleaned out and refilled with fresh water before proceeding into the aquifer.

Advantages

(a) Extremely rapid drilling at large diameter, especially in gravels and soft sediments The bit dislodges the formation and the pieces are immediately sucked up the drill pipe rather than being crushed and reduced.

(b) Rapid production of representative samples The cuttings spend negligible time in transit before appearing at the surface and therefore breakdown and separation are minimal.

(c) No risk of mudding - off the aquifer Although some silty particles may temporarily lodge in the wall of the well these are readily removed during the development stage of construction.

(d) Reverse circulation drilling uses a smaller pump horsepower than direct drilling at large diameters because the velocity of the return water to lift the cuttings is a function of the drill pipe diameter rather than the borehole diameter.

(e) Temporary tubes unnecessary below short conductor pipe Reverse circulation relies on a 'head' of water superimposed upon the natural water table and it is this positive pressure which may be of only about 2 metres which supports any unstable formation.

Disadvantages

(a) Equipment of high capital cost To cope with the weights involved in rotary drilling at large diameter these rigs are generally heavy and expensive. However, light rigs are now being made for smaller diameter elementary reverse circulation drilling.

(b) Water table must be fairly near to the surface The circulating water is usually drawn up the drill pipe by a suction pump, although this can be augmented by a compressed air lift. However, several criteria demand a reasonably high water table, not least of which is the undesirability of cascading the return water down the annular space.

(c) Very large quantities of back-up water should be available Apart from the water required to fill the well and lagoon, there is bound to be a transfer of drilling water from the unlined annular space to the aquifer and this can reach extremely high proportions.

1.4 Rotary - Air

A. Direct Circulation - Air

The use of air as a drilling fluid offers a number of advantages and, with slight modification, most standard rotary rigs can be used. The direct circulation principle of mud drilling is followed and provision is generally made for the injection of small quantities of water when required to keep down dust. Foam may also be used.

B. Reverse Circulation

In large diameter wells it becomes impractical to provide the quantity of air sufficient to lift the cuttings up the annular space between the drill pipe and the wall of the well. To overcome this limitation a shrouded tool assembly is employed together with double-walled drill pipe. The compressed air is passed to the bit down the annulus within the drill pipe and the shroud conducts the cuttings and used air back up the central bore of the drill pipe (Fig 5.)

Advantages

(a) Loss of circulation does not prevent drilling The air is not used to support the formation.

(b) Impossible to mud off an aquifer The air and cuttings will not enter the aquifer. When water is reached the principle of the air lift pump will come into play and may even commence initial hole cleaning.

(c) Drilling water normally not required Very small injected quantities are sometimes beneficial when traversing the damp zone between dry rock and the water table.

(d) Rapid return of cuttings The rising air velocity in the annular space is at least 1,000 m/min. and therefore a rapid presentation of cuttings is ensured. A large air compressor is therefore needed and capable of 7 to 20 bar according to the depth to be drilled below the water level.

Disadvantages

(a) Inability to support caving formations No pressure is exerted on the wall of the well.

(b) Efficiency falls with increase in depth below water The air pressure required increases with depth below the water table and therefore less air pressure remains to clean the cuttings.

(c) Usually desirable to provide mud back-up system If knowledge of the strata conditions is uncertain, it may be impossible to drill entirely with air. An alternative method of drilling should be available.

(d) Necessary to have heavy compressor capacity

C. Down-the-hole hammer drilling

Another rotary/percussive system using air is that known as Hammer drilling. Air under pressure is fed down drill pipes to a pneumatic hammer below which is fitted a bit set with tungsten carbide inserts. Most of the power is applied where it is most needed - at the bit - and the only rotational equipment is that necessary to change the position of the teeth for each blow on the bottom of the hole (Fig. 6) This tool represented a major step forward as penetration rates in hard rock increased enormously.

D. Foam Drilling

Introduced during the last few years - and still under development - is a type of drilling incorporating conventional rotary equipment or down-the-hole hammer but differing in that a foam-producing fluid is injected into the drill string at the surface. This combines with the air whilst passing down the drill pipe and expands into a foam when leaving the bit. The foam rises in the annular space relatively slowly and, being substantially stiff, transports the cuttings to the surface.

Advantages

Foam permits drilling to take place where (a) mud cannot be circulated due to well losses or where drilling water supplies are a problem, and (b) insufficient annular air velocity is available to clean the hole adequately. The required capacity of the air compressor is greatly reduced.

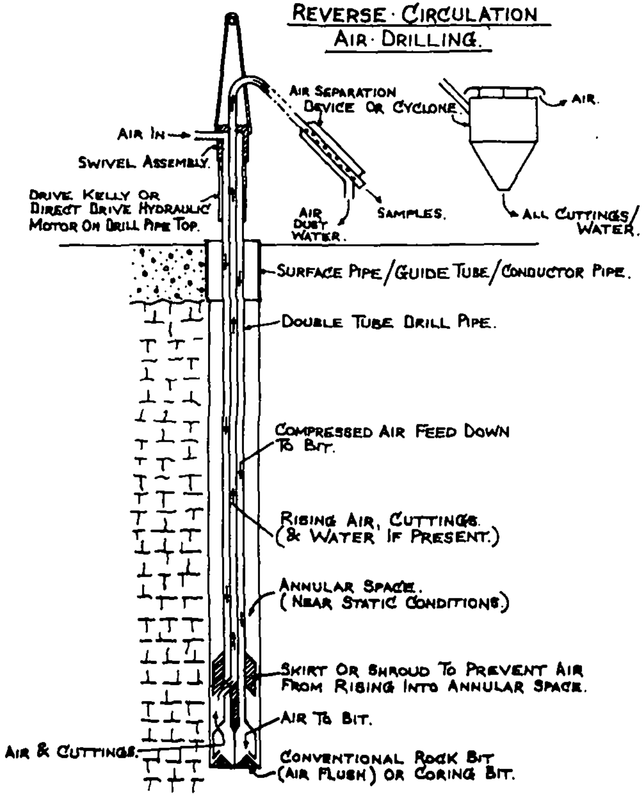

REVERSE · CIRCULATION
AIR · DRILLING.
AIR SEPARATION DEVICE OR CYCLONE.
AIR.
AIR IN
SWIVEL ASSEMBLY.
DRIVE KELLY OR DIRECT DRIVE HYDRAULIC MOTOR ON DRILL PIPE TOP.
AIR DUST WATER.
SAMPLES.
ALL CUTTINGS/ WATER.
SURFACE PIPE/GUIDE TUBE/CONDUCTOR PIPE.
DOUBLE TUBE DRILL PIPE.
COMPRESSED AIR FEED DOWN TO BIT.
RISING AIR, CUTTINGS. (& WATER IF PRESENT.)
ANNULAR SPACE. (NEAR STATIC CONDITIONS.)
SKIRT OR SHROUD TO PREVENT AIR FROM RISING INTO ANNULAR SPACE.
AIR TO BIT.
AIR & CUTTINGS.
CONVENTIONAL ROCK BIT (AIR FLUSH) OR CORING BIT.

FIG.5.

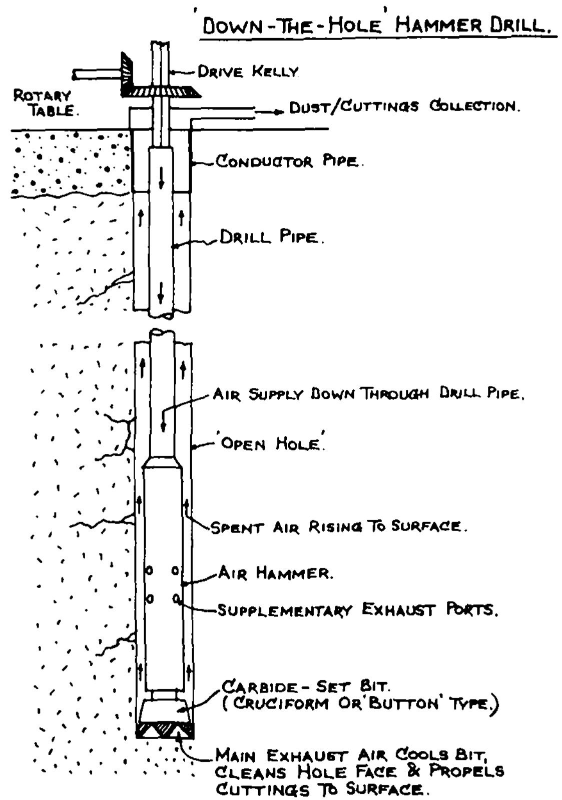

'DOWN-THE-HOLE' HAMMER DRILL.
DRIVE KELLY.
ROTARY TABLE.
DUST/CUTTINGS COLLECTION.
CONDUCTOR PIPE.
DRILL PIPE.
AIR SUPPLY DOWN THROUGH DRILL PIPE.
'OPEN HOLE.'
SPENT AIR RISING TO SURFACE.
AIR HAMMER.
SUPPLEMENTARY EXHAUST PORTS.
CARBIDE-SET BIT. (CRUCIFORM OR 'BUTTON' TYPE.)
MAIN EXHAUST AIR COOLS BIT, CLEANS HOLE FACE & PROPELS CUTTINGS TO SURFACE.

FIG.6.

1.5 OTHER METHODS

A. Jetting

This is a form of hole-making which is applied to small diameters and shallow depths through soft materials and is a very simple method. Hollow rods are lightly rotated and reciprocated whilst a high pressure flow of water is passed through them to the bit below. The rods (or pipes) may be fitted with various types of jetting chisels but in very soft ground they may be used with a cutting shoe only, to form the actual final lining and screen column.

A variation of this method requires a regular reciprocating motion of the rods which, in conjunction with a non-return valve, causes a pulsating flow of water and debris to rise within the pipes to discharge at the surface whilst replacement water is added to the annular space.

B. Hydraulic Tube Racking

Used for very large diameter shallow wells in sands and gravels, steel tubes are gripped at the surface by an hydraulic mechanism which imparts a slow and continuous semi-rotating movement. The breakdown of horizontal friction allows the weight of the tubes to be applied to vertical penetration. The bottom of the tube column is serrated and, as the tubes descend under their own weight further tubes are welded at the surface. The lower part of the column would be perforated to allow entry for the water.

The gravel inside the tubes is removed by a hammer-grab or a conventional grab if space permits. The only plant required is the racking device and a crane to place tubes and operate the grab.

C. Auger Drilling

This is not generally a water well drilling method in itself but is sometimes combined with another system of making holes. It can be applied very effectively in clay at or near the surface where conditions of infall do not lead to problems with holding the aquifer open. Unstable formation can sometimes be supported by static bentonite or circulated bentonite through hollow auger rods (or tubes).

D. Coring

This type of drilling is usually associated with small diameter exploration work in rock formations and, as such, is little used in water well drilling - except perhaps as part of another major method. However, it can be applied to some larger diameter work and is described in more detail in Part 2.

Basically a tubular cutting shoe (crown) set with industrial diamonds or tungsten carbide is rotated into rock whilst water is fed into the drill pipe to remove the fine cuttings. The central core of rock thus produced is enveloped by a tube above the crown as depth is made. When the tube is full it is lifted to the surface - the initial movement of which snaps off the core with a special catcher - to be emptied and re-inserted into the borehole. The removal of the core can be carried out speedily by wire line methods introduced in recent years.

E. Down-Hole-Motor (Turbine) Drill

In this system a conventional rotary rig is used, but the drill pipe is not rotated. Instead, the pipe is used solely to conduct drilling fluid down to a turbine motor which directly drives the rock bit, the spent mud from which carries out its normal functions in the hole and recirculates.

A variation of this theme is found in a drilling machine powered by submersible electric motors mounted within a device which can either be lowered on solid reverse-circulation drill pipes or on flexible hoses and steel wire ropes. If the latter arrangement is used there is provision for directional corrections to be made by remote control from the surface.

F. Vibration Equipment

A facility for vibration and hammering is now being incorporated into a few special drilling rigs. Added to the standard mechanisms of rotation and downthrust is a unit, mounted above the kelly or swivel, which generates vibratory waves, these being transmitted down the drill pipe and/or casing. Downward or upward hammering forces can also be provided (Fig. 7).

The system uses double-tube drill pipe and there is no annular space as such between it and the formation. The vibration causes a state of semi-fluidity in the particles immediately adjacent to the pipe and almost eliminates friction. This enables the slowly rotating drill pipe to travel down through unlined unstable ground without becoming seized, thereby obviating the need for columns of temporary casing. Cuttings are removed by air or mud circulation which is passed down the pipe annulus, across the cutting assembly, and up the centre drill pipe.

Part 2

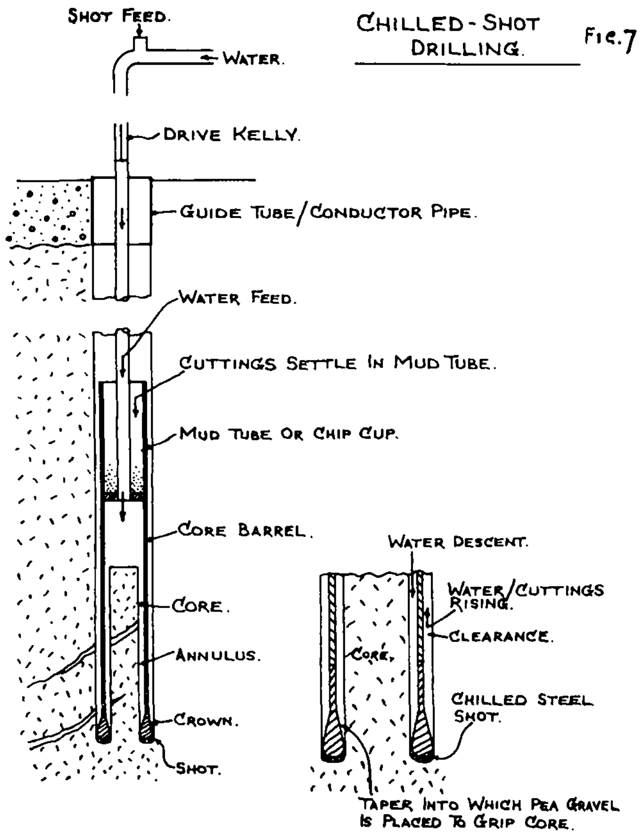
LOW COST DRILLING

From the foregoing general description of methods available to the well driller it will be apparent that most of the modern drilling rigs are costly, complicated and - under the right conditions - efficient.

When we come to examine these conditions it will be seen that severe limitations are placed on the selection of such equipment. Difficult access, weight restrictions, slender lines of communication, limited drilling water, semi-skilled servicing personnel, absence of workshop facilities, unreliable spares back up, adverse climate etc., can combine to render a sophisticated rig immobile very quickly.

It is often under just the above circumstances that a low cost method of drilling may be shown to be the best. Production speed may be low, but not subject to serious breakdown, and the very low capital cost of plant may permit several drilling units to be deployed.

At this point is must be stressed that one vital ingredient cannot be reduced: that of supervisory skill. It is absolutely essential that the driller is thoroughly experienced in the type of work which he is doing and with the equipment that he is using and the below-ground conditions that he is likely to meet. Alternatively, a good but relatively inexperienced driller closely supervised by a drilling superintendent meeting the above requirements may suffice.



CHILLED-SHOT DRILLING. FIG. 7

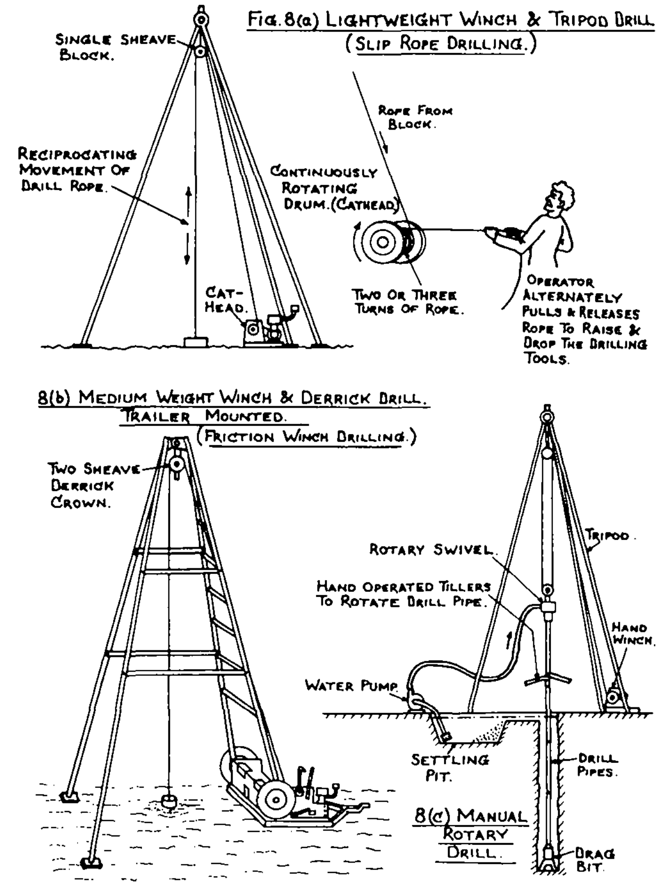


FIG. 8

On the other hand it may not be essential to provide a skilled drilling crew as a locally recruited team may produce one or more men of natural aptitude to receive training and accept responsibility to operate the following two systems.

2.1 Lightweight Cable Tool/Percussion Drilling

One of the cheapest and well-tried methods of mechanically drilling small water wells is that employing a tripod and motor-driven winch (Fig. 8). It is the mechanical version of a system still in use in some developing countries where the equipment is constructed from local materials and the energy is provided by manpower.

Although conventional lightweight cable tools are employed, there is no mechanism to impart a 'spudding' or reciprocating motion to the tools. This is achieved by one of two ways: (a) the 'slip-rope' method where the drilling rope is loosely wound round a constantly rotating drum and the driller alternately pulls and releases the free end of the rope, or (b) the friction winch method where the driller engages and disengages a purpose-built clutch to raise and drop the tools.

At first sight this appears to be a crude and tiresome method but in practice two drillers can achieve remarkably high productivity.

The equipment consists of a tripod made of timber or steel which carries a crown wheel at the top. There may be cross bracings to aid stability. The height is usually dictated by the lengths of lining tubes to be handled.

Power is provided by an i.c. engine driving a reel (cat-head) for slip rope drilling or a clutched drum for winch drilling. Alternative power may be provided by an agricultural tractor or even the bare wheel centre of a vehicle.

Several manufacturers offer light drilling rigs weighing about 1500 Kgs. which can be towed behind a Land Rover type vehicle and which are easily erected and dismantled. Depths to 100 m are possible under ideal conditions and starting diameters up to 375 mm are quite common. Fitted with a 15 BHP air-cooled engine, these rigs cost approximately £7,000 with tools.

2.2 Medium Weight Cable Tool/Percussion Drilling

In order to uprate the capacity of the previously described equipment it is only necessary to increase the strength of the components and the weight of the drilling tools. The slip rope method becomes impractical with heavy tools, therefore, it is usual to install a heavy duty friction winch which may be of the double-drum type, in which case one line would be used for percussion drilling and the other for handling casing. It is not essential to add the complication of a sandline; instead, the drilling line is fitted with a quickly detachable connection just above the tools which allows them to be removed and temporarily replaced by a bailer or sandpump for the purpose of removing the cuttings from the well.

A power unit of 20/30 BHP is necessary and may be an air-cooled diesel engine, although where servicing facilities for diesel injectors and fuel pumps are not available a petrol engine may be preferred. In the latter case a light high-revving engine might be attractive from the point of view of cost and weight but, in the interests of long life, should be avoided. Conventional cable percussion tools are generally used but locally fabricated tools have been successfully employed in a number of countries.

It should be noted, however, that low cost 'home made' tools may prove a disadvantage in the long term unless of substantial design. A cheap tool becomes expensive when lost down the hole.

Regarding drilling capacity, wells of up to 1.5 m diameter are not unknown and, depending on the derrick rating, depths in excess of 200 m are possible.

The cost of such equipment largely depends on the design and materials of the derrick, but would be in the order of £14/25,000 with tools. A comparable trailer-mounted percussion rig would be approximately £30/60,000 with tools.

2.3 Chilled-shot Rotary Drilling

In some parts of the world where all or part of the ground formation consists of firm rock it may be worth considering the application of the Chilled-shot method of rotary drilling which was in common use in the U.K. until 1950.

To the existing equipment described under section 2.2 above it is only necessary to add a rotary table, water swivel, drive kelly, drill pipe and a core barrel to enable large diameter coring to take place. A simple belt drive from the winch to the rotary table would provide the power transmission.

The core barrel is lowered into the ground below the drill pipes which in turn are rotated at the surface by a drive kelly sliding through a matching aperture in the rotary table. The bottom of the barrel is thickened and tapered upward internally to form a crown which cuts a hole of sufficient size to accommodate the barrel above and produces a core which passes freely into the barrel.

A small flow of water is pumped down the drill pipes and some chilled steel shot fed down with it. This shot falls to the bottom where it becomes trapped beneath the crown. A relatively light loading is applied, but as only a few pieces of shot are supporting it, the rock fails under the face of the crown and progress is made. The weight of the rods on the rotating shot crushes the rock beneath.

The shot itself is worn away and must be replaced periodically by adding more to the water feed.

The fine cuttings produced are washed up past the barrel and fall back into a sand tube receptacle formed in the top of the barrel; meanwhile the central core is 'swallowed' by the barrel. When the barrel is full, pea gravel is pumped into the annuler space between the core and the barrel to lodge in the tapered crown and grip the core for removal.

This method has been discontinued in those parts of the world where labour costs are high, due to the rather slow rate of drilling. However, where labour is inexpensive, there is no other system which can drill firm rock at medium to large diameter for so small a capital outlay and with such low running costs.

In the event of there being more than two or three metres of overburden present, the upper section of the well would be drilled with the cable tools.

2.4 Manual Rotary Jetting

Where the strata consist of clay and stable sands it is possible to drill quite rapidly at up to 300 mm diameter employing the rotary jetting method with rotation provided by manpower (Fig. 8C) Depths to 50 m are possible and the only motorised item is that required to drive the circulating pump - although in some cases a large hand powered pump has been used.

The equipment consists of a tripod or derrick to handle the drill pipe and casing, a hand-winch, a water swivel, and a column of drill pipe to which is screwed the cutting bit.

To operate the drill the bit and the first drill pipe are suspended in the derrick and lowered onto the ground as water is pumped through them. Rotation is applied manually through 'tillers' or chain tongs whilst downward feed is provided from the hand winch.

The cuttings are dislodged by the bit and carried to the surface where they separate from the water whilst in the settling pit. Drill pipe is added to the column as depth increases until sufficient penetration of the aquifer has been achieved, whereupon the tools are removed and the casing and screen tubes inserted.

The above method is cheap, labour intensive, and can be used to drill wells very rapidly under suitable geological conditions.

2.5 Conclusion of Part 2

There is no one perfect drilling rig for all conditions.

A very large drilling programme financed by an international agency would probably warrant the use of modern rotary rigs of the simplest design, i.e. conventional rotary tables, chain pull-down, minimal hydraulics and a slow speed rather than automotive type power unit.

For a large self-financed drilling scheme in a developing country where hard rock is unlikely to be encountered and the ground consists of cohesive sedimentaries, the cable tool/percussion method may be most suitable. It would be worth considering trailer-mounted rigs, preferably from a manufacturer offering a reputable well tried model with adequate spares back up. Hydraulic attachments should be avoided if possible unless required for hard rock drilling conditions where a hydraulic rotary attachment may prove useful.

Where cost is the prime consideration the friction winch and tripod equipment with cable tools, or the manual rotary method with power pump may be the most suitable choice, geological conditions permitting.

Part 3

MOTORPUMPS

There are two categories of motor pumps used for abstracting water from wells: shallow well pumps and deep well pumps.

The former are generally of the suction type where the pump/power unit is placed at ground level and a suction line enters the well to draw water to the surface.

This places a practical limit of about 7 metres on the vertical distance between the pump and the pumping water level in the well - although in a large diameter well it may be possible to fix the pump on a staging down the well just above standing water level.

Deep well pumps are installed with their pump-end submerged below the water table, and in the case of the electric submersible kind, the motor is also submerged.

3.1 Shallow Well Pumps

A. Centrifugal Pumps The universally available centrifugal pump and engine unit possesses some attractive advantages for drawing water from a shallow well, e.g. simplicity, ease of maintenance, proven performance, and robustness. At slightly higher cost but with no more complication a self-priming type offers the advantage over the standard design by reducing the 'dry pumping' period if a foot valve should fail, thereby prolonging the life of the gland packings. As in all low cost long term power requirements a rugged engine should be selected.

B. Jet Pumps Although usually associated with deep wells, the above-ground centrifugal pump has been modified to incorporate a jet feature in order to increase the discharge pressure far beyond that possible with the standard type. This is achieved by re-circulating some of the pumped water through a jet and venturi chamber in the suction side of the pump, thus providing increased thrust. This three-fold increase in pressure (head) is, however, gained at the expense of volume which is reduced proportionally.

C. Diaphragm Pumps In cases where only small quantities of water are required and the pumping water level is near to the surface it may be worth considering the diaphragm pump - especially if the well tends to discharge solids with the water.

Low initial cost, simplicity, with slow speed moving parts and ease of maintenance all combine to make this an attractive pump under the right conditions. The main disadvantage is its inability to generate much discharge pressure and its low volume output.

D. Chain Pumps These are only mentioned in passing as they require sufficient working space within the well such as to rule them out for use in drilled wells.

Of either the continuous bucket or multi-disc type, the former operates on the same principle as the navigation dredger, whilst the latter draws close-fitting discs up to the surface through a partially submerged pipe.

They can be fabricated in local workshops from readily available materials and are surprisingly efficient if soundly designed.

Note: Reciprocating Pumps as for deep wells can be used for suction lift or with working cylinders below the water table.

3.2 Deep Well Pumps

In some countries the choice of pumping plant is partly influenced by tradition, e.g. vertical spindle pumps are installed where perhaps electric submersible units would be more suitable.

FIG. 9. VIBRO-PERCUSSION ROTARY DRILLING.

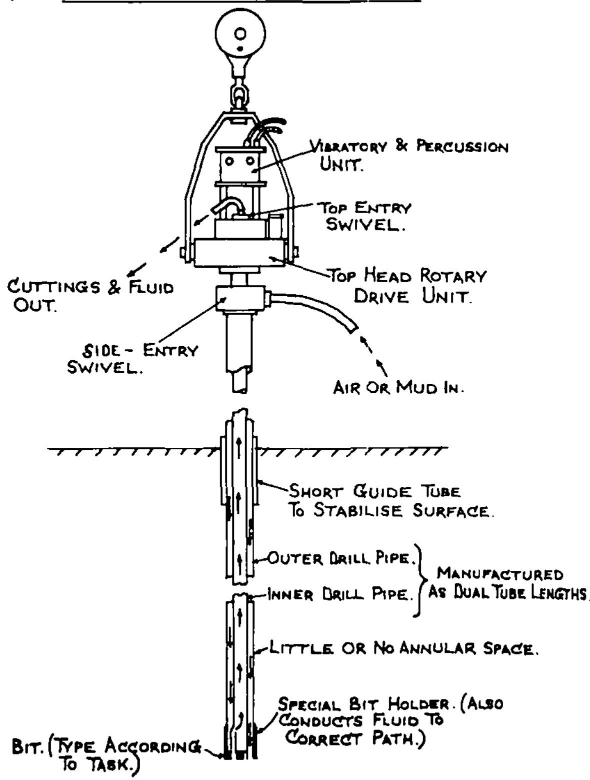
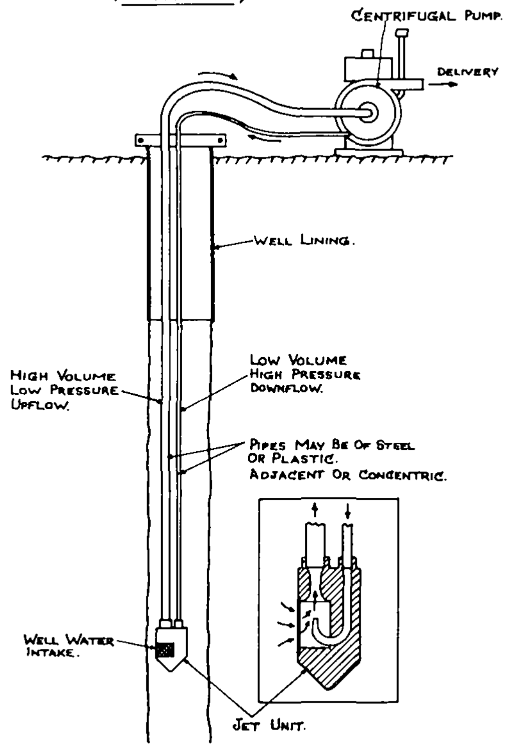


FIG. 10. DEEP WELL JET PUMP. (DIAGRAMMATIC)



We have seen the difficulties which arise when deciding on the most suitable method for drilling the well. There are also problems when it comes to selecting the pumping method, but perhaps less so because the parameters are more in focus where the requirement is to lift a certain quantity of water against a known pressure for a specific task.

A. Reciprocating Pumps Also known as piston, cylinder or plunger pumps, they evolved many centuries ago and with the advent of steam power grew to enormous size in the industrialised countries, only to be relegated to more modest duties when efficient vertical spindle pumps became available.

Reciprocating pumps are now manufactured in a range covering most installation depths and for moderate outputs. Provided that no sand enters the well to wear the piston assembly, most of these units will run for extremely long periods with minimal attention - especially those powered by a heavy single-cylinder oil engine running at low r.p.m.

A typical 6 BHP pumping plant might lift 110 l/min. against 150 m head or 230 l/min. against 60 m head and the cost ex works would be in the region of £8,000.

Advantages are relatively low initial and running costs, simplicity, reliability and ease of repair and maintenance. Disadvantages are the necessity to have hoisting facilities to remove the sucker rods and piston (but required for servicing most pumps), a rather limited output from small wells due to the cylinder diameter restriction, and the need to drill a reasonably straight well to prevent sucker rod chafing.

The reciprocating motion to the pump rods is obtained from a beam driven by a crank or direct from a crank.

B. Jet Pumps The common deep well jet pump consists of a centrifugal pump mounted at ground level to which is attached two pipes of unequal diameter. These pipes pass down the well to a point below the anticipated pumping level and are connected to a jet and venturi assembly.

Water at high pressure is pumped down the smaller pipe and passes through the jet into the venturi tube thus causing a pressure drop which draws well water into the combining cone (Fig. 10). The incoming water mixes with the jet water and flows up the larger pipe to the centrifugal pump where it is forced into the delivery main whilst a proportion is continuously by-passed to operate the jet.

The advantages of the deep well jet pump are that there are no working parts down the well, the jet unit can be lowered on plastic flexible pipes thus obviating the need for a crane or tripod, and the pump unit is simple.

Disadvantages are the relatively inefficient performance - especially for deeper installations - and the need to maintain full prime in the pump to ensure a rapid and reliable start-up.

C. Vertical Spindle Turbine Pumps This type of pump superseded the old reciprocating plant and enabled very large quantities of water to be drawn from wells of small diameter and, by multiplying the stages, to generate high heads.

The pump, of axial, centrifugal or mixed flow design, is installed down the well below a column of pipes (rising main) which carry a concentric drive shaft supported by bearings located at rising main pipe joints. At the surface this shaft is driven by a power unit which may be direct-coupled (as with an electric motor) or be indirectly driven through a right-angle gearbox. Some shaft lubrication is provided either by the untreated well water, a filtered by-pass, or shielded oil feed.

A very wide range of duties are available and a variety of power sources can be utilised. Also some control of the output may be obtained by varying the speed.

Disadvantages are the requirement for a reasonably straight well, fully trained personnel for installation, and the number of moving parts subject to wear.

D. Electric Submersible Pumps Very commonly used, this type of pump consists of a centrifugal turbine pump to which is directly coupled an electric motor. The power sizes can be from a fractional h.p. to several hundred h.p. and in all cases the motor/pump unit is designed to be as slim as possible without unduly sacrificing efficiency.

The unit is suspended on rising mains from the surface to which is clipped the electric power cable. The motor casing is cooled by the movement of the surrounding well water whilst the internal heat is transferred by a closed circuit of clean water within the pump.

Although the long slim motor/pump unit is not quite as efficient as a surface motor, the absence of a long drive shaft and bearings balances this deficit against the turbine pump. The electric submersible pump can be installed in crooked hole (clearances permitting) without fear of operational failure. It requires less expertise in install and in clean non-aggresive water may run for several years without attention. For deeper wells it is cheaper than a vertical spindle type.

Disadvantages are the necessity for a well yielding solids-free water, the high cost of motor repairs, and the supply of electricity at a steady voltage; many developing countries have public electricity supplied at such varying voltages between phases that the consumer's plant may be damaged without very substantial protective instrumentation.

E. Helical Rotor Pumps Often known as a 'Mono' pump, the helical rotor pump is a positive displacement rotary pump comprising a helical rubber stator in which a steel helix revolves (Fig 11). The pump is suspended in the water by the rising main pipes through which the drive shaft is carried in bearings to the surface. The power source may be selected according to the site circumstances, some of the smallest units being manually operated.

These type of pumps are manufactured in a range of sizes covering the low to medium flow rates, e.g. from 20 to 850 l/min. and for heads as high as 230 m. Regarding costs, these range from £1,200 to £5,000 excluding power unit. Power requirements run from ½ to 40 BHP.

The major advantage of the helical rotor pump is its ability to pump some sand without suffering serious damage; it is also efficient and very robust and can be run at variable speeds.

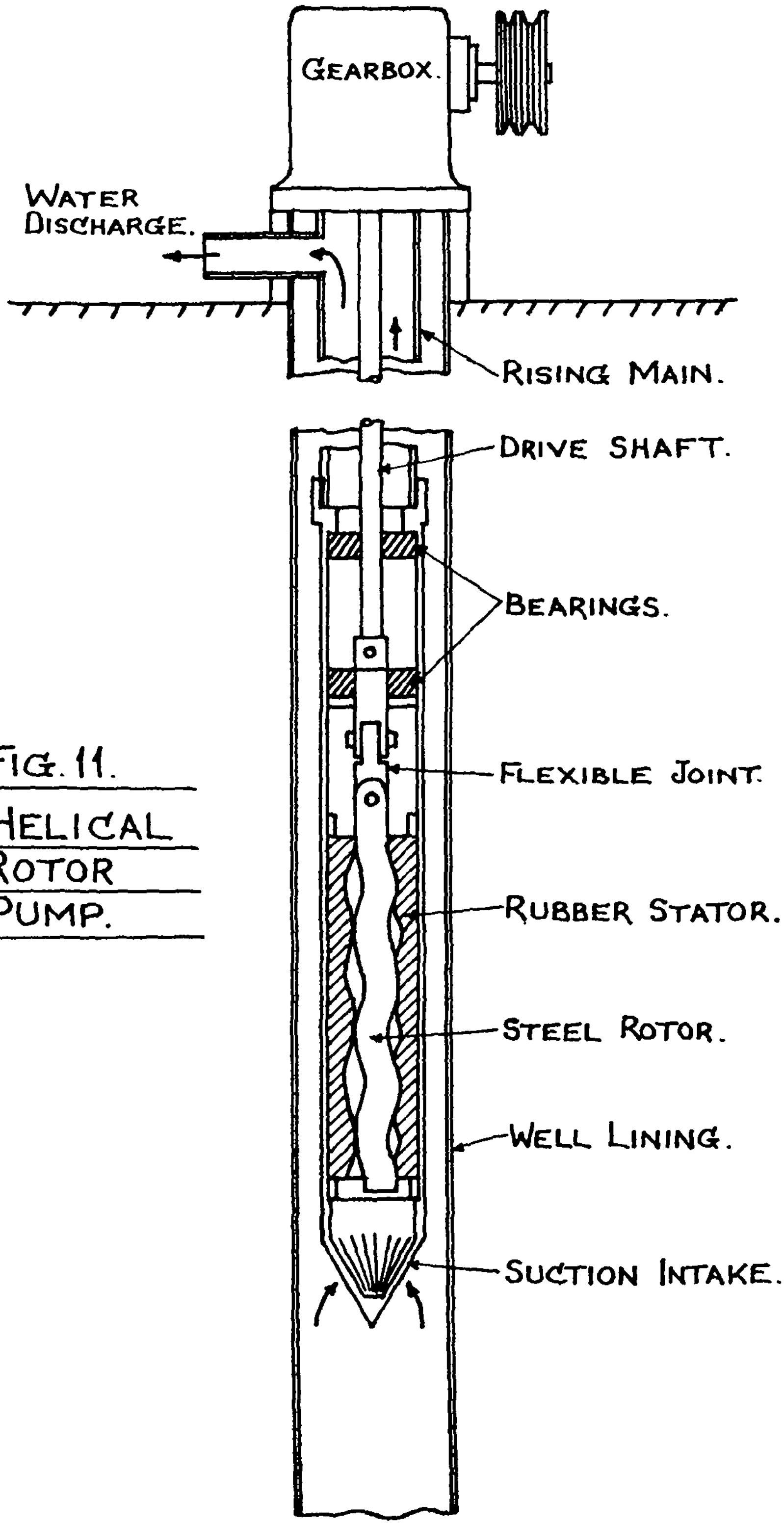


FIG. 11.
HELICAL ROTOR PUMP.

3.3 Conclusion of Part 3

For shallow well water abstraction where the pumping water level does not exceed 7.0 m below surface it is difficult to improve on the self-priming centrifugal pump, especially where high outputs are obtainable, whilst for very modest requirements from water standing near to the surface a diaphragm pump would provide a cheap and easily maintained pumping unit.

The majority of modern wells are drilled down to a lower aquifer with the aim of ensuring a reliable output and this brings them into the deep well category; in consequence one has to turn to a more complicated and therefore more costly method of pumping from them.

For low outputs from a deeper sand-free well of small diameter the reciprocating pump has much to commend it. Running at low speed for long periods it will charge a storage tank capable of satisfying peak demands such as the watering of animal herds, as well as regular village requirements.

Again with the accent on moderate running speeds, but for greater outputs, the helical rotor pump is a natural choice - especially when powered by an engine of proven reliability.

Finally, opinions will always differ on the interpretation of 'low-cost' and 'appropriate' technology - depending on the position of the viewer. However, one frequently committed error has been the well-intentioned expenditure of one hundred per cent on a water well scheme in a developing country which, upon completion allows little or nothing at all for maintenance. Without a thorough training programme for indigenous staff and provision for service spares covering several years any such scheme is doomed to failure - and the magnitude of the failure is directly proportional to the level of the technology.

PUITS FORES MECANIQUEMENT ET MOTO-POMPES

par P.K. Cruse

Résumé

Les anciennes méthodes de forage faisaient appel à des systèmes à percussion, encore usités actuellement - outillage de forage à cable.

Dès que les moteurs furent utilisables pour des unités mobiles, les systèmes rotary firent leur apparition, notamment à la suite des développements des forages pour puits de pétrole - des matériels sont ainsi devenus complexes, coûteux et particulièrement performants.

La technologie de forage des puits d'eau à faible coût, et compte tenu des conditions spécifiques d'un pays en voie de développement, doit combiner la fiabilité des machines modernes avec la simplicité des méthodes traditionnelles. Elle nécessite un personnel compétent. Ceci s'applique également aux moyens d'extraction de l'eau des puits déjà forés.

Dans cet article, l'on trouvera une présentation des divers systèmes de forage et de pompage couramment utilisés ainsi que des suggestions relatives aux méthodes les plus adaptées aux pays en voie de développement.

Paper 7

Poste de traitement d'eau complet sans electricité ni mecanique

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- RESUME :

Les besoins des pays en voie de développement en postes de traitement complet, d'eau destinée à la consommation, se caractérisent principalement par la définition d'une technologie répondant aux critères suivants :

1 - Facilité de mise en oeuvre d'ensembles mobiles de petits ou moyens débits, à l'échelle du village ou de la petite agglomération.

2 - Equipement simple, facile à exploiter.

3 - Maintenance limitée presque exclusivement au matériel disponible dans le pays.

4 - Coût d'achat et coût d'exploitation les plus limités possible.

Il est proposé des postes de traitement qui permettent de mettre en oeuvre une décantation avec coagulation, une filtration, une distribution de réactifs et de désinfectant, ainsi qu'un stockage d'eau traitée. Ces postes ne demandant pas de Génie civil sont transportables sans convoi spécial et sont particulièrement remarquables par :

- l'absence de besoin énergétique (électricité, fuel),

- l'absence d'organes mécaniques en mouvement,

- l'emploi de doseurs sous pression et gravitaires simples,

- la qualité de la technologie employée et sa simplicité.

Le coût d'exploitation se limitant à la consommation de réactifs et le coût d'achat étant limité, une telle solution conduit à un coût minimal du m3 d'eau produit.

...

- INTRODUCTION :

Les besoins en poste de traitement d'eau, destinés à la consommation, des pays en voie de développement comme ceux des petites agglomérations dispersées dans les pays déjà développés, se caractérisent par la définition d'une technologie répondant aux principaux critères suivants :

. Facilité de mise en oeuvre d'ensembles de petit ou moyen débit à l'échelle du village ou de petites agglomérations.

. Facilité d'exploitation et de surveillance.

. Maintenance réduite et faisant presque exclusivement appel aux matériels disponibles dans le pays.

. Coût d'achat et coût d'exploitation aussi limités que possible.

I - SCHEMA DE TRAITEMENT :

Il est bien évident que le choix de la technologie d'un poste de traitement d'eau est lié à la qualité des eaux à traiter et qu'il ne saurait être question de définir un schéma universellement valable pour tout type d'eau de surface comme d'eau souterraine. Nous nous bornerons ici au cas d'eaux de surface suffisamment turbides pour nécessiter une chaîne de traitement complet, en circuit ouvert, fonctionnant sans électricité sur le site et comprenant :

- une préchloration éventuelle,
- une floculation-décantation avec mise en oeuvre d'un coagulant et si nécessaire d'un réactif correcteur de pH,
- une filtration,
- une neutralisation pour ajustement du pH de l'eau traitée,
- une désinfection finale,
- un stockage d'eau traitée.

Nous admettrons également que :

- Il ne s'agit pas d'eaux brutes particulièrement polluées et susceptibles de nécessiter des étapes de traitement complémentaires.

- L'eau brute est amenée à disposition du poste de traitement, par exemple par une adduction gravitaire ou par un pompage situé hors du site.

- L'eau traitée est à stocker dans une réserve posée au sol, d'une capacité de 1 à plusieurs heures suivant les conditions de distribution.

...

II - DEBITS :

Si l'on considère un niveau de besoins de 50 à 100 litres/jour/habitant, on peut satisfaire la demande d'agglomérations jusqu'à 10.000 habitants avec des postes d'un débit de 21 à 42 m3/h en marche 24/24 h.

Nous considèrerons donc des postes allant au moins de 5 à 50 m3/h, ce débit étant susceptible d'être doublé, ne serait-ce que pour un fonctionnement qui peut être limité à 12/24 h si les possibilités de contrôle le nécessitent.

III - POSTES DE TRAITEMENT TRANSPORTABLES, FONCTIONNANT SANS ELECTRICITE DE 5 à 30 m3/h, POUR EAUX MOYENNEMENT CHARGEES :

Les schémas n° 1 et 2 montrent le principe du poste préconisé qui n'utilise ni électricité ni appareil mécanique en mouvement. Ils comprennent tous deux :

. Pour la décantation, un décanteur-floculateur sous pression.

. Pour la filtration, un filtre autolaveur.

. Pour l'injection des réactifs de coagulation, de correction de pH et de désinfection, une distribution sous pression ou gravitaire de solutions avec des doseurs sans mécanique susceptible de coincement.

. Pour le stockage de l'eau traitée, une ou plusieurs citernes posées au sol.

1) Décanteurs :

L'emploi de décanteurs sous pression posés au sol permet l'alimentation d'un filtre ouvert et d'une citerne d'eau traitée, également installés au sol.

a - Pour les postes de petit débit (5 m3/h) un décanteur vertical du type

CIRCULATOR (fig.3) permet d'obtenir un mélange intime des réactifs à l'eau brute et sa floculation, sans organe mécanique, uniquement par recirculation interne des boues floculées concentrées, déjà formées et décantées. Cette recirculation est obtenue par un éjecteur à faible perte de charge où les vitesses locales sont limitées pour éviter le bris systématique des flocs et dont le refoulement dans un cône lentement divergent évite toute turbulence également néfaste à la floculation.

L'eau décantée est reprise uniformément sur la section supérieure du décanteur par un collecteur perforé où, là encore, les vitesses d'approche sont faibles pour éviter toute succion locale importante pouvant engendrer une remontée des boues en cours de décantation.

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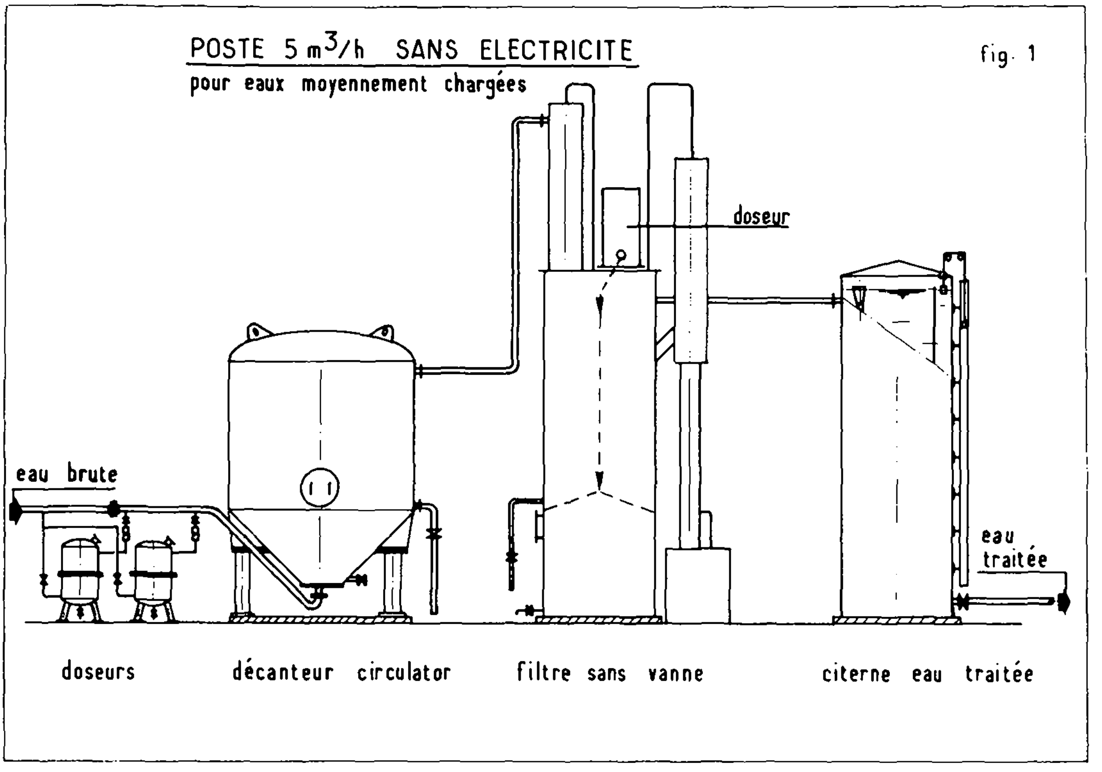
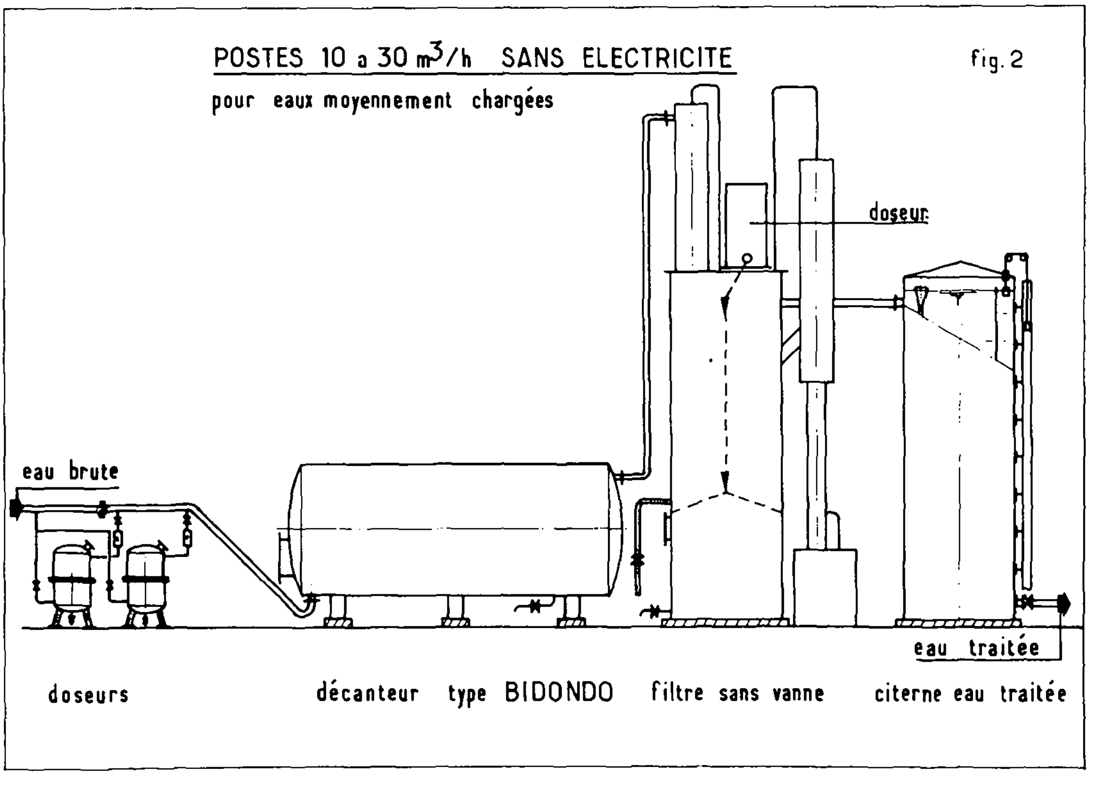

POSTE 5 m3/h SANS ELECTRICITE
pour eaux moyennement chargées
fig. 1
doseur
eau brute
eau traitée
doseurs
décanteur circulator
filtre sans vanne
citerne eau traitée


POSTES 10 a 30 m3/h SANS ELECTRICITE
pour eaux moyennement chargées
fig. 2
doseur
eau brute
eau traitée
doseurs
décanteur type BIDONDO
filtre sans vanne
citerne eau traitée

Les boues stockées dans la partie conique inférieure viennent se déverser dans un concentrateur d'où elles peuvent être extraites soit manuellement en discontinu, si le responsable de l'exploitation est à proximité,ou en continu à petit débit en cas de simple passage d'un surveillant dans la journée.

b - Pour les postes de débit moyen 10 à 30 m3/h, les décanteurs, toujours sous pression, sont horizontaux (fig. 4). Ils travaillent sur le même principe de floculation par recirculation des boues déjà formées à l'aide d'un éjecteur.

Suivant leur capacité, les postes comportent un ou deux décanteurs standards de 10 ou 15 m3/h de capacité unitaire qui sont utilisés depuis longtemps en Afrique.

Ces deux types de décanteurs, verticaux et horizontaux ont l'intérêt d'être peu sophistiqués, de ne pas se boucher et de ne nécessiter aucun entretien mécanique puisqu'ils sont dépourvus de tout organe en mouvement. Toutefois, bien que largement dimensionnés, ils ne peuvent pas stocker une quantité importante de boues, aussi doit-on limiter leur emploi à des eaux brutes ne dépassant pas 500 mg/l de matières en suspension.

2) Les filtres :

Les filtres doivent pouvoir être lavés avant d'atteindre leur crevaison et cela en l'absence de l'exploitant qui n'est pas supposé être à proximité en permanence : pour cela, la seule solution consiste à utiliser un ou plusieurs filtres autolaveurs tels que schématisés fig. 5 .

L'eau décantée arrive au sommet d'un pot de mise en charge (A) et chute vers un compartiment de dégazage (B) avant de pénétrer dans le compartiment filtre (C). Ce compartiment de dégazage évite un entrainement d'air en direction du filtre qui dérèglerait le système d'amorçage automatique du siphon de lavage (S).

Après passage sur une couche de sable fin (D) ou sur un lit bicouche, l'eau filtrée est reprise par un réseau de tuyauteries perforées (E) en plastique, disposées en étoile, vers une cheminée centrale (F) qui l'amène dans le compartiment supérieur (G). Ce compartiment sert au stockage de l'eau de lavage et de départ d'eau filtrée (H) vers la citerne d'eau traitée.

Après un lavage, le compartiment d'eau de lavage est à niveau bas. Pendant une demi-heure environ, l'eau filtrée sortant du filtre sert à remplir ce compartiment et la citerne d'eau traitée n'est plus alimentée : ceci entraine donc déjà la nécessité de disposer d'une réserve d'eau traitée d'une capacité d'au moins une heure.

Le compartiment d'eau de lavage, une fois rempli,la citerne d'eau traitée est à nouveau alimentée et le niveau d'eau décantée s'élève progressivement dans le pot de mise en charge au fur et à mesure de l'encrassement du lit filtrant. Pendant ce temps, de l'air est comprimé dans le siphon (S).

Lorsque le lit filtrant atteint l'encrassement maximal fixé par construction, l'air comprimé au col du siphon s'échappe de lui-même par la tuyauterie d'amorçage (J) et le siphon s'amorce automatiquement, assurant le lavage du lit filtrant par mise en expansion, pendant un temps de 4 à 5 minutes.

...

fig. 3/4

POSTES 5 à 30 m^3/h SANS ELECTRICITE – DECANTEURS SOUS PRESSION

DECANTEUR VERTICAL
TYPE CIRCULATOR

DECANTEUR HORIZONTAL
TYPE BIDONDO

Ⓐ entrée d'eau brute
Ⓑ éjecteur de recirculation des flocs
Ⓒ concentrateur de boues/extraction
Ⓓ reprise de l'eau décantée

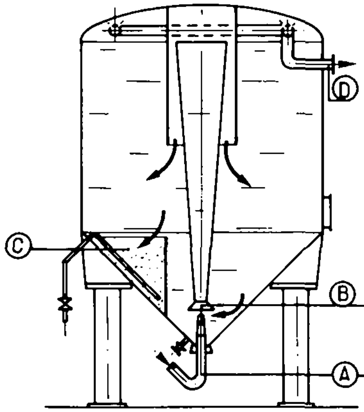


fig 3

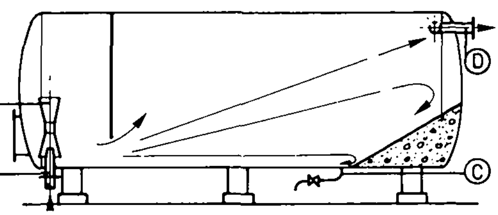
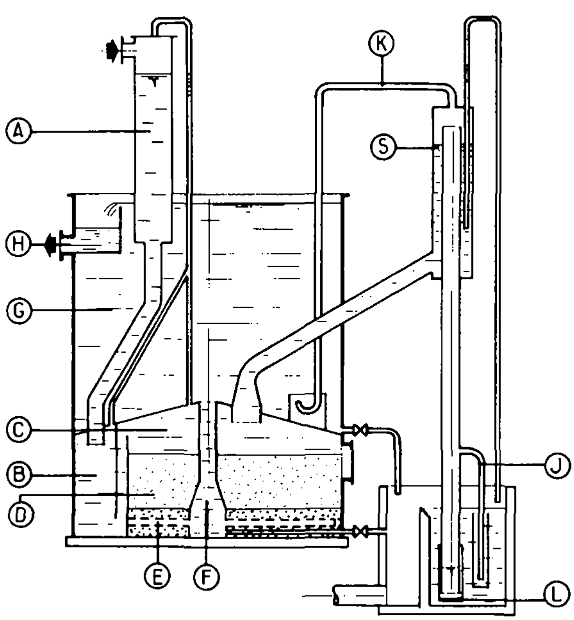


fig 4

fig. 5

FILTRE SANS VANNE



Ⓐ pot de mise en charge de l'eau brute
Ⓑ compartiment de dégazage
Ⓒ filtre
Ⓓ lit filtrant
Ⓔ collecteur d'eau filtrée
Ⓕ cheminée centrale
Ⓖ compartiment eau lavage
Ⓗ départ d'eau filtrée
Ⓙ tuyauterie d'amorçage
Ⓚ tuyauterie de désamorçage
Ⓛ limiteur débit eau de lavage
Ⓢ siphon

Quand le niveau d'eau dans le compartiment de lavage (G) atteint le minimum fixé, une tuyauterie (K) met à l'atmosphère le col du siphon et le désamorce.

L'eau décantée, qui n'a pas cessé d'arriver pendant le lavage, est à nouveau filtrée et le cycle recommence.

L'intérêt d'un tel type de filtre est évident :

- Il se lave automatiquement en l'absence de toute intervention humaine pour un encrassement constant fixé à l'avance et ajusté, si nécessaire, à la mise en route. Le lit filtrant ne risque pas d'être "surencrassé" au point de ne pas pouvoir retrouver son état initial de propreté après lavage. Ce dispositif donne donc l'assurance que le lavage sera toujours fait à temps.

- Le gain d'encrassement du lit filtrant est ajustable et atteint un maximum de 1,50 à 1,70 m suivant la vitesse de filtration nominale. Il y correspond une chute géométrique entre le pot d'entrée et la sortie d'eau filtrée de 1,90 à 2,10 m; cette chute est nécessaire pour avoir un temps de filtration entre lavages suffisamment long et une perte d'eau de lavage limitée, ce que l'on ne peut obtenir avec des filtres ne disposant que de 1,50 m de chute soit 1 m à 1,10 m de gain d'encrassement.

- La qualité de l'eau filtrée est assurée par une granulométrie de sable suffisamment fine pour ne pas risquer de crevaison prématurée dans les conditions d'encrassement définies précédemment.

- La quantité d'eau de lavage disponible dans le compartiment situé au-dessus du filtre est constante et de 2,80 à 3 m3/m2 de filtre : elle est bien adaptée à la perte de charge maximale fixée par construction et est toujours disponible.

- La vitesse moyenne de lavage est réglée une fois pour toutes à la mise en service en fonction de la granulométrie du milieu filtrant, par ajustement d'un dispositif (L) d'obturation du débouché aval du siphon.

- Enfin, il faut dire et avoir conscience que ce type de filtre est conçu pour fonctionner avec des eaux bien décantées, sinon, le pourcentage de perte d'eau de lavage devient rapidement important : il est donc nécessaire de disposer d'une décantation efficace et conçue avec un large dimensionnement.

3) Citerne d'eau traitée :

L'eau traitée peut être utilisée directement sur place, ou peut alimenter gravitairement un réseau plus ou moins important. Suivant le mode de distribution, la réserve d'eau traitée doit pouvoir représenter un stockage d'une à plusieurs heures. Ceci est réalisable par utilisation d'une ou plusieurs citernes verticales mises en parallèle.

Ces citernes couvertes sont en matière plastique de qualité alimentaire et équipées de tous les accessoires nécessaires (indicateurs de niveau, trou d'homme, vidange, trop-plein, etc). Grâce à leur positionnement sur le sol, ces citernes permettent de disposer d'une mise en charge hydraulique au départ de la distribution.

...

4) Réactifs :

Il est bien évident que l'on a intérêt à limiter au plus juste le nombre de réactifs mis en oeuvre pour simplifier l'exploitation, mais la nature des eaux à traiter en impose le choix.

Nous supposerons ici que l'on doit pouvoir mettre en oeuvre une partie ou la totalité des réactifs suivants :

. Au niveau de la décantation :

- un composé chloré pour la préchloration, qui peut être de l'eau de Javel ou de l'hypochlorite de calcium (HTH),
- un coagulant, en général du sulfate d'alumine,
- un correcteur éventuel de pH (carbonate de soude ou soude).

. Au niveau de la sortie d'eau filtrée :

- un correcteur de pH final (carbonate de soude ou soude),
- un composé chloré pour la désinfection.

a - Doseurs sous pression pour injection avant décantation :

Les différents réactifs en solution à injecter peuvent être dosés par des pots à déplacement placés au sol tels que représentés fig. 6 . Ils sont constitués d'une poche en matière plastique où l'on place une solution préparée à concentration constante dans un bac auxiliaire. Cette poche délivre un débit constant et ajustable, de solution à concentration constante, par compression extérieure assurée par la pression différentielle donnée par un diaphragme placé sur la tuyauterie d'amenée d'eau brute.

Ainsi, réalise-t-on une distribution de réactif proportionnelle au débit d'eau à traiter quoiqu'il soit recommandé de travailler à débit constant pour simplifier le contrôle du traitement.

Le débit de solution est repéré par un rotamètre placé sur la solution claire distribuée et donc pas susceptible de bouchage.

b - Doseurs gravitaires pour injection dans l'eau filtrée :

Les réactifs de correction de pH et de désinfection sont injectés au débouché de l'eau filtrée dans la réserve de lavage du filtre. Comme cette réserve représente un temps de contact supérieur à 30 minutes, elle donne l'assurance d'une bonne désinfection avant stockage dans la citerne d'eau traitée.

Des doseurs gravitaires très simples (fig. 7) permettent de distribuer les solutions de carbonate de soude ou de soude ainsi que la solution désinfectante.

...

- Ce doseur consiste en un flotteur lesté équipé d'un orifice débitant à l'atmosphère sous une charge constante au fur et à mesure que le bac se vide. La solution dosée est évacuée par une tuyauterie souple reliée au point d'injection.

- Le réglage du débit est obtenu par obturation partielle de l'orifice, avec une vis pointeau située au centre du flotteur : l'étalonnage est aisé : il suffit de jauger le débit avec une éprouvette graduée en plastique placée au débouché du doseur,avant son départ pour l'injection.

- Les 2 doseurs de correction de pH et de désinfection sont installés au sommet du filtre sans vanne dont la réserve supérieure est couverte pour éviter l'entrée des mouches, moustiques ou autres insectes.

Ainsi, ces 2 types de doseurs sous pression et gravitaire permettent le dosage simple et précis de diverses solutions, sans emploi de robinet à flotteur ou d'autre organe mécanique susceptible de coincement.

5) Constitution des différents postes de traitement :

Le tableau (fig. 8), définit 5 postes adaptés aux débits de 5 à 30 m3/h.

. Comme on peut le constater, aucun élément n'a un diamètre supérieur à 2,60 m et une longueur supérieure à 5 m : aucun transport exceptionnel n'est donc à prévoir.

. Si nécessaire, deux groupes semblables peuvent être accolés en conservant un seul doseur par réactif : il est donc possible, avec ces équipements, de traiter des débits pouvant atteindre 60 m3/h.

. Ces différents postes complets peuvent être simplifiés par suppression de la décantation lorsque la qualité de l'eau brute permet de se limiter à une coagulation sur filtre.

IV - POSTES DE TRAITEMENT TRANSPORTABLES, FONCTIONNANT SANS ELECTRICITE, DE 10 à 50 m3/h POUR EAUX CHARGEES :

La décantation d'eaux susceptibles d'être assez chargées en matières en suspension nécessite la possibilité de stocker un volume de boues suffisamment important, d'une part pour en permettre le tassement et, d'autre part, pour éviter de voir le décanteur totalement envahi par les boues. Les décanteurs sous pression doivent alors être remplacés par des décanteurs ouverts avec un volume conséquent, spécialement réservé aux boues.

1) Schéma de traitement :

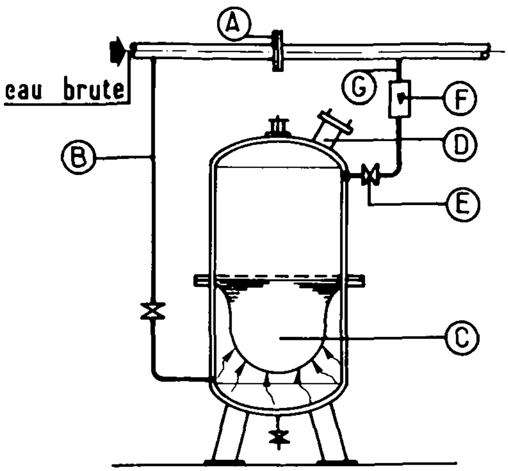
Le schéma d'un poste complet (fig. 9) ne diffère du schéma précédemment examiné, que par la partie décantation, les filtres, citernes d'eau traitée et doseurs de réactifs restant semblables.

...

DOSEURS DE SOLUTION

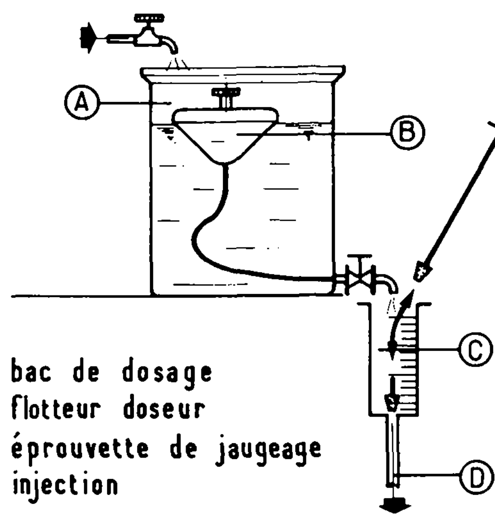
fig. 6_7

DOSEUR SOUS PRESSION



Ⓐ diaphragme
Ⓑ eau motrice
Ⓒ membrane
Ⓓ orifice de chargement de solution
Ⓔ vanne de réglage de débit
Ⓕ rotamètre
Ⓖ injection

DOSEUR GRAVITAIRE



Ⓐ bac de dosage
Ⓑ flotteur doseur
Ⓒ éprouvette de jaugeage
Ⓓ injection

POSTES DE 5 A 30 M3/H SANS ELECTRICITE

FIG.8

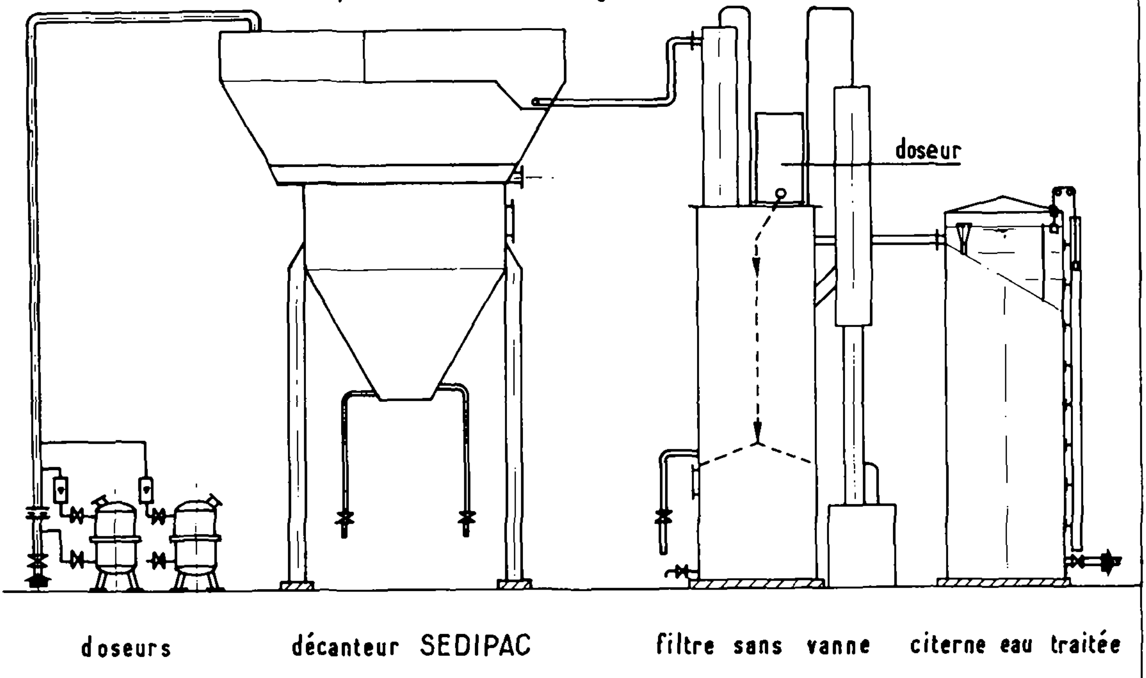
POUR EAU BRUTE MOYENNEMENT CHARGEE

DEBIT EN M 3/H	DECANTEURS			FILTRE SANS VANNE	CITERNES EAU TRAITEE		
	TYPE	NOMBRE	DIMENSIONS UNITAIRES	DIMENSIONS	NOMBRE	DIMENSIONS UNITAIRES	CAPACITE UTILE
5	VERTICAL	1	Ø = 2,30 M H = 3,80 M	Ø = 1,20 M H = 4,70 M	1 MINI	Ø = 1,60 M H = 4,50 M	8,5 M3
10	HORIZONTAL	1	Ø = 2,00 M L = 5 M	Ø = 1,60 M H = 4,70 M	1 MINI	Ø = 2,00 M H = 4,50 M	13,3 M3
15	HORIZONTAL	1	Ø = 2,50 M L = 5 M	Ø = 2,00 M H = 4,70 M	1 MINI	Ø = 2,50 M H = 4,50 M	20,8 M3
20	HORIZONTAL	2	Ø = 2 M L = 5 M	Ø = 2,20 M H = 4,70 M	1 MINI	Ø = 2,50 M H = 4,50 M	20,8 M3
30	HORIZONTAL	2	Ø = 2,50 M L = 5 M	Ø = 2,60 M H = 4,70 M	2 MINI	Ø = 2,50 M H = 4,50 M	41,6 M3

POSTES 10 a 50 m^3/h SANS ELECTRICITE

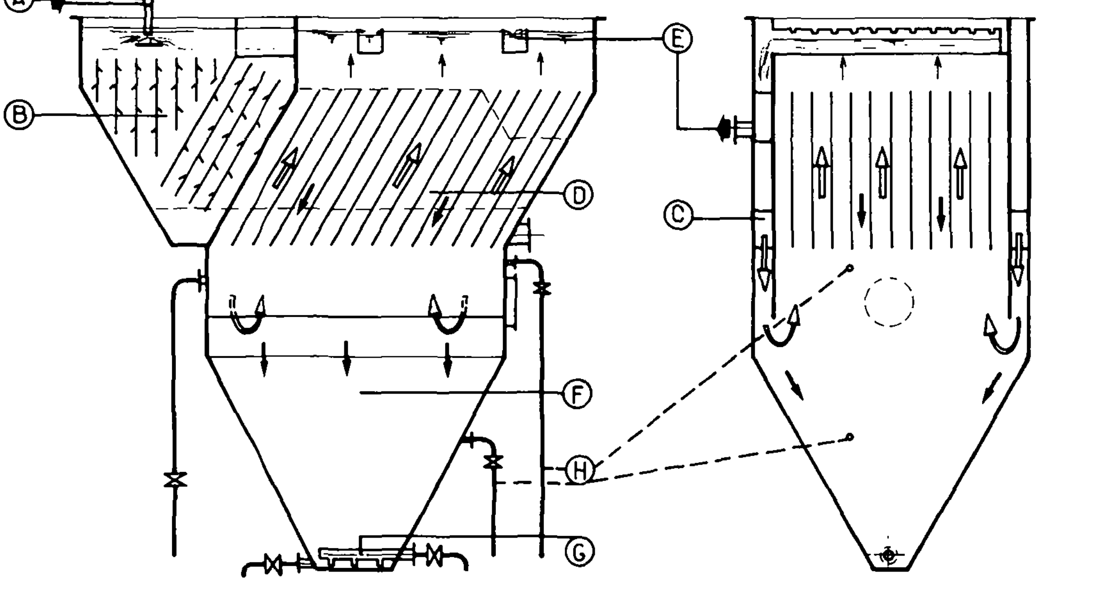
pour eaux brutes chargées

fig. 9



DECANTEUR OUVERT TYPE SEDIPAC

fig. 10



- Ⓐ arrivée d'eau brute
- Ⓑ floculateur
- Ⓒ canaux distributeurs d'eau floculée
- Ⓓ zone de décantation modulaire
- Ⓔ reprise et sortie d'eau décantée
- Ⓕ concentrateur de boues
- Ⓖ extraction des boues
- Ⓗ prises d'échantillons

2) Les décanteurs (fig. 10) :

Ceux-ci sont du type SEDIPAC ouvert, installés en charge sur les filtres.

Ils comprennent :

- un mélangeur en ligne sur la tuyauterie d'alimentation d'eau brute,
- un floculateur à plaques équipées de cornières permettant une floculation en milieu concentré d'un temps de contact de 15 à 20 minutes,
- une partie décantation équipée de modules inclinés d'un diamètre hydraulique suffisant pour en éviter le bouchage, avec :
  . un volume important sous les plaques pour stockage, concentration et extraction des boues,
  . une reprise de l'eau décantée par goulottes déversantes à créneaux.

3) Constitution des différents postes de traitement :

Le tableau (fig. 11), définit les différents postes réalisables en éléments métalliques transportables sur plate-forme de camion.

V - CONCLUSIONS :

Les postes de traitement décrits permettent donc de traiter des eaux soit moyennent chargées, soit plus fortement chargées :

- avec un matériel transportable sur camion, sans convoi spécial, et susceptible d'être installé au sol sans travaux de Génie civil,
- avec des équipements susceptibles de fonctionner sans disposer d'électricité et sans organe mécanique en mouvement.

Ces postes peuvent donc être stockés en magasin et installés très rapidement en tout endroit où l'on dispose d'une alimentation en eau brute. Ils sont donc parfaitement adaptés à la satisfaction immédiate de la demande, et en particulier de celle résultant d'inondations ou de cataclysmes, privant d'électricité ou d'eau potable toute une population.

Ces postes sont faciles à exploiter : ils demandent seulement la préparation de solutions à concentration constante et l'ajustement de la dose de réactifs à la nature de l'eau, ce qui nécessite 1 à 2 visites par jour.

Leur maintenance est limitée aux vidanges et nettoyages d'appareils assez espacés, ainsi qu'à l'entretien de la peinture, aucun organe mécanique en mouvement n'étant susceptible de tomber en panne.

Enfin, si l'on compare le prix de tels postes à celui de postes comportant du Génie civil, ou un réservoir surélevé pour le lavage de filtres sous pression, ou des groupes susceptibles d'entrainer des pompes ou organes mécaniques, ces prix se révèlent plus intéressants, surtout si l'on tient compte de la fiabilité du matériel, du coût de la main-d'oeuvre non spécialisée qu'ils requièrent, et du très faible entretien qu'ils nécessitent.

Le coût d'exploitation se limitant à la consommation de réactifs et le coût d'achat étant limité, les postes décrits conduisent à un coût minimal du m3 d'eau traitée.

POSTES DE 10 À 50 M3/H SANS ELECTRICITE
POUR EAU BRUTE CHARGÉE

DEBIT EN M 3/H	DECANTEURS S E D I P A C			F I L T R E SANS VANNE		C I T E R N E S EAU TRAITÉE		
	NOMBRE	DIMENSIONS UNITAIRES	SURFACE DE DÉCANTATION	NOMBRE	DIMENSIONS UNITAIRES	NOMBRE	DIMENSIONS UNITAIRES	CAPACITÉ UTILE
10	1	2,70 x 2,70 M H = 4,40 M	3 M2	1	Ø = 1,60 M H = 4,70 M	1 MINI	Ø = 2 M H = 4,50 M	13,3 M3
15	1	3,42 x 2,70 M H = 4,55 M	4 M2	1	Ø = 2 M H = 4,70 M	1 MINI	Ø = 2,50 M H = 4,50 M	20,8 M3
20	1	4,47 x 2,70 M H = 4,60 M	5 M2	1	Ø = 2,20 M H = 4,70 M	1 MINI	Ø = 2,50 M H = 4,50 M	20,8 M3
30	2	3,42 x 2,70 M H = 4,55 M	2 x 4 M2	1	Ø = 2,60 M H = 4,70 M	2 MINI	Ø = 2,50 M H = 4,50 M	41,6 M3
40	2	4,47 x 2,70 M H = 4,60 M	2 x 5 M2	2	Ø = 2,20 M H = 4,70 M	2 MINI	Ø = 2,50 M H = 4,50 M	41,6 M3
50	2	5,26 x 2,70 M H = 4,80 M	2 x 6 M2	2	Ø = 2,40 M H = 4,70 M	3 MINI	Ø = 2,50 M H = 4,50 M	62,4 M3

FIG. 11

Comprehensive Water Treatment Station using no Electrical or Mechanical Equipment

SUMMARY

The needs of developing countries for plants for the comprehensive treatment of water intended for human consumption can be defined by an engineering technology to fulfil the following criteria:

1. Easy installation of small or medium-capacity mobile units suitable for villages or small townships.

2. Simple equipment that is easy to operate.

3. Maintenance limited virtually exclusively to the resources available within the country.

4. Purchase price and operating costs as low as possible.

The proposal is for treatment stations which allow for sedimentation with coagulation, filtration, addition of reagents and disinfectant, and the storage of treated water. These stations, which require no civil engineering work, can be carried on standard trucks and are particularly notable for:

- the absence of any need for energy (electricity, fuel oil),

- the absence of moving mechanical parts,

- the use of simple pressurised and gravity-fed dosing equipment,

- the quality and simplicity of the technology used.

As the operating cost is limited to that of the reagents used, and as the capital cost is low, a solution of this kind leads to a very low cost per m3 of water produced.

Paper 8

Design and Fabrication of a Low Cost Package Water Treatment Plant for Rural Areas in India—A Developing Country

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ABSTRACT

The paper deals with the design of a low cost package water treatment plant for treating surface waters. The design is based on unconventional technology. The low cost package plant mainly consists of tapered pebble-bed flocculator, plate settler and filter unit. In tapered pebble-bed flocculator water is initially subjected to very high velocity gradient and subsequently to lower and lower velocity gradients. The plate-settler unit is quite compact and efficient even for 1/10th detention time compared to the conventional clarifiers. The filter unit is same as conventional rapid sand filter.

The advantages claimed by the unit are that it involves no mechanical equipment or electric power for its operation except for a diesel pump. It is simple and easy for operation and maintenance and hence does not require skilled operator. It is quite compact and hence can easily be transported to the required place with the help of a truck. It treats surface waters.

INTRODUCTION

Safe water of acceptable quality is a basic need of any community. But a number of developing countries have not as yet been able to extend this facility to the majority of their population. The problems of developing countries are different in many respects than those of developed countries. Seventy percent of their population stays in rural sector while the corresponding figure for developed countries is only 35 percent (1) as can be seen from Table 1.

TABLE 1. Population percent in Rural Areas

Developed Regions				Developing Regions			
U.S.A.	U.S.S.R.	Europe	Ave	India	Asia (South)	Asia (East)	Ave
26.6	43.7	36.5	34.1	80	76	74.5	75

The per capita consumption in developing countries especially in the rural and semi urban sector is in the range of 50-200 l/d while in the developed countries it is in the range of 300-1200 l/d.

Package plant in a conventional way may be defined as that which is compact, transportable, single unit comprising of all the treatment processes and capable of producing finished water comparable to any of the conventional field installation. A high degree of mechanisation and automation may be involved.

To suit the requirements under rural situations of developing countries, package plant may have to be different from that conceived in a conventional sense.

Transplanting full scale units for turn key operation from developed countries to developing countries could never solve their problem for the simple reason that the number, the level of literacy, the social values, the economic standards involve numerous parameters that are not common between the developed countries and the developing countries. The problems of the developing countries are too huge to be solved by copying and the needs are too large to be met by borrowing. Facilities for repairs, maintenance and communication are next to nil in rural areas of the developing countries. The skilled personnel required for maintenance of such plants is also not easily available. The report (2) published by the National Environmental Engineering Research Institute (India) stated that in India the performance of existing conventional water treatment plants involving a lot of mechanical equipments was not satisfactory. The flow recorders, gauges and dosing equipments were out of order in majority of the plants. Pretreatment units were not giving desired performance. In filtered water, counts of coliform and fecal streptococci were often recorded.

The package water treatment plant for rural areas in the opinion of the author should fulfil the following requirements.

The plant should be

(a) Sturdy,

(b) Simple to operate and easy for inspection and access to any of its parts,

(c) reliable,

(d) with least mechanical equipment and running cost,

(e) able to operate without electrical energy,

(f) fabricated with least capital cost,

(g) easy for transportation and installation with least construction work at site,

(h) able to treat surface water.

In the present paper, attempt has been made to design a package water treatment unit to suit the rural areas of a developing country, taking into account above mentioned requirements.

PROPOSED PACKAGE PLANT

The plant is designed for treating surface waters. The design is based on unconventional technology which has been developed and tested by public health engineering group (3,4,5,6,7,8,9) at the Visvesvaraya Regional College of Engineering (VRCE), Nagpur, India.

The package plant consists of the following

1) Alum dosing unit :- This is a large size plastic bucket for storing alum solution.

2) Pebble-bed-flocculator :- This arrangement is in place of conventional flash mixer and paddle flocculator which require mechanical equipment and electrical energy for their operation. In this unit, water is initially subjected to very high velocity gradient as in flash mixer and subsequently to lesser and lesser velocity gradients. Thus in this unit, water is subjected to tapered velocity gradient which is more effective compared to uniform velocity gradient achieved in conventional paddle flocculators.

3) Plate Settlers :- This arrangement is in place of conventional clarifier requiring mechanical scraper for collection of sludge. Plate settlers are quite compact and efficient and require less than 1/10th detention time compared to a conventional clarifier. Like the pebble bed flocculator, this unit also does not require mechanical equipment or electrical energy for its operation.

4) Filter Unit :- This is same as the conventional rapid sand filter. Slow sand filter though more simple and easy for operation, cannot be considered for surface waters which normally contain high turbidity.

(5) Chlorination Unit :- This unit is similar to alum dosing unit. This contains solution of bleaching powder.

FLOW DIAGRAM OF THE PLANT

The flow diagram is shown in Fig. 1. Raw water from the surface source such as a river is pumped with the help of a diesel pump (P_1) to the elevated tank (E.T.) wherefrom it goes by gravity to the pebble bed flocculator (P.B.F.). A precalculated constant dose of alum solution is continuously added to the raw water before its entry to the flocculator. The flocs formed in the flocculator are settled out in the plate settler unit (P.S.). The clarified water goes to the rapid sand filter (RSF). The predetermined constant volume of chlorine solution from the bleaching powder solution tank (BPST) is added to the filtered water. Finally the water is stored in the pure water reservoir (PWR). The diesel pump (P_2) is for cleaning the filter and the plate settler.

DESIGN OF PACKAGE WATER TREATMENT PLANT

This section deals with the design aspects of various units included in the plant. More emphasis has been given to pebble bed flocculator and the plate settler unit.

<u>Data Assumed for Design</u>

a) Source of water - surface water
b) Population - 1000 persons
c) Water consumption - 90 lphd
d) Average turbidity of water - 100 NTU

It is assumed that the plant operates for 8 hours a day in the initial stage.

Design Details

a) Alum dosing unit :- This is nothing but a large size plastic bucket of around 12 litre capacity with an opening at the bottom and a lid. A brass tube which slides up and down with the help of a guide has a orifice at its lower end. The wooden float maintains a constant head of 8 to 10 cm which is independent of the solution level in the bucket, thus ensuring a constant discharge. One of the two valves at the outlet, which is closer to the bucket is the control valve to adjust the required flow of solution. The adjacent outer valve is the shutoff valve for temporarily stopping the flow and restarting the same at the required time. The details of the alum dosing unit are shown in Fig. 2.

b) Design of pebble bed flocculator :- It has been shown by earlier workers (10,11,12,13) that tapered velocity gradient is more efficient than uniform velocity gradient. But a number of tanks are required, one for each velocity gradient. Besides, mixing time and paddle speed in each tank will have to be progressively changed. This arrangement conflicts with the modern concept of compactness of the plant. So it was decided to use a compact unit in the form of a pebble bed flocculator to achieve tapered velocity gradient which depends on factors such as size of pebble, rate of flow, head loss across the bed, etc.

The pebble bed flocculator has been tested and used successfully at many places (14,15,16,17) in India. The preliminary experiments required for the design purpose were conducted in the college laboratory.

The flocculator unit was fabricated using 6 cm diameter, 50 cm long perspex pipe. Necessary arrangements for measurement of head loss were made by providing nozzles at a spacing of 15 cm.

Pebbles of sizes 0.5 cm to 1 cm were filled in the flocculator unit to a depth of 30 cm. The head loss across 15 cm depth of pebble bed for various rates of flows were recorded at steady state. The temperature was also recorded for every observation.

These pebbles were then replaced by pebbles of size 1 to 2 cm and again the head losses were measured. Table 2 shows the effect of rate of flow on head loss and velocity gradient (G).

The velocity gradients for different rates of flow were calculated with the help of the following equation

$$G = \sqrt{\frac{h \cdot g \times Q}{\nu \cdot f \cdot Vol}} \qquad \cdots \qquad \cdots \qquad \cdots \qquad (1)$$

where

h = head loss in cm.
g = gravitational acceleration cm/sec/sec.
ν = kinematic viscosity
f = porosity (0.4)
Vol = Volume of bed (cm^3)
Q = rate of flow (cm^3/sec)

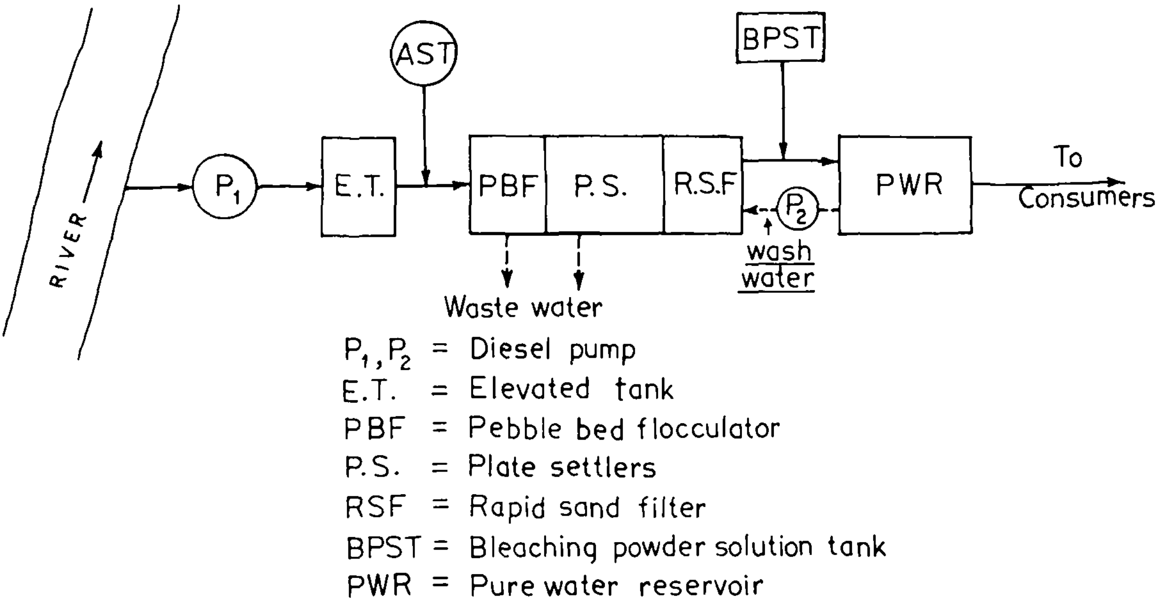

Fig. 1. Flow Diagram

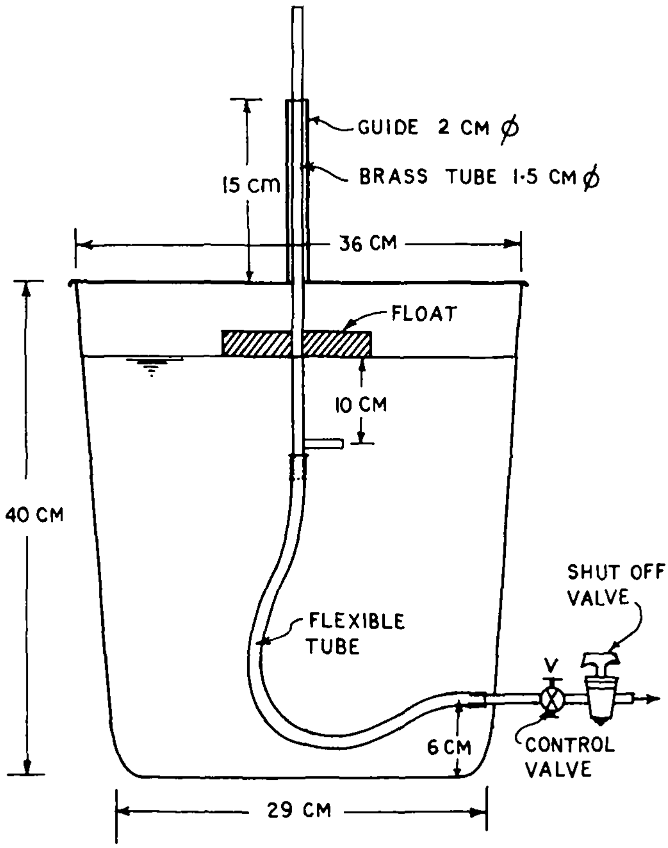

Fig. 2. Alum Dosing Unit

TABLE 2. Effect of rate of flow on head loss and G.

Sr. No.	Rate of flow (cm^3/sec)	Velocity of flow (cm/sec)	Head loss (cm)	G (Sec^{-1})	Remarks
1	22	0.78	0.4	75	Pebble size 0.5 to 1.0 cm.
2	30	1.06	1.0	136	
3	45	1.60	2.0	240	
4	58	2.05	5.0	437	
5	100	3.54	6.8	638	
6	120	4.24	8.9	822	
7	170	6.01	14.3	1242	
8	17	0.60	0.3	98	Pebble size 1 to 2 cm
9	33	1.17	1.1	152	
10	62.5	2.21	3.0	344	

Figures 3,4,5,6 were drawn connecting head loss and discharge and head loss and velocity gradient. Fig. 3 and 4 being for 0.5 to 1.0 cm size of pebble and Fig. 5 and 6 for 1 to 2 cm size.

It was decided to obtain following velocity gradients in the flocculator. The details are as indicated in Table 3.

TABLE 3. Details of Velocity Gradients for Proposed Flocculator

Velocity Gradient (Sec^{-1})	Size of pebble (dia.) (cm)	Depth of pebble (cm)
1200	0.5 to 1.0	20
400	0.5 to 1.0	20
200	0.5 to 1.0	20
	1 to 2	20
50	1 to 2	20

The above velocity gradients cover a wide range. The total depth of media is 100 cm., the bottom three layers (each layer 20 cm deep) of pebbles consist of size 5 to 1C mm and the fourth and top layer of pebble size 10 mm to 20 mm.

Calculations for the Design of Tapered Flocculator

The velocity gradient is a function of head loss which in turn is a function of rate of discharge and hence velocity of flow for a constant cross sectional area of a flocculator.

With the help of the Figs. 3 to 6, the head losses and the corresponding discharges and the velocities of flow were found out for the above mentioned velocity gradients.

In order to achieve different velocity gradients, necessary cross sectional areas are to be found out after arriving at the velocity of flow at each section for a predetermined rate of flow. The total water requirement/day = 90 cubic metres. Since the plant operates for 8 hours a day

$$\text{Rate of flow} = 11.25\ m^3/hr. = 3125\ c.c./Sec.$$

Sample Calculation to find the flocculator dimensions for the velocity gradient of 1200 Sec^{-1} :- From Fig. 3 and 4, it can be seen that for 1200 Sec^{-1} value of G, the corresponding discharge is 167 $cm^3/sec.$, which results in velocity of flow (V) of 5.91 cm/Sec. for 6 cm. dia. laboratory model.

$\therefore$ For 1200 Sec^{-1}, velocity of flow V = 5.91 cm/Sec. If A is the area in the package water treatment plant for 1200 Sec^{-1}, then

$$A\ V = 3125$$

$$\therefore\ A = \frac{3125}{5.91} = 529\ cm^2.$$

If 1 m is the length of the flocculator, then width of the flocculator = 5.3 cm. The depth of the flocculator has been decided as 20 cm. for this velocity gradient.

Dimensions of flocculator for other velocity gradients :- Similar approach was adopted to calculate the dimensions of the flocculator for the rest of the velocity gradients viz., 400, 200, 100 and 50 Sec^{-1}.

Table No. 4 shows the worked out dimensions of the flocculator.

The designed flocculator is shown in Fig. 7. I is the inlet at the bottom of the flocculator for the alum mixed raw water. The water is then subjected to highest velocity gradient of 1200 Sec^{-1}. This mixing is quite intense and hence is equivalent to flash mixing. The water is then subjected to sequentially lower velocity gradients.

The predetermined quantity of alum is added in the inlet raw water pipe. The plastic bucket containing alum solution is kept above raw water level in the raw water tank to avoid the reverse flow.

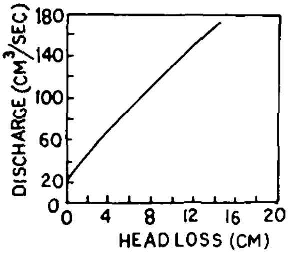

Fig. 3. Headloss Vs Discharge for 50 mm to 100 mm media

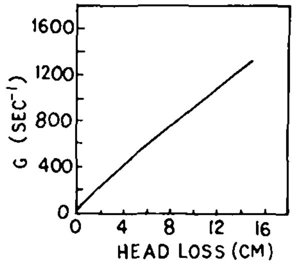

Fig. 4. Headloss Vs G for 50 mm to 100 mm media

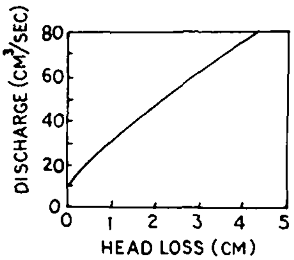

Fig. 5. Headloss Vis Discharge for 100 to 200 mm media

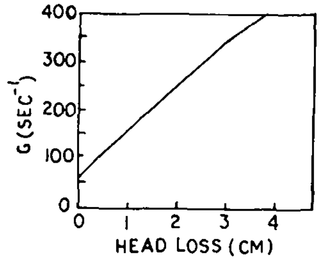

Fig. 6. Headloss Vs G for 100 to 200 mm media

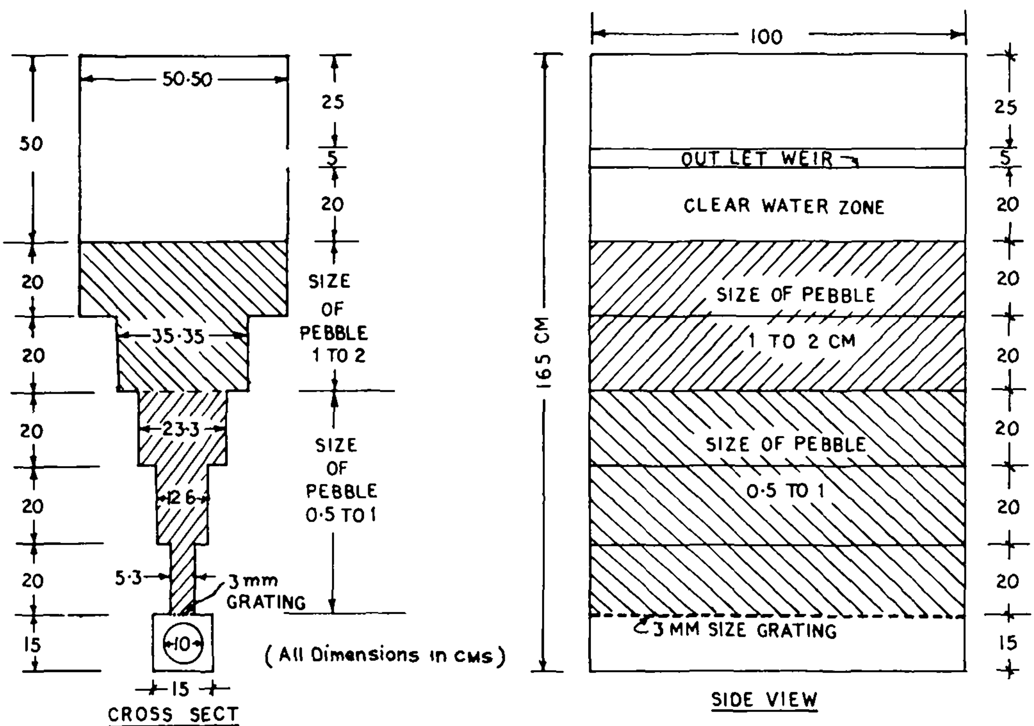

Fig. 7. Pebble Bed Flocculator

TABLE 4. Dimensions of the Flocculator

Velocity gradient	Velocity of flow V	Length	Width	Height
Sec^{-1}	cm/Sec	cm	cm	cm
1200	5.91	100	5.3	20
400	2.48	100	12.60	20
200	1.34	100	23.30	20
100	0.88	100	35.35	20
50	0.62	100	50.50	20

(c) Design of Plate Settlers

The plate settlers have numerous advantages compared to conventional sedimentation tank. A few important advantages are listed below

1) Plate settler unit more compact,
2) Ten to thirty minutes sedimentation time is sufficient compared to 3 hour time for conventional plant,
3) No mechanical equipment or electrical energy is required for its operation,
4) It can take higher overflow loadings,
5) No skilled operator is required to handle the plant.

The present design is based on Yao's (18) approach. Following assumptions are made for the design.

1) Overflow rate $= V_{SC}$

$= 30$ cum/m^2/day

2) Constant for dimensional adjustment of the overflow rate

$= C$

$= 8.64 \times 10^2$

3) Critical value of the parameter indicating performance of a high rate settling system

$= S_C$

$= 1$ for parallel plates

4) Average velocity of flow through a settler

$= V_o$

$= 0.25$ cm/sec

5) Depth of a settler $= d$

$= 5$ cm

6) Kinematic viscosity of the fluid $= \nu$ at $30^{\circ}C$

$= 0.804 \times 10^{-2}\ cm^2/Sec.$

7) Angle of inclination of the settler to horizontal

$= \theta$

$= 40^{\circ}$

If l is the length of the settler and L is the relative settler length, then

$$L = l/d \quad \ldots \quad (2)$$

Also the overflow rate is given by

$$V_{S_C} = C.K.\frac{V_o}{L} \quad \ldots \quad (3)$$

$$\text{where}\quad K = S_C \left(\frac{L}{\sin\theta + L\cos\theta} \right) \quad \ldots \quad (4)$$

Substituting the value of K in Eqn. 3 and rearranging the terms, we get

$$L = \frac{1}{\cos\theta}\left[\frac{C}{V_{SC}} \times \frac{S_C}{1} \times \frac{V_o}{1} - \sin\theta\right] \quad \ldots \quad (5)$$

Substituting the assumed values

$$L = \frac{1}{0.766}\left[\frac{(8.64 \times 10^2)}{30} \times \frac{1}{4} - 0.643\right]$$

$$= 8.56$$

The relative length for the transition region

$$L' = 0.058\,\frac{V_o \times d}{\nu} \quad \ldots \quad (6)$$

After substituting the given values, we get

$$L' = \frac{0.058 \times (0.25) \times 5}{(0.804 \times 10^{-2})} = 9.02$$

In this design L' works out to be greater than L and hence a total relative length of 2 L is used instead of $(L + L')$

$\therefore$ The total relative length

$$2L = \frac{l}{d} \quad \ldots \quad (7)$$

$\therefore\ l = (2 \times 8.56) \times 5$

$= 85.6$ cm

$\simeq 86$ cm

$\therefore$ Detention time $= \dfrac{86}{(0.25 \times 60)}$

$= 5.74$ minutes

It was decided to provide 100 cm length of the inclined plate settler in place of 86 cm. This gives a detention time of

$$\frac{100}{0.25 \times 60} = 6.67 \text{ minutes}$$

$$\simeq 7 \text{ minutes}$$

Volume of plate settler for 7 minutes detention time

$$= \frac{11.25 \times 7}{60}$$

$$= 1.33 \text{ m}^3$$

Assuming the width and the length of the settler unit to be 1.25 m and 2 m respectively the volume of the settler

$$= 2 \times 1.25 \times 0.643$$

$$= 1.61 \text{ m}^3$$

$$> 1.33 \text{ m}^3 \text{ and hence safe.}$$

Now 5 cm clear distance gives horizontal projection of 7.8 cm. If 2 mm is the thickness of the plates then number of plates provided

$$= \frac{200}{8} + 1 = 26$$

out of which the first plate acts as partition wall and prevents direct entry of water from the flocculator unit to the sedimentation unit. The plates could be of a suitable noncorrosive material such as PVC etc.

The weir to collect settled water is in the form of a centrally located channel of 40 cm width and 20 cm depth along the length of the tank. It has 5 cm deep, 90° V notches spaced 15 cm c/c. The bottom of the channel is kept 15 cm above the top of the settler.

The total length of the weir is 4 m.

The sludge accumulated on the plates, could be cleaned out with the help of filter back wash water.

Check for weir loading :-

$$\text{Weir loading} = \frac{\text{m}^3/\text{day (Total Flow/day)}}{\text{m (length of weir)}}$$

$$= \frac{90}{4}$$

$$= 22.5 \text{ m}^3/\text{m/day}$$

Hence safe.

The plate settler unit is shown in Fig. 8.

(d) Design of Filter

The design criteria for a rapid sand filter is used for the proposed filter which includes the design of the following

1) Size of filter unit
2) Filter media
3) Supporting media
4) Underdrain system
5) Inlet, outlet and back wash system.

They are discussed in brief below :

1) Size of filter unit :-

Rate of rapid sand filtration = 4.8 $m^3/m^2/hr$.

$\therefore$ Area of filter $= \frac{11.5}{4.8}$

$= 2.4\ m^2$

$\therefore$ Provide a filter of size = 2 m (length) x 1.25 m (width)

1.25 m width being common for the plant.

2) Filter Media :- Sand is used as filter media. Sand specifications are as follows :

a) Effective size = 0.40 mm.
b) Uniformity coefficient = 1.4

Recommended depth of filter media is normally 75 cm. but the general experience is that the first few centimeters of the sand bed acts as filter media and the rest act as supporting media. Hence in the present design, depth of the sand bed adopted is 60 cm.

3) Supporting Gravel Media :- The depth of gravel media adopted is 30 cm. The different sizes of gravel used are as shown in Table 5.

TABLE 5. Size and depth of Gravel Layers

Position of layer	Size range (mm)	Depth of layer (cm)
Bottom layer	38-20	10
Middle layer	20-12	10
Top layer	12-05	10

4) Underdrain System :- The underdrain system consists of manifold and laterals, their diameters being 150 mm and 100 mm respectively. The lateral spacing is 400 mm. The twin orifices of the laterals are 6 mm in diameter at a spacing of 50 mm. The orifices face downwards making an angle of 45° to vertical as shown in Fig. 9. The underdrain system is protected with the gravel media.

5) Inlet, Outlet and Backwash System :-

(i) Inlet :- The troughs provided at the top of the filter unit serve the purpose of inlet as well as wash water gutters.

The water from the sedimentation tank comes to the main trough in the filter and gets uniformly distributed along the three longitudinal troughs. The main trough has a width of 30 cm while the longitudinals have 20 cm. The depth of 20 cm is provided for either types of troughs. The rest of the details are same as that described for plate settler unit.

(ii) Outlet :- The outlet pipe is kept above the level of sand surface to ensure that the sand bed is always immersed in water to avoid air logging and negative head etc.

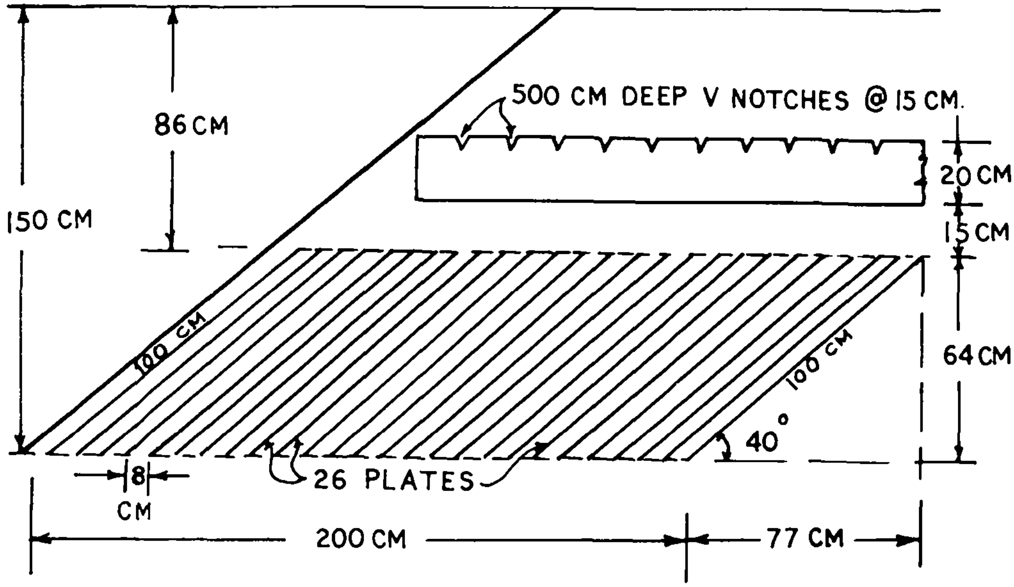

Fig. 8. Cross-section of plate settlers

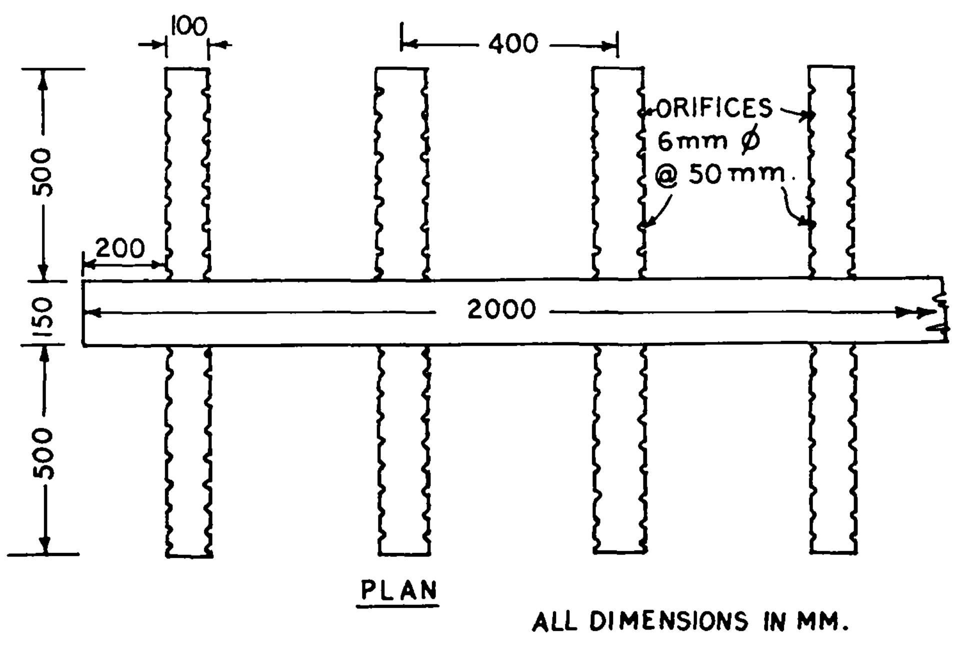

Fig. 9 Filter Underdrain System

(iii) Backwash System :- The cleaning arrangement for the filter is the usual back wash system for rapid sand filter. The wash water enters from the bottom and then collects in the troughs and finally taken on the plate settlers. Thus the plate settlers are also cleaned with the help of water for filter wash. The combined wash water is then taken out of the plant through the outlet located at the bottom most portion of the plant.

CHLORINATION UNIT

The chlorination is done with the help of bleaching powder solution which is stored in a plastic bucket. The chlorine solution is injected to the filtered water under constant head so that same constant volume of solution is injected during any time of the process. Rest of the details are same as alum solution dosing unit.

THE LAYOUT OF THE PLANT

The detailed plan and elevation of the plant and the relative positions of various proposed units are shown in Fig. 10.

The river water is pumped to the elevated tank E.T. with the help of a diesel pump P_1. The excess water in E.T. is drained out with the help of an overflow pipe.

B_1 is the alum solution bucket located above the level of E.T. to avoid reverse flow of water towards bucket. Alum solution is injected before water enters the flocculator unit.

The flocculator is made of M.S. (mild steel) sheet coated with a suitable corrosion proof paint. It is kept in position in the outer main container, with the support of horizontal rods which are fitted to the main container. The alum solution mixed raw water enters the flocculator through an inlet in the form of a perforated pipe provided at the bottom of the flocculator unit. The water has an upward direction in the flocculator. The flocculated water enters the main container through the outlet weir provided to the flocculator.

The water enters the plate settler unit (PS) from the bottom and travels upward in the PS. The clarified water collects in the trough provided at the top and finally taken to the filter.

The flow is downwards in the filter. The filtered water is then chlorinated and finally collected in the clear water tank. The chlorine solution is stored in the plastic airtight bucket B_2.

During normal operation of the plant, valves V_1, V_2 and V_5 are opened and valves V_3, V_4, V_6 and V_7 are closed.

At the time of back washing, valves V_4 and V_6 are opened and valves V_1, V_2, V_3, V_5 and V_7 are closed and pump P_2 is started.

For draining out the flocculator unit, valves V_1 and V_4 are closed and V_2 and V_3 are opened.

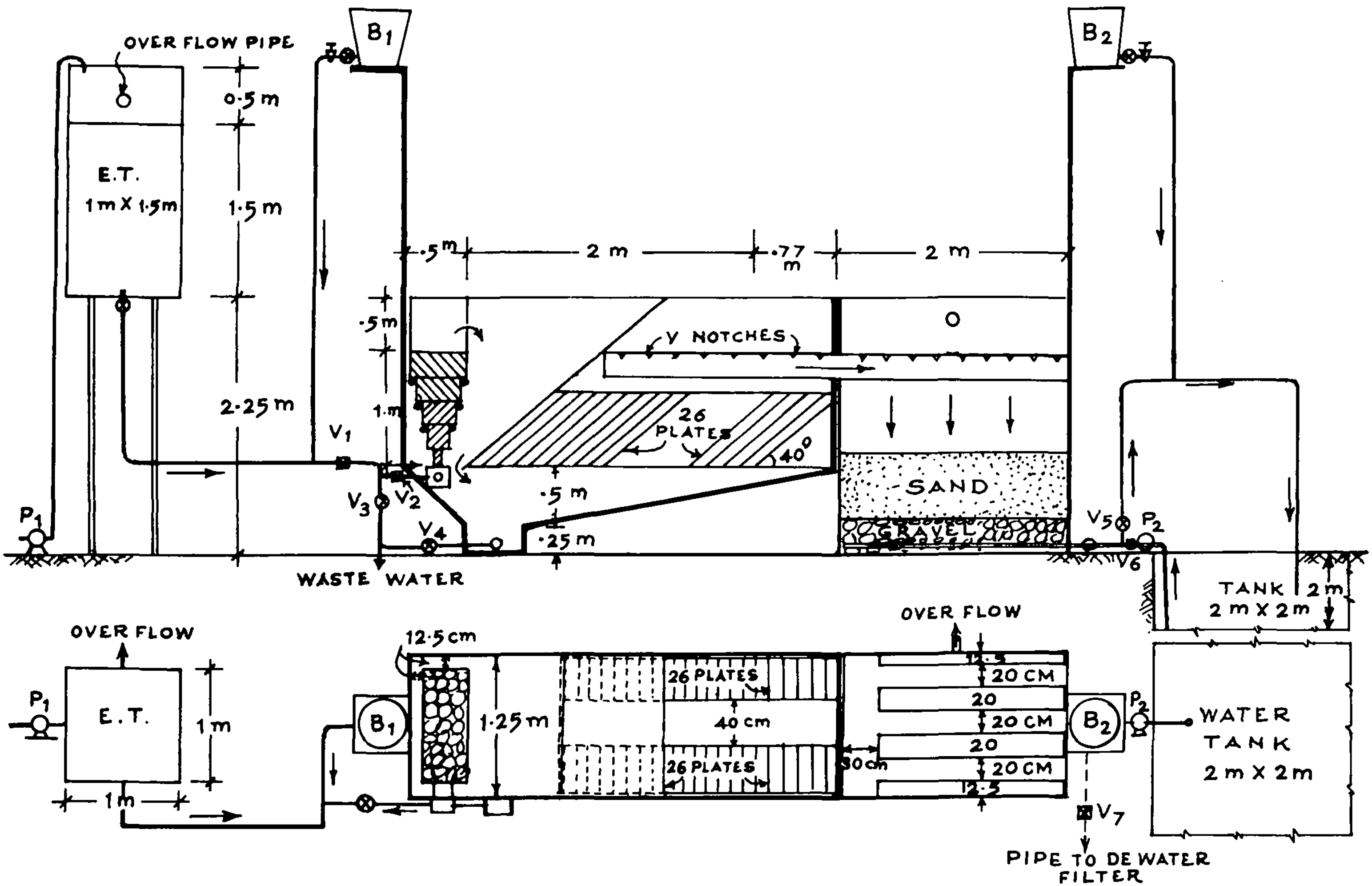

Fig. 10. Layout of the Plant

To dewater the filter unit, valves V_5 and V_6 are closed and V_7 is opened.

COST OF PACKAGE PLANT

The cost of the package plant works out to be approximately Rs. 20,000 i.e., £ 1150. If the plant could be manufactured in large number at one place and transported to the place of site, then cost could be still reduced down.

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Conception et Fabrication d'une Station de Traitement de l'Eau Compacte et a Faible Coût pour les Régions Rurales de l'Inde.

RESUME

L'Inde est avant tout un pays de villages où vit environ 80% de la population. Moins de 10% de la population rurale dispose d'eau potable. En Inde, dans presque tous les villages, la source d'eau est une eau de surface. Très peu de villages bénéficient intégralement d'eau souterraine (eau de puits).

Le présent rapport traite d'une installation de faible coût pour le traitement des eaux superficielles. Sa conception est basée sur une technologie non conventionnelle qui a été développée et testée par un groupe d'ingénieurs du Visvesvaraya Regional College of Engineering (VRCE) à Nagpur en Inde.

L'installation proposée pour le traitement de l'eau comprend un floculateur avec couche de galets, un décanteur, un filtre à sable rapide et des doseurs des solutions de chlore et d'alun. Le floculateur avec couche de galets remplace le mélangeur et le floculateur à aubes conventionnels qui requièrent du matériel mecanique et de l'énergie électrique pour pouvoir fonctionner. Le floculateur avec couche de galets est formé de galets de 0,5 a 2 cm. Lorsque de l'eau non traitée passe par ce lit avec une solution d'alun, elle est soumise initialement à un gradient de grande vitesse, puis à des gradients de vitesse de moins en moins grandes. Le décanteur remplace le clarificateur con entionnel comprenant un racleur mécanique pour ramasser les boues. Le décanteur est très compact et efficace même avec un temps de détention de 1/10eme comparé au clarificateur conventionnel. Le décanteur n'a pas besoin d'ouvriers qualifiés y compris pour son entretien, car il n'y a ni matériel mécanique ni energie électrique. Le filtre à sable rapide est identique à un filtre à sable conventionnel. Le nettoyage du filtre se fait avec le lavage par retour habituel. L'eau de lavage passe aussi sur les décanteurs pour nettoyer les plaques. Les installations de chloration et de dosage de l'alun sont en plastique. La solution est ajoutée sous pression constante pour pouvoir injecter un volume permanent de solution.

Parmi les principaux avantages de cette installation à faible coût pour le traitement de l'eau, citons le prix de revient et l'aspect compact de l'installation. Elle est facilement transportable sur camion du lieu de fabrication au site. Le montage sur place est simple, ne nécessitant aucun materiel mécanique ou énergie électrique, hormis une pompe diesel. Elle est de fonctionnement rustique et facile. L'opérateur peut acceder aisement à toutes les parties de l'installation dans le cas où il y aurait une panne quelconque.

Paper 9

The Compact Water Works (CWW)

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1. Introduction

Today I would like to show you a water treatment system which has been developed in cooperation with the consulting engineering firm of Dr.Kuros/ Teheran to meet the demand for clean drinking water in Persia and similar countries. As I shall explain later, the compact water works comes under the heading of "low cost technologies" in drinking water preparation, on account of its low initial and operating costs. This does not mean, however, that it can be classed as a mobile facility, such as those to be found on containers as a temporary water supply.

The CWW is a permanent water treatment plant of the highest quality, for supply drinking water to local communities, being made of materials specially chosen for a long and useful service life.

2. Uses

The CWW represents the latest development in technology in the field of small size treatment systems, and is suitable for use at all places where it is not possible to connect a local community to a central water supply. Communities of this category are as a rule remote areas comprizing up to 1000 inhabitants, often located in hard to reach geographical terrains.

Raw water can be treated coming from rivers or brooks, from irrigation systems or even groundwater. This compact water works is designed specially not to need an electrical supply, the gravitational flow of the water being used to motivate all feeding and conditioning stages.

To keep the cost of operating the system to a minimum, the various stages in the process were united to form a system which uses natural cleaning processes and requires the addition of specific chemicals only after substantial operational intervals.

3. The treatment process

The raw water coming in for treatment is substantially surface water, therefore the various stages were designed so that the organic and inorganic pollutions contained in the water are almost completely eliminated in a pre-treatment stage in which flocculation and sedimentation processes are resorted to.

The most important part of the conditioning process is the slow sand filter, which after a breaking-in period forms in the top strata a biologically active zone. This biological slime, the so-called biological mat, is composed of an accumulation of micro-organisms which eliminate the pathogenic organisms, i.e. germs, which can be contained in the raw water, by the process of decomposition.

Besides eliminating of the germs, the slow sand filter also effects a fine filtration of the water. Suspended solids and fine particles coming from the sedimentation stage get completely removed by this filter.

Because of the profound effect of the slow sand filter, a biological digestion of dissolved organic substances such as ammonia and nitrates is effected.

The water which leaves the slow sand filter is treated with small quantities of chlorine compound for its auto-sterility, and enters a clear water tank, from which the user can draw off his supplies by operating a piston or diaphragm pump.

4. Mechanical system

Normally the raw water will enter the conditioning system by a natural fall. If however the local conditions preclude this, the water can be pumped by an hydraulic ram. These simple pump systems work with a ratio of flow rate to pumped water of 5 : 1 to 8 : 1, for an inlet gradient of the pumped water of 1-1/2 m. The pumping heads thus achieved lie between 10 and 50 m column of water, depending on the ratio of flow rate to useful flow. These hydraulic rams require almost no maintenance and have been used successfully in the developing countries for decades now.

The raw water flows into the conditioning system through a manually-operated check valve to an inlet regulator. The level of the clear water tank actuates the conical control valve by means of a float mechanism, which ensures that the flow of water which is to be treated passes through the sundry stages at a constant rate. This manner of control is maintained by means of a very simple spring-loaded system, requiring absolutely no attention.

The raw water enters a feeder tank, the purpose of which is to maintain a constant level for supplying the first chemical feeder. The main flow passes down the adjustable gravity line, in which the flow of raw water gets mixed with oxygen through means of thin-film aeration and in which the secondary flow goes with the chemicals added. The chemical required for eliminating the turbidity substances, usually a salt of a heavy metal such as aluminium or iron sulphate, is delivered from a feeder vessel by means of a volumetric dosing device, into a depressurized mixing chamber. This chamber supplies a dosifier with the requisite quantity of chemicals, this being done fully-automatically and by the simplest means possible. The dosifier itself gets driven by a small pilot wheel and is connected either to a dosing spiral or a dosing centrifuge. Quantities of up to 1/10 ml/sec. can be metered in this way. For larger quantities, a so-called tilting cup device takes over.

The water which is now mixed with the chemicals passes into an annular reaction/sedimentation tank surrounding the conditioning system. This gives the chemicals a chance to destabilize the colloids, causing adsorptive cohesion of the dirt particles and deposition on the bottom of the settling basin. The resulting hydroxide flocs infiltrate an appropriately large sedimentation section. The sedimentation basin requires to be cleaned only after every 100 days or so, this being effected by opening the foot valve, whereby the sludge liquor carries the dirt into a collector which is located outside of the CWW. Any residual amounts can be forced through by hand. Closing of the foot valve again puts the CWW back into commission.

The pre-cleaned water now flows through a multi-stage cascade into the flood chamber of the slow sand filter. The purpose of this cascade ist a further aeration of the water, while at the same time preventing the water from scouring the material of the slow sand filter.

The slow sand filter comprizes a drainage system embedded in a coarse gravel stratum. The filter material itself is composed of fine-grained coarse gravel. The stratum thickness is 1 metre. The flood chamber is approximately 60 cm high, and is defined as the clearance from the end of the filter bed to the annular dividing wall to the reaction/sedimentation zone.

The level of water above the filter material will only be a few centimetres for freshly-cleaned water, and rises with increasing run and sludge accumulation to the full height of the flood chamber, this being achieved by passing the conditioned water in the outlet part of the slow sand filter through a standpipe in the same plane as the crest of the filter sand. Thus the filter material is always under water and remains biologically active. After a breaking-in period, a so-called biological mat is formed on the top of the filter, which, as already explained, is an accumulation of micro-organisms. As the run continues the thickness of this biological mat will also increase, and at the end of the run, which can last several months, this biological slime of approx. 5-8 cm thickness gets removed by hand, cleaned, and returned. This procedure is effected in a few hours only.

The adequately-dimensioned clear water tank enables the CWW to remain in commission as a supplier of drinking water during the time the sludge collector is cleaned and the biological mat is removed, and also during cleaning and returning of the filter sand.

Another version of the CWW has the sand of the slow filter resting on a nozzle bottom equipped with some 64 nozzles/m^2, this preventing the filter material from getting into the clear water tank. This nozzle bottom has also the job of ensuring that the CWW back-washes relatively rapidly with the help of a wash water pump assisted where appropriate by a scavenging blower, in other words, a mechanical cleaning is possible. I will come back to this washing method later on.

Using a nozzle bottom has the added advantage that the CWW can be used as a rapid filter at any time, by resorting to a coarser-grained sand material, whereby filtering rates and hence the throughput can be raised by some 10 times that of normal slow sand filter operation. If mechanical washing and cleaning of the CWW is desired, the annular partition between the filter assembly and the sedimentation chamber serves as an overflow for the incoming sludge liquor.

Opening of the foot valve in the sedimentation tank causes the incoming sludge liquor to flow with the settled dirt particles into the external sludge thickener, thus completely dispensing with the need for manual cleaning of the sludge liquor chamber.

The standpipe on the clear water side performs a function similar to that of the first level vessel on the raw water side. Here also is used some of the water flow to form a pilot to a second dosifier, whereupon it gets returned mixed with chemicals to the main water flow. As already mentioned, these chemicals serve for keeping the clear water collector tank free of germs.

The second dosifier is suitably designed as a tilting cup mechanism. As a kind of pumping station, this tilting cup meters at intervals the fluid chemical additive, into the pilot flow and thence into the main water flow, all this being effected solely by the free fall of the water. As with the dosifier no. 1, no maintenance is required at all, except for the periodic replenishment of the chemical.

The conditioned water passes into the clear water tank and in here flows through a baffled section, to prevent a dead storage. Water can be drawn from the clear water tank by the user by operating a diaphragm pump.

The CWW process has been subjected to extensive technical tests and trials, over a period of over a year, and its usefulness has been proven. The findings obtained from these tests have been written down in a report, from which I want to quote one or two passages in the following.

Sampling point		1			2			3		
Turbidity 1)	NTU	5	16	43	0,5	0,7	5,5	0,06	0,07	0,09
Suspended matter 2)	mg/l	20	40	100	4	8,5	15	-	-	-
Aluminium 3)	mg/l	5	10	25	1	3	6	0,01	0,05	0,07
Duration	d	-	-	-	-	-	-	120	90	38
Zeta potential	mV	- 18			- 9			-		
E.-coli 4)	$100\ ml^{-1}$	500-3000			300-1000			E 99% 5)		
O_2 saturation in %		64			85			105		

1 = raw water

2 = pre-treated water

3 = filter effluent after final aeration

1) = Hach turbidimeter 2100 A
2) = German Standard Procedure HZ
3) Spectrophotometer Perkin Elmer 550
4) = German Standard Procedure K 5
5) = E = rate of elimination

Performance of the pilot plant:	0,5 m^3/hour
Filtration speed:	0,12 m/hour
Layer height of the slow sand filter:	1 m
Granulation of sand:	0,3 - 0,7 mm ø (diametre)
Period of biological ripening:	6-9 weeks according to characteristics of raw water

The tendency of the test results achieved coincide with those stated by the National Research Centre for Environmental Technology of Nakpur (India) (NEERI) and by the International WHO Reference Centre for Municipal Water Supplies of The Hague. The CWW thus complies with the parameters of the World Health Organization instructions.

5. Notes concerning the CWW design

There are 2 modular CWW types which, however, may be expanded to meet the kind of application concerned.

Type 1 for a capacity of 1 to 3 m^3/hour. Provided the daily per capita consumption is 80 litres, it corresponds to a water work sized for a community of 300 to 1000 inhabitants.

Type 2 for a capacity of 3 to 6 m^3/hour. Provided the daily per capita consumption is 80 litres, this water work will supply drinking water for a community of 1000 to 3000 inhabitants.

Any type comprizes two entirely self-sufficient treatment plants, each of them being dimensioned to one half of the overall capacity. This embodies the substantial advantage that 50 % of the treatment capacity remain available in the period when one section is being cleaned.
The clear water reservoir, therefore,may be dimensioned to a correspondingly smaller size.

The design concept of the two CWW models provides one cylinder-shaped body each, which - by means of 2 bulkheads each of concentrically decreasing diameters - is subdivided into individual cylindrical ring segments. Thus, the necessary conditioning chambers are created, the water flowing by free fall through one after the other from outside to inside.
The outer cylindrical ring segment is used as a reaction and sedimentation chamber for water pre-treatment, the central ring chamber houses the slow sand filter with its submerging chamber, and the remaining inner cylinder compartment is used as a control chamber for the valves and fittings.

Under the proper treatment section, the clear water collector reservoir is placed which is equipped with guide walls to ensure improved flow.

On principle, the two CWW types may be manufactured of reinforced concrete or of corrosion-resistant steel. From the size point of view, the type 1 has been dimensioned to allow the transport of each of the prefabricated treatment halves by means of a low-bed trailer to the construction site.

The type 2 also has been systemized to prefabrication elements allowing transport to the place of installation; because of its outside dimensions it, however, requires welding on site.

Furthermore, one may imagine to combine reinforced concrete and steel, in which case it will be preferable to make the treatment section of steel sheet, the clear water collector tank being built on site of waterproof reinforced concrete.

6. Planning, Shipment, Installation, Putting into Operation, Maintenance

Planning

At first, the authority competent for the drinking water supply of small communities will have to decide upon the type of the CWW to be installed. It will be advisable that a local engineering office is entrusted to survey the construction activities.

After that, it has to be decided upon the material to be used for the CWW. It is recommended that the clear water tank preferably should be made of reinforced concrete, and the treatment section of easy-to-handle steel sheet. The design specifications required in this connection will be supplied by the licencer against payment of a royalty.

The following operations will have to be performed:

1. Construction of a small intake weir in the flowing waters.

2. Installation of a raw water feed pipe from the intake weir to the treatment section.

3. Earthmoving for the waterworks and a supernatant collector and infiltration basin.

4. Performance of construction work for the clear water tank.

5. Supply and installation of the complete treatment unit.

6. Roofing of the treatment plant, if necessary, by structure of masonry, steel or wood.

Shipment / Supplies

The CWW comprizes the following supply components:

1. Steel sheet structure for the treatment section and, if applicable, for the clear water chamber.

2. Interconnecting pipes and fittings.

3. Corrosion-resistant plastic components, plus filter bottom and dosage equipment and/or chemicals storage tank, as far as applicable.

4. Filter sand.

5. Chemicals.

Re item 1: The steel sheet structure usually is made and supplied by a qualified local contractor who may receive the order either indirectly from the licencer or directly from the competent authority.

Re item 2: The necessary pipes and fittings usually are procured from local sources, too, to be installed by said qualified contractor.

Re item 3: The corrosion-resistant special plastic components will be supplied by the licencer under a contract concluded with the competent authority, to be shipped to the site and to be mounted (in accordance to instructions given) by the local contractor.

Re item 4: The filter sand required usually will be available from a pit close to the construction site, to be sieved by the plant operator to achieve the adequate grain size and to be filled into the plant.

Re item 5: The chemicals required for plant operation may be supplied by the licencer or be made available directly by the purchasing authority.

Installation

The CWW will be erected by the selected qualified contractor in accordance to the licencer's instructions. The CWW type 1 may be shipped in completely pre-assembled condition (2 sections), and just has to be mounted and bolted to the clear water section. For the CWW type 2, pre-fabricated components will be supplied which have to be welded on the construction site.

Mounting of the special components supplied by the licencer, as instructed by him, will not involve any problem. After that the corrosion protection will be made.

Putting into operation

The CWW usually will be put into operation by qualified personnel trained by the licencer. This includes practical instructions to the owner's operating personnel, too.

Maintenance

The current maintenance of the compact water work CWW usually will be done by the owner. To this effect, he will get a checklist included in the licencer's documents giving detailed instructions for the maintenance of the CWW.

Most substantial work to be performed is:

1. Re-filling of the chemicals from the storage tank into the dosage vessels. (approx. every 8 to 14 days).

2. Cleaning of the sludge sedimentation chamber (approx. every 6 months).

3. Removing of a sand layer (height approx. 7 cm) from the slow sand filter cleaning and re-putting it into the filter (approx. every 2-4 months).

Optional equipment for the CWW is a nozzle assembly in the slow sand filter area. This attachment allows cleaning of the CWW within a few minutes' time by a specialized CWW maintenance crew using a special vehicle equipped with flush water pump and blower. The wash water required for reverse circulation is derived from the clear water tank. This mobile maintenance and cleaning service is also able to provide the concentrated chemicals to the CWW. It is recommended to draw up such a mobile maintenance service whenever care has to be taken of at least 20 CWW installations within a range of 200 km. This maintenance service may be organiszed by the local authority, or by a contractor conjointly run by the users of the facilities.

7. Summary

The Compact Water Works (CWW) is a water treatment facility which can be classed as a low cost technology for drinking water supply, on account of its low initial and operating cost, but which is not on a par with the mobile systems often to be found on containers.

The CWW is a top-quality permanent water conditioning plant for supplying drinking water to local communities of up to 1000 inhabitants, located at some distance from more populous centres, often in geographically inaccessible rural areas.

The CWW requires no electrical power for the water conditioning process, the hydraulic gradient of the water being completely adequate to effect all the conditioning and dosing stages.

The water treatment process proceeds as follows:

- Admixture of a flocculation agent
- Reaction
- Sedimentation
- Filtering through a slow sand filter, which works biologically and eliminates germs as well as fine particles
- Addition of a sterilizing agent (germicide)
- Clear water tank

As its name implies, the CWW requires little space for its installation. The space needed by the process is occupied by a series of concentric annular rings. Various versions can be supplied, for throughputs ranging from 1 to 10 m^3/hour. The optional installation of a nozzle bottom enables the water works to be equipped at any time with a rapid filter assembly, allowing the capacity to be raised to about 10 times the original throughput. Electrical power will however be needed for this.

The maintenance of the CWW is very simple indeed and can be effected by untrained personnel in a very short time. This work comprizes layer removal of the slow sand filter and the periodic replenishing of the chemicals, and this only at intervals of a few weeks/months.

The results of lengthly pratical trials coincide with the tendency published by the National Research Centre for Environmental Technology of Nakpur, India (NEERI) and the International WHO Reference Centre for Municipal Water Supplies, of The Hague. Thus the CWW meets all the parameters laid down by the World Health Organization.

The CWW can be supplied as a complete pre-fabricated unit, or - in the case of the larger type - can be assembled on site from pre-fabricated components. The work of installation can in the main be effected by local personnel, thus obviating large foreign exchange lots, excepting of course the expenditure for licence fees and specific special components.

If a number of CWW units are required to be installed, it would be advisable to have a centralized mobile maintenance service available.

The Compact Water Works thus represents a genuine enrichment for the countries of the developing world, so as to solve current problems within the framework of improvements to the infrastructure of the country, and in particular, the supply of good clean drinking water, and to safeguard this requirement for the future.

Paper 10

Roughing Filters as Pre-Treatment for Slow Sand Filtration

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SUMMARY

Operational difficulties with Slow Sand Filters are experienced in East-Africa and probably also in other developing countries. These problems reduce the credibility of Slow Sand Filtration, which still remains an appropriate water treatment process, and promote the implementation of more sophisticated purification methods which are mostly unsuitable for developing countries. The operational problems with Slow Sand Filters are mainly caused by the use of such inappropriate technologies or by insufficient pretreatment.

Horizontal-flow filters have the advantage of unlimited length and hence possess a large silt storage capacity. In Europe such filters are operated at high filtration rates of 5 - 10 m/h. Investigations on Lab-scale in developing countries with high turbid water indicate the necessity to reduce the filtration rate to 0.5 - 1 m/h if the filtrate should meet the quality standard required by Slow Sand Filters. Field tests will have to prove the efficiency and the design guidelines for the discussed Horizontal-flow Roughing Filter which being simple in construction and operation will be an appropriate pretreatment process for Slow Sand Filtration.

1. APPLICABILITY OF SLOW SAND FILTRATION

The raw water to be treated by Slow Sand Filtration (SSF) is mostly drawn from rivers, impoundments and lakes. Surface water as such often shows a big variation of its bacteriological, chemical and physical property. Usually SSF can cope with the annual bacteriological and chemical fluctuation but suffers when the physical characteristics show for instance high Turbidity and Suspended Solids concentration. According the WHO/IRC - Manual [1] raw waters exceeding a mean Turbidity value of 10 NTU (Nephelomatric Turbidity Unit) or reaching a value of 50 NTU or more for a few weeks a year must be pretreated.

Application of turbid water with a high Suspended Solids concentration to SSF will cause operational problems. Short filter runs requiring frequent cleaning and preventing a proper development of the biologically active top-layer (Schmutzdecke), which is essential for the production of a bacteriological safe water, will be the result.

2. PRETREATMENT METHODS

The conventional methods of pretreatment in order of increasing efficiency are
- sedimentation
- storage
- filtration
- flocculation combined with sedimentation and / or filtration

In practise the last two methods are predominant. Experimental tests and practical experiences show that plain sedimentation in most cases is insufficient to produce a water which meets the physical standards required by SSF. Also the provision of a large storage volume does not necessarly guarantee water physically fit for SSF. Flocculation which depends on the external supply of chemicals impairs an independent pretreatment method, applicable in rural areas of developing countries. Out of the listed pretreatment possibilities filtration remains as the most practicable process.

3. CONSIDERATIONS ABOUT FITRATION

According of the size of the filter medium used, filtration can be classified into
- Slow Sand Filters, SSF (∅ 0.15 - 0.35 mm)
- Rapid Sand Filters, RSF (∅ 0.5 - 2 mm)
- Roughing Filters, RF (∅ > 2 mm)

As most of the solids are retained at the top of the filter bed SSF is mainly a surface filtration with a small silt storage capacity. Hence after clogging these filters are cleaned by scraping off the first few centimeters of the clogged filter bed. RSF and RF allow a deeper penetration of the suspended matter and are therefore considered as space filters with a large silt storage capacity. Space filters require more complicated equipment for their regeneration as the entire filter medium has to be cleaned. With RSF the filter bed is fluidized by a backwash process using high wash-water rates and even compressed air to support the scour of the sand grains requiring the installation of rather sophisticated equipment. The coarse filter material of RF impairs such a backwash process. The solid matter retained in the filter bed will be removed by flushing or, where this process is not efficient enough, by excavating the filter material, washing and replacing it.

A second classification criteria is the flow direction. Vertical downward filtration is usual with SSF and RSF. To take advantage of stratification after backwashing vertical upward filtration is occasionally used with RSF. Vertical filtration asks for a vertical alignment of the structures required to contain the supernatant water, the filter medium and the drainage system. Structural constraints limite the height of the filter box which restricts especially the depth of the filter bed. Such shortcomings can be avoided by applying horizontal filtration for which the filter length is practically unlimited.

The combination of the advantages from the two preceding paragraphs leads to a special filter type, the so-called Horizontal-flow Roughing Filter (HRF). HRF have a large silt storage capacity owing to the coarse filter media and long filter length. This in its turn allows the run of such filters over years without any cleaning resulting in simple and unpretentious operation. Once silted up the HRF can easily be cleaned manually.

Concerning technology HRF appear qualified for an appropriate and selfreliant pretreatment of surface water prior to SSF application. The purification efficiency under tropical conditions of such filters especially the capacity to reduce high Turbidity and Suspended Solids concentration remains to be proved.

4. HORIZONTAL-FLOW ROUGHING FILTERS (HRF)

4.1 Examples in Europe

There are several water treatment plants in Europe using HRF. As examples only two are mentioned and briefly described, the in the literature well described Dortmunder Treatment Plant and the recently constructed plant in Aesch, both applying HRF prior to SSF-basin for artificial groundwater recharge.

Around 20 years ago the Dortmunder Stadtwerke (W-Germany) constructed gravel filters to reduce the silt load from the river Ruhr water. These filters have the considerable length of 50 to 70 m and are operated with a filtration rate of 10 m/h. The raw water has an average Suspended Solids concentration of about 8 ppm and it exceeds the value of 20 ppm for 30 days a year.

Since 1976 water from the river Birs has been pretreated with HRF at the treatment plant of Aesch (Switzerland). These filters consist of a length of 15 m and are operated at a maximum filtration rate of 8 m/h and at an average rate of about 5 m/h. The mean Suspended Solids concentration of the river Birs amounts to about 7 ppm. Since their setting into operation 4 years ago the HRF were not cleaned and the succeeding SSF had a filter run of 3 years before the first cleaning.

4.2 Applicability of HRF in Tropical Countries

Uneven annual rainfall distribution, deforestation and land cultivation methods which promote soil erosion are the main causes of highly turbid and silted surface water in tropical countries. As an example the results of a survey [2] on the water quality of surface water in Tanzania are shown in the following table :

	wet season	dry season	annual
average Turbidity (NTU)	41	28	35
average Susp. Solids (ppm)	96	42	69

Unlike rivers in moderate climate the rivers in tropical countries mostly carry much higher loads of Suspended Solids. Such different water quality properties toghether with local conditions will influence the decision on the water treatment technology to be applied, its design and its operation. Design guidelines valid in moderate climates are not necessarly applicable under tropical conditions.

To investigate the design and the performance of HRF under such conditions studies at the Asian Institute of Technology in Bangkok, Thailand and at the University of Dar es Salaam, Tanzania were performed.

The investigations at Bangkok are described in Reference [3] and may be summarized as follows. After preliminary laboratory tests field tests using a 5 m long HRF were run twice for a period of 40 days. The raw water Turbidity varied between 25 and 140 JTU with an average of about 60 JTU. The tested HRF produced an effluent of 10 to 20 JTU by a constant filtration rate of 0.6 m/h. Effects of different filtration rates or of different filter material for the HRF on the purification efficiency were not explored in that study but were taken up in the investigations carried out at the University of Dar es Salaam [4]. The results of this work will be presented in the following.

4.3 HRF-Studies at the University of Dar es Salaam

Short term filter tests with vertical filter columns (Ø 20 cm, filter bed length 1 m) were run with filtration rates between 0.5 and 8 m/h and with the following aggregates :

- Coral Limestone, ex Kunduchi, Dar es Salaam (graded in 5 fractions from 2 to 64 mm, surface extremly porous, rounded edges)
- Limestone, ex Msolwa, Morogoro (graded in 3 fractions from 4 to 32 mm, dense and rough surface, sharp edges)
- River-Sand, ex Matema, Lake Nyasa (one fraction 1 - 2 mm, Quartz and basaltic grains, rounded edges)

The results of the filter tests graphed in Fig. 1 show that the property of the gravel surface does not much influence the purification efficiency in respect of Turbidity reduction. As expected finer aggregates have a higher purification capability than coarse material. Remarkable is the sudden increase of the removal efficiency with filtration rates below 2 m/h. This incontinuity is probably caused by a change of the flow conditions. According to Todd [5] one can

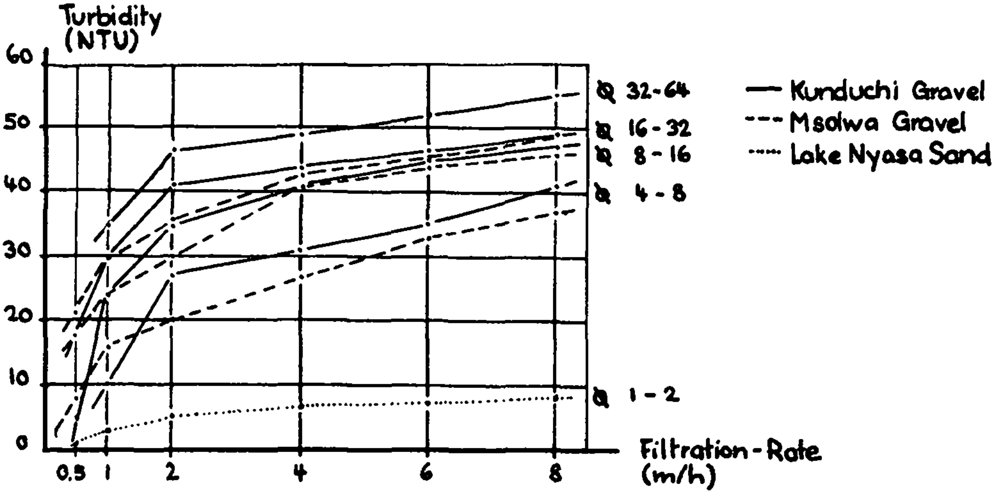


Fig.1 Turbidity reduction in a 1 m long vertical filter bed in correlation with filtration rate for different aggregates

expect laminar flow conditions in porous media at a Re-number (Reynolds-number) smaller than 1. The transition zone starts somewhere between a Re-number of 1 and 10 and Re-values bigger than 600 to 1000 describe turbulent flow conditions. The correlation between Re-number, different sizes of aggregate and filtration rate are graphed in Fig. 2. For the tested Kunduchi gravel laminar flow probably exists below a Re-value of about 7. In this zone a reduction of the Re-number achieved by decreasing the filtration rate and/or by the use of finer filter material results in a large increase of the removal efficiency. Hence the Re-value should be checked during the design of roughing filters.

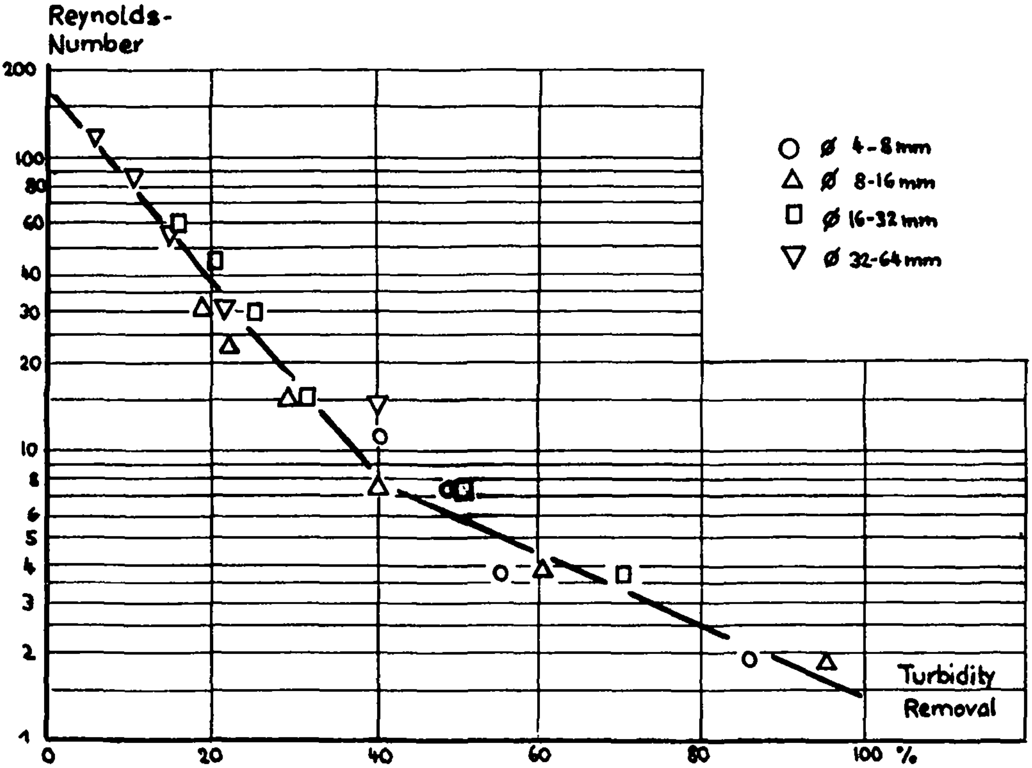
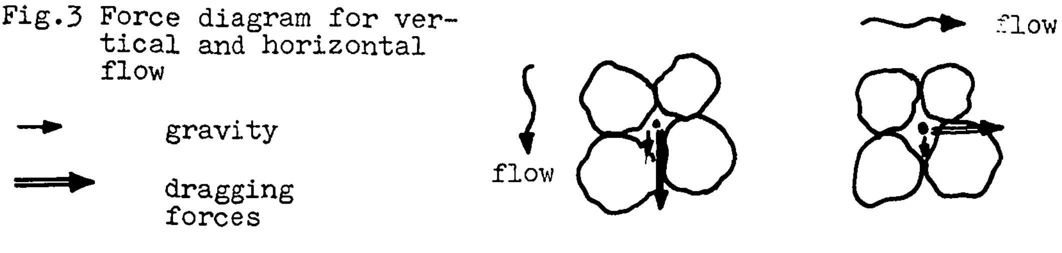


Fig.2 Turbidity removal in correlation with Re-number for Kunduchi aggregates in a 1 m long vertical filter bed

As sedimentation is the major process in the removal of discrete matter by roughing filters vertical and horizontal filter tests were run simultanously to investigate any influence of the flow direction in respect to Suspended Solids reduction. The force diagram of Fig. 3 shows that gravity, which is responsable for sedimentation and dragging forces, which cause scour are either parallel or perpendicular to each other. At the applied filtration

Fig.3 Force diagram for vertical and horizontal flow



rates up to 8 m/h the tests showed no significant difference for vertical and horizontal filtration, thus allowing the results from vertical filtration tests to be applied to horizontal flow filters.

To test Turbidity reduction of a HRF an open channel with the dimension of 0.35 * 0.40 * 15.00 m was filled with different gravel fractions according Fig.4. The Turbidity decrease along the filter length for different filtration rates is graphed in the same Fig. The most important conclusion from these filter tests is the fact that under given conditions (made-up water, filter configuration) only low filtration rates of 0.5 and 1.0 m/h are able to reduce the Turbidity to a value of 10 NTU or less.

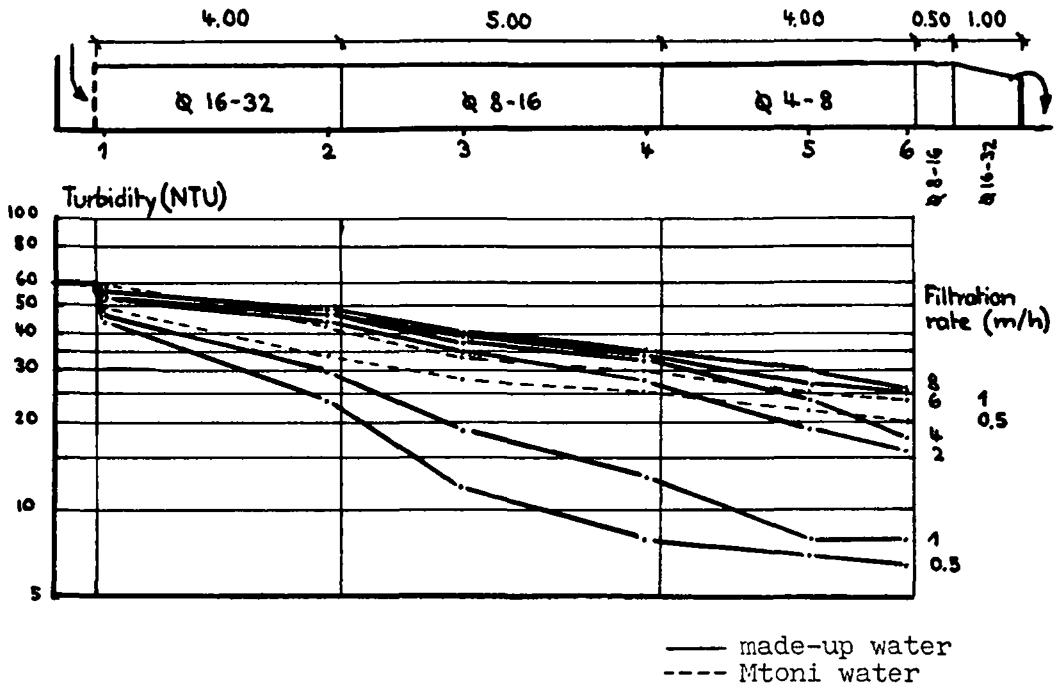
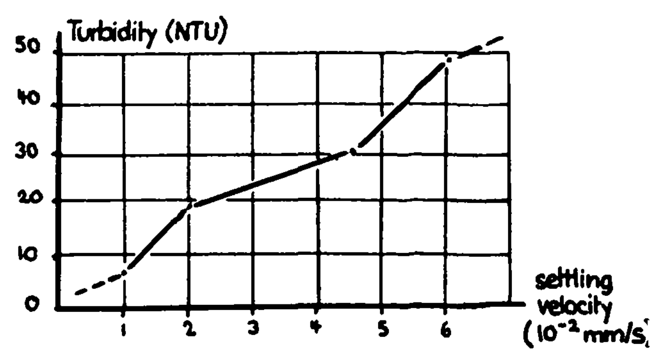


Fig.4 Longitudinal section of HRF with Turbidity removal in correlation to filtration rate

All the tests discussed so far were performed with made-up water. Tapwater was mixed with water-borne sludge to a Turbidity of 60 NTU and to a Suspended Solids concentration of about 100 ppm. The settleability of the sludge in the made-up water shown in Fig.5 is considered to be moderate.

Fig.5 Settling test in absolut quiescent conditions for made-up water described by Turbidity change



In a final experiment the HRF-model filtered water from a natural water source (Mtoni water). The initial Turbidity was recorded to be again 60 NTU and its reduction is graphed in Fig.4. By applying filtration rates of 0.5 and 1 m/h the effluent still had a Turbidity of about 20 NTU. In spite of this relative high value the Suspended Solids concentration in the filtrate amounted to 4 ppm as shown in Fig.6. Obviously the residual Turbidity was caused by the true Colour which was recorded to 150 mg Pt/l. Turbidity is used as parameter that is easy to determine. But critical for SSF operation are the Suspended Solids which at high concentrations cause rapid clogging of the filter. Therefore Turbidity as the decisive quality parameter for raw SSF water should be used with care. For the tested Mtoni water HRF would still have a satisfactory performance.

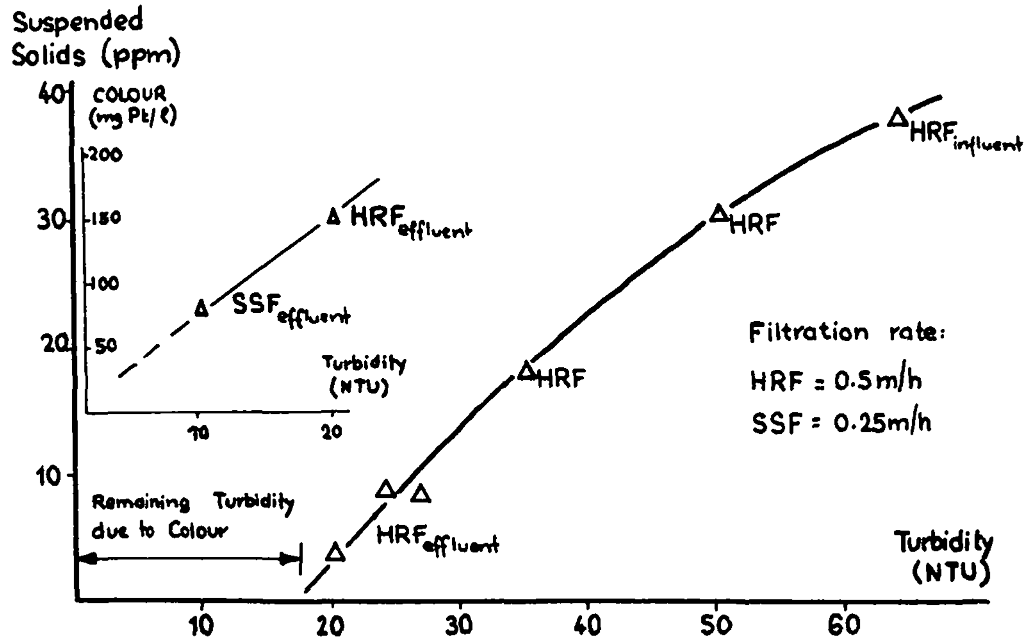


Fig.6 Reduction of Turbidity and Suspended Solids concentration by Horizontal-flow Roughing Filtration for Mtoni water and decrease of Turbidity and Colour for HRF-filtrate by Slow Sand Filtration

The Lab-results will be checked by field tests. Short term tests at different major treatment plants in Tanzania using SSF with or without chemical flocculation and sedimentation as pretreatment are under progress. A long term test with a pilot plant using HRF and SSF will start its operation soon. The results of these field tests will be presented at the Conference.

5. CONCLUSION AND OUTLOOK

HRF present a simple and self-reliant pretreatment method for surface water prior to SSF. They are indepent from external imputs (e.g. chemical supply) and do not require skilled manpower for operation. Lab tests show that filtration rates between 0.5 and 1 m/h should probably be applied in contradiction to the experiences in Europe where HRF are run at 5 to 10 m/h. The Lab results have to be verified by field tests. The conclusions of the field experiments are anticipated for mid-1981.

If HRF proves to be the appropriate pretreatment for SSF the use of these 2 filter types will have a broad application in the rural water supplies of developing countries.

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Abbreviations

SSF Slow Sand Filter(s)/Filtration
RSF Rapid Sand Filter(s)/Filtration
HRF Horizontal-flow Roughing Filter(s)/Filtration
RF Roughing Filter(s)/Filtration

NTU Nephelomatric Turbidity Unit
JTU Jackson Turbidity Unit

References

[1] Slow Sand Filtration for Community Water Supply in Developing Countries, WHO/IRC, December 1978

[2] Slow Sand Filter Research Project, Report 1, University of Dar es Salaam, February 1980

[3] Horizontal-flow Coarse-Material Prefiltration, Asian Institute of Technology, Bangkok, July 1977

[4] Slow Sand Filter Research Project, Report 2, University of Dar es Salaam, July 1980

[5] Ground Water Hydrology, Todd, 1959

Filtres Dégrossisseurs Horizontaux comme Prétraitement avant Filtration Lente

RESUME

Les difficultés de fonctionnement des filtres lents, en Afrique Orientale et probablement dans d'autres pays en voie de développement, réduisent la crédibilité du procédé bien qu'il demeure une méthode adaptée pour le traitement des eaux. Elles provoquent également la mise en oeuvre de méthodes de traitement plus sophistiquées qui ne conviennent pas aux pays en voie de développement. Les problèmes rencontrés avec les filtres à sable lents viennent notamment d'un traitement préalable insuffisant ou de technologies inadaptées.

Les filtres à flux horizontal ont comme avantage une longueur non limitée et peuvent ainsi permettre de stocker une grande capacité de limon. En Europe, ces filtres fonctionnent à des vitesses élevées, 5 à 10 m/h. Des études en laboratoire indiquent que dans les pays en voie de développement, il est nécessaire, pour traiter de l'eau à forte turbidité, de réduire la vitesse de filtration à 0.5 - 1 m/h afin d'atteindre des standards de qualité suffisants.

Pour leur mise en oeuvre pratique, des essais préalables sur place doivent etre conduits afin de définir les concepts du filtre à flux horizontal, qui conviendra dans le processus envisagé.
Rappelons que ces filtres sont de construction et de fonctionnement très faciles.

Paper 11

Various Aspects of the Optimization of Surface Water Treatment in Tropical Countries

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RESUME

A l'aide de deux exemples concrets réalisés l'un au Nigéria, l'autre en Guyane on montre comment, lors due développment d'une technique optimale de traitement des eaux de surface en pays tropical, deux aspects sont essentiellement à considérer. On montre d'une part comment certains détails de conception permettent un contrôle et une maintenance aisée de l'installation. Ceux-ci incluent entre autres: un cheminement de l'eau et des trop-plein pouvant être suivis visuellement, l'emploi d'un minimum de pompes standardisées, des valves à commande soit pneumatique soit manuelle, des pompes doseuses simples et robustes et couplées aux pompes d'eau brute, un choix des matériaux et du type de construction acceptant une certaine marge de tolérance.

La filière, de traitement appliquée doit d'autre part être parfaitement adaptée à l'eau brute utilisée et permettre d'obtenir la qualité de l'eau requise avec un investissement et des frais d'exploitation minima. Quand on n'a pas à faire face à des pointes saisonières élevées en matières organiques ou minérales, une microfloculation sur filtre multicouche permet un traitement direct sans floculation-décantation préliminaire. Cela conduit à des économies substancielles sur les frais d'investissement ainsi que sur les dépenses en floculants.

ABSTRACT

The example of two real cases, the one realized in Nigeria and the other one in Guyana, shows how the development of an optimal treatment technology for surface water in tropical countries requires two main aspects to be considered. It is shown how certain design characteristics lead to an easy control and maintenance of the plant; these are for example water channels and overflows that can be followed visually, the use of a minimum of standardized pumps, valves with pneumatic and manual operation possibility, simple and resistant metering pumps which are coupled with the raw water pump,

a choice of materials and a type of design that allows a certain tolerance margin during the plant erection phase.

On the other hand the treatment has to be perfectly adapted to the raw surface water used, in order to achieve the required water quality with minimal investment and operating cost. When one has not used to be faced with extreme seasonal values of suspended solids or organic matter, a microflocculation on a multilayer filter will allow a direct treatment without a preliminary stage of flocculation-sedimentation. This brings substantial savings in the investment cost and in the expenses that would otherwise be incurred for the higher amount of flocculant.

INTRODUCTION

The optimisation of a water treatment technique is not necessarily synonymous with extensive automation or complex technology. Rather, it begins with the account of the hydrological, geographical and economic factors in the particular case, and putting into operation a scheme which will produce potable water of the required quality while at the same time minimising investment and running costs. In areas where the industrial supply centres may be far away, everything concerning the control and maintenance of the installation, as well as the renewal of chemicals, assumes prime importance. It will also be necessary to take account of the fact that both small modular and large installations must be individually dimensioned for each new situation in order to give the best results under specific conditions.

These two main objectives, easy maintenance and the optimisation of the process for the water to be treated so as to minimise operating costs, are illustrated here by means of two actual examples, one in Nigeria, the other in Guyana. Maintenance and control are facilitated by the use of treatment units which, permit simple manual operation and need only a few different spare parts. Treatment by direct filtration, i.e. with flocculation on the filter and no prior sedimentation, is in many cases well suited to the waters encountered in tropical rivers. It allows, very good removal of the humic and colouring materials generally present in the water to be achieved with a greatly reducted flocculant usage.

INSTALLATION OF A LOW-COST EASY-MAINTENANCE TREATMENT PLANT

Basic Principles

Having chosen the treatment process on the basis of the raw water quality and the standards required (see following section) it is then a question of putting the project into realisation.

Bearing in mind economic considerations and the fact that many items have to be imported and transported over long distances, the steps taken can be based on the following principles. The first consideration is to look for ways of reducing construction costs and conceive a process which uses the least possible energy. It is essential that the operation of the plant is easy to control and that in all cases this should be performed manually, above all whether the local labour situation makes this possible. Finally, it is imperative that all maintenance work can be carried out locally because the importation of parts may sometimes cause a delay of several months. On should, as far as possible, try to use materials and equipment which are obtainable locally. This is also advisable for the filter media as the filter can then be specifically designed for it. The process and water flow should be arranged so that any unnecessary expenditure on energy is avoided. These criteria are all the more relevant in that the installations under consideration employ medium or large units, requiring specific engineering and construction operations on site which exceed simple assembly.

Technically this is implemented as follows:

- use of the minimum number of pumps and avoidance of all intermediate pumps.
- water flow which can be followed visually and equipped with overflows and expansion pipes which enable the use of detection instruments to be avoided.
- utilisation of soft seals in manual valves, and in the case of automatic valves and equipment, choice of a pneumatic or hydraulic transmission system which can be repaired on the spot.
- pipes which are thick enough to tolerate a certain amount of corrosion, thereby avoiding problems associated with applying or repairing interior coatings.
- linking operation of the raw water and dosing pumps; the latter, nevertheless, remain critical points in the process.
- civil engineering which avoids the need for expensive final adjustments by allowing generous tolerance margins.

<u>Water treatment installation in Nigeria</u>

The treatment process is designed for water from a river containing up to 1000 g/m3 of suspended solids.

The installation (see Fig. 1 and 2) comprises

- a water intake with screen and integral raw water pumping station
- cascade aeration
- flocculation sedimentation, Sulzer OPUR system with recirculation of sludge permitting optimal utilisation of chemicals (aluminium sulphate and lime)
- rapid filtration in 4 open sand filters
- chlorine disinfection
- two service reservoirs

Throughput is 400 m3/h

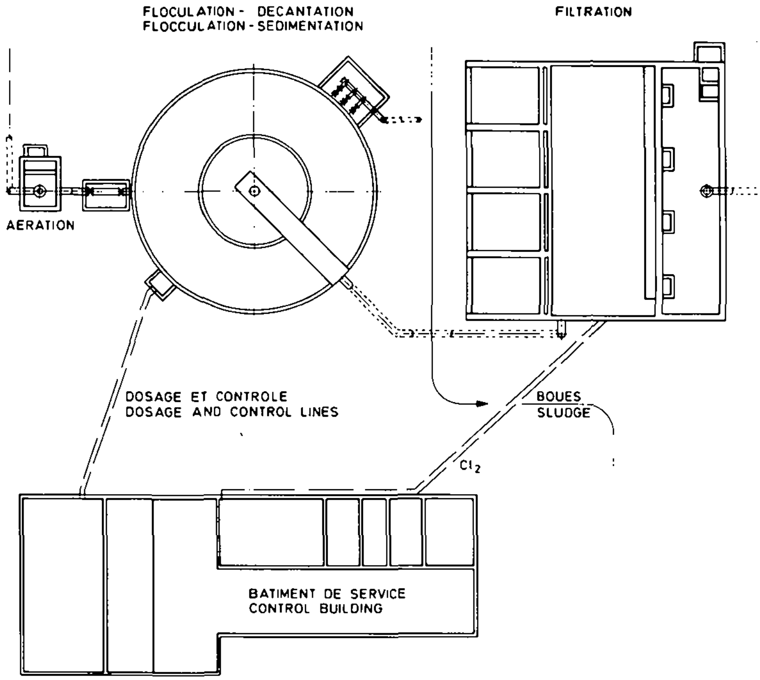

Fig. 1 Water treatment in Nigeria:
Site plan

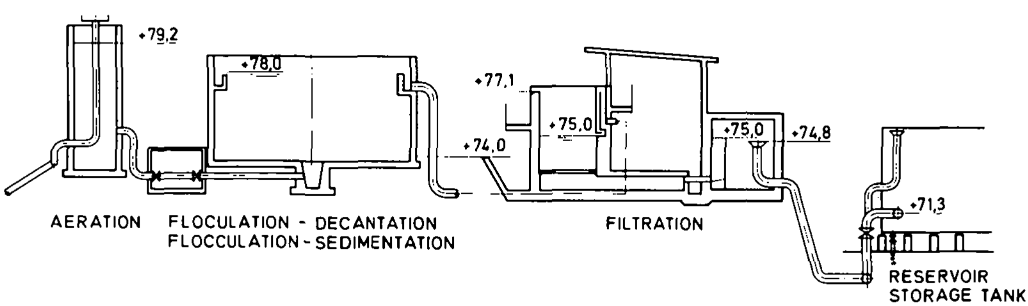

Fig. 2 Water treatment in Nigeria:
Hydraulic diagram

Special design features of the plant

The installation is designed to eliminate any intermediate pumping by utilising gravitational flow across all the units between the cascade and the service reservoirs. The base of the sedimentation is fitted with valves which enable the settled sludge to be run off manually under gravity. The wash water vessel has an outlet at level 74.8 which permits storage of sufficient water for one complete filter washing before the water is diverted to the service reservoirs. If these are closed the water overflows into an open channel, alerting the station personnel.

Following the flocculation sedimentation the water is distributed equally to each of the filters. A system of patented submerged partitions ensures that the water is spread uniformly over the whole
1 filter surface without disturbing the filter medium, (see Fig.3). Thanks to the system of regulation by overflow, the water level after washing is just above the surface of the filter medium, and its subsequent rise gives an indication of how clogged the filter has become. The state of the filter is thus easy to check visually. When the maximum load is exceeded part of the water flows into the backwash water channel and the noise signals the fact.

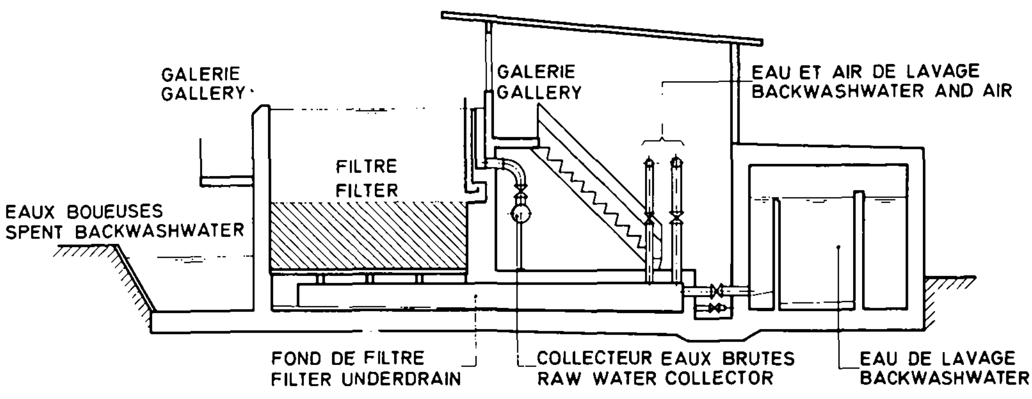

Fig. 3. Water treatment in Nigeria:
Filtration unit

Filter washing is carried out manually with visual checks on the washwater pump settings and aerators from the covered gallery. The dirty water outlet valve is opened after stopping the pumps. It is the only hydraulically operated valve in the whole plant. Experience has shown that the entire plant operation, including washing, can be efficiently and safely conducted by local personnel after a short instruction period.

TREATMENT BY DIRECT FILTRATION

Treatment of water from the river Le Comté (Cayenne, Guyana)

The Le Comté river is subject to an equatorial régime which in general characterizes the surface waters in Guyana (1). The water is relatively difficult to treat because of its colour, which is high all through the year (see table 1). This colouration is mainly due to the presence of humic acids and iron, which can only be eliminated efficiently if flocculation, generally effected by addition of aluminium sulphate, is carried out at a pH of between 5 and 6. (2) (3). Although below the values usually employed to obtain aluminium hydroxide precipitation it is quite appropriate for this type of tropical water (see Fig. 6 and 7). The water in question is low in hardness and the pH adjusted by addition of lime.

As the average suspended solids concentration is between 20 and 50 g/m3 and does not exceed 200 g/m3, and in the absence of excessive quantities of organic matter, a decision in favour of a direct filtration process was taken. The flocculation - sedimentation stage before filtration - as used in the old plant - was thus no longer necessary. Even the flocculation installation it self was now superfluous; a microflocculation could be induced in the filter water inlet pipe.

Treatment by direct filtration with multi-layer filters had already been applied successfully to various types of surface waters from lakes and rivers (4) (5).Analogous results have been obtained at installations in Australia (6) and the U.S.A. (7). These show that, with a significantly reduced investment cost and using 3 to 7 times less flocculant than plants involving flocculation sedimentation, direct filtration provides water of equivalent quality. It is due to the fact that the capacity of the low concentrations of flocculant to form stable flocs is augmented during their passage through the filter pores.
Furthermore, direct filtration produces less sludge. It is well qualified to cope with suspended solids concentrations of up to 200 g/m3. The capacity and efficiency of direct filtration were confirmed by tests undertaken during the design stage and by the results obtained in practice (see below).

The treatment plant

The treatment process realised for the new installation serving Cayenne is outlined in Fig. 4. It consists of

- a raw water intake sited 42 km up-stream of Cayenne.
- a store for chemicals (aluminium sulphate, lime, hypochlorite)

- an oxydation facility ahead of the station
- a static mixer to provide destablisation energy during the introduction of flocculating agents at the inlet of the pipe leading to the filters.
- a set of three 3-layer filters.
- facilities for final pH adjustment.
- a calcium hypochlorite doser for disinfection of the distribution system.
- two storage tanks ahead of the pumps to the service reservoirs about 30 km from the station.

The nominal output of the installation as described is 10'000 m3/day.

<u>Flocculation-filtration with the multilayer filter</u>

When a granular bed is used for flocculation-filtration the highest efficiencies are attained if filtration takes place throughout the depth of the bed. This gives the best possible retention of solids per volume of filter. Multilayer filters (2 or 3-layer) offer the best way of achieving this objective. The Guyana filters therefore consist of three layers as follows:

	Grain Size (mm dia)	Density (g/cm3)
Pumice stone	2 - 3	1.1
Hydroanthracite	1.2 - 2	1.5
Quartz sand	0.6 - 1	2.4

The lower layer of fine sand retains the smaller particles left in the water after passage through the two upper layers. These, with their larger grain size, retain the larger particles and flocs, but not all at the filter surface where they would greatly increase head-loss and reduce the filter running time. It is noteworthy that although the best retention of hydroxide flocs occurs in the empty pore spaces of these upper layers, the hydraulic shear forces experienced by the flocs are reduced thanks to the bigger grains.

The phenomena encountered in flocculation-filtration involve many parameters, from the characteristic colloid-chemistry of the water to be treated to the parameters of flocculation and filtration such as flow-rate, quantity of flocculant added, reaction time, depth and granular properties of the various layers, etc.

Methods have been developed for achieving optimum operating conditions in each particular case where multi-layer filtration is applied (5) (8)

The results illustrated in Fig. 5 show the difference between a mono-layer filter with fine sand and a multi-layer filter. Run-times are clearly longer in the case of multi-layer filtration, yielding tangible savings in wash-water. One may note here that in the case of the Cayenne installation, with an average suspended solids concentration of 35 g/m3, filtration runs of 40 h are possible before the head-loss reaches 1.5 m water.

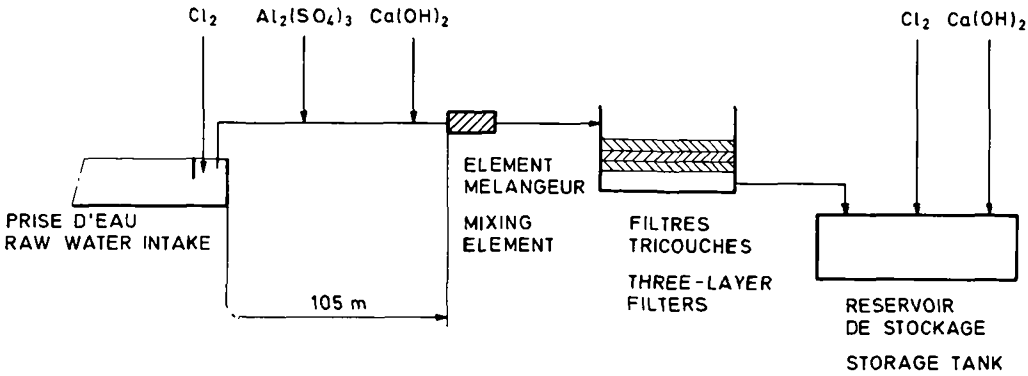

Fig. 4 Guyana: the treatment process

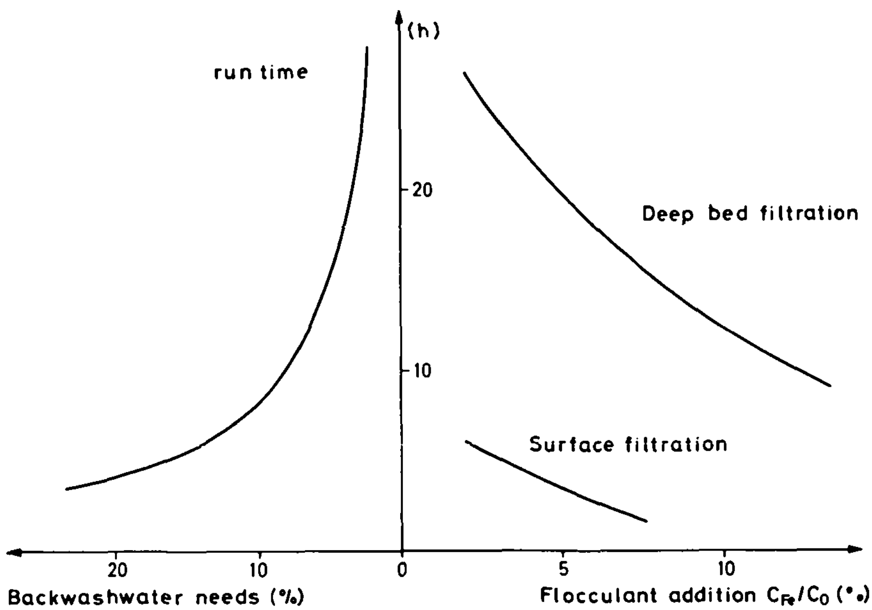

Fig. 5 Comparison of deep bed (multi-layer) and "surface" "mono-layer) filtration under typical operating conditions.

The corresponding wash-water requirement is 1.2 %. Moreover, because of the greater pore space available in a multi-layer filter, more flocculant can be utilized compared with a "surface" (mono-layer) filter. The multi-layer filter therefore represents the optimum technique for direct water filtration. It is very efficient in removing algae and micro-plankton. Experience has shown that this type of filter gives satisfactory results even when raw water suspended solids concentrations of the order of 200 g/m3 are reached.

Operational Results

A series of tests carried out in parallel with the plant and in the laboratory enabled the optimum operating conditions for flocculation to be established (4).

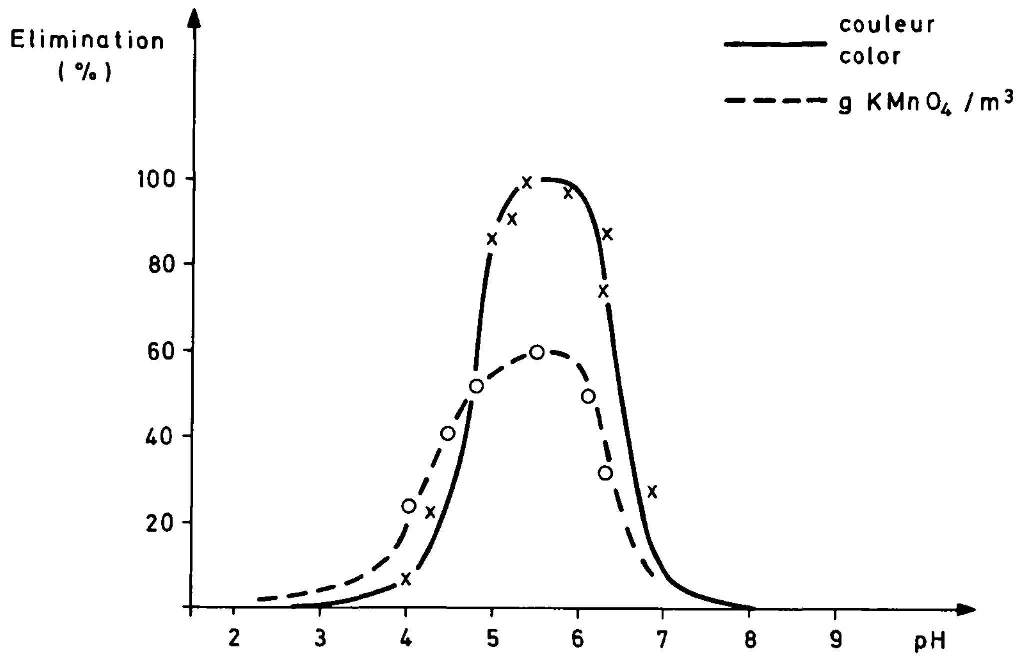

Fig. 6 Treatment of river water in Guyana:
Removal of oxygen-demanding and coloured matter during flocculation-filtration with .58 g/m3 Al as a function of pH.

Analysis	Water			
	Raw	Entering Filter	Leaving Filter	Neutralised and Chlorinated
pH (-)	6,28	5,56	5,55	9
Colour (Pt/Co)	80	80	0	0
Turbidity (FTU)	9,5	-	0,05	0,05
Al (g/m3)		0,55	0,07	0,07
Fe (g/m3)	0,58	0,58	0,02	0,02
NH_4^+ - N (g/m3)	0,38	0,38	0,06	0,06
$KMnO_4$ demand (g/m3)	27,3	-	8,9	-
Cl_2 total (g/m3)		0	0	2

TABLE 1. Operational results

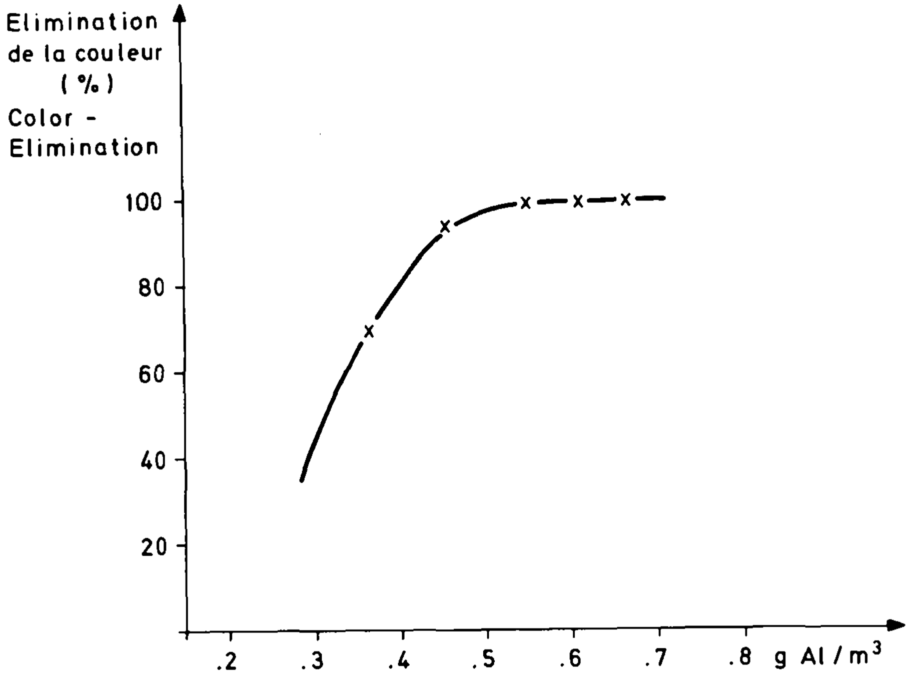

Fig. 7 Treatment of river water in Guyana:
Removal of colour as a function of flocculant dosage before filtration.

Figures 1 and 2 indicate the optimum conditions for flocculation-filtration. These results are in agreement with those from analogous cases (2) (3) where the water being treated contained, respectively, fulvic and humic acids.

The first results showed that oxidation by chlorine ahead of the plant did not bring any improvement in the efficiency of flocculation-filtration. The iron partly responsible for the colour is in fact entirely present in the ferric state as a complex with humic acids, and does not interfere with flocculation-filtration. Chlorine oxidation merely risks increasing the difficulty of eliminating the fulvic acids and is thus unnecessary. Pre-oxidation has therefore been discontinued in favour of direct flocculation-filtration with the following concentrations of chemicals:

Aluminium sulphate	6.3 g/m3
Lime	9.0 g/m3
Calcium hypochlorite	4.5 g/m3

(for disinfecting the distribution system)

The results obtained are given in table 1. It is particularly important to note the absolute values as well as the removal efficiencies obtained by flocculation-filtration in the case of colour elimination (100 %), iron (97 %), oxygen demand (67 %), ammoniacal nitrogen (84 %) and turbidity (95 %).

CONCLUSION

It has been shown that the optimisation of treatment plants for tropical countries involves several considerations, including the technical realization of the plant as well of the development of a treatment process best adapted for the water concerned. An essential requirement is the need for flexibility in operation. In addition to the aspect of the equipment and its suitability for large variations in load, there is the choice of process itself. It must, for example, be capable of maintaining efficiency if there is a gradual change in the quality of the raw water. This has certainly been the case in Guyana, where the river water has almost doubled in oxygen demand and colour during the past four years. Despite this, the flocculation-filtration plant has had no difficulty in achieving the results expected.

ACKNOWLEDGEMENT

The autors want to aknowledge greatfully the contribution of Mr. J.C. Ginocchio, Mr. L.Antoine, Dr. R.Sales et Dr. J.Scutt to the paper presented here.

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Paper 12

Design of Water Treatment Plants in Developing Countries

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I. Introduction

The goal of water treatment is the production of safe and appealing water without excessive cost to the consumer. Some primary considerations and guiding principles in water treatment plant design include:

1. the overall design should be simple, reliable, durable, and economical

2. treatment processes should be integrated to provide efficient production of high quality water at minimal cost

3. excess hydraulic capacity in channels, pipes, and basins should be provided for possible future hydraulic overloading

4. plant flow should be uniformly distributed in each treatment unit through hydraulically balanced designs and layouts

5. plant processes and equipment should be simple to operate and maintain

6. service from manufacturers and chemical suppliers should be readily available

7. local conditions including climate, soil conditions and available labor should be evaluated

8. simple process controls and a justifiable degree of automation should be provided

9. imported equipment and materials should be minimized

10. disposal sites for plant wastes should be readily available

11. public attitudes toward plant location and plant operation should be evaluated

12. plant safety features should be provided

13. good access to all process units and a centralized plant operational control board should be provided

14. investigations on alternative sources of electrical power and raw water should be conducted to ensure an uninterrupted water supply and low cost treatment

All the items listed above are important considerations in water treatment plant design. However, items 1 through 9 are especially relevant in developing countries.

In general, technological conditions in developing countries are different from those in industrialized nations; thus, applications of higher technology have often been of little value to these countries. Therefore, it is imperative that engineers familiar with local conditions appraise new water treatment technologies and modify proven technologies to meet local needs. Major considerations that local engineers must evaluate are:

1. limitation of available capital

2. availability of both skilled and unskilled labor

3. availability of major equipment items, construction materials, and water treatment chemicals

4. applicability of local codes, drinking water standards, and specifications for materials

5. influence of local traditions, customs, and cultural standards on construction and operation of a facility

6. influence of national sanitation and pollution policies on construction and operation of a facility

2. Design Recommendations for Developing Countries

Conventional water treatment includes the following units:

- plant flow measurement
- coagulant feeding
- flash mixing
- flocculation
- sedimentation
- filtration
- disinfection

Presently, many water treatment plants that have high quality water sources employ the direct filtration treatment mode (no sedimentation). Furthermore, water treatment plants with an excellent quality of raw water commonly employ in-line filtration (no flocculation and sedimentation).

In developing countries, selection between conventional treatment, direct filtration or in-line filtration should be evaluated thoroughly. The conventional treatment mode is a more conservative and fail-safe system. Direct or in-line filtration should only be adopted when a comprehensive evaluation of raw water quality demonstrates the excellence of the raw water source.

(2.1) Plant Flow Meters

For developing countries, the Parshall flume is probably the most suitable type of flow rate measuring device. Advantages of the Parshall flume over other flow meters are:

- the Parshall flume can be made entirely from concrete which is usually available locally
- the Parshall flume contains a hydraulic jump which minimizes clogging and enhances chemical mixing

The most important design consideration for Parshall flumes is to locate them at high enough elevations such that outlet head loss does not constrict flow through the flume at anticipated peak and future flow rates.

(2.2) Chemicals and Chemical Feeders

The two basic types of chemicals necessary to efficiently operate most water treatment plants are coagulants and disinfectants. Fortunately, aluminum sulfate (alum) and chlorine are generally the most readily available water treatment chemicals in developing countries. Synthetic polymers are usually hard to obtain but natural polymers such as sodium alginate and starch may be available for use as coagulant aids. If pH control is necessary, lime, not caustic soda, should first be considered mainly because of its lower cost and better safety in handling.

Chemical feeders should be simple in design and easy to operate. A continuous, revolving cup type volumetric feeder is both accurate and durable, and ideal for use as a coagulant feeder in developing countries. This type of feeder has a feed range of over 1:300 and can also be paced to the plant flow rate. It is very important to have coagulant continuously fed; thus, an on-off type coagulant feeder should not be used.

For chlorine application, typical chlorine solution feeders should be used in developing countries. If chlorine usage is high, one ton chlorine cylinders with liquid chlorine gas evaporators should be used. However, if chlorine demand requires use of many one ton cylinders, manifolding the cylinders to one chlorinator may be preferred over providing evaporators for each cylinder.

(2.3) Flash Mixing

The most practical type of flash mixing in developing countries is utilization of a hydraulic jump downstream of a Parshall flume. Mixing in this manner requires no moving mechanical parts and incurs no operation and maintenance costs. In general, G-values from 500 to 1000 S^{-1} with 1 to 10 seconds mixing time are used as design guides. If the Parshall flume is not used, other effective flash mixing systems such as the pump injection type may be considered.

(2.4) Flocculation

Flocculation can be accomplished through the use of mechanical mixers in baffled tanks or by using baffled channels. Although variable speed mechanical mixers can input various degrees of mixing intensities, significant capital, operation, and maintenance costs often preclude their use in developing countries. Thus, baffled channels are preferable for use in developing nations. However, if conditions permit, mechanical flocculators may be used. In either case, preliminary jar tests would prove quite valuable in establishing design criteria for both flocculation and sedimentation.

Baffled channels can either be the over-and-under type or the round-the-end type. Over-and-under baffled channels have very little flow short circuiting and possess good mixing characteristics, but are prone to high head losses across the channel. For this reason, round-the-end type baffled channels are often used in large scale plants. The biggest drawbacks of this flocculation system are the variations in mixing intensity and mixing time due to the variations in plant flow. Plant flow can be maintained within a 1 to 3 range if enough clearwell and distribution system storage are provided.

Other important design considerations for enhancing flocculation include:

- the velocity gradient, G
- flocculation time, T
- utilization of tapered mixing

Commonly, values of GT range from 10^4 to 10^5 based on a minimum flocculation time of 15 minutes. Tapered slow mixing is also commonly practiced.

(2.5) Sedimentation

The two basic types of sedimentation processes are the horizontal flow type process and the upflow clarification process with a sludge blanket or with slurry recirculation.

Upflow type clarifiers are almost exclusively proprietary with internal mechanisms and structures designed by equipment manufacturers. These units perform quite well under certain conditions:

- relatively constant hydraulic loadings
- relatively constant raw water quality
- careful operational control by qualified operators

Because these conditions are difficult to satisfy continuously, selection of the upflow clarification process requires extensive evaluation, possibly with the aid of pilot plant studies.

The horizontal flow type sedimentation process, on the other hand, is commonly used for the following reasons:

- the process is more tolerable to shock hydraulic and water quality loads mainly due to a longer detention time

- the process gives predictable performance under most operational and climatic conditions

- the process is cost effective for large scale plants (say over 0.5 m^3/sec flow rate)

- the process has low operation and maintenance costs

For developing nations, horizontal flow type sedimentation tanks are often used. Manual, rather than mechanical, sludge removal is also preferred, with manual hosing operations required every 3 to 4 months, depending on solids loading. Most common basic design criteria for horizontal flow settling tanks are:

- surface loading - 20 to 80 m^3/m^2.day

- effective water depth - 3m minimum

- mean flow velocity - 0.25 to 0.5 m/min

- detention time - 2 to 4 hours

High rate settling modules such as lamella separators and tube settlers are the products of recent water treatment technology. If properly designed and installed, these modules improve clarification efficiency and permit two to three times higher surface loading rates on the basins. However, continuous sludge removal is required below the settler modules. Thus, a mechanical sludge removal unit must be provided. For this reason, use of high rate settling modules in developing countries is somewhat limited.

(2.6) Filtration

Water filtration is a physical and chemical process for separating suspended and colloidal impurities from water by passage through porous media. Although basic filtration design concepts have changed little in the past 30 or 40 years, some modifications, such as multi-media filtration, deep-bed coarse single medium filtration and use of polymers as filter aids, have improved particle removal substantially.

Presently, the most commonly used type of filter is the downflow type gravity filter. The two basic types of downflow gravity filters are:

- rapid sand filters 700 to 750 mm in depth with an effective size of sand between 0.5 and 0.6 mm. Common filtration rates range from 120 to 180 m^3/m^2.day.

- high rate dual media filters consisting of a 200 to 250 mm depth of 0.5 to 0.6 mm effective size sand and a 450 to 500 mm depth of 1.1 to 1.2 mm effective size anthracite coal (or an 1800 mm depth of 1.7 to 1.8 mm effective size coarse sand). Common filtration rates range from 240 to 600 m^3/m^2.day.

The major difference between the two types of gravity filters is the pretreatmant requirements. Rapid sand filters require a high degree of pretreatment (5 turbidity units or less in the filter influent) so as to minimize the amount of floc carried to the filters. High rate dual medial filters, on the other hand, perform best with small size but well coagulated floc. Turbidities up to 10 turbidity units can be effectively treated by high rate dual media filters if a small amount of polymer is added ahead of the filters.

Filters can also be classified by the type of filter rate control system used. Constant rate filtration and declining rate filtration are the two basic types of control systems, with the latter type requiring simpler control mechanisms.

For developing countries, rapid sand filtration with declining rate filtration controls may be used. However, declining rate controls have the following limitations:

- initial high turbidity levels following backwash due to high initial filtration rates

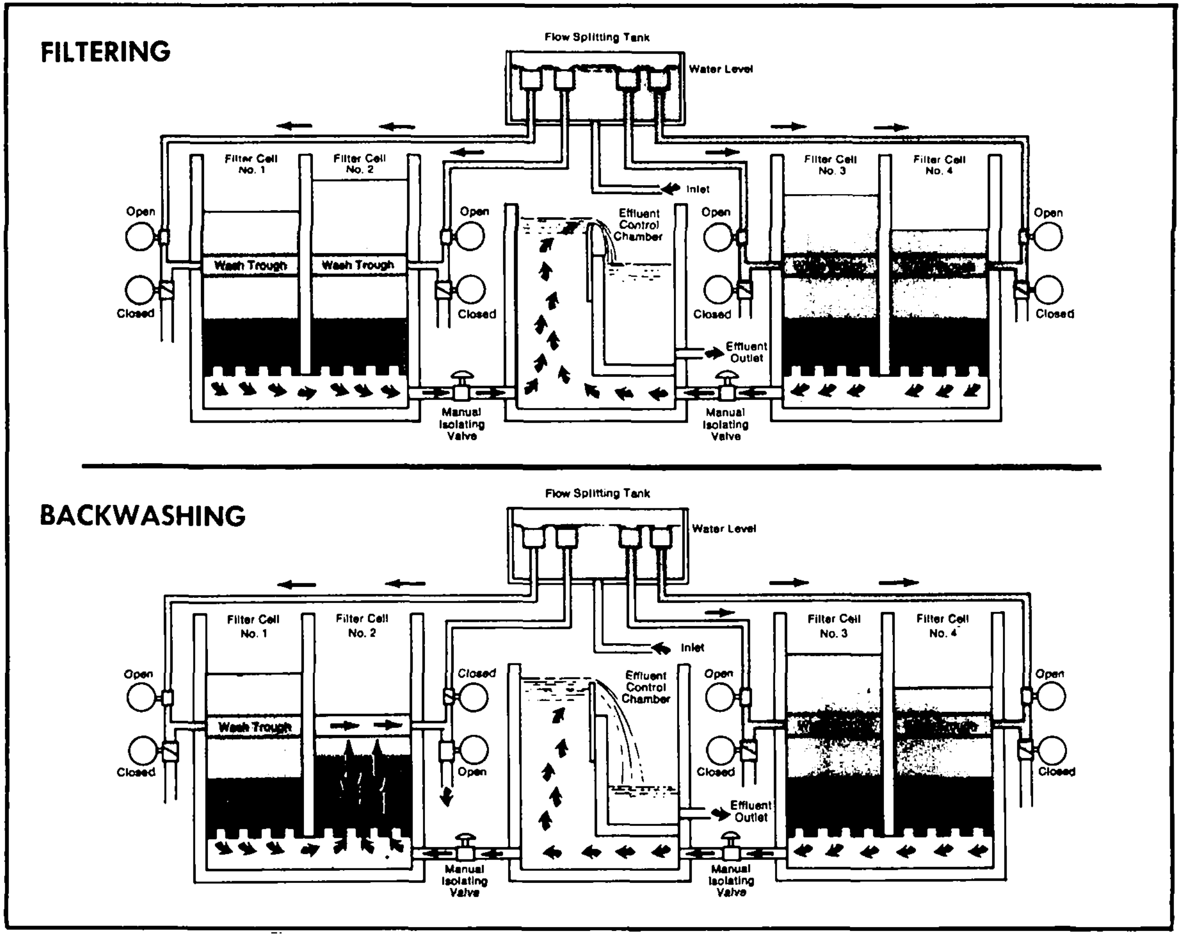
- possible overflow due to simultaneous clogging of many filters

Therefore, use of another filtration control system in developing countries may be considered.

A rising level self-backwash type filter with equal hydraulic loading to each filter on-line is an ideal type of filtration system for developing nations. This filtration system is reliable in performance, simple, and inexpensive to operate. A schematic of the operation of this type of filter is given in Figure 1.

Basically, the rising level self-backwash type filter is a constant rate filtration system without any mechanical equipment. An influent flow splitting device, such as a weir, provides equal hydraulic loading to each filter online. An effluent control weir in the common filter effluent channel, rather than an individual flow control valve on each filter effluent line (a valve is provided for isolation purposes only), is provided to control filtration rates. The effluent weir hydraulic gradient is usually maintained 0.45 to one meter above the lip of the filter troughs in each filter cell.

At the start of a filter run, the water level in the filter cell is about 0.3 meters above the water level in the effluent control chamber prior to flowing over the weir. As the filter bed clogs, the water level rises until it reaches a predetermined elevation for backwash. The backwash is achieved by using filtered water from other operating filters. No pumps or elevated wash tanks are required. However, a minimum number of filters are needed to provide sufficient water for backwash. As a general rule, four filters are required unless small sized pumps supplement the backwash water with treated water from a clearwell. The required number of filters may be determined based on:



Schematic of a Rising Level Self-Backwash Filter

Figure 1

(1) Minimum plant flow rate

(2) Design filtration rate, and

(3) Quantity of filtered water in the clearwell or distribution system

Another important item in filter design is selection of an auxiliary scour wash system to aid in backwash. Without auxiliary scour, a filter bed is susceptible to mud balls soon after the filter is back in service. Air scouring and surface wash are two basic types of auxiliary scour systems. A fixed grid type surface wash system may be best suited for developing nations because of its simplicity in design and its lack of moving parts. However, the rotating arm type surface wash system may also be used.

Filter underdrains are also an important consideration in filter design. Typically, however, the auxiliary scour system selected dictates the type of underdrain system used. Nevertheless, a simple, durable, and reliable underdrain system should always be chosen. Precast concrete laterals or strong polyvinyl chloride pipe laterals meet the above criteria and can be used in conjunction with a fixed grid surface wash system.

3. Conclusion

Two early steps in design of water treatment plants are evaluation of raw water characteristics and selection of the unit processes to be employed. Depending on the quality of the raw water source, conventional treatment, direct filtration, or in-line filtration can be adopted. However, conventional treatment is recommended for use in developing countries because of the fail-safe nature of this treatment scheme.

Detailed design of water treatment plants in developing nations requires special consideration of many factors. Paramount among these factors is the availability of capital, raw materials, equipment, chemicals, and skilled labor. Proper evaluation of these factors can lead to construction of a highly efficient facility that requires little operation and maintenance.

It is the author's hope that this article provides practical and low cost design guides for the design of water treatment plants in developing countries.

CONCEPTION DE LA STATION DE TRAITEMENT DE L'EAU

par Dr. Susumu Kawamura

Résumé

Enumérant les composantes principales d'une installation de traitement de l'eau en pays de développement, ainsi que les facteurs les plus importants pouvant influencer sa conception, l'auteur propose un certain nombre de recommandations :

- simplicité de la conception et de l'entretien qui doivent permettre l'emploi de la main d'oeuvre locale et des matériaux disponibles sur place .

Paper 13

The roles of upflow filtration and hydraulic flocculation in low-cost water technology

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SUMMARY

There are three levels of technology required for water treatment in less developed countries: for rural, urban and metropolitan applications. The problems of successfully devising and operating water treatment units for small and isolated communities in developed countries are almost the same as for small but urban communities in less developed countries. The problems include: ease of access for maintenance and supply of chemicals; adequate attendance by unskilled personnel; robustness, reliability and simplicity of plant and equipment, and availability and reliability of electricity.

A difference between developed and less developed countries is that the latter tend to be in the tropical zones of the world. Therefore the nature of the material to be removed by water treatment can be different or more extreme. However, regardless of this, upflow filtration has been found to be an effective process for treating these kinds of waters at all technological levels.

There are various types of upflow filters appropriate for different applications. These include, filters with coarse or fine media, filters operated with constant or declining filtration rate and automatic downwash coarse bed filters. Variable declining filtration rate control might be particularly attractive for some applications such as where surges in influent quality and quantity might be a problem. Some of the other attractions of upflow filtration in the tropics can include: low cost, low media requirement, adaptability and low operating headloss. The breadth of application ranges from a shallow bed of pebbles when used as a simple strainer or as a hydraulic flocculator preceding lamella sedimentation to a filter with carefully selected wide or narrow size range media to be used with coagulation either on its own or in conjunction with another clarification or treatment process.

An upflow filter with a bed of only coarse pebbles used as a hydraulic flocculator is likely to find limited application because of its relatively poor flocculation performance. Hydraulic flocculation using pipe or baffled chamber arrangements are better since their design can be more adequately specified and their performance predicted and easily adjusted. However, beds using smaller media can successfully combine flocculation and filtration, as in contact filtration.

INTRODUCTION

It has been proposed (1) that the design criteria for water treatment plant for less technically and economically developed communites should be:

(i) Maximum hygienic protection, as expected for developed communities.

(ii) Minimum utilisation of equipment, since less developed countries tend to be importers rather than producers of equipment.

(iii) Maximum use of local materials and the labour force within the region.

(iv) Slight or no automation, since investment should generate employment opportunities and skilled maintenance is rarely available.

Another criterion which should be included for the majority of countries is:

(v) Optimal use of prime energy resources for construction and operation.

The extent to which these criteria and the simplicity, rather than the sophistication, of the solution will apply will depend on whether the particular situation being considered is rural, urban or metropolitan.

In a metropolitan situation the level of technology which should be used in a less developed country approaches that which is typical in most developed communities regardless of country.

Some countries have concentrated on developing technology at urban levels appropriate for their circumstances. For example in South America (1) a water treatment plant design has evolved which incorporates hydraulic flocculation, lamella sedimentation and 2-valve operated filtration back-washed by gravity. India (2) has also given particular attention to hydraulic flocculation and lamella sedimentation as well as indigenous filter media.

The problems of successfully devising and operating water treatment units for small and isolated communities in developed countries are almost the same as for small but urban communities in less developed countries. The problems include: ease of access for maintenance and supply of chemicals; adequate attendance by unskilled personnel; robustness, reliability and simplicity of the plant and equipment, and availability and reliability of electricity or hydromechanical energy.

In the rural situation in less developed countries water supply and treatment can become a problem very different from the urban and metropolitan situations. It becomes greatly associated with social tradition with the basic task being one of education and the introduction of fundamental concepts of domestic and communal hygiene. This is a prerequisite to the effective introduction of appropriate equipment and techniques for water storage and treatment in addition to appropriate sanitation.

The purpose of this paper is to draw attention specifically to the roles of upflow filtration and hydraulic flocculation in the three levels of water treatment technology.

UPFLOW FILTRATION CONCEPT

Some of the basic types of upflow filters are shown in Fig. 1. In upflow filtration the direction of flow during filtration is usually in the direction of the natural gradation of decreasing size of the filter media.

In practice (3) there is little gradation of media size in the main part of an upflow bed. The usual exception is the coarse support media and commonly a very thin layer of fine material on the top of the bed. This fine material originates from the preparation of the main filter sand and as silt reaching the filter. Thus the theoretical advantage of upflow filtration of filtering through progressively smaller media is only partly achieved in practice. The advantage of a conventional upflow filter over a conventional downflow filter more often arises from the usually deeper bed of filter sand and from the absence of the formation of a mat or other kind of high density deposit at the entry to the filter bed.

When an upflow filter in its fully developed form is to be used at normal or higher filtration rates, 5 to 15 m/h, a grid across and just below the surface of the filter bed, as used in the Immedium filter (3), is necessary. This maintains the structure of the filter bed against the upward forces on the bed which increase as the filtration run progresses. This in turn ensures filter run length and filtration efficiency. As an alternative some designs, as widely applied in Eastern Europe (4), like the biflow or AKX filter, use a collector placed lower down than the grid. In these designs the part of the filter bed above the collector is used as a downflow filter to provide a counteracting downward force rather than a resistance.

With some types of upflow filter the only features which can be unattractive are the backwashing when it is in the same direction of filtration and that under certain loads quality breakthrough can be unpredictable (3). Although these might discourage its use for terminal filtration in high technology situations they should be of no concern for all other applications. When coarse media only is used in upflow filters, adequate cleaning of the filter bed can be achieved by just a rapid draindown, or downwash.

The ultimate efficiency of an upflow filter is dependent on the same factors as a downflow filter such as filtration rate, nature of the particles to be removed and the nature of the filter media. The last factor governs in what way and how much suspended material is entrapped within and subsequently removed from the filter bed. In general, suspended particle removal is improved by decreasing filtration rate, decreasing media size, increasing media depth, increasing suspended particle size and decreasing suspended particle concentration. These factors can be illustrated by the results of work by the WRC (3) which included operating an upflow filter in comparison with an anthracite-sand downflow filter, as shown by Figs 2, 3 and 4. Similar results have been produced in other countries (5,6,7).

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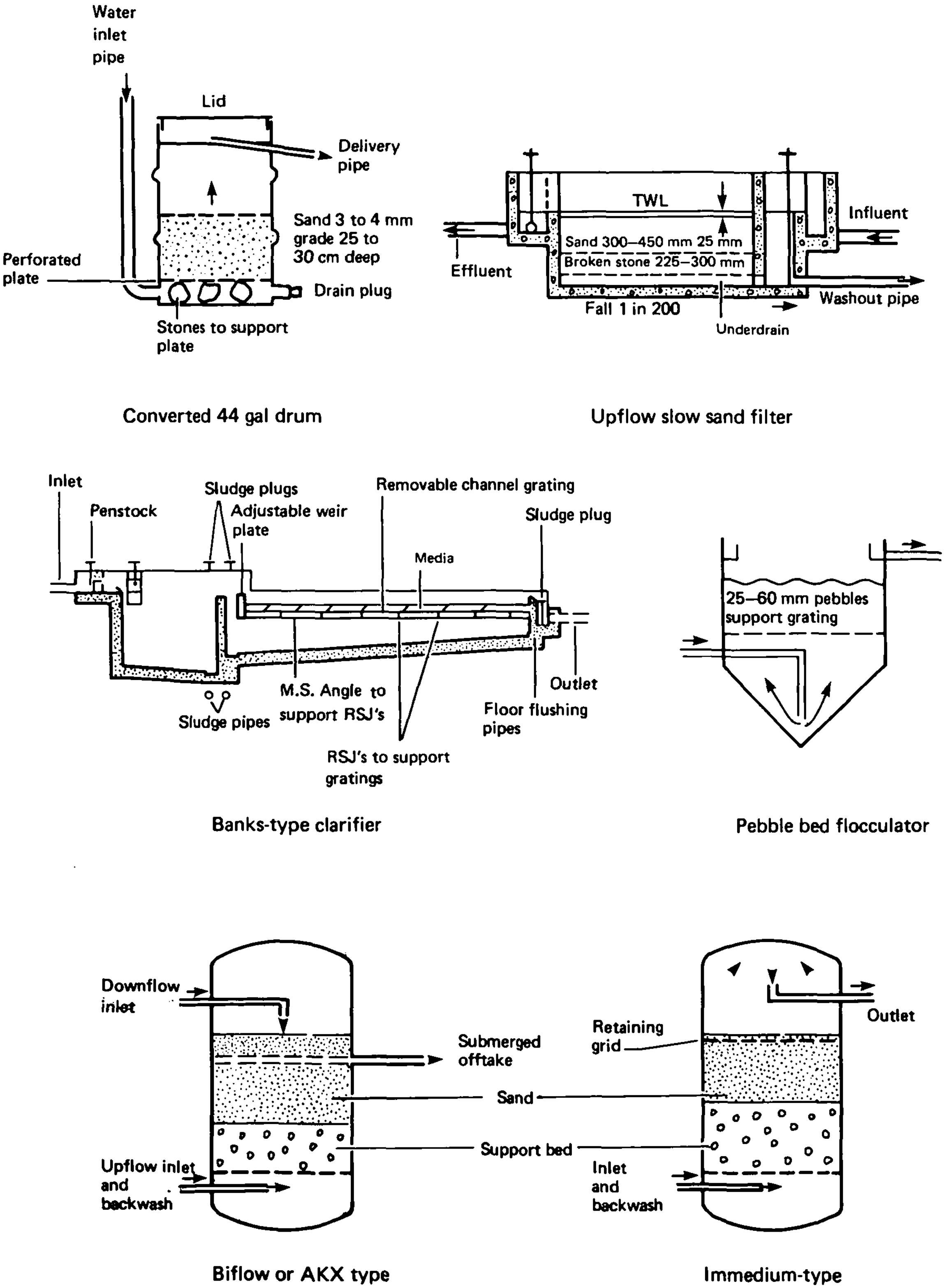

Fig. 1. Types of upflow filters

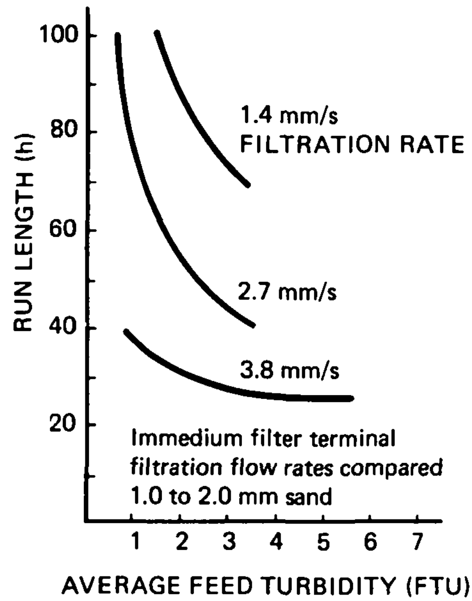

Fig. 2. Effect of feed turbidity on run length for filtration rates shown

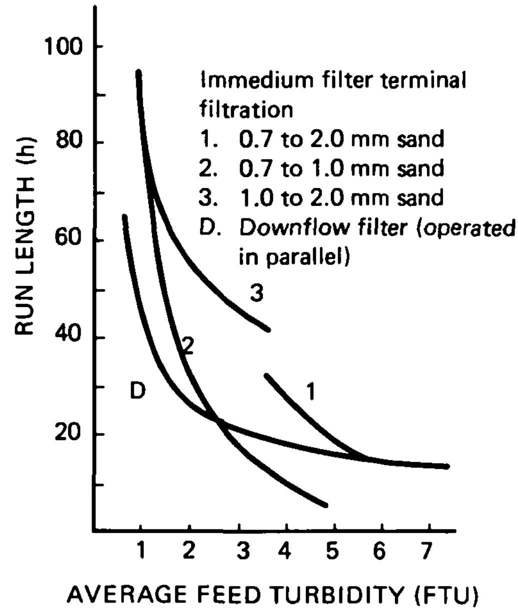

Fig. 3. Effect of feed turbidity on run length for filtration rate of 2.7 mm/s

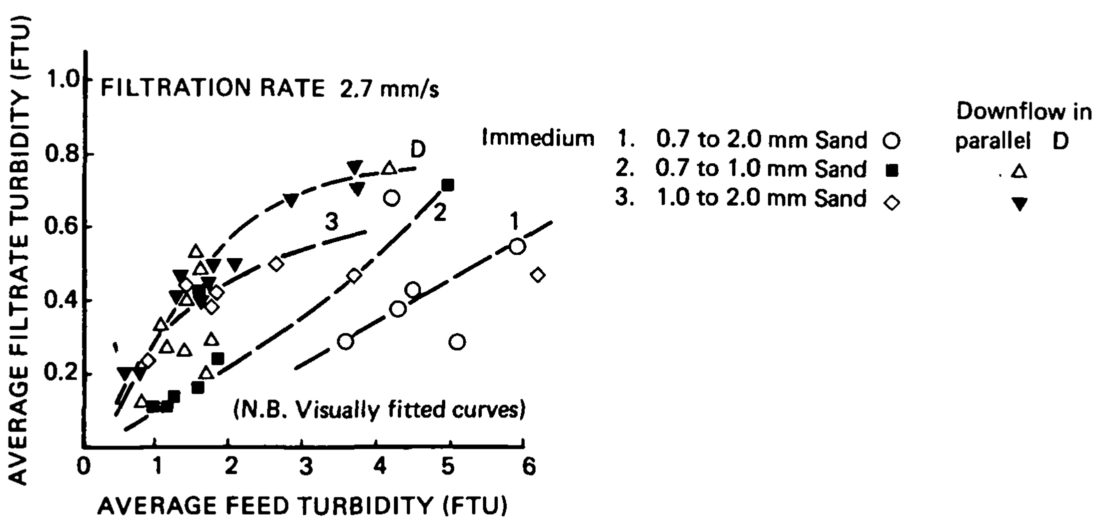

Fig. 4. Effect of feed turbidity on filtrate turbidity for terminal filtration

In practice there will be a compromise between removal efficiency and utilisation of the filter. Thus with appropriate attention (3) to economic filtration rates and media size, an upflow filter can produce longer filtration runs, better filtrate quality and develop less headloss than a conventional downflow filter.

EFFECT OF UPFLOW FILTRATION

The main purpose of introducing a water treatment process where there was previously none, as in a developing community, is usually to remove suspended matter. The value of this is not so much in the improvement of aesthetic quality but mostly in the reduction of health risk which is very approximately in proportion to the reduction of suspended matter.

In tropical waters suspended matter tends to be found in a wider range than is likely in temperate waters of developed countries. The nature of suspended matter and the effectiveness of upflow filtration in their removal is summarised in Table 1.

Most tropical waters contain material in more than one group. Therefore it may not be always adequate to use just upflow filtration. In such circumstances, upflow filtration can form a useful component in a multi-stage treatment system. For example, even if upflow filtration alone cannot adequately clarify a water it might substantially relieve the load on slow sand or other filters and on disinfection requirements.

Table 1. Effectiveness of upflow filtration in removing suspended matter

Suspended matter in tropical waters	Effect of upflow filtration
a. Coarse particulate organic matter including vegetable debris, algae, large free swimming organisms may all be present in tropical lakes and rivers in concentrations upto and exceeding 250 mg/l.	Very substantial removal with high filtration rates and a coarse filter media. Backwashing is very easy.
b. Coarse inorganic soil solids are common in rivers and streams but rare in lake waters. Concentrations can be relatively low at normal flow rates but can be very high following rain.	Very substantial removal, as (a), but generally advantageous to pretreat by sedimentation in order to extend filter run length.
c. Bacteria, parasite eggs and other organisms of faecal origin are common in rivers and lakes because of runoff. They may also be significant in wells and boreholes because of runoff and airborne pollution.	Up to 50% removal can be achieved without difficulty using lower filtration rates and a finer media than for (a) and (b). This in conjunction with the removal of other organic matter will reduce the chlorination load.

Table 1 continued.

d.	Colloidal and semi-colloidal soil particles mostly derived from runoff can exceed concentrations of 2000 mg/l in rivers after rain. Some boreholes and springs in E. Africa have up to 160 mg/l semi-colloidal SiO_2 present.	Colloidal material is very difficult to remove by filtration alone. But, if coagulant is dosed immediately before the filter without a flocculation tank, filtration can be in the same range as rapid gravity filtration. Backwashing needs careful attention.
e.	Small motile organisms, commonly motile algae and certain dangerous parasites such as cercaria of bilharzia.	Motile organisms are difficult to retain in granular filters. Bilharzia control must include slow sand filters and chlorination with adequate contact time or very long retention.
f.	Precipitated iron compounds, which can be in surface and groundwaters in concentrations upto about 25 mg/l.	After cascade or trickling filter most of the precipitated iron oxides can be removed by filtration. However, precipitation is not successful in some cases.

TYPES OF UPFLOW FILTERS

(a) The simplest form of upflow filter using a shallow layer of coarse media in an oil drum (8) can be expected to be no better than a good strainer for the removal of coarse inorganic and organic matter. About 50 per cent of coarse particulate matter can be removed by 0.15 m of 8 to 10 mm pea gravel. Accumulated matter is removed by quickly draining the filter. Increasing the bed depth to about 0.3 m and decreasing media size to about 3 to 4 mm will give more effective filtration in the range 12 to 36 m^3/m^2d and is the limit for using just rapid draining to clean the bed as a daily routine.

(b) Upflow filters in their more developed form will include deep beds of graded media of about 0.5 to 2 mm on top of supporting layers of coarser sand and gravels with facilities to clean the filter bed by a conventional backwash procedure starting with a rapid draindown. Sand with a size range of 0.7 to 2.0 mm to a depth of 1 to 1.5 m with efficient backwashing will produce a noticeable sand size gradation whereas a sand with a size of 0.7 to 1.4 or 1.0 to 2.0 will have negligible gradation (3). The finer at least some of the media is the more effective the filtration will be at removing not only coarse matter but also precipitated iron and destabilised, coagulated colloidal particles.

The upflow backwash rate must be adequate to expand the bed so that it is at least fluidised enough to put a stout pole, such as a broom handle, down into the bed to the top of the supporting coarse sand or fine gravel and stir it around. But this does not have to be part of the backwash procedure.

(c) One of the problems of upflow and rapid gravity filters is maintaining filtered water quality with fluctuating filtration rate or raw water quality. A means of overcoming this is to use declining rate filtration (1,9). When there is only one to three filters this means using constant declining rate on each filter, or when there are at least four filters using variable declining filtration rate. Although most work, including that carried out by WRC on declining rate operation relates to downflow filtration, the general principles are also applicable to upflow filters; there is a bank of four variable declining rate Immedium-type upflow filters at the Fobney Treatment Works of Thames Water Authority, England.

Figure 5 illustrates the fundamental differences between constant rate, constant declining rate and variable declining rate filtration.

Constant declining rate means the filtration rate is governed directly by the resistance to flow through just the one filter. Thus to ensure there is enough filtered water available at all times there must be a large enough filtered water storage tank. The size of this tank can be reduced when there are several constant declining rate filters and the operation of each filter is phased so that their combined output reflects water demand.

Variable declining rate occurs when the feed to all the filters is arranged to cause the same inlet water head on each filter. The result is that the proportion of the total flow to all the filters that is filtered by an individual filter is in inverse proportion to the headloss that has developed through that filter relative to the other filters. Thus the cleanest filter has the highest filtration rate and the dirtiest filter the lowest filtration rate. Further, when there is an increase in the quantity or quality of the load to the filters then this is taken up by each filter in proportion to its relative ability.

For variable declining rate filtration it is important that the backwashing routine is adhered to such that when the inlet head reaches a predetermined level only the longest operated filter must be washed.

A number of advantages have been claimed (1,9,10) for variable declining rate filtration relating to simplicity of construction and operation, better filtrate quality, coping better with flow surges, more filtered water per backwash and lower capital and operating costs.

(d) The need for satisfactory and appropriately timed backwashing of filters has already been mentioned. In coarse media upflow filters, when just a rapid draindown might be adequate for most or all wash occasions, the necessary frequency of such downwashing could be achieved automatically by a simple syphon arrangement. The syphon on the inlet to the filter is initiated when the inlet head reaches a specific level. The discharge of the syphon should be at least 4 times the inlet rate to ensure a rapid draindown.

(e) When coagulation is used with upflow filtration, the coagulant can be added before entry to the filter media to provide delay time that is more than perhaps adequate for flash mixing thus allowing some floc growth to occur before filtration. Alternatively the delay time is sometimes minimised, as in contact filtration (11) to ensure virtually all flocculation takes

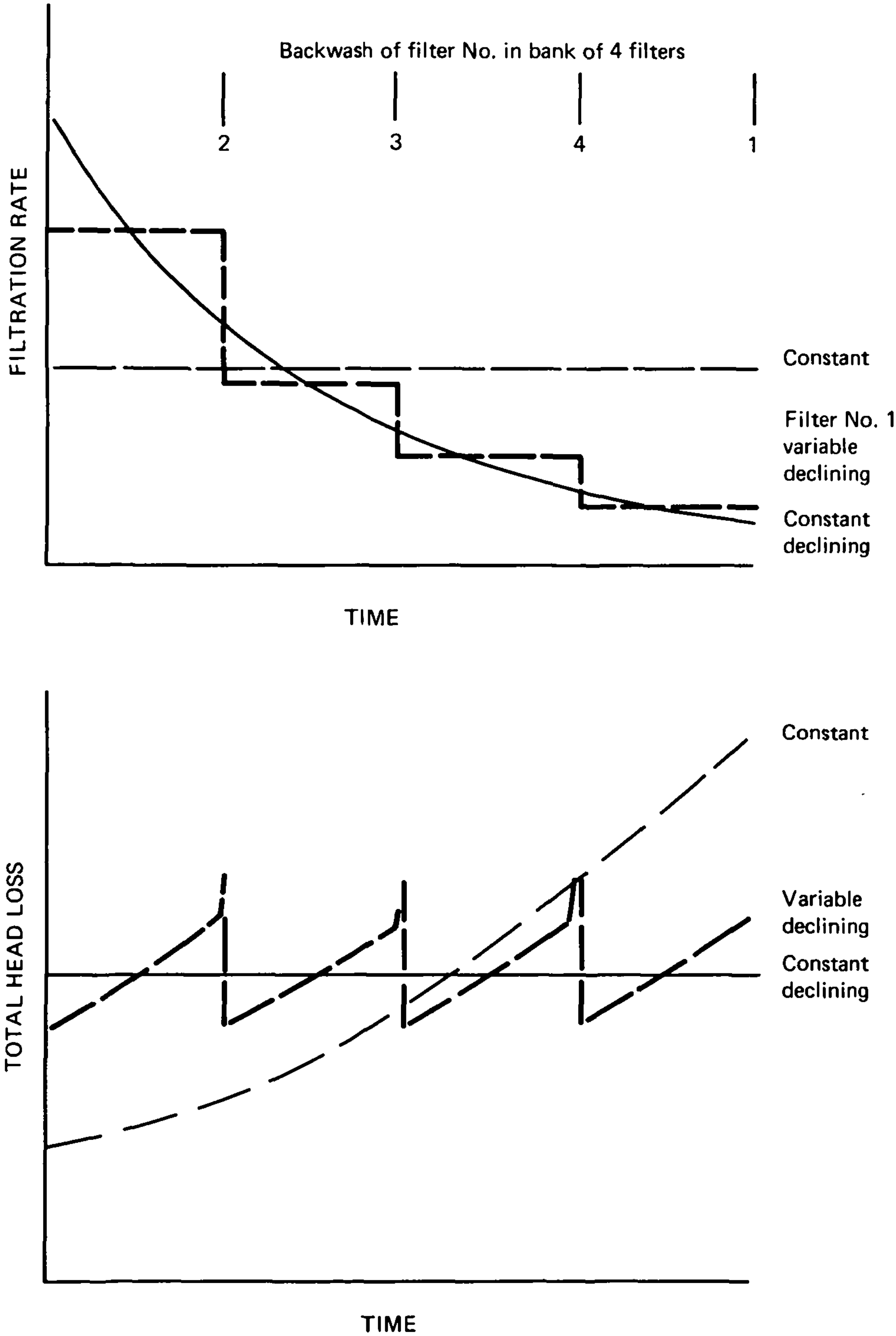

Fig. 5. Comparison of constant and declining rate filtration

place within and at the surface of the filter media. An extension of this approach is to use coarse media so as to still use the upflow filter for hydraulic flocculation but not necessarily filtration. A Banks-type clarifier as used in effluent polishing (12) is intended to achieve both, with pea gravel, 6 to 25 mm and depth 0.1 to 0.3 m. A coarse pebble bed (2), 25 to 60 mm and depth about 2.5 m, would be only for hydraulic flocculation.

ADVANTAGES OF UPFLOW FILTERS

There are particular advantages of upflow filters for the tropics as well as temperate zones.

Low cost

The necessary structures can be relatively small and simple in comparison to the alternatives. The smallest practical unit can be constructed at low cost from a 40 gal drum (8). With just a daily simple downwash it has no major operating cost other than for any associated pumping.

Low requirement of filter media

The quantity of filter media required is very much less than for slow sand or horizontal sand (8) filters. The grade of media need not be important because the depth of media and filtration rate can be adjusted accordingly. Depending upon objectives a functional wide size range media can have advantages both when grading of the media does or does not occur through its depth.

If sand is not available then indigenous substitutes can be used. Examples are broken brick as in Bangladesh, broken charcoal but with a retaining grid as in E. Africa or coir and other fibres as in S.E. Asia, though fibre materials have limited but adequate lives.

Adaptability

In addition to variations in structure and media other modifications can be made.

(i) A layer of limestone in the filter bed can raise the pH of acid waters enough to avoid the need for preparing and dosing lime solution.

(ii) The clearwater zone above the filter bed can be used for clear-water storage and incorporate the disinfection when using a simple chlorine feeding device.

(iii) Coagulation can be easily incorporated by using a simple solution feeding arrangement to the water inlet. This would allow upflow filtration to be used successfully for certain variable quality raw waters when the coagulation may only be needed periodically, such as after intense rainfall or during prolonged drought.

Low operating headloss

Coarse media filters will develop very little headloss. A bank of variable declining rate filters will have less variation and therefore a lower ultimate headloss than constant rate filters, Fig. 5.

Single stage treatment

A high quality drinking water can be produced providing the load due to coagulant and raw water solids is not too great, Fig. 4, or variable, such as when treating stored raw waters (5,7) or slow flowing rivers (3,6). If a coagulant concentration in terms of iron or aluminium has to exceed about 2.0 mg/l then a cationic polymer can be used instead (7).

UPFLOW FILTRATION APPLICATIONS

Upflow filters can be used for various applications (4) besides those already mentioned. The following considers the extent of the range of applications.

Single stage filtration without coagulation

(i) Depending on media and suspended particle sizes, between 40 to 70 per cent of suspended solids can be removed without coagulation.

(ii) High quality water can be produced when only precipitated iron has to be removed.

Single stage filtration with coagulation

(i) Providing the load due to coagulant and raw water solids is not too great, so that iron or aluminium concentrations are less than about 2.0 mg/l, or a cationic polymer is used, a high quality drinking water can be produced, when using sand about 0.5 to 2 mm.

(ii) Providing adequate attention is given to coarse straining and coagulant dose selection, this can be used for simple emergency water treatment units, when using sand about 0.5 to 2 mm.

(iii) In less developed situations an adequate water can be produced from poor sources by dosing the coagulant into a coarse sand or fine pebble bed, about 3 to 4 mm, to achieve what is sometimes regarded as contact flocculation and filtration.

Filtration without coagulation preceding other treatment

In this application it is used basically as a coarse filter or fine strainer but is useful for:

(i) Reducing solids loads going onto slow sand filters, as at Layer-de-la-Haye, England.

(ii) Reducing solids loads and coagulant requirements for downflow filters (3).

(iii) As the only pretreatment for reverse osmosis with tubular membranes, used successfully by WRC.

Filtration with coagulation preceding other treatment

(i) To protect slow sand filters from colloidal and semi-colloidal particles, as at Fobney, England.

(ii) To reduce the fouling rate of reverse osmosis membranes.

(iii) To produce in conjunction with rapid gravity filtration a high quality drinking water when one stage of filtration is inadequate to cope with the solids load; this should be very suitable for treating stored raw waters requiring high yet relatively constant coagulant doses.

(iv) To act as a hydraulic flocculator when using a shallow bed of large gravel or pebbles, although some solid-liquid separation will occur as in a Banks-type clarifier as used in effluent polishing (12).

Filtration following other treatment (terminal filtration)

(i) Slow upflow filtration following sedimentation without coagulation (13) can be useful for small isolated treatment works for the reduction of coarse particulate matter or high precipitated iron concentrations.

(ii) When intensive aeration is not suitable yet some pH adjustment is feasible and necessary, sedimentation preceding filtration without coagulation is more a means of providing time for reactions rather than settlement to occur.

(iii) Sedimentation or filtration without coagulation preceding upflow filtration with coagulation could be useful for producing high quality water from high turbidity raw water with the minimum of coagulant.

(iv) Clarification with coagulation preceding filtration is necessary when coagulant doses can be too great and fluctuate too much for filtration alone to cope; it was for this reason that a dissolved air flotation plant was added for preliminary clarification before upflow filtration at Glendye, Scotland (14).

HYDRAULIC FLOCCULATION

In hydraulic flocculation the turbulence required for flocculation is provided by the water flowing through the system. For a given design and a given flow-rate the energy input to create the turbulence can not be easily changed to adjust for changing raw water conditions, although in some designs this is possible (1).

Besides the energy input, the flocculation time is also an important parameter for efficient flocculation. In general, increasing the flocculation time increases the flocculation effect. Therefore, if the flow through a hydraulic flocculator is reduced, the energy input and with it the turbulence created is decreased but at the same time the flocculation time is increased. Within certain limits the two influences on flocculation cancel each other out.

Practical experiments by the WRC using a pipe flocculator and a baffled tank flocculator have shown that a hydraulic flocculator can be operated over a wide range of flow rates with little change in flocculation performance.

The advantage of hydraulic flocculation is that mechanical agitation units which are usually very inefficient in electrical energy usage and also require maintenance are not required. In hydraulic flocculation no electrical energy is required provided a head of water between 0.5 and 1.0 m is available. On the other hand, if the raw water has to be pumped it has only to be pumped an extra 0.5 to 1.0 m to overcome the energy lost in the hydraulic flocculator.

Investigations by the WRC (15) have shown that provided the hydraulic flocculator is designed correctly, flocculation equivalent to that of a multi-stage, 3 to 4 stages, mechanical flocculator can be achieved with approximately half the flocculation time. Figure 6 illustrates results of a comparison between hydraulic and mechanical flocculation. However, a cheap and simple large scale design has yet to be proven. Whereas mechanical flocculation is usually carried out in open channel flow, compact open channel flow is less attractive for hydraulic flocculation because of the headloss of 0.5 to 1.0 m which has to be taken into account when designing the channel.

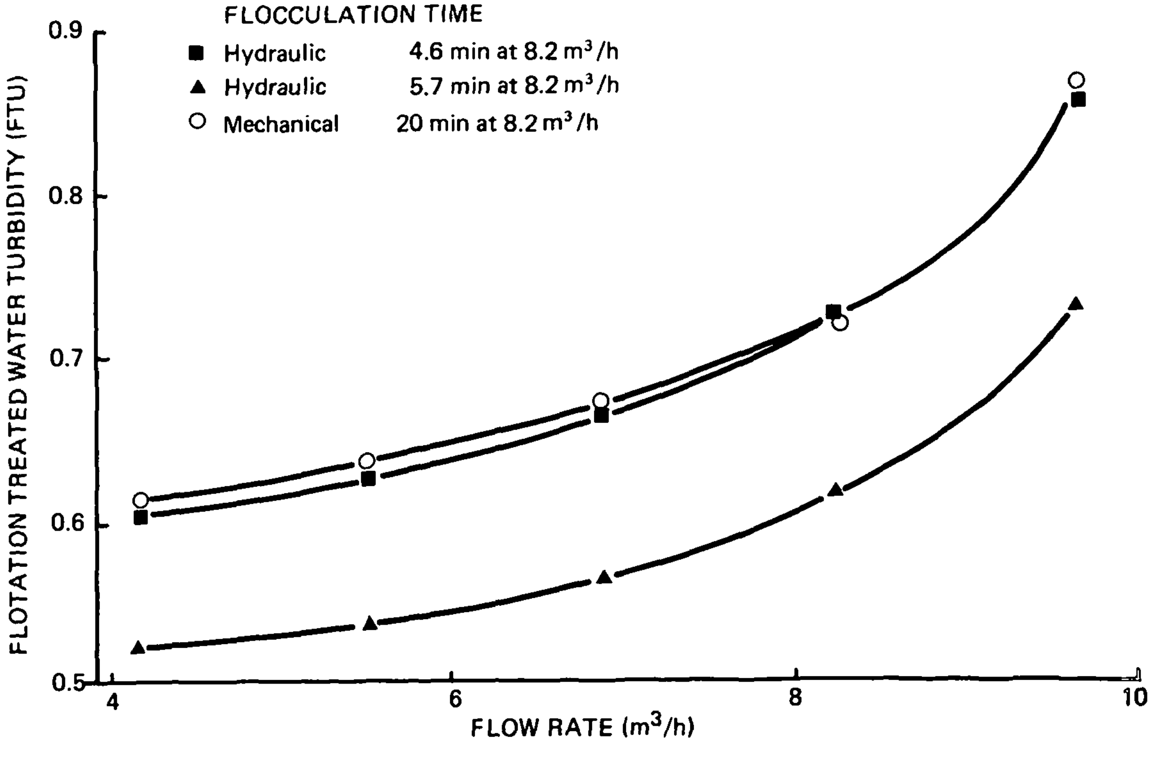

Fig. 6. Comparison of mechanical and hydraulic flocculation used on a dissolved air flotation plant

The WRC investigations using pipe and baffled tank hydraulic flocculators have shown that $G \times t$ values of about 50 000 are required for optimum flocculation for both mechanical and hydraulic flocculation preceding dissolved air flotation as the solid-liquid separation process, for raw water conditions encountered in the UK. (G is the mean velocity gradient in sec^{-1} and t is the flocculation time in sec.)

For the hydraulic flocculator it was found that as design parameters a bulk velocity of about 0.5 m per sec, a flocculation time of about 5 mins and a mean velocity gradient G of between 150 and 200 sec^{-1} would result in optimum flocculation for dissolved air flotation. If sedimentation is used as the solid-liquid separation process the aim would be to form larger flocs to increase their settling velocity. For this application a lower G value and longer flocculation time would be more suitable. If upflow filtration is used as the solid-liquid separation stage a higher G value and probably a shorter flocculation time would produce the most suitable floc.

Hydraulic flocculation using a packed column, of 25 mm Pall rings, gave unsatisfactory flocculation for dissolved air flotation. Therefore, coarse pebble bed flocculators (2) would seem to have only limited application, as distinct from fine pebble bed Banks-type clarifiers and contact filters.

CONCLUSIONS

(i) There is a wide range of applications for upflow filtration at all levels of technology in both developed and developing countries. Greater use of upflow filtration can therefore be expected.

(ii) Because of design and operational problems upflow filtration is least likely to be used for final filtration when a high performance is required.

(iii) An upflow filter with a shallow bed of very coarse media may also be regarded as a hydraulic flocculator. However, better hydraulic flocculation can be achieved, using pipe or baffled tank designs, which can be as good as mechanical flocculation.

(iv) Hydraulic flocculators can operate over a wide range of throughputs with little need to make other adjustments excepting the coagulant dose.

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LES ROLES DE LA FILTRATION DE SURECOULEMENT ET DE LA FLOCCULATION HYDRAULIQUE DANS LA TECHNOLOGIE DU TRAITEMENT DE L'EAU A FAIBLE COUT

par R. Gregory, H.T. Mann et T.F. Zabel

Trois niveaux de technologies existent pour le traitement de l'eau dans les pays moins développés : respectivement pour ces zones rurales, urbaines et metropolitaines. Les problemes de definition et de mise en oeuvre d'installations de traitement de l'eau pour des communautés petites et isolées en pays développés sont sensiblement les mêmes que pour des communautés petites mais urbaines en pays moins développés. Ces problèmes sont notamment : la facilité d'accès pour l'entretien et l'approvisionnement en produits chimiques ; une surveillance adéquate avec un personnel non-spécialisé ; solidité, fiabilité et simplicité des machines et de l'équipement ; disponibilité et fiabilité de sources d'énergie

Une des différences entre les pays développés et les pays moins développés provient du fait que ces derniers ont tendance à se trouver dans les zones tropicales du monde. Donc la nature des matériaux à éliminer par le traitement de l'eau peut être différente ou plus extrême. Dans ce contexte, la filtration s'est révélée être un procédé efficace pour le traitement de ces eaux à tous les niveaux technologiques.

Differents types de filtres de surécoulement conviennent à des applications différentes. On trouve ainsi des filtres gros ou fins, des filtres fonctionnant avec un débit de filtration constant ou décroissant et des filtres automatiques avec déflexion vers le bas par couche grossière. La maitrise des variations du débit de filtration convient particulièrement pour certaines applications comme celles où les changements de la qualité et de la quantité de l'affluent serait un problème. Parmi les autres aspects avantageux de la filtration de surécoulement en pays tropicaux, on peut citer : le bas prix, le peu de matériaux de filtration nécessaire, l'adaptabilité et la baisse minime de pression en cours de fonctionnement. La gamme des applications s'étend de la couche mince de galets utilisée comme simple passoire ou comme flocculateur hydraulique précédant une décantation lamellaire jusqu'au filtre dont le matériau est chosi avec soin dans la gamme des granulométries pour être utilisé après coagulation, isolement, ou en conjonction avec un autre procédé de traitement ou de clarification.

Un filtre à surécoulement avec une simple couche de galets, et utilisé comme flocculateur hydraulique risque d'avoir une possibilité restreinte d'application à cause des faibles possibilités en matière de flocculation. Mieux vaut la flocculation hydraulique utilisant un tuyau ou une écluse avec chicane, de conception plus aisée, et dont les performances sont mieux controlables. Pourtant, des couches utilisant des matériaux plus fins peuvent intégrer avec succès flocculation et filtration, comme c'est le cas pour la filtration par contact.

- prise en compte des contraintes économiques, des politiques gouvernementales, des coutumes et des traditions.

- prise en compte d'une expansion possible du système dans le contexte de la région et limitation au minimum des importations de produits chimiques ou autres.

Au plan technique, il recommande :

- le caniveau Parshall pour la mesure des débits ;

- doseur de chlore ou de sulphate d'aluminium du type roue rotative à godets ;

- mélangeur flash avec ressort hydraulique en aval du caniveau Parshall ou du type pompe à injection ;

- flocculation par mélangeurs mécaniques dans des réservoirs ou des rigoles avec chicanes ;

- procédé de sédimentation avec débit horizontal car plus sûr quand les conditions sont variables ou incontrôlables ;

- filtration rapide ou filtration avec rincage automatique etc.

Une étude détaillée des critères et des choix possibles permet la construction d'installations efficaces d'un entretien simple avec une technologie à faible coût.

Paper 14

Development of Simple and Economic Filtration Methods for Rural Water Supplies

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1. INTRODUCTION.

1.1 The Urgent Need.

Number of water treatment plants are likely to be constructed for the supply of potable water to the villages in the developing countries during the next decade. As the conventional water treatment methods are fairly costly in their construction and maintenance, an intensive search is going all over the world for the development of low cost water treatment methods with the use of new techniques.

Even though the slow sand filters are recommended for village water supplies, there are some problems in providing these filters for turbid raw water sources. With the provision of storage basins, coagulation and sedimentation before slow sand filters, the cost of construction and maintenance goes very high. In the case of rapid gravity filters, due to various mechanical arrangements adopted for mixing, flocculation and clarification in the pretreatment, these filters also become costly, particularly for the small units for villages. In the case of pressure filters there is difficulty in obtaining reliable pressure pretreatment units for treatment of turbid waters. Further there are maintenance problems for all these treatment methods in the small capacity plants. Hence the development of simple and economic filtration methods for the small capacity rural plants has become an urgent need.

1.2 Development of New Methods.

Considering the various problems in the design and construction of the conventional treatment plants, the author has developed three new designs for the construction of simplified water treatment plants in his Ph.D. thesis (1978), which are explained in this paper. For the treatment of low turbidity water sources, Ramtek plant has been designed, while for the treatment of turbid water sources Varangaon and Chandori plants have been developed. The principal design criteria for these new methods are given in Table 1.

2. RAMTEK PLANT.

2.1 Design and Construction.

The plant was constructed in 1973 for a population of

0.02 million with a capacity of 2.4 mld. The source of supply is an irrigation tank and the average raw water turbidity is within 10 to 20 J.T.U. with an occasional turbidity of 300 to 500 J.T.U. The treatment plant is designed for an hourly pumping rate of 0.1 mlph, which supplies intermittent water supply to the town. The plant comprises of two separate units and each unit consists of one gravel bed prefilter chamber followed by one dual media filter bed. Both the units are open to sky with a control room on the outlet side. The flow diagram of one unit is shown in Fig. 1, and the photograph of the plant is shown in Fig.2.

2.2 Prefilter Chamber.

Raw water after addition of alum dose and taking through the mixing channel is introduced through the bottom of the prefilter chamber. The dimensions of the prefilter chamber are 2.0 m x 3.5 m with 3.0 m water depth. The under drainage system at the bottom of the chamber consists of one mild steel manifold of size 200 mm x 300 mm, with 50 mm dia side perforated pipe laterals provided at 20 cm centres. Graded gravel of 50 mm to 10 mm dia are placed for a depth of 1.7 m from the bottom to the top. The direction of flow in the prefilter is upward and the surface loading rate is 6.75 mph. The gravel bed provides ideal facilities for flocculation, and the floc formed due to continuous recontacts get consolidated, and when it reaches at the top of the bed, it settles at the top due to sudden drop in the upward velocity. The sludge thus settled at the top of the bed can be drained out through the perforated draining pipe system provided at the top of the gravel bed. This can be operated periodically depending on the raw water turbidity, and the same can be judged from colour of the drained sludge. In addition to this draining facility, the sludge settled in the voids and at the bottom of the gravel bed can be removed by draining out the same by hydrostatic pressure through the under drainage system. Further a back wash can be given periodically with full pressure to clean the gravel bed effectively.

2.3 Dual Media Filter Bed.

Water from the top of the prefilter chamber is introduced on the top of the dual media filter bed and is then filtered in the downward direction. The dimensions of the filter chamber are the same as those given for prefilter chamber, including the under drainage system at the bottom. The dual media consist of the top coarse coconut shell media of average 1 mm to 2 mm size, which is provided for 35 cm depth over the sand bed. The fine sand media having effective size of 0.45 mm and uniformity coefficient of 1.5 is provided for 55 cm depth, over the supporting graded gravel bed of 45 cm depth at the bottom.

The filter bed is cleaned by giving hard wash for 7 to 8 minutes, when the expansion of the filter media is achieved

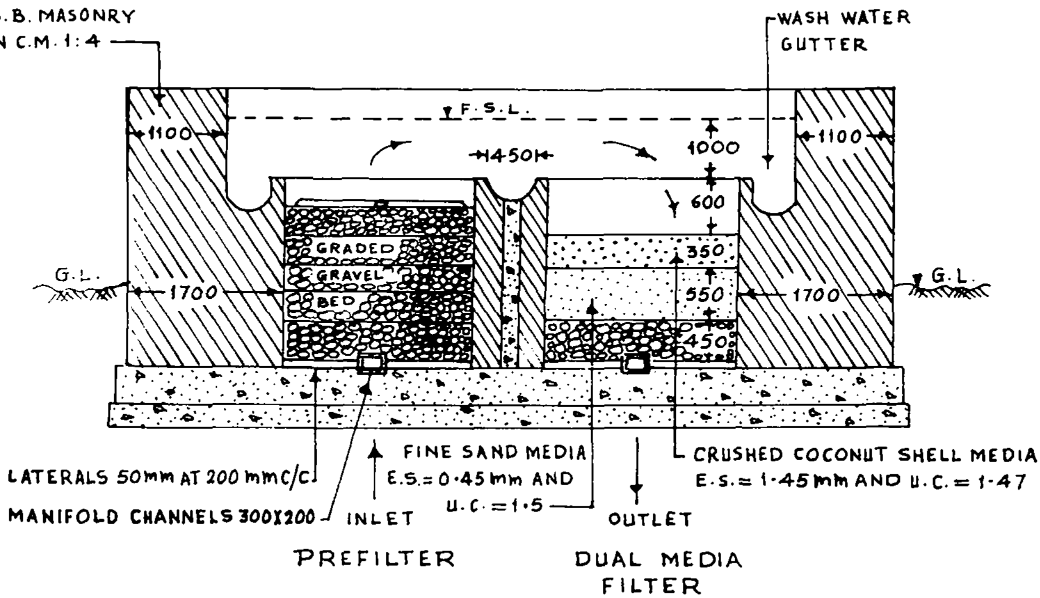

FIG.1. FLOW DIAGRAM OF RAMTEK FILTER.

ALL DIMENSIONS IN MILLIMETRES.

FIG.2. PHOTOGRAPH OF RAMTEK PLANT.

at 30% to 40%. The rate of filtration is 6.75 mph which is controlled by a manually operated sluice valve before a 'V' notch chamber in the control room. A simple gravity chloronome is provided to give the dose of gaseous chlorine in the control chamber.

2.4 Plant Observations.

i) From the actual plant observations the turbidity after prefilter was generally within 20 J.T.U. even when the raw water turbidity was between 300 to 500 J.T.U., with occasional increase upto 1000 J.T.U. The headloss in the prefilter chamber was negligible. The filtered water turbidity was maintained below one J.T.U. during the filter runs for a limiting headloss of two metres. Even though the filter bed was designed for a filteration rate of 6.75 mph, one dual media bed was operated for a higher rate of 9.65 mph for one year. The average length of filter run for this higher rate of filtration was found to be 88 hours for a maximum headloss of two metres, when the plant was operated intermittently for 4 to 8 hours daily. The wash water consumption was less than one percent.

ii) Use of the Crushed Coconut Shell Media.

This media was used for the first time for filtration in the high rate dual media filter beds at Ramtek, and the general performance of this new media was found to be very satisfactory. Even though the media is organic in nature, there is no sign of deterioration of the media after a period of six years of its use in the filter beds at Ramtek.

The specific gravity of the coconut shell media is about 1.43 when it is soaked in the water. Its colour is brownish when dry but turns to black when it is soaked in water. The media is angular, hard and tough and microscopic observations show a compact and uniform structure. Its solubility in 20% HCL is 0.7% in 24 hours, and the durability tests of the media shows about 2.5% loss in weight, when the media was continuously backwashed for 100 hours. During laboratory study with different coal media the coconut shell media was found to be superior. Further the cost of this media may also be cheaper than the other coarse media, as the coconut shell is a waste material and it is mainly used as fuel. Though the coconut shell is available in large quantities in Philipines, Indonesia and India, it may also be available for use in some other coastal countries. However its availability for large scale use may have to be ascertained.

3. VARANGAON PLANT.

3.1 Design and Construction.

The plant was constructed in 1977 for a population of 0.035 million in five villages. The source of supply is the Tapi river and the maximum turbidity during rainy season is

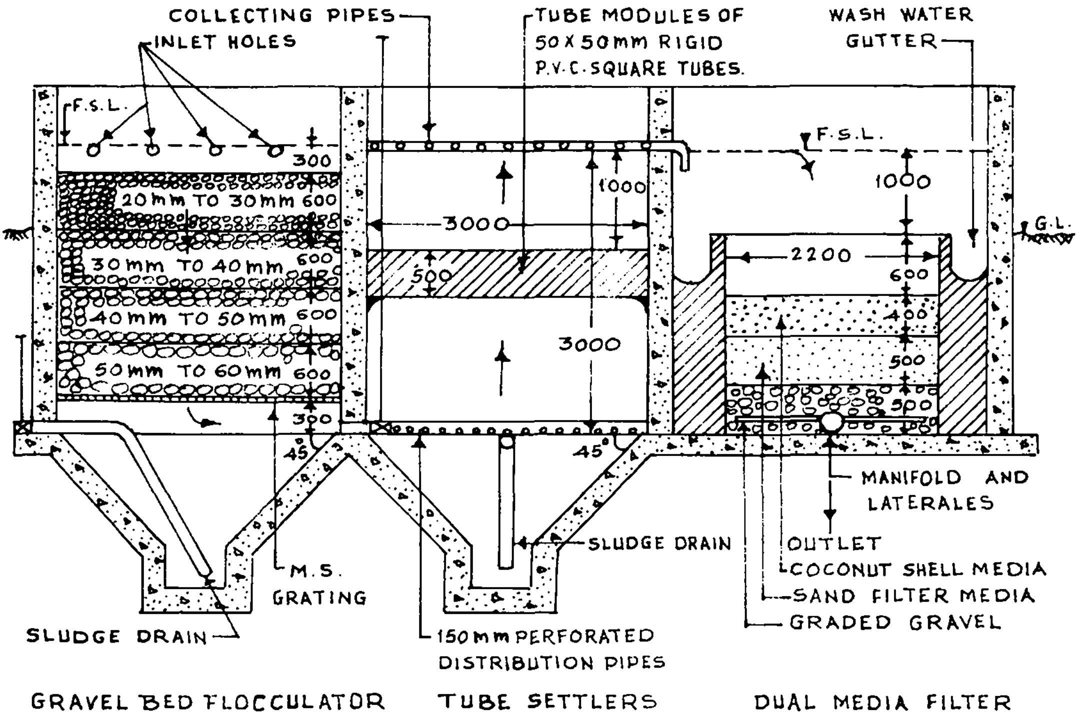

FIG. 3. FLOW DIAGRAM OF VARANGAON TREATMENT PLANT

ALL DIMENSIONS IN MILLIMETRES.

FIG.4. PHOTOGRAPH OF VARANGAON PLANT.

more than 3000 J.T.U., while the average turbidity is between 30 to 50 J.T.U. The plant is designed for a pumping rate of 0.175 mlph with a capacity of 4.2 mld. The plant comprises of two pretreatment units in parallel and each unit consists of one gravel bed flocculator chamber and one tube settling tank, which are followed by three dual media filter beds. Raw water after addition of alum dose is taken through the mixing channel to the top of the gravel flocculator chamber. All the beds are open to sky with a control room on the outlet side. The control room provides the declining type rate control system, with one master control valve before weir chamber. The pure water pumping machinery is also installed in the control room. Alum solution and dosing arrangements are provided on the first floor, while the back wash tank is provided on the top of the chemical room. The flow diagram is shown in Fig.3 and the photograph of the plant is shown in Fig.4.

3.2 Gravel Bed Flocculation Chambers.

The size of each chamber is 3.0 m x 3.0 m with 2.5 m depth of gravel, with 0.3 m water depth on the top. Graded gravel of 60 mm to 20 mm sizes are placed from the bottom to the top on mild steel grating, for supporting the gravel on the top of the hoppers. Two hoppers are provided at the bottom with 45° slopes, to drain out the sludge by hydrostatic pressure. The direction flow is downward and the surface loading rate is 9.7 mph. Arrangements are provided to desludge the gravel bed by draining out water to waste. For cleaning the bed effectively, back wash arrangements are also made, for giving the back wash periodically.

3.3 Tube Settling Tanks.

The flocculated water from the gravel bed is introduced at the bottom of the tube settler and just above the top of the hoppers, through 150 mm dia four perforated distribution pipes. The size of each tank is 3.0 m x 6.0 m with 3.0 m water depth above the top of the hoppers. A layer of 50 mm x 50 mm size rigid PVC square tubes, 60 cm in length is provided to cover all surface area. The modules of PVC tubes were fabricated by fixing the tubes at 60° angle in opposite directions, and were then installed in the tanks. The direction of flow is upward and the flow through rate is 6.6 mph through the open area of the tubes, with detention period of about 35 minutes in the tube settling tanks. The settled water is collected through the perforated collecting pipes of 100 mm dia in the central collecting channel, and is then introduced on the dual media filter beds. Four hoppers are provided with 45° slopes in these tanks, and sludge is drained out by hydrostatic pressure by opening respective sludge valves.

3.4 Dual Media Filter Beds.

There are three filter beds and size of each bed is 4.0 m x 2.2 m with 3.0 m water depth. The under drainage system consists of 300 mm dia mild steel manifold with 50 mm dia PVC pipe perforated laterals placed at 20 cm centres. The filter media consist of 40 cm depth of coarse coconut shell media of average size 1 mm to 2 mm, over the fine sand bed of 50 cm depth. The effective size of fine sand is 0.5 mm with uniformity coefficient of 1.5, and the media is supported by graded gravel bed of 0.5 m depth. Even though the designed rate of filtration is 6.6 mph, two filter beds were operated at a higher rate of 10 mph for one year, which showed satisfactory quality of filtrate. The filter beds are cleaned by hard wash with 30% to 40% of expansion of media for about 8 to 10 minutes. The chlorine dose is given in the control chamber.

3.5 Plant Observations.

The actual plant performance with pretreatment consisting of gravel bed flocculator, followed by tube settling tanks was found to be very satisfactory even for higher turbidities above 3000 J.T.U. The settled water turbidity was generally below 20 J.T.U. while the filtered water turbidity was between 0.5 to 1.0 J.T.U. The average filter run was seen for 40 hours for a maximum headloss of 2 metres. The wash water consumption was about two percent.

4. CHANDORI PLANT

4.1 Design and Construction

The plant was constructed during 1979-80 for a population of 0.015 million of the Chandori village. The source of supply is the Godavari river and the maximum turbidity is more than 3000 J.T.U., while the average turbidity is between 30 to 50 J.T.U. The plant is designed for a pumping rate of 0.04 mlph. The plant comprises of one unit of pretreator followed by one unit of dual media filter bed. Both the units are open to sky with a control room on the outlet side. The pure water pumps are also installed in the control room. The elevated service reservoir is used for giving back wash. The flow diagram is shown in Fig.5 and the photograph of Chandori type plant is shown in Fig.6.

4.2 Pretreator Unit.

Raw water after additinn of alum dose and passing through the mixing channel is introduced through the bottom of the pretreator unit. The dimensions of the pretreator are 4.0 m x 2.2 m with 3.6 m water depth. It is a gravel bed flocculator-cum-tube settler unit. Graded gravel of 60 mm to 20 mm sizes are placed for 1.5 m depth over the under drainage system. The PVC tube settler modules are provided for 0.5 m depth and covering all the surface area over the gravel bed, but keeping a clear spacing of 0.9 m below the tube settler zone. Three 150 mm dia perforated C.I. pipes settled water collectors are provided at 0.6 m above the top of tube settler. The surface

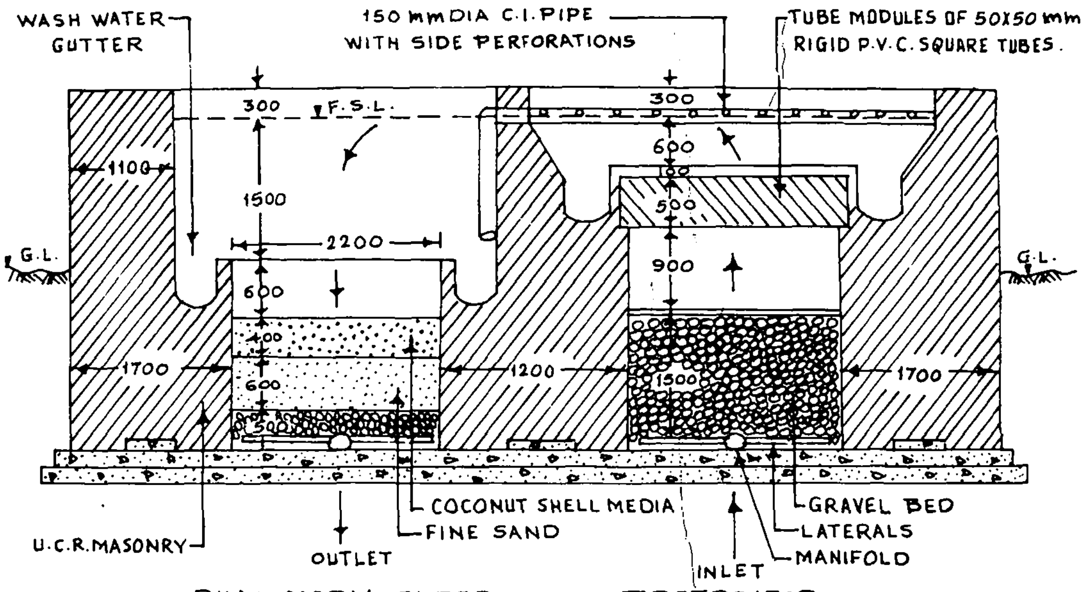

FIG.5. FLOW DIAGRAM OF CHANDORI TREATMENT PLANT

ALL DIMENSIONS IN MILLIMETRES.

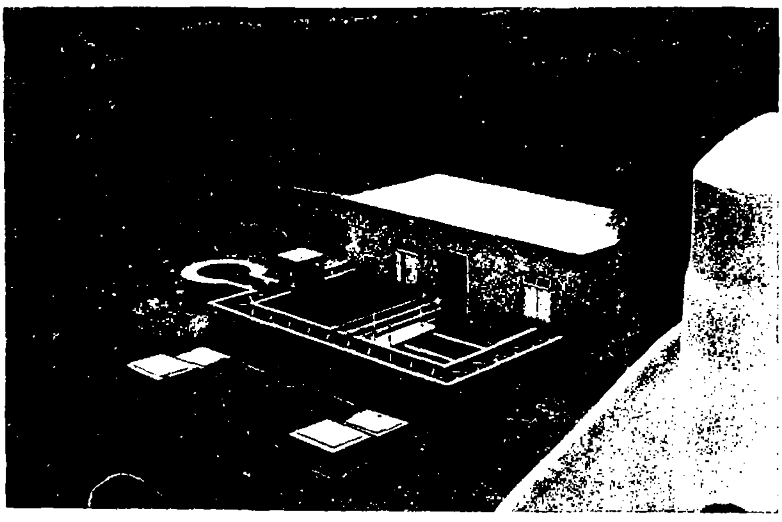

FIG.6. PHOTOGRAPH OF CHANDORI TYPE PLANT.

loading rate on the gravel bed is 4.5 mph, while for the actual tube opening area it is 5.7 mph. The total detention in the pretreator unit is about 45 minutes. The direction of flow is upward, and the raw water after passing through the gravel bed and the tube settling zone is introduced on the filter bed through the collector pipes at the top. The cleaning of the gravel bed is similar to the procedure explained for the prefilter bed in the Ramtek plant.

4.3 Dual Media Filter Bed.

The dimensions of the filter chamber are 4.0 m x 2.2 m with 3.6 m water depth. The details of the dual media filter bed are similar to those given for the Varangaon filter bed. One weir chamber is provided in the control room where chlorine dose is given.

4.4 Plant Observations.

The plant was put into operation from October 1980 and various observations are being conducted. The pilot plant results are satisfactory. The Chandori plant has special advantage for two stage construction. For lower turbidity sources the tube settler in the pretreator and the coconut shell media in the filter bed can be omitted in the first stage. However the same can be introduced at a later stage for augmentation of the plant output, or for obtaining longer filter runs with improvement in the quality of filtrate.

5. SOME COMMON ASPECTS.

5.1 Design Aspects

All these three plants have been constructed in the gravity masonry side walls with RCC roof only on the control room. This type of structure was adopted mainly to utilise the local material and unskilled labour in the villages. This type of massive structure is generally found leakproof. Further there is advantage of providing mixing channels and walkways on the top of the side walls. However, the RCC structure can be cheaper for these designs, if such facilities are available in the villages, All these designs can also be adopted with some modifications for fabrication of package plants, which will be cheaper and will have some more advantages.

All these plants have been found considerably simple for construction and maintenance the main reasons being, the absence of mechanical equipments, plants having compact designs with higher surface loading rates, and these can be built with local material and labour. Even though the dual media filter beds with coconut shell media have been adopted in all these designs, rapid sand bed can also be adopted. The use of gravel bed flocculation and tube settlers in pretreatment and the use of coconut shell media in the dual media filter beds are the new techniques adopted in the development of these small capacity unconventional treatment plants.

5.2 Cost Aspects.

The cost of construction for Ramtek plant was U. S. $

15625/- for Varangaon plant was $ 50,000/-, for Chandori plant was $ 18,750/-, and these costs were between 30% to 50% of the construction costs for the same capacity conventional plants. Table 2 showing the comparative costs of construction of the simplified plants as developed by the author, and the conventional treatment plants is enclosed. The costs of RCC and package plants can still be reduced. Further the over all costs can be reduced if the elevated service reservoirs are utilized for back wash facility.

6. CONCLUSIONS.

Considering the proposed "International Drinking Water Supply and Sanitation Decade" during 1981 to 1990, the provision of safe drinking water supply facilities will have to be provided on the priority basis. The new treatment methods as explained in this paper have shown satisfactory results for number of such plants constructed in the Maharashtra State in India. Table 1 giving the recommended general design criteria based on the actual plant observations, shows that there is considerable flexibility in the design of such small capacity plants for village water supply schemes. The new treatment plants are found simple for construction and maintenance and are also found considerably cheaper as compared to the costs of the conventional plants. It is therefore felt that these new methods may be able to help in solving some of the important problems in providing simple and low cost water treatment plants for adoption in the rural and semi-rural areas, particularly in the developing countries.

7. ACKNOWLEDGEMENTS.

The author is grateful to Mr. P. K. Nagarkar, Director and Chief Engineer, Maharashtra Engineering Research Institute, Nasik, Mr. S. T. Khare, Chief Engineer and Member Secretary, Maharashtra Water Supply and Sewerage Board, Bombay, and Dr. A. G. Bhole, VRCE Nagpur, for giving their valuable encouragement during the Ph.D. study on this work.

TABLE 1: Recommended design criteria for the simplified treatment plants

Design criteria 1	Ramtek plant 2	Varangaon plant 3	Chandori plant 4
I. RAW WATER TURBIDITY.			
i) General Recommendations	For low turbidity sources	For high turbidity sources	For moderate turbidity sources
ii) Average range in J.T.U.	10 to 30	30 to 100	30 to 100
iii) Maximum range in J.T.U.	300 to 500	1000 to 5000	1000 to 2000
II. PRETREATMENT.			
i) Mixing unit	Mixing channel	Mixing channel	Mixing channel
ii) Type of gravel bed units	Prefilter	Flocculator	Pretreator
i) Direction of flow	Upward	Downward	Upward
ii) Surface loading in lph/m^2	4000 to 7000	4000 to 10000	4000 to 8000
iii) Volumetric loading in lph/m^3	2000 to 3500	2000 to 5000	2000 to 4000
iv) Depth of the gravel bed in m	1.5 to 2.0	2.5 to 3.0	1.5 to 2.0
3) Tube settling tank	Not adopted	Tube settler	Gravel bed-cum-tube settler
i) Surface loading in lph/m^2	-	5000to 10000	4000 to 8000
ii) Detention period in minutes	-	30 to 50	30 to 50
iii) Depth of the tank in m.	-	3 m above hopper	3.5 to 4.0

1	2	3	4
iv) Direction of flow	-	Upward	Upward
v) Size of PVC square tubes	-	50 mm x 50 mm	50 mm x 50 mm
vi) Depth of tube settler	-	0.5 to 0.6 m	0.5 to 0.6 m
III. DUAL MEDIA FILTER BED.			
i) Surface loading in lph/m^2	4000 to 7000	5000 to 10000	4000 to 8000
ii) Dual media details			
a) Coconut shell media depth	30 to 40 cm	30 to 50 cm	30 to 50 cm
average size in mm	1.0 to 2.0	1.0 to 2.0	1.0 to 2.0
b) Fine sand media depth	50 to 60 cm	50 to 60 cm	50 to 60 cm
Effective size in mm	0.45 to 0.55	0.45 to 0.55	0.45 to 0.55
Uniformity coefficient	Below 1.5	Below 1.5	Below 1.5
iii) Back wash method	Hard wash	Hard wash	Hard wash

Conversion : 1000 lph/m^2 = mph

TABLE 2 : Comparative costs of construction of the simplified and conventional treatment plants

Sr. No.	Name of water treatment plant (District)	Year of Construction	Capacity of the plant in mld	Cost of conventional plant based on quotations figures in '000 $	Cost of simplified plant as per new techniques in '000 $	Approximate saving due to adoption of simplified plant in '000 $
1.	Ramtek (Nagpur)	1973	2.40	56.25	15.63	40.62
2.	Surya Colony (Thana)	1976	0.66	25.00	12.50	12.50
3.	Varangaon (Jalgaon)	1977	4.22	100.00	50.00	50.00
4.	Kandla Port Trust (Gujarat)	1977	2.00	43.75	25.00	18.75
5.	Bhagur (Nasik)	1978	2.00	43.75	24.00	19.75
6.	Murbad (Thana)	1978	1.00	31.25	18.75	12.50
7.	Jejuri (Pune)	1978	2.40	56.25	25.00	12.50
8.	Akola (Nagpur) (Dual media beds only)	1978	2.00	25.00	12.50	12.50
9.	Dhulia dairy (Dhulia)	1979	1.50	37.50	18.75	18.75
10.	Chandori (Nasik)	1980	1.00	31.25	18.75	12.50

Développement de Méthodes de Filtrage Economiques et Simples pour la Fourniture des Eaux Rurales.

RESUME

En Inde, le rapport population rurale/population globale est de 80% environ. Le Gouvernement de l'Inde a prévu un vaste programme d'investissement pour le secteur rural, comprenant l'alimentation en eau potable. Cela a constitué pour le pays un problème essentiel: pouvoir développer un traitement de l'eau nécessitant à la fois un fonctionnement et un entretien simples, et un coût peu élevé.

En examinant les divers problèmes de conception, de construction et d'entretien d'installations de petite capacité, l'auteur a développé trois nouveaux types d'unités dans le cadre de ses travaux de recherches pour sa thèse de doctorat (1978). Pour traiter les sources d'eau peu chargées en impuretés, une installation Ramtek fut construite en Inde en 1973, avec préfiltrage sur graviers puis double filtration. Cette installation a pour particularité d'utiliser le gravier pour le pré-traitement, tandis que la coque broyée de noix de coco est utilisée pour la premiere fois dans la double filtration. Pour traiter les sources d'eau fortement chargées d'impuretés, l'installation Varangaon fut construite en 1977. Ce nouveau modèle comprend deux unités de floculateurs à lit de gravier et à tubes-decanteurs avec trois unités de double filtration. Pour traiter l'eau moyennement chargée en impuretés, l'installation Chandori fut construite en 1980. La caractéristique spéciale de cette installation est l'adoption d'un pré-traitement qui combine le floculateur à lit de gravier et les tubes décanteurs avec la double filtration.

Les installations sont de construction simple et d'entretien facile et la performance des installations s'est avérée satisfaisante. Le coût de la construction est généralement de 30 a 50% du coût des installations conventionnelles de mêmes capacités.

Paper 15

A training course in the optimization of water treatment plant performance for operators

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INTRODUCTION

Very few of the water treatment plant in operation today were designed to treat the specific raw water under the variety of conditions which occur throughout the year in any specific locality. In most cases the designer did not have the information on the characteristics of the raw water over a reasonably long period of time in order to know the range of the variable involved. Furthermore, the means and methods of determining design parameters from laboratory testing of raw water have not been well understood until recently. Another important reason is that the cost of obtaining good base information may have been prohibitive. The designer therefore applied criteria which had proven satisfactory in other treatment plants. It is for this reason there are so many very similar plants from any one group of designers. It is important that the treatment plant operators understand this and seek to optimize the plant performance regardless of design. This will be a continuous procedure during certain times of the year whenever the raw water is changing due to increases or decreases in rainfall, temperature, or any other factor that would change the treatability, such as turbidity, color, pH, temperature, alkalinity, etc.

Since the demand in all cities is increasing, most treatment plants sooner or later are pushed to their maximum and often severely overloaded. The result is poor water quality. It is important, therefore, to optimize operation in order to: (1) economize chemicals which in many places are imported and very expensive; (2) improve treated water quality; and (3) by improving efficiency of operation to provide an opportunity to increase plant capacity while maintaining water quality.

The optimization method utilizes simple procedures and only a small amount of equipment, glassware and laboratory space. Any experienced operator can learn it quite readily.

Settled water turbidities as a measure of clarification are the basis for the use of bench scale laboratory testing as an important and effective method of optimizing operation. This is valid procedure since turbidities are directly related to the quantities of all other suspended solids in the sample. Other more precise methods of determining the microbiological and particle content of a water sample are available but they are far more expensive, complex, and time-consuming.

As a matter of fact, bench scale work, as described here, is essentially a miniature pilot plant operation which can provide essential and applicable information. Obviously, judgement and care is necessary in scaling up the bench scale testing data. Experience, however, is excellent in the successful application of this type of information on a plant scale. Once the information has been obtained in the laboratory, it is possible to use the plant or a part of it for confirmation without excessively complicating the routine operations. This has been done in many plants and is an economical and practical procedure.

PARAMETERS.

The important parameters which can be determined within a practical range of application in the plant are:

1. Dispersion of coagulant, polymers and other reagents.

   a. Effectiveness of the rapid mix based on the limits of G values as applied by the stirring machine (see G vs RPM curves).

   b. Optimum dosages of coagulants, polymers and lime if required.

   c. Effect of coagulant application, means and method.

   d. Effect of coagulant dilution.

   e. Optimum time and sequence of application of chemical reagents.

2. Flocculation

   a. Optimum total time.

   b. Optimum G value and time for each basin or optimum step-down values.

   c. Optimum compartmentation.

   d. Prediction of effect of over and under agitation on clarification.

   e. Tolerable velocities in transporting flocculated water between units in the treatment plant.

3. Settling

   a. Settled water turbidities at various basin loadings.

   b. Expected turbidity carryover for any settling basin loading.

4. Filtration

   a. Floc load on the filter.

An important rule in running jar tests and one of the reasons they do provide good information is that only one variable is tested on each test run. The procedure then is to determine the optimum for each variable one at a time (the work usually starts with the determination of the optimum coagulant dose).

EQUIPMENT AND LABORATORY SPACE.

The equipment required is the following:

1. Stirring machine with variable speed control from 0 to approximately 150 rpm and it should have places for at least four 2-liter beakers. Some machines have space for six beakers.

2. Turbidimeter, light scatter type, with a range of 0.01 to approximately 1000.

3. pH meter.

4. Two-liter square jars with siphons (Figure 1).

5. Curves showing velocity gradient vs. rpm of stirring machine (Figure 2).

6. Miscellaneous glassware for holding samples, mixing reagents, making dilutions, etc.

7. Pipettes - 0-1 ml, 5 ml, 10 ml, 25 ml and 100 ml.

8. Graduates - 10, 25, 100 and 500 ml.

9. Stopwatch with sweep second hand.

The reagents necessary are:

$Al_2(SO_4)_3$ - stock - 10% concentration.

$FeCl_3$ - stock - 10% concentration (for each days use, dilute stock solution to 0.1% where 1 ml of coagulant solution will provide 1 mg of coagulant).

Polymers - cationic, anionic and non-ionic (manufacturers will provide samples) - make up polymer stock solution from dry reagent at 0.1% concentration.

Distilled water

Raw water

Lime solution - 1% - (make up daily).

The space requirement is relatively small - perhaps about 10 to 12 feet of laboratory bench with electric outlets with appropriate voltage for the equipment. A sink with rinse water is also important to washing glassware and disposing of samples.

LABORATORY TESTING.

The testing program begins with the determination of the optimum dosage of the coagulant. If the plant is treating the same water with the same coagulant, the test run can be made around the plant dose. If it is a different water, then the range must be fairly broad and then narrowing down to finally arrive at the optimum dose. The procedure is as follows:-

1. Fill each jar to the 2-liter mark with raw water.

2. Carefully pipette the dose destined for each jar (0.1% coagulant solution and place in a small beaker beside the jar.

3. If alkalinity is needed, measure out the lime and dose in advance of the coagulant, stirring slowly.

4. Decide on the mixing regime and record in the notes (Figure 3). If nothing is known which could give a clue to the optimum, use this one:

   a. Maximum rpm of stirring machine for coagulant dispersion.

   b. After 5 sec., reduce stirring to 100 rpm - 2 min.
      60 rpm - 3 min.
      20 rpm - 15 min.

      Total - 20 min. flocculation time.

5. After 20 minutes of flocculation, stop the stirring machine and begin the settling cycle. In order to better approximate the clarification, it is good practice to allow a lag time of about 20 sec. from the time of stopping the flocculation to the beginning of settling. This is about the time required for the fast rotary motion to subside. In a plant, it is about the time required for the flocculated water to go from the flocculation to settling basins before settling begins.

   Start timing the settling then at 20':20" or start with the watch set back to zero. Since settling velocity curves are not important in determining the coagulant dose, samples may be taken from each jar at 2 min. and 5 min. Just prior to the sampling time, the siphon should be drained. Since there are at least four jars two people need to be available to take the samples to avoid a serious time lag between sampling each jar. With a little practice, one person can take two samples at the same time so with two people sampling, the four can be taken simultaneously.

   The procedure is repeated for the 5-minute samples. Shortly before the time the siphon is drained to waste and exactly at five minutes, a second sample is taken.

   For the light scatter turbidimeter 30 to 35 ml is sufficient so that the sampling can be taken quickly and the liquid level in the jar is not lowered significantly.

6. With four jars, there will be 8 samples for determining turbidities. The turbidimeter should be checked for calibration for the range anticipated. Some instruments are calibrated for over 10 ntu or under. Turbidity readings are then made on each sample and recorded. Samples should be agitated carefully before pouring into the turbidimeter tube. A direct comparison of turbidities for each group of samples, 2 min. and 5 min., can be made.

7. After the first trial run the process is repeated, choosing dosages around that which, in the first run, gave the best clarification, i.e., the lowest turbidity. The range of turbidity of the 5 min. sample for a near optimum dosage should be in the range of 2.0 - 5.0 ntu.
   (When the optimum or near-optimum dosage is determined, this quantity of coagulant should be used for all subsequent work. This same procedure should be repeated for all coagulants that may be utilized).

8. Take pH readings of each jar of the trial run, giving optimum clarification and record. Those jars with higher dosages should give lower pH readings.

The second variable to be determined will be the dosage of the polymer. In this case, the objective will be to find the lowest possible polymer dose which will significantly improve clarification. The reason being that polymers add to the treatment cost. Some are very expensive - as much as $2/lb. and therefore must be applied sparingly.

In checking out the polymers, the same procedure is followed as in the case of the coagulants. The only modification will be that along with the coagulant dose for each jar, it will be necessary to pipette the polymer dose for each. Using the same mixing regime, the time for dosing the polymer should be just before reducing the mixing speed from 60 to 40 rpm, or at about 5 min. 30 sec. Here again, all the polymer dosing should be done at the same time.

Experience has shown that the most effective use of the polymer is to add it in a very dilute solution after the floc is well formed. Normally the floc is well formed at this point in the process.

The polymer may be dosed in varying amounts such as 0.05, 0.10, 0.15, and 0.20 mg/l in the four jars. After 20 min. of flocculation with the same mixing regime as in determining the coagulant dose, the settling cycle begins and sampling is also the same.

In this case, however, the least turbid sample will probably be the one with 0.20 mg/l polymer dose. This however, may be an expensive dose. A smaller dose giving a turbidity almost as low would be a better solution from the overall point of view of economy vs. clarification.

Once the dosages are determined, the testing on mixing time and energy input begins. This has such a wide range of possibilities that experience is very helpful in arriving at an optimum reasonably fast. In coming up with some trial alternatives, an idea of the plant flocculation system is important. If the job is to improve an existing plant, the system is there on the site to analyse. In all probability, an existing plant would have vertical paddle, horizontal reel, oscillating paddle, or walking beam type. Some of these systems are readily adapted to the step down regime of energy input in the flocculation and some are not. Obviously, in new plants the designer can specify a system that will give the desired results. In an existing system, every effort is necessary to utilize the existing equipment, but sometimes the old system must be abandoned. For example, if 4 or 6 or 8 compartments are already available, the bench scale work should be carried out with mixing regimes which are compatible. Theoretically, for 20 minutes of flocculation in four compartments, the flow would allow five minutes in each. The testing work then could be oriented accordingly. Similarly, for 8 compartments, there would be a 2.5 min. for each. In a horizontal reel system, the possibilities are much greater because it can be made into a continuous channel with G values and time varying as the water flows through the system.

Experience in sorting out this complex matter is fairly consistent for clarification of colored and turbid waters. A short period of high energy agitation followed by a relatively long period of low agitation either with or without a polymer has been very effective. This is only a general observation and could be wrong in any specific case. This is, however, a profitable place to start since a beginner could easily get lost in the maze of possibilities.

An example of a beginning then could be the optimum coagulant dose with 0.1 mg/l of polymer added just prior to the speed reduction of 5 min. then continue for another 15 minutes at 20 rpm.

The sampling now, however, is a little different in that the objective is to construct a settling velocity distribution curve which will guide the design of the settling basins. The settled water samples, therefore, should be taken after 1, 2, 3, 5 and 10 minutes in order to obtain sufficient data. Possibly the 8-minute sample could be eliminated since only under the most unusual circumstances would a basin be designed for such a low loading. It does, however, provide relevant data as described above except for the number of samples and the use of the turbidity data. The settling velocity curves plot turbidity remaining in the sample on the vertical axis (log scale) against settling velocities in cm/min on the horizontal axis (arithmetic scale). (Figure 4).

At this point in the bench scale work the coagulant and polymer dose along with mixing regime have been determined. Settling velocity distribution curves have been constructed from the turbidity samples taken from the test run on mixing regimes. Obviously the testing for optimum mixing regime involves many trials to arrive at the optimum.

The optimum mixing time is the next variable to be determined. This is done by using the optimum mixing regime and stirring the jars for different times from 10 to 50 or 70 minutes. The optimum coagulant and polymer dosages are used. In other words, the only variable is mixing time. For example:

Jars	1	2	3	4	5	6	7	8
Time (min)	10	20	30	40	50	60	70	80

The jar No. 1 would be removed or the stirring paddle stopped after 10 min. and turbidity samples taken at 1, 2, 3, 5 and 8 min. The procedure would be the same for each succeeding jar until all were completed. The data are recorded but in this test flocculation time in minutes is plotted on the horizontal scale while turbidity is plotted on the vertical scale. Typically the clarification improves very rapidly during the first 10 to 20 minutes. After this time the curves flatten out with relatively little improvement then after 30 to 40 minutes the turbidities begin to increase indicating the beginning of floc deterioration due to excessive mixing. The weaker the floc the more rapid will be the deterioration. The accompanying figure is typical of the clarification relative to time. (Figure 5).

The other variables which need to be determined are coagulant dilution, effect of and amount of sludge return, sequence of chemical addition. The optimums for these variables are determined in the same manner as that described for determining flocculation mixing regime and the data are plotted in the same way.

This laboratory testing program results in the accumulation of a large body of information upon which design and operation decisions can be made. These include the design of physical structures as well as the process. It should be understood that the data represent the experience in treating the water under the conditions at the time of the testing work. If the raw water changes in turbidity, temperature, alkalinity or in other ways throughout the year, the testing should be done under these changed conditions.

From the information obtained in the testing work decisions can be made on treatment process design which would include:

1. Most effective coagulant and dosage range of each.

2. Alkali dosage range and periods of year required.

3. Most effective polymer and dosages.
4. Time and sequence of chemical dosage.
5. Most effective process at extreme ranges of raw water variability.
6. Chlorination procedure to minimize THM formation.

The decisions on physical process would include:

1. Rapid mix, coagulant dispersion system.
2. Flocculation system, hydraulic, paddle, reel, etc.
3. Optimum flocculator time under extreme conditions.
4. Optimum input energy under all raw water conditions.
5. Flow control - compartmentation or other.
6. Permissible flow velocities, ports, channels, etc.
7. Settling basin loadings under various conditions.

After the many variables have been determined by the bench scale work the testing moves into the plant to try to find out how the plant process compares with that in the laboratory. This starts by dipping two liter (same square jars) samples from the outlet of the flocculation basin, allowing them to settle the same as in the laboratory, taking samples at the same times 1, 2, 3, 5, and 10 minutes and constructing settling velocity distribution curves. That is, the existing plant treatment process (dosages of all chemical reagents) and the plant mixing and flocculation system can be evaluated by comparison with laboratory performance. Also the performance of the plant flocculation system can be compared to that of the laboratory by taking plant dosed and mixed samples and flocculate them in the laboratory. (Figures 6 and 7).

In both these cases there is wide discrepancy between plant and laboratory performance. This indicates that there is ample possibility for improvement in the plant flocculation system. In Figure 7 the post operating performance is indicative of the result of plant performance under the modifications introduced in both process and the physical plant.

In most plants a portion of the poor flocculation is overcome in the settling basin which in these cases function as a flocculation as well as settling system.

One other variable should be tested and that is the effect of pH adjustment in clarification. This is done by fixing all variables except the pH which is varied by adding lime. The pH range is usually varied from 3.5 to about 7. and titration curves plotted with lime dosage against pH. Figures 8, 9 and 10 show three different titration curves. It should be noted how different each water reacts and therefore how important it is for the operator to know as much as possible about his specific water.

<u>EXAMPLE.</u>

Example of test run with mixing regime as the variable:

Raw Water Info -	Turbidity 8 ntu	pH - 7.1	Temp. - 19°C
	Color - 80	Alk. - 23 mg/l	Source - Reservoir

Coagulant - $FeCl_3$ Dose 30 mg/l

Alkali - None

Polymer - 985N Dose 0.10 mg/l (Dosed after 4.5 minutes of flocculation).

Mixed Regimes	Jars 1	2	3	4
	2 min - 100 rpm	1 min - 100 rpm	5 min - 100 rpm	5 min - 100 rpm
	3 min - 60 rpm	1 min - 80 rpm	5 min - 40 rpm	15 min - 20 rpm
	15 min - 20 rpm	8 min - 50 rpm	10 min - 20 rpm	
		10 min - 20 rpm		
Total Time	20 min	20 min	20 min	20 min

Results

Sampling Times	Settling Velocities cm/min	Jars 1	2	3	4
		Turbidities, ntu			
1	10	8	15	17	9
2	5	3.2	6	5	3.5
3	3.3	2.1	3.5	4.0	2.4
5	2.0	1.5	2.5	2.5	1.9
10	1.25	1.4	2.3	2.4	1.8
pH		4.8	4.9	4.9	4.8
Color Remaining in Filtered Samples		8	14	13	6

Jars 1 and 4 are significantly better than Jars 2 and 3. This indicates that this water flocculates best with a long period of gentle mixing.

APPLICATION BY OPERATOR.

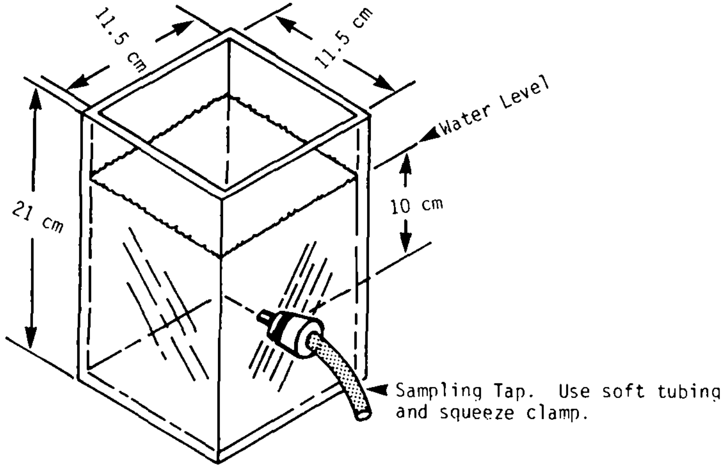
The operators can utilize the information obtained from the testing program to improve the plant performance. Part can be applied immediately, for example, like the dosages and information on sequence of chemical application. These are directly under the control of the operator and he can modify them easily. After testing during the extremes of the raw water conditions he will know the dosage ranges for the different conditions and can apply chemicals accordingly.

If there is a wide discrepancy between the laboratory results and the plant performances determined by sampling through the plant as indicated in Figures 6 and 7, then much improvement in performance can be attained.

Some of the physical changes in the plants may be beyond the operator's capacity especially those involving hydraulic design. In these matters he should obtain engineering assistance. There are, however, some simple improvements within the range of operator initiative.

One of the serious defects in most plants is the initial mixing of the coagulant. It is done very poorly and inefficiently in a majority of plants. This defect is often easily corrected simply by applying the coagulant through a drip diffuser at a weir parshall flume or through a pressurized diffuser in a channel or pipe line (Figures 11 and 12). Flocculation systems are also poor performers. These can be improved by changing the mixing energy simply by modifying the rotation speed of mechanical flocculators of changing the baffles in hydraulic units. Assistance for more extensive modification in both flocculation and settling basins can be obtained by consulting the literature and from associates.

The important message here is to begin testing to better understand how any specific water reacts and optimize the treatment process. This will begin the chain of events which will reduce chemical consumption, improve water quality and provide opportunities to increase plant capacity.



Settling Velocity = Surface Loading or Basin Overflow Rate

m^3/m^2/min or ft^3/ft^2/min = velocity

$$\frac{d}{t} = \frac{10\ \text{cm}}{\text{min}} = \text{cm/min}$$

Sampling Time-t (min)	Settling Distance-d (cm)	Settling Velocity (cm/min)	Relative Settling Basin Loading m^3/m^2/day	Relative Settling Basin Loading gal/ft^2/min
1	10	10	144.0	2.50
2	10	5	72.0	1.25
2.5	10	4	57.6	1.00
3	10	3.3	48	.83
4	10	2.5	36	.63
5	10	2.0	28.8	.50
6	10	1.7	23.9	.42
8	10	1.25	18.0	.31
10	10	1.0	14.4	.25

Source: Water and Air Research, Inc.

FIGURE 1. TWO-LITER JAR FOR BENCH SCALE TESTING

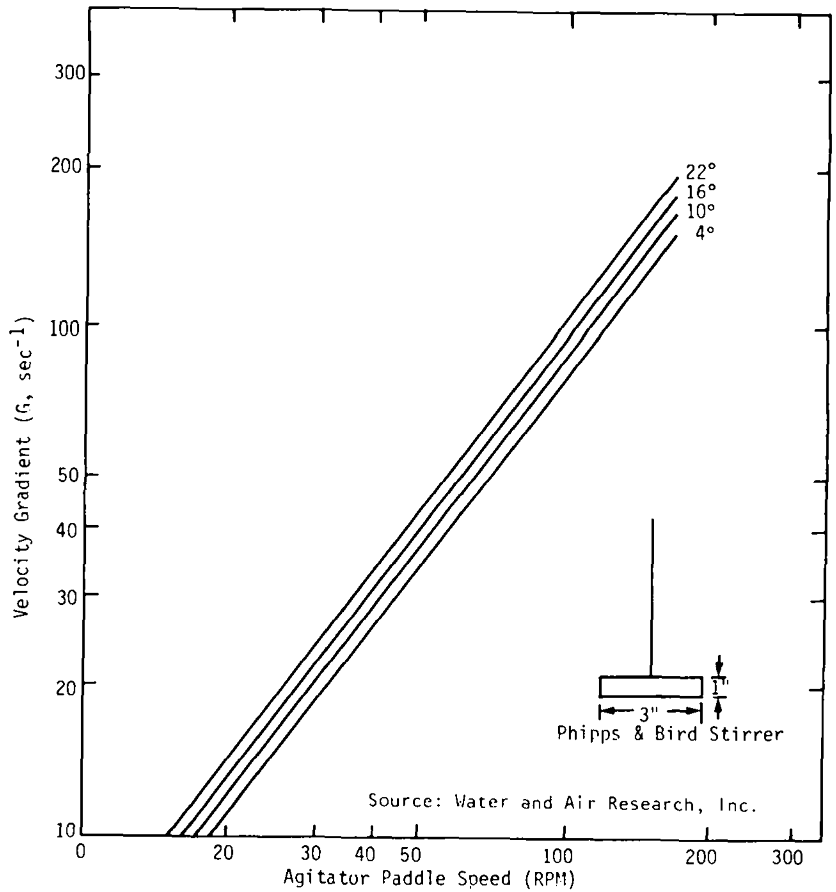


FIGURE 2. Velocity Gradient vs. RPM for a 2-Liter Square Beaker, Using a Phipps and Bird Stirrer

FIGURE 3. JAR TEST REPORT

Source: WATER AND AIR RESEARCH, INC.

Test No.________ Jar Test Equip.________ Source of Water________

Date Tested________ Dosing Method________ Date Collected________

By________ Sampling Method________ Turbidity________ pH________

Alkalinity________ Temp.________

Jar No.				1		2		3		4		5		6	
Rapid Mix		RPM													
		Duration, Sec.													
Flocculation		RPM													
		Duration, Min.													
REAGENTS				mg/L	mL	mg/L	mL	mg/L	mL	mg/L	mL	mg/L	mL	mg/L	mL
Coagulant____ Conc. of Solution, %____															
Alkali____ Conc. of Solution, %____															
Other____ Conc. of Solution,____															
				TEST RESULTS, CONCENTRATION REMAINING											
Sample	Time of Settlg Min.	Depth of Sampling cm	Settling Velocity cm/min	Turb. ftu	%	Turb. ftu	%	Turb. ftu	%	Turb. ftu	%	Turb. ftu	%	Turb. ftu	%
A	1														
B	2														
C	4														
D	8														
E	16														
		Al^{+++}													
F		pH													
		Alkalinity													
		Color													

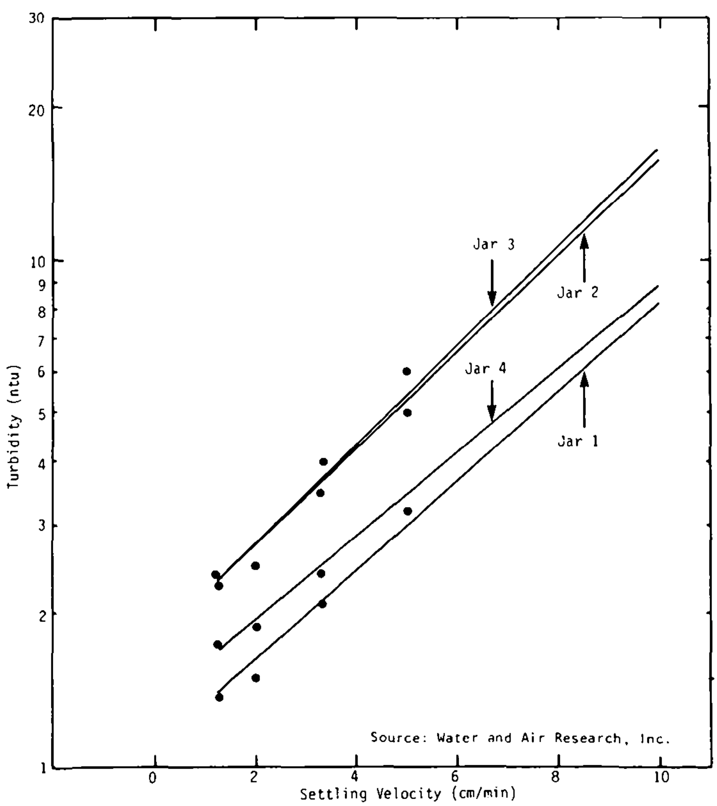


FIGURE 4. Example - Settling Velocity Distribution Curve

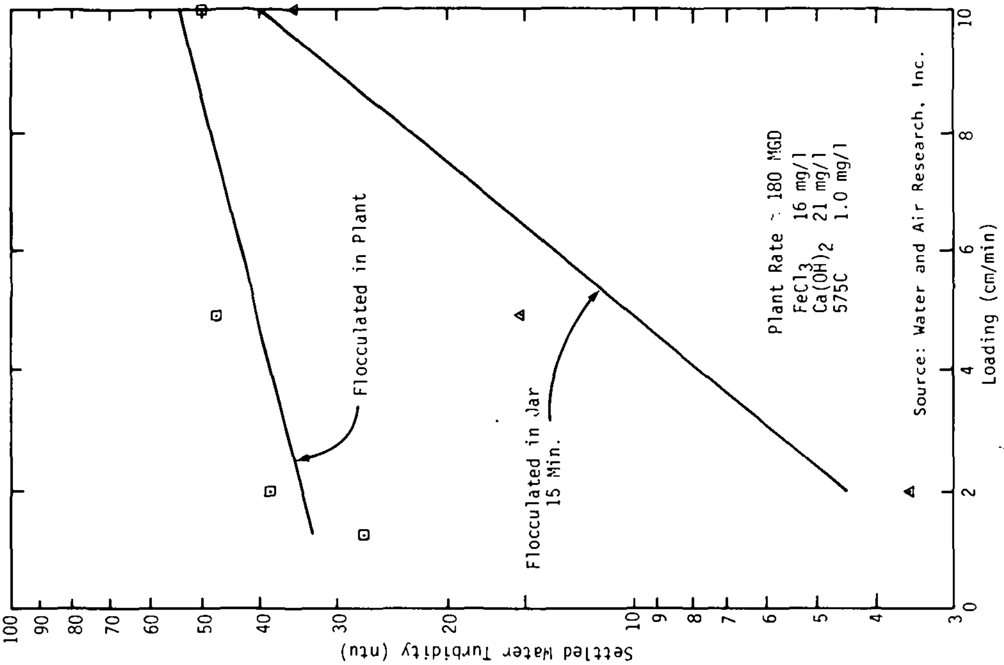


FIGURE 6. Comparison of Settled Water Turbidities of Plant Flocculated and Lab Flocculated Samples. Water Temperature 5°C

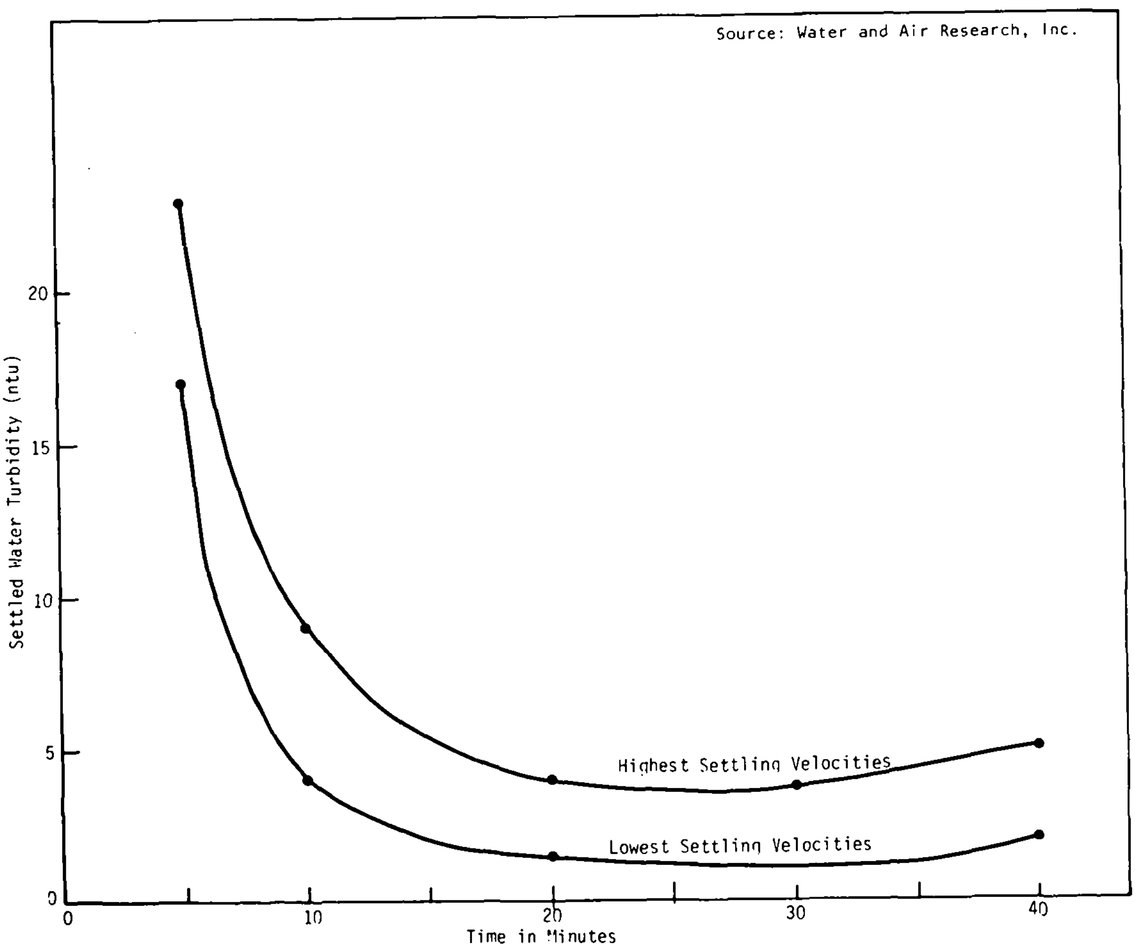


FIGURE 5. Typical Curves of Clarification Related to Flocculation Time

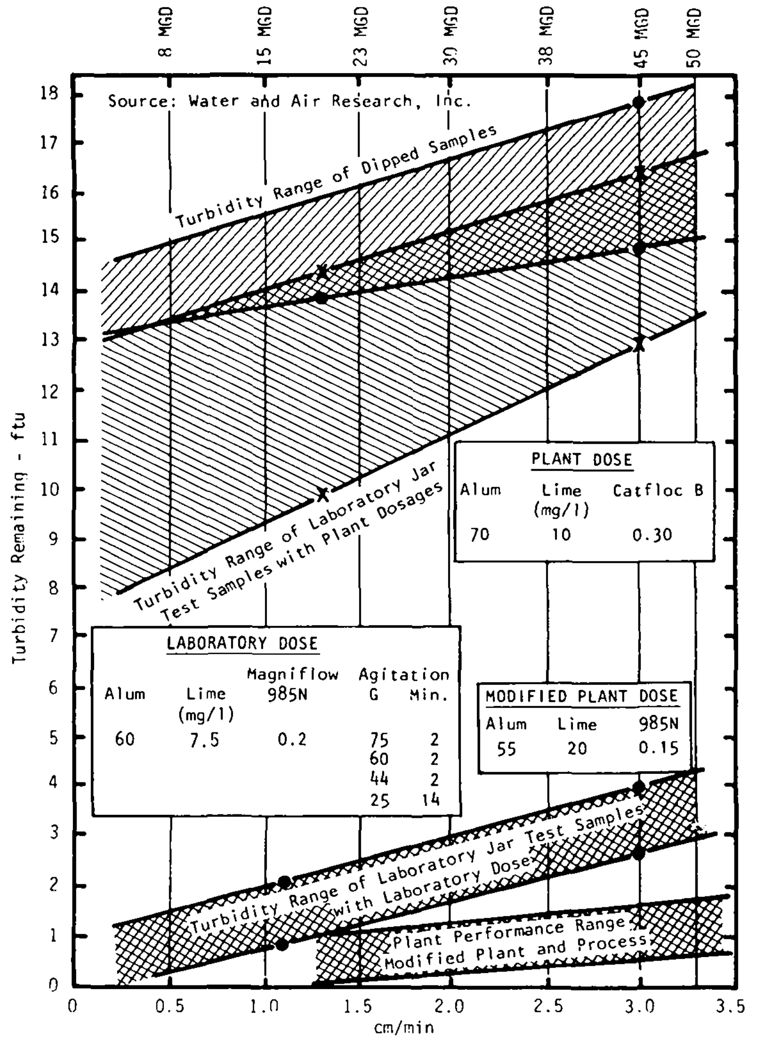


FIGURE 7. COMPARISON OF PERFORMANCE OF PLANT, PLANT DOSE AND PROCESS, LABORATORY DOSE AND PROCESS, AND PERFORMANCE OF PLANT UNDER MODIFIED CONDITIONS

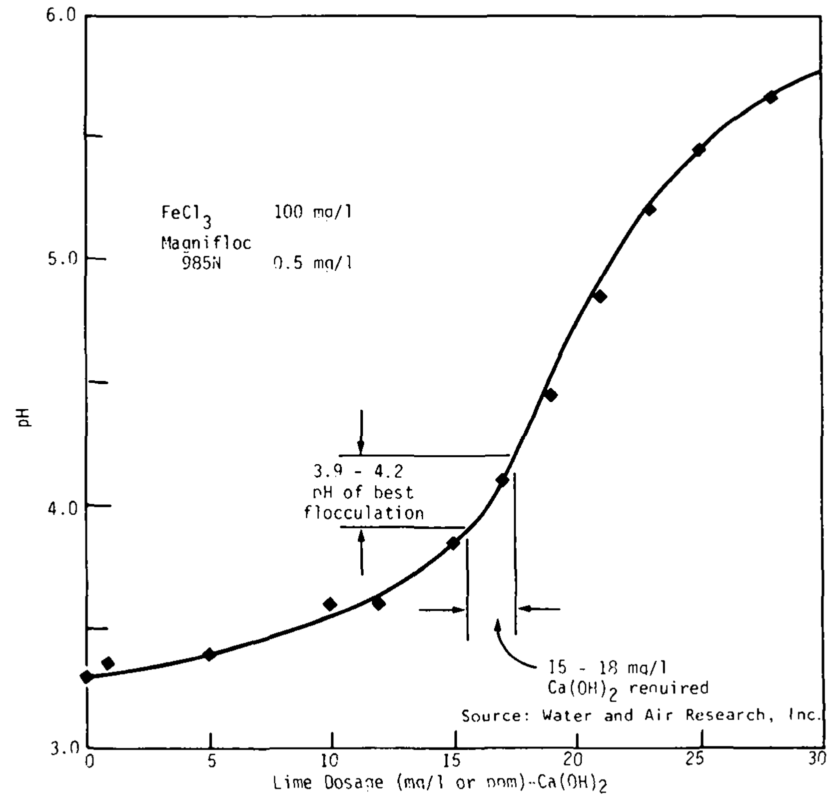


FIGURE 8. Titration Curve for 100 mg/l Dosage of $FeCl_3$ with Lime

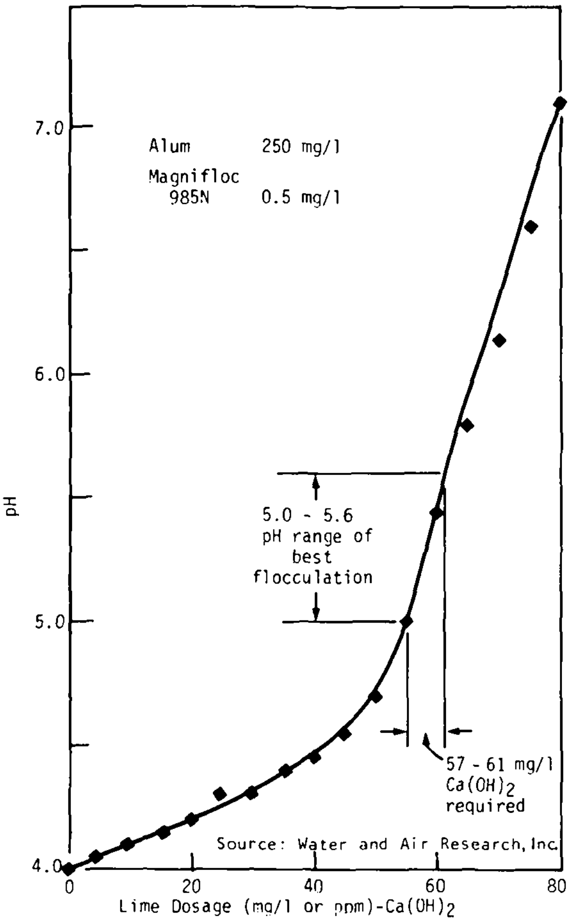


FIGURE 9. Titration Curve for 250 mg/l Dosage of Alum with Lime

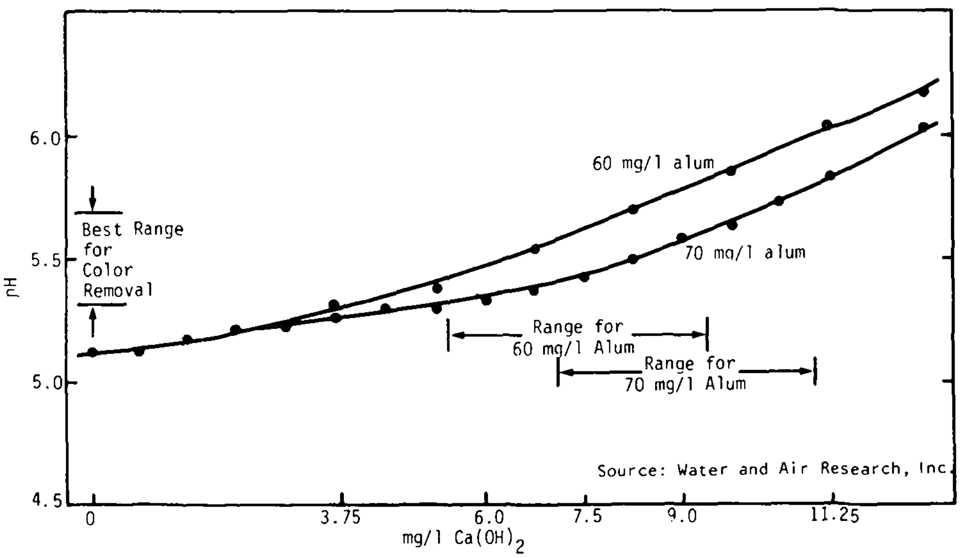
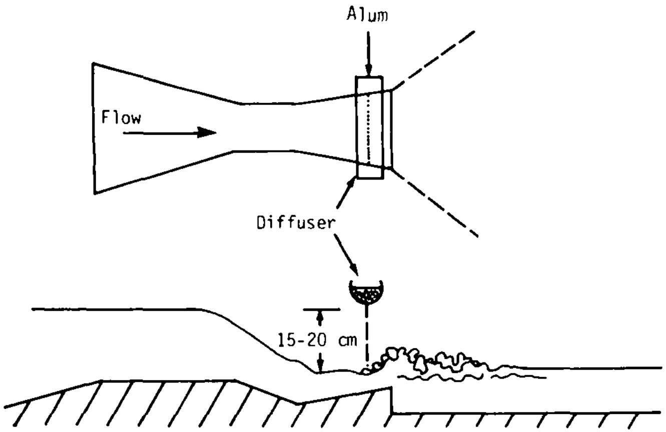
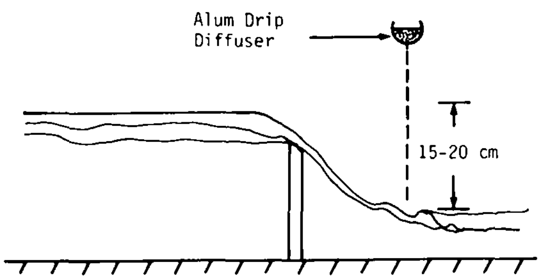


FIGURE 10. Titration Curves



ALUM FEED AT A PARSHALL FLUME THROUGH DRIP DIFFUSER



ALUM FEED AT A WEIR THROUGH DRIP DIFFUSER

DIFFUSER SPECIFICATIONS:

1 - Use corrosion resistant material.

2 - Design diffuser dosing capacity to maintain a liquid level of 4 to 5 cm over the orifices.

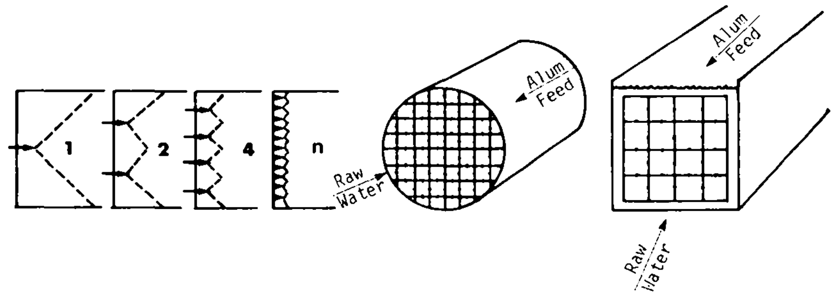
3 - Provide largest number of orifices possible to improve coagulant distribution across section of flow.

4 - Orifice diameter should be determined to minimize clogging while also satisfying specification #3 above.

Source: Water and Air Research, Inc.

FIGURE 11. ALUM FEED AT A PARSHALL FLUME AND AT A WEIR THROUGH DRIP DIFFUSER

SUBMERGED DIFFUSER IN PIPE OR CHANNEL



1. Divide the flow into the smallest possible segments by providing the largest number of orifices practicable. (High orifice density.)

2. Assure equal dosing throughout the water cross-section by designing a head-loss into the orifice system of the diffuser.

3. When preparing alum solution from dry $Al_2(SO_4)_3$, all solids must be removed by sedimentation or filtration to eliminate the possibility of clogging diffuser pipes and orifices.

4. Design must provide for quick and easy removal of the diffuser for inspection and cleaning.

5. Place diffuser so that dosing orifices face the flow.

Source: Water and Air Research, Inc.

FIGURE 12. SUBMERGED DIFFUSER IN PIPE OR CHANNEL

GENERAL JAR TEST PROCEDURE

1. Preliminary
    1. Prepare chemical reagents for dosing.
    2. Set up equipment and arrange glassware.
    3. Decide on dosages, sequence of dosing, and flocculation stirring regime.

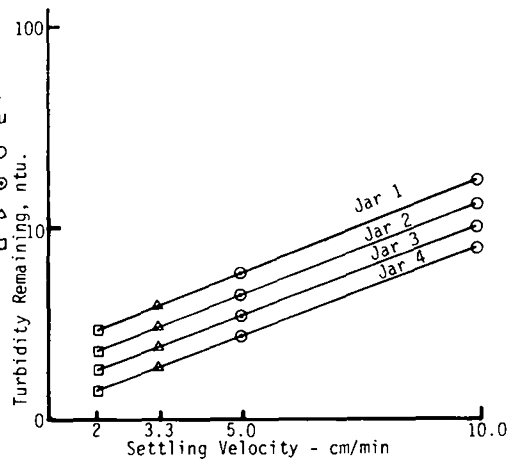
2. Preparation for Test Run
    1. Fill all jars to 2-liter mark with raw water.
    2. Pipette reagents into small beakers and line up beside jars in order of dosing sequence.
    3. Normal sequence of reagent addition - lime, coagulant, polymer.

3. Rapid Mix
    1. Turn stirrer to maximum speed.
    2. Dose lime if required and stir one minute.
    3. Dose coagulant, all jars simultaneously.
    4. Reduce machine speed to initial mixing speed of flocculation regime.

4. Flocculation
    1. Stirring regime: 100 rpm, 2 min; 60 rpm, 3 min; 20 rpm, 15 min; total = 20 min.
    2. Dose polymer just prior to reducing speed to 20 rpm.
    3. Stop machine at end of 20 min.

5. Settling
    1. Let jars sit 20 sec. and start timing for settling.
    2. Take samples of 30-40 ml at 1, 2, 3, and 5 min.
    3. Drain siphon just prior to taking each sample.
    4. Sample all jars simultaneously into small beakers.

Sample min.	Settling Velocity cm/min	Jars 1	2	3	4
		Turbidities, ntu			
1	10	○	○	○	○
2	5	⊙	⊙	⊙	⊙
3	3.3	△	△	△	△
5	2	□	□	□	□



Source: Water and Air Research, Inc.

Cours de Formation des Opérateurs à l'Optimisation des Performances d'une Unité de Traitement de l'Eau

RESUME

Tres peu des installations de traitement de l'eau, aujourd'hui en service, furent concues pour de l'eau brute dans la diversité des conditions qui surgissent dans un lieu donné au cours d'une année. Généralement le concepteur n'avait pas assez d'information sur les caractéristiques de l'eau brute pendant une periode de temps suffisamment longue pour connaître la gamme des variables en jeu. De plus, les moyens et les méthodes pour déterminer les concepts depuis les essais en laboratoire d'eau brute n'ont pas, jusqu'a récemment, été suffisamment bien compris. Une autre raison importante est que le coût pour obtenir de bonnes informations de base pouvait être prohibitif. Donc le concepteur appliquait des critères qui s'ètaient avérés satisfaisants dans d'autres installations de traitement. C'est pour cette raison qu'il y a tant d'installations très semblables venant d'un groupe donné de concepteurs. Il est important que les opérateurs de l'installation de traitement comprennent ceci et cherchent à optimiser la performance du matériel en dépit de sa conception. A certaines époques de l'année, lorsque l'eau brute change à cause de l'augmentation ou de la baisse des chutes de pluie, des modifications de température ou d'autres variables jouent sur la qualité, tels que la quantite d'impuretés, la couleur, le pH, la température, l'alcalinite, etc. L'optimisation du materiel constituera une procédure continue. La demande dans toutes les villes augmente, donc la plupart des installations de traitement seront a plus ou moins brève échéance saturées. Le resultat en est une médiocre qualité de l'eau. Il est important donc d'optimiser le fonctionnement afin de (1) économiser les produits chimiques qui sont souvent importés et très coûteux ; (2) améliorer la qualité de l'eau traitée et, (3) en améliorant l'efficacité du fonctionnement, augmenter la capacité de l'installation à qualité constante. La méthode d'optimisation utilisé des procédés simples, peu de materiel, que l'on trouve dans tous les laboratoires. Un opérateur expérimenté peut l'assimiler facilement.

La mesure de la turbidité de l'eau constitue la base des essais en laboratoire. D'autres méthodes plus précises sont utilisées pour déterminer le contenu microbiologique et les particules d'un échantillon d'eau, mais sont plus coûteuses, plus complexes et moins rapides.

En fait, le travail sur banc d'essai, tel que décrit dans ce document concerne essentiellement le fonctionnement d'une installation pilote miniature qui peut fournir de l'information utile. Une fois l'information obtenue en laboratoire, il est possible d'utiliser l'installation ou une partie de celle-ci pour confirmer sans trop compliquer les operations de routine. Ceci a été fait sur maintes installations ; c'est un procédé économique et pratique.

Paper 16

Optimal design of water distribution systems

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ABSTRACT

A method for optimal design of water distribution systems using a computer programme based on linear programming gradient (LPG) method is presented in this paper. No network solver is required in this approach. It incorporates the flow solution into the optimization procedure. The programme can be used to design water distribution systems, if the critical consumption pattern at various nodes are known. Operational decisions are included explicitly in the design process. The solution is obtained in terms of available pipe diameters, head at source, and capacity of the booster pump if required at a specified location. A solved example is also presented. Various data have been taken into account according to existing modes of practice in India.

INTRODUCTION

Water distribution systems are designed to deliver water from sources to consumers, using various components such as pipes, reservoirs, pumps and valves. There is an urgent need for the least cost design of water distribution systems in India under severely limited resources in finance. In fact the desired level of water supply to be delivered to consumers is increasing in recent years and it has become increasingly necessary now for engineers to be able to minimize expenditure on water distribution systems which form the major component in total expenditure needed to provide adequate and safe drinking water. It is estimated that the financial implication for water supply in India is to the tune of Rs. 3318 crores for the period 1981-90.

Considerable amount of work has been done so far in determining the economical sizes of pipes in the distribution system, height of the elevated reservoir, capacity of pumps, as well as the operation

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*Now graduate student at Cornell University, Ithaca, New York, U.S.A.

schedule of pumps and valves. Most of these techniques dealing with looped water distribution network employ nonlinear optimization techniques along with a network solver; such as Hardy Cross algorithm or Modified Newton-Raphson algorithm. Works in this area can be found in the papers by Jacoby, Kally, Watanatada, Shamir and Rasmusen (1-5). Some investigators (6,7) did not use a conventional network solver, but they had to make certain assumptions regarding the hydraulic solution of the network e.g., a fixed head distribution in the network. The Linear Programming Gradient (LPG) method as proposed by Alperovits and Shamir (8) incorporates the flow solution into the optimisation procedure, without making any assumptions about the hydraulic solution of the network. It enables optimization for multiple loadings and explicit inclusion of operational decision.

This paper presents a method for optimal design of water distribution systems using a computer programme based on the LPG method. No network solver is required in this approach. The programme developed here deals with the problem of designing an optimal water distribution system for a single loading pattern and gives the solution in terms of the available pipe diameters, head at the source (elevation of the reservoir or the horse power of the pump), and capacity of the booster pump if required at a specified location.

MODEL FORMULATION

Considering the design of a looped water distribution system supplied from one or more sources under gravity for a given set of demands at the nodes, the problem is formulated as given below:

For every pipe to be provided between any two nodes, a list of candidate diameters is specified. The algorithm has to provide the lengths of segments of different diameters from the above list so that the sum of the segment lengths is equal to the length of the link connecting the nodes. Let x_{ijm} be the length of m^{th} diameter pipe in the link connecting nodes i and j, then

$$\sum_m x_{ijm} = L_{ij} \tag{1}$$

where L_{ij} is the length of the link connecting nodes i and j.

Let J be the hydraulic gradient for a pipe to be obtained by using the Hazen Williams Equation,

$$J = \alpha\left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{2}$$

where Q is the discharge, D the pipe diameter, and C the Hazen Williams coefficient. α is a constant to be decided by the system of units (for Q in lpm and D in mm, $\alpha = 6.0922711 \times 10^6$). Let Q_{ij} be known throughout the network in such a manner that the flow continuity at the nodes is satisfied.

At every mode of the network, the pressure has to be more than a prespecified head, hence

$$Hmin_n \leq H_s \pm \sum_{i,j}\sum_m J_{ijm}\, x_{ijm} \tag{3}$$

where H_{min_n} is the minimum head to be maintained at node n, H_s is a known head, say at the sources and i,j correspond to the pipes forming the path between the node of known heads and mode n.

The algebraic sum of head losses in a loop should equate to zero and it should be equal to a known value for a path between two sources having known heads.

$$\sum_{i,j} \sum_{m} J_{ijm} \quad x_{ijm} = b_p \tag{4}$$

where b_p is equal to zero for loops. i,j correspond to appropriate links in the network.

The objective is to minimize the cost of the system which is

$$\sum_{i,j} \sum_{m} C_{ijm}\, x_{ijm} \tag{5}$$

where C is cost per unit length.

Hence a linear program can be written for minimization of (5) subject to the constraints (1), (2),and (4) which will give the optimum results for the given flow pattern along the pipes. This itself may serve as a satisfactory design for the distribution system if the designer is interested in maintaining the given flow pattern along the pipes. Such a situation may be a reasonable requirement as lumping of demands at the nodes is only an assumption and demands are actually distributed throughout the pipe. So the above model which satisfies the exact flow requirements along the pipes can be of great help to designers.

However, if flow requirement along the pipe is not a criterion governing the design, then a change in the flow pattern may further reduce the cost of the system. The flow pattern is to be changed in such a fashion that the flow continuity at the nodes is not disturbed. This can be achieved by making equal corrections in all pipes forming a loop. Let ΔQ_p be the change in flow in loop p, then

$$\frac{\partial(\text{Cost})}{\partial(\Delta Q_p)} = \frac{\partial(\text{Cost})}{\partial b_p} \; \frac{\partial b_p}{\partial(\Delta Q_p)} \;\pm\; \sum_{r} \frac{\partial(\text{Cost})}{\partial b_r} \; \frac{\partial b_r}{\partial(\Delta Q_p)} \tag{6}$$

where r corresponds to all other paths which have one or more members in common with loop p. The sign of the second term in the above equation depends on whether or not path r uses link i,j in the same direction as path p.

$$\text{Now} \quad \frac{\partial(\text{Cost})}{\partial\, b_p} = W_p \tag{7}$$

where W_p is the dual variable of the constraint corresponding to path p. In above expressions b_p holds for

$$b_p = \sum_{i,j} \sum_{m} \quad J_{ijm}\, x_{ijm} = \sum_{i,j} \sum_{m} \left[\alpha\, Q_{ij}^{1.852} C_{ijm}^{-1.852} D_{ijm}^{-4.87} x_{ijm}\right] \tag{8}$$

Then $$\frac{\partial b_p}{\partial(\Delta Q_p)} = \frac{\partial b_p}{\partial(Q_p)} = \sum_{i,j} \sum_m [1.852\alpha\, Q_{ij}^{0.852} C_{ijm}^{-1.852} D_{ijm}^{-4.87} x_{ijm}]$$

$$= \sum_{i,j} \sum_m [1.852(\Delta H_{ijm}/Q_{ij})] \tag{9}$$

Since we are interested only in the relative magnitude of the components of the gradient vector, the constant 1.852 can be avoided and we can write for the loop p

$$G_p = \frac{\partial(\text{Cost})}{\partial(\Delta Q_p)} = W_p \sum_{i,j\in p} \left(\frac{1}{Q_{i,j}}\right) \sum_m \Delta H_{ijm} \pm \sum_r W_r \sum_{i,j\in p} \left(\frac{1}{Q_{i,j}}\right) \sum_m \Delta H_{ijm} \tag{10}$$

The step size β along this gradient vector to decide the magnitude of changes provided should minimize the objective function by finding β from

$$\min_\beta \, [\, LP(\vec{Q} + \beta \quad \vec{\Delta Q})\,] \tag{11}$$

However, in the present work, only some heuristic laws were used to decide about the step length.

Incorporating Pumps in the Design

Decision variable for pumps is taken in terms of the head added by it. Hence the cost data required is the increase in cost due to a unit increase in the head to be supplied by the pump. However, due to nonlinear variation of the pump cost with its capacity, some approximation is required and an iterative procedure is needed to nullify the effect of approximation. If one denotes by XP(t) the head added by pump number t, then the head constraints for paths with pumps become

$$HMIN \le H_s \pm \sum_t XP(t) \pm \sum_{i,j} \sum_m J_{ijm}\, x_{ijm} \tag{12}$$

where the first summation is over the pumps in the path. The loop equations are also modified in the similar fashion. The procedure to take into account the nonlinearity of the capital cost is as follows:

1. Assume values for the cost per HP for each pump location.
2. Solve the linear program with these values as the coefficients of the XP's in the objective function.
3. For the resulting XP after the LP has been solved, compute the cost per HP.

If all values are close to those assumed, this step is complete and one proceeds to a flow iteration by the gradient method. Otherwise one takes the new costs and solves the programme again. This procedure is found to work well.

Incorporating Reservoirs in the Formulation

The decision variable for a reservoir is the elevation at which it is to be located. An initial elevation is assumed, then XR is the additional elevation where the reservoir is to be located, relative to its initially assumed elevation (XR+ is addition to reservoir height and XR- is deduction from its height). Path equations have to be formed between the reservoir at node s and nodes in the network. For node n,

$$HMIN_n \leq HO_s + XR_s^+ - XR_s^- \pm \sum_{i,j} \sum_m J_{ijm} x_{ijm} \qquad (13)$$

where HO_s is the initial elevation of the reservoir at node s. The coefficient of XR_s in the objective function is the cost of altering the height of the reservoir by 1 unit.

Selection of Candidate Diameters

At the outset, the list of candidate diameters for each link is based on a suitable hydraulic gradient say 0.001. Out of the available pipe diameters, the diameter is selected which gives the nearest hydraulic gradient to the above value. Candidate list consists of 3 pipe diameters, one larger and one smaller than the one selected above. If the first selection of candidate diameters results in an optimal LP solution, the list may have to be modified after the flows in the network have been changed by the gradient move. The modifications in the list of candidate diameters are based on the following rules:

a) If in the optimal LP solution a link is made entirely of one diameter, then for the next LP the list is made of 3 consecutive pipe diameters, the existing one being in between the other two.
b) When in the optimal solution a link is made of 2 diameters, the list for the next LP is made of these two, plus one adjacent to that diameter which has the larger of the two lengths.

Both cases result in a list of only 3 diameters for each link. In going from one LP solution to the other, the list for a particular element may remain unchanged, or a diameter may be dropped off one end of the list and a new one added at the other.

Overview of the Algorithm

a) The first step is to decide about the initial flow distribution knowing the demand at each of its nodes.
b) Objective function and constraints are formulated as described previously.
c) The LP is solved to give the optimum results for the initial flow distribution. The gradient vector is calculated knowing the dual variables of the LP.
d) The magnitude of the gradient vector is calculated as follows:

$$\text{MAGNITUDE} = (G_1^2 + G_2^2 + \ldots + G_n^2)^{1/2} \qquad (14)$$

where $G_1, G_2, \ldots, G_n$ are components of gradient vector.

If the MAGNITUDE is less than prespecified value the problem has been solved. If not so, the flows are to be changed in the gradient direction.

e) The gradient vector is normalized by dividing every component by MAGNITUDE.

f) The gradient vector is multiplied by a trial step size to give the required flow corrections in loops.
g) After applying these corrections, if any of the flow direction changes, the step size is reduced by a factor (say half) and new flow corrections are calculated. Program is terminated if the step size becomes less than a prespecified value.
h) For the new flow pattern, the LP is formulated and solved.
i) If the optimum cost of the new LP comes out to be less than the preceeding LP, the new gradient vector is calculated and control is shifted to step (d). If the optimum of the new LP is more than the preceding one then the step size is reduced by a factor and the preceding gradient move and flow pattern is used to get the new flow pattern and control is shifted to step (h). If the step size becomes less than a prespecified value, the program is terminated and the present solution is taken to be the optimum.

DESIGN OF WATER DISTRIBUTION SYSTEM USING THE PROGRAMME

A computer programme in Fortran has been written (9) based on the above algorithm. The programme can be used to design a new water distribution system given the consumption pattern during peak hours at various nodes of the system. The following are salient features regarding the use of the programme:

i) The critical consumption pattern, i.e. the peak demand should be known.
ii) An initial flow distribution along the pipes is to be given as input data. The given flow distribution should satisfy the continuity of flow requirement at all nodes of the system.
iii) In case of more than one source of supply, the amount of water to be withdrawn from different sources should be decided beforehand, i.e., the programme does not take into consideration the optimal allocation of resources.
iv) The suitable positions for the booster pumps should be given as input data. If a pump is required at a specified position to minimize the cost, the program will give the output in terms of the head to be added by the pump at that point.

Example

The set of available diameters and the cost of pipes are given in Table 1. The cost of pumps is taken to be Rs. 1000/- per installed Horse Power. The cost for the source of supply is to be given in terms of the increase in cost for 1 metre increase in head at the supply mode and the program gives the increase or decrease in head required to minimize the cost. As the useful life of various components in the distribution system will be different so all the capital costs are converted to annual costs and the total annual cost of the system is minimized.

The capital costs can be converted to annual costs by multiplying with the capital Recovery Factor (CRF).

$$\text{CRF} = \frac{i(1+i)^n}{(1+i)^n - 1}$$

in which i is the rate of interest and n is the estimated life of the component. Assuming 10 per cent rate of interest and 15 years and 40 years to be the lives of pumps and pipes respectively, the factors come out to be:

CRF(pump) = 0.13147; CRF(pipes) = 0.10225

To calculate the increase in cost due to increase of supply head of pump by 1 m, the following procedure is adopted:

Assuming the pump cost to be Rs. 1000/- per installed horse power the increase in cost due to 1m increase in head

$= 1000 \times 2.7323 \times 10^{-4} \times Q = 0.27323 \times Q$ Rs./m

where Q is water supplied by pump in lpm.

Assuming that the pump operates at this rate for 8 hours a day, the increase in annual cost due to operation

$$= \frac{0.163 \times Q \times 8 \times 365 \times 0.25}{1000 \times 0.8} = 0.14873\ Q \text{ Rs./m}$$

where cost of power = Rs. 0.25/KWh and efficiency of pump = 0.8. Hence increase in cost due to 1 m increase in supply head

= Capital Cost x CRF + Operation Cost

$= [(0.27323 \times 0.13147) + 0.14873]Q$

$= 0.18465 \times Q$ Rs./m.

Distribution network for the problem is given in Fig. 1. There is a pump serving at the source and a booster pump at pipe No. 9. Node No.1

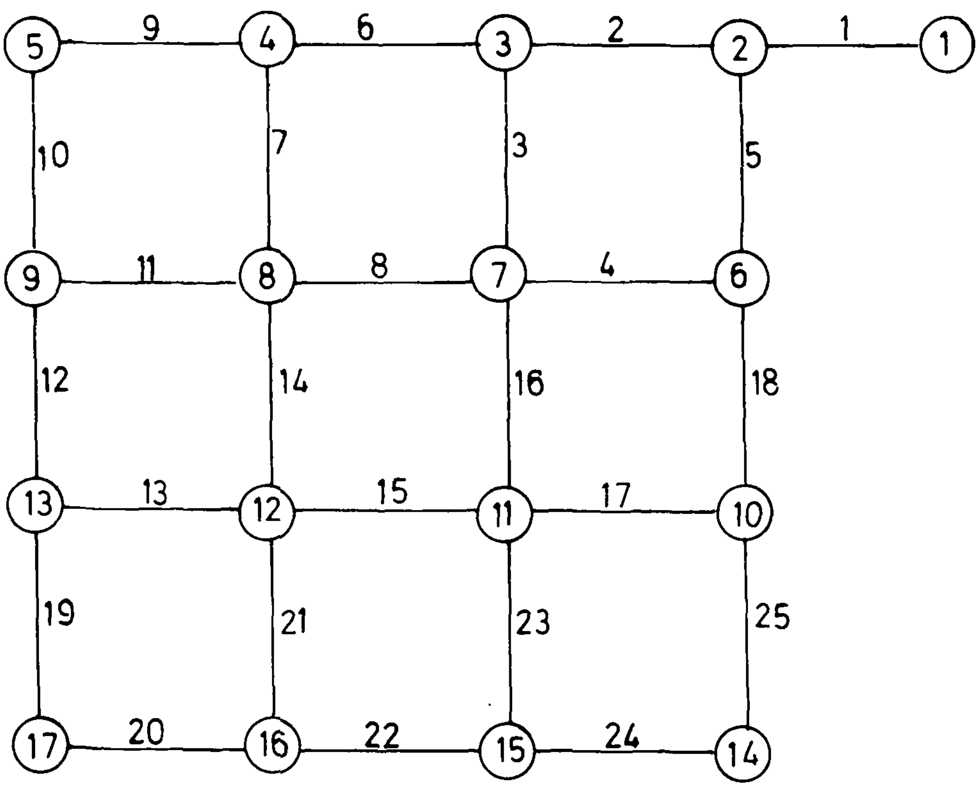

FIG.1 SAMPLE NETWORK

is supply node and rest all nodes have 100 lpm demand. Hence supply at node 1 is 1600 lpm. Increase in cost due to 1 metre increase in supply head is, therefore, 295.44 rupees.

The initial flow distribution and candidate diameters for each pipe are given in Table 2. The structure of initial linear programme is given in Table 3. The optimum design obtained from execution of the programme is given in Table 4. The program gives the lengths of pipe segments of adjacent available diameters to be provided between any two nodes in odd figures. These may be rounded off to the nearest suitable values andcheckedfor the heads available at the nodes. This has not been done in the present case and the pipe lengths have been reported upto two decimel places (These should be rounded off if possible to get the multiples of a preassigned value, say 1 metre).

CONCLUSIONS

The salient features of the method, as presented in this paper, are the following:

1. The programme does not require a network solver. It incorporates the flow solution into the optimization procedure, without making any assumptions about the hydraulic solution of the network.

2. Operational decisions are included explicitly in the design process.

3. The method yields a design that is hydraulically feasible and is closer to being optimal than the one from which the search is started; this holds true even when the optimization procedure is terminated prematurely.

4. Algorithm can be used to satisfy exact flow requirements along the pipes if required.

5. The program gives the solution in terms of available pipe diameters.

It is hoped that the present work may be helpful for the efficient solution of the complex problem of optimally designing and operating a water distribution system. It may also be mentioned that nonlinear optimization techniques require a high degree of experience in applying these to complex problem in order to give initial values of various parameters; and when measures are taken to take into account the variable demand pattern and discrete nature of some of the decision variables, the nonlinear problem becomes very difficult to solve.

TABLE 1. Available Pipes, Their Costs, and Hazen-William Coefficients

CRF for pipes = 0.10225

S.No.	Pipe diameter (mm)	Cost Rs/m	Annual cost Rs/m	Hazen-William coefficient
1	15.0	2.00	0.2045	140.0
2	20.0	3.00	0.30675	140.0
3	25.0	4.50	0.46012	140.0
4	32.0	5.00	0.51125	140.0
5	40.0	7.00	0.71575	140.0
6	50.0	8.00	0.818	140.0
7	65.0	11.00	1.12475	140.0
8	80.0	15.50	1.58487	140.0
9	100.0	22.00	2.2495	140.0
10	125.0	35.50	3.62987	140.0
11	150.0	47.00	4.80575	140.0
12	200.0	83.00	8.48675	130.0
13	250.0	112.00	11.452	130.0
14	300.0	145.00	14.82625	130.0
15	350.0	187.00	19.12075	130.0
16	400.0	221.00	22.59725	130.0
17	450.0	278.00	28.4255	130.0
18	500.0	329.00	33.64025	130.0
19	600.0	463.00	44.34175	130.0

Note: 1) According to the present practice in U.P. Jal Nigam, India upto 150 mm diameter, PVC pipes are provided and beyond this size, AC pipes are used. The costs and Hazen-William coefficients are taken accordingly.

2) 15 mm pipe has also been taken in the set of available pipes in the present case but usually such small pipes should not be considered while designing the distribution system. The optimum results will come out to be different if small pipes are not included in the list of available pipes.

TABLE 2. Pipe Data for the Sample Problem

Pipe No.	Initial flow (lpm)	Candidate diameters (mm)	Pipe No.	Initial flow (lpm)	Candidate diameters (mm)
1	1600.0	200, 250, 300	14	200.0	100, 125, 150
2	1170.0	200, 250, 300	15	10.0	32, 40, 50
3	300.0	125, 150, 200	16	200.0	100, 125, 150
4	10.0	32, 40, 50	17	10.0	32, 40, 50
5	330.0	125, 150, 200	18	220.0	100, 125, 150
6	770.0	150, 200, 250	19	90.0	63, 80, 100
7	300.0	125, 150, 200	20	10.0	32, 40, 50
8	10.0	32, 40, 50	21	100.0	80, 100, 125
9	370.0	125, 150, 200	22	10.0	32, 40, 50
10	270.0	100, 125, 150	23	100.0	80, 100, 125
11	10.0	32, 40, 50	24	10.0	32, 40, 50
12	180.0	80, 100, 125	25	110.0	80, 100, 125
13	10.0	32, 40, 50			

TABLE 3. Initial Linear Programme for the Problem

Begin	End Node	Numbers of pipes connected between the two nodes
		Pressure Equations
1	14	1,5,18,25
1	15	1,2,3,16,23
1	16	1,2,6,7,14,21
1	17	1,2,6,9,10,12,19
		Loop Equations
2	2	2,3,-4,-5
3	3	6,7,-8,-3
4	4	9,10,-11,-7
6	6	4,16,-17,-18
7	7	8,14,-15,-16
8	8	11,12,-13,-14
12	12	13,19,-20,-21
11	11	15,21,-22,-23
10	10	17,23,-24,-25

TABLE 4. Optimum Results for the Problem

Pipe No.	Flow (lpm)	Length (m)	Segment 1		Segment 2	
			Diameter (mm)	Length (m)	Diameter (mm)	Length (m)
1	1600.00	1000	150	1000.00	-	-
2	1177.45	1000	150	1000.00	-	-
3	307.09	1000	80	1000.00	-	-
4	2.53	1000	15	1000.00	-	-
5	322.54	1000	100	15.45	80	984.55
6	770.36	1000	125	1000.00	-	-
7	292.13	1000	100	556.24	80	443.76
8	9.43	1000	20	947.68	15	52.32
9	378.23	1000	100	1000.00	-	-
10	278.23	1000	100	786.84	80	213.16
11	2.03	1000	15	1000.00	-	-
12	180.26	1000	80	1000.00	-	-
13	1.77	1000	20	34.20	15	965.80
14	199.53	1000	80	1000.00	-	-
15	11.50	1000	32	1000.00	-	-
16	200.19	1000	80	1000.00	-	-
17	2.82	1000	20	435.61	15	564.38
18	220.01	1000	80	1000.00	-	-
19	82.04	1000	63	1000.00	-	-
20	17.96	1000	50	1000.00	-	-
21	109.26	1000	80	11.61	63	988.39
22	8.70	1000	32	1000.00	-	-
23	91.51	1000	63	705.89	50	294.10
24	17.19	1000	40	43.33	32	956.67
25	117.19	1000	63	1000.00	-	-

Optimum pumping head at node 1 = 35 + 32.51 = 67.51 m

Minimum cost of the system = 46819.07 Rs/year

Minimum cost for initial flow distribution = 47581.68 Rs/year

No booster pump is required at pipe no. 9.

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CONCEPTION OPTINALE D'UN SYSTEME DE DISTRIBUTION

par Rajendra S. Solanki et Dipak K. Ghosh

Résumé :

Cet article présente une méthode d'optimatisation de la conception d'un système de distribution d'eau basé sur les techniques de programmation linéaires et employant l'informatique.

Le programme écrit en langage "FORTRAN" est utilisable lorsque les modèles de consommation, à differents noeuds, sont connus. Il ne nécessite pas de spécialiste du calcul des réseaux. Il permet de définir des solutions optimales en terme de diamètre des conduites, des pressions à la source et des dimensionnements des surpresseurs.

Ce document présente un exemple d'utilisation ; les diverses donneés qui ont été prises en compte correspondent aux pratiques usuelles en Inde.

Paper 17

Appropriated water pumping systems and their applicability in semiarid regions investigated in the Dodoma region in Tanzania

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O. SUMMARY

Within the following study three groundwater-lifting-systems (flapping - vane - deep - well pump, flan - blower - Archimedes - screw, and Persian wheel) will be shown which use regenerative driving energies and offer support towards self-reliance to the local people by the economic effects.

The region of Dodoma (Tanzania) has a semi-arid climate with brambly vegetation and possesses a large groundwater storage. The inhabitants are semi-nomadic cattle-breeders by tradition, called Wagogos. Since 1973 they have settled down by the policy of Ujamaa (founding villages), and they have to learn new forms of agricultural production, like irrigation - and dry-field-farming.

Reckonings of the power capacity of the regenerative energy sources such as man, animal and wind will be given to enable the engineer to design water-lifting-systems according to the regional conditions. The technical, socio-cultural, ecological and political aims, according to the development objectives like self-reliance, technical reliability, avoiding new dependences, are discussed and assessed within seven items as well as the transferability. Power capacity, discharge, head, and efficiency are exactly described; also the constructions, materials used, and the possible effects to the Wagogos are studied.

1. INTRODUCTION

This report is based on the work of AKWIE in 1979 (Water Resources as an Instrument of Development Policy, study group at the Technical University of Berlin).

It shall extend the range of suitable lifting systems for irrigation in developing countries not to be restricted to modernize pumping devices. The idea of promoting traditional systems is lead by the political aim of strengthening self-reliance.

Criteria of assessment are given which reflect the regional conditions in technical, socio-economic and cultural aspects to decide what kind of lifting system is appropriate to the location. That means not only criteria based on the natural sciences are important.

The location of the example, Dodoma region in Tanzania, was chosen by the author, who spent there three months in summer 1979, performing an examination of small scale irrigation. Appropriate ground-water-lifting-systems for this region are proposed and their effects to the local population are described.

2. REGIONAL CONDITIONS FOR WATER-LIFTING IN DODOMA REGION

Situated in the central highlands of the United Republic of Tanzania Dodoma region covers an area of 41 000 km^2. Within this region approx. 6% of the Tanzanian population is living with a density of 16 inhabitants/km^2.

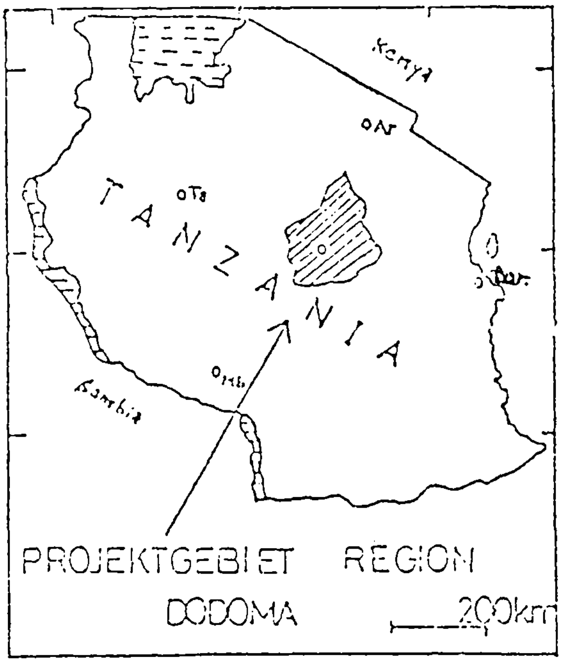

Figure 1 Dodoma Region Tanzania

The region, including the new capital Dodoma (25 000 inhabitants), is formed by large dry plains with an average height of 1 300 m a.s.l., and a semi-arid climate is dominating. Predominantly, the natural conditions are determined by the distinct dry-season over seven months, May to November, and by the single wet-season, December to April. The long-term mean precipitation is h = 572 mm/a. The average wind-velocities during the irrigation-period permit the use of wind-energy for groundwater-lifting.

Groundwater-discharge takes place in two aquifers in this region, normally, one of these aquifers is lying 30 m under ground. The other aquifer consists of hundreds of kilometers of alluvial channels, showing impermeable bottoms. Because of the low lifting head the latter aquifer can be used better with respect to the local energy sources. The water of these Wadis also often feeds salt swamps and cannot be used elsewhere.

The traditional economy of Dodoma region was restricted to collecting and hunting activities and semi-nomadic life-stock according to the natural conditions. Today, life-stock forms the greatest amount of monetary income in this region. Estimations done for the year of 1967 yielded the following percentage of production output in Dodoma region: 40% in subsistence agriculture, 41% in secondary and tertiary production and 19% in monetary agriculture [9]. Agricultural production took place in dry-field-farming for subsistence, irrigation farming was unknown.

To develop irrigation farming the third Five-years-plan 1975-80 scheduled 2,272 ha of irrigated farm land in this region [4]. The main industrial product is wine, which is sold on the national market, only. Furthermore, the infra-structure of the region is on a low level. There only exist some roads and one railway line, and the problems of traffic are very severe due to fuel shortage. In sommer 1979, e.g., all Diesel oil was rationed and no reliable energy supply existed. Dependend on the traditional economic system the typical kind of settlement was the spread settlement up to the seventieth of our century.

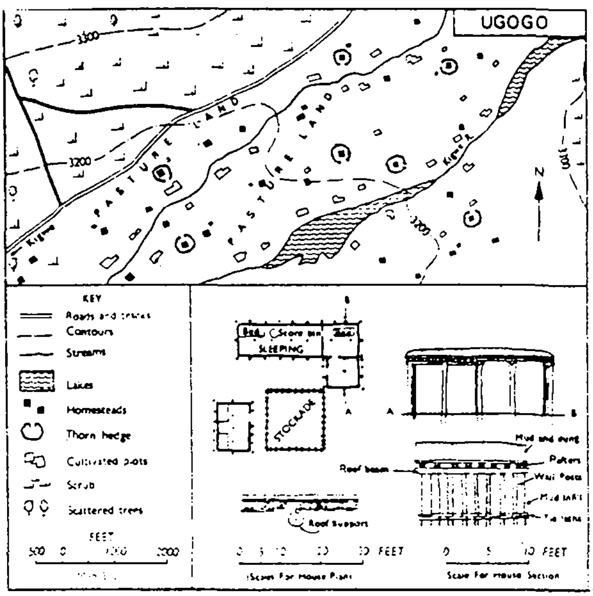

Figure 2 Settlement of the Wagogos

Settlements similar to villages exist only in small number, mainly at the stops of the railway Dar-es-salaam/Tabora/Mwanza-Kigoma and along the main streets Iringa/Dodoma/Arusha and Morogoro/Dodoma/Singida, which mostly fullfil economic and less administrative or residential functions. This mainly was changed by the policy of Kjamaa within the "Operation Dodoma", in 1973/74, when all spread-settlements were abolished. Today, the inhabitants of the region live in 359 villages with approx. 130,000 households cultivating an agricultural effective area of approx. 3,000 km^2, nearly exclusively in extensive dry-field-farming. Every village includes a primary school; furthermore, a number of medical stations, village-workshops, village-shops, as well as eight community-developing-centres were built and supplied with attendants and materials. In spite of these approaches in planning, the economic development of this region hardly was promoted due to several reasons (hasty performance of administrative acts, wrong choice of location, etc.). The region further is straightened to the capital Dodoma as sub-supplier; it is the poorest region in Tanzania but one, namely Singida.

The way of life of the Wagogos has not changed basically by the recent phase of village-founding (Ujamaa), and it is performed within the extended family living together in a horse shoe shaped complex of buildings. The family structured patriarchally, and polygamy still exists. Every extended family - that means the men - owns a particular property of the ground and the life stock. In the actual communal-villages the communal property is strongly propagated, but only very few villages show complete communal property. Traditionally, there is a division of work with respect to the sexes; the men manage the life stock, the scale, and the trade, as well as the monetary part of agriculture; the women are responsible for the basic-food-production, the water-supply, i.d., the carriage of the water, and the household. Handicrafts qualifications are to be found in the production of weapons and jewellery, made by smithes. Furthermore, great earthen vessels are burned, fabricated by special craftsmen.

3. BASIC DESIGN OF THE WATER DEMAND FOR IRRIGATION

To determine the possible lifting systems the gross irrigation requirement was calculated for the region. This is done via the climatic water account of potential evapotranspiration and long-term precipitation, including the gross irrigation efficiency, which takes the conveyance losses into account, and the coefficient of plant covering, which represents the relation between the evapotranspiration as a function of growth of the specific crop (here maize) and the evaporation of the Class A Pan.

The calculation results in a daily water demand of 7.88 mm/d, i.e., a lifting discharge of 2.73 l/s ha irrigated ground, given 8 hours per day irrigation period. These values are taken as approximate ones to reflect the technical implementation of the appropriate lifting systems.

The given groundwater yield can cover the above water demand as experiences with wells show.

4. CONDUCTIVITY OF HUMAN, ANIMAL AND WIND POWER DRIVE

Capacity (i.e. discharge, load) of lifting systems driven by man power is dependend mainly on productive manpower. Herefor table 1 shows computations of 18th and 19th century, which are valid for well-fed people (mean value).

Table 1: Productive power of man working with a handle

	continuous piecework	wage - work or piecework with interruptions
tractive force at handle	98 N	78 N
speed of handle	1.0 m/s	0,78 m/s
daily working time	8 h	8 h
power	99 W	61 W

Productive power of draught animals is connected with some kind of power transmission. For a whim gin drive table 2 and table 3 are valid for every single draught animal.

Table 2: Productive power of horses working with a whim gin

draught animal	tractive force	velocity	daily working time	power
healthy, strong horse (400 - 500 kg) piecework	638 N	1,0 m/s	6 h	638 W
weak horse (300 kg) without piecework	441 N	0,9 m/s	8 h	397 W

When several horses or other draught animals are working with one whim gin, the productive power for each animal is decreasing.

Table 3: Production power for oxes, mules and donkeys working with a whim gin

draught animal	tractive force	velocity	daily working time	power
ox	638 N	0,6 m/s	8 h	283 W
mule	294 N	0,9 m/s	8 h	265 W
donkey	137 N	0,8 m/s	8 h	110 W

The worldwide increase of prices of fossil energy and their limited resources promotes the utilization of regenerative enery sources. Wind power is very suitable as regenerative energy for water lifting systems.

Wind power is dependend on the general wind direction. Furthermore, local winds within a zone of 30 m above the ground are usable for wind-mills.

By using wind power you need data about distribution of wind velocity and a windforce duration curve.

Maximal possible wind power is given by the formula:

$$P_{max} = \frac{\rho_L}{2} A \cdot v_o^3 \cdot 0{,}593$$

ρ_L : density of wind in kg/m^3

A : area exposed to wind force in m^2

v_o : undisturbed wind velocity in m/s.

Basic design criteria for the use of wind-mills are recapitulated, besides the mentioned socio-economic conditions, annual series of wind performance, annual series of irrigation requirement and a long term value of wind velocity.

5. PROBLEMS OF IMPLEMENTATION OF GROUNDWATER-LIFTING-SYSTEMS

If you want to initiate a new lifting system, you have to see the local problems. As described above people in Dodoma region had no experience in irrigation technology. Besides of this, there is a big change from a semi-nomadic to an agricultural way of life.

You have to adapt water lifting technology to the regional and social conditions. You have to pay attention especially to the drive of the lifting system, as its reliability is very important.

6. CRITERIA OF ASSESSMENT FOR THE CHOICE OF AN APPROPRIATED GROUNDWATER-LIFTING-SYSTEM IN DODOMA REGION

Criteria of assessment are given by personal and written experience. Decision cannot be made just by hydrological and meteorological data. Many social failures in planning irrigation systems for developing countries demand a new approach in respect to the evaluation and analysis of the aim.

6.1 Analysis of objectives

In this essential part of any planning, the aim of development is prescribed and thereby its assessment.

You can call the general aim an improvement of life conditions for the rural population in countries of the Third World by irrigation farming. With a view to groundwater-lifting-systems the aims are increasing of agricultural yield, increasing of standard of self-reliance, security in food supply, opening up a monetary market in zones of subsidiary agriculture. You also get new jobs, and technical and general knowledge is increasing. When irrigation technology is innovated, new forms of community life are coming up. The semi-nomadic Wagogos in Dodoma region for example cannot live their traditional way of life any longer.

Groundwater-lifting-systems are an important part of the irrigation system. You therefore need a safe energy supply for the lifting system. Water requirement has to be adapted to groundwater discharge.

Besides this, there should be an equilibrium between groundwater discharge and drawdown and the open capillary fringe for deep rooting plants. Altogether there should be a well-balanced hydrological cycle with its main factors, discharge, evaporation, transport of atmospheric water vapor, change in water storage. This is necessary to prevent spreading of desert region.

The lifting-systems should have a simple technical construction, so that many people can be skilled for repairing.

6.2 Criteria of assessment

Many questions can be summarized in seven categories for the evaluation of groundwater-lifting-systems:

1. Technical cenception, capacity and reliability of lifting-systems.

2. Organizational and operational pre-conditions of site, as well as the existence of a market for supply and outlet.

3. Ecological consequences for the site.

4. Evaluation of local and regional organisations, which are able to implement lifting-systems.

5. Requirements of lifting-systems in comparison with development and abilities in the region.

6. Socio-economic relations and values of the population in the site in regard to lifting-systems.

7. Direct effects on income and employment of the population inside and outside the region.

Additional criteria of transferability are:

8. Usual industrial standard in the region.

9. Transmission of know-how which is independent of the industrial country.

A detailed example is explained in [4].

7. FEASIBLE GROUNDWATER-LIFTING-SYSTEMS FOR DODOMA REGION

You have to consider the following lifting-systems as a suggestion which adapt hydrological, meteorological and technical conditions. The final system has to be chosen after investigating the Dodoma region.

Criteria of assessment, mentioned above, shall help to find an appropriated solution. Therefore, you have to see the aims and to value the judgement as an example for a development of the poorest by the concept of "self-reliance". Therefore you have to decide with the persons concerned.

7.1 Flapping-vane-deep-well-pump

In Dodoma region the lower aquifer is 30 - 61 m below the surface. For this aquifer a flapping-vane-deep-well-pump is very suited. It pressures windpower directly on a piston without

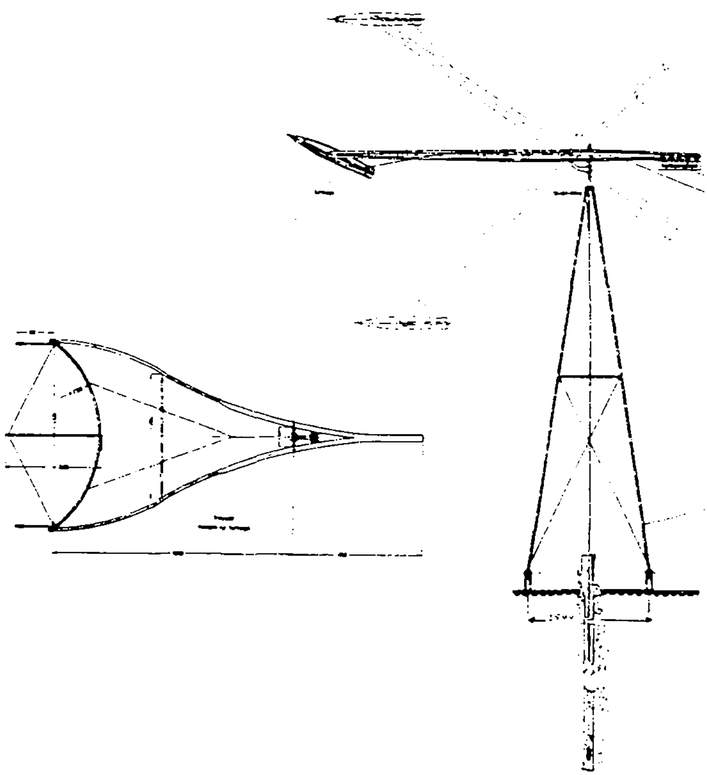

Figure 3 Flapping-vane-deep-well-pump

The vane can swing freely about its horizontal main axis between an upper and a lower angular stop. The pivotal point of the vane is behind the resultant of the vane lifting force or buoyancy in the wind direction, so that the vane-depending on whether its effective approach angle is positive or negative-is swung by the lifting force to the upper or lower angular stop.

The vane can thus assume two angular positions relative to the longitudinal axis of the lever arm. At the same time, during the up-and-down movement of the lever arm the effective approach angle is continuously changed. The up-and-down movement of the lever arm, and hence the working motion of the reciprocating pump, therefore derives from the periodically changing direction and magnitude of the lifting force of the vane. Although the vane approach angle can attain a large value at the upper or lower maximum position of the lever arm, there is practically no danger of a break in the flow and hence a loss of lifting force, since owing to the periodically changing circulation around the vane there is always a stable starting boundary layer.

The total height of the pylon is 10 m.

Piston pumps are suited for small discharge and big delivery head (up to h_{max} = 60 m).

Introduced here is a wind-driven piston pump in which the movement behaviour of the component parts of the windpower machine and the pump is matched, and in which the technology is much less demanding compared with that of the rotary wind machines.

The wind machine adapted to the motion of the reciprocating pump must provide directly an up-and-down movement in the frequency range of 0.2 to 1 l/s without another motion converter being interposed. The piston pump must have a long stroke so as to ensure that any gas bubbles which may be present in the well water do not adversely affect or prevent delivery.

Delivery can be calculated with the equations mentioned in chapter 4.

$$Q = \frac{0{,}593 \cdot \rho_L \cdot \eta \cdot A_F \cdot v_o^3}{2 \cdot \rho_W \cdot \delta \cdot h_{max}} \qquad m^3/s$$

ρ_L	=	air density	=	1,293 kg/m^3
ρ_W	=	water density	=	1 000 kg/m^3
A_F	=	vane surface	=	6 m^2
g	=	gravitational constant	=	9,81 m/s^2
h_{max}	=	manometric head	=	40 m

v_o	=	wind velocity	m/s
η	=	efficiency of pump	0.9

If you now consider a concrete example you obtain the following result. In July you can irrigate with one vane pump an area of 0.35 ha, in October an area of 0.57 ha (daily depth of irrigation $h_{B,d}$ = 7.88 mm/d).

This kind of groundwater-lifting-system is very suited for the irrigation of small areas with a low-lying aquifer.

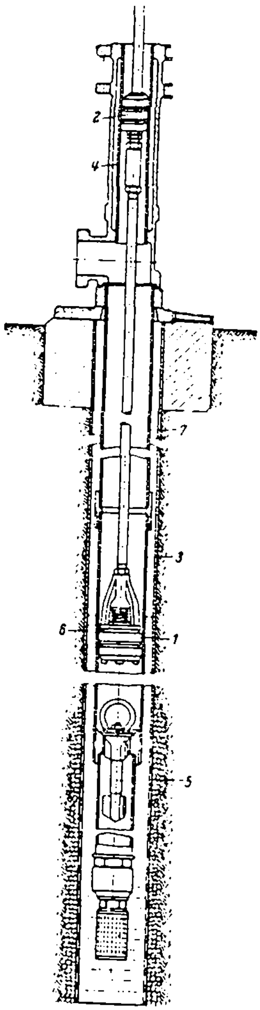

Figure 4 Deep-Well-Piston-Pump

The utilization of wind energy with this system increases the independence of the persons concerned, or a country like Tanzania has to import fossil energy and has to accept prices of the world market.

7.2 Flan-blower-Archimedes-screw

This groundwater-lifting-system shall be used for the groundwater flowing in the sandy river bed. For this flan-blower "Comet" shall be used as drining element, as this has been used in the region of Dodoma since about 1950.

Figure 5 Flan-Blower "Comet"

To transmit the rotation movement of the flan-blower on the delivering fluid an Archimedes-screw is chosen, as there are less problems with gearing.

Figure 6 Archimedes Screw

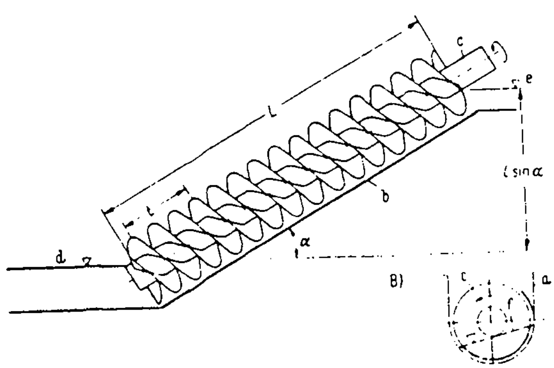

Figure 7 Scheme of Archimedes Screw

a trough

b bottom of the trough

c in direction to the arrow rotating, multiple-thread screw

d,e lower and upper water-surface

t distance between two turns (pitch)

l length of the screw pump

$l \cdot \sin \alpha = h_{geo}$ geodetic pressure head

A) side-view

B) cross-section

In an open trough a, which is placed with an inclination of about 30^o a screw is rotating with a small slackness. The water is lifted by the shear effect of the screw. This Archimedes screw can also be made of wood by notching spiral deepenings in the axis and setting wooden shingles in it.

The elevating capacity of every pitch is

$$P = Q \cdot \rho \cdot g \cdot t$$

with

ρ density in kg/m^3

g gravitational constant m/s^2

t pitch in m

Q transported fluid in m^3/s

In the example calculation for Dodoma region, the geodetic pressure head h_{geo} is 5 m, as the depth of the river bed is in this range. The efficiency is 0.65 and the wing surface of the windmill "Comet" is $A_F = 10.2\ m^2$ for a radius r = 1.8 m. In July a system in this region can irrigate 3.4 ha and in October 5.5 ha. To irrigate during calm, a reservoir must be built.

The efficiency of this system in opposition to the flapping vane deep-well pump is much better, so that this system can be installed in a great number. During calm manual working is also possible if a winch is fixed at the end of the screw. But then there will be more problems with gearing, because an additive separating coupling for the pump rods must be constructed. The self-reliant construction and maintenance is also secured, as in the "Arusha Appropriate Technology Project" flan blowers are built which are not very efficient ; but they are built with material available in Tanzania. Therefore it is secured, that the domestic population is able to handle this system.

Social and cultural effects are similar to those of the faapping-vane-deep-well-pump. This system will not lead to a dependance from industrial countries. Also the existing know how support the development strategy "self-reliance" for the concerned persons.

7.3 Persian Wheel

This system is suited for the upper aquifer in the sandbed. Hereby oxen shall be used as driving force.

Figure 8 whim gin with a short shaf

At a whim gin the draught animal is moving in a circle with a radius of about 4,5 - 6,o m. It is drawing a draught equippment which set the whim gin in rotation.

The rotation of the whim gin is transmitted on the shaft by bevel gears, ring gears or other draught gears. At this shaft a big wheel with spokes is fixed directly to the wheel. This whim gin can be constructed variously. Different characteristics are particularly horizontal or vertical shafts.

The wheel with spokes is either made of two wheels connected by handles or of one wheel. The wheel can be completely made of wood, but there are also new constructions where whim gin, wheel and handles are made of skel [7]. The single buckets are made of clay, sheet or earthen ware and depending on the pressure head directly fixed to the wheel or rotating at an endless rope or chain (Fig.8).

By measuring at a Novia-lifting-system, an efficiency of $\eta = 0.6$ was calculated.

Table 4: Discharge depending on geodetic head for single draught animals

capacity from table 2 + 3	unit	donkey	mule	ox	normal horse	strong horse
capacity from table 2 + 3	W	110	265	383	397	638
efficiency	1	0,6	0,6	0,6	0,6	0,6
transported fluid at h_{geo} =						
1 m	l/s	6,7	16,2	23,4	24,3	39,0
2 m	l/s	3,4	8,1	11,7	12,2	19,5
3 m	l/s	2,2	5,4	7,8	8,1	13,0
5 m	l/s	1,3	3,2	4,7	4,9	7,8
8 m	l/s	0,8	2,0	2,9	3,0	4,9
10 m	l/s	0,7	1,6	2,3	2,4	3,9
15 m	l/s			1,6	1,6	2,6

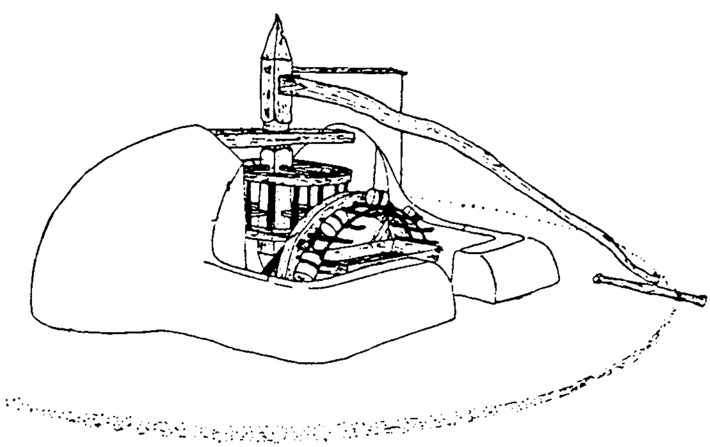

Figure 9 Noria in Dodoma region

As for the construction of whim gin, pull rod and wheel with spokes, tropical iron wood shall be used to be protected against termites. The buckets shall be calcined of clay as native women are able to do this by tradition. When calculating with h_{geo} = 5 m, for example, a lifting time of 8 hours and two oxen for draughting you obtain water available to irrigate 3.4 ha. For this reason you have to strengthen the whim gin according to this pull and to take more bucket strings.

In Dodoma region there are many oxen one can use as draught animal. Many people keep these animals for cultural reasons or these animals represent property and they don't keep them for meat production. These animals are used to the climatic conditions and to the feed in this region. To care for neats the Wagogos have good experience by tradition. You cannot use every neat for draughting, because these animals are not used to this work. But there exists a special program to train oxen for draughting. According to this program, draught oxen available shall be used for the groundwater-lifting-systems. Therefore, you can expect a a relative great availability of draught oxen and a high delivery of water.

All materials for the construction and maintenance can be found in Tanzania. By using buckets of clay for lifting you will use existing traditional know-how and dexterity. To construct a whim gin, a wheel with spokes, the bow with buckets and the dug well, it will be possible in this region, you only have to transmit the know-how required. There is no ecological infringement like for example contamination of groundwater by incorrect treatment of Diesel oil.

The use of neats for water lifting will lead to a change in the traditional partition of work as women never before cared for neats. That's the same for men in water supplying. By using buckets of clay, there shall remain a constructive part in water supplying for women. The costs of construction for one of this groundwater-lifting-systems will be 5.000 Tsh (approx. 1.350 DM) for a working time of 10 - 15 years. According to the Water Development Plan, it is suggested to construct the water-lifting-systems by self-reliance. For this reason, a local industry in coordination with a mechanical engineer, a master craftsman, a smithy, a joinery and a machine shop shall be installed. The Novia-lifting-systems can be completely built in this factory, so that you will have new jobs.

For small irrigation systems the Novia system is a suitable groundwater-lifting-system for the Dodoma region.

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Paper 19

Piped Water Supply to Low Income Households

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ABSTRACT.

Households with an income of less than US \$ 80 (1976 level) per capita per annum are often regarded as being unable to pay for a private water connection and generally public taps are considered the only appropriate means to provide water for this category.

A socio-economic survey conducted in six medium sized cities in Indonesia revealed a clear preference for private connections over public taps, however, also in the low income categories.

Experience with a single-tap private connection, the yard connection, that was extensively used in the city of Bogor, proved this type of connection to be very successful, while the percentage of unpaid water bills was not significantly higher for the low income groups using these connections than for other income groups.

Moreover, a cost comparison between yard connections and public taps with and without a watchman showed the yard connection to be an attractive solution, for the consumers and the water company both.

INTRODUCTION.

In 1977 the technical and financial feasibility of piped water supply to the population of six medium sized cities, located on three islands in Indonesia was studied. The cities varied in size from 28 000 to 143 000 inhabitants and the combined population represents nearly 2% of the 25 million Indonesians living in cities of more than 10 000 inhabitants. The estimated population growth hovers around 3% per annum, with exception of the largest city, where extremely densely populated built-up areas hardly leave room for new housing areas.

The master plans were aimed at metered supply in urbanized areas with a minimum population density of 40 inhabitants per hectare. Connected consumers should pay for the new waterworks, but interest free equity participation (28% of capital costs) was allowed to maintain reasonable tariffs: US \$ 0.10/m^3 for small consumers (e.g. less than 10 m^3/month) and US \$ 0.20/m^3 for larger quantities. These rates were combined with fixed monthly fees of respectively US \$ 0.25 and US \$ 0.50.

In the existing situation obsolete and intermittantly working public supply systems in some of the cities served a small proportion of the population. Most people relied on local springs, heavily polluted rivers and in particular on shallow wells, the latteroften located inside the house or yard.

A socio-economic survey revealed that household incomes varied from less than Rp 10 000 (month (US $ 25) to RP 80 000 and more; the average being Rp 39 000/month (US $ 94) or US $ 161 per capita per annum, so slightly above the Indonesian average at that time.

People said to be prepared to spend 2-6% of their income on piped water, an insufficient basis to determine the willingness and the capacity to pay for a private connection. In developing countries public taps or standpipes are often provided to serve the lower income groups. In this study the analysis could be carried one step further, both from a theoretical point of view as well as based on facts in the real world. These facts were provided by the users of a new metered supply system, in operation in a similar township, serving large parts of the city including low income areas.

METERED SUPPLY TO LOW INCOME HOUSEHOLDS.

In Bogor, near Jakarta on Java metered water supply was in operation since 1½-2 years and the application for connection had been highly successful, in particular for the low cost "yard connection", a single supply point inside the yard.

The relation between income and water consumption proved to be significant when established on a per capita basis and it was found that 24% of the households with incomes that were previously considered too low for a private connection to the piped system did have a yard connection and paid for it according to realistic, cost covering tariffs.

The survey in Bogor came as an addition to the surveys in the six cities and the Bogor sample of 111 interviews could not be considered directly representative for the whole city. However, the fact that 58% of the total population was connected and its similarity to one of the six investigated cities in aspects like household income and size made the interpretation in the following table a probable one.

Relation between income and water consumption.

		income per capita per annum (US $)			
Real figures total		less than $80	$ 80-200	more than $200	total
Number of households	(units)	25	50	36	111
Average water consumption	(lcd)	70	95	135	102
Ditto excluding leaks & waste	(lcd)	68	83	119	91
Standard error of mean	(lcd)	7	7	7.5	-
Interpretation total					
Percentage of households	(%)	24	38	38	100
Water demand	(lcd)	70	95	135	102
Connected households	(%)	30	55	80	58
% of income spent on water	(%)	4.8	2.6	2.1	3.1

Households with an income (1976) of less than US $ 80/capita/annum are often regarded as being unable to pay for a connection.

Therefore, in many countries it is accepted policy to provide safe, piped water to lower income groups through public taps. The reasoning usually being that it is a low cost solution and consumption is limited by the carrying capacity of the people and therefore also a low cost solution for the water company.

The socio-economic survey in the six cities revealed a clear preference for private connections over public taps. The expressed willingness of the low income groups to pay for a yard connection would appear somewhat unrealistic in view of their financial capacity. In Bogor, however, it was found that approximately 25% of the lower income group applied for - and effectively got - a yard connection.

Approximately 75% of the connected low income households managed to pay the one time connection fee of Rps 10 000 (or US $ 25), so 25% unpaid connection fees. The fact that the percentage of not-yet-paid connection fees in the middle income group was nearly as high indicates that this is at least partly due to a rather tolerant attitude of the water company in Bogor.

With a view to unpaid monthly water bills the situation in Bogor was found to be more problematic. The typical monthly water bill represented 4.8% of the monthly income of the household. The percentages of the connected households with unpaid connection fees and unpaid water bills are indicated in the following table.

Unpaid connection fees and water bills
(percentage of 111 interviewed households)

income category	unpaid connection fee	unpaid water bill(s)
low	24%	40%
middle[1])	22%	28%
high	3%	8%
average	16%	24%

[1]) middle income group: $ 80 - $ 200 per capita/annum.

Though the sample of 111 households was not representative for the entire city, the results indicate that the majority of the low income households manages to cope with the costs of a connection and the cost of water. The obligation to pay was not pushed very hard by the water company as some households had accumulated unpaid bills for six months, while regulations prescribe to cut off supply after one month. The preference for piped water at the price of nearly 5% of income led to a reconsideration of the argument that public taps are a low cost solution from the viewpoint of the water company.

COMPARATIVE COSTS OF PUBLIC TAPS AND OF YARD CONNECTIONS.

The real cost price of water, including capital costs, varied from $ 0.20 to $ 0.30. For social reasons a stratified tariff is proposed which includes a considerable element of cross-subsidization. The same is true for the connection fee for yard connection and house connections. A cost comparison between a yard connection and a public tap, the latter with and without a watchman is presented in the following table.

Cost comparison of yard connection and public tap, per household.

Description	unit	yard connection	public tap serving 120 people 1) with watchman	without watchman
Average household Consumption	l/c/d	55	20	20
Monthly consumption	m^3/month	11.6	4.2	6.83 2)
Real costs:				
- of water	Rp/month	1160	420	683
- of connection	Rp/month	414	414	414
- of watchman	Rp/month	-	700	-
Total real cost	Rp/month	1574	1534	1097
Tariff to consumer	Rp/m^3	40/80 3)	125 4)	-
Expenses by consumers				
- connections	Rp/month	104 5)	-	-
- water	Rp/month	528	525	-
Total expenses per household	Rp/month	632	525	-
Subsidy per household	Rp/month	942	1009	1097

1) 120 people, or 17.14 households of 7 people each
2) 25% losses of water in case of public tap without watchman
3) Rp 40 for first 10 m^3/month
4) Rp 5 per 40 litres
5) connection fee 25% of real cost.

It follows that total costs are highest in case of the yard connection. However, it delivers almost a triple quantity of safe water and results in improved health of the low income groups. The subsidy per household is lowest in case of the yard connection and people are paying approximately the same as in case of the watched public tap. Only the unwatched public tap would be free of charge to the consumer with a considerable risk that losses are much higher than the assumed 25%.

For these reasons the yard connection was adopted. Public taps will be installed only as long as the final capacity of the water network is not yet fully utilized.

The case of the experience with metered and paid water supply to low income groups in urban areas indicates that piped water supply has a high priority in the budget of low income groups. This story constitutes an argument to comply with these preferences instead of judgement from outsiders.

DISTRIBUTION DE L'EAU PAR CONDUITES AUX MENAGES DE FAIBLES REVENUS

par Sippo F. Postma,

Résumé :

On considère généralement que les familles ayant un revenu inférieur à 80 U.S. $ (niveau 1976) par personne et par an ne peuvent payer un raccord d'eau privé et, que les robinets collectifs publics constituent le seul moyen approprié de distribution d'eau à cette catégorie de personnes

Une étude socio-économique sur six villes de taille moyenne en Indonésie montre cependant une préférence marquée pour les raccordements privés plutôt que les robinets collectifs, y compris les catégories à faibles revenus.

Les expériences faites en matière de raccordement privé à un robinet unique, - raccordement à une cour -, notamment utilisé dans la ville de Bogor, ont confirmé le succès important de ce type de raccordement. Le pourcentage de factures pour l'eau qui ne furent pas payées n'était guère plus élevé pour les groupes à faibles revenus utilisant ces raccordements que pour les autres catégories de revenus.

De surcroît, une comparaison des coûts du raccordement de cour et des robinets collectifs publics avec ou sans gardien ont montré que le raccordement à une cour était une bonne solution, à la fois pour les consommateurs et pour les services d'eau.

Paper 20

House Installations in Urban Developments— Use of Modern Materials in an Acceptable Way

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Introduction

The ultimate aim of a national sanitation policy will be an acceptable hygienic watersupply all over the country and at an acceptable price level. Methods of watersupply by means of public standposts or even yard connections have to be considered as interim solutions, however several circumstances can make public standposts for the time being as the most feasible solution, especially in rural areas and scattered urban zones.

House connections seem to be, where possible, the opportune target not only from a sanitation point of view but also from a socio-economic point of view. At one hand the rentability of a watersupply company relies on the most assured possibilities of income from waterbills. At the other hand, the standard of living and the self-respect of the people are increasing with the availability of a house connection. The responsibility of a consumer is growing when he is more and more convinced of the relation between potable water, sanitation and costs. But there is more than talking about house connections only. For the watersupply companies their system terminates at the stopcock of the house connection; so there seems to be no special interest in the house installation, left alone the rather probable leakage and wastage of potable water and the prevention of backsiphonage.

Review of problems with house installations

It is reasonable to see the different components of watersupply systems in relationship with view points as investments and responsibilities. Within the layout of a watersupply system, excluding house installations, the transport and distribution part already takes 50% or even 70% of the total investments. Regarding planning the total sum however, this may be influenced by the application of different ways of distributions, i.e. with public standposts or with house connections.

Expressed in investments per capita (1) this is shown in table 1, which indicates as an example that the investment costs per capita for a house connection is at least three times the investment costs for a public standpost.

Table 1 - Total costs per capita of different types of watersupply and sanitation (1978 US $)

		Urban	Rural
Watersupply	House connections	120	150
	Public standposts (with storage and minimum distribution)	40	40
	Hand pumps	-	25
Sanitation (waste water and excreta disposal)	Full water borne sewerage	250	250
	Septic tanks	100	-
	latrines	30	20

With reference to house installations - and one can only refer to house installation in case of the existance of house connections - some remarks can be made. House installations as they are, can be completely different and cover a large range between simple systems (with short service pipes, few fittings and only one tap and sink), and advanced systems (with a multi-tap installation, toilets, kitchen-, laundry- and bathroomequipment with an adequate full water borne sewerage system). The costs of the indicated, complete kind of total house installations can be illustrated by some U.K. figures (2) in table 2, which only gives an impression of the differences which are significant in the western world.

Table 2 - Average investment costs per house (January 1975 prices in £)

Watersupply	Watertreatment, pumping and conservation	£ 140
	Water mains	£ 550
	House connections	£ 100
House installation*	Watersystem	£ 700
	Sewerage system	£ 300
Sewerage	House drains	£ 250
	Public sewers	£ 1200
	Sewage treatment	£ 200

*estimated figures

The individual investment costs in this table of the house installation and the in-house sewerage system have been compared with some other components of the watersupply and sewerage systems.

The costs of the house installations in comparison with all other components are considerable.
Although the exact costs of a particular house installation are depending on the level of installation comfort in the house, the costs can be expected to be considerable even with the most simple solutions

In view of the responsibilities the watercompany is bound to control the whole, excluding the house installation. Furtheron, the watercompany has the task to inspect the house installation, while the consumer has the responsibility for it. This includes that the costs of house installations are for the account of the individual consumer.

In view of the financial impact of house installations it may be clear that a watercompany has at least a moral responsibility in the field of technical assistance and education regarding the planning and design and the operation and maintenance of house installations.
It may be expected from a watercompany to have the capability to design a house installation, to act as an advisor or as an supervisor.
The watercompany has the know-how on materials and hydraulics, which may not be expected from the consumer.
So, the consumer can only act as the purchaser, i.e. the buyer of the different elements of a house installation. Nowadays, the consumers in developing countries willing to complete their house installations are bound to buy on the free market, expensive and sometimes inadequate installation elements like taps, sinks, wc-pans, bathroom-equipment etc..
These installations are often modelled on those encountered in the Western industrialized countries, were high water pressure and expensive living quarters form a prime consideration in demanding a high safety level for sanitary installations.
The equipment which is finally installed is in general not adapted to each other, has no warrant for quality, is sometimes inferior (even with high costs) and is available in a rather excessive and unnecessary choice.

It may be concluded that the materials and equipment are not appropriate and certainly not appropriate to the available budgets.

A further conclusion is that in general the efforts in optimizing the house installation are of an extremely difficult nature. This situation is the result of e.o. too vague and passive responsibilities in this field. It is clear that, in view of the enormous amount of money which is involved, there is a task for the different national governments and also a task for the watercompanies, which is delegated to them by these governments.

Consideration of possible policy on house installations

To set up a policy to improve the indicated situation one has to refer to an appropriate technology.

Appropriate technology can be described generally following a W.H.O.-document (3):

- The word "technology" means not just a device but any association of techniques, methods and equipment which together contribute towards solving a problem.
  "Appropriate" means that the technology is not only scientifically sound but acceptable to users, providers and decision makers and the like, that it fits within local cultures and that it is capable of being adapted, further developed and manufactured locally where ever at low costs.

Some basic criteria can be conducted from this definition:
- Products in this context should be made locally, of locally available materials and at an acceptable quality level.
- Designed house installations should be acceptable in an economical, cultural and social way.

These criteria regarding the design of house installations and local manufacturing of elements may be considered in more detail.
In developing countries a large part of the population lives in rather modest housing where occasional water leakage would not result in substantial material damage. Under such circumstances less costly materials and sanitary equipment could be used and the house-owner could install most, if not all of this equipment.

However this requires a rather different approach of the design, manufacturing and marketing of these materials and equipments.

What is needed in the first place are

APPROPRIATE TECHNICAL REQUIREMENTS:

- The requirements on durability of the installed materials has to be based on the expected period of use, which depends on the income situation and the technical life time of a low-budget house.
  An illustration is given in table 3 where the global expectancy of service time (4) of some elements of house installations in the western world are presented.
  The appropriate technical requirements in developing countries of familiar elements may differ from the mentioned figures but will always relate to the relevant expectancy of service time.

Table 3 - Global expectancy of service time of some elements of house installations.

Pipes:	Galvanised steel	20	-	25	year
	Plastics	25	-	30	year
	Copper	30	-	40	year
Taps		5	-	10	year
Sinks, wash-basins		15	-	20	year
WC-pans		15	-	20	year
Flushing cisterns		15	-	20	year
Water meters		5	-	20	year

- The use of standards, which permits during the design phase to be more precise in specifications and gives facilities during the follc wing operation and maintenance phase.

- The uniformity, the use of standard design and availability of prefabricated assemblies (do-it-yourself). The advantage is production in greater series, of a restricted variety of elements. An acceptable quality level with internal and external control may be easily assured.

Technical specifications (quality requirements) should be ready before developing further organizational procedures. These specifications should allow to lead to:

- the development of standard designs. Standard designs have to be developed for different situations with a free choice to some possibilities.
- The drafting of guidelines for local manufacturers. The same specifications for all manufacturers excludes false competition between manufacturers, whilst maintaining the essential quality level of the products.

As far as standards are concerned it should be mentioned that the first aim of standardization always has been an economical one by the elimination or avoidance of useless differentation of products. This differentation of products has been caused by independent production by different manufacturers on different specifications of clients. This aspect can be illustrated by the continuous growing variety of fancy shaped elements for house installations, from taps and washbasins to water closets and bathroomequipment in the western world. This implies a great diversity of products on the relevant market but probably also on every potential market in the world.
The first step in standardization has been as far as possible, the elimination of a lot of different types of a certain product, e.g. the standardization of pipe diameters and lengthes.
The next step, and second target now, can be indicated as fitness for purpose; this has led to certain acceptable testing specifications which may be used as production guidelines for manufacturers at one hand and purchasing guidelines for the clients at the other hand.
A client can be a watercompany but also an individual consumer.
The last one cannot be regarded to be an expert in this, so it may be seen as a general task for the government or a delegated task for the watercompany to oblige manufacturers to produce on such harmonised testing specifications and to inspect the quality of products to be delivered to the individual clients.

Standardization is available for distribution pipes, materials, fittings, accessories for water meters and partly for house installation elements like pipes, fittings, taps, waterclosets, flushing cisterns and similar products.

MATERIALS

There is an immediate need for a full range of all components for house installations e.g. from pipes, fittings and taps until prefab blocks with their components such as wash-basins and shower bathes. This range should contain a limited number of items, although the consumer must be allowed to a competitive choice.
The development of prefab blocks of composition materials (plastics with e.g. gravel), produced at low costs, should be encouraged.
The expectancy on service time of these blocks should be related to the expectancy on service time of the whole dwelling.

Materials, which are available and appropriate for pipes and fittings one could mention:
plastics (PVC and PE) and copper.

Although copper is not a modern material, it has certain advantages in view of its small dimensions, easy handling, strength and high resistance to external damages.
If the specifications for copper are adapted to the considered service time expectancy for developping countries, and by that being lower than the specifications in industrialised countries, the advantage should be considerable, otherwise the same specifications should be used, so that re-use will be possible.
The range of fittings as available on the western market should be reduced considerably as the need is rather small for that variety.

PVC-pipes and fittings are rather cheap and may be manufactured locally; the compounds are available on the international market and the costs of the manufacturing equipment are modest.
The plastic pipes and fittings allow easy handling whilst installment can be done by fairly unskilled labour.

This means providing a good instruction is given.
In this instruction the susceptibility to external damage should be clearly stated.

Polyethylene pipes(PE) have similar advantages as PVC pipes, although the price is a bit higher. Additionnally, the flexibility of PE makes it more appropriate for the use in service pipes.
Local manufacturing is similar to the manufacturing of PVC.

Water borne sewerage systems, essential in a total household system, may use the same kind of materials as used in the house installation. Thus plastics may be used, but in respect of the appropriate technica requirements for a non-pressured system, as for house drains, clay products may be used also.

Financing

From experience it is known that the investment costs for house connections may form a considerable barrier to all income groups to have equal access to domestic water supply.
Most of the potential consumers in urban developments are not able to save or to borrow the money for the investment for the initial house connection. It may be clear that the potential consumers in rur areas, where the income level is generally lower and the investment costs considerably higher, the access to domestic water supply is even more difficult.
The normal practice of watercompanies however, is to charge the full costs of the installation of the service pipe and the water meter at the moment that the domestic supply is realised.
As shown in a recent survey (5), several African countries however, have realised a payment by installments or high or full subsidies.
In areas with good distribution systems in operation, it is also in the watercompanies' interest to encourage the access to the system on a low cost policy.

The establishment of a revolving fund for house connnections is frequently a workable alternative to recover immediately the connection costs.
The customer is then charged with the full costs of the connection by equal monthly installments during one or two years, including an interest at normal commercial rates in addition to the billing for consumption.

A possible solution to cover the costs of house installations, even low cost installations, may be found at a similar way, in which the watercompany acts as an intermediary if necessary.
Another way may be found by integrating the house installation as a standard practice in the construction costs of a new house.
In view of the available workmanship the installation is easy to be optimised in that situation.
The watercompany remains with its inspection task, whilst the investment costs come within the rent.
It may be a governmental policy to have only houses costructed with an incorporated minimum installation.

Recommendations

It is recommended to start the study of the following aspects in relation to the means to reach the objectives.

1. Appropriate technical facilities.
2. Organisational consequences of a policy to increase the number of house installations.
3. Facilities for commercial credit.

With reference to "appropriate technical facilities" the following may be indicated:

1.1 Development of appropriate quality requirements; the design of standard components and the drafting of instruction manuals for do-it-yourself installation.

1.2 Making an analysis of the regional market prospects for recommended products with recommendations concerning the production prognoses best suited to local conditions.

1.3 Development of small and medium-sized enterprises and industries in the field of water supply and sanitation.

Regarding the "organisational consequences" the following needs are stipulated:

2.1 To initiate and control the effects of a policy in this field, starting with a study on responsibilities and delegation of tasks.

2.2 To make use as far as possible of commercial possibilities already in operation.

2.3 To recommend the use of regulations and standards promoting the use of better products for sanitation and water supply consistent with local capabilities.

Regarding the "facilities for commercial credit" the following is recommended:

3.1 To establish a revolving fund for house installation and to use e.g. the regular waterbill to assure the recovery of the costs on a modest commercial basis.

3.2 To integrate low cost house installations into the construction of low cost houses. A basis minimum should be maintained for that installation. The house owner should be free to extensions under the condition that the watercompany deals with this for inspectio and advice.

Summary

In developing countries there is a great need of appropriate house connections and in consequence house installations.
The installations already present are often styled following those encountered in the industrialized countries.
In developing countries a large part of the population has rather modest housing conditions, where the need prevails of materials and sanitary equipment on a low cost basis and where the house-owner could install mostly himself, if not all, of this equipment. This as far as this equipment is not integrated at the construction of the dwellings as a preferred situation.
However, this asks for an appropriate approach in the design, the local manufacturing, the installation instructions and the financing of these installations. This also asks for a national policy, in which at least the government and the watercompanies should be involved.

Some recommendations in this direction are given, which should become operational to form the first steps into a development where the ultimate aim is an acceptable and comfortable house installation in every house.

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Paper 21

Studies on Development and Performance of Fixed Bed Disinfector

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ABSTRACT

A simple and easy to operate and maintain device to disinfect the water in rural areas of the developing countries has been developed. Indigeneously available sorbents like resin, activated rice husk and activated coconut shell were employed along with activated carbon to adsorb chlorine and iodine when placed in disinfectant rich medium and release the same when contacted with water. Chlorine being a very strong oxidising agent irreversably reacted with all sorbents and thus not available for disinfection. Iodine being a milder oxidising agent reversibly sorbed on the sorbents and was released iodine for disinfection. The sorption of iodine on these sorbents can be expressed by Langmuir's isotherm. The release of iodine from iodine loaded sorbents appear to be function of flow rate, disinfectant demand of water and initial iodine loading. The resin disinfector charged with 67 percent of iodine saturation capacity yielded maximum amount of water. Resin disinfector produced more disinfected water based on per gram of sorbent while rice husk disinfector yielded more water per gram of iodine sorbed than others. The turbidity present in influent water did not affect the performance of resin disinfector. On the other hand the turbid water induced production of more disinfected water.

INTRODUCTION

The United Nation's Water Conference at Mar Del Plata, Argentina in 1977 highlighted the importance of community water supply and decided to observe the current decade (1981-90)as 'International Water Supply and Sanitation Decade. Goals of the Decade is to water to all and sanitation to most. The rural population of the developing countries forms a major fraction of their population. In India, for instance over 80 percent of the population i.e.,502.1 millions(as of 1979)live in half million villages. There had been little efforts to supply safe drinking water to the rural population as reflected by allocation of only 9000 million rupees($ 1125 million)during last 30 years while the much smaller urban sector received a larger proportion. Logically the water supply and sanitation pludge by UN is to be directed to solve the mannoth task of supplying water to the rural population. It requires more than 3000 million rupees ($ 340 million) per year for the Decade to achieve this.

In order to achieve economically the aim it is necessary to review and even modify the water quality standards. The primary objective of rural water supply scheme should be protection against transmission of water borne diseases like typhoid, cholera, infectious hepatitis, dysentry and dracontiasis. Physical and chemical qualities of water can be secondary one. Higher values of certain impurities like turbidity, salinity and hardness than those permissible can be allowed since the local population can tolerate such quality through long usage (1). Secondly, it is more important to develop the low cost water technology preferably requiring least skilled operation and maintenance. Kardile (2) had developed sand-coconut dual media filters and non-mechanical flocculators made of packed gravel bed which successfully eliminates the operation and maintenance problems besides being cheapter. According to salome and Venkobachar (3) utilisation of unconventional media like crushed coconut shell where cheaply available results in the operation of filters at a higher rate besides providing longer service time before back wash.

The conventional chlorination systems used in urban water schemes are unsuitable due to prohibitive cost and skilled operation and maintenance. In most water works despite available technical assistance, it is not uncommon to find the chlorinators going out of order after a short while of their installation. Consequently, it is appropriate to develop a simple, cheap, self-regulatory, easily operatable and maintainable device to replace the conventional chlorinators in rural water supply schemes. The same device in a smaller version should be suitable for incorporation in hand pumps to disinfect the handpump waters in villages

METHODOLOGY

In this investigation multitude of materials were evaluated for their potential to sorb the disinfectants like chlorine and iodine when placed in disinfectant rich medium and release the same when water with disinfectant demand is contacted with them. The sorbents investigated were locally and economically available strongly basic anion exchange resin (De Acidite-N), activated rice husk, and activated coconut shell. Activated carbon (Fitra Sorb - 400) had been used as control. The investigation included iodine as a disinfectant though it is costlier than chlorine to avoid the problems associated with its handling particularly in remote rural areas. Moreover, in the hilly areas of north-east provinces of India and Nepal iodinised salt is supplied to reduce the goiter epidemics. Thus, the supply of iodinated instead of chlorinated water would certainly ensure the required dose iodine to the public. The tube well water (dechlorinated) of I.I.T. Campus, Kanpur injected with desired density of *Escherichia coli* served as test water. The concentration of *E. coli* both in feed and treated water was evaluated by plate count employing eosine methylene blue agar (4). The disinfectant residuals were determined by Diethyl-p-phenylene diamine (DPD) sulfate titri-metric method (4). Turbidity of water was determined by Hach turbiditimeter.

The experimental methods included the standard sorption and tests to evaluate the kinetics of sorption and sorption equilibria of above mentioned sorbents with respect to chlorine and iodine. The reaction mixture consisted of known concentration of disinfectant and sorbent. The reaction vessel was agitated in a laboratory shaker. Aliquots were withdrawn from the reactor at frequent intervels for estimation of disinfectant. The equilibrium times obtained from these kinetic runs were used in sorption equilibria tests to evaluate the saturation capacities of various sorbents for chlorine and iodine. To evaluate the parameters influencing the release of disinfectants from the sorbents column desorption studies were conducted. The disinfecting abilities of disinfectant loaded sorbents were found by conducting both batch and continuous system experiments.

EXPERIMENTAL RESULTS AND DISCUSSION

The disinfector contemplated for rural areas containing sorbents should not only have good sorptive capacity for the disinfectants but also disinfecting potential. All the sorbate-sorbent systems investigated were subjected to these two tests. Those fulfilling the above attributes are eligible candidates in the development of fixed bed disinfectors. Experimental results and discussion in this regards are presented herein.

SORPTION

The equilibrium kinetics of iodine and chlorine by resin, activated carbon, activated rice husk is presented in Fig.1 and 2. The saturation time of iodine for resin, activated carbon, activated rice husk and activated coconut shell is about 2, 2, 3.5 and 18 hours respectively. The equilibrium time for chlorine appears to be greater than 24 hours for all sorbents. Due to probable chemical oxidation of sorbents by chlorine there existed always a concentration gradient between chlorine in liquid and solid phases which resulted in the non attainment of equilibrium. The sorption of iodine on resin and activated carbon is very rapid in the initial 15 min indicating instantaneous sorption due to external surface reaction. The sorption rate was much smaller subsequantly. Data on sorption equilibria for different sorbents appear to follow Longmuir's isotherm indicating a monolayer deposition. The linearised form is

$$\frac{C_e}{q_e} = \frac{1}{b}\,\frac{1}{Q^o} + C_e\,\frac{1}{Q^o} \qquad (1)$$

where C_e = concentration of iodine remaining in solution at equilibrium.

Q_e = weight of iodine sorbed per unit weight of sorbent at C_e.

Q^o = weight of iodine sorbed per unit weight of sorbent to form a monolayer (saturation capacity)

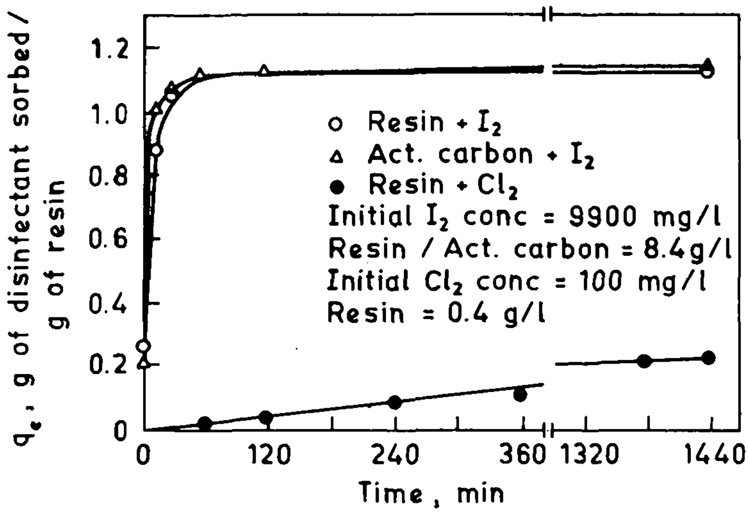

Fig. 1. Kinetics of sorption of iodine and chlorine on resin.

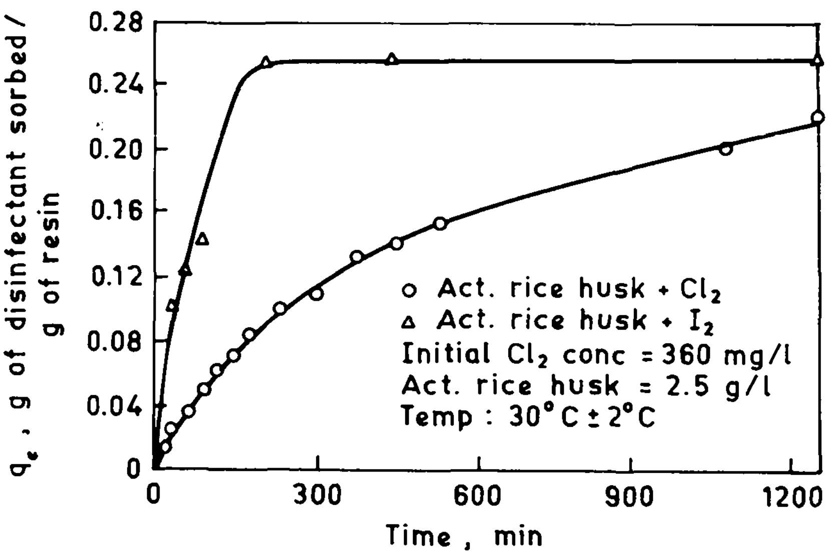

Fig. 2. Kinetics of sorption of iodine and chlorine on act. rice husk.

b = constant related to enthalpy of adsorption

The linearised plot of Langmuir's isotherm for resin-iodine system is presented in Fig.3 as a sample. The saturation capacities of iodine for resin, activated carbon and activated rice husk are respectively 1.25, 1.06 and 0.32 gram of iodine per gram of sorbent. The active surface areas responsible for this sorption can be calculated using the expression

$$\Sigma_s = Q^o N_{av} \sigma^o \qquad (2)$$

where Σ_s = active specific surface area

N_{av} = Avogadro's number

and σ^o = area of iodine molecule (24 sq.Å)

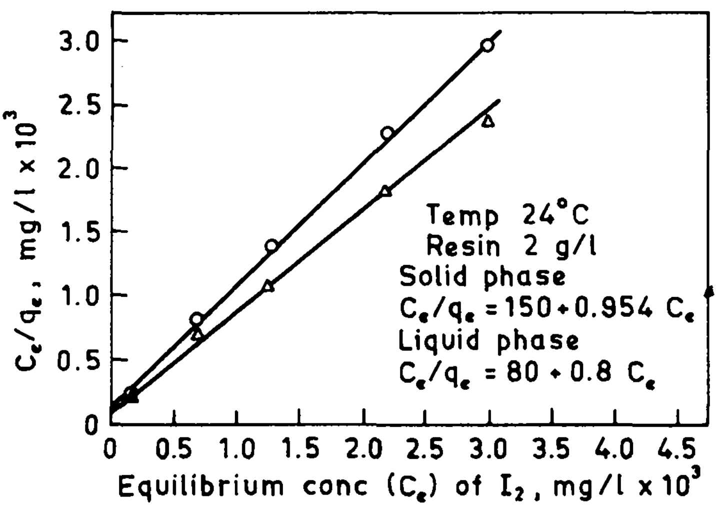

Fig. 3. Linearised form of Langmuir isotherm for resin.

The specific surface areas for resin, act carbon and act rice husk are 710, 585 and 182 sq.m/g respectively. These are orders of magnitude in excess of respective surface areas of sorbents. This tends to indicate presence of internal pores for diffusion of iodine particularly more in resin and act carbon.

DESORPTION

An important attribute of sorbents of fixed bed disinfectors is controlled release of disinfectants when water is passed through them. The batch desorption studies indicated chlorine could not be detected from chlorine loaded sorbents while iodine was released. Chlorine being a strong oxidising agent has probably irreversibly reacted with sorbents whereas iodine has not. Column desorption studies were conducted on iodine loaded sorbents to study the effect of parameters like flow rate and disinfectant demand of water. The experimental step up for this investigation is presented in Fig. 4. The release of iodine as a function of flow rate for both resin and activated carbon is shown in Fig. 5. More iodine was released with the increase in flow rate upto 20 $M^3/M^2/$ beyond which there was a decrease both for act carbon and resin. Both turbulance due to increased flow and contact time appear influence the desorption of iodine. During the initial stages increased turbulance rather than decreased contact time induced more iodine release. However beyond the threshold flow rate, significantly decreased contact time rather than much more increased turbulance is an important factor. Through the columns, waters containing various concentrations of disinfectant demanding chemical (arsenious trioxide) were passed to assess the response of the disinfectors to such situation. Iodine concentration in the effluents water from the disinfector increased linearily with the increase in concentration of arsenious trioxide in the influent water. The fact that the disinfectors can release iodine on demand is confirmed.

DISINFECTION

The most important step is to evaluate the disinfection potential of these sorbents. Batch tests with chlorinated and iodinated sorbents using test water containing 10,000 E. coli /ml indicated 100 percent inactivation bacteria with iodinated sorbents within few seconds as against no inactivation with chlorinated sorbents even after prolonged contact time. It appears that chlorine has irreversibly reacted with sorbents and hence not available for bacterial inactivation. Number of columns containing iodinated resin, activated carbon, activated rice husk and activated coconut shells charged with 1, 1, 0.24 and 0.16 grams of iodine per gram of sorbent were prepared to receive the turbid free test water containing 10,000 E. coli/ml at a flow rate of 16.0 $m^3/m^2/hr$ (25 ml/min). Control columns containing uniodinated sorbents were employed to assess the removal of bacteria by filtration. The data on service time is presented in Fig. 6. The bacterial breakthrough occured as soon as residual iodine could not be measured in the effluent. The production of bacterial free water for various sorbents is given in Table 1.

TABLE 1. Yield of Potable Water from various Disinfectors

Disinfector	Depth (cm)	Contact time (sec)	Total amount of Iodine sorbed (g)	Volume of potable water (l)
Resin (5 g)	10.0	24	5.00	150
Act. carbon(2.5g)	15.0	12	2.50	45
Act. rice husk (5 g)	21.6	52	1.20	50
Act. coconut shell (5 g)	10.0	24	0.82	10

It is evident from the above table that resin disinfector produced more water than others on the basis of per gram of sorbent used in the disinfector. Activated rice husk produced more water per gram iodine adsorbed on any sorbent. This is due to the longer contact time (52 sec) available between the bacteria and iodinated rice husk. The longer contact time is due to more space occupied by lighter rice husk though in terms of weight it was identical with resin and act coconut shell. Considering both iodine sorbed and contact time resin, act. carbon and act. rice husk can serve as candidates for the development of fixed bed disinfector while activated coconut shell is much inferior. Rice husk occupies more space and consequently produces more head loss where as resin and activated carbon consume more iodine for the production of equal volume of water incurring lower head loss.

The Fig. 6 indicates excess release of iodine than required which imparts taste and odour to water above 1.0 to 1.5 mg/l (5) during the initial stages. In order to reduce the excessive iodine release and increase the volume of water produced per gram of iodine sorbed on resin, initial iodine loading on the resin was decreased. The result on iodine release per different amounts of iodine adsorbed per gram of sorbent is presented in Fig. 7. The iodine loading is expressed as q/Q i.e. amount of iodine on resin (q)/practical maximum capacity (Q). It is interesting to note that the time required for disappearance of iodine from disinfectors with $q/Q=1$ and 0.67 is almost same. The dose iodine released for 0.67 is also sufficient to inactivate E. coli. This strongly suggests that charging the resin beyond $q/Q = 0.67$, besides being not economical produces more complicated taste and odour problems. The same 150 litres of water could have been produced by resin disinfector as indicated in Table 1, even if it were charged with 3.3 grams of iodine instead of 5.0 g. Thus, this makes resin slightly more attractive proposition than rice husk.

Figure 8 evaluates the quantities of disinfected water produced for different initial iodine loading 2 g of resin portions loaded with different iodine concentrations were placed in the columns through which water containing E. coli (10,000 Ne/ml) was passed at 16.0 $m^3/m^2/hr$ yielding a contact time of 10 min. There is a steep increase in quantity of water produced upto $q/Q = 0.67$ beyond which

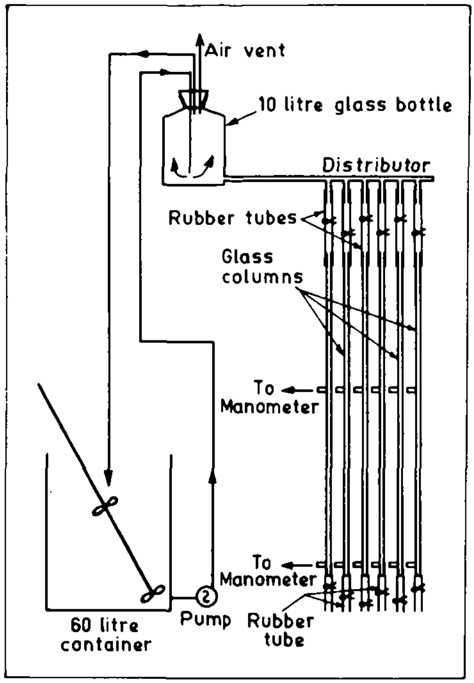

Fig. 4. Experimental setup.

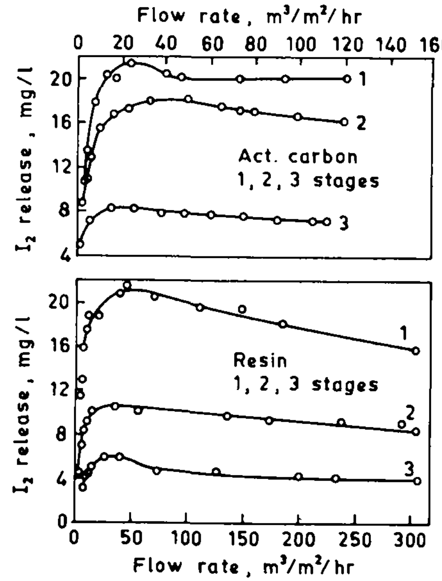

Fig. 5. Release of iodine vs flow rate.

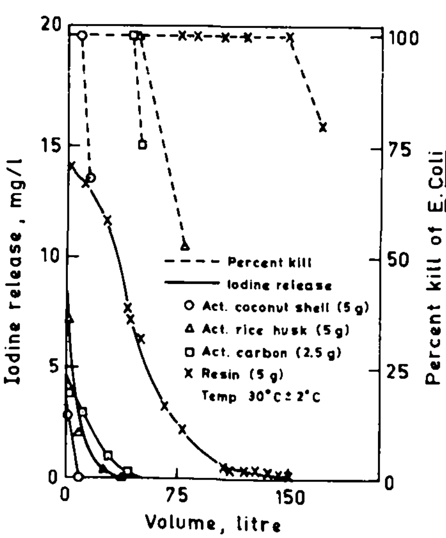

Fig. 6. Plot of iodine release and percent kill vs volume of water disinfected.

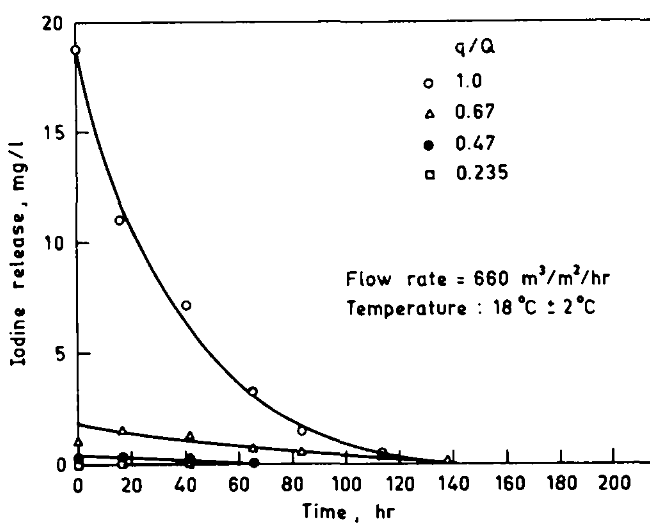

Fig. 7. Release of iodine with time.

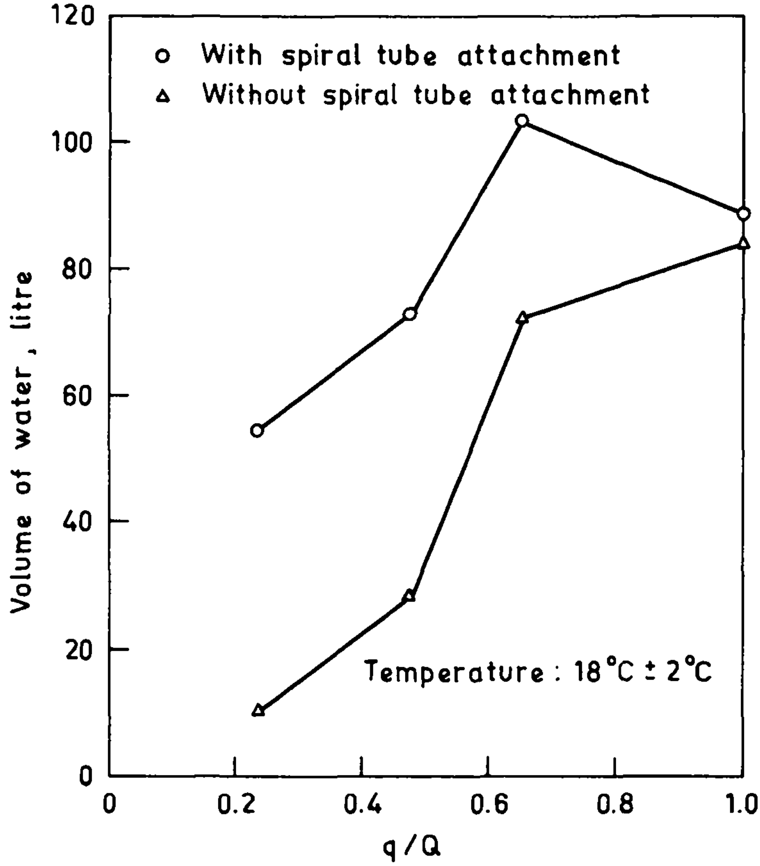

Fig. 8. Volume of water disinfected vs extent of iodination of resin.

it decreases. This confirms that optimal iodine loading is 67 percent maximum iodine sorption capacity of resin should be adopted. In order to further increase the service time the water from the disinfector is passed through a spiral tube extention of of 1 meter to provide an additional contact time between released iodine and bacteria. The service of time of the disinfector is maximum for $q/Q = 0.67$ for this also.

The potential of disinfector to handle turbid waters containing bacteria was evaluated. Test waters containing identical bacterial concentration and different turbidities namely 8-16 NTU and 33-55 NTU were passed through identical resin disinfectors and the results are presented in Fig.9. The production of water was 200 litres and 320 l without and with spiral attachment respectively. An identical disinfector receiving turbid free water without spiral extention produced only 150 litres. It is interesting to note that more disinfected water could be produced when influent had turbidity. This appears to be due to light coating around the resin grains by the retained turbidity within the disinfector. This was, however, confirmed when a slight increase in loss of head was observed due to small fraction of turbidity retained. In rural

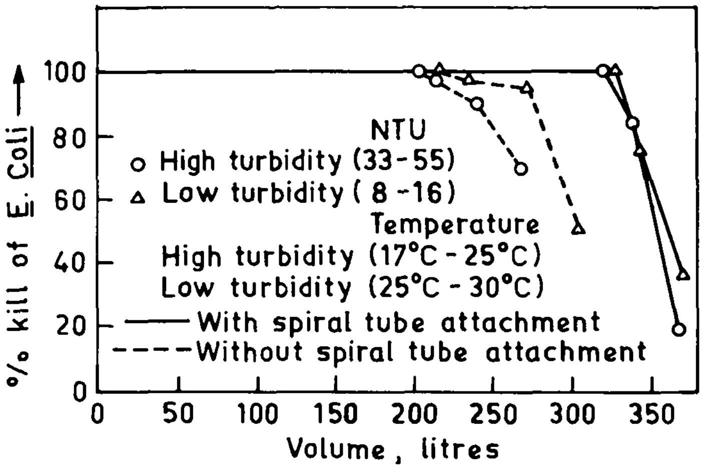

Fig. 9. Performance of iodinated resin disinfector handling turbid water.

water supply schemes as bacteriological quality is more important than physical quality, presence of turbidity in excess than permissible can advantageously be used to produce more disinfected water than in the absence of turbidity.

Further investigation to assess whether inactivation of bacteria by disinfector is due to direct contact between iodinated sorbent and bacteria or due to contact between released iodine and bacteria was conducted. The inactivation due to direct contact can be evaluated by estimating the frequency of contacts between sorbent grains and bacteria. This can be done using the principle of filtration. Assuming diffusion and interception as dominent transport mechanisms by means of which bacteria establishes contact with resin grain and calculating the single collector efficiency using filtration fundamentals, it was shown that the contact rate between bacteria and resin is only 17/sec. while actual bacterial flux rate through the column is 417/sec. The detailed analysis of this is presented elsewhere (6). It clearly indicates that the major contribution towards inactivation of bacteria is not the direct contact but contact between released iodine and bacteria.

CONCLUSIONS

Based on this investigation the following conclusion may be drawn.

1. Among chlorine and iodine, only iodine is suitable to develop fixed bed disinfector with sorbents like De Actidite-N resin, activated carbon, activated rice husk and activated coconut shell. Chlorine irreversible reacts with these sorbents and hence not available for disinfection.

2. Sorption isotherm follow Langmuir's equation with saturation capacities of 1.25, 1.06, 0.32 of iodine for resin, activated carbon and activated rice husk respectively.

3. The iodine release is a function of flow rate, disinfectant demand of water and initial iodine loading. The iodine release from resin disinfector charged to 67 percent its iodine saturation capacity is found to be optimum.

4. The batch disinfection studies using iodinated sorbents showed instantaneous inactivation of E.coli while no kill was observed with chlorinated resins.

5. The resin disinfector produced more disinfected water per gram of resin than others while rice husk disinfector produced more water on the basis of per gram iodine sorbed on it.

6. The resin disinfector charged to 67 percent iodine saturation capacity yielded more water than at other iodine loading.

7. Water containing bentonite turbidity upto 55 NTU can be disinfected without any problem. On the other hand slight turbid of influent water helped to control to release iodine from the disinfector and thus produced more disinfected water.

8. The inactivation of E.coli is due to released iodine in the disinfector rather than the its contact with iodinated resin.

9. The iodinated rice husk disinfector is useful when there is no limitation on space and initial available head is more while resin disinfector is attractive when only compact space and low operating heads are available.

FURTHER WORK

While this paper is being prepared the investigation on both theoretical and practical aspect of fixed bed disinfectors is continued. Instead of activated rice husk, raw rice husk is investigated interms its ability to yield disinfected water when iodinated to different degrees. Utilisation of Coconut shell to achieve both filtration and disinfection in the same reactor. The theoretical aspects include the mass transfer phenomenon occuring in resin disinfector and the role of retained turbid particles in the disinfector on iodine release. Beyond this, efforts will be directed towards developing suitable sorbents for chlorine sorption. It is hoped the authors would be able to update the finding at the time of presentation.

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Etudes sur la Mise au Point et le Rendement d'un Désinfectant a Lit Fixe.

RESUME

Pour réaliser les buts de la Décennie de l'Assainissement et de l'Alimentation en Eau des Nations Unies, il est nécessaire de développer une technologie de l'eau à bas prix et de modifier les standards de qualité de l'eau. Des essais furent entrepris pour développer un dispositif simple de désinfection de l'eau afin de la protéger contre les maladies en provenance de l'eau dont souffrent les populations rurales dans les pays en voie de développement. Le système de désinfection proposé devrait pouvoir remplacer la chloration, plus coûteuse, dans les projets d'alimentation en eau des campagnes, ce système ètant incorporé dans les pompes à main afin de produire une eau potable.

Différentes résines disponibles localement, les balles de riz, des coques de noix de coco ainsi que du charbon actif importé furent examinés et leur capacité d'adsorbtion des désinfectants comme le chlore et l'iode comparées. L'adsorbtion de l'iode s'exprime ici suivant l'isotherme de Langmuir.

Le chlore est fixé irréversiblement par les adsorbants et n'est donc pas libéré au contact de l'eau, tandis que l'iode est facilement libérée. La libération de l'iode des adsorbants semble être en fonction de la vitesse du flux, du niveau d'infection de l'eau et de l'iode présente initialement. Dans les études de désinfection intermittente les adsorbants iodés se sont montrés supérieurs aux adsorbants chlorés en ce qui concerne la stérilisation des E.coli présents dans l'eau étudiée. Parmi les désinfectants iodés, celui qui contenait de la résine a permis de traiter plus d'eau par gramme d'adsorbant que d'autres tandis que l'assainisseur avec balle de riz traitait plus d'eau par gramme d'iode adsorbée. Le système à base de résine s'est avéré le plus efficace lorsqu'il ètait à 67% de capacité de saturation en iode plutôt que lorsqu'il ètait rempli a 100%. La turbidité présente dans l'eau n'a pas affecté la performance du système à base de résine. Par contre l'eau a forte turbidité gène la désinfection en agissant sur la libération de l'iode. Le mécanisme de stérilisation des E.coli semble venir de leur contact avec l'iode libéré plutôt que de leur contact avec la résine iodée.

Paper 22

Scope and Limitations of Disinfection of Water by Ultra-Violet Radiation

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1. INTRODUCTION

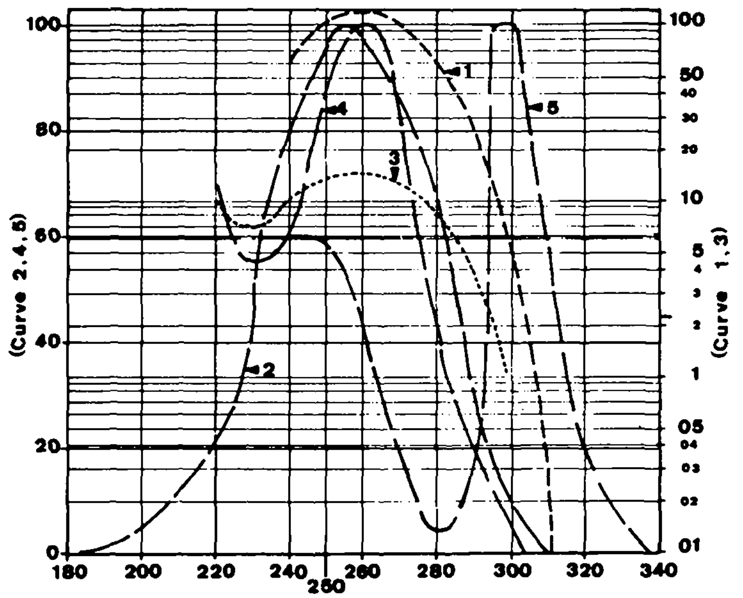
1.1. The bactericidal effect of radiant energy from sunlight was first reported in 1877 (1). Ultra-violet (U.V.) radiation is part of the sunlight which reaches the earth's surface and is merely confined to wavelengths higher than 290 nm, this means that the most energetic wavelengths are absent (2). The highest bactericidal efficiency is actually recognized to be obtained between 250 and 260 nm with a maximum at 255 nm.

1.2. The action of U.V. radiation in water should not be based on the formation of ozone which requires quanta, corresponding to wavelengths lower than 220 nm and dry air or oxygen (3). Although the wavelength of 254 nm is widely considered as the most efficient, the optimum value may vary in function of the type of organism (4). U.V. sources, when immersed into water can produce hydroxyl and peroxy-radicals which have in certain circumstances been advanced as the disinfecting agents. Iron salts, according to the theory of NOFRE, could have a catalytic action (5, 6).

The efficiency of disinfection with U.V. light may be affected by dissolved organic compounds. This is particularly the case of amino-acids which inhibit the formation of free radicals (5) at concentrations of 3.5 g/m^3 and higher, as well as an efficient disinfection (7).

The most advanced theory of the bactericidal action of U.V. light supposes that the photochemical alteration of the DNA hinders the bacteria from reproducing. The DNA absorption spectrum is similar to the bactericidal efficiency as a function of the wavelength *(see Figure 1)*. Different structual changes induced by U.V. light are reported in the literature (9), the most frequent are the formation of a thymidine-dimer structure, DNA leakage and cross-linking. These interactions are believed to be the most important causes of death by U.V. radiation. However, the hydroperoxide-oxidation is apparently more compatible where bacterial revival is encountered and which is sometimes observed.

Figure 1 : Typical U.V. spectral data (1) Mercury lamp emission; (2) relative germicidal action; (3) DNA-absorption; (4) spectral sensitivity for conjunctivitis; (5) spectral sentivity for erythema.



1.3. An action worth being mentioned is the synergic effect in disinfection by U.V. radiation, by H_2O_2 (10) and also by U.V. and ozone (11). However the method still required further development for large scale applications; moreover it remains expensive.

1.4. In some instances, bacterial aftergrowth can take place (6) as is the case with ubiquitous germs. This effect is similar yet less abundant than in the case of aftergrowth obsereed in ozonized waters. In U.V. disinfection, the effect has been attributed to growth-stimulating necrohormones.

1.5. Although U.V. disinfection is not a specifically low-cost technology, its general merits and drawbacks can be quoted as follows :

- with the doses applied in disinfection, there is no appreciable change of chemical composition or properties of the water during the application;
- <u>the equipment demands very little attention, maintenance is simple and does not require especially skilled staff</u>;
- installation is simple, automatisation largely feasible and control facilities easily built in;
- <u>there is no need of supply, stock or handling of chemicals, consequently the potential hazards of these operations are eliminated</u>;
- there is no long-term residual germicidal effect and the action required appropriate design and operation;
- the renewal of the lamps determines an important part of the operational cost of the process; whence for "economical" reasons this operation is sometimes neglected;
- the control of the efficiency of the process cannot be based on a residual action and must be immediate when in current operation;
- although less relevant, professional risks associated with the use of U.V.-irradiation are those of erythema (maximum at 296 nm) and conjunctivitis (maximum risk at 260 nm) *(see Figure 1)*. Goggles with normal glass are adequate for the protection of the eyes and the sources are best not directly visible. The TLV for maximum daily irradiance at 254 nm, expressed in mW/m^2 are 8h : 2 - 2h : 4 - 1h : 16 - 0.5h : 34 - 0.25h : 66 - 0.082h : 200.

1.6. This contribution tends to give accommodations formulated for design as no official general rules have been formulated to this day by Health Authorities. On the other hand, the existing literature is often related to one particular (commercial) equipment. The method is undoubtly efficient but may sometimes fail through incorrect operation or inadequate design. The most important items for design are marked as follows : •

2. TYPES AND AVAILABLE SOURCES OF U.V. LIGHT

2.1. Emission yield

2.1.1. The most used available sources are the low-pressure mercury cathode lamps with a vapour pressure in the order of 10^{-2} - 10^{-3} mbar. The emission spectrum *(see Figure 1)* is most important in intensity at the resonance line of 253.7 nm ($\geqslant$ 98 %) in the vicinity of the frequency of the most thorough bactericidal action (i.e. a monokymatic source).

Lamp types exist in the hot cathode type and in the cold "incandescent type". The hot cathode type requires a starter to warm up the discharge gas with an appropriate ballast. The optical efficiency yield at 254 nm in watts related to the watts of the nominal electrical input can vary from 15 % (6) to 25 - 30 % (12, 13).

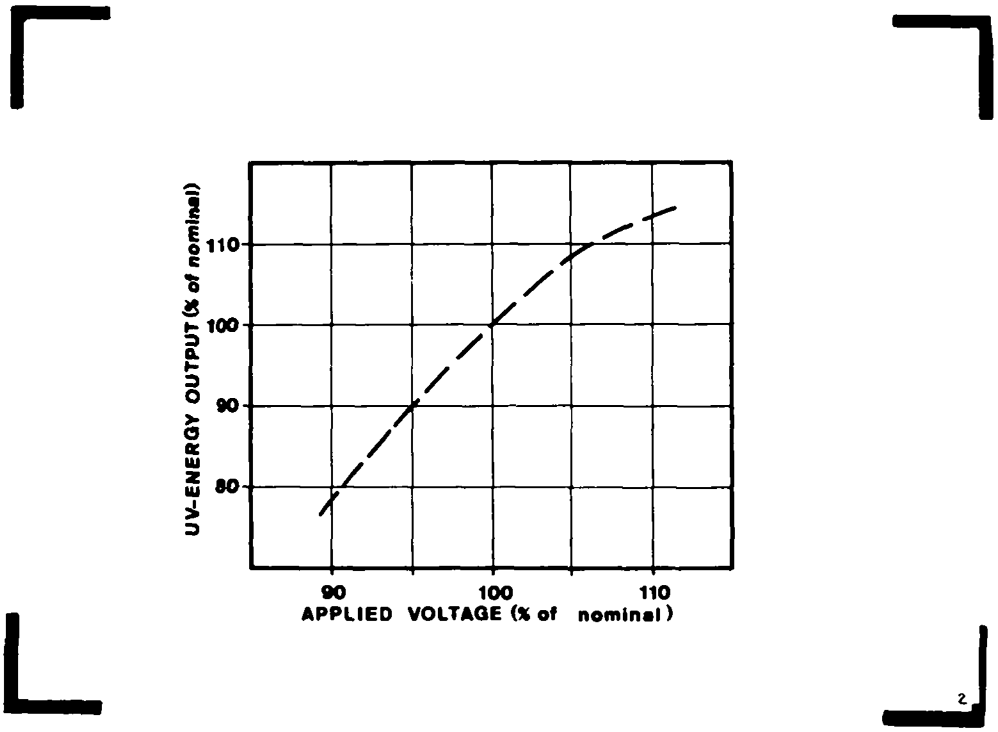
2.1.2. In design, it is most important to consider that a starting-up period of ± 5 minutes is necessary to reach the full emission yield of the hot cathode. Therefore, automatisation may not be based on a start-stop principle working in function of the water flow. The optimum temperature for emission of 254 nm light is of about 40° C (14)

2.1.3. The optical yield at 254 nm of a cold cathode mercury vapour lamp is much lower than that of the hot cathode type e.g. (12, 13) : 1.5 -2 % of the nominal watt input. Whence this type is of less use in water treatment but the lamps are easily installed and operated with a classical source of electrical power e.g. 220 V 50 Hz. The start-up is immediate.

2.1.4. The medium-pressure mercury vapour lamp (1-3 atm) which is a polykymatic source, should also be mentioned to complete the list. The ratio of U.V. energy output to the nominal electrical input rarely exceeds 5 % and in this case only part of the energy is emitted at 254 nm. Lamps based on gas-continuum discharges are not yet applied in this specific technique as they are more suited for the ozone-active range below the 220 nm wavelength zone (15).

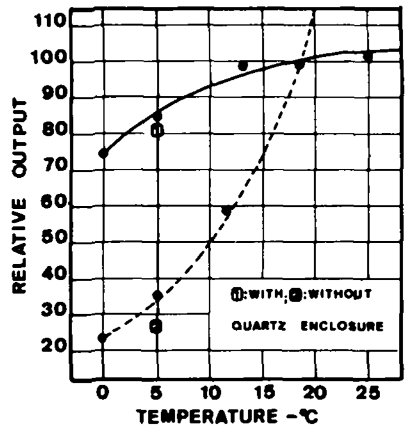
2.1.5. The net U.V. energy output depends ± linearly on the voltage of the electrical current according to *Figure 2*. Hence the design U.V. energy must be considered as the lowest occurring voltage owing to fluctuations in the net.

Figure 2 : U.V. output as a function of applied voltage



• 2.1.6. The change in energy output of U.V. light as a function of the lamp temperature is illustrated in *Figure 3*. Lamps without quartz enclosure when immersed are often not safe to be used in water disinfection, especially with water changing in temperature. For safety reasons, design energy is to be estimated at the lowest temperature encountered in practice.

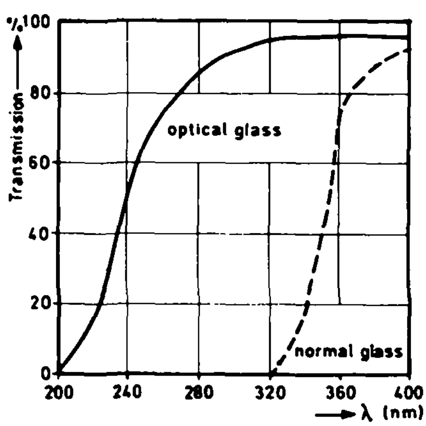
Figure 3 : Temperature dependance of U.V. output



2.2. Transmission and reflectance yield

2.2.1. The transmission yield of clean optical quartz amounts to about 60 % of the emission. Quantitative glass transmission curves are indicated in *Figure 4*.

Figure 4 : U.V. transmission yield of optical glass



2.2.2. The reflectance must be considered particularly in the case of indirect action on the water of U.V. light reflected in the gas phase. The reflectance of suitable materials e.g. metallic aluminium and special aluminium paints amouts to 75 - 80 % (it is worth being mentioned that stainless steel attains a reflectance yield at 254 nm of hardly 25 %. Magnesium oxide and calcium carbonate can present reflectance yields of 70 to 80 %. Therefore, reflecting precipitates on the external surface of the immersed parts of the lamps can be a major cause of failure in the disinfection efficiency).

During periods at which the water stagnates, the layer near the wall of the immersed lamps becomes warmer promoting scaling and precipitation, consequently lowering the light transmission yield. For design : immersed lamps must not be mounted in a hydraulic circuit where stagnation occurs, thus the installation of the lamps at the entrance of service reservoirs or water-towers is preferable to that at the exit of these premises.

Also for easy maintenance and replacement of the lamps the disinfection unit is best mounted into a by-pass system. Thus the valves in the direct pipeline must be of guaranteed tightness to avoid any side-way contamination with unexposed water.

2.2.3. According to BEER's law, water absorbs the U.V. light as follows :

$$I = I_o \quad e^{-kd}$$

in which I_o is the incident intensity, I the intensity at depth d, and k the absorption index. When expressed in cm^{-1}, the following values of k are typical : distilled water : 0.007 - 0.01; drinking water of good quality : 0.02-0.1; strained sewage effluent : $\simeq$ 0.3 - 0.4 . Thus, in clear waters, the absorption of the U.V. light is not a significant factor lowering the active U.V. intensity at cm distances. In existing practical designs, water layer thickness varies between 10 to 30 mm for clear waters and between 5 to 25 mm for sewage as has been specified in scientific papers; according to technical papers it would range from 25 to 120 mm. From our own experience, we recommend a maximum penetration depth of 25 mm.

2.2.4. In the technology by which the emission source is placed at a short distance in the air above the water surface, the absorption in the air is neglectible, however part of the emitted intensity (40 - 50 %) is lost in function of the reflectance of the source. In such indirect irradiation, the distance in the gas phase is to be kept as short as possible e.g. 2 to 10 cm and the thickness of the water layer is optimum between 3 cm for clear water and less than 0.5 cm for sewage.

2.2.5. Practical waters, as purveyed in most cities, have normal absorption coefficients which are below 0.1; however situations have been reported with k-values reaching 0.5 (6,14). If a given water has a marked tendency to form deposits, a regular cleaning of the tubes must be carried out on routine basis. This is one of the obvious disadvantages of U.V. disinfection (17). Furthermore, for turbid waters and sewage, a more reliable technology of indirect disinfection by placing the lamps in the air phase can be carried out.

Dissolved ions, such as Ca, Mg, Na, Al and Fe in amounts up to 0.4 ppm (18), have no marked effect of decreasing the efficiency unless precipitates deposit on the lamps. As for iron at $\geqslant 5\ g/m^3$, it significantly reduces the efficiency. Organic matter such as amino-acids at 3 - 4 g/m^3 (5), Orzan S and tea extracts at colour intensities lower than 10 degrees can diminish the intensity of the received U.V. light by 50 % and consequently the efficiency of the treatment. A 10 g/m^3 increase of BOD can decrease the U.V. intensity received per unit surface by a factor of 4 (17).

Turbidity lower than a standard value of 2.5 g SiO_2/m^3 does not affect the bacterial killing rate providing the matter remains suspended and does not deposit on the lamps (19). The effect, if any, can be overcome by reducing the water flow by 40 percent. Even at higher turbidity for instance between 40 and 100 g SiO_2/m^3 the disinfection may remain sufficient; however it is no longer reliable (20).

With concentrations of suspended matter below 15 g/m^3 the efficiency is generally not adversely affected. If these concentrations increase and exceeds 15, or even up to 30 g/m^3, the intensity diminishes by a factor of 4 and consequently the methods must be abandoned.

2.2.6. To summarize, the reported data on the inhibition of the disinfecting ability of the 254 mm U.V. light of different impurities are conflicting. The following guidelines can be formulated for the use of different equipment of conventional light intensities :

- It is necessary to reconsider the reliability of the U.V. method for disinfection if :

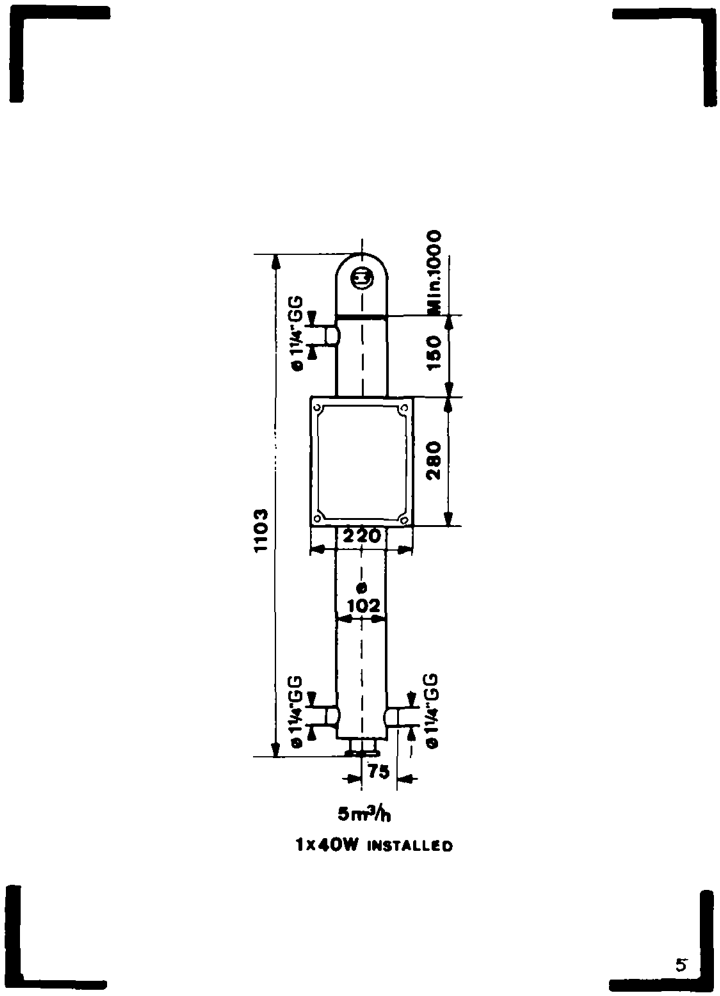
  - the turbidity is higher than 40 g SiO_2/m^3
  - the colour exceeds 10° HAZEN
  - the iron concentration is higher than 4 g/m^3
  - the BOD often exceeds 10 g/m^3
  - the suspended solids are higher than 15 g/m^3
  - the amino-acids (+ proteins) exceed 3 g/m^3
  - the calcium carbonate deposits form on the lamps

2.2.7. Special guidelines for appropriate design in the case of treatment of charged water are the following :

- It is necessary to foresee specific mounting conditions for the equipment :

  - indirect disinfection without contact between the lamp and water is recommended
  - the building-in of automatic cleaning wipers can increase the efficiency e.g. a cable driven wiper in stainless steel plate with rubber brushed around the tubes, sweeping off the deposits and therefore promoting mixing of the water which is to be treated (16)
  - the mixing can increase the disinfection yield
  - a recommended alternative for water carrying suspended solids is a vertically installed reactor with an up-or down-flow contact and purge system *(see Figure 5)*. This installation is preferred to tubes with a horizontal water circulation system
  - symmetrical water circulation must be ensured so as to avoid short-circuiting
  - besides specific technologies for waste-water or sewage, the design of the disinfection units are best foreseen as several contact systems in series enabling a sufficient disinfecting capacity when one of the units is put in by-pass for cleaning. As a general recommendation, at least three units are to be foreseen for a disinfecting capability of two
  - a built-in U.V. intensity meter reaching the active zone can complete the equipment as well as a totalizing- recording system of the burning time of the lamps.

Figure 5 : Typical vertical upflow U.V. disinfection unit



3. <u>MIXING IN U.V. DISINFECTION</u>

Basically, the disinfection capacity depends both on the U.V. intensity which reaches the bacteria and the contact time. Mixing, as an additional parameter in design, can play a very important role in the compromise-contact time versus killing rate. <u>Through adequate mixing, at least a four fold water flow can be treated in comparison to that in ± static conditions</u> (4). In some instances, a statistical contact number of 20 is recommended for sewage (16).

Different technologies are applied in order to promote mixing e.g. perpendicular introduction of the U.V. emitters in the main horizontal pipe (21), static flow mixing (22), mechanical wiping (16) and baffles (e.g. conical) built in the tubes to promote cylindrical flow around the U.V. lamps (4).

No specific quidelines are available at the moment for the mixing intensity. According to experience of existing operation conditions, a G-value of 300 to 800 s^{-1} can be reached. <u>However, more investigation is still needed on the importance of the mixing conditions in U.V. contactors.</u>

4. LETHAL DOSES AND POWER REQUIREMENTS

4.1. The basic mathematical relation expressing the killing action of U.V. radiation is acknowledged to occur according to a monoparticular kinetics :

$$Nt/No = \exp - k \ It$$

in which Nt= number of germs (average number) in a given volume which survive at the time t

No= number of germs at initial time

k = decay constant

I = *intensity of the lethal radiation*

t = time of exposure

(It)= radiation dosis

the relation can also be formulated as

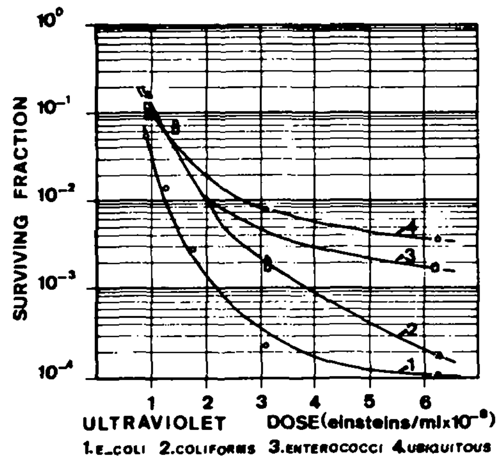
$$N_t/No = \exp - (It/(It)_L)$$

in which $(IT)_L$ is the lethal dosis, that is the intensity during a time t.

4.2. It is a generally recognized fact that the bacterial killing depends on the total active light dosis at 254 nm and not independently on the light intensity and irradiation time (23).

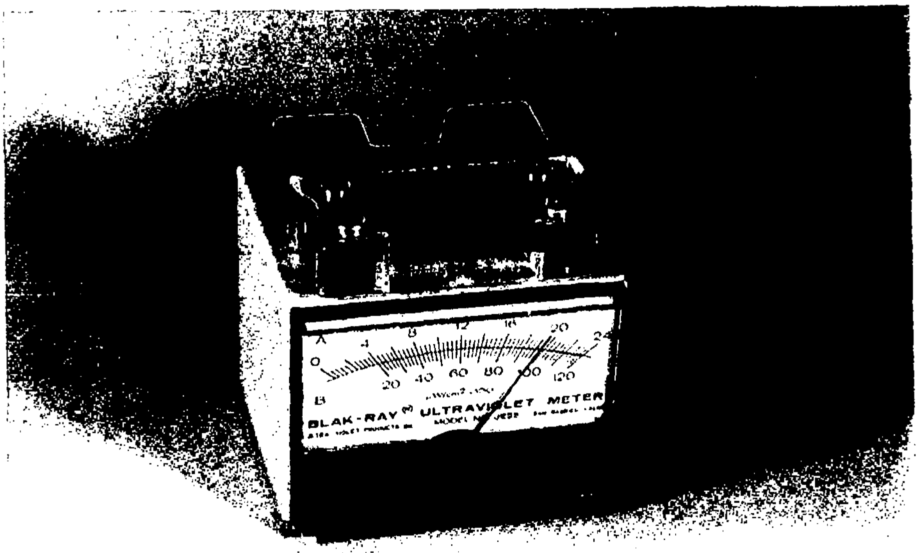
The U.V. (quantum) dosis is most accurately expressed in the unit : Einstein/unit volume. According to the data given in literature (8,24), some basic decay rates indicate about 6 to 8 10^{-2} Einstein/m^3 ($\simeq$ 280 J/m^2 for λ = 254 nm) as a safe dosis for the most usual water-borne bacteria. However, some individual differences may exist for certain species *(see Figure 6)*.

Figure 6 : U.V. dose to killing rate relationships



4.3. The most accurate scientific measuring in full scale systems, is based on chemical actinometry (25). A very suited actinometer is potassium ferri-oxalate : which is a 0.006 Molar solution in 0.1 N sulphuric acid irradiated in the system as is operated in practice. At given time periods, samples are withdrawn and analysed for photoconversion Fe^{+++}/Fe^{++}. The quantum efficiency is of 1.25 mole per Einstein at 254 nm. The actinometric methods also accounts for the incidence of the mixing conditions. For operational control, the most practical measurement of the "black ray" intensity is often controlled (if controlled at all !) by the means of photoelectrical cells *(Figure 7)*.

Figure 7 : Black ray intensity meter



For practical purposes, the energy required to kill bacteria through irradiation is most commonly described in terms of the U.V. radiation energy density necessary to kill or desactivate waterborne organisms, at a rate of about 99,99 percent and expressed in mW sec/cm^2 which is equal to 10 J/m^2. The light intensity demanded in practice is the real incident intensity which is best measured at the very site of application, as the various mixing conditions may play a dominant role in comparing static, flowing and/or agitated irradiated waters.

Unfortunately, in most experimental work and practical uses, the active doses are expressed on an empirical basis, indicating the geometry, flow-conditions and nominal (electrical) power of the lamp. These descriptions enable an analysis of the relative sensitivity of bacteria and organisms but are hardly interpretable on an absolute basis to formulate design criteria.

4.4. In moist conditions or in water, bacteria are less sensitive to U.V. radiation than when in dry air. For instance, a factor of at least 10 must be considered in the interpretation of data of different origin (12). Table I indicates reported data for the achievement of a 99.99 % killing rate. From the given data for the fore-mentioned killing rate, doses for less performing requirements may also be computed with the equation :

$$(Nt/No) = \exp - [\, It/(It)99.99]$$

Legend of Table I

(★) *Values also corrected for all energy reducing factors (turbidity, colour...)*

(★★) *Data may depend on starting volumic concentrations*

(★★★) *Necessary exposure times up to 40 - 300 seconds have been reported*

(Estimated values, computed from the experimental conditions as published)

TABLE I : KILLING RATES UV - 254 nm

ORGANISM	J/m^2 OPTIMAL CONDITIONS (9,26)	J/m^2 PRACTICAL CONDITIONS (*)
E. coli	66	130-400 (**) <240>
Enterobacteria (general)	34-76	240
S. thyphimurium	150	250
Ps. aeruginosa	105	?
Clostridia	?	500
Ubiquitous (total plate count)	?	(≈500) (**)
Resistant bact. (Micrococcus radiodurans)	2.000	?
Brewers yeast	66	250
Fungi	130-175	1.200
Mould spores	?	<2.500
Coli-phagi (27)	40	?
Viruses (Entero)	?	150-1.500 (***)
Animals (micro)	?	(5.000)
Microalgae & phytoplancton (28)	?	1.000
Algae (green-blue)	?	(10.000-25.000)

4.5. Several conclusions can be drawn from Table I :

• 4.5.1. E.Coli is representative as a test organism for the enterobacteria-group. Most often bacteria of this group require a lower dose for a given lethal rate. S. Thyphimurium and Ps. Aeruginosa are potential exceptions requiring further study. The U.V. sensitivity of faecal streptococcus is similar to that of E.Coli. Water bearing E.Coli and V.Cholerae, treated by U.V. has been found free of cholera as long as E.Coli is absent in the treated water; whence in this technique, the absence of E.Coli is also a valuable criterium to avoid spreading waterborne cholera.

• 4.5.2. A safety factor of 1.6 to 1.7 applied to the radiation dose necessary to eliminate E.Coli is proposed as design parameter in drinking water disinfection (29).

4.5.3. Most enteroviruses are more or less as sensitive to U.V. radiation as enterobacteria (8). However, longer exposure times, at given intensities have been reported in some instances. Consequently, this may be due to encapsulation or a cluster effect (multisite killing kinetics), determining a lethal lag-phase. In comparable operating conditions Clostridium perfringens has been found to be less sensitive to U.V. irradiation than the A-type, poliomyelitic virus (19). Whence this bacterium could be a valuable criterium for the viricidal action of this technique.

4.5.4. U.V. light disinfection of higher organisms, animalcules e.g. Gammarus, Daphniae, Euglena etc... implies longer exposures, for instance several minutes at conventional light intensities. Estimated dosis values according to literature data range 5000 J/m^2 (2).

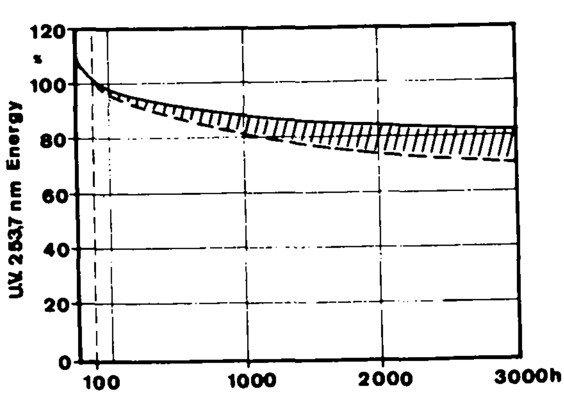
4.5.5. U.V. irradiation is not an efficient algicidal treatment : high doses at especially long exposure times may be necessary. In particular conditions, e.g. swimming pools or aquariums, the method can be efficiently applied in the recycled sub-stream.

4.5.6. As a general observation in practice, ubiquitous bacteria as a total plate count are recovered in U.V. irradiated water even when Coli and enterobacteria are eliminated through the treatment (30) *(see Figure 6 also)*. Ubiquitous Ps. Fluorescens species, e.g. Ps. Cepacia exposed at a conventional dose of 250 J/m^2 at 254 nm are reduced in number by a factor of 10^6 to 10^8 when cultivated in the dark during the subsequent analysis of the water. However, if cultivated in the presence of light, a photochemical repair (?) occurs susceptible of indicating bacterial counts increased by a factor of 10 compared to the initial number (31). When operating the immersed-lamp technique, stand-by periods should be avoided during which the lamps are to be switched off and the water maintained in a static condition in the system. During these periods e.g. nights, the bacterial aftergrowth takes place forming slimy deposits on the outer wall of the lamps thus hindering the efficiency during the following periods of service. The most frequent species are Pseudomonas and Achromobacter (32) (and modified Pseudomonas species?).

4.5.7. The radiation intensity emitted by lamps of the low-pressure hot cathodetype decreases within the time of use. A major cause is the inside deposal of condensed mercury on the walls of the tube. General trends are given in *Figure 8*.

• The necessary design energy is best fixed on the basis of a 1000 to 1200 h use of lamp emission intensity, equal to 75 % of the original emission intensity. Beyond 7500 - 10.000 hours of continuous operation, the
• emitted intensity decreases to 60 - 65 % of the initial irradiation. Therefore, after a year's operation, lamps are to be renewed even if their behaviour remains apparently correct.

Figure 8 : Drop-in radiation energy on aging of U.V. lamps



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Continual measuring of the dosis can enable monitoring of the renewal rate. Moreover, in laboratory experiments, age and operation conditions of the lamps may not be overlooked; furthermore, records on this aspect which is related to the lethal doses are scarce where fundamental investigation is concerned.

5. RELIABILITY IN DESIGN

5.1. In current practice, when designing equipment on the basis of the lethal dosis at a given rate of E.Coli, it is highly recommended to overdimension the available irradiation dosis. The absolute minimum dosis of 160 J/m^2 is often retained; however according to data in Table I, if considering a 99.99 % killing rate, it is recommended to foresee 250 J/m^2. This quantification is to be based on the most unpropituous circumstances likely to be encountered in water treatment practice : e.g. water temperature, aging lamps, voltage drop, turbidity etc...

Consequently, in the case of drinking water disinfection, a safety measure, or overdimensioning by a factor of 1.6 to 1.7 is recommended seeing the E.Coli may be more sensitive than sporulating bacteria or certain enteroviruses.

5.2. The doses applied for disinfection range 1/10 of those for extensive photochemical oxidation of dissolved organic compounds (33,34) e.g. in the preparation of apyretic water, at conventional irradiation intensities, long exposure periods - several hours for instance - are required. Nevertheless, the photochemical changes in structures of dissolved compounds that may occur on U.V. irradiation, during current disinfection practice (with PAH for example), remains a question open to further investigation.

5.3. Actually, the design of the irradiation intensity is usually based on the active dosis concept, that is :

$$N/No = \exp - k\ I\ t$$

To express the dosis, the active energy received is to be computed preferably on the basis of the largest surface of the water layer e.g. the inner wall of a tubular contactor, the emission lamp being placed at the center of the cylinder. This is the safe approach as measuring the dosis (J/m^2) on the basis of the average surface between the outer lamp surface and the inner surface of the cylinder, implicates a very reliable mixing in the water layer.

5.4. The approximation of the irradiation energy, received at a given distance of a source compared to the known energy measured at a reference distance, can be approached by an inverse square depreciation law (27) :

$$\frac{I\,d_x}{I_R} \qquad \frac{d_R^2}{d_x^2}$$

5.5. In the most accurate designs, a photoelectric dose-meter is incorporated in the outer wall of the contact vessel, thereby guaranteeing permanent reading and eventual automatic alarm. If deposits of turbidity occur in the system, the detection warning is of the highest over-all reliability. Furthermore, failure of the U.V. source or unsufficient radiation energy can very well monitor the shut-down of the water flow.

6. ACTION TIME

Although the germicidal action of U.V.-quanta is reported on the basis of the active dose, the action time is generally acknowledged as important in practical design. There is however some doubt as whether appropriate design is to be expressed as energy dissipated into the given volume rather than on the basis of a dose.

For quanta of 254 nm in actual systems, the irradiation time is equal to :

$$t = \frac{D(\text{in } J/m^2)}{E(\text{in } W/m^2)}$$

in which D is the necessary dosis, (e.g. 250 J/m^2) for a given germicidal rate (e.g. 99.99 % for E.Coli) and E the intensity of irradiation.

In the systems applied, the theoretical contact time ranges between 1 and 300 seconds with the most frequent values of 7 to 15 seconds. The contact time can significantly be shortened by built-in mixing blades. The ideal solution is that each part of the water should approach the tubes at a distance of 0.6 mm and this twenty times during each flow transit period.

7. EXISTING EQUIPMENT

Most U.V. disinfection units for water are presently based on the immersed lamp system. Quartz jackets are used in this case. There exists no completely immersible lamps of the water-proof type, as all the electrical contacts are located outside the water zone. Standard annular space between lamp and inner wall of the cylinder is of 1 inch, that is equal to 25 mm.

Basic units exist for the flow range of 1 to 100 m^3/h but this figure can be increased by operating several units in a parallel disposition. When installed vertically, up-flow contact columns are best suited in the case of the possible presence of turbidity or settling matter. Typical examples are given in *Figures 5, 9 and 10.*

Figure 9 : High rate U.V. disinfection lamp installed in a pipe

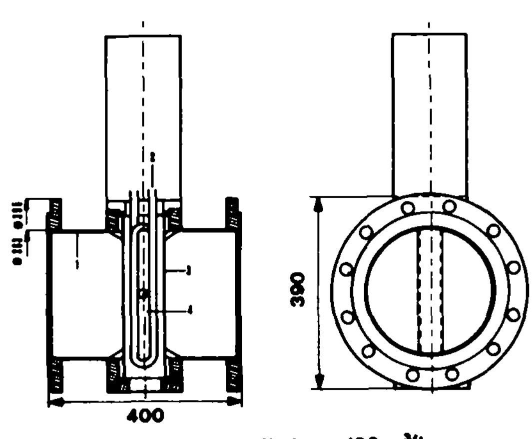
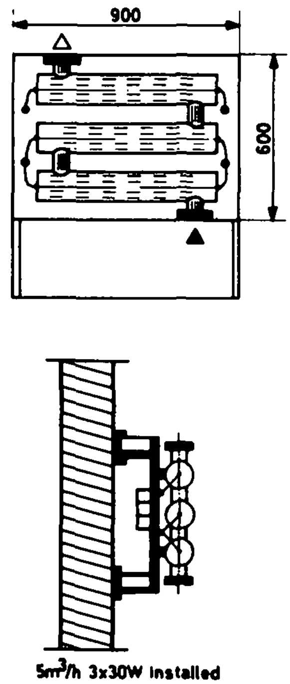


Figure 10 : U.V. disinfection apparatus with three units is series

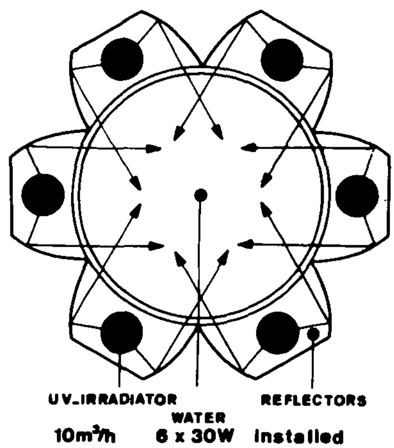


Most equipment resist to 16 bar water pressure, however nominal design is often based on 10 bar.

Actual units without direct lamp to water contact are in full development; this may enable to apply the method to more turbid waters including sewage. Moreover, the use of optical-glass lamps, instead of quartz can lower the cost of the lamps by a factor of 4 to 5.

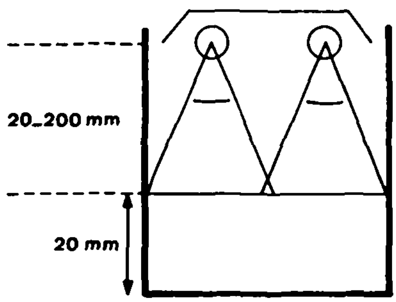
A particular approach to this technology, is to place the lamps outside the water tube which is constructed in quartz in the active irradiation zone.

Figure 11 : U.V. lamps installed outside the water pipe-line



Another method is to place the U.V. lamp with appropriate reflectors at a close distance (less than 5 cm) above the free water surface, flowing in a shallow channel. For drinking water quality, the thickness of the water layer is best maintained below 20 mm. The zonal distribution of the light intensity implicates a maximalflux-angle of 45 °. This irradiation technique is in full development (see *Figure 12*).

Figure 12 : U.V. irradiation at the surface of a channel



8. COSTS

Water disinfection by U.V. is not specifically a low-cost technology, although the contact time is 1 % below that required in chlorination. Whence, building depreciation costs are negligible in the U.V. disinfection.

Investment costs and operational costs for the U.V. contact equipment are proportionnal to the contact time, at least within the limits of 5 to 30 seconds. At a 15 second contact time, the over-all cost is estimated equivalent to that of chlorination at 3 g/m^3 (17) (in present conditions this is equal to 0.1 BF/m^3). This global cost is consistent with literature data which results in 0.12 to 0.17 BF/m^3 (8). However, several components of the global cost are often overlooked in such comparisons.

TABLE II : OPERATION COSTS OF UV-DISINFECTION PLANTS (C.I.B.E., Belgium, 1980)

Flow rate m^3/s (nominal)	1	2	3	4
	0.1410^{-3}	1.410^{-3}	1.410^{-3}	5510^{-3}
Installed lamps (Wh)	30	40 (*)	120(**)	4.800(**)
Number of lamps	1	1	3	120
Cost of equipment (installed)	40.000	400.000	300.000	9.000.000 (estimated)
Annual depreciation rate	4.000	40.000	30.000	(900.000)
Energy costs/year (***)	700	930	2.800	112.000
Attendance/year (at 580 BF/hour)	30.000	60.000	60.000	120.000
Replacement of lamps	2.000	3.500	11.000	440.000
Total (annual)	36.700	104.430	103.800	1.572.000
Cost per m^3 (BF) (at 50 % of nominal capacity)	16,6	4,7	4,7	1,82

(*) based on 180 J/m^2 or (**) 240 J/m^2

(***) based on 2,6 BF/kWh

In Table II figure the costs of different technologies in use or controlled by the C.I.B.E.

The systems compared are all of the submersed lamps type, with a quartz jacket and operated on a continuous basis for a light emission at variable flow rates of the water, within the limits of the nominal flow capacity. An average use of 50 % of the maximal water flow is estimated in the cost computation.

In columns 1 & 2 the systems are single lamp systems, different in technology than those in columns 3 & 4 which are based on the principle of two operational units, plus one continuously fed as stand-by. The first obvious conclusion is that regarding the over-all cost there exists a scale factor making larger equipments comparatively less expensive than the small units. However, annual depreciation is consequently heavier than for low nominal flow rates. This often leads to the use of U.V. disinfection for smaller plants only, although global costs decrease by the increase of flow capacities.

Attendance and inspection are the major cost-impact factors especially where small scale units are concerned. On the basis of the net operational cost (without depreciation or attendance) the principal expenses are those for the renewing of U.V. sources in due time, which 75 to 65 % of the total. Disinfection of water by U.V. light is labour intensive, however no special qualification is required for operation.

9. CONCLUSION

Disinfection through U.V. irradiation can be a simple and reliable alternative method applicable in remote areas where the availability of appropriate chemicals is scarce. The method does not require specially skilled staff however the design must be very well adapted to the local circumstances.

Basic parameters for design as reported here are : water temperature, available current voltage, global properties of the water and mixing conditions in the contactors. The operation of the system is best continuous and permanent.

Appropriate irradiation doses must be recommended as 250 J/m^2 and 500 J/m^2 for a guaranteed reliable bactericidal and viricidal effect. Representative test organisms are E.Coli or Ps.Aeruginosa and Clostridium perfringens.

The key to the success of the method is the replacing of the lamps in due time and this at least once a year.

Different other design criteria are commented upon in this paper.

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BUTS ET LIMITES DE LA DESINFECTION DE L'EAU PAR LES RAYONS ULTRA-VIOLETS.

Dr. W.J. Masschelein

Résumé

Ce document étudie en detail l'état actuel de l'utilisation de rayons ultra-violets pour la désinfection de l'eau.

Il présente la théorie du procédé dont l'efficacité varie selon les bactéries à détruire et selon les types d'installation.

Le désinfection par rayons U-V n'est pas une technologie à bas prix car les lampes qui en constituent un élément intégral doivent être changées au moins une fois par an.

Ses avantages résident dans la simplicité des modes opératoires, dans leur possibilité d'automation et dans l'absence de produits chimiques, malgré la complexité de la mise en place en un lieu déterminé.

La qualité de l'eau , sa température, le courant électrique doivent être connus en permanence et le procédé doit être utilisé en continu.

Paper 23

Water Disinfection Practice and its Effect on Public Health Standards in Developing Countries

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ABSTRACT

The relevant public health standard of an area can be rapidly assessed by consulting local hospitals or dispensaries. The results of one such survey in the Middle East indicate a high percentage of treatment given for diseases of sanitary origin. The routes of infection are not confined to the direct water-borne faecal/oral route, and local assessment of waste disposal practice and some limited on-site analysis of water sources indicate that the improvement of public health standards requires action to be taken in several aspects of sanitation and water supply if disinfection is to be successful.

The basic methods for water disinfection are discussed and chlorination is selected as the most suitable technology for developing areas. The chemical basis of water chlorination is briefly discussed and the merits of various types of chlorine-bearing substances compared. Practical details of chlorination by diffusion chlorinators, by batch chlorination and by drip-feed chlorination are discussed and details of improvised equipment are given. The importance of chemical tests is stressed and details given for the measurement of free chlorine. These tests are important where it is necessary to measure the chlorine dose required for a given supply; two methods are given for determining this.

The importance of water disinfection as part of an integrated public health improvement plan and the need for public co-operation are stressed.

INTRODUCTION

One of the major tasks of civil administrations in developing areas is the improvement of public health in both urban and rural populations. Thorough statistical surveys of the standard of public health over a widely distributed population need to consider many factors including nutrition, childbirth, and medical treatment for a wide range of injuries and diseases. Rapid surveys of local medical services will often indicate that treatment is most frequently given for many infectious diseases that are water-related. Although the proportion varies widely from place to place, it is common to find that the largest category is that derived from these diseases.

The main source of these diseases is infected people, and the means of infection may include the direct faecal/oral route, non-human vectors, and contact with the wastes of infected people that may be either airborne or water-borne. Public health protection must therefore be concerned with treatment of the sick and measures to reduce the spread of infection by improving sanitation and improving the water supply. The effectiveness of these developments can be shown by comparing the incidence of sanitary disease in developed areas with that in less developed areas. As the development of these facilities improves, statistics can demonstrate their effectiveness. This can be illustrated by Fig. 1 which shows the decline in the death rate due to sanitary disease among British soldiers in India. A major reduction in mortality occurred in the last half of the 19th century, a period before the widespread introduction of either sewage treatment or water treatment. This improvement was brought about by relatively simple measures of hygiene. The subsequent introduction of organised sanitation and water treatment caused a further rapid decline.

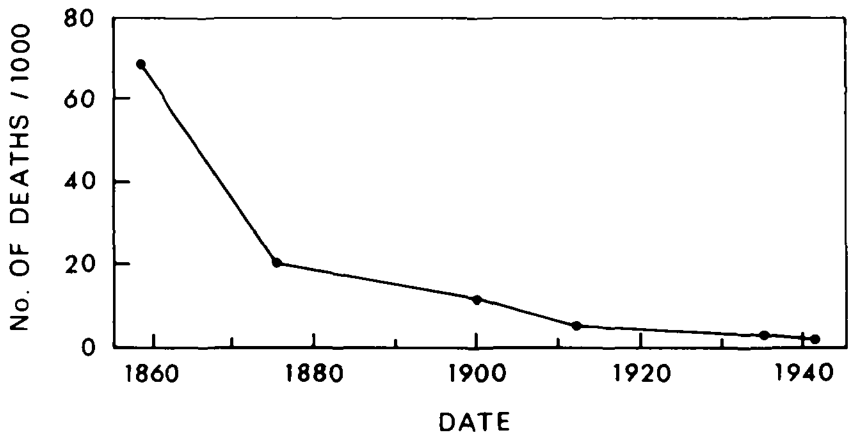

Fig. 1. Death rate of British soldiers in India attributed to diseases of sanitary origin (after Maj. Gen. Sir J W D Megaw, late Medical Advisor to the Secretary of State for India)

THE ASSESSMENT OF APPROPRIATE PUBLIC HEALTH DEVELOPMENTS FOR LOCAL WATER PROBLEMS

The Water Research Centre has carried out a number of on-site investigations of local water and sanitation problems in several areas in Africa and Asia for some of its consultant members. During a recent survey, 26 rural towns and villages in the Middle East were visited. Local water sources were sampled and analysed on site, with the results shown in Table 1. The bacterial quality of the water indicates faecal pollution in every case tested, falaj waters being more exposed and having inferior quality to well waters. In every source tested the water was clear and bright. The towns had local medical facilities varying from a simple dispensary without a qualified doctor to a small but fairly well equipped hospital with several doctors available. The percentage of patients treated for diarrhoeal diseases (no detailed diagnoses were consistently available) and intestinal worms indicates a high level of infection of sanitary origin. There was also significant seasonal malarial infection derived from mosquitoes breeding in irrigation ditches. There was no local bilharzia. Diseases in the last two categories are likely to be much more prevalent in other areas of Asia or Africa. The environment of the area surveyed was predominantly dry desert with few water sources, mostly derived from underground. The absence of water-borne sewage results in few points of intensive faecal

Table 1. Water quality and health in some towns in the Middle East

Note - Water Source W = well F = falaj (qanat)

Town No	Population (10^3)	Water source	Daily water consumption (1)	No of hospital cases daily	% cases treated for: Diarrhoea	Intestinal worms	*E. coli* (MPN/100 ml)	NH_3 (mg N/1)	Total solids (mg/1)
Coastal towns									
1	15	W	100	250	10	3	26	0.4	755
2	10	W	15	170	41	-	< 6	0.6	560
3	7	W	10	200	25-30	2	30	1.2	520
4	6	W	>7.5	200	40-50	10	6	0.45	740
5	6	W	36	250	30	2	30	0.5	520
6	5	W	75	300	40	30	>30	0.3	1200
Inland towns									
7	36	W	-	400	25	4	< 6	1.1	520
8	25	W	-	800	40	15	14	0.45	2000
9	10	W	-	400	10	10	14	0.65	600
10	10	W	-	100	25	-	< 6	0.35	600
11	8	W	110	450	18	1	6	0.6	640
		F					>30	0.4	840
12	5	W	-	100	30-35	20	< 6	0.4	600
		F					>30	0.4	660
13	5	F	-	50-80	15	5	>30	0.5	400
14	5	W	-	250	20	few	30	0.6	1280
		F					>30	0.5	2040
15	5	W	60	400	60	10	30	0.5	1220
16	5	W	-	80	25	12	14	0.3	96
17	4.5	W	-	180	3	7	8	0.5	900
		F					-	0.4	760
18	3	W	-	90	10	-	> 3	0.4	700
		F					-	0.4	440

pollution, but there are many areas where no sanitary facilities are provided and defaecation in the open is the common practice. Almost no direct water pollution can occur in dry soil, but since almost all wells and falaj points are open there is a high risk of airborne and fly-borne pollution. An increased use of water without organised waste disposal would be likely to increase the risk of polluting water sources.

The monsoon and paddy regions of India and S.E. Asia have abundant surface water which is still used for consumption without treatment in spite of current well-drilling developments. Where sanitary facilities are inadequate this results in greatly increased health risks due to direct pollution. In many of the wet tropical areas of Africa and S. America the same hazards occur, together with bilharzia. These conditions severely complicate the task of protecting drinking-water sources and increase the need to organise simple latrine programmes to replace the dangerous custom of defaecating on the fields. The development of water supplies must consider the quantity and availability of the supply as well as its quality. Although the standards usually proposed for water quality embrace many chemical and physical properties there can be no doubt that the absence of disease organisms is the most important. Disinfection can provide almost complete protection of a water supply, but care is needed to eliminate those routes of infection that may occur between disinfection and consumption. The prudent selection of water sources, the prevention of pollution by good sanitation, and the clean transport and storage of water all play important roles.

THE PRINCIPLES OF DISINFECTION

The dictionary definition of disinfection is to cleanse from infection. Many technologies have been employed to disinfect water, including boiling, ultra-violet radiation, filtration, and treatment with controlled doses of chemicals that can eliminate disease organisms without harming the consumers. All the methods used have limitations and could be rejected by those that would be satisfied only by perfection. Developing communities may be remote, and may lack every kind of resource from cash to skill. The most appropriate procedures are therefore those requiring the least expenditure of resources. Simple low-cost techniques for properly applying very small quantities of cheap disinfectant chemicals make low demands on local resources, can be extremely effective, and as they have been in use since the turn of the century can be said to be well tried and reliable.

In Table 2 the main characteristics of most important chemical disinfectants are compared. Chlorine is the cheapest, and easiest to transport and store safely in its solid forms. It is the oldest established chemical disinfectant and is widely used throughout the world, consequently more experience is available in its use than for any of its rivals. For these reasons, and for those given in Table 2, it is recommended as the best choice in developing areas.

Chlorine can be employed in 4 main ways. Large communities can economically use chlorine gas supplied in cylinders, although supply difficulties can be a major problem. Small electrolytic chlorine generators are currently available that can use sea water or brine to generate chlorine on site. They are, however, sophisticated machines dependent on abundant cheap electricity. Sodium hypochlorite is commercially available as a solution in various strengths containing from 1 to 15% of available chlorine. It is a useful material but slowly decomposes on storage and cannot safely be used without careful testing. Calcium hypochlorite is readily and cheaply available as bleaching powder

Table 2. The characteristics of some water disinfectant chemicals

Disinfectant	Advantages	Disadvantages
Chlorine	Very effective. Can be applied on wide range of scale. Simple equipment can use solid, solutions or gas. Running costs low.	Good control needed. Works best with clear water. Good safety practice necessary when using gas.
Iodine	Effective. Can use simple equipment.	Running cost high. Good control necessary.
Ozone	Effective. Removes many coloured impurities. No chemical supplies needed.	Equipment is costly. Needs good electricity supply and engineering support. Running cost high.
Silver	Fairly effective. Can use simple equipment.	Running cost high. Only works well in fairly pure water.

containing about 30% available chlorine. High Test Hypochlorite is a similar material but with about 70% available chlorine and is consequently pro rata more costly.

The chemical reactions of chlorine in natural waters are complex. Chlorine will react readily with many oxidisable substances including ferrous and manganous compounds, nitrites, sulphites, and many forms of organic matter. Some organo-chlorine compounds have been shown to have carcinogenic properties but only in high concentrations. Chlorine also reacts with ammonia to form chloramines which themselves have slow, mild disinfecting properties. All the above reactions reduce the total disinfection power of chlorine which is able to penetrate micro-organisms at a rate dependent on their physical structure. Amoebic cysts and bacterial spores with resistant cell walls require higher doses or longer exposure periods than most vegetative bacteria. Viruses are much simpler structures, and though they do not have resistant cell walls they are less vulnerable than most bacteria. Chlorine reacts in water to form hypo-chlorous and hypochlorite ions. The equilibrium is pH-dependent; thus at pH 5 almost all the chlorine is in the form of hypochlorous acid and highly active, while at pH 10 most of the chlorine is in the form of hypochlorite which is much less powerful. In the pH range of most natural waters, small doses of free chlorine are able to render inactive 90-99.9% of E. coli and other enteric bacteria, faecal coliforms, and faecal streptococci, and 50% or more of many types of virus, 100% of the miracidia or the cercaria of bilharzia.

It must be emphasised that chlorination of drinking water has little effect on worm eggs which are however readily removed by settlement, has no effect on malaria transmission, and does not reduce the risk of bilharzia infection, which is caused by skin contact with infected surface water.

Although disinfection with chlorine may appear to be a complex process to under-stand and a difficult process to operate, disinfection can be accomplished with very low concentrations of free chlorine. A practical system can be adopted in which a dose of chlorine is applied to water that will satisfy all of the chlorine-consuming reactions (chlorine demand) and leave a residual concentration which is able to cleanse infections. The process is not instantaneous; if a suitable reaction period is allowed to satisfy the chlorine demand - 30 minutes is usually ample - the dose may be represented by a simple sum:

Free chlorine dose = free chlorine residual + chlorine demand.

The free chlorine residual may be set at a concentration low enough to ensure no ill effects to the consumer, and to achieve low cost, but high enough to satisfy a potential chlorine demand that may result from the system of distribution. Residual concentrations of free chlorine may be as low as 0.2 mg/l where the supply is clean and the distribution system not vulnerable to contamination. Where the supply is turbid or polluted, or exposed to unsafe distribution or storage in unsterile containers, a safer residual concentration would be 1 mg/l. Concentrations recommended are commonly in the range 0.2 to 0.5 mg/l of free Cl_2.

The chlorine demand of a given water source must be determined on site. Waters from large underground aquifers not exposed to faecal pollution usually have a consistently low chlorine demand. Some large lakes can also be fairly consistent. These sources present few operational problems. Tropical rivers are, however, rapidly affected by rainfall run-off which can introduce extremely large increases in suspended solids of both organic and inorganic origin. Algae can be present in high concentrations at some seasons. Surface-water sources therefore must be tested more frequently to ensure the proper chlorine dose is maintained.

PRACTICAL METHODS OF CHLORINATION FOR REMOTE AREAS

1. Diffusion chlorination

Many communities in remote areas rely upon hand-dug wells. These have much to recommend them: quality is usually uniform and they are protected by the soil from evaporation or faecal pollution. Poor well structures often admit pollution from surface run-off and airborne and insect-borne pollution. Pit latrines or sewage discharges should not be sited too close. Well waters are usually clear but may be turbid in silty soils and may contain iron salts up to 30 mg/l, and other non-toxic salts in unacceptable concentrations. Some wells may contain harmful concentrations of lead, arsenic or fluoride. A rapid survey of the condition of the well and site, the key chemical properties of the water, and the health risks indicated by local medical records will enable the necessary actions to be taken to improve the supply. These may include removal of debris, improvement of the lining and walls, the provision of a cover and either a fixed bucket or handpump. Disinfection may be carried out with periodic additions of chlorine but a cheap simple diffusion chlorinator may provide adequate continuous chlorination which is far superior.

In principle a container of hypochlorite is suspended in the free water and allowed to diffuse chlorine into the water at a rate which will allow chlorine to enter the water to replace that removed by the chlorine demand and the water withdrawn for use. It is essential that sufficient free water is present to allow a reaction period of 30 minutes; this may require enlargement of the bottom of the well.

Three types of diffusion chlorinator are shown in Fig. 2. Type (a) is a vessel made from porous pottery and filled with hypochlorite solution. Type (b) is a common clay pot half filled with a mixture of 1.5 kg bleaching powder and 3 kg

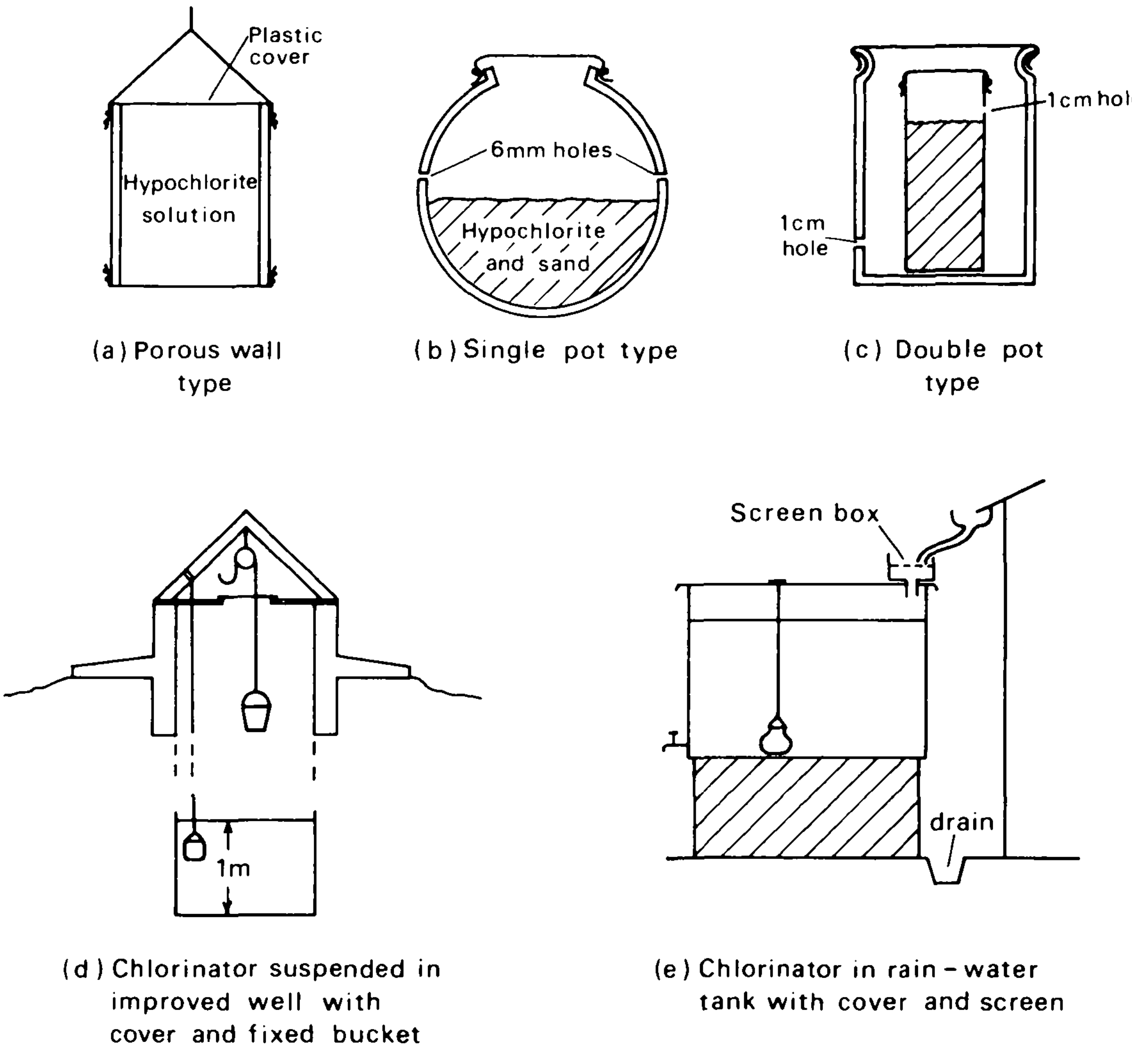

Fig. 2. Diffusion chlorinators and how to use them

of sand. Two 6-mm holes drilled a little above the level of the mixture will diffuse enough chlorine to provide a residual chlorine concentration from 0.2 to 0.8 mg/l for about a week in a well with a free volume of 9-13 m^3 and a withdrawal rate of 0.9-1.3 m^3/d under conditions in India demonstrated by NEERI. Type (c), also recommended by NEERI, consists of a perforated pot containing a mixture of 1 kg bleaching powder and 2 kg sand placed within a second perforated pot. It has been found to maintain 0.3-0.5 mg/l free chlorine for 2-3 weeks in a well with a free volume of 4 m^3 and a withdrawal rate of 0.37-0.45 m^3/d.

Although the performance of types (b) and (c) have been established by NEERI in local conditions in India, any of the three types must be adopted with due care. Tests are necessary at least for the first few weeks to confirm the size and number of chlorinators to employ in each specific case. If the raw water is hard, calcium-carbonate scale may form which impedes diffusion. This effect can be reduced if about 5% of Calgon (sodium hexametaphosphate) is added to the bleaching powder. Materials such as wood or metal can react with chlorine and should not be used in chlorinators unless they are protected.

Testing for free chlorine is necessary to check the effectiveness of every kind of chlorination system. A suitable test for well chlorination is the starch iodide test.

Starch iodide test for free chlorine - Dissolve a small crystal of potassium iodide in a sample of water (about 50 ml). A few drops of starch solution will show a faint blue colour at about 0.15 mg/l free chlorine; higher concentrations give stronger colours. Indicator papers can be obtained which employ a similar reaction, are easier to use and more precise.

Palin's DPD test - Simple low-cost test kits are available employing DPD (diethyl-p-phenylene diamine) reagents in tablet or liquid form and a simple block or disk comparator. This gives more precise and reliable results and provides a capacity for reliable investigation and control of water quality that can become a sound foundation for future development.

The installation of a diffusion chlorinator is shown in Fig. 2 (d) together with the improvements such as watertight head works, cover and fixed bucket that may be necessary. In many areas stored rainwater is an important source. The first run-off from roofs is often contaminated and should be discarded, and additional hygienic safety may be provided by installing a diffusion chlorinator as shown in Fig. 2 (e). Prolonged storage of rainwater low in salt content and suspended matter provides ideal operating conditions.

2. Batch chlorination

Where storage tanks served by intermittent hand or power pumps are used, batch chlorination can provide more consistent final quality even where the source is of variable quality as from a river. The system shown in Fig. 3 shows two tanks that are alternately filled by a pump, and a preliminary treatment stage such as screening or filtration. One tank can be in service while the second is filled, dosed with the correct quantity of hypochlorite, mixed and allowed to stand for the minimum reaction period (30 min) before connection. This permits a continuous treated supply to be maintained at all times. Using very variable water, it is necessary to test the chlorine demand of every tankful treated, though with stable water quality this may be less frequent.

Chlorine demand test - The simplest method is the multiple-dilution test. A stock solution of hypochlorite can be prepared, containing approximately 1% free chlorine. This will be used for both the test and the chlorination. A series of 5 or 6 one-litre samples are drawn and dosed with hypochlorite, 1 drop in the first, 2 drops in the second and so on. Each sample is then mixed, stood in the shade for 30 minutes, and tested for free chlorine by any available method to indicate the lowest dilution required to provide a concentration of 0.2-1 mg/l.

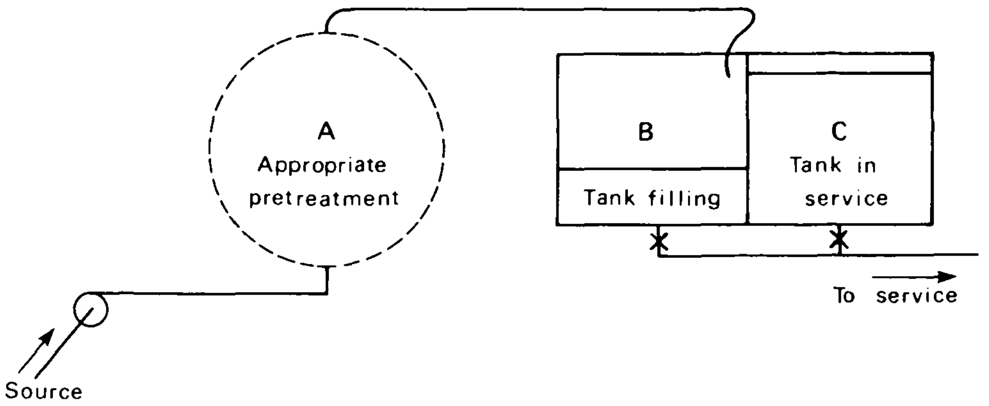

Fig. 3. Batch chlorination with two tanks

To chlorinate the tank to this level, add 50 ml of hypochlorite solution per m^3 of tank volume for every drop used in the lowest acceptable test.

3. Continuous chlorination

Large systems supplied by a pump at a constant rate or by gravity from a reservoir may employ simple drip-feed chlorinators. The system shown in Fig. 4 (a) shows raw water flowing from the source through the necessary preliminary treatment to a mixing vessel to which hypochlorite solution can be added at a rate proportional to the rate of flow and the chlorine dose established by the same test given for batch chlorination.

Many designs of drip-feed doser have been published, but it is clear that this information is not readily available in developing areas as a result no doubt of communication problems. Three simple designs are given in Fig. 4 (b) (c) and (d), which can be constructed at low cost by most village craftsmen. They comprise a container for the hypochlorite solution fitted with a simple means of controlling the drip-feed which can be adjusted to provide the correct dilution ratio with the flow of water. Minor fluctuations in chlorine demand or flow will cause a variation in the residual free chlorine concentration, but provided this remains between 0.2 and 1 mg/l satisfactory disinfection is accomplished.

CONCLUSIONS

Although the disinfection of water supplies can make an important contribution to development programmes aimed at improving the standard of public health in developing countries, it must be seen in the context of a larger integrated development programme including sanitation, and health education. The most simple techniques are recommended, relying wherever possible on equipment that can be made on site, with minimum use of materials that must be imported. The co-operation of the user population is essential if the maximum health benefit

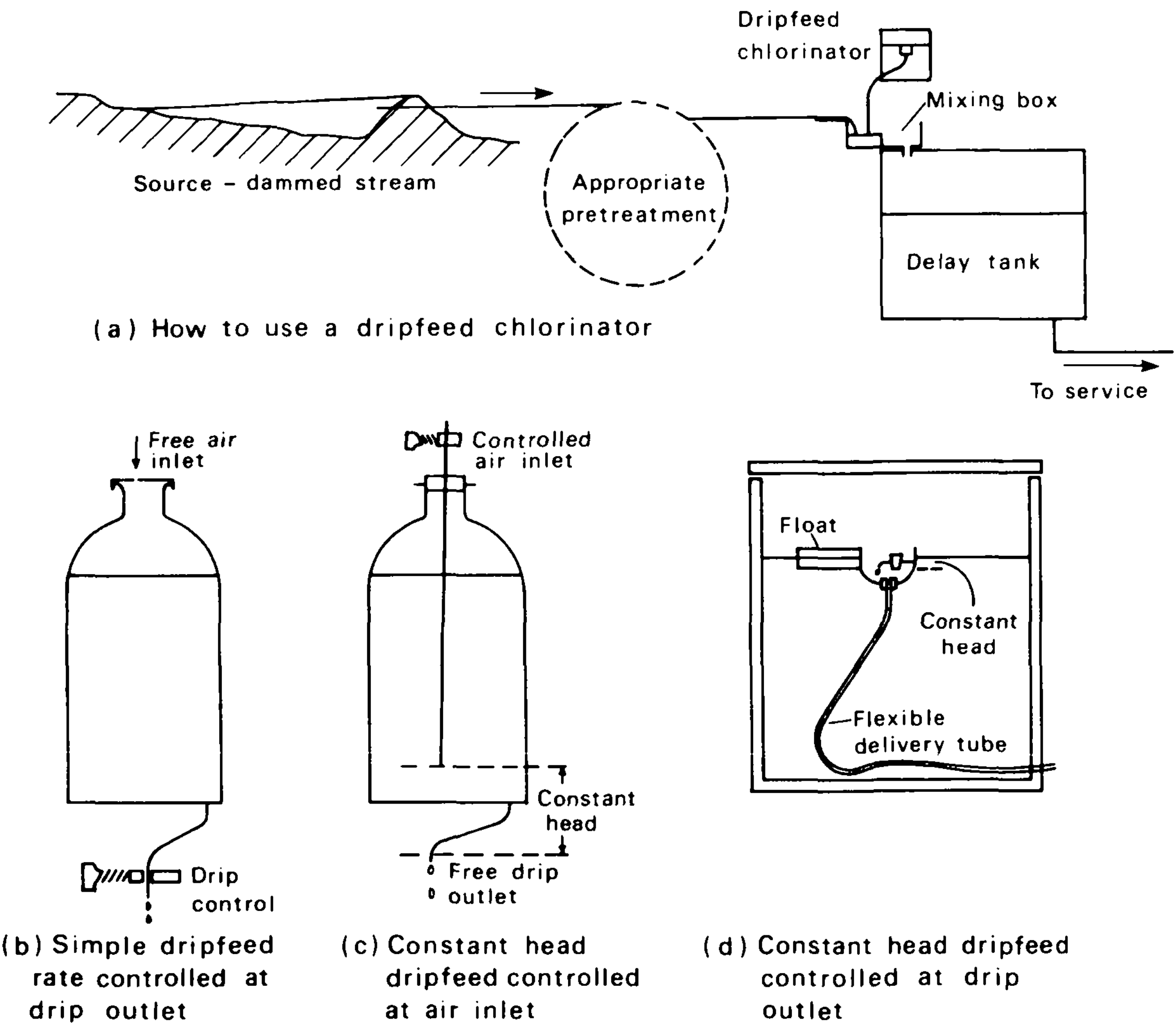

Fig. 4. Drip-feed chlorinators

is to be achieved. This requires them to avoid using untreated water where treated water is available and above all to improve unhygienic practice of waste disposal and water usage that may prejudice the benefits of water disinfection which must be maintained as a continuous process with as few interruptions as possible.

La Pratique et la Désinfection de l'Eau et ses Effets sur les Normes de Santé Publique.

RESUME

Le niveau d'hygiène d'une région peut être rapidement évalué en consultant les hôpitaux ou dispensaires du lieu. Les résultats d'une telle étude au Moyen Orient indiquent un pourcentage de traitement élevé pour les maladies en rapport avec l'hygiène. Les voies d'infection ne sont pas limitées aux eaux fécales/buccales, et une évaluation locale de la façon de disposer des eaux usées ainsi qu'une analyse sur place, même restreinte, des sources d'eau indiquent que l'amélioration du niveau d'hygiène nécessite une action sur plusieurs aspects concernant à la fois l'alimentation en eau et l'assainissement.

Après étude, la chloration a été choisie comme la technologie convenant le mieux aux pays en voie de développement. La base chimique de la chloration de l'eau est ici évoquée brièvement et les effets de divers genres de substances chlorées sont comparées. Les détails pratiques de la chloration par diffusion, par intermittence et en continu sont discutés et des détails sur les matériels correspondants sont donnés. L'importance des essais chimiques est souligné et des détails sont donnés pour mesurer le chlore libre. Ces essais sont importants lorsqu'il faut mesurer la quantité de chlore requise pour une quantité d'eau donnée ; deux méthodes sont données à cet effet.

Le document insiste également sur l'importance de la désinfection de l'eau, dans le cadre d'un plan d'amélioration de l'hygiène et avec la coopération du public.

Paper 24

Demand, Finance and Payment for Water Supply in Less Developed Countries

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It is now generally accepted that there is an appropriate level of technology suitable for each circumstance. That to introduce the most advanced systems into a backward area is wrong and will prove ineffective in practice. I am arguing that the same is true with regard to finance and economics, and that a simple system that can be easily understood, and operated may well be the system appropriate to the circumstances.

In considering the economics of providing a water supply and sanitation in a less developed country there are many different factors to be decided; and a rather different attitude of thought may have to be employed compared with a richer and more developed country.

DEMAND

With water supply there may be a need but no 'effective demand', if 'effective demand' means the amount which potentially would be bought at a certain price at a certain time. The first problem therefore may be to convert that need into demand. If the income per head of the population is very low this may be impossible until the general economy of the country improves.

Closely allied to this problem of realisation of need and inability to pay, is the concept of benefits. Can the installation of a proper water supply be justified by traditional cost benefit analysis?

That there is a benefit from clean water and proper sanitation need not be argued but to translate that into money terms in order to justify the expenditure is probably false in many less developed areas. It is said that over half the hospital beds in the world are filled with people suffering from water-born diseases; and yet in many of the areas that we are considering there may be no hospitals or even doctors, so that to translate clean water benefits into the idea of less hospital costs is unhelpful, equally to say that less illness means great ability to work and earn wages when sufficient work may not be available, is again a false analysis.

In fact as is the case with many monopolies largely run publically, cost benefit analysis is not much value; the benefit has to be assumed, or at any rate considered to be someone else's responsibility. The duty of those seeking to supply water and provide sanitation is to make those schemes cost effective.

If we decide that there is a benefit and that some schemes however simple should go ahead, then if there is no effective demand we come to the problem of finance. Both capital payment in the first instance, then payment for maintenance and running costs later.

FINANCE

This is the most difficult aspect to discuss as each country will have it's own methods for providing capital, or subsidies.

I have heard it said that money is a limited resource. Technically money in itself is not a resource, it is merely a proxy for men and materials which are the true resources. This is not merely an academic argument as on the ground it is the availability of local skills and materials that matters. A project financed from outside with all the hardware and expertise also coming from outside, will be less likely to operate successfully, and continuously than a system so run that when repairs and renewals become needed, they can be provided locally and also the continuing skills for operation. A total package delivered as a present from elsewhere will benefit the country that gives it more than the one that receives it in the long term.

DEPRECIATION

The level and rate at which depreciation should be charged in the accounts is probably the most difficult decision, that needs to be taken. No charging system that attempts to relate charges to full economic costs can be calculated until a depreciation policy has been decided.

In writing this paper I am bound to consider that 'developing countries' means what it says, i.e. that we are considering countries that are developing a better economy, and a higher standard of living for it's population, however slowly. This point has a bearing on depreciation policy and in fact on the whole problem of self generation of capital. If the hope is realistic that the economy is improving then it may be easier for the next generation to afford a good water supply. If this is so, but only if it is so, then the charge for depreciation need do no more than meet the capital redemption part of the financing charges. If the original loan is for 25 or even 50 years, this means that the debt will be repaid in that time, and if any of the assets last longer than that, as many should, they will appear as nil value on the books. But in fact the next generation will not be as lucky as might appear at first sight, as there will be no money available when replacement becomes due, and historically there has always been long term inflation however slight the replacement of the worn out assets and will be considerably more expensive in money terms. If the generation then needing to replace is in fact better off, it may be alright, and they may in fact demand a more sophisticated system that would be more expensive in real terms anyway. But this method of financing is not to be recommended if money can be found to finance a proper depreciation policy based on the current cost of replacing those assets. Not to do this is to build up a large replacement problem for later on and is not good practice; however the immediate problem is how to finance and maintain a good water supply where none may have existed before; so it may be that temporarily the future must take care of itself.

PAYMENT

There can be two alternative objectives considered when deciding what method of payment is appropriate:-

1. Payment to recover all true costs.

2. Payment designed to control use and act as an indicator of future demand.

The first method is basically an accountancy exercise with the second you also include an element of value which tends to be more of an economists' concept.

In providing any of the essential services the closer the relationship between payment and use made of the service by the consumer, the better. The further away from that ideal you get the closer you get to taxation rather than charge.

However in the countries that we are considering the ideal may well be impossible to start with. Almost certainly the costs of installing and maintaining a system will have to be recovered by taxation to begin with.

But even here there is a choice. If payment is to be made by central government then they will raise that money by a loan or general taxation, either by personal taxes or company taxes or on various forms of purchase tax. If there is no local authority competent to raise the necessary funds then there is no alternative to central government payment.

However, if there is some form of local authority already in existence then local taxation would be preferable in that it brings the relationship between payment and user, closer together. The most usual form of local taxation is a property tax, but there are alternatives such as a poll tax, local income tax, or sales tax. On the whole a property tax is easier and cheaper to collect, and ease and cheapness of collection is extremely important.

It may be that central government will have to subsidise the system to start with, but this is not reducing the cost but merely transferring the payment to central government taxation.

If the country's economy is sufficiently advanced so that a charge related to use can be brought in, then it is almost inevitable that water meters should be installed. But some costs will inevitably be increased as water meters require regular maintenance, also they must be read and recorded, and a system of billing must be introduced. To install meters that are not regularly serviced and read, is a waste of effort and money. A billing system whereby accurate bills can be sent out regularly, is absolutely essential and requires skills that may not be too abundant in many places.

If it is decided to install meters then, as mentioned earlier, thought must be given early on whether it is intended just to recover costs or whether some control of use is required.

If recovery of costs alone is wanted, then a two part tariff is preferable. In many cases 90% of costs are fixed costs in advanced systems. So that these costs continue regardless of the amount of water supplied. It is the availability of the supply that is expensive, the variable costs which mostly mean power for pumping and chemicals is quite a small proportion of overall costs.

VALUE

If however some attempt is to be made to control water use by charge than there should be no two part tariff, all the charge should be levied on the volume related part of the charge, without doing that too small a proportion of the charge will be within the control of the consumer, and by turning off a tap he will not be able sufficiently to affect his bill; at what point in price you do in fact get such elasticity in the demand is likely to vary considerably from country to country, and will depend on the proportion of available income that each household is able or wishes to spend on this service.

Water supply is normally a monopoly, but it is in fairly small quantities essential to life, so that, truly to charge by value would be unreasonable. The value of the next cup of water to someone dying of thirst is presumably infinite. But there are methods by which a community as a whole can decide by way of charge how much they want more water. Long run marginal cost pricing is the usually recommended method of dealing with this problem where a monopoly exists, but it is extremely complicated to work out, and basically depends on the cost of the next supply of water that would need to be brought in. I cannot imagine that the situation in those parts of the world that we are considering here would justify this sophistication.

There is one further point that needs consideration while discussing payment. Should all water be paid for at the same rate by volume?

The probability is that in most cases costs fall as volume increases; if there is abundant cheap raw water available then a tariff falling as quantity increases can be justified and this system used to be applied in many places. Unfortunately there are very few places where there is abundant, and cheap, raw water any longer; so that again there has been a move towards charging by value, and a rising tariff is fairly usual. The first block of the tariff is fairly low priced as this is needed for drinking and hygiene,so that a very real benefit is obvious, but after that the demand becomes more one of personal choice, i.e. garden watering, car washing etc. and therefore the need to conserve water is felt to be important, and a higher price can be justified.

If a rising block tariff is decided on, a judgement - probably subjective - has to be made as to how large the first low priced block of the tariff should be, and should it be the same for each house regardless of the size of the household.

TRAINING

Possibly the most important part of any scheme is training. Training of the whole community. The obvious training requirement is for those who run the system but what is frequently forgotten is training for the community in water use and water needs. In many schemes training is probably the most cost effective part of the expenditure. It is not just the paint you put on to make the job look smart at the end, it is really a vital part of the foundations. The World Bank recognises this point and it is now possible to get part of a loan for a water supply scheme set aside for training. It may be helpful to send one or two key personnel overseas for training periods but the bulk will have to be done on site, but that of the whole local community it is inevitable should be undertaken within that community. I cannot stress too strongly that to make a scheme cost effective it must be effective in its application and continued use. You can sometimes manage with quite a low cost to produce an effective scheme, all the money spent in the world is wasted unless the objective is both achieved and, most importantly, maintained.

La Demande, le Financement et le Paiement de la Distribution de l'Eau dans les Pays Sous-Developpés.

RESUME

Il est maintenant généralement admis qu'il y a un niveau de technologie approprié qui convient dans chaque cas particulier. Il serait faux d'introduire les systèmes les plus avancés dans une zone arriérée et en pratique cela ne s'avérerait pas efficace. J'affirme que ceci est également vrai pour les aspects financiers et économiques, et qu'un système simple pouvant être compris et mis en oeuvre facilement pourrait bien être le système qui convient dans tel cas donné.

En examinant l'économie de la fourniture en eau et de l'assainissement dans un pays en voie de développement, il faut tenir compte de bien des facteurs ; et avoir façon de penser assez différente de celle qui existe dans un pays plus riche et plus développé.

Paper 25

Les conditions d'une gestion de qualité a faible coût dans les pays en développement

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ABSTRACT

The creation of a water distribution service requires a minimum of competent local operators, of money, and of technical assistance.

Restrictions in these various areas may make it necessary to dismiss what is too expensive or too complicated, to concentrate on the necessary, and to put off what can wait.

So, objectives must be ranked by priority.

INVESTMENT OBJECTIVES

Before water can be sent through pipes or put in storage, it must be drawn from its natural milieu. This is usually done by means of pumping. Investments in pumping equipment then are naturally entitled to first priority.

Piping is next. It can represent up to 80 % of the investments of a water distribution service. Piping must be kept to a minimum. Pipes must not be oversized, and the materials used in their construction must be chosen in view of their resistance to corrosion and the facility with which they can be repaired.

As for treatment plants, when money is scarce or personnel is wanting in number or competence, clarification objectives must remain modest and limited to simple chlorination methods.

MANAGEMENT OBJECTIVES

The first management priority is the regular supply of spare parts and sterilizers.

The second priority is the inspection and maintenance of material : pumps, pipes, and treatment stations. Pipe maintenance is particularly necessary, because allowing leaks to develop will result in the need to increase the size of pumping stations and that of treatment plants, if there are any.

Next, good accounting management must be ensured, the formation of personnel must be cared for, and an efficient administrative infrastructure must be established.

Pour la gestion de leurs services d'eau, ce sont les pays en développement qui ont le besoin le plus pressant d'une gestion de qualité à faible coût. Epaulés par les aides bilatérales ou multilatérales, ils développent des efforts importants dans ce sens. Il faut cependant reconnaître que ces efforts sont loin d'aboutir aux résultats que l'on pourrait attendre en connaissant la valeur et la volonté des hommes qui les mettent en œuvre. Trop souvent la qualité des services rendus (quantité, qualité de l'eau) est insuffisante, ou le prix de revient est excessif, l'un d'ailleurs n'excluant pas l'autre, malheureusement.

Cela provient de ce que les pays en développement doivent affronter à un certain nombre de difficultés spécifiques. Il semble important d'analyser ces difficultés afin de dégager les différents axes d'action le long desquels il faudra faire porter les efforts pour améliorer progressivement la situation.

La première difficulté, souvent évoquée, est le manque de cadres moyens. Le développement de l'enseignement technique pour les jeunes en âge scolaire, la création ici et là de centres de formation pour les agents déjà engagés dans la vie professionnelle, la multiplication des stages de formation pour les cadres dans des sociétés de distribution d'eau mieux assises et disposant d'une longue expérience, porteront certainement leurs fruits à terme.

Mais il faut reconnaître qu'il manque souvent au jeune technicien entrant dans la carrière professionnelle ou au stagiaire de retour dans son service d'eau, deux éléments essentiels dont disposent les agents des pays industrialisés. C'est d'abord une organisation structurée qui le contrôle, qui le pousse et l'aide à donner le meilleur de lui-même dans l'exercice de sa tâche. C'est aussi le contact avec des agents confirmés dans la pratique du métier. C'est par ce contact que celui qui sort d'une formation acquiert ou développe le goût du travail bien fait, la satisfaction de tirer du matériel mis à sa disposition les services pour lesquels ce matériel a été conçu, en lui dispensant les soins nécessaires. C'est un ensemble de façons de faire, de façons de prendre le travail, de procédés qui ne peuvent qu'échapper en général au discours scolaire et qui en fait s'intègrent progressivement dans la personnalité de l'agent.

Une difficulté moins souvent évoquée et pourtant lourde de conséquences réside dans l'imprécision des informations qui parviennent aux décideurs (Direction Générale, Directions Régionales, Directions Techniques et Administratives) sur ce qui se passe réellement. Pris par les problèmes de difficultés budgétaires et des problèmes de personnel, surchargés du fait du petit nombre de cadres de haut niveau dont ils peuvent s'assurer le concours, obligés parfois de rentrer dans des détails du fait du manque de cadres moyens, ils ne sont plus en mesure de contrôler ou de faire contrôler les renseignements qui leur parviennent, et par conséquent ne peuvent pas réagir opportunément aux insuffisances.

Nous savons tous pas exemple combien il est difficile de connaître avec une bonne exactitude le rendement d'un réseau, le soin qu'il faut apporter à la surveillance de l'exactitude des indications fournies par les appareils de comptage : aux points de production d'une part, aux points de distribution d'autre part.

De même la surveillance de la qualité bactériologique de l'eau distribuée suppose des laboratoires fiables, bien équipés et pas trop éloignés des agglomérations à contrôler, des agents bien au courant des précautions à prendre pour effectuer les prélèvements dans de bonnes conditions, un choix des points de prélèvement en nombre suffisant et avec une bonne répartition, une périodicité relativement fréquente des prélèvements, un transport des échantillons jusqu'au laboratoire à la fois soigneux et rapide, un ensemble de procédures de contrôle à déclencher en cas de résultats douteux.

Sur ces deux sujets : rendement des réseaux, qualité bactériologique de l'eau, particulièrement simples et évidents pour une bonne gestion de services d'eau - comme pour bien d'autres - on voit sans peine que la tâche de contrôle ou même de simple connaissance des données techniques (et il en est de même des données comptables) est loin d'être aisée.

Enfin la troisième difficulté qu'il semble indispensable d'évoquer est celle que l'on peut éprouver généralement à assurer un entretien et des réparations de bonne qualité, ceci afin de maintenir les installations dans un état de service constamment analogue à celui pour lequel elles ont été conçues initialement. L'éloignement des sources d'approvisionnement d'une part, les difficultés de transport à l'intérieur de ces pays d'autre part rendent difficile, longue et coûteuse l'acquisition des pièces de rechange. La disparité des matériels ne rend pas aisée la tenue d'un magasin pour ces pièces de rechange même par des spécialistes dont d'ailleurs ces sociétés ne disposent pas en nombre suffisant.

De plus, devant les obligations propres au service d'eau, qui doit assurer un service continu et si possible sans défaillance, la tendance du haut en bas de l'échelle est de faire appel à la débrouillardise et au bricolage (parfois très astucieux) lorsqu'un incident se produit.

Disons tout de suite que cette tendance est parfaitement justifiée dans ses motivations premières, et qu'elle a son principe dans un désir louable d'assurer le service public quelles que soient les difficultés. Mais malheureusement c'est au détriment du long terme : l'état du matériel se dégrade et sa durée de vie diminue.

Cette tendance - souvent imposée par les circonstances - ajoutée aux contraintes budgétaires et à l'information insuffisante des décideurs, se développe au point de retarder les opérations d'entretien et de réparations, de se contenter de faire au mieux avec ce que l'on a, d'utiliser du matériel dégradé et donc inadapté. Cette situation entraîne toute une série de conséquences dont trois au moins sont graves :

- les prix de revient augmentent, ce qui ajoute aux contraintes budgétaires
- la qualité du service diminue
- le personnel loin d'acquérir les bons réflexes et les bonnes habitudes dont il a été question ci-dessus perd le goût et le sens du travail bien fait.

Progressivement des données, pour la plupart extérieures à la société des eaux elle-même, qu'elles soient techniques ou économiques deviennent les causes d'une sorte de défaut interne dont les conséquences ne peuvent qu'être coûteuses. En un mot, ces sociétés sont amenées souvent à négliger la gestion au sens économique du terme pour se contenter de colmater les brèches.

Dès lors, tentons de définir comment faire pour améliorer cette situation, essentiellement quelles erreurs redresser, dans quelles directions faire porter les efforts, tout en sachant qu'un rapport de cette nature ne peut être que partiel, et qu'il devra être adapté voire corrigé dans chaque cas particulier.

En premier lieu, il semble que les critères de choix du matériel mis à la disposition des services d'eau de ces pays doivent être précisés sinon modifiés. Ces matériels doivent être :

- aussi homogènes que possible dans une agglomération et même dans un pays
- résistants, fiables, d'une longue durée de service
- bien adaptés au personnel qui sera chargé de les exploiter.

Parler de matériels homogènes, c'est évidemment aborder un sujet délicat et peut-être heurter des habitudes souvent honorables. Mais il est parfaitement anormal que dans un pays où il n'y a que quelques dizaines de milliers de branchements on trouve quatre marques de compteurs individuels différents ou plus, que dans une station qui comporte une dizaine de pompes, celles-ci proviennent de trois fournisseurs différents. De tels exemples pourraient être multipliés.

Le système de la mise au concours élargi est certes justifié dans de nombreux cas, celui des crédits liés parfaitement compréhensibles, mais si nous voulons rendre l'entretien plus facile et moins coûteux, il sera nécessaire d'imaginer des procédures qui en limitent les inconvénients majeurs tels que ceux que nous avons évoqués ci-dessus.

Organiser un parc de pièces de rechange rationnel est déjà délicat dans un pays industrialisé où les fournisseurs sont à proximité, où un personnel administratif compétent sait parfaitement définir, individualiser et stocker les pièces nécessaires, où des techniciens ont été habitués à s'adapter à des matériels différents. Que dire d'un pays éloigné des fournisseurs avec les difficultés de communication que cela implique, où les frais de transport à l'intérieur du pays sont élevés et enfin où les difficultés de recruter du personnel compétent en quantité suffisante sont très grandes ?

Rappelons également la nécessité de prévoir un matériel d'une grande robustesse, facile à entretenir et à réparer et d'une longue durée de vie. Les installations de pompage et de traitement, les canalisations, les appareils de robinetterie et de fontainerie devraient être envisagés aussi sous cet angle.

Pour arriver à ce résultat, il faudra infléchir certaines habitudes qui sont les nôtres et amener les fournisseurs à s'adapter à des nécessités qui ne sont plus (ou pas encore) celles avec lesquelles ils sont confrontés actuellement.

Même un peu plus onéreuse à l'achat, et d'un rendement inférieur de quelques points, une pompe tournant moins vite peut être préférable dans de nombreux cas.

De plus il n'est pas rare de constater que dans nos pays les caractéristiques des matériels changent rapidement : ce qui a été mis en vente il y a quelque 10 ou 15 ans est souvent abandonné, y compris la fabrication des pièces de rechange.

C'est une pratique conforme à l'environnement des pays industrialisés, même si elle entraîne parfois pour ceux-ci des frais excessifs. Les données des problèmes eux-mêmes y changent rapidement : pollution des eaux de surface, caractéristiques des nappes souterraines, déplacement et exigences de la population. La main d'œuvre y est chère, elle était rare il n'y a pas si longtemps mais les techniciens adaptés aux matériels les plus modernes se trouvent aisément, et donc la recherche de la meilleure performance devient immédiatement rentable.

Par contre pour les pays en développement le rapport prix installé sur prix d'achat d'un matériel est beaucoup plus élevé, l'allongement de la durée d'amortissement technique est donc un élément favorable.

Dans le cadre de la décennie de l'eau, un effort important va être fait dans le monde entier, le marché des fournitures aux pays en développement va donc s'étendre, et il est normal d'alerter les fournisseurs sur les contraintes de ce marché qui ne pourra être l'extrapolation pure et simple de celui des pays industrialisés. Au contraire des agglomérations de plus en plus éloignées et isolées vont être équipées et les contraintes évoquées ci-dessus seront encore plus vivement ressenties. Les frais de renouvellement ou de réparations seront élevés, le personnel difficilement contrôlé, il faudra donc lui donner un matériel qu'il puisse maîtriser. A cette condition il comprendra également que s'il l'entretient mal il pourra être inéluctablement sanctionné. Le contrôle en sera facilité d'autant.

Ajoutons aussitôt que la recherche de la robustesse ne signifie pas que les technologies fines d'automatismes ou autres ne puissent pas être envisagées-loin de là. Il est certain par exemple que des dispositifs de sécurité sur les pompes seront souvent nécessaires, le surveillant pouvant toujours être sujet à des défaillances. De même certains pays qui disposent de sources d'énergie à bon marché peuvent être tentés d'utiliser l'ozone en remplacement du chlore cher en devise. Il faut simplement vérifier que la solidité, la facilité de réparation et d'entretien et la durée de vie de ces installations sont compatibles avec les besoins des utilisateurs.

Il faut maintenant aborder le problème de la conception même des projets. Pour créer ou améliorer les services d'eau, il sera toujours nécessaire - du moins dans un avenir prévisible - de faire appel à des bureaux d'études de haute technicité et détenant une large expérience qui leur permette de détecter la meilleure solution technique à chaque problème qui se pose.

Mais il semble utile de favoriser dès maintenant la création ou le développement d'organismes nationaux ou régionaux chargés aussi bien d'étudier des améliorations ou des petites extensions que d'en contrôler l'exécution ou d'en vérifier la qualité après mise en route. Il s'agit-là de travaux quasi-quotidiens d'autoadaptation que les directions doivent engager si elles veulent améliorer la qualité de leur service. Ces travaux, en général de faible envergure, sont déclenchés lorsqu'une installation ne donne pas pleine satisfaction à un moment donné. Les organismes chargés de ces études pourraient être soit des bureaux d'études locaux composés de spécialistes du pays ou du groupe de pays où ils exercent, soit tout simplement une direction des études et travaux de la Société Nationale des Eaux du pays concerné.

Pour cela il sera nécessaire de faire un effort particulier en faveur de la formation de quelques ingénieurs d'études. Après avoir acquis la formation de base dans les écoles ou instituts spécialisés, ils auront à acquérir le savoir faire indispensable au cours de stages d'application spécialement préparés et suffisamment étoffés, et par un travail en commun avec des collègues étrangers.

Une fois rodés ces organismes acquerront le poids nécessaire pour devenir les interlocuteurs des grands bureaux d'études étrangers ; ils pourront alors éclairer leurs collègues sur les contraintes spécifiques qui seront rencontrées dans le secteur qui fait l'objet de l'étude, sur le type de matériel qui convient, sur les capacités du personnel qui sera recruté pour exploiter ce matériel. Un échange fructueux pourrait intervenir entre ceux qui disposent de la connaissance internationale sur les techniques et ceux qui, de l'intérieur, connaissent et vivent les contraintes locales. Ainsi pourraient être mieux évités les inconvénients déjà signalés ci-dessus.

Mais cet organisme national, lui-même, aura le poids et la compétence voulue non seulement par les valeurs de ses hommes mais aussi par la connaissance de ce qui se passe réellement dans les services d'eau du pays, en un mot si la Direction Générale, convenablement alertée lorsque des problèmes se posent, lui demande d'étudier les solutions adéquates. Or nous avons vu que c'était une des difficultés auxquelles on se heurte souvent : l'information doit circuler de façon précise et rapide. Un effort d'organisation et de rigueur semble nécessaire.

Comme dans tous les cas semblables et dans tous les pays, un conseil extérieur peut grandement aider à améliorer cette situation, à condition qu'il sache se mettre à l'écoute, s'adapter aux conditions locales, tenir compte des habitudes et de l'environnement, tout en appréciant les possibilités de les infléchir. Toutefois il est évident que c'est de la Direction Générale que devra venir l'impulsion nécessaire et l'effort principal.

L'information vient des opérateurs, par définition. Ceux-ci devront sentir qu'ils seront entendus et compris des échelons supérieurs. De même chaque échelon devra sentir qu'on lui sera reconnaissant de donner des informations exactes, que ses suggestions seront écoutées et éventuellement étudiées et qu'en aucun cas il ne lui sera fait grief de ses remarques si elles sont faites dans un but de rigueur et d'amélioration du service rendu.

Les documents émis par les opérateurs devront être synthétisés et commentés à chaque échelon, de façon à ce que chaque niveau de décision dispose des informations nécessaires à l'accomplissement de sa tâche sans être submergé par des détails inutiles.

Les informations d'ordre comptable dont la rigueur risque parfois d'être négligée par des cadres de formation surtout technique ont également besoin d'être connues avec précision. Dans un premier temps la comptabilité générale et surtout la comptabilité analytique n'ont pas besoin d'être très complexes, mais il est nécessaire qu'elles soient aussi exactes que possible et qu'elles permettent de dégager les chiffres essentiels grâce auxquels pourront être détectées les insuffisances et les charges excessives. La comptabilité deviendra alors un outil de gestion car elle permettra de prendre des décisions pour redresser les anomalies.

En particulier, elle doit permettre au Gouvernement du pays concerné de prendre des décisions motivées sur le prix de vente de l'eau à appliquer dans le pays pour permettre à la Société Nationale des Eaux de disposer des ressources nécessaires pour une bonne gestion (y compris les frais de renouvellement). Des spécialistes existent pour effectuer de telles études, mais leurs conclusions seront d'autant plus fiables que la comptabilité sera mieux tenue.

Dans cette communication il n'a pas été donné aux problèmes de formation la place qui leur revient en réalité. C'est d'abord parce que le temps est mesuré, mais aussi parce que la prise de conscience de la nécessité de la formation et du recyclage est maintenant assez générale pour qu'il soit moins nécessaire d'insister sur ce point.

Il peut simplement être utile de préciser que l'amélioration à attendre de la formation ne pourra être que progressive. Cette amélioration sera d'autant plus notable que la réinsertion du stagiaire dans son cadre de travail sera faite avec plus de soin sur deux plans essentiels déjà évoqués : la qualité du matériel mis à sa disposition, l'organisation d'ensemble de la Société des Eaux.

Un effort considérable déjà amplement amorcé va se développer au cours des dix prochaines années pour effectuer des travaux de distribution d'eau potable dans les pays en développement. De nombreux organismes vont être concernés par cette tâche : les Gouvernements de ces pays, les Directions Générales de leurs Sociétés des Eaux, les organismes de financement, les agences d'exécution, les consultants internationaux de toutes natures, les entrepreneurs, les fournisseurs de matériels, etc.... Le modeste propos de cette communication est d'aborder différents aspects de ce truisme fondamental : un service d'eau est fait pour être exploité dans les meilleures conditions et à la satisfaction des utilisateurs, c'est-à-dire pour assurer la distribution d'eau en quantité suffisante sans gâchis et sans contraintes excessives. C'est simple mais cela demande de la part de tous beaucoup de rigueur dans la réflexion et dans l'élaboration, beaucoup de souplesse dans l'application, beaucoup de volonté dans la poursuite de l'objectif final, en un mot beaucoup d'efforts et pas seulement financiers pendant dix ans ... et au-delà bien entendu.

Paper 26

Renewable Energy Sources and Water Treatment

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ABSTRACT

In this article we are considering the possibilities of renewable energy sources within the context of low cost technology for water supply and water treatment. The problems of the sources we use today (fossil fuels and uranium) are outlined. Some data are given about the renewables and also the energy use of several energy-intensive processes in the field of water supply are mentioned. Cabo Verde is used as an example of a country where wind energy is being introduced on a large scale. It is stressed that, since there are several sources and many conversion systems, a proper choice is difficult to make. Finally, the possibilities of multiple purpose applications are mentioned.

INTRODUCTION

Energy is nearly always needed in the field of water supply and water treatment. Until now, mostly fossil fuels have been used. For the time being these energy sources will remain important and one of the preliminary demands is to use this energy more efficiently. However, the use of fossil fuels causes many problems (see 2). These problems are so immense that many people believe that a total change of the present energy system has to be brought about as soon as possible. One of the problems is cost. This is true both for the single customer, who cannot afford the money, as well as for a country as a whole, because there is not enough foreign currency available.

The use of renewable energy sources (sun, wind, organic materials, etc.) may be an answer to the problems connected with the use of fossil fuels and uranium, and is often also applicable within the concept of low cost technology.

In this article we will look at water supply from the point of view of the energy supply with renewables in connection with low cost technology. First we will briefly state the problems that we are facing with the energy sources we are using today and especially third world aspects will be mentioned. Secondly, we will deal with the renewables and we will describe the systems in which they are used. Thirdly, we will give some examples of applications. Finally, we will discuss some aspects of costs and we will make some general remarks.

2. ENERGY SUPPLY

The use of fossil fuels and nuclear energy causes many problems. We will first mention some general problems (1,2) and then aspects especially connected with third world countries.

The general problems can be divided into five main areas:
1. Environmental problems (e.g. the increase of carbon dioxide in the atmosphere, or the storage of radio-active waste).
2. Exhaustion of supplies.
3. Dependence on countries rich in raw materials.
4. Economical aspects. Fossil energy has become expensive. For instance after 1979 the prices of common energy have risen by a factor of about two.
5. Complexity and vulnerability of the energy infra structure. This especially holds for the western world.

The most important advantage of the renewable energy sources is the fact that the problems mentioned above are far less severe or completely absent. So it is of utmost importance to develop these sources as quickly as possible.

Several aspects apply particularly to third world countries.
1. While analysing the possibilities of renewables, we have to realize, that there is often more energy from these sources available in these countries compared to Western countries. Solar energy and organic materials are two examples of this fact.
2. There may be not enough foreign currency to buy the fossil fuels or a nuclear power plant. If a country can produce energy from its own sources, foreign currency is available for other purposes.
3. In many countries the infra structure needed for transport of energy is not available. It is not only the tremendous amount of money needed to install such a complete energy system but also the fact that even if the system has been installed after some decades, problems will be generated because of decreasing supplies. For this reason many people state that it is more attractive for developing countries to start immediately with the introduction of renewables in full extent.

These points are independent of culture, social structure and technological skill of the countries in which the energy has to be used. However, the following question could be raised: "which energy sources are most appropriate for a certain culture". The answer on this question has been outlined within the topic of Appropriate Technology.

Renewables can be applied to a broad range of applications taking into account local demands, such as adaption to available skills (3). The renewables are suitable, for both simple and advanced concepts, besides,they can be utilized in centralized and decentralized schemes. So it seems quite possible that each country, or part of a country (rural areas) develops renewable energy systems on its own way from its own sources.

3. RENEWABLE ENERGY SOURCES AND SYSTEMS

We will treat the main characteristics of the renewable energy sources and will deal with several conversion systems in general. Of each energy source we will give the most usual forms in which the energy is converted.

Historical background and future expectations

It is not the first time in history that people have high expectations of renewable energy sources. Between 1955 and 1960 also much attention was given to these sources, especially for third world countries. However, the development stopped. Firstly because of the existence of cheap abundant fossil fuels and secondly because third world countries usually follow the tendencies of the western world. When the development came to a standstill in the developed countries, this also happened in other countries. As research, development and introduction has been started again in developed countries, it is expected this also will take place in third world countries.

The expectations of the renewables are very high. In the U.S.A. a program has been started and it is anticipated that 20% of the energy can be supplied by renewables in the year 2000. Several european countries have a planning aim of 5 to 10%. As these energy sources are more attractive for third world countries, 10 to 20% seems feasible.

Solar energy (4,5,6)

To get an idea of the amount of energy we can harness, first some numbers are given. Perpendicular to the solar rays, the radiation density is about 1 $kW.m^{-2}$ and an annual average energy of about 2000 $kWh.m^{-2}$ is received. So, if the conversion efficiency is 70% (heat) or 20% (electricity), then 1400 $kWh.m^{-2}$ (thermal), resp. 400 kWh (el.) can be obtained.

a) Solar collectors.
These systems are especially suitable for the production of low temperature heat (below 100°C). A collector consists of an absorber behind a glass plate, which collects the radiation. The heat is transported by water or air to a storage vessel. A lot of development is being done in this area which has resulted in reliable simple systems. All we have to do is to adapt these collectors to local conditions and needs.
Energy form: low temperature heat.

b) Concentrating systems.
To obtain high temperatures, concentration of radiation is necessary. Several systems are being tested these days and it is not yet clear what will be the "best" system. There are uncertainties about reliability, lifetime and the maximum amount of energy which can be obtained. The production of mechanical energy (electricity) is mostly performed by a Rankine or Stirling cycle. A lot of research and development will be necessary before the optimal system is available.
Energy form: high temperature heat (above 100°C); mechanical energy or electricity.

c) Distillation equipment (solar still).
Such a device consists of a container with brackish or salt water over which a glass plate has been placed in such a way that a greenhouse-like construction is obtained. The water is heated by radiation, evaporates and condenses at the tilted glass plate. Then the water flows along the glass plate into a gutter and is drained away. It is a simple reliable system which is being used at many places. The production is about 4-6 $l.m^{-2}.day^{-1}$.

d) Solar cells.
These cells convert light directly into electricity. The efficiency is between 10 and 20%. Until now only silicon has been used as a material for solar cells. The advantages of these cells are: simplicity, hardly any main-

tenance, no moving parts, long lifetime and the practically limitless amount of silicon. A few years ago intensive R & D programs have been started to produce cheap cells. Once such a process is available, nearly all countries can make these cells themselves. For instance today countries like Iran, Egypt and Mexico are building their own factories for the production of cells. Energy form: electricity.

Wind energy

The power that can be obtained from the wind per m^2 swept rotor area lies between $0.1\ V^3$ and $0.25\ V^3$ depending on the type of system (V is the average wind speed in ms^{-1}). For instance, the energy from a simple water pumping windmill, operating in a moderate wind regime with an average annual wind speed of $4\ ms^{-1}$, that is available to lift the water is about 60 kWh per m^2 swept rotor area per year. The power is about 7 watts.

Since ancient times windmills have been used for water pumping and several kinds of industrial purposes such as grinding, manufacturing of cloth, paper, oil.

Modern wind energy research and development is aiming at wind energy systems in a wide range of installed powers: from a few watts to about 5 MW. The development of electricity generating windmills receives most attention, although several programmes are dealing with mechanical water pumping systems especially for developing countries (7).

Besides the classical horizontal axis windmill, the vertical axis "Darrieus" rotor is subject to substantial research efforts. In the near future from experiences with mills of both types it will be possible to determine which type under which circumstances is most attractive.
Energy form: mechanical- and electrical energy.

Organic materials

Wood and other organic (waste) materials belong to the oldest energy sources used by mankind. Wood is normally used for cooking and heating purposes. Conversion efficiencies are often rather low. For this reason many people are looking for better burning systems. Also conversion of wood by gassification or by fermentation is possible. The latter process appears to be attractive because the residue is usefull as a fertilizer. However, intensive research has still to be done on optimising the processes. Another process is the conversion of organic materials into alcohol or other useful fluids. Research on these processes has started recently. There are two reasons why organic materials are not used to a large extent as a source of energy. Firstly, nature has a low conversion efficiency (about 0,5 to 1%), so large areas would be needed. This means that about 10 - 20 kWh of energy in the form biomass is obtained per m^2 per year. Secondly, the land is needed for food production.
Energy form: like fossil fuels.

Hydropower

Several types of water turbines are available (3). The choice of turbine will depend on the head of the water and the local possibilities of manufacturing a certain type. The efficiency of hydropower systems can be very good (90%).

In recent years especially small systems have been introduced (China). Also the hydraulic ram opens new perspectives for the use of hydropower to pump water in areas with a very specific terrain condition. The maximum power available from a water flow of 1 m^3s^{-1} is 9.8 10^3 hW (h is the head in m). Energy form: mechanical or electrical energy.

Muscular power

Our muscles are still an important source of power. However, a more efficient use of it is quite possible. The amount of energy that one person can produce is less than 1 kWh per day.
Energy form: mechanical.

Radiation cooling

Especially in dry areas the cooling effect of the emission of radiation during the night can be utilised. The system can be simple: a black surface with a plastic cover, transparant for infrared radiation. However, there are not many experiments reported in literature. Over a period of 10 hours the maximum amount of energy per m^2 emitted to the environment is 3.2 kWh at 0°C. In practice num-like 0.7 kWh have been obtained (8). This means that about 7 kg of ice can be produced per m^2 per day. A solar still produces about the same amount of pure water.

Other sources (9)

Other renewable energy sources are: geothermal energy, energy from the temperature gradients in the oceans, tides, energy from waves, etc. However, presently these sources seem not very interesting from the point of view of water supply.

4. ENERGY CONSUMPTION IN WATER SUPPLY AND WATER TREATMENT

For nearly all processes in the field of water supply and water heatment energy is needed. We will pay attention to some of them.

Desalination

One of the processes which consumes a lot of energy is desalination of salt- (sea) or brackish water. In Table 1 for several processes the energy needed to obtain pure water from sea water per m^3 is given. These numbers have been compiled from several literature sources (10 - 15).

This table shows that a solar still needs relatively much energy for each m^3 meter of pure water produced. This does not mean, however, that it is not an attractive system. Especially from the point of view of low lost technology it may be the best choice for producing potable water.

Until now the multiple flash system has been mostly used. However, the reverse osmosis (R.O.) is gaining field fastly. It is not only the low energy consumption that makes the R.O. system attractive. Two other very important aspects are: Firstly, the energy use is about proportional to the salt content. So, for the production of clean water from brackish water, less energy is needed compared with sea water. Secondly, usable water is obtained as a by-product of the use of R.O. processes for the treatment of waste-water.

Table 1. The amount of energy in kWh needed to obtain 1 m^3 pure water from sea water; th = thermal, e = electrical.

Solar still	1500 kWh (th)
Multiple flash	65 kWh (th)
Vapour compression	12 kWh (e)
Electrodialysis	5-10 kWh (e)
Reverse osmosis	5-10 kWh (e)
Minimum energy	0.7 kWh (e)

Theoretically, the minimum energy needed for the desalination of sea water is 0.7 kWh (e) per m^3. One can see from Table 1, that we still can gain a factor 10. It must be a challenge for research workers to create systems which approach this number. Keeping in mind the enormous amount of sea water, it is somewhat reassuring that the water supply does not has to be hopeless. If an energy use of say 1.5 kWh (e).m^{-3} (twice the minimum) can be obtained, energy costs will be low for the production of pure water even with todays energy prices. In this context we have to remark that renewable energy sources can normally deliver energy at a price lower than todays common prices.

In connection with Table 1 it can be noticed that the land use in case of solar energy as an energy source is large. In Table 2 the number of m^3's of pure water produced per m^2 solar collector surface has been given. We assume that about 1400 kWh (th) m^{-2} and 400 kWh (e) m^{-2} per year can be obtained. Looking at the numbers of Table 2 we have to keep in mind the fact that about 1 m^3 of water per m^2 per year is needed for the production of crops.

Table 2. The numbers of m^3 pure water per year one can get from sea water per m^2 collector surface for several desalination systems.

Solar still	0.9
Multiple flash	22
Vapour compression	33
Electrodialysis	40-80
Reverse osmosis	40-80

In Table 3 some numbers are given which are of importance for the applications mentioned below.

Table 3. Energy needed per m^3 of water for several processes.

Freezing	93 kWh
Heating from 0^oC - 100^oC	112 kWh
Evaporation	600 kWh
Pumping (h meter height)	$2.73.10^{-3}$ h kWh

Pumping of water

A very important application is pumping of water. It can be seen from Table 3, that the energy needed for this purpose is very low compared with desalination (Table 1). For instance to pump 1 m^3 of water over a head of 10 m only 0.001 part of the amount of energy to desalinate 1 m^3 of sea water is needed.

Desinfection

Water from rivers or lakes can be made potable by desinfection. To do this, distillation is not needed and heating for some time near the boiling point will do. In Table 3 the amount of energy for heating water from $0^{o}C$ to $100^{o}C$ is given.

Hot water supply

For many purposes hot water or steam is needed. We have to realize that in many countries 50% of the energy is used as low temperature heat (< $100^{o}C$) and 20% for high temperature heat (> $100^{o}C$). The application may vary from hot water supply of a hospital to high temperature water in industrial or agricultural processes. If steam has to be produced considerably more energy is needed than in the case that only hot water is needed (see Table 3).

Other applications

An example is the production of ice or low temperatures. In this case the energy needed per m^3 is about the same as for heating water to the boiling point and much less than for evaporation. Three basic ways can be used: obtaining low temperatures by absorption cooling, the use of the heat pump mechanism or by radiation cooling.

5. APPLICATIONS

The applications of renewable energy sources such as wind, hydro and wood in connection with water supply and treatment date already from ancient times. Radiation cooling has been applied in old Persia. In the field of solar energy the first solar stills have to be mentioned, that were built in Chile in 1872 to provide drinking water for the mules in the nitrate mines.

However, during the last decades there has not been much interest in the renewables, until recently when an impressive rebirth could be observed. We will concentrate on the recently started applications. As not all applications can be described in full extent we will go into some detail on an application of wind energy at the Cape Verdian Islands (West Africa).

The fact that so many alternative energy systems are being introduced today proves that these systems are already very attractive as low cost technologies.

Wind energy applications at the Cape Verdian Islands

The Republic Cabo Verde is an archipelago consisting of 9 populated islands situated about 460 kilometers west of the coast of Senegal. Its population consists of about 300 000 people living on a surface of 4000 km^2. The archipelago lies within the Sahel zone and suffered severe draughts during the last 10 years.

Cabo Verde is particularly suited to apply wind energy systems because of the trade winds. The wind speed at typical windmill sites is about 7 ms^{-1}, the wind direction being very constant.
The most promising applications of wind energy (W.E.) on these islands are:

1. Water pumping for irrigation, domestic supply and cattle watering.
2. Generation of electricity to be fed into small local grids or in connection with an isolated diesel set.
3. Desalination of sea water and brackish ground water.
4. Small scale cooling plants.

ad 1

The Government of Cabo Verde aims at pumping all water with renewable energy sources within a period of 20 years.

Most water is obtained from deep wells and hand dug wells. In Cabo Verde 75% of these existing and future wells can be equipped with wind energy systems. These systems may consist of piston pumps directly driven by wind rotors or submersible turbine pumps powered by electricity from a wind turbine. For the first type of system the ministry of Rural Development is preparing a local production and implementation scheme.

By applying W.E. systems instead of the usual diesel sets an annual saving of about 870 m^3 oil could be realised, being almost 10% of Cape Verde's total oil import in 1977. Besides conservation of energy and saving of foreign currency, the use of W.E. systems has the advantage that local employment in the windmill production and maintenance sector can be created. The expected water costs of a windmill driving a piston pump and pumping from a deep well of 55 meter depth are about \$ 0.04 per m^3, whereas a diesel set would do it for about \$ 0.07 per m^3 (16). (For the diesel set no storage tank, whereas for the windmill a 2 days storage tank was included).

Electrical W.E. systems for water pumping are presently being installed but these are experimental set ups of which still experience has to be gained before conclusions on the technical and economical feasibility can be drawn.

ad 2

As only fuel costs for diesel generated electricity presently amounts already to about \$ 0.15 - \$ 0.30 per kWh and windmills can generate electricity for about \$ 0.20/kWh (17), it is evident that windmills could be successfully utilised as fuel savers in parallel with local grids such as those of the capital Praia and Mindelo. Initiatives to realise demonstration plants on the basis of commercially available windturbines are being prepared.

ad 2, ad 3, ad 4

A lot of smaller towns and villages have a diesel generator for lighting and small industries. The generation costs usually are very high because of fuel and maintenance costs and the bad load pattern.

A considerable improvement of the economics can be obtained in two ways:
1. Switching wind turbines in parallel with the diesel set.
   This leads to a decrease of fuel costs and as the operation time of the diesel sets will decrease, also the maintenance costs will do.
2. Adding desalination and/or cooling plants to the load.
   This offers the possibility of load regulation, leading to a better power factor. Because of the built-in storage capacity of these plants the varying wind energy can be utilised to a much higher degree.

In Cabo Verde, locations where the above described systems can be applied, can be found along the south-east coasts of the main island of Santiago, in fishery-villages and at farms in the interior parts of most islands.

6. COST

The term "low cost technology" implies that those systems are selected which can solve the problems in the most economical way. However, the following points have to be taken into account.

1. We would like to stress the point that not only strictly economical arguments in the selection process should play a role. The question whether a system is appropriate to the local situation or not, should be treated as equally important. This for instance means that besides the selection of an energy source, also the local availability of raw materials, labor, skills capital, workshop, etc. should be taken into account.

   From development projects carried out in the past, the actual involvement of the population appeared to be crucial for the long term success of these projects. If for instance on economical arguments, diesel sets were selected to pump water and these sets could not be maintained and repaired by local people, the low operation times would cause the diesel sets to be too expensive on the long run.

   Renewable energy sources offer suitable possibilities to involve the local population in the projects, leading to regular maintenance and thus longer lifetimes. Many types of solar stills and windmills can be mentioned in this respect.

2. In making cost calculations, the method which is being used has to be made explicit. Which are the interest and inflation rates? Are the calculations based on annuity methods or are shadow prices being used? Practice shows that the methods often may influence the results of the cost comparison considerably.

   In making these comparisons, it must be realized that prices of fossil fuels differ enormously from country to country and even within one country. Tax regulations and subsidies by which certain systems are priveliged (such as electricity and kerosine) make cost comparison even more complicated.

3. Most articles which were published between the oil crisis of 1973 and 1979 when a second increase of oil prices took place, in general claim cheaper energy from renewables than from fossil fuels. As oil prices still tend to increase this surely holds true at present.

7. GENERAL REMARKS

Sources and applications

In the previous section the aspects which play a role in the selection process of an energy system were discussed. Here we will pay some attention to the selection process itself.

The selection process implies: 1) the selection of the energy source, 2) the form in which the energy is converted, and 3) the system to be used. E.g. in a desert it is useless to use organic materials as an energy source. The use of wind energy in areas with a low wind regime, such as tropical forests, is a bad choice. Cooling can be done with electrical or mechanical energy (heat pumps), heat (absorption cooling) or radiation (radiation cooling); desalination by the use of heat (multiple flash) or mechanical or electrical energy (reverse osmosis or electrodialyses). The available skills may impose restraints on the complexity of the systems to be chosen.

Another important aspect which has to be kept in mind is the development stage of a certain technology. Systems which are in an early stage of technical development are not suitable to be implemented in areas, which lack the necessary research facilities.

Anticipated cost developments as a result of research should be taken into account. For instance the research on solar cells, which are reliable and commercially well introduced devices, is aiming at a considerable decrease of the cost of cells.

Solar concentration systems, on the other hand, are in an early stage of development and the reliability and final prices are still uncertain. Windmills for waterpumping are commercially available, although a decrease in cost is expected from the on-going research. Most wind turbines for electricity generation are in the technical development stage.

Solar collectors for low temperature heat is a product of which no important changes can be expected.

The fact that there are so many possibilities to choose in realizing a renewable energy system can be considered as a disadvantage of the renewables.

Multiple purpose

Often the energy from an energy system can be used for different purposes at the same time. The waste heat of an electricity power plant, e.g., can be utilized for desalination.

An example of how local circumstances determine the combination of applications is an irrigation system in the U.S.A. Solar cells are being used to convert solar energy into electricity. During the time that no water is needed for irrigation the electricity is used for the desalination of brackish water by reverse osmosis or, alternatively, the production of fertilizer (18, 19).

An advantage of multiple purpose systems is that the demand pattern can be easily matched to the varying energy supply of most of the renewables. A disadvantage, however, is that the selection processes which were discussed before, become yet more complicated.

8. CONCLUSION

Renewable energy sources become more and more attractive as alternatives for the common fossil and nuclear energy sources, because of the following reasons:
- absence of problems connected with the use of fossil and nuclear energy.
- most countries possess renewables in some form which could make these countries economically independent.
- generally speaking, presently renewable energy is cheaper than fossil and nuclear energy.
- renewable energy systems are suitable for decentralised applications in remote, rural areas.
- renewable energy systems can be used in a wide range of applications and they can be designed appropriate to specific local circumstances.

A disadvantage of the renewables is: The great number of possible choices between renewable energy sources, conversion systems and energy forms makes the design of an alternative energy system in a given situation quite complicated. The fact that a number of systems are still in full development make choices even more complicated.

It is emphasized that in selecting an energy system not only "common" or "standard" economical calculations should be made, but that these systems also have to be appropriate to the local situation.

There is a need for local and international consultancy organizations which are not only able to carry out feasibility studies, but can also provide specific designs and give assistance in the execution of implementation projects (20).

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DES SOURCES ENERGETIQUES RENOUVELLABLES POUR LA DISTRIBUTION DE L'EAU

par H. Beurskens, C. Daey Ouwens

Résumé :

Cet article étudie les avantages des sources d'énergie renouvellable utilisant une technologie à bas prix pour le traitement de l'eau, par rapport aux ressources usuelles de l'énergie fossile ou nucléaire à base de pétrole ou d'uranium :

- les problèmes associés à ces formes d'énergie, tels que la pollution atmosphérique et les déchets radioactifs sont absents.

- presque tous les pays ont de telles sources d'énergie et pourraient donc avoir une certaine indépendance économique.

- l'énergie renouvellable revient moins cher en général que les énergies citées.

- les systèmes d'énergie renouvellable s'adaptent bien aux applications rurales décentralisées.

- de pareils systèmes peuvent avoir de nombreuses utilisations et être faits sur mesure selon les besoins d'une localité.

Le bon choix, en matière de système à utiliser, est malheureusement difficile à faire, compte tenu notamment du grand développement des recherches en ce domaine.

Il doit prendre en considération les données d'une situation en plus des facteurs proprement économiques. Cet article appelle à la constitution, à l'échelon local et international, d'organisations de conseils capables de faire les études nécessaires et d'entreprendre les projets.