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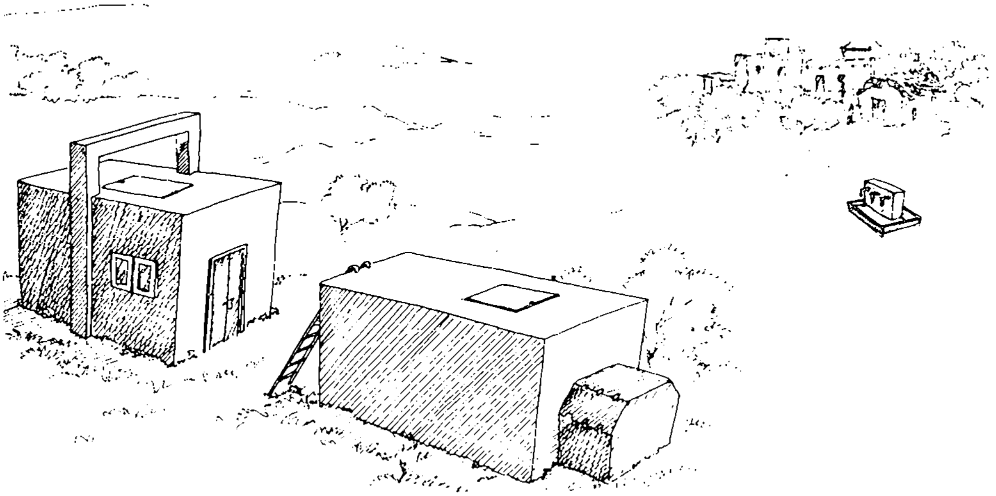
LOW COST RURAL WATER SUPPLIES AND SANITATION

A DESIGN MANUAL

FIRST EDITION

FOR
THE GOVERNMENT OF BALUCHISTAN
PAKISTAN



UNITED NATIONS CHILDREN'S FUND
PAKISTAN

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AND SANITATION

A DESIGN MANUAL

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FOR
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PAKISTAN

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LOW COST RURAL WATER SUPPLIES AND SANITATION

PREFACE

1. In 1985, both the Government of Baluchistan and UNICEF found that the manner in which rural water supply designs and construction were then being handled in the Province were leading to very expensive schemes, difficult operation and maintenance and a rate of implementation far below what was needed. No clearcut policies for the sector were available.

2. At a Donor Meeting in Quetta on 22nd September, 1985, policies were spelled out, and guidelines for future designs suggested. In 1986 and 1987 these were tried and showed that further refinements were possible.

3. In 1988, following agreement with Government, UNICEF developed a system of design and construction which is believed to offer many advantages over the former methods employed. The system is covered in this Design Manual. The system is fast, being computer based; it eliminates the need to rely on Consultants whose expertise varies very widely in Pakistan; it provides standardised designs allowing for bulk ordering of supplies and remarkably quick production of drawings; it provides both standardised Specifications and Tender Documentation (including Bills of Quantities); it eliminates the need for overhead reservoirs which are costly, slow to construct and a danger in a seismic area; it allows for community operation and maintenance; and, perhaps best of all, the system can provide low unit costs allowing for greater service coverage within a fixed budget.

4. During development of the system, it was seen that the "Modular Approach" could easily be extended to other areas of Pakistan, and thus this Manual was somewhat widened in scope for the purpose. Hence, not all aspects of this Manual are specific nor confined to Baluchistan. Indeed, it is believed that the approach may well have applicability well beyond the borders of Pakistan.

5. The Manual addresses itself to rural water supply and sanitation systems. For water supplies, communities in the range of 400 to 20,000 are generally accommodated which are typical to Pakistan. The Manual is written for engineers in Government water and sanitation agencies, and the system has already been demonstrated as being capable of being handled by Assistant

Engineers in Baluchistan. It is only necessary for designers to have a good basic understanding of the behaviour of water.

6. In sanitation, only on-site family excreta disposal systems are addressed. However, as it has been noted that the success of introduction of sanitation to communities appears to depend more on community behaviour than on technology, both are addressed in the Manual.

UNICEF, Quetta
February, 1989

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These design notes are intended to be part of the public domain; and may be copied and used without further reference to the authors. The authors take no responsibility for the accuracy of these notes, nor for any misinterpretation of them. The "Modules" will also be part of the public domain and may be obtained by public or private institutions wishing to use them upon application to UNICEF, Islamabad (WES Section). Appropriate fees may be charged to private sector applicants; while public sector bodies would generally have them provided free of cost as part of a technical assistance package.

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The World Bank, (UNDP Projects),
Water and Urban Department,
1818 H Street NW,
Washington DC 20433 U.S.A.

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1. BACKGROUND

This section reviews in some detail the reasons why this Manual became necessary; and generally how the present designs came to be. Later sections deal with the specifics of designs for ground-water and surface-water schemes together with variations such as springs, infiltration galleries, handpumps on dug-wells and finally, simple latrines.

1.1 **Costs**: In 1985, construction was underway in water supply schemes to provide facilities to around 25,000 people in some 15 different locations. The cost *per capita* (inclusive of drilling and casing boreholes where ground-water was used) averaged Rs. 2,000. This equates to US $ 125 at the exchange rate at the time. In one community, the *per capita* cost rose to above Rs. 14,500, and no less than 44 km of pipeline was contemplated for only 690 people. At the level of Rs. 2,000 per capita, the cost of placing water supplies in rural communities only in Baluchistan which boasts less than 5% of the population of Pakistan, would have been around Rs. 800 crore (nearly US $ 500 million). Not only that, but designs were assumed to cater only for a 20 year period, implying that this level of investment might be needed every 20 years. Obviously, this was untenable and more modest levels which could still provide the necessary water supplies were proposed.

This line of reasoning is applicable to other areas, too. Development finance is limited from whatever source it may come, and to provide the maximum number of people with improved water supplies implies designing and constructing to the lowest feasible unit cost without compromising function.

The present system provides, at 1988 prices ruling in Baluchistan for ground-water systems, a unit cost of around Rs 300 *per capita*, with larger schemes being less costly. This price is inclusive of drilling and casing. Surface-water schemes tend to be a little more costly than ground-water schemes. Obviously, prices will vary depending on location and nature of the community to be served.

1.2 **Designs**: Designs of schemes were generally undertaken by private sector Consultants. While some of the designs were functional, the time taken to produce those designs was generally between four months and a year. Some of the results showed inaccuracies in designing and a possible unfamiliarity with small water supply systems. The specifications relating to materials in slow sand filters have been particularly problematic. It would seem inappropriate to employ technical consultants to perform designs and then have to recheck every part of those designs, if an alternative could be developed which would allow for both designing and quick and accurate cross checking by Government itself.

In 1986, the TAG (Technology Advisory Group of the World Bank) computer software for designing of water (and sewerage) systems became available and was offered to UNICEF, Quetta, purely for information purposes. It was examined and seemed to offer a method whereby previous design and checking deficiencies could be overcome.

There was some delay between receipt and investigation of the system since not all computers are compatible with all software. Access to the software and agreement with Government concerning the development of a Modular System for design and construction was complete in late 1987.

In early 1988 serious development started and, to avoid interruptions in placing water supplies in communities, private sector Consultants were employed to perform a small number of further designs while the other work took place. This is significant in that the consultants were required to follow substantially the system which was being developed. Their designs were received in a matter of weeks. They were cross checked on the computer and found to be substantially accurate.

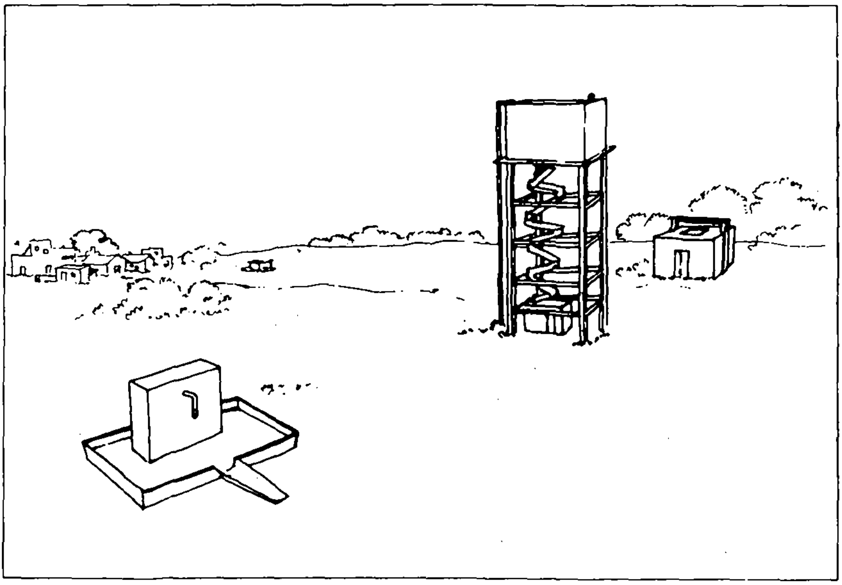
At the time that this was in hand, it was decided to train Government Assistant Engineers in how to handle the TAG software. This proved to be very quick indeed. Even for engineers unfamiliar with computers, no more than three days were required before designs could be produced which were adequate. It was noted that the more familiar the engineer was with how to design by hand, the better the design, indicating that it was still necessary to have technical competence fully to exploit the potential of the computer.

1.3 **Construction of water supply systems**: With the designs that were employed between 1982 and 1985, anything between two and four years were required for construction. This was a commentary on the capacity of the Contractors and the prevailing weather patterns which alternate between very hot (flash sets) and very cold (frost heave and spalling of newly laid concrete).

It had been decided that to ensure recipient perceived ownership of systems, that communities should participate by digging many of the pipe trenches. Coordinating community work-forces (which are only available when not involved in agricultural activities) and the Contractors, proved to be exceptionally difficult. Perhaps the most time consuming item for construction was the overhead water reservoirs, which were typically between 90 and 225 cubic metres (20,000 and 50,000 gallons), and up to 21 metres (70 feet) high.

1.4 **Overhead reservoirs**: Quetta city suffered a devastating earthquake on 31st May, 1935. Half of the city was totally destroyed. Had there been high overhead tanks in the city at the time, there is little doubt that they would nearly all have collapsed. Baluchistan lies in a highly active seismic area. Thus it seems reasonable to suppose that such structures will constitute a decided danger in communities where they are placed. Purely from a design point of view, such structures are only necessary in a 24-hour, fully pressurised system. Other systems such as direct pumping do the same job but cost considerably less, and can be constructed more quickly.

It was reasoned that the capacity of the former OHRs (overhead reservoirs) was excessive. Sizing should be based upon the balancing storage necessary only. This is the balance between the outflow during periods of supply (only intermittent supplies are provided in rural areas), and the inflow from the source/borehole during that same period. Calculations showed that very much smaller tanks were appropriate.



A TRADITIONAL PIPED WATER SUPPLY SYSTEM
WITH OVERHEAD RESERVOIR (OHR)

At the same time that this reasoning was proceeding, it was wondered whether elevated tanks were even necessary. Direct pumping as an alternative was very seriously considered but was discarded for two reasons. First, loadshedding (a frequent event in Baluchistan) would result in there being no water at all available; but, secondly and more importantly, the size of pump required was the same as for an OHR and experience had shown these pumps to be too big to handle locally when a malfunction had taken place. Thus a ground level tank sized purely for balancing was selected.

A ground level tank (GLT) does not produce the head/pressure required for the water to be driven to its destination, but by the simple expedient of the addition of a small, locally made centrifugal pump, one can achieve the same effect as the former OHR. And locally made centrifugal pumps are inexpensive and can be repaired and maintained locally.

1.5 **Illegal Connections**: Systems having a specified number of (street) taps will produce water at all points of the system if correctly designed. Most of the systems did at 'handover' on the completion of construction. However, shortly thereafter, taps which were distant from the source curiously dried up and failed to supply any water at all even when all valves were open. The reason was that certain people were unwilling to allow their women to collect water in the public street—or they simply required the convenience—and they had arranged to "tap" into the pipes which passed their homes. Attempts to disconnect them on the basis that they were denying others water, met with no success. Guns of all descriptions are freely available in this society and can be remarkably persuasive instruments.

This problem has, it appears, been known about for many years but no practical solution has been found. To restore systems to provide sufficient pressure to supply both the illegal connections and the distant taps requires raising the level of the OHR (an impractical solution); or constructing a booster station at the source, for which no budget is available with Government. However, discussions with pump manufacturers provided a remarkably simple answer. At the time of installing the centrifugal pumps on the GLTs, one has only to supply one or two additional "blind" stages to the pump. Later, when the illegal connections are taken and the pressure drops, simply adding impellers will provide some additional pressure. If more power is required beyond the immediate design of the motor, then, if the original motor was arranged for a vee-belt connection, it only need be traded in for a new and slightly larger motor which can easily be installed. Costs for this arrangement are surprisingly low. Indeed, they are so low that Government has the ability to say to communities that if they "damage" their own systems by taking connections which are not designed for, they can restore the system *at their own cost.*

Loadshedding is a problematic feature of rural water supplies. If all motors are electrically driven, when there is no power, there is no water. It was thus decided that when centrifugal pumps are placed on GLTs, not one, but *two* pumps would be placed – the second one being diesel driven as stand-by. This will at least make the balancing storage available during power outages, for drinking purposes. Diesel driven centrifugal pumps tend to be more expensive than electrically driven alternatives; but both are very inexpensive compared with the costs for OHRs.

1.6 **Community behaviour with street taps and community tanks:** Those familiar with rural water supply systems on the sub-continent will recall having seen vandalised street taps quite frequently. Water may be seen gushing out of a pipe from which the tap appears to have been torn off. If this is what is done to facilities which are provided free-of-cost, might there be some reason why people behave this way; and is there some way in which to overcome the problem? Besides, where taps have been removed, considerable wastage occurs; and as the communities are more and more paying for the operation of their own systems, this fact only leads to their wasting their own money quite apart from causing drainage problems.

It would appear that there is a "flow" from street taps which satisfies users; and anything less than that causes frustration. The less the water flows, the greater the frustration until a point is reached where they take matters into their own hands. The same mechanism appears to apply to the operation of handpumps, too.

Recently, some considerable study has been undertaken to determine what is a satisfactory flow. The general consensus appears to be that a flow of 0.225 litres/sec will satisfy people. Flows greatly in excess of that will force the bucket out of the hand of the user; and flows somewhat less than that will cause frustration.

The designer needs to be aware of the head loss which occurs through any kind of tap; and what the community reaction is to the particular type of tap that may be employed. Recently, the WEDC Group from Loughborough University in England has tested a number of different tap types to determine their Q vs H characteristics. While there are a number of reasons for having undertaken the work, one of the most important is that self-closing taps are being increasingly used, and they have some limitations. Many of this variety of tap have a high head loss associated with them. This becomes important when such taps are used with community tanks where the head available is low. This becomes apparent when the relevant drawing in the Modules is seen. Annex IV presents a graph of Q vs H for one type of self-closing tap.

It has been asked whether street taps are generally the right way in which to provide communities their water supplies? In some areas of the world, community storage tanks with taps are used. Even in parts of Sind, this technique has been used. It has some advantages over the street tap system of water supply in that it provides water points which, when the tank has water, allow water collectors the freedom to choose their time of collection to suit themselves, rather than depending on the whim of the operator as to when he is willing to switch the system on. How frequently have we not all seen queues of people waiting for the operator to switch on? Some in the community will have to walk a few yards further to collect their water from such tanks, but the additional walk should be compared to the frustration with slow system operators and the limited access provided by a single water point.

It is noted that community tanks provide one special feature which can be of considerable use to adapt a water supply to the furture living patterns in the villages. When engineers design systems, they make the assumption that the community will grow by a fixed percentage each year. They also assume that each village will grow by that same amount, each year. Obviously, this is not the case. Some will grow more; and some, less. For staged designs–such as are being considered in the Manual–some schemes may well require additional storage capacity some years in the future. Ideally, the additional storage should be where the water is to be used. The simple expedient of adding community tanks *where they are needed* will provide a useful method of increasing the total necessary storage at the appropriate time.

Designers will note that the Modules provide only one arrangement at the tap, whether it is a street tap or one from a community tank. As communities in different parts of the country use a range of different sized containers of varying heights, they may need to take this into account. If, for instance, the tap is too high for the *kalshi* being used to collect water, some spillage will occur. If the tap were lower, this would be avoided. Also, in some areas, water is collected in jerry cans. Does the tap arrangement allow for ease of collection in such a case? Only those who are active within the communities will be able to determine the answers to such questions.

1.7 **Maintenance**: In large water authorities where economies of scale can apply, it is generally accepted that around 3% of the capital worth of the undertaking is required every year in maintenance. For small, rural water supply systems in an area where large distances must be covered between systems, it is reasonable to estimate at least 5% as being the annual maintenance requirement. Taking the capital worth at Rs 800 crore (US $ 500 million) for total coverage of the province, then the *annual* maintenance requirement would be around Rs 40 crore (US $ 25 million). The allocation in 1985 amounted to around Rs 3 crore.

Conventional wisdom among donor agencies is that communities should both pay for and operate their own systems. Frequently, both loan and grant finance has this as a requirement. However, can one really expect communities to handle 40 HP motors and pumps when such equipment has to be sent overseas for repair when it malfunctions? And can one also expect a community of around 1,000 people to repair reinforced concrete structures which are over 20 metres high? At the same time, Government has scarce resources for which there is fierce competition, and maintenance—a "non-productive" activity—is generally the first sector to be cut when cuts are applied. Taken together, this means that sustainable water supply systems are ones which should be inexpensive (*not* cheap); should be quickly constructed to avoid community frustration; should employ local materials and equipment if at all possible; and should be so designed that the resulting system can be operated and maintained without the need for government intervention. Small GLTs with local centrifugal pumps appear to answer most of these requirements.

1.8 **Policy**: In 1985, no clear-cut policy for the provision of rural water supplies existed in the province. General guidelines were followed from other areas, but considerable variation existed. It was thus at the Donor Meeting on 22nd September, 1985, that the Government very clearly spelled out what the Baluchistan Provincial policy would be from that time onwards. It is shown in Section 2.1.

1.9 **Demography**: For designing water supply systems, engineers normally will adopt the national growth rate and project it 20 years hence. They then design for a community that it *is assumed will exist* at that time, staging as necessary. In the designs from 1982 to 1985, this was not the case. No attempt was made to stage designs so that expansion could take place if the community did, in fact, grow at that rate. There was an added complication in that the engineers had taken census data which gave rural growth rates greater than it is biologically possible to attain. The fastest growth rate in the world is around 4.3% to 4.4%, seen in Kenya. The Baluchistan provincial growth rate for 1972 to 1981 was shown to be nearly double that rate. Resulting designs were as much as ten times oversized.

Growth patterns in the developing world are generally similar. As industries are created, so migration takes place towards them, for that is where the jobs are. Industries are normally all urban or peri-urban based. Urban growth rates are normally very much higher than the national average. In fact, Karachi boasts one of the highest in South-East Asia; and Karachi must thus be drawing for some of its growth, from Baluchistan. If urban growth rates are higher than the national average, then it stands to reason that the rural growth rates must be lower.

It is said that the national average growth rate is 3% per annum; and that Karachi's growth rate is between 6% and 8% per annum. Thus one can expect an *average* rate in *rural* Baluchistan somewhat below 3% per annum. Naturally, there will be variations which will generally depend upon employment opportuntities; but it would seem unreasonable to design all systems for a population (20 years hence) 3% per annum greater than the present. It would seem more sensible to assume that some will grow at that rate, and some will not; and make provision so that if the population does grow, then some years hence the system should be capable of being adapted to provide the increased population with their requirement. This is known as "staged" design.

1.10 **Computers and engineering designs**: The formulae which apply to piped water supply systems are remarkably simple. However, because the systems require simultaneous solution of many simple equations, either good guesswork or a very tedious series of calculations is required to obtain convergence of the solutions. This is know as iterative solving of the equations. Formerly, one was sometimes tempted to "guesstimate" to avoid the tedium of the long series of calculations. Some of these "guesstimates" were not accurate; or the solution was very costly.

Computers themselves are relatively inexpensive when one exploits their power fully. They also provide speed. It seemed entirely reasonable to look at what could be done with computers to assist in the design of water supply systems. Computers are particularly good when applied to a series of iterative calculations, so that when the TAG software was offered and examined, the timing was fortuitous.

1.11 **Water supply and Health**: Normally, water is seen as a basic need and one which leads to an improvement in health in the community in which it is placed. An examination of many national rural water supply schemes shows that this is more promise than fact. It has been noted that unless there is sufficient understanding in the community that the water provided must be used for hygiene—and how—and *used*—then the promised health improvements remain as dreams. Children still die of diarrhoea.

This Design Manual does *not* address itself to these issues except where some aspects of latrines are concerned, but it is noted that the investments that are made in the "hardware" of water supplies are very costly and both Donors and Government are likely to look at the return on their investment. Hence it will be necessary to make provision for real hygiene improvements (and who make these except the users of the water?) if a continuing flow of funds to the sector are to be assured.

2. DESIGN AND GENERAL METHODS

2.1 **Policy**: Section 1.8 noted the Donor/Government Meeting in 1985. That meeting provided the following policies for rural water supply schemes in Baluchistan:–

Availability: Each person should have access to 10 gallons per day. These were not defined as Imperial gallons but were assumed to be. This is 45.4 litres/capita/day. It is noted that no time period was given during which this water should be able to be collected. It is further noted that the type of supply was not laid down (i.e. 24-hour or intermittent), but as even major city supplies are generally intermittent, it is assumed to be intermittent too.

Growth Rate: Piped water supply designs should be able to accommodate a 3% annual growth for up to 20 years.

Staged Designs: Designs should, where possible, be staged to avoid having idle capital.

Unit Costs: In 1985, units costs (*per capita*) should be aimed to average Rs 500 (US $ 29), but flexibility should be allowed for special cases. This was not official Government policy but purely for guidance given the very high costs seen earlier.

Yard Taps: The Government will only provide street taps or community tanks. Those wishing to take yard taps or house connections must pay the full price.

O & M: Operation and routine minor maintenance will be by the recipients. Government does not have the staff to run all community water supplies in the province, nor does it have a budget to allow for the large distances that must be covered for anything but major breakdowns. Major breakdowns will be the responsibility of the Public Health Engineering Department following handover of the schemes to the recipient communities. (This policy was stated later, after the creation of that Department.)

Cost Recovery: There was no stated policy on cost recovery in 1985 and, at the time of writing in February, 1989, this is still the case.

2.2 **Population, Location and Topography**: In designing any piped water supply system, it is necessary to know what flow is required at what point. This means that one must know how many people are to be supplied (accurate population count); where they are to be supplied (the location of that population); and the accurate topography of the area (topographical survey.) Without these data, one cannot *design*; one simply is making crude estimates. For the purposes of this Design Manual, these are mandatory. It is noted that the 29 water supply systems that have already been designed using the system described in this Manual have all had such data.

With respect to the present population (P_p), the data are collected house by house. This has the advantage that one can obtain age distribution and precise location at the same time. Since Government policy is that future population should be based on a growth rate of 3%, one can compute the future population (P_f):

$$P_f = (1.03)^{20} \times P_p$$

$$= 1.806 \times P_p$$

It may come as some surprise to readers that it will be unnecessary to have to deal with P_f at all despite it being required as part of Government Policy. This is elaborated in the following two sections.

2.3 **Water Demand**: It has been observed that water collectors place limits on the number of water collection trips that they are willing to undertake no matter how close the water is to their homes. This means that while policy requires an *availability* of 45.4 lpcd, the more usual amount *utilised* may be around 13 lpcd. With time, this amount used may rise somewhat. As noted earlier, some doubt attaches itself to what precise growth rate will actually apply to any community; making the choice of design flow complex. The design flow must—above all—match the behaviour patterns of the communities.

Taking all these factors together, it has been decided that the "staging" of the system to accommodate growth will *not* involve the addition of physical facilities except in certain structures like surface water storage basins (see later chapters). Where pipes are concerned, "staging" will be effected by operating the system for longer hours. Where communities do grow at the rate of 3% per annum, and where such a community collects the full "policy" 45.4 lpcd, such communities will need to operate their systems for 6 hours per day. If the community uses less than the "policy" demand, the system need operate for fewer hours. Where the growth rate is less than 3%, the necessary hours of operation are further shortened. This implies that if there is no growth and people only collect 13 lpcd, that the system need only operate for less than an hour per day under ideal conditions. Leakage will, naturally, affect the hours of operation. See Section 2.5 in this respect.

2.4 **Determination of the Required flow**: To be able to design a water supply system, one must have certain basic data. For the determination of the flow required at the tap, one must have:

a. Exact numbers of people to be served and their location;
b. The period over which they will gain their requirement, daily;
c. Whether intermittent flow is used, and if so, how;
d. Possible growth rates; and
e. Whether staging is possible, and if so, how.

All parts of the water supply system will depend on these calculations like tank sizes, hours of pumping, pipe sizes and the like; so it is important that the data are accurate and reliable.

No time limit has been placed for production of the 45.4 lpcd (10 Igpcd). However, observations in communities indicate that water is usually made available for 1 to 2 hours in the morning; and 1 to 2 hours in the afternoon/ evening. It is thus assumed that the 45.4 lpcd must be made available in around 3 to 4 hours per day. For design purposes, however, because pipelines cannot cheaply and easily be dug up and replaced, it will be assumed that a future population must be able to be provided their full 45.4 lpcd each in a six hour period. The flow (1/sec or Igpm) in the pipelines will be the same for both present and future populations. By implication, present communities should be able to collect their full quota in less than 3½ hours per day. This is a repeat of the previous section but is deemed to be fundamental to the understanding of the basis of this system, that it is intentionally repeated.

Flow will be determined by the formula:

$$Q = \frac{(1.03)^{20} \times P_p \times 45.4}{6 \times 60 \times 60} \text{ litres/sec}$$

where P_p = Present population.

On this basis, the peaking factor will be 1.0. See Section 2.5.

Since both the Standard Specifications used in Pakistan, and the TAG computer software are based in the SI system (metre-kg-sec system), the flows will be reflected in litres/sec. Engineers wishing to obtain flows in imperial units (ft-lb-sec system) may refer to a number of handbooks which give the necessary conversions.

Following is a table which gives the flows for different sized groups. Where groups differ from these, flows may be deduced by simple arithmetic.

TABLE 2.1

Present Population	Flow in litres/sec.
100	0.38
200	0.76
300	1.14
400	1.52
500	1.90
1000	3.80
2000	7.60
3000	11.40
4000	15.20

2.5 **Wastage, Leakage and Peaking Factors**: It will have been noted that no factors have been allowed for these. There are several reasons why this is so.

Every drop of water wasted by messy collection, spillage during transport or otherwise, costs money. This is because every drop of water whether used or not, has to be pumped at some stage. The power consumed has to be paid for by someone. Designing for wastage only encourages wastage. Besides, both the Government and the Donors seek cost effective use of their scarce resources, and both are becoming less willing to finance both unnecessarily wasteful and non-productive activities.

The mode of operation of systems appears directly affected by the behaviour patterns of communities. If community members have to wait long hours for buckets to fill because there are many leaks in the pipeline and resulting residual pressures at the taps are low, then they are likely to do something about it. If those same community members have to pay for all the water that leaked and never even reached the taps, they are likely to do something about it on the assumption that they pay the real price for the energy to pump the water. Again, as for wastage, leakage is not something which either Government or Donors are willing to subsidise. Thus, it should not be encouraged by being incorporated in the design.

It should be remembered that the systems are being designed to cater for Government policy where design yields are as much as six times greater than is actually utilised by present populations. Thus, rather than provide a "laziness" factor by allowing for wastage and leakage, no allowance is made beyond an understanding that communities will operate the systems for as long as they are willing to pay, or are prepared to carry the filled containers.

With respect to the Peaking Factor, this is normally only associated with urban, 24-hour fully pressurised systems. Patently, the systems addressed by this Manual are not urban; they are not 24-hour; and they are only pressurised during limited daily operation. Typically, when the systems are turned on, *all taps are open*. What this means is that the full design flow must be available at *all taps* during operation. This in turn means that the full flow at a Peaking Factor of 1.0 is required at design.

2.6 **Balancing of the Systems**: Experienced designers will know that the assumptions that are made during the design will provide a system which does not behave exactly as designed. Pipe friction coefficients are inaccurate. "Satisfactory" residual heads vary quite widely. Computer generated solutions are not always entirely practical, and minor modifications are often used to ease the problems of construction. These all contribute to a final system where flows may be rather higher than ideal; and it remains for the system to be "governed" to provide flows which are satisfactory.

Many of the standard texts will note that excess head may be destroyed using special valves. While this is indeed correct, the behaviour of the communities in which the systems are placed may not have been fully taken into account. Typically, one sees people who wish to have a higher flow at their taps, breaking into and fiddling with such valves. This unbalances the *whole system*, resulting in other taps receiving *less* than the ideal flow, with consequent frustration. Also, such flow regulating valves are relatively expensive since they have to be placed next to *each* tapstand. Thus, a solution is required which will be relatively easy to install with less than fully skilled labour; which does not lend itself to tampering by the community; and which is inexpensive. Orifice plates appear to be the ideal solution.

An orifice is simply a hole drilled in a disc, which is placed in the pipe conveying the water to the tap. It may be placed very close to the tap, which means that any competent plumber can walk around the system installing such orifices in the nipples beside the taps. All he needs in his pocket is a number of discs which are drilled at slightly different diameters which will give slightly different head losses, depending on how much head requires to be destroyed. Annex IV gives the graphical solution to a wide range of head loss requirements. These can be translated by the designer on to the final drawings depending on the requirements of the design which may require more (or less) head to be destroyed at individual taps. Annex IV gives the derivation of those head losses.

2.7 **Borehole Pump Tests**: The success (or failure) of all designs for groundwater based water supply systems – the majority in the Province – is primarily based on the yield from the borehole, and its reliability. Yet the attention that is paid to testing of boreholes appears to be less than necessary. So important is this aspect of rural water supply system design deemed to be that a separate section is devoted entirely to the subject, Section 5.

2.8 **Borehole or Source Yield**: From Sections 5 and 6.2, the reliable long-term Yield is determined. This flow, Q_b, is used in the determination of the size of the Ground Level Tank (GLT), dealt with in detail in Section 2.9. It is reiterated that if this flow Q_b is not determined reliably, the system cannot perform satisfactorily, which leads to community frustration and a loss of credibility by the Government Engineers.

For boreholes, standard texts say very little about how to judge whether a long-term yield is adequate or not for any particular community. For Baluchistan in the areas under consideration, it is accepted that if the borehole long-term yield will not provide the required total volume of "policy" yield for the population 20 years in the future *in 6 hours or less*, then the borehole is deemed not to be adequate. This may seem to be a stringent requirement. However, given the lack of regulation of groundwater extractions in the province with consequent falling of the water table (e.g. Pishin), this is a necessary precaution. The balance between extraction and recharge is fragile in this arid area.

2.9 **Ground Level Tanks/Clear Wells**: It has been noted earlier that the basis for the design for GLTs and clear wells (which serve the same function as GLTs, but which are associated with surface water schemes) is to provide balancing storage between the borehole/source and the outflow pump.

As will be seen from an examination of the "Modules" (standard drawings), both ground level tanks and clear wells come in a range of sizes. They are as follows:

Also, it should be noted that the GLT is sized for the present population only, and that staging will take place later if it is found that the population grows sufficiently to warrant it. Later in this section, more will be said about methods of staging.

If the *future* population must be able to obtain their 45.4 litres in a six hour period (twice daily at 3 hours each time), then the present population must be able to receive—at the same flow rate—their 45.4 litres each in 3.32 hours (or 1.66 hours twice daily). Some typical design examples follow:

TANK SIZE = A − B Where A (Outflow) = Pres. Popn. x 22.7 litres
B (Inflow) = Borehole Yield in 1.66 hrs.

Example 1: Present population = 2,750
Borehole/pump Yield = 5.67 l/s

A = 2,750 x 22.7 = 62,425 litres
B = 5.67 x 3600 x 1.66 = 33,884 litres

Tank size = A−B = 28,541 litres

Use 33 cu m tank

———— ** ————

Example 2: Present population = 4,000
Borehole/pump Yield = 6.81 l/s

A = 4,000 x 22.7 = 90,800 litres
B = 6.81 x 3600 x 1.66 = 40,697 litres

Tank size = A−B = 50,103 litres

Use 45 cu m tank
(Use 1.9 hrs, 2 x daily)

———— ** ————

Example 3: Present population = 1,117
Borehole/pump Yield = 4.54 l/s

A = 1,117 x 22.7 = 25,356 litres
B = 4.54 x 3600 x 1.66 = 27,131 litres

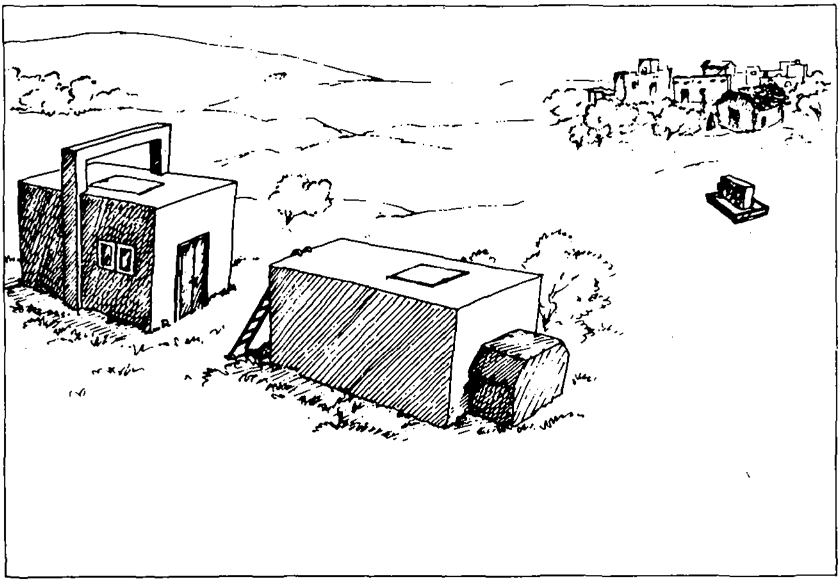
Tank size = A−B = −1,775 litres

Use 11 cu m tank
and Q_b = 4.24 l/s

———— ** ————

Ground Level Tanks:	11, 23, 34, 45, 68 and 91 cubic metres.
Clear Wells:	11, 23, 34, 45, 68 and 91 cubic metres.

For the GLTs (used with boreholes), long-term yields on boreholes in the province typically range from 2 to 10 l/s (30 to 150 Igpm), and communities generally range from 400 up to 6,000. Such communities will require flows (See Section 2.4) of from 1.5 to over 20 l/s. While other provinces may have larger communities to serve than these, the same general principles of design will apply to them. For the sizing of the GLT, the difference between outflow and inflow will determine the balancing storage required. Since this is rarely exactly the same as one of the standard GLTs in the Modules, the next size larger is generally selected.



THE MODIFIED PIPED WATER SUPPLY SYSTEM
WITH GROUND-LEVEL TANK (GLT)

Before considering design examples, it should be noted that the size of the GLT will depend on mode of operation. What this means is that it will be operated *twice* per day, and that only *half* the "policy" yield will be required during any particular operation period such as morning or afternoon supply. Thus, a community of 1,000 people will only require 1,000 x 22.7 = 22,700 litres availability in the morning; and another 22,700 litres in the afternoon.

It is not necessary to view the GLT (or clear well) as the only location at which the balancing volume may be stored. It can, just as easily, be stored in community tanks where the water will be collected. This factor will be employed for staging. As an increasing volume may be needed, it can be created by adding community tanks. This gives considerable flexibility to the system. Also, there are a number of sociological reasons why community tanks are favoured, and these are elaborated in Section 2.12, later.

For surface water systems, it is well known that the "source" (the slow sand filter) provides a small flow for a long period as compared to the flow from a borehole which normally provides a high flow for a short period. Thus, if exactly the same design approach is used for clear wells as for GLTs, the resulting clear wells would be very large. Mass-flow diagrams confirm this fact. Thus, for surface water schemes, it has been decided that to limit the size of the clear wells, some 50% of the necessary future balancing storage will be assumed to be sited in the communities in community tanks. The operation of such systems will, of necessity, be somewhat different from ground-water schemes. This will be covered in the respective operation manuals to be developed later.

While it may be more applicable to an operation manual it is, nevertheless, probably useful to note that after the morning or afternoon supply has been made, the borehole pump will be operated until the GLT is completely full. This is necessary to avoid the case where load-shedding occurs just before the supply is to operate which will deny the community any water at all. If the GLT is always left full, then if load-shedding takes place at an inconvenient moment, there will at least be enough water for drinking purposes available in the GLT. This can be pumped even when no electricity is available as noted in Section 2.10, later.

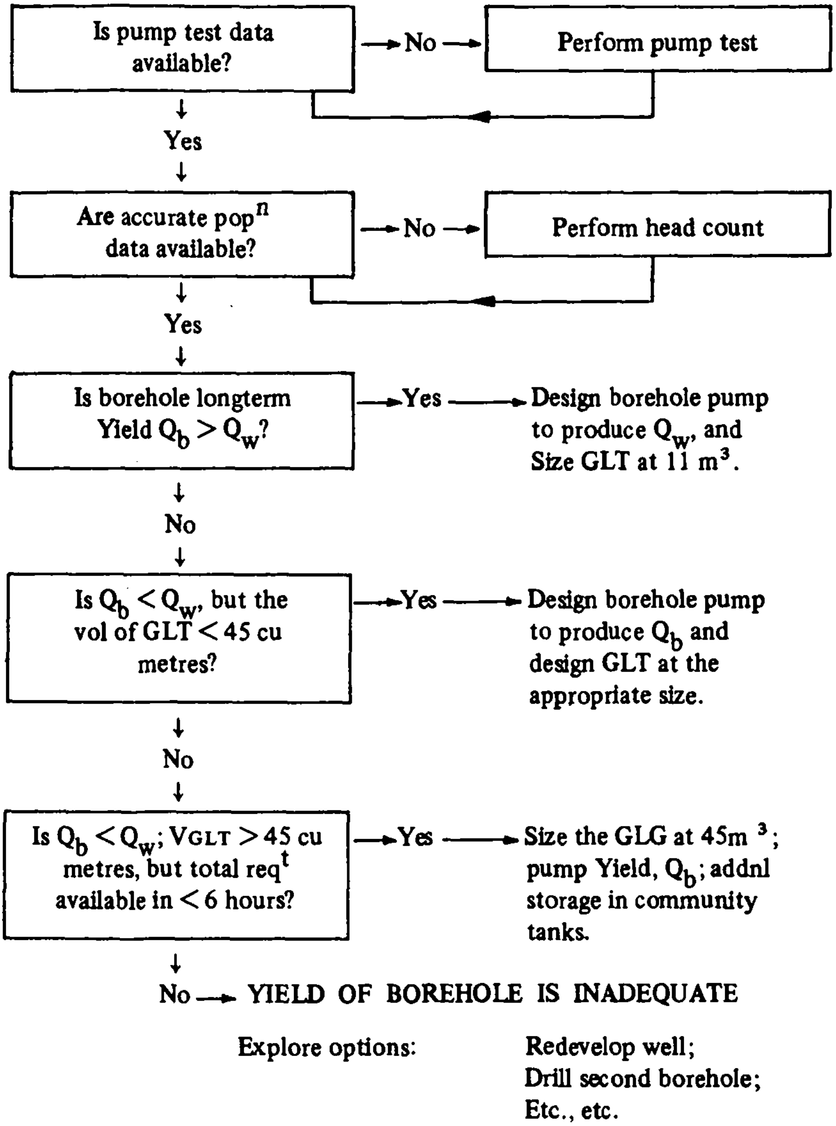
Engineers frequently favour flow diagrams when faced with a long series of decisions since such diagrams give both the sequence to be followed and the action to be taken when faced with a choice. As the GLT sizing, the borehole yield and community size are all inter-related, such a flow diagram may be a useful tool in clarifying the sequence and choices. It follows.

GROUND LEVEL TANK/PUMPING PLANT DESIGN FLOW DIAGRAM

Q_b = Borehole long-term yield, generally in l/s (See Sect 6.2)

Q_w = Flow required for community (See Section 2.4)

$$= \frac{P_p \times 1.03^{20} \times 45.4}{6 \times 60 \times 60} \text{ litres/sec where } P_p = \text{Present population}$$



Is pump test data available? → No → Perform pump test

↓ Yes

Are accurate pop^n data available? → No → Perform head count

↓ Yes

Is borehole longterm Yield $Q_b > Q_w$? → Yes → Design borehole pump to produce Q_w, and Size GLT at 11 m^3.

↓ No

Is $Q_b < Q_w$, but the vol of GLT < 45 cu metres? → Yes → Design borehole pump to produce Q_b and design GLT at the appropriate size.

↓ No

Is $Q_b < Q_w$; $V_{GLT} > 45$ cu metres, but total req^t available in < 6 hours? → Yes → Size the GLG at 45m^3; pump Yield, Q_b; addnl storage in community tanks.

↓

No → YIELD OF BOREHOLE IS INADEQUATE

Explore options:

Redevelop well;
Drill second borehole;
Etc., etc.

2.10 **Centrifugal Pumps on GLTs and Clear Wells**: As noted earlier, the GLTs (and clear wells for surface water schemes) will receive not one, but two centrifugal pumps each to provide the requisite flow and head. Where electrification has not reached the area yet, both of the pumps will be diesel driven. Where it has, one will be electrically driven and the other, diesel. The general layout for these pumps will be found in the Modules.

It is assumed that the designer has a solid understanding of pumps and motors since the Universities feeding professionals to the relevant Departments cover the subjects in their curricula. For those less familiar with the subject, reference is made to the bibliography where a range of standard texts on the subject may be found.

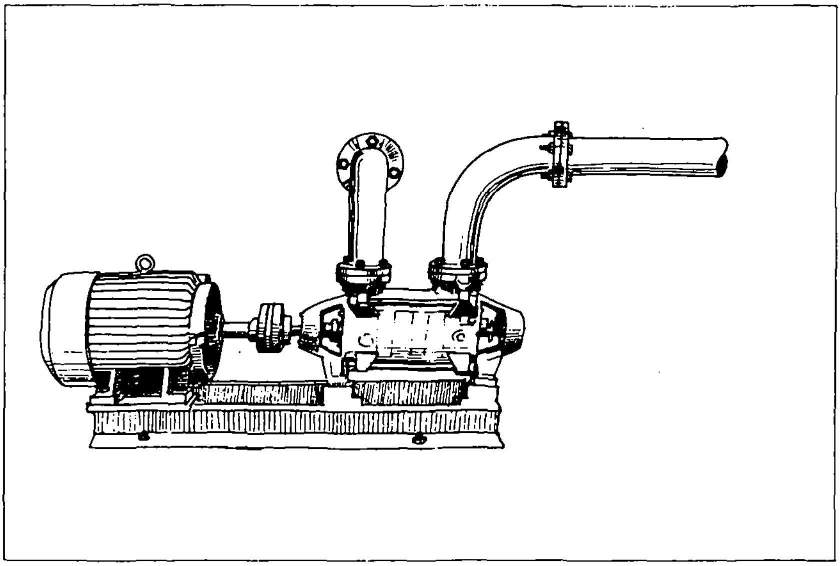
As Baluchistan—and indeed other parts of Pakistan—present a range of temperature and elevation conditions which affect the power output of diesel engines, it has been deemed necessary to provide a table showing the effects of ambient temperature and elevation on the performance of diesel engines. It should be noted that ambient temperature does *not* refer to the maximum but rather to the average daily temperature. This is important because the water supply systems will rarely be operated when the maximum temperature is encountered. The Table in Annex III presents diesel engine performance data. The designer will need to consult synoptic station data for the nearest equivalent station to determine what ambient temperature should be used; and accurate mapping for the elevation.

Annex II presents two sets of pump performance curves together with a detailed worked example of a pump design. The curves are referred to as Type A and Type B. These are used solely to demonstrate the methodology that should be followed when selecting a centrifugal pump for the duties envisaged. In practice, since neither Government nor UNICEF is permitted to nominate from whom pumps will be bought (bids must be called), the process becomes iterative. This means that bids are called and the manufacturers/suppliers perform the design in the first instance. The role of the Government engineer is thus to check that the pump offered meets all the design criteria.

Type A is 2½" x 2" (suction x delivery); while Type B is 3" x 2½". These two appear to cover the range of flows and heads encountered in Baluchistan. For small communities which are < 1,000 in population, Type A may be rather large, resulting in operation at below optimum efficiency. However, as the power requirements are modest—being in the range ¾ to 1½ horsepower—this is deemed not to be a problem.

Characteristic data are only available for two speeds of operation, namely 1450 and 2900 rpm. These are the speeds normally associated with three-phase electric motors.

For higher heads (in excess of around 10 metres), one can either use the higher speed (2900) with a single impeller; or the lower speed (1450) with multiple impellers. Selection should be based on obtaining the highest efficiency as indicated by the characteristic curves. Two "blind" stages must be supplied with each pump to allow for raised pressures which may be called for if illegal connections are taken from the distribution pipework. "Blind" stages are those which are complete in all respects (volute casing and shaft); but are without the associated impellers. Installation of the impeller(s) raises the pressure while maintaining the former flow, in theory. In practice, the flow will have increased, thereby reducing the total head obtainable. Nevertheless, head will be recovered as compared to the original system; and, while the modifications will rarely, if ever, give precisely the hydraulic design necessary, they will provide water to many or all of those parts of the system which have "dried-up".



A MULTI-STAGE, ELECTRICALLY DRIVEN
CENTRIFUGAL PUMP

Diesel engines commonly run at around 2200 rpm; but pump characteristic curves are not available for this speed. The curves should be obtained from the manufacturers of pumps for this speed, before making any selection.

This implies that it will be mandatory for pump manufacturers or suppliers to provide pump performance curves with their bids – a practice not frequently followed in Pakistan at present. For the diesel motors, there are three options: *First* is to design for 2200 (or the rated speed of the diesel engine) providing pump curves are available – and use a direct, flexible coupling drive between pump and motor. *Secondly*, one can design for either 1450 or 2900 rpm as appropriate, and drive via a gearbox to achieve the correct speed of operation of the pump. Both these options would provide a neat and compact arrangement. The *third* alternative is to have the pump offset from the motor, and driven by vee-belts on pulleys sized to provide the correct pump speed, which would be appropriate for any of the speeds noted above. This has the disadvantage of requiring rather more space than the other options.

In fact, it has been decided to adopt the third option irrespective of whether the driver is electrical or diesel powered. There is a very good reason for this. In the future, when adding impellers to achieve a higher system head, the energy requirements will, of course, be increased. This implies the need to place a larger motor when the impellers are added. If motors (diesel or electric) are directly coupled to the pump or gearbox, alignment of the replacement becomes a very difficult task, and is rarely undertaken satisfactorily. On the other hand, if the drive is by vee-belts and pulleys, alignment is very easy to achieve since both the pulleys can be shifted along the axes of their shafts, and the frame can be slotted to allow movement to, or away from the pump.

Returning to factors to be taken into consideration with respect to the centrifugal pumps; it should be noted that the NPSH (Net Positive Suction Head) *must always be checked*. Referring to the curves Type A and Type B, it will be seen that the NPSH is rather low, being in the range 1½ to 2 metres. The problem appears to be that minor losses in the pipework between the ground level reservoir and the pumps become substantial if small diameter pipes are used. For the Modules, it will be noted that the pipe size adopted is standardised at 4″ϕ, which is really only necessary for flows in excess of 16 l/s.

In Annex II, Table 1 presents what is considered to be the best selection of pump arrangement related to the population size. This should be used for guidance only, and the particular design should be checked in all respects. Annex II, Table 2, indicates how impeller sizes can be selected to provide very nearly the precise flow and head arrangement required.

For the diesel driven stand-by unit, it is suggested that exactly the same pump is used as for the electrically driven unit to allow for inter-changeability; which means that the pulley system must be designed to provide the same speed of operation of the pump.

Surge protection is deemed not to be required until the head rises above 40 metres; a most unlikely event in rural water supply systems in the Province. In fact, as no system in Baluchistan is likely ever to operate where valves slam closed at times to provide a simultaneous surge, no surge calculation is presented. For the reader who is interested, reference is made to Parmakian's "Waterhammer Analysis" which deals elegantly with the subject, indicating that the problem is generally only encountered in non-elastic, long piperuns, where water velocities are relatively high.

Operation of the centrifugal (distribution) pumps will be carried out by manual push-button. A low level regulator will be placed in the ground level tank (or clear well in surface water schemes) to ensure shutoff to avoid running the pump dry. The regulator will be an electrical relay for electrically driven pumps; and a mechanical device for diesel driven pumps. This regulator will over-ride the manual push button, so that operators will be unable to start the centrifugal pump(s) if there is inadequate water in the supply tank.

How to determine the flow requirement of the pumps has already been described in earlier Sections in the Manual, but the methodology to fix the design head has not. Naturally, it follows from the design of the pipelines which will dictate what head loss will occur, and thus what head is required to be imparted to the water to overcome that loss. This is described in the following Section.

2.11 **Pipelines**: Designers will know that there are many different ways in which to compute hydraulic friction loss in pipelines. Amongst others, the formulae include those developed by Manning, Darcy-Weisbach, Hazen-Williams and Colebrook-White. Some are easy to use and some, not. Some are very accurate, and some not. Some apply to a narrow range of Reynold's Numbers, and some not. The standard texts provide all the necessary details. However, it should be noted that *all* formulae depend for their accuracy on the accuracy of choice of one or more constants; and experience has shown that this is, perhaps, their greatest weakness.

The constant(s) which require to be fixed also vary with time. This is because deposits build up inside the pipelines changing the hydraulic friction characteristics for any particular flow. The designers will know this. Thus, in the selection of which formula to use, it is perhaps necessary to select the formula(e) for which performance coefficients are best known. For this reason the Hazen-William's Formula has been selected. By happy coincidence, the Technology Advisory Group selected this formula for computing hydraulic losses, probably for the same reason. See Annex V.

Having selected the formula, it then becomes necessary to know what pipe material is to be used so that the coefficients which must be applied may be determined. For rural water supplies in Baluchistan and in most other areas where pipes are used, upvc is normally the material of choice. Only this material is considered, and if the designer wishes to employ another material, he is referred to the standard texts for the choice of coefficients to be employed. This may include GI for small spring development schemes in Azad Jammu and Kashmir; or ductile iron in hilly, rocky areas of NWFP where burial of the pipe is not feasible.

Many water sources in Baluchistan display a degree of hardness. This implies that deposits are likely to build up in the pipes, depending on the pH. As designs are to be for systems which must have a *minimum* life of 20 years, then it is reasonable to be somewhat conservative in the selection of the Hazen-Williams coefficients (HWCs) for different diameters of pipe. Following are the coefficients for new pipe shown in "Twort, Hoather and Law" which is a standard text on water supplies. Also in the table are those coefficients which should be used in the design programme "BRANCH" (See Section 2.13 and Annex V).

TABLE 2.2

upvc pipe ϕ inches (mm)	Hazen-Williams Coefficient	
	Probable	Use in '"BRANCH"
6" (155)	149	141
4" (105)	148	140
3" (82)	147	139
2" (55)	145	137
1½" (43)	143	135
1" (29)	140	132

When calculating the optimum pipe diameters, it is necessary to specify acceptable head losses per unit length of pipe. If the minimum head loss is too low, resulting pipes tend to be large in diameter which obviously does not give a cost effective solution. If, on the other hand, this factor is too high, then while the pipes tend to be smaller in diameter, the resulting head loss can make for a greater required head to be produced by the centrifugal pump(s). Experience has shown that the maximum and minimum head losses should be as follows:

Maximum Head Loss: 10 m/km for normal conditions;
25 m/km where high heads must be destroyed.

Minimum Head Loss: 0.2 m/km for all conditions.

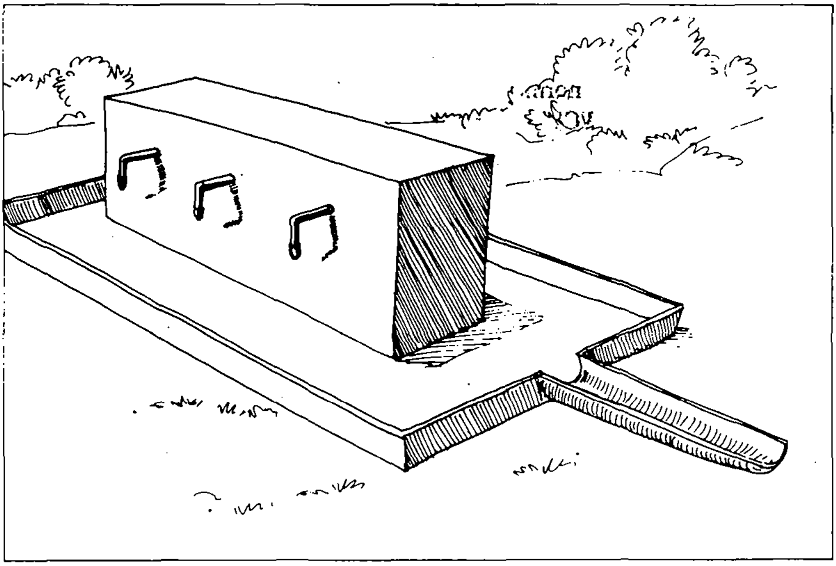
It will have been seen that no factor is allowed for to accommodate the minor losses which are normally encountered in pipelines. It will also have been seen that the HWCs that have been selected are rather lower than normal, and these conservative values are deemed to include for minor losses.

Residual heads at external nodes (offtakes) are dealt with in the following Section.

To ensure the feasibility of maintaining pipelines in any given community, it is necessary to compute the length that each person, as it were, will be responsible to maintain. It should be remembered that every community is made up of men, women and children; and that only a very few of the community members will be involved in the upkeep of all elements of the system. If, for instance, a community of only 690 people has a system which has 44 km of pipeline to maintain, each person will be responsible for 44,000/690 = 63.8 metres of pipe. In reality, it would mean that those looking after the pipeline – say some 20 people – would have to maintain 44,000/20 = 2,200 metres each. This would not be feasible. The designer will no doubt know that every metre of pipeline in these communities is buried. Experience has shown that a reasonable average per person in the community is around 3 to 3½ metres. An inspection of the pipe requirements for schemes in Annex VIII reflects figures of this order of magnitude.

Where communities are very scattered in nature, and where to connect all villages to the water supply scheme will involve great lengths of pipeline, alternatives should be sought. In certain areas, dug wells are already used; and many of these can be adapted to have handpumps installed. The designer will know that while piped supplies are typically available for two to three hours per day, handpumps are available for 24 hours. If community handpumps are used such as the Afridev-type, repairs normally only take 30 to 60 minutes and require only two tools. Piped systems normally take longer than this to repair, and require many more tools. Perhaps best of all is that handpumps can be maintained by women, releasing the men for other, more productive activities.

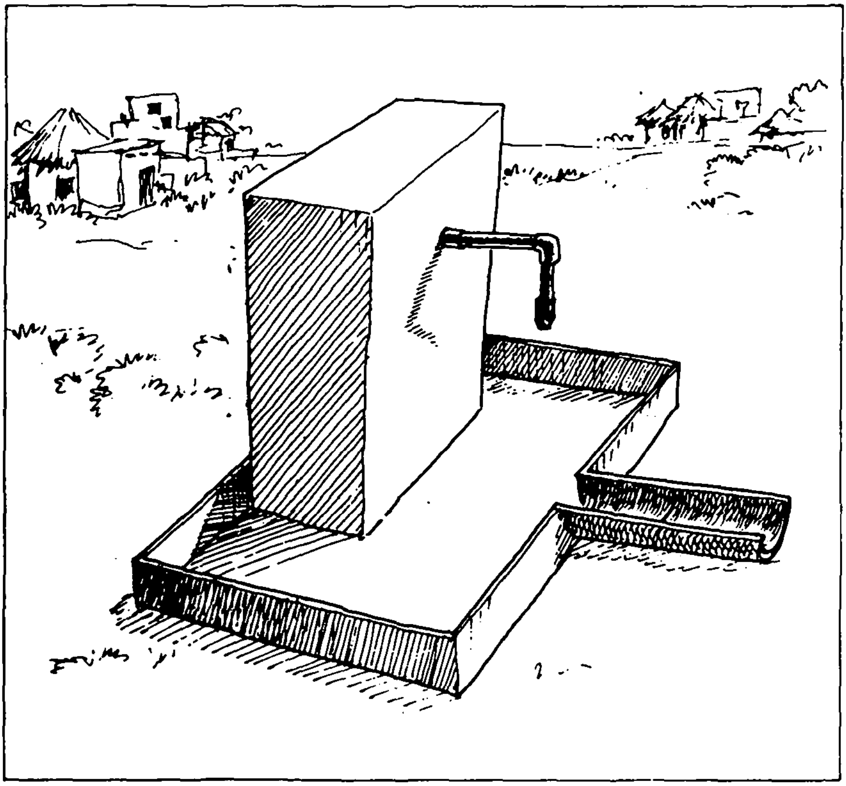
Section 2.10 mentions, *inter alia*, the problem of surge, sometimes called waterhammer. Designers should try, wherever possible, to avoid pumping into long pipelines which are at a higher elevation. Such pipelines can prove problematic when there is a power interruption and the column of water tries to run back against a non-return valve. Alternatives should be sought.



A COMMUNITY TANK WITH MULTIPLE TAPS

2.12 **Street Taps and Community Tanks**: These items may be found along or at the end of pipe-runs. In computer terminology, they are referred to as external nodes for this is where water is allowed to flow out of the system.

For water to flow through a tap, there must be a certain amount of pressure to cater for the friction loss within the tap itself. Also, for there to be a flow, there must be head available over and above that which is lost in friction. The sum of these two heads is usually called the "Residual Head". Generally, for ½"ϕ or ¾"ϕ taps, a residual head in the range 4 to 10 metres is required. Annex IV, "Flow at the Street Tap" deals with this in detail.



A SINGLE-POINT STREET TAP-STAND

A normal ¾"ϕ tap with a residual head of 10 metres is likely to force the bucket out of the hand of the collector unless he/she opens the tap very carefully. Thus, a residual head of 10 metres should only be contemplated where a community tank is planned. For standard ¾"ϕ taps, a residual of 6 to 8 metres is more appropriate.

Satisfactory yields from street taps are generally thought to be around 0.225 litres/second. Referring to Table 2.1 in Section 2.4, this implies that one tap should serve 60 people. With families being around 8 to 10 per household, this means one tap per 6 to 8 families. Putting in more taps than this will result in lower residual heads and lower yields—causing frustration in the community.

Should any design provide one external node with an excessively high residual head, this energy can be dissipated by placing an orifice in the pipe behind the tap or in the adjacent main pipe. The orifice will have to be small to be effective, usually in the range 4 to 10 mm. Annex IV presents a nomograph to determine the appropriate size of orifice.

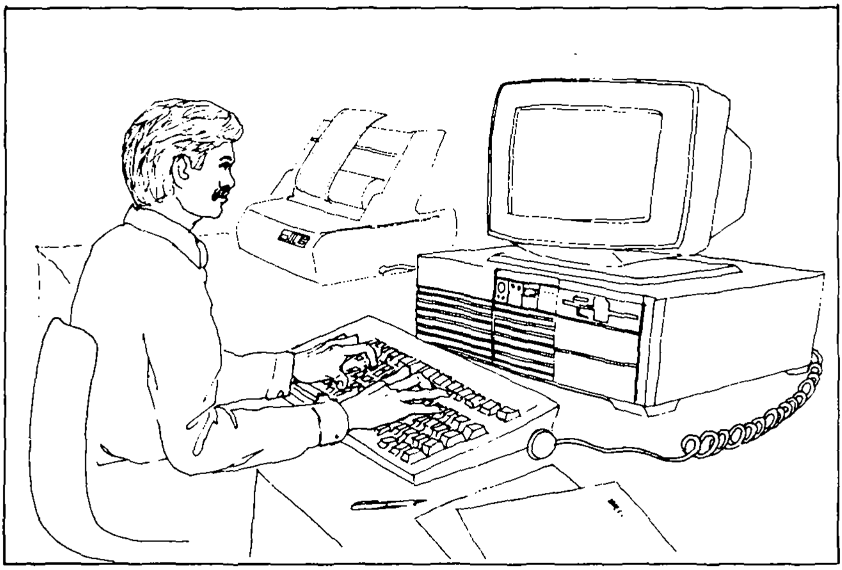
2.13 **Computer Design of Pipelines**: The TAG software for the design of piped water supply and sewerage systems includes one programme entitled "BRANCH". This, naturally, will compute pipe sizes for branched systems. It is not the function of this Manual to reproduce the instructions that come with the TAG software, because each of the programmes has its own set of instructions which are clear. Thus the reader is directed to those instructions.

All the data required by that programme must be entered by the designer. This will take an experienced user around one to two hours per "cluster" (group of villages numbering between 400 and 6,000 persons). It must be stressed that the data must be *accurate* because if they are not, the solutions that are generated will be meaningless. In computer jargon there is a term "GIGO" which refers to "Garbage In; Garbage Out". It is indeed tempting to believe that because the computer is sophisticated; because it doesn't usually make mistakes; and because the software originates from the World Bank, that anything that is placed on the computer will give accurate results. This is NOT so. Checks must be made at all stages.

When the data have been placed on the computer, they must be saved. In some offices, it may be wise to save during data input since an interruption in the power supply will usually mean the loss of all data entered since the last save operation. Lap-Top computers are quite useful in this respect since many of them work on batteries and they do not switch off as soon as the power is lost.

Following the save operation, the network is "run" or simulated. This particular operation takes between 10 and 120 seconds per simulation depending on the complexity of the pipe network. The results are displayed or printed as required. Conditions that have not been met (like residual pressures) are highlighted in the output, making it easy to identify where the problems are. At this time, the designer's skills are brought to bear.

If the residual pressures are low, it may be possible to raise the source level (referred to as "Reference Node Grade Line" in the programme). It may also be possible to enlarge certain pipes which may be destroying head unnecessarily—but this could be an expensive way of solving the problem. When the changes have been made, the network is again simulated until all the necessary conditions are met.



AN ENGINEER DESIGNING AT THE COMPUTER

The designer should always aim to keep the residual pressures at the lower end of the scale to avoid having to destroy head needlessly. All head that has to be destroyed has to have been added as energy beforehand.

When a final, satisfactory solution has been reached, the designer has it printed out for record purposes. On the printout will appear the "Reference Node Grade Line". This is an elevation. It will normally be above the actual ground level at that point which is the source (GLT next to the borehole). The difference between these two levels is the head which must be produced by the centrifugal pump(s). Obviously, the total flow for the system is that flow which each of the two centrifugal pumps must be able to produce by itself; this is because only one of these pumps is working at a time.

The data on pipe sizes and head to be destroyed are now ready to be transferred on to the Master Plan of the pipe network. This Master Plan is one of the only drawings that cannot be included in the Modules, for obvious reasons. Each agency which uses this system will develop its own way of handling the production of the set of drawings for tender/construction; but it is suggested that up to three versions of the Master Plan be produced. On one will appear the details of pipe lengths, diameters, valve chamber locations and the like. On a second will appear the details of the locations (and orientation) of the structures covered by the Modules – like pump-house + GLT or storage tanks + slow sand filters/clear well; or street taps/community tanks. On the third might be the suggested sizes of orifices which are required for destroying head to allow the system to be balanced, which will allow the particular person responsible for this some idea of the number and range of orifices which will be required.

A typical printout from the computer programme is shown on Annex VII. Following this, the Modules are applied, the methodology for which is described in the following Section.

2.14 **Application of the "Modules** : So far, various aspects of the design of rural water supply schemes have been dealt with, and some mention has been made of the Modules. The Modules refer to the set of drawings which accompany this Design Manual, Specifications, Bills of Quantities, Tender Documents and Computer Software. Those drawings include a range of different sizes of GLTs for instance, and it is necessary to use only the drawings which apply to the particular GLT which has been selected following the procedures described in this Manual.

When the full design has been completed, Master Plans must be prepared. These are noted in Section 2.13. Thereafter, the appropriate drawings for all parts of the scheme must be taken from the set of original negatives (mylars). These must be duplicated as mylars, and the originals returned to the set of Modules. Naturally, the associated Bills of Quantities that apply to the chosen Modules, must be selected.

At this stage, one has a set of drawings which will apply to the particular water supply system, but which is missing the name of the scheme. The name is added to all of these drawings, and the location of each item indicated on the Master Plan. Also, it is probably wise for the scheme drawings to be numbered sequentially so that the contractors involved will not be confused by there being a number of drawings "missing". The "missing" drawings are, of course, those that do not apply to the particular scheme. It might also be wise to produce a list of all drawings which apply to the site so that when updating of any drawing takes place, it can be noted on this list, and all parties will know which drawings are current.

It should be noted that the original drawings which are "mylars", will not last for ever particularly if they are not treated with the utmost care. It is thus recommended that agencies which use the system have *two* sets of originals. The primary set should be kept in good conditions of temperature and humidity, and no dust should be allowed to contaminate them. It would be this primary set which would be used to produce the set of working originals; known as the secondary set. When the secondary set becomes worn, cracked or creased, then a new secondary set can be produced from the primary set. It will be the secondary set which will be used to produce the drawings for each scheme.

Each agency using this system will have its own working methods. When the Modules have been used to produce the full set of drawings for the scheme, the agency will almost be ready to go to tender assuming that materials will be ready at the time that construction is to take place. For going to tender, the agency will require:

Design and drawings;
Specifications;
Bills of Quantities; and
Tender Documents (Conditions of Contract).

The three documents (Spec., BoQs. and Tend Docts) are all available on "floppy disks". The type of disks being employed currently are 5¼" and the documents are, at present, in Wang IWP and Lotus 1-2-3 Version 2 spreadsheet formats. The documents can be photocopied or they can be printed directly from the computer; but if the latter, they will need to be in the same "language" and programme compatible with the above. 3½" disks can be arranged if required; particularly if the whole system is to be given to the agency as part of an assistance package. If they are not, but an agreement is made to supply the agency with the documents only, it will be necessary for the agency either to have compatible software, or for that agency to undertake translation of the originals into their own WP and spreadsheet formats, themselves. It is noted that considerable numbers of printer codes are used in the document, and that printer codes vary widely between manufacturers. Also it should be pointed out that character translate tables are often unique to a particular computer operator.

3. OPERATION, MAINTENANCE AND REPAIR

Since this is a Design Manual, this section seeks only to alert the designer to certain needs or aspects of operation, maintenance and repair of systems. Operation Manuals need to be developed as separate documents which, for Baluchistan at least, may be very difficult indeed since the literacy rates amongst system operators are very low.

3.1 **Operation**: Operation – for the purposes of this Manual, and following stated Government policy – is the responsibility of the community benefitting from the water supply system. Thus, when the designer is faced with choices with respect to equipment which will be operated by the community, he must be aware of the capability of the operator from that community to handle it. For instance, it may not be realistic to introduce sophisticated electrical controls since when they malfunction, they are unlikely to be able to be repaired within the community or even the nearest town. Such controls as are introduced should be locally available (in general) so that local repair is possible.

Where instructions need to be given to operators concerning how to handle the system, thought should be given as to whether written instructions can even be read; and if they can, are they in the right language even? If pictures are to be used to convey any particular instruction, it is first necessary to test whether operators understand what the picture means. It is not sufficient to test the picture with a colleague who is, perhaps, rather better educated than the intended user of the instruction; for he is likely to see in the picture a different message from that which the operator will see. Communicating with pictures is a very specialised skill.

It is believed that each environment in which this design and construction system is employed, will need to develop its own set of Operator Manuals. These manuals may need to go with tailormade training programmes.

3.2 **Maintenance**: Minor maintenance is, by Government policy, the responsibility of the community. This will include the repair and replacement of taps, repair of leaks and *must* also include the repair of small motors and pumps too. However, there appears to be some ambivalence on the part of Government engineers as to the capacity of the communities to perform these simple tasks; and there is, sometimes, a belief by the communities that they have a right to expect maintenance by Government staff despite living in very remote areas.

Earlier in the Manual it was noted that the financial capacity of Government (and indeed the Donors) is limited. If communities are encouraged to think that they have a right to free-of-charge services such as maintenance of their systems, this will inevitably lead to frustration on their part, and should be avoided. Thus, it is the responsibility of Government staff clearly to point out to communities what their responsibilities are in this matter. Better still, before placing a system in a community, the Government should obtain a written agreement with the community what their responsibilities are to be, and what those of the Government are.

It will have been noted that considerable attention has been paid to the problem of illegal connections. That some people will take – or try to take – illegal connections is not in doubt. The rôle of the Government staff in this matter is to ensure that the community understands that when this happens, it will be their responsibility to modify the system to recover the head to supply the distant taps.

It will also be seen that this design-and-construct system allows the Government a useful "lever" in this. When the distant taps dry up and the community is obliged to install additional impellers to the blind stages in the centrifugal pumps, they will have had to remove the pump/motor sets for repair. If they are capable of this small operation, then they are capable of maintaining their own systems. This should be pointed out to them. Also, particularly in Baluchistan, many communities are running their own irrigation systems. Boreholes are drilled for private sector farmers; borehole pumps are installed; electrical switchgear placed; and the system is operated – including both routine operation and maintenance – entirely by the farmer himself. Thus, to say that communities are incapable of operating and maintaining their own systems is somewhat unrealistic.

3.3 **Repairs and Rehabilitation**: Major repairs and rehabilitation are, again according to Government policy, the responsibility of the Public Health Engineering Department (PHED). This has been stated by the Provincial Government following the creation of the PHED. For schemes initiated by other agencies of Government (such as BIAD), the schemes will become the responsibility of PHED for major repairs immediately after handover to the community.

It should be noted that the whole design concept for the Modules seeks to reduce the need for Government intervention to the extent possible. It is not realistic – in Baluchistan at least – for a Government employee to drive 200 km to repair something minor. As the Modules are intended to be dynamic (changing in response to conditions in the future), any modifications which are proposed should have the need for community maintenance as a prerequisite.

The Modules provide a great asset to Government – that of standardisation. The inventory that will be required to cater for major repairs will be basically the same for all schemes.

With respect to rehabilitation, little need be said in this Design Manual, except in as far as how designs should be undertaken to avoid having to rehabilitate systems in the future.

The sums of money required to provide rural water supplies to communities are calculated on the assumption that once placed, such systems will last for ever. This is NOT the case. All systems will wear or malfunction at some stage. Those systems which are well maintained will last longer than those that are not. Those that are designed to allow for ease (and low cost) of maintenance are likely to last longer than others that are not designed with ease of maintenance as a design requirement.

4. ORDERING, STORING AND FLOW OF MATERIALS

This Section examines facets of ordering and inventory control.

4.1 **Ordering of Materials**: When planning rural water supply schemes, consideration must be given to the ordering of stock which is either required in large quantities (such as pipe, fittings and specials), or which takes considerable time to manufacture or procure (such as pumps or special valves).

Obviously, it is ideal to arrange to have such items delivered only shortly before they are required, as this saves considerable space and ensures that stocks are always fresh. However, this implies that accurate designs are to hand before stock is ordered. Also it means that a carefully considered and agreed WORKPLAN is available indicating *which* schemes are to start *when*; that accurate population data have been collected and are available; that topographic surveys that are accurate and reliable have been completed; and that borehole pumping tests have been completed which are reliable. Inaccurate data can make for very wasteful investments.

On the assumption that all the necessary data are to hand, then computer designs are performed on all the planned schemes for up to three years in advance. Such designs must, obviously, be accurate. They should take into account the range of residual heads appropriate, and should also avoid high heads caused by pumping to higher elevations for very small population groups.

When the computer designs have been completed, they are tabulated by scheme, and by pipe size. Annex VIII shows an example. It should be noted that a small allowance (5%) is made *at the planning stage* to allow for minor changes to take place later. However, this stock represents the total, and requires revision during the plan implementation; which means that on completion, no excess stock is held.

Secondly, the plan must be compared with this listing. The example in Annex VIII distills the plan into a listing of *which* schemes will be implemented *when*. This allows for a summary of what pipe lengths are required by *what* date. Such calculations are summarised at the end of Annex VIII.

This same pipe stock flow table—shown in Annex VIII —clearly indicates *what* shortfall of pipe will occur, *when*. Ordering takes place to ensure that sufficient stock is to hand when it is needed.

This type of calculation is undertaken for each item that is required to be supplied to the schemes; but must be done sufficiently far in advance to ensure timely delivery.

4.2 **Storing Materials**: Designers throughout the world are aware of the need to store materials in a way that will ensure that they are not damaged before use. Earlier, in Section 4.1, mention was made of ordering materials so that they are received only shortly before use. This will reduce needed storage space, and will ensure that the supplies are fresh.

Several materials will degrade with time, with exposure to moisture and/or with exposure to sunlight/heat. These include cement, items made from iron or steel, and upvc. They also include chemicals that are used in water supply treatment such as bleach (sodium hypochlorite). Obviously, a Design Manual should not deal definitively with the storage of materials; but it may be useful to mention some of the basic needs for materials that are likely to be used in bulk.

Upvc pipe degrades in both sunlight and high temperatures. Direct sunlight tends to make upvc brittle, and should be avoided at any cost. Some manufacturers will mix an ultra-violet (UV) inhibitor into the base material, but even this is only partially successful in reducing the effects of the ultraviolet light. Thus, upvc pipes should always be stored away from direct sunlight. To the extent possible, such pipes should also be stored in an area which is cool and well ventilated.

Theoretically, galvanised iron (GI) products are supposed to withstand the effects of moisture. This is only true if the galvanising is of a very high quality—which is rarely the case. Thus, GI products should be stored where the effects of damp are reduced to the minimum. Air should *always* be allowed *under* such products.

Cement is perhaps one of the most badly stored items of all. It should be kept dry since cement depends on water to start the chemical process which leads to setting. It should be well ventilated; and, as for other items, air must always be allowed to circulate freely on all sides and *under* the cement. This can easily be arranged by suitable use of wooden slats. Cement should never be stacked more than 12 bags high; and should be stacked in squares of not more than 4 x 15 bags at a time. Further, each stack MUST be dated on arrival; and the principle of "First In, First Out" must be rigorously applied.

Bleach is not commonly used in rural water supplies because it has so many problems associated with both storage and use. This is indeed reasonable, since if the water supply system is properly designed and operated, there are very few cases where the addition of bleach will do anything to improve the quality of the water. Rather, bleach will often impart a foul taste to the water. However, where bleach is to be used, very great care must be exercised in its storage. Bleach, whether in powder or liquid form, is adversely affected by direct sunlight, by heat and, in the powdered from, by humidity.

Storage routines such as the use of bin and master cards are not dealt with in this Manual. The reader is directed to a number of standard texts on the subject. It should be noted however, that the efficiency of storage can have a profound effect on the ability that the Organisation has to meet deadlines.

4.3 **Flow of Materials**: Section 4.1 and Annex VIII present some practical examples of planning when and where materials need to be. What they do not highlight is the complexity caused by having some materials already in stock; some on order; some in the field ready for use; and some which have recently been consumed. For a successful flow of newly constructed—and indeed maintained—projects, the engineer must be able to determine at all times, the flow of the materials he needs.

For materials currently in stock, one needs an up-to-date inventory. There are several ways in which this inventory can be made. The usual way is to have master cards and bin cards. The same thing can be achieved on a computer, *but only if the system which was formerly run by hand is accurate.* Also, computers have this nasty habit of losing all their data when the electricity supply is interrupted.

Whatever system is used, it must be checked periodically. This is best done once per year in an Annual Inventory. At least one agency of Government in Baluchistan does this, and the resulting document is very valuable. It gives every item in four different locations, both at the end of the previous year and at the end of the current year. There may be around four to five hundred different items on the inventory; and the process of compilation is greatly accelerated by neat stacking and ease of accessibility.

A similar system must be available for materials on order. Too frequently, projects suffer long delays because the engineer did not follow-up on materials which were supposed to have arrived but did not. Also, some engineers do not plan sufficiently far ahead to allow for the delay between when an item is ordered and when it is delivered. Typically, pipes or other items which are procured overseas require a lead-time of around nine months. Even locally available materials will take up to four months to obtain because of the processes through which the Departments or Agencies must go, such as tendering, adjudication and the like.

Which then leads on to those materials which have been released from Government stocks for use, but have not been consumed as yet. Again, track must be kept of these materials beyond ensuring that *challans* and *gate-passes* have been issued.

When all three are put together—materials needed and on order; materials already in stock; and materials released for incomplete projects—one can begin to see the complexity of the matter. The success of the Department or Organisation concerned in providing completed projects may well depend as much on stock control as it does on good designs and good site supervision.

5. BOREHOLE PUMPING TESTS

The following Section provides perhaps the most important design parameter for ground water based schemes. It has thus been included in the Manual. Without reliable and accurate borehole pumping test data, the whole design may be in jeopardy.

5.1 The Importance of Pumping Tests and some Basics Relating to the carrying out pump Tests and Calculating Aquifer Parameters:

5.1.1 *Introduction*: The Science of Ground Water Hydrology is concerned with evaluating the occurrence, availability and quality of ground water. Although many ground water investigations are qualitative in nature, quantitative studies are necessarily an integral part of the complete evaluation of occurrence and availability. The worth of an aquifer as a source of water depends largely upon two inherent characteristics—its ability to store and to transmit water.

Thorough knowledge of the geological framework is essential to understand the operation of the natural plumbing system within it. Ground water hydraulics is concerned with the natural or induced movement of water through permeable rock formations. The principal method of analysis in ground water hydraulics is the application, generally by field tests of discharging boreholes, of equations derived for particular boundary conditions. Some of these equations are very complex, constructed by mathematicians who fortunately have been overtaken by even better mathematicians who have made the equations simple in nature and easy to solve by straight line semi-log techniques.

5.1.2 *Terms Used*: Two fundamental terms are commonly used. These are the transmissibility and the storage coefficient. The transmissibility (T) of an aquifer is defined as the rate at which water flows through a unit width of the aquifer under a unit hydraulic gradient. The definition of storage coefficient is that volume of water an aquifer releases from, or takes into storage per unit surface area of the aquifer, per unit change in head.

Both of these definitions are difficult to understand in non-mathematical terms. Nevertheless, intelligent statements about their values can be made without understanding their meaning. The importance of both "T" and "S" is discussed below. Of the two, transmissibility is particularly important.

Several assumptions (e.g. Is the aquifer confined or unconfined; is it of infinite areal extent etc., etc.) govern the validity of the equations used to obtain "T" and "S". These conditions are listed in standard texts but a sufficiently good estimate of "T" and "S" can generally be computed even if the field conditions are not satisfied. The assumptions need not concern the reader.

The units of "T" are either square metres per day (m^2/d) or, in Imperial Units, galls per day per foot (g/d/ft or g/(day x ft)). The coefficient of storage is dimensionless.

If the transmissibility and coefficient of storage can be determined for a particular aquifer, *predictions* of considerable significance can usually be made. Some of these are:

1. Drawdown in the aquifer at various distances from a pumped borehole.
2. Drawdown in a borehole at any time after pumping starts.
3. How multiple boreholes in a small area will affect one another.
4. Efficiency of the intake portion of the borehole.
5. Drawdown in the aquifer at various pumping rates.

As is evident, some of these predictions are of vital importance, particularly predictions (1), (2) and especially (5). Prediction (4) is important in large yielding boreholes when the efficiency is a factor to be considered and proper borehole design with a gravel pack, etc., can influence efficiency significantly. Prediction (3) is important in high density borehole areas but at the moment this is of little significance in Baluchistan. Before describing the technique of a pump test, it is important to understand clearly the meaning of common terms related to pumped boreholes. The definitions are presented in the figure on the following page.

Static Water Level (SWL): This is the water level at which water stands in a borehole when no water is being removed from the aquifer.

Pumping Water Level (PWL): This is the level at which water stands in a borehole when pumping is in progress.

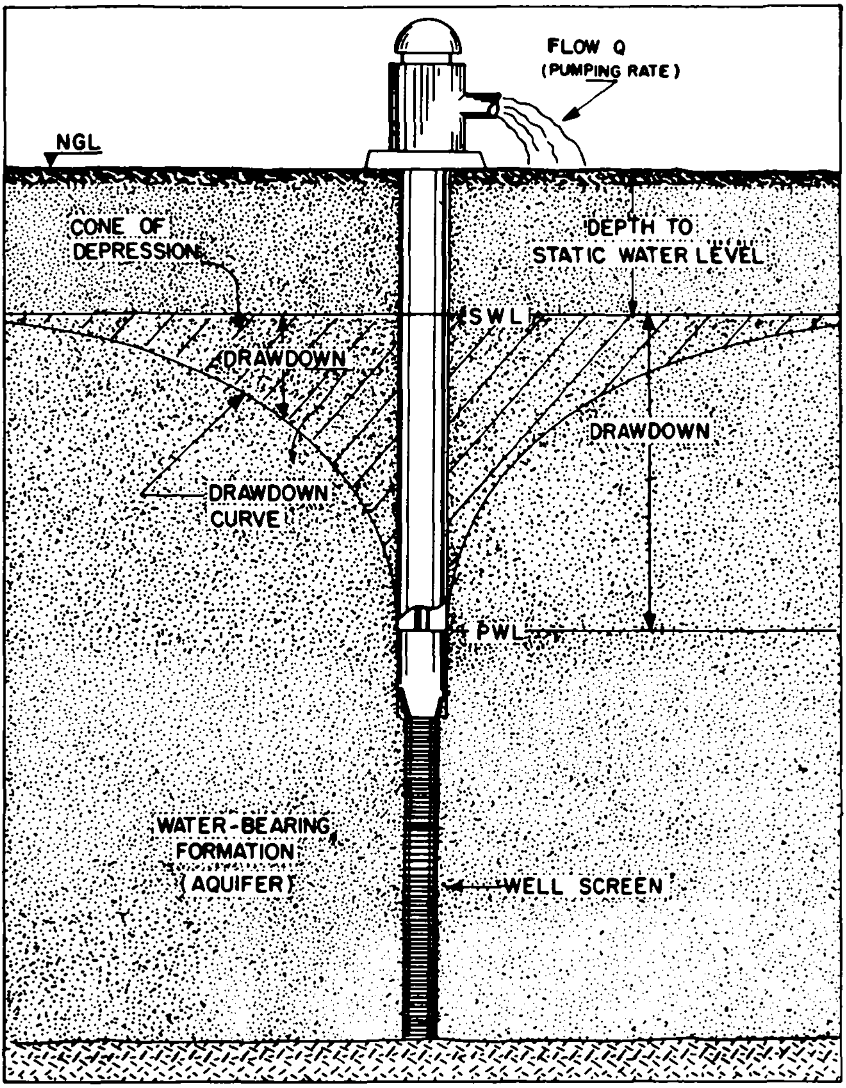


Fig. 1: TERMS USED IN THE TEXT RELATING TO BOREHOLE AND WATER LEVELS

Drawdown: Drawdown is the difference measured in metres (or ft) between the static water level or piezometric level and the pumping water level.

Residual Drawdown: After pumping is stopped, the water level rises and approaches the static water level observed before pumping began.

During water level recovery, the distance between the water level and the initial static water level is called Residual Drawdown.

Well Yield: Yield is the volume of water, per unit of time, discharged from a borehole, typically quoted in litres/sec or Igpm.

Specific Capacity: Specific Capacity of the borehole is its yield per unit of drawdown, usually expressed as litres/min, per metre of drawdown (or gallons/minute, per foot of drawdown).

The numerous equations which are used in pumping tests are all based on Darcy's basic flow equation of groundwater flow towards a pumping borehole. This equation is more than 100 years old. The early equations which were used to calculate the aquifer characteristics depended upon the borehole reaching equilibrium conditions—i.e. when the water level no longer decreased during a pumping test. In Pakistan, it is felt that whenever boreholes are pump tested (alas, this is not nearly frequently enough), equilibrium is reported to be reached. This is at variance with the usual situation where equilibrium conditions are seldom reached and the pumping water level is *not* constant. There may be two reasons for the fact that equilibrium conditions "appear" to be reported more than expected. Either the measurements are not correctly carried out or the pump is small in relation to the potential aquifer yield. A 1.25 l/s (1,000 Igph) pump in a borehole capable of yielding 63 l/s (50,000 Igph) will certainly result in equilibrium conditions. In general, however, the water level is always going down as pumping continues. The curve describing this is asymptotic from the pumped level to the static water level and in real terms equilibrium conditions are seldom reached. But after several hours, because the rate of water level decrease is so small, it appears to be constant. Nevertheless, it is easy to calculate the transmissibility and the storage coefficient of an aquifer even when non-equilibrium conditions exist.

The practical aspects of a pump test are now addressed. It is a simple procedure, but the sequence of events must be strictly followed even though this requires considerable patience.

There are two separate activities involved during the pump test. One is the sequential measurement of water levels once pumping has started; and the other is the measurement of the flow of the pump at several specific times during the pump test to ensure that it remains constant.

5.2 MEASUREMENT OF PUMPING TEST PARAMETERS:

5.2.1 *The measurement of water levels*: There are many ways in which water levels can be measured; the use of a chalked tape, electrical meter, a float type mechanism, sonic devices, and several others. Only *two* methods of measuring water levels are addressed:

(1) *The use of a wetted survey tape*: This is the simplest and certainly the cheapest. The ideal tape is a 100 metre steel survey tape, which is usually 10 mm (3/8″) wide. The bottom two metres are chalked with normal school black-board chalk and the tape (with a weight at the end) is lowered into the borehole. Assume that the water level is around 50 metres. Lower the tape to 50 metres and accurately hold momentarily the 50 metre mark on the tape at the top of the casing. Then wind the tape up. If the water level was 49 metres then the lowermost 1 metre of the tape will be wet. The line between dry chalk and the wet tape is very sharp and the depth of water can be measured to ± 2 millimetres.

If, on the other hand, the water level had been 50.3 metres, then of course the tape would have been dry, because the tape had been held at the 50 metre mark at the surface. The tape is again lowered, this time, say, momentarily holding the 51 metre mark at the top of the casing. On winding up the tape, 0.70 metres will be wet.

It is again noted that this is a simple, accurate, method of measuring water levels. It obviously needs speed if water levels are required at one or two minute intervals, but this is easily possible, if the water level is not too deep.

(2) The second simple method of measuring water levels is by the use of *commercially available water level recorders*, which are usually based on a simple electronic circuit, whereby once contact is made with water, the water acts as a short circuit and a light or a buzzer is activated at the surface. Obviously this is a more effiecient and a more rapid way of measuring water levels. It is of course far more suited to pump test measurements where they must be taken at frequent intervals early in the pump test.

Water level measurements during the pump test may be taken in the pumped borehole or in an observation borehole anywhere from 10–100 metres from the pumped borehole. Results for T & S (The Transmissibility and Storage Coefficient – see later discussion) are more valid using measurements from the observation borehole, but often an observation borehole is not available. S values which are calculated from pumped borehole data are seldom accurate.

5.2.2 *Frequency of water level measurements*: Following the discussion of measuring devices, the next important consideration is the *frequency* with which water levels must be taken when a pump test is performed. Ideally, the following general procedure should be adopted:

From 0 – 10 mins : Measure depth every 1 minute;
From 10 – 20 mins : Measure depth every 2 minutes;
From 20 – 60 mins : Measure depth every 5 minutes;
From 1 – 2 hours : Measure depth every 15 minutes; and
From 2 hrs onwards : Measure depth every 1 hour.

These figures are rule-of-thumb and obviously can be changed. The reason why frequent measurements are taken early in the pump test is that the rate of change of water level is most rapid during the first 10 minutes and thereafter slows down and becomes, as noted before, asymptotic (unless boundary conditions intervene; see later sections).

5.2.3 *The duration of a pump test*: Generally speaking, if confined conditions exist, (the reader is referred to the standard texts for a discussion of confined and unconfined aquifers), a 24 hour test is sufficient.

In unconfined situations the test should be longer; but these are ideal requirements. It is suggested that in the first instance, *an 8 hour test* be considered an standard and mandatory. In other words, pumping takes place continuously for 8 hours irrespective of whether confined or unconfined conditions exist, and water level measurements are taken at intervals as indicated above. Pumping for longer periods is considered much better.

5.2.4 *Flow Measurement of the Pump*: The other activity, apart from water level measurements, is a measurement of the *flow of the pump*. This may be measured in a variety of ways, either by use of a bucket and stop watch, or a small V-notch or a circular orifice weir and a piezometric tube.

For flows of up to 2 1/s (25 Igpm), a 20 litre (4 gall) bucket and a stop watch is quite sufficient and of acceptable accuracy. This may be illustrated by an example:

> Suppose the flow rate of the pump is 1 1/s. It will take approximately 20 seconds to fill a 20 litre bucket. If the error in the timing is one second – and this is a reasonable error by somebody

only moderately trained—then this represents an error of ± 1 in 20 or ± 5% in the flow rate. This is quite acceptable. If 5 consecutive readings are taken of the time to fill a 20 litre bucket, it is quite reasonable to assume that the mean of 5 readings will have an error of ± ½ second which means an error in the flow rate of ± 2.5% which is more than acceptable in ground water investigations. If, however, the flow is around 4 l/s, then it will take only approximately 5 seconds to fill the 20 litre bucket and, assuming a timing error of 1 second, the resulting error in flow is ± 20%. This means that for flow rates in excess of 2 l/s, either a 200 litre (44 gallon) drum should be used or a small V-notch or a circular orifice weir employed.

This very simple error analysis shows clearly how easy it is to obtain more than acceptable flow results from the bucket and stop watch method with a 20 litre bucket if the flow rates are relatively low; or the use of a 200 litre drum (or some *known* large volume) for higher flow values.

It is important to check the flow rate approximately every half hour to determine that it is constant, because the calculations of the aquifer characteristics are dependent upon a *constant* flow rate during the pump test.

5.2.5 *What is Practical*: Having dealt with the *ideal*, it must be accepted that it may in some instances not be possible to follow these procedures precisely. It is found, for example, that many engineers in Sind province, say that they are unable to measure rapidly dropping water levels even using the electrical water level recorder. This may only be a reflection of a lack of adequate training, because it is possible even though it requires concentration and speed. If one had to make a few rules as to the *minimum requirements* for a pump test, these would be as follows:

1. The measurement of water level *before* pumping starts (vital).
2. Water levels at, say, 5 minute intervals during the first hour of the test; and levels at hourly intervals until the pump test has been stopped. Obviously too, soon after pumping begins, one determines the flow, and subsequently checks that it is constant.

These would be the *minimum* requirements to allow some meaningful statements to be made about crude approximations of the aquifer characteristics, borehole yield and specific capacity.

5.3 **Calculations of Aquifer Characteristics**: As noted earlier, the mathematics involved in hydraulic flow towards a borehole are complex and some of the equations used to calculate transmissibility and storage coefficient are very difficult to solve. In Section 5.1.2, the definitions of both "T" and "S" have been given with some notes. It is possible to obtain sufficient accuracy in "T" and "S" values even if the field assumptions required to satisfy the equations are not met.

5.3.1 *Solution using Cooper and Jacob's Equation*: Only one of the simplest methods of calculating the transmissibility and storage coefficients will be presented. This is Cooper and Jacob's Equation which allows the use of a semi-log graphical solution. Fig 2 indicates how the drawdown is plotted versus time on semi-log graph paper. That figure results from the data shown in Table 5.1 and is used as a calculated example.

TABLE 5.1

DRAWDOWN MEASUREMENTS IN OBSERVATION WELL
122 m FROM PUMPED WELL

Time since pump started in min	Drawdown. (m)	Time since pump started in min	Drawdown. (m)
1	0.049	24	0.482
1½	0.082	30	0.518
2	0.116	40	0.573
2½	0.140	50	0.610
3	0.162	60	0.643
4	0.204	80	0.683
5	0.235	100	0.725
6	0.265	120	0.759
8	0.302	150	0.799
10	0.341	180	0.829
12	0.369	210	0.856
14	0.396	240	0.878
18	0.436		

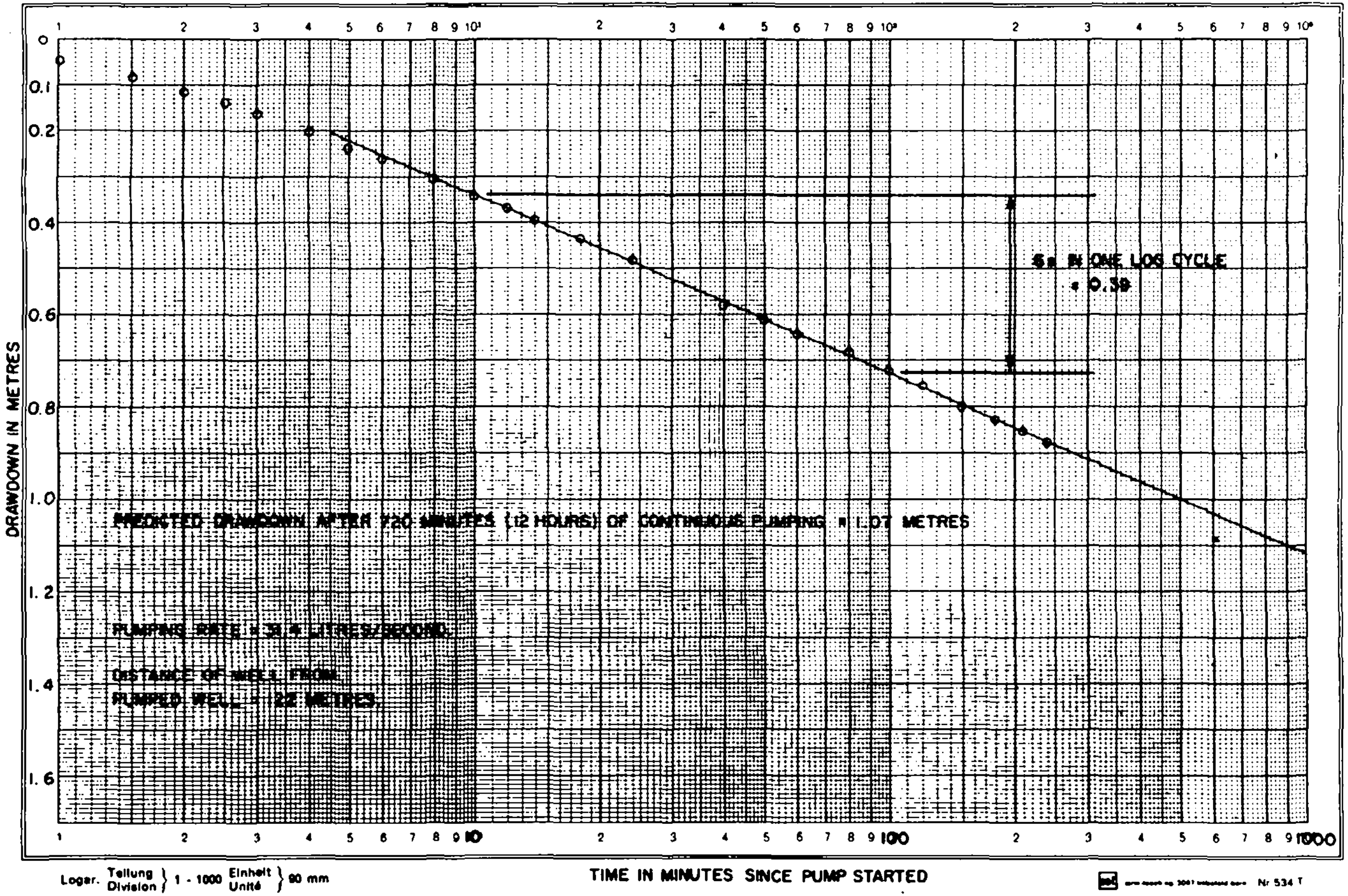


Figure 2: WHEN DATA FROM TABLE 5.1 ARE PLOTTED ON SEMILOGARITHMIC GRAPH PAPER, MOST OF THE PLOTTED POINTS FALL ON A STRAIGHT LINE.

Obviously, if water level measurements are taken frequently early in the pump test, the curved part of the graph may be defined. This is important in terms of the calculation of the storage coefficient which tells one how much water is available from the acquifer.

It is the slope of the line that defines the transmissibility. The drawdown results are shown in Table 5.1 and the resulting graph in Figure 2.

5.3.2 *The Transmissibility* (T) is calculated from:

$$T = \frac{15.84 \times Q}{\delta s} \qquad \text{Units: } \frac{\text{metres}^2}{\text{day}}$$

Where Q = Pumping rate in litres/sec (1/s).

δs = Slope of time drawdown graph; expressed as the change of draw-down between any two times of the log scale whose ratio is 10 (one log cycle), in metres.

In the example (Figure 2): $Q = 38$ 1/s

$\delta s = 0.4$ m

$$\text{Thus } T = \frac{15.84 \times 38}{0.4} = 1{,}505 \text{ metres}^2\text{/day.}$$

How the transmissibility may be used to determine what pumping plant should be used on the borehole is shown in Section 6.2.

5.3.3 *The Storage Coefficient* (S) is also readily calculated from:

$$S = \frac{1.875 \times T \times t_o}{r^2}$$

Where T = Transmissibility in m^2/day.

t_o = Intercept of the straight line at zero drawdown in *days*.

r = Distance in metres from the pumped borehole to the observation borehole.

In the example $T = 1{,}505\ m^2/day$;

$t_o = 1.44$ minutes or 0.001 day; and

$r = 122$ metres.

$$\text{Thus, } S = \frac{1.875 \times 1{,}505 \times 0.001}{(122)^2}$$

$$= 1.9 \times 10^{-4}$$

It is important to note that storage coefficients calculated from drawdown data from pumped boreholes are not reliable. This is because in cases where no observation borehole is available and the water level measurements must be taken in the borehole itself, it has to be assumed that the factor "r" (distance from the borehole) is half the diameter of the borehole itself. It is further noted that if there is a nearby borehole to the one on which the pumping test is being carried out, the nearby borehole may be used as the observation borehole.

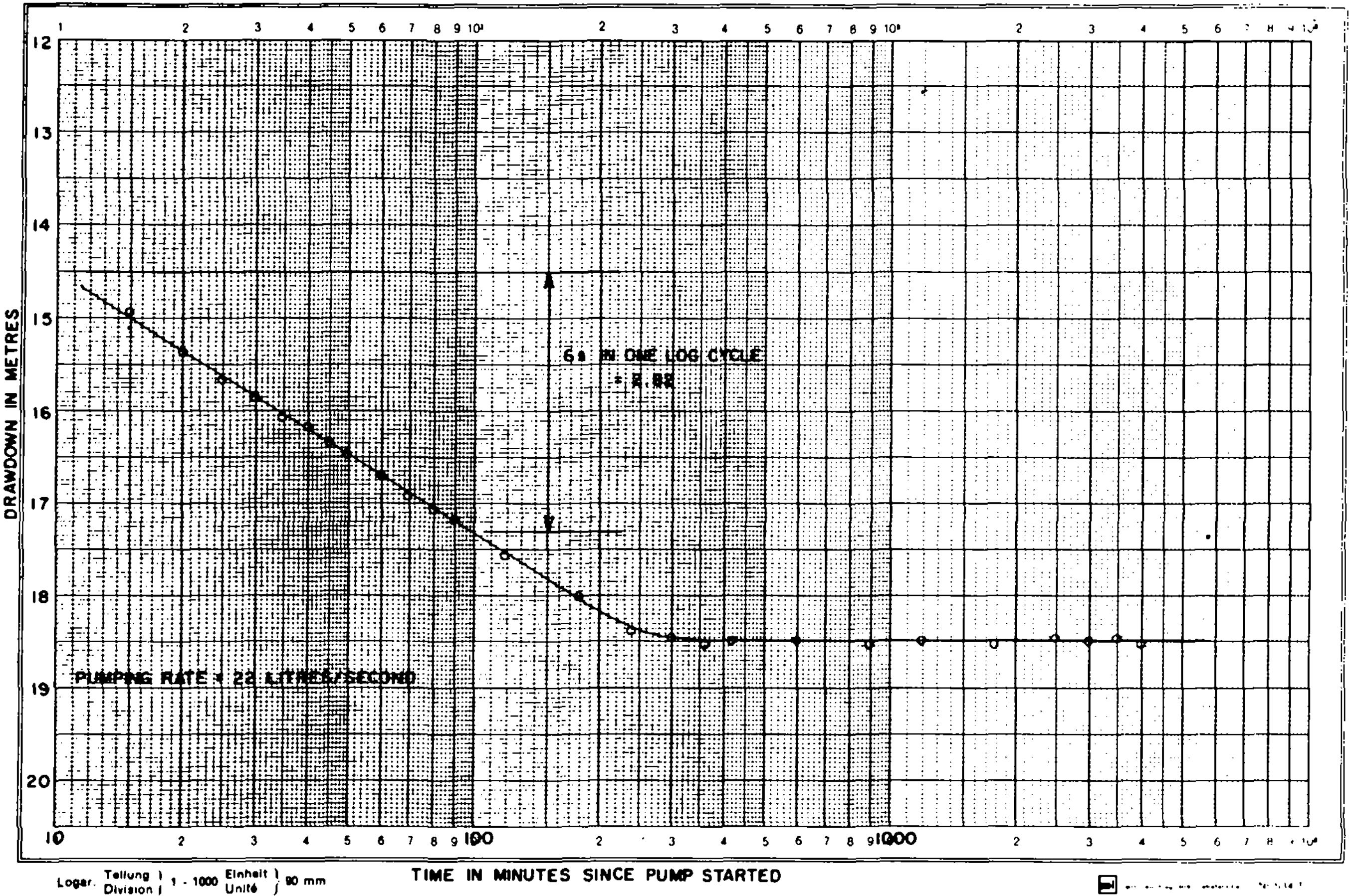


Figure 3: WHEN RECHARGE TO THE AQUIFER OCCURS WITHIN THE ZONE OF INFLUENCE OF THE WELL, THE SLOPE OF THE TIME-DRAWDOWN CURVE BECOMES FLATTER. THE HORIZONTAL LEG INDICATES THAT RECHARGE EQUALS WELL DISCHARGES AFTER 240 MINUTES OF PUMPING.

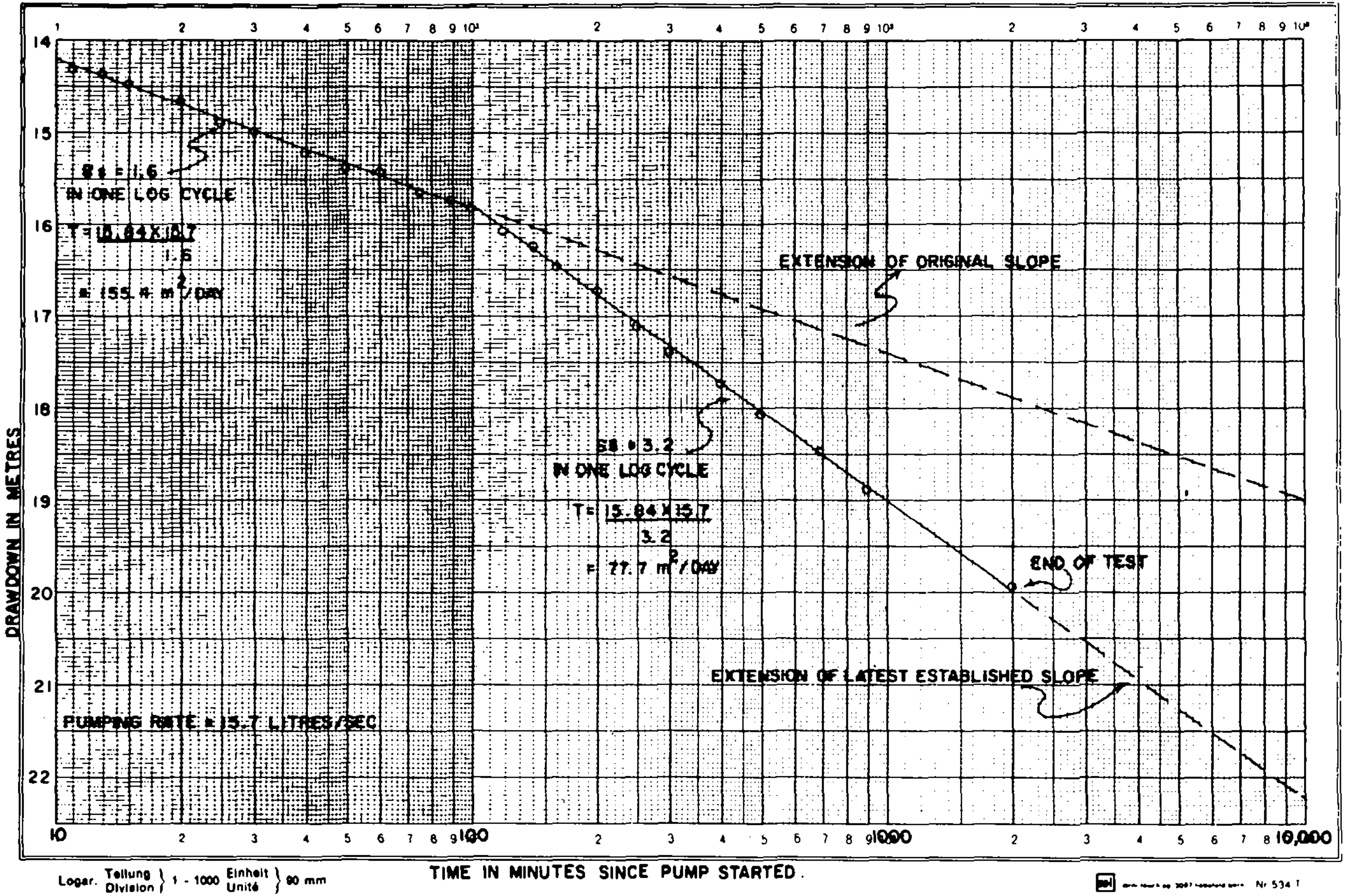


Figure 4: STEEPENING OF THE TIME-DRAWDOWN, SEMILOG CURVE INDICATES A LIMITED AQUIFER. THE EXPANDING CONE OF DEPRESSION ENCOUNTERED ONE OR MORE IMPERVIOUS BOUNDARIES AT THE TIME SHOWN BY THE CHANGE IN SLOPE.

It is important for the reader to have an understanding of what the change in slope means in the previous two graphs (Fig. 3, and Fig. 4).

In Fig. 3, the second straight line illustrates that for a constant pumping flow, the rate of fall of water level (or drawdown) has decreased indicating that the cone of depression has reached a more permeable boundary. The following figures illustrates an example:

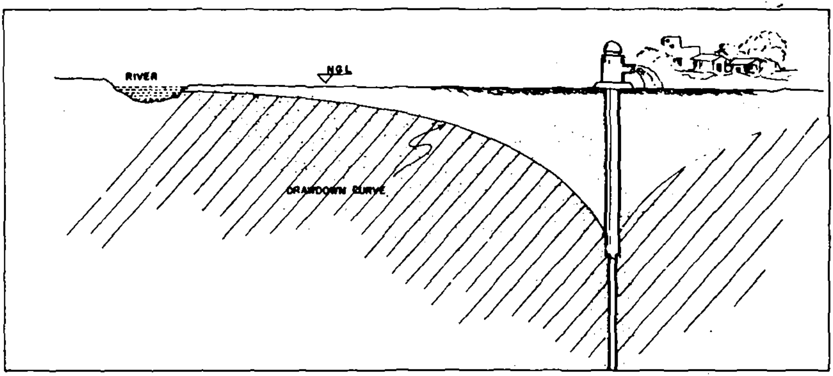


Fig. 5: WHERE THE CONE OF DEPRESSION HAS MET A MORE PERMEABLE BOUNDARY (See Fig. 3)

If, on the other hand, the cone of depression meets an impermeable boundary such as crystalline rock, the drawdown curve becomes steeper such as shown in Fig. 4. This is the opposite of what occurs above.

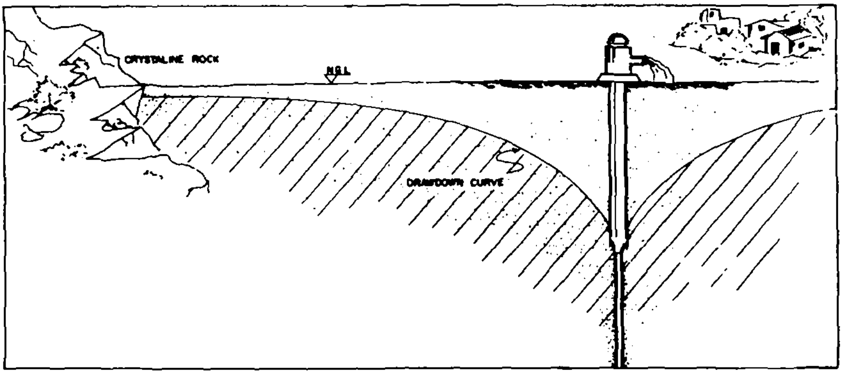


Fig 6: WHERE THE CONE OF DEPRESSION HAS MET AN IMPERMEABLE BOUNDARY (See Fig. 4)

In the case illustrated in Fig 3, "T" must be calculated from the first part of the graph, while in Fig. 4, "T" must be computed from the second part of the graph. In practical terms it is evident that the total volume of ground water in the pump test in Fig. 4, is limited, whereas in Fig. 3, this same limitation does not exist.

It is thought that this explanation of the pumping test procedure is not too complex. It should be easily understood by engineers and hydrogeologists alike. It is reiterated that a few simple measurements can define the validity and the security of the yield of a particular borehole. However, pump tests do not absolutely guarantee the security of a water supply. For instance, in confined conditions a 24 hour pump test will fully describe the aquifer charateristics, and predictions made upon such a pump test may be looked upon as valid. In unconfined ground water conditions, a short pump test can be misleading.

It is noted that an 8 hour pump test is considerably better than no pump test at all; and still better than, say, a 4 hour pump test. These may seem exaggerated statements but they illustrate the importance that the longer a pump test is carried out particularly in unconfined conditions, the better.

5.4 **Conclusions**: There appears to be only one area of the country in which pumping tests on boreholes should not be carried out as standard. This is in the Tharparkar Desert where there is a very delicate balance in the aquifer. Sweet water, being very slightly lighter than saline water, tends to float at the top of the aquifer. If a borehole pumping test is carried out on a borehole drilled in such an environment, there is a real danger that upconing of the salty underlayers will occur. Thus, in Tharparkar, pumping tests should not be undertaken. This has already been noted by the Chief Engineer, PHED, Sind, who has set a limit to extractions from any borehole in the area at 1.25 l/s (1,000 Igph). But, with the possible single exception of the Tharparkar Desert area, pump tests *should always* be carried out.

The current practice in Baluchistan, where flow rates and water levels are quoted, and specific capacities calculated, becomes meaningless if a *time* has not been assigned to the drawdown. It is absolutely vital that if a drawdown is mentioned that the time after pumping began when the drawdown was taken is recorded. A drawdown without the associated time figure is of limited value and can be dangerously misleading.

Finally, it must be stressed once again that if a borehole is drilled and a proper pumping test not carried out on it, then millions of rupees can be wasted. The design of any scheme is always first and foremost dependent on the security and availability of the water supply; and without a properly conducted pump test the security and availability of the water in the particular borehole cannot be stated.

Pumping tests are vital and should always be undertaken for any piped rural water supply scheme based on a ground water source.

The figures in this Section are based on "Groundwater and Wells" by Driscol. See Bibliography.

6. PARTICULARS RELATING TO GROUNDWATER BASED SCHEMES

This Section examines aspects of the design and construction system which relate only to groundwater based schemes.

6.1 **Borehole Drilling, Development and Pumping Test**: It cannot be stressed too much that the success of the whole scheme will depend upon the yield, quality of water and reliability of the source. The Manual does not set out to instruct drillers in how to drill boreholes; but the engineer designing a scheme for which the borehole has not been drilled, cased and pump-tested properly, does so at the risk of wasting considerable sums of money.

An example may serve to show the importance of the earlier comments. In one district of Baluchistan, plans were agreed to provide water supply systems to eight different communities in 1988. Seven of those were to be based on boreholes, all of which had been drilled. One was to be based on a spring, the yield of which had not been determined. No pumping tests had been performed on any of the boreholes. On inspection, it was seen that the spring had dried up, and one of the communities had already been supplied with water by another agency. Consultants were engaged and designs were required for only six of the eight schemes. When the pumping tests were performed, no time was assigned to the yields that were recorded so that reliability of the reported flows was somewhat questionable. Even so, two of the six schemes could not provide adequate water even for the present populations. As a result, the consultants only could produce the designs for four of the original eight schemes, and materials for the other four remain unused in stock.

The reader's attention is again drawn to Section 5 which deals with pumping tests on boreholes.

6.2 **Design of Borehole Pumping Plant**: Such plant should never be designed in the absence of valid, reliable and recent pumping test data. To design plant without a test leads to the possibility of drying the borehole and totally destroying the pump and motor set.

Since few sites exist in Baluchistan where a centrifugal pump can draw water from a borehole, only "top driven" turbines are considered here. While submersible pumps are feasible, few local firms are capable of handling them, and they are thus not considered. Also, submersibles are less efficient than top-driven turbines. See Annex I.

Section 5 dealt in detail with borehole pumping tests. It is vital to note that a full understanding of the meaning of the graphs in Section 5 is necessary before trying to design any pumping plant.

Designers will know that the power of a turbine is dependent on both pump flow and the head through which the water is to be pumped. A study of the drawdown curves in Section 5 will also show that–in general–the higher the flow from a borehole, the greater the drawdown. What this means is that even if a borehole will provide a larger flow than is necessary for the community, one *never* installs a pump to provide such a flow because when the flow increases above what is necessary, *so does the head.* This in turn means that the required pump power goes up not arithmetically but geometrically. A simple example with demonstrate what is meant:

Static water level, say = 0 metres (at ground level)

Drawdown at 5 l/s = 10 metres

Drawdown at 10 l/s = 20 metres

Horsepower at 5 l/s (h @ 10 m) = k x 50 horsepower.

Horsepower at 10 l/s (h @ 20 m) = k x 200 horsepower.

This clearly shows that–for this case at least–if one doubles the flow, the horsepower required does not double, it rises by a factor of *four.* As communities are having to pay for the power to pump the water, one thus needs to ensure the minimum feasible horsepower. Some engineers will note that doubling the flow will halve the time to pump the water, which is true; but it must be remembered that even so, the cost per litre of the "fast" pumped water is still twice the "slow" pumped water, in this case.

It would appear that one should thus design for the minimum feasible flow to satisfy the community demand. This is not entirely so because the size of the GLT also depends upon the flow from the borehole. If the flow from the borehole is very small, the GLT becomes very large; and thus the capital cost of the scheme rises. On the other hand, if the flow from the borehole is greater than or equal to the community demand flow, the GLT will be small, but the cost of pumping will perhaps be higher than necessary.

The determination of Q_b–the well long-term yield–which is referred to in Section 2.9, appears complex. However, if the checklist which is shown below is followed, the value of Q_b may be obtained quite easily:

1. If both "T" and "S" have been computed (See Section 5), the potential yield of the well or its specific capacity after a certain period of pumping can be obtained.

2. If no value of "S" is available (i.e. if no observation borehole was used in the pump test) an educated estimate must be made concerning the yield. If the pumping test has revealed no impermeable boundary conditions, then a safe "rule-of-thumb" is to pump at a rate where the *drawdown equals one half of the total column available*. For example, if:

   Depth of borehole D = 100 metres
   Static water level, SWL = 20 metres

   Water Column available = D – SWL = 80 metres

   Safe long-term Yield Q_b is when
   drawdown is ½ x column available;
   or at ½ x 80 m i.e. @ 40 m *or less*

   This implies a Pumped Water Level PWL = 20 + 40
   = 60 metres

If, on the other hand, impermeable boundary conditions do exist, the estimated initial yield will have to be regularly reassessed as pumping continues and the *drawdown is carefully monitored.*

Whether or not boundary conditions exist it is *absolutely mandatory regularly to measure the drawdown* in the borehole both during pumping and before pumping begins in the morning *at least once a month.*

If the pumping test is reliable, and if the drawdown is small, the aim should be to provide pumping plant which provides the community demand flow. This will give the minimum size of GLT. Only if the drawdown is great is there a need to vary the size of the GLT. This is well illustrated in the flow diagram, Section 2.9.

When the flow that is to be used for the calculations has been selected as noted above, and the associated drawdown determined, then the procedure to design the plant is as follows:

There are three steps in pump design:–

1. Calculate the pump power at 100% efficiency.
2. Size the pump from estimated efficiency.
3. Size the motor from estimated efficiency.

The following system of pump and motor design should be viewed as *approximate* only; and is used to guide the manufacturer/supplier in the requirements of the plant. The actual pump performance curves should be used to determine whether the pump is suitable for the specified duty. This is covered in nearly all standard texts on the subject, but an example of a pump performance curve is presented in Annex II. A discussion is also presented concerning the options when the curve does not fit the precise specification.

1. *Pump Power*: (At 100% Efficiency)

Where P = Pump power in horesepower
h = Total lift (static + dynamic), metres
q = Flow in metres ^{3}/sec (from pump test data)
1 HP = 0.746 kilowatts

$$\text{Then } P = \frac{9.81 \times q \times h}{0.746} \text{ horsepower}$$

2. *Pump Size*: (At Actual Efficiency)

Since a turbine cannot be 100% efficient, one has to put a little more power into the pumping than it actually needs to pump the required amount of water. Turbines in the range of 2 to 5 horsepower typically display an efficiency of ± 70%.

$$\text{Thus "Pump Power"} = \frac{P}{0.70} \text{ horsepower}$$

3A. *Motor Size* (Electrical):

Motors convert electricity into mechanical power. Again, they are not 100% efficient, and thus the amount of electric power consumed by the motor is more than the "pump power" noted in 2), above. Typically, electric motors in the range 2–5 horsepower display an efficiency of 90%.

$$\text{Motor size} = \frac{P}{0.70} \times \frac{1}{0.9} \text{ horsepower}$$

When ordering pumping plant, this is the horsepower quoted in the first instance. When the manufacturer's bid is received, a precise check is run as shown on Annex II.

3B. *Motor Size* (Diesel):

Diesel engines are used where electricity from the national grid is not available. They are considerably less efficient than electric motors and are adversely affected by both high temperatures (making the air less dense) and altitude (which has the same effect). When designing a diesel engine for this application, one has to perform two sets of calculations:

(i) Establish the engine power requirement at 20°C and at sea-level; and

(ii) Adjust both for maximum average shade ambient temperature, and for elevation. Annex III presents the power fall-off with both temperature and altitude.

If E_e = Engine efficiency at 20°C and at sea-level

$$\text{Then Engine H.P.} = \frac{P}{0.70} \times \frac{100}{E_e}$$

If F_f = Falloff Factor due to temperature and altitude

$$\text{Then Engine H. P.} = \frac{P}{0.70} \times \frac{100}{E_e} \times \frac{1}{F_f}$$

Warning: Engineers usually want to design a "safety factor" into any pumping plant they may need to order. While this might make them believe that the equipment so ordered "will do the job" better if it is oversized, they run the real risk of overpumping the borehole; drying it up and thus destroying the equipment. Engineers who do this display a lack of understanding of how pumping plant works. Not only is oversized equipment heavy and difficult to handle, but it runs the risk of damaging the water supply itself. Also, most turbines are water-lubricated. Overpumping can dry-up the lubricant itself.

Manufacturers or suppliers of pumping plant are in the business of making money. They wish to sell bigger plant than ordered, as more money is made this way. They often quote pump and motor efficiencies somewhat below the actual efficiencies at which their equipment will perform. *Accept-*

ing the efficiencies "guaranteed" by manufacturers may lead to overdesign and risk of drying the pump.

Communities have a mistaken belief that if they place a bigger motor on the pump with which they are supplied, they will obtain more water. This is not so. This is because driving a pump at a higher speed or a higher potential power input than that for which it was designed, will drop the efficiency. There are many examples of this in Baluchistan; the faster speed taking *longer* to pump the same volume of water as before. An inspection of the pump performance curve will show why.

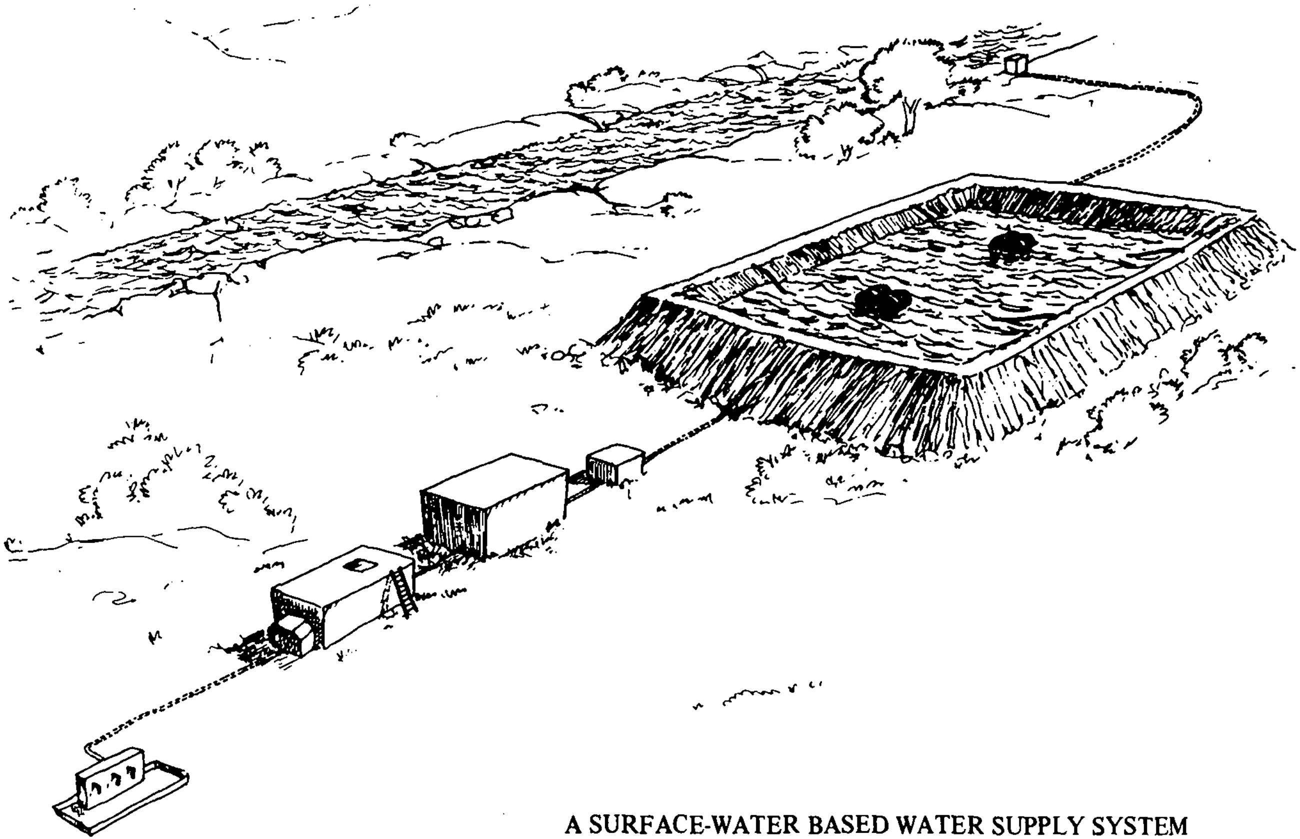
In summary; design with care and *do not overdesign.*

6.3 **Pumphouse Design**: The pumphouse has been designed to accomodate either an electrically driven turbine or one with a diesel driver. No design considerations are thus required for this structure. At the time of writing, all items have been checked to ensure that they are common in Baluchistan like brick sizes, diameters of reinforcement, cube strength of concrete, structural steel member sizes, and even the tiles that are used between I-beams in the roof structure. However, it is recognised that some items may vary in size or availability in different parts of the country making small amendments necessary. It is envisaged that these will be undertaken by the respective agencies on the *original* mylar master drawings. The associated Bills of Quantities MUST be amended at the same time.

6.4 **Ground Level Tank**; This has been completely covered in Section 2.9 and will not be repeated. The reader is thus referred to those Sections.

6.5 **Pipelines**: The reader is referred to Sections 2.11 and 2.13 where pipeline design is covered.

6.6 **Street Taps and Community Tanks**: Street taps and community tanks are dealt with in Section 2.12.



A SURFACE-WATER BASED WATER SUPPLY SYSTEM

7. PARTICULARS RELATING TO SURFACE-WATERBASED SCHEMES

This Section examines aspects of the design and construction system which relate only to surface-water based schemes.

7.1 **Source Yield, Reliability and Seasonality**: The Indus Basin contains the largest irrigation project in the world. Any domestic water supply based on the water from the Indus is thus likely to be very reliable indeed; and yet this is not so because water supply schemes are often based on canals conveying the water to the agricultural areas. These require to be cleaned periodically, and the date and length of period of cleaning is not well established. This is particularly important since, if one wishes to design a pond to carry over between when the canal is closed and when it is reopened again, this period must be known very accurately. The Irrigation Department has verbally quoted figures ranging from 15 to 45 days. Obviously, the pond based on a closure period of 15 days will be one third the size of a pond based on a 45 day period. In reality, it was discovered that no canal is closed for more than 21 days because of pressure to have water in it again. Communities alongside the canal depend upon it for their domestic water requirements.

7.2 **Pre-Sedimentation Tank**: The design of the pond (pre-sedimentation tank) which holds water before slow sand filtration, is based on a 30 day retention period. Freeboard is allowed at 1.5 metres on all basins. All basins are designed to provide a balance between cut and fill. Factors are allowed for seepage and evaporation leaving the full volume required for "policy" yield for the present population. Evaporation losses are based on daily evaporation figures for winter when the canals are closed; and seepage based on clays/ loams where the water table is high. Reference has been made to the Irrigation Department Pat Feeder Canal PC-1 Document in this regard. In addition, siltation of the basin will take place over time which will require cleaning; but for those years between cleanings, an allowance has been made. It is not possible to obtain accurate data from any source regarding the silt loadings carried by the canals at different times of the year. However, all factors are conservative. First, the closure period is over-estimated; next, communities generally take of the order of 13 lpcd while the policy is to provide 45.4 lpcd; and all factors are conservative. Together, this implies an "over-estimate" of some 200% to 300%.

This is one civil structure which lends itself to staging. If the community does increase in size; and if it, at the same time, demands up to the policy volume for which the design is made, then it is perfectly possible for another basin to be constructed in the future. However, experience tells us that this is most unlikely, at least for the next 10 to 15 years. Sufficient space should be available to allow for the furture expansion if it becomes necessary.

There will be times when these designs are being used (such as in Sind for instance) when the incoming water might carry a very heavy silt load. This would be when water is taken from the main river during the snowmelt period. For such cases, horizontal roughing filtration may be a useful solution; and this is dealt with in Section 8.1.

Water is placed in the pre-sedimentation tanks via floating inlets. This is to avoid scour on the tank walls. The retention period will of course depend on the demand but is not less than 15 days. Calculations show that with anticipated silt loadings, a retention of only one or two days will suffice; for the silt particles after that time will only be colloidal and will remain in suspension for removal by slow sand filtration. The retention is purely to cater for canal closure periods.

The intake from the canal to provide the water to the pre-sedimentation tanks is so designed to be self cleansing. The gate and floor slope of the intake attest to this. The flow in the pipes between the canal and the tank(s) provides a flow velocity higher than that in the canal. Silt carried at the lower velocity in the canal will obviously be carried by the faster moving water in the pipes. Any modifications in the future should take this into account.

The outlet from the pre-sedimentation tanks to the slow sand filters is by an identical arrangement to the inlets. This allows for water to be skimmed from the upper layers in the tank, since lower layers can be oxygen starved and carry heavier silt loads.

7.3 **Slow Sand Filters**: The water from the pre-sedimentation tanks is passed to the slow sand filters via a small centrifugal pump. As the slow sand filters require a fixed, small flow, a suitable arrangement has been incorporated into the structures to allow excess water to run back to the tanks. A good operator will be able quite accurately to adjust the flow with valves using the flow meter before the slow sand filter (SSF) to obtain no more than is required.

The sand (media) in the SSF must have an Effective Size d_{10} of 0.15 to 0.30 mm; a Uniformity Coefficient $d_{60}/d_{10} \leqslant 3.0$; the maximum particle size should be $\leqslant 3$ mm; and the minimum size $\geqslant 0.1$ mm. These specifications have been derived from extensive experience in the field, and may be found in the standard texts. Variation outside these specifications *will* make the SSF malfunction, so it is important to insist on the strict adherence to the specifications. Satisfactory filtration rates are in the range 0.1 to 0.2 m^3/hour per m^3 of filter media. For design purposes, the rate is taken as 0.1 m^3/hour to allow for periodic cleansing of one of the units; at which time the flow rate would double on the other, but still be in the acceptable range. SSFs work best under continuous conditions. Thus, for design purposes, a 24 hour regime has been used; but this will provide, in the beginning, 250% more water than the community is likely to want to use.

While the filter units are designed to cater for the 45.4 lpcd for the existing population, this is already some 250% over-designed and allows for considerable flexibility. Two units will always be built as a minimum; which allows for the cleaning of one while the other serves the community. One unit by itself is still around 75% more in capacity (at the lower filtration rate) than is presently utilised by the communities. Increases in community size may be catered for by adding one or more units as necessary.

Details of aprons around the units are not examined in this Manual since they are purely to allow for cleaning the filters. However, it should be noted that training for operation is crucial to the successful functioning of the filter units at which time the aprons will be necessary. Also, the procedure to clean filters is not examined in the Manual since it belongs in an operation manual. However, it should be noted that by covering the top sand layer of the SSF with a suitable geotextile, it *may* be possible greatly to simplify the cleaning procedure. This is because the biological filter should build on the geotextile rather than in the top layer of sand. Cleaning the filter may thus involve simply cleaning the geotextile which should be considerably easier than cleaning the sand. This is presently being tested; and results are awaited.

From the SSF, the water flows by gravity to a partially sunken holding tank which is called the clear well.

7.4 **The clear well**: This is, as noted, partially sunk to allow the water to enter under gravity. As such, there are structural problems associated with the structure. When it is empty and there is flooding in the area, there is some risk that it will try to float. This has been addressed in the design; but it is felt that a large tank of this type is a danger and should be avoided if possible.

The performance of the clear well is identical to the GLT. This means that it will be provided with two centrifugal pumps and be operated in much the same fashion as for the GLT. However, the designer will have seen that the inflow will be somewhat lower than for the ground-water based schemes. What this means is that—theoretically at least—the clear well should have a volume of 50% of the "policy" 45.4 lpcd; or around 22.7 litres per person if the incoming flow can cater for that amount. It cannot. The system is operated generally early in the morning and late in the afternoon, implying that inflow between morning and afternoon operation is less than 12 hours. At the same time, it is well known that the community will only take about 30% of what the Government has determined must be available if demanded.

The above notes are confusing; but this reflects how communities behave. They do not behave mathematically. Thus, to simplify how the system will be operated, it has been decided that the clear well will be sized to store up to 25% of the daily "policy" requirement per person (45.4/4 litres per person); and that the balance—if required—will be provided through community tanks. This also means that the clear well pumps must be operated rather longer than they would be for ground-water based schemes. The precise timing of operation of the pumps at the clear well will obviously be dictated by the community. The following table indicates the sizing of clear well associated with what population.

TABLE 7.1

Present Population persons	Clear Well m^3
0 – 1,000	11
1,001 – 2,000	23
2,001 – 3,000	34
3,001 – 4,000	45
4,001 – 6,000	68
6,001 – 9,000	91

Staging of the system will take place by adding community tanks at the required locations and by operating the system for longer hours.

7.5 **Centrifugal Pumps**: The centrifugal pumps on the clear wells will be designed in exactly the same way as for those on the GLTs; namely that the head and flow requirements will be taken from the computer design of the pipelines. See Section 2.10.

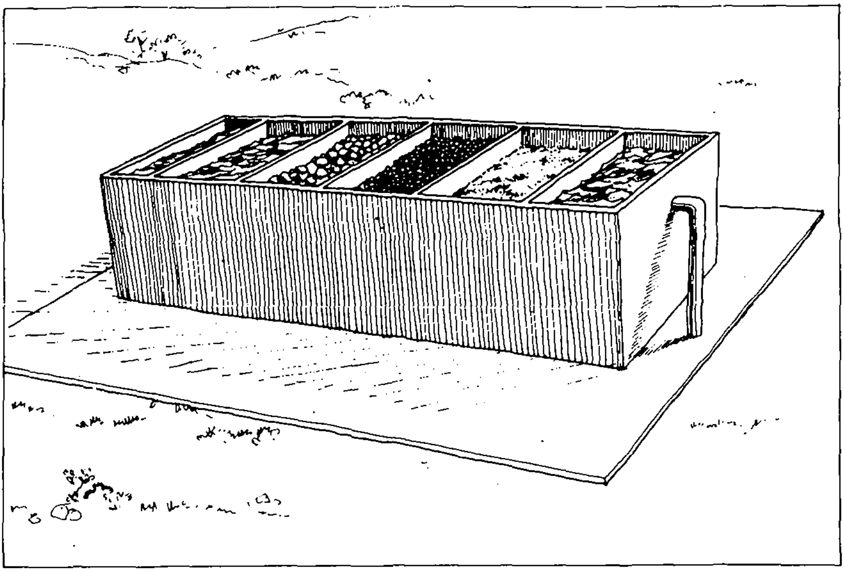
7.6 **Pipelines**: The pipeline design is identical to that for the ground-water based schemes. Please refer to Sections 2.11 and 2.13.

7.7 **Street Taps and Community Tanks**: The conditions pertaining to street taps and community tanks may be seen in a number of earlier sections, and will not be repeated.

8. SUNDRY NOTES ON SPECIAL FEATURES

This section examines certain features that are sometimes associated with water supply systems, and include horizontal roughing filters (HRFs), infiltration galleries, spring development schemes and handpumps.

8.1 **Horizontal roughing filters**: (HRFs): HRFs are a quite recent development in civil engineering techniques to improve the quality of water. They appear to be most successful in removing heavy silt loads from incoming water, but they do NOT replace slow sand filters. Rather, they should be used together with slow sand filters.



A HORIZONTAL ROUGHING FILTER

HRFs are used after water has been held in a holding basin for one day to remove the heaviest particles, and the water from the HRF then passes to slow sand filters to provide the final "polish" to the water before delivery to the communities from the clear well. They are most effective in lightening the load placed on the slow sand filters. They allow—by comparison with slow sand filters—a quite rapid flow for a large reduction in silt content of the water. The HRF separates the silt particles from the clear water, passing the silt back to the holding tanks or, better still, back to the river or canal.

Naturally, if the source for the community is to be from a canal where there is a canal closure period allowing water to stand in the pre-sedimentation tanks for some 15 days, there is no need to place an HRF, because the presedimentation process will remove all but the colloidal particles in 15 days. Thus, the HRF would be useful where the source is a river.

Where the HRF is to be used, two HRFs would be placed with two holding tanks and two slow sand filters to allow for periodic cleaning. Staging would be to add one unit of each as appropriate. Naturally, space must be available for future expansion.

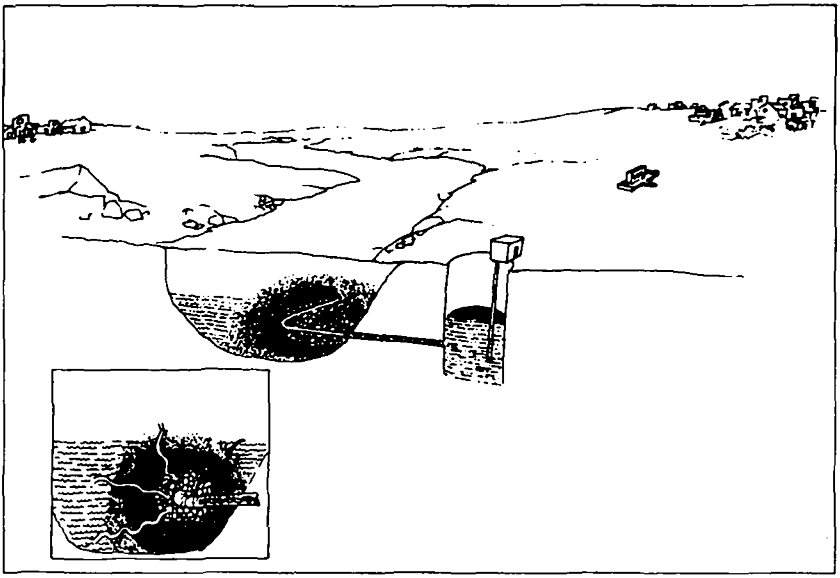
Design parameters include that the HRF is sized to provide for the 45.4 lpcd for the present population; that incoming turbidity should not exceed 100 mg/1 of suspended solids; and that the flow velocity should be in the range 0.5 to 1.5 metres/hour. These design parameters have been taken from the IRC publication listed in the bibliography; and it is suggested that when HRFs are placed, the designer should study these publications.

The noted publications indicate that there are a number of construction details which are crucial to the full functioning of the HRF, such as underdrains. There are, in addition, a number of operation details which show the need to train operators with great care and to ensure that such operators are well managed. Without the correct maintenance, the HRF will not provide the service which it can. This is also true for the management and maintenance of slow sand filters.

8.2 **Infiltration Galleries**: Infiltration galleries collect water from a water bearing layer and bring it to a wet well from which water may be pumped directly to communities. The water bearing layers may be in river beds (the majority), or they may be near a water body such as a *tarai* (a crude but effective holding pond which collects runoff in parts of Sind). The gallery is usually designed to effect the filtration so that none is required after the wet well.

The Modules will show that the water is collected in a perforated pipe which is wrapped in a geotextile. The open area available gives a flow suitable for a population of up to 4,000 people—providing that a sufficient head is available. This head is almost impossible to define. It will depend upon the ease with which the water can pass through the material from which the water is being collected. If it is evenly graded coarse river sand, the head will be very low. If, on the other hand, it is a "tight" clay, the necessary head will be very high as water passes very slowly through such material. Variations may be accommodated by increasing the length of the collector pipe.

Simply by placing a pipe in a river bed is no guarantee that water will be available, even if it was available at the time of original inspection. The sub-surface flow in a river is seasonal and is as variable as the rainfall. If the water supply is to be secure, so must be the sub-surface flow. It is thus necessary for the designer to obtain data to demonstrate the flood-frequency, rainfall, runoff and the like. What will be seen is that there will be times when there is inadequate water available even for a small community. This may only be once in 10, 25 or 50 years. The community should be so informed. Pump tests on the source in the absence of these data can be very misleading, although pump tests should be carried out.

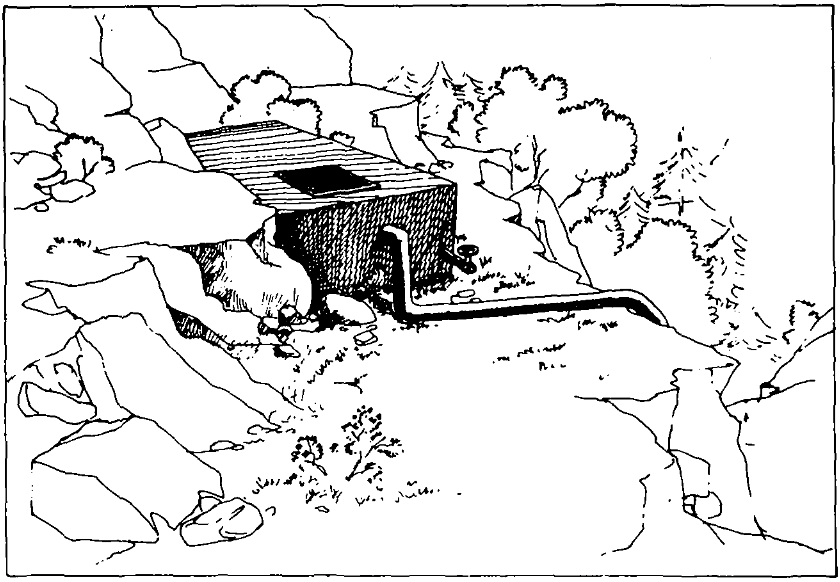


AN INFILTRATION GALLERY (SECTIONAL VIEW)

An example of where water has "run out" is the Akra Khor Dam near Gwadar on the Makran Coast. Communities taking water from this dam have had to travel long distances to fetch water because the dam dried up for lack of inflow. An examination of the flood-frequency in the river might have prepared the community for what was seen as a catastrophic event. Another area near this dam is served by an infiltration gallery where the source dried up at the same time. This would indicate that it was a dry year when both rainfall and runoff were low. The designer is referred to the hydrology texts which deal with how to determine flood frequency data.

8.3 **Spring Development**: There are, particularly in the mountainous regions of the country, many springs which can be used for a domestic water supply. Indeed, in Azad Jammu and Kashmir, one of the main water sources is springs.

The Manual does not set out to describe how spring development schemes should be handled since there are a number of excellent publications which do it well. Thomas Jordan's Handbook on Gravity Water Systems deals comprehensively with the subject. However, the Modules present both an example of spring capping and a break-pressure tank.



AN EXAMPLE OF SPRING CAPPING

The spring capping is to provide some protection to the source. It not only collects water and puts it into the pipe delivering to the community, but it provides an overflow and a system where small particles are allowed to settle out, and which may be flushed periodically from the system by a valve. It is important to note that this structure should be securely "keyed" into the water course to stop it running away downhill when water flows in the stream.

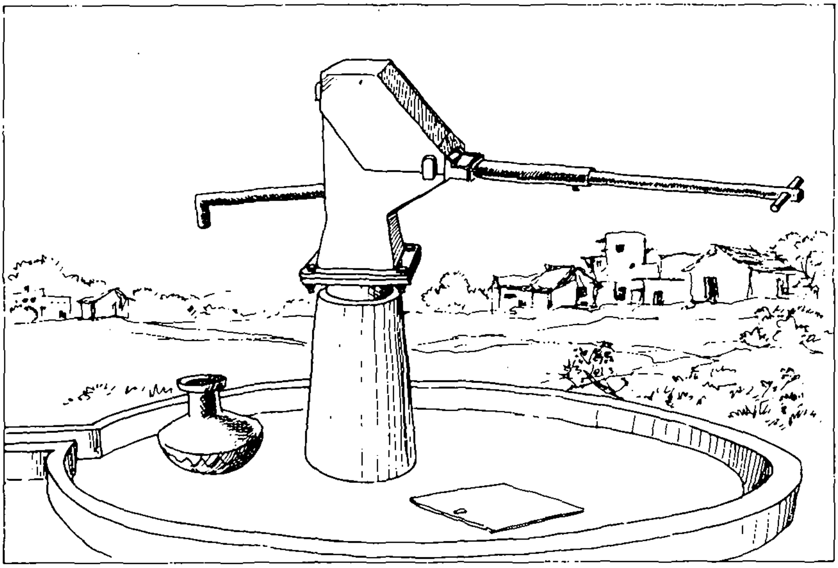
From the spring cap, water is taken by pipe downhill. Depending on whether the area suffers frosts, so the pipe may be led overground or should be buried or lagged to avoid freezing. Suitable pipe types include GI and HDPE (High Density Poly-Ethylene). What is important to note is that the pressure in the pipes should not rise above the working pressure rating of the pipe (including surge pressures). Break pressure tanks should be provided each time the pressure reaches the limit.

The break pressure tanks have a device on them to ensure slow and positive closing of the valve when the tank is full. Simple ball type cistern valves *should never be used* in these tanks without the arrangement shown on the Module drawing. This is because such valves can open and close in response to waves in the open tank; and such cycles of opening and closing rapidly are the cause of surge pressures which tend to fracture pipes. The reader may consult Parmakian's Waterhammer Analysis for details of how surge pressures are caused, and their magnitude. Surge is a problem mainly in long pipe runs where water velocities are high—such as where the pressure is near the pipe pressure rating.

At the time of writing, no other structures are presented in the Modules although a storage tank probably should be. For those wishing to use the Modules for a storage tank in a pure gravity system, they should employ the 11 m^3 GLT without the centrifugal pumps. Tap stands and community tanks would be essentially the same as in the Modules, with modifications to suit the water collection vessels. Please note that the Second Edition of the Modules and Manual are likely to include considerably more detail on spring development (gravity fed) schemes. They may even be dealt with separately.

8.4 **Handpumps**: There are some people who believe that handpumps are the most important community water supply system in the world. This may well be true as it has been estimated that about 2 billion (2,000,000,000) people will eventually receive their water supply through handpumps. On the other hand there are some people who believe that since Agricola (De Re Metallica, 1556) reported some early handpumps used in mining dewatering operations, handpumps are now considered to be "obsolete".

It is true that handpumps may not have been refined much until around the middle of the twentieth century, but since that time, there has been a revolution in handpump design. Today, handpumps can pump efficiently from 50 metres (150 feet). Today, some of them use plastic bearings which can be replaced in a matter of seconds, and which easily outperform even robust roller bearings. Today, handpumps are designed which can be maintained with ease even by women using very simple tools indeed. The design effort which has gone into these refinements is equivalent to that which has sent men to the moon.



THE AFRIDEV-TYPE HANDPUMP
Sometimes called the "INDUS" handpump

The World Bank has recently produced the definitive publication on Handpumps called "Community Water Supply: The Handpump Option". The reader is referred to that publication. It covers nearly all aspects of relevance to the designer.

Several points arise from this. First, it is NOT necessary for any handpump to be "developed", for there are probably too many available already. The annexes to "The Handpump Option" demonstrate this fact very well. Secondly, that despite there being so many handpumps available, very few provide the performance and reliability necessary to make them a viable option. Thirdly, as spare parts are necessary for *any* mechanism, and as handpumps will be used in other parts of the country (for community use), it makes good logistic sense to use the *SAME* handpump throughout Pakistan. The type which will be used is the Afridev (called the "Indus" locally), which has a proven record and is aready being made in Pakistan. Thus spares will be *locally* available.

Putting handpumps on machine drilled boreholes in a country where there are already hundreds of thousands of dug wells does not make sense. As the Government programme for the supply and installation of handpumps cannot hope to provide "full coverage", it should be viewed as one of demonstration to show what community handpumps can do. When communities see what the handpumps can do, and that they can provide a highly reliable service, then they will be inclined to purchase their own. Accordingly, communities will need to understand what their investment will be—which must include the well or borehole. As boreholes cost lacs of rupees and dug wells cost only the labour to make them, the Government programme will only provide handpumps for installation on existing dug wells, or, in some instances, hand drilled boreholes.

Herein lies a problem. Government will obviously seek existing dug wells to ensure that the programme moves forward quickly. However, existing dug wells are almost exclusively privately owned. How can one place a "public" handpump on a "private" well? Quite obviously, it will be necessary to negotiate with the present owner to obtain his agreement to put the well into public ownership. This process of negotiation will take time and staff; but MUST end with the written agreement of the original owner that the well will now pass into public ownership. Without this written agreement, the handpump should never be placed.

Returning to the handpump and its use: While Government may require 45.4 lpcd, water collectors will normally only take as much as they are willing to pump (or carry). As a human can only produce power in the range 50–200 watts; then if pumping is from 50 m below ground level, the yield will be between 0.08 and 0.3 l/s, or ¼–1 m^3/h—enough to satisfy Government policy for only 6 to 24 persons per hour. However, this should be compared with the volumes taken by bucket from the same well.

As yield is proportional to lift and power input, the following parameters should be used to determine the normal arrangement:

TABLE 8.1

Depth to Water Table (mtrs)	User Group Size for 10 hours pumping	
	2 pumps/well (Persons)	1 pump/well (Persons)
10	1500	750
15	900	450
30	450	225
50	300	150

A check should be made that the well recovers sufficiently quickly where larger user groups are supplied.

As the community grows (if it does), more handpumps may be installed. However, please note that the handpumps are expected to perform so well that communities will wish to purchase and install their own. THIS IS TO BE ENCOURAGED.

All operation *and* maintenance will fall to the community. The pump is designed for just this eventuality, and comes with the complete necessary tool kit – one spanner and one extension piece!

However, please note that it is the responsibility of Government (BIAD/LGRD/PHED/SAZDA etc.) to train recipient communities in how to maintain their pumps. This will normally be done at the time that the pump is installed. The person assigned the task of looking after the pump is called the "Caretaker" and, because the task is so easy, "Caretakers" may be selected from among the women in the community, provided always that this is acceptable to the community as a whole. Besides, handpumps tend to malfunction (even if this is a rare event) during use. Water is normally collected by women, often when the men are at work away from the community. Must the women always wait for water until their menfolk arrive back?

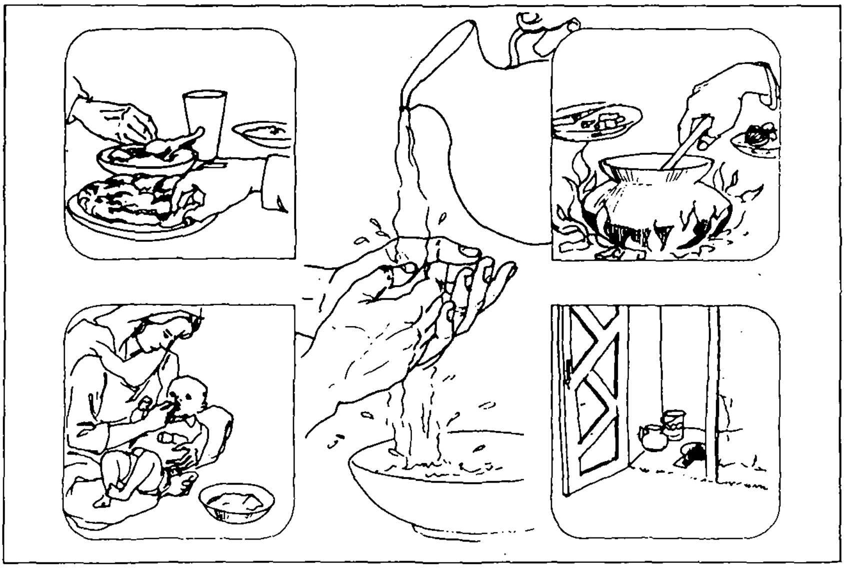
From the record of the Indus pump to date, it has not been found necessary to have an elaborate major breakdown backup system. However, for the rare occasions when a major breakdown occurs, the same system which is used to install the pumps in the first place, should be used. However, it is not feasible for instance, to have a mobile workshop for such eventualities in Baluchistan, and communities should be encouraged to seek the services of local mechanics for necessary repairs.

9. HEALTH AND ENGINEERING DESIGN

It may come as some surprise to engineers to see a Section in the Manual – however short it is – about health. They might wish to recall the title of the major water supply agency in the country. It is called the *Public Health Engineering Department*. As such, engineers should at least be aware of the health aspects of the programme.

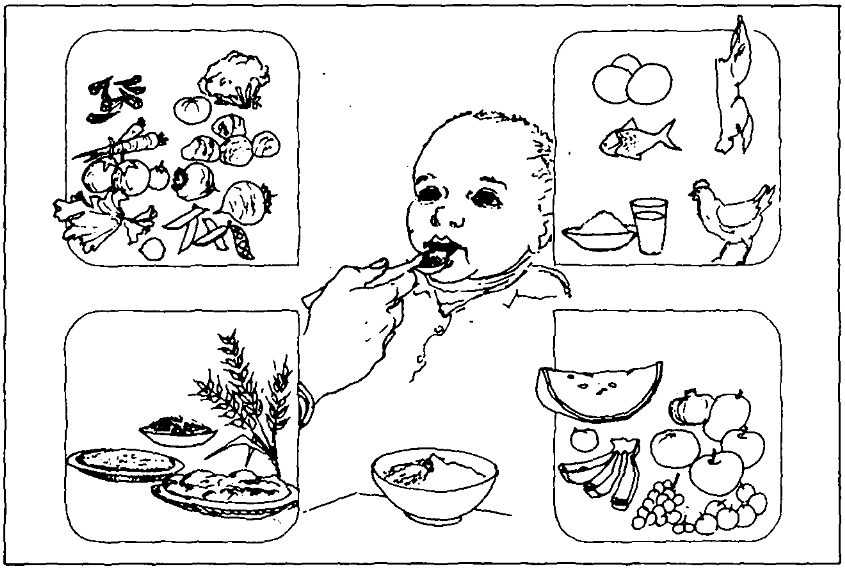
9.1 **Water and Health**: Considerable work has been done recently to show the public health connections between water and diarrhoea. Prior to this work, many people believed that all diarrhoeas were water borne. This is not so. Indeed, a surprise to many is that perhaps the major killer diarrhoea – rotavirus – is also vapour borne. This means that the major transmission route may be when one child speaks to another, where the virus is carried on the vapour from the lungs. This also means that improved water supplies and good sanitation may do little to prevent this diarrhoea.

Nevertheless, there are a number of diarrhoeas which do rely on water for their transmission. Perhaps of far greater importance is the fact that cleanliness can substantially reduce the risk of transmission of a number of diarrhoeas by hand-to-hand contact. *But this only happens if the water is used correctly.*



PERSONAL HYGIENE: HANDWASHING
Washing with soap can reduce shigella
hospital admission rates by upto 80%

Another aspect which must be mentioned is that well nourished children generally do not die from diarrhoea. They have sufficient body reserves to combat the disease. On the other hand, children who are malnourished are more likely to die. The worse the malnutrition, the higher the risk of death. And, very surprisingly, poor quality water can be a cause of *malnutrition*. If a child suffers bout after bout of diarrhoea, he can quickly lose weight and become malnourished. Good hygiene undoubtedly reduces the risk of many types of diarrhoea. However, simply by having a good water supply does not mean that people will automatically use it.



GOOD NUTRITION IS THE BEST PREVENTION
AGAINST DEATH FROM DIARRHOEA

It is thus very important that designers give thought as to how they can encourage communities to want to use the water that their designs are providing. For instance, people will not be inclined to wash their clothes regularly if the only place where they can do so is in a mud puddle. People will probably not be inclined to collect water in larger quantities (for hygiene purposes) if the place where they collect the water is poorly drained and messy. They will become frustrated if the flow from the tap is lower than it should be. They will waste water if the tap is not at the correct height for the particular water container that they are using. And the only way in which these design "deficiencies" may be overcome is if the designer studies the communities he is serving. Better still, if the designer takes the time and trouble to talk with community members to find out *what they want and need*, only then is there a likelihood of designs meeting the needs; of designs encouraging people to use the water that is provided; and, consequently, the standard of personal hygiene going up with lower rates of diarrhoea.

PERSONAL HYGIENE: WASHING OF CLOTHES
Designs should take into account the
practical needs of water users

9.2 **Sanitation and Health**: Very many diarrhoeas are associated with faeces. In theory, one could make a substantial reduction in diarrhoea rates if all faeces were separated from people because the source of most of the diarrhoeas would be cut off. In practice, this is not possible. Traces of faeces find their way into the environment, such as on the hands of people who do not clean themselves adequately after defecation. Nevertheless, it is essential that as much excreta as possible is removed from the immediate environment or the gains that might be made from improving the water supply will be completely lost. Hence the need for sanitation.

So important are the health aspects of sanitation that they are noted in some detail in the following sections on latrines.

9.3 **Communicating Health Information**: Designers will have noted that there is increasing emphasis on "health education". At the same time, the authorities who use these "buzz words" rarely tell one exactly what is meant by "health education".

"Health Education" means giving people the information they need to look after their own health. Ideally, it also means encouraging them to act on that information.

Traditionally, "health education" has been seen as a process where an engineer or a health educator stands up in front of an audience and lectures to them about the disease cycle. Then he goes home satisfied that his audience has been "health educated". His audience is usually so polite that they never ask any questions because they simply do not understand what he was talking about. They go home with a warm feeling that they have been "health educated", eat their food and feed their children with dirty hands, and wonder why it is that their children fall ill next morning with diarrhoea. After all, they have been "health educated" – this shouldn't happen, should it?

This example may seem a little extreme but too often reflects actually what is going on where health education is even attempted. Today, however, it is becoming apparent that to provide people with the information which they need to look after their own health, one needs to speak to them in a language which they can understand. What use is it for instance, telling a mother that a virus will kill her child if she doesn't know what the virus looks like, nor where it may be found? Even if it were possible, putting her in front of an electron microscope to see the virus would still leave her perplexed. She would be unable to understand the magnification; and even if she could, she would probably ridicule the fact that something so small could kill her child.

Many communities are essentially illiterate. For instance, it has been estimated that in Baluchistan, less than 1% of all adult females are functionally illiterate. True, some 3% to 4% may be able to sign their names, but they cannot read. Is it thus feasible to provide them with written information about what they can do for their own health? Obviously, pictures and the spoken word are necessary. Television is not very helpful since it reaches only those in the Quetta valley where most of the literate women in the province live. Obviously, pictures are needed for the health educators to support their spoken message.

Sadly, the problem becomes even more complicated. Even if agencies had health educators in sufficient numbers, the information conveyed by pictures is not always readily understood. This is particularly true for illiterate societies. Where engineers may readily understand a drawing (which is in two dimensions) and be able to imagine what the item looks like in three dimensions, illiterate women would only see an incomprehensible clutter of lines. The science of conveying information using pictures is, sadly, highly specialised. However, some help is at hand.

In Peshawar, there is an Organisation which specialises in the production of health education materials. It is called HERC, or the Health Education Resource Centre. This Organisation is presently developing a wide range of pictorial health education materials. The difference between this body and almost all others is that they are testing each picture with intended users to ensure that the picture(s) convey(s) what is intended. But even they face some problems. For those pictures which are being developed with some written message, which language should be used for the message?

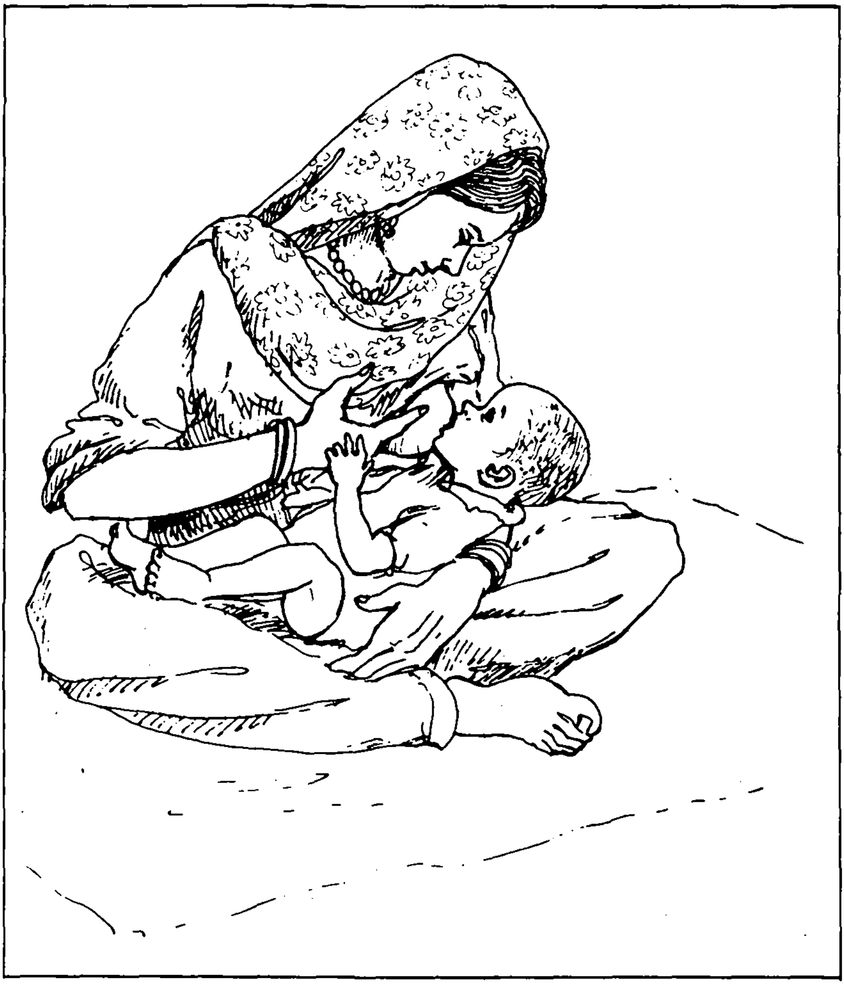
As may be seen, "health education" is not an easy business; but without it, can the designer expect that his designs are going to provide the health benefits that are supposed to come with improved water and sanitation?

9.4 **Diarrhoeal Disease Control**: The frequency with which diarrhoea occurs after a water supply and latrines have been placed in a community will be *identical* to the frequency before. The "hardware" of water systems and latrines appear to play no individual part in the reduction of diarrhoeal morbidity nor mortality, unless hygiene behaviour of the users changes substantially at the same time. Thus, if the objective is to reduce diarrhoeal morbidity and/or diarrhoeal mortality, a new water supply and a latrine programme by themselves will not be useful. Indeed, it will be a waste of money, and lead to frustration. Paying lip-service to "health education" will also be a waste of money. If the objective is to control diarrhoea, then piped water, latrines and "health education" will not do it.

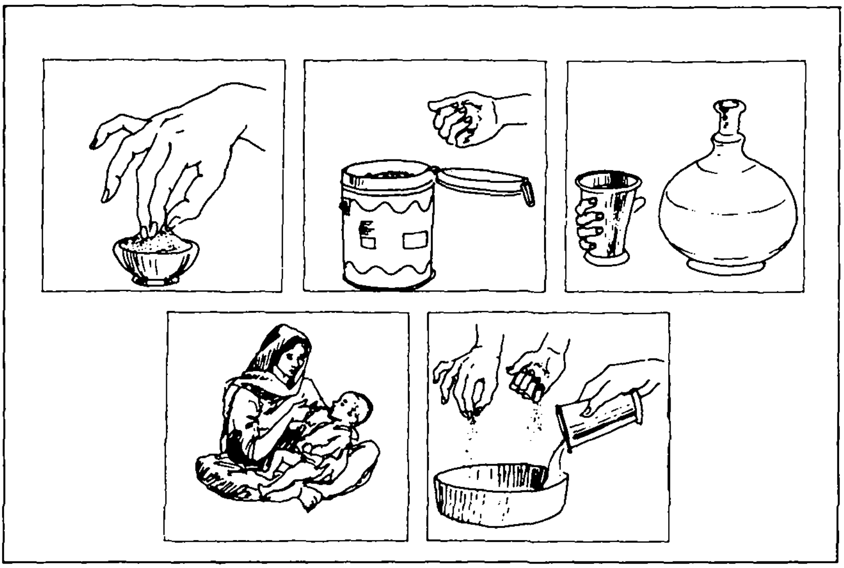
The whole gamut of control measures must be applied together. These will include:

Water, sanitation and hygiene education;
Soap for handwashing;
Nutrition for the mother to avoid low birth-weight;
Nutrition of children, particularly females;
Food hygiene;
Breastfeeding;
Vaccines (particularly measles);
Oral rehydration therapy; and
Chemotherapy with IV as necessary.

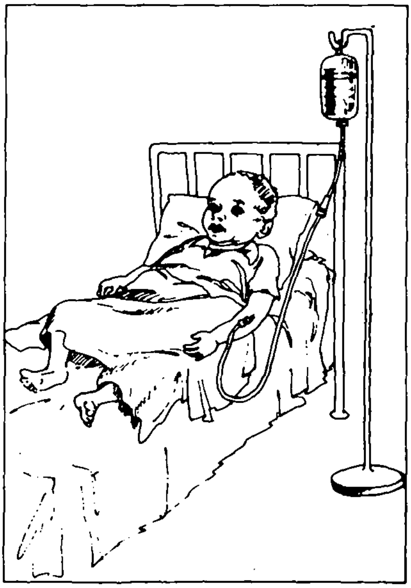
Unless connections are made between these various items which together control diarrhoea, their cumulative effect is unlikely to be measurable. Thus health workers from different disciplines need to make those connections for the communities they serve. Engineers and sanitarians need to talk with LHVs. Dais must be briefed and monitored. Doctors need to talk to peripheral health workers; and common problems solved. Planners need to monitor progress and adapt to changing conditions.



BREASTFEEDING GIVES EARLY IMMUNITY
Statistically, bottle-fed babies
have a higher mortality rate



ORAL REHYDRATION THERAPY:
EFFECTIVE FOR ALL DIARRHOEAS
If the objective is the control of diarrhoea, then ORT
must be used alongside water supply and sanitation
and the other measures known to be effective



INTRAVENOUS (IV) THERAPY
Outdated and dangerous
It has many dangers from dirty
needles. Doctors are more and
more returning to ORT as the
best "solution" to dehydration

SANITATION

It is a well established fact that the public health gains obtained by providing an improved water supply can be almost totally lost if the same attention is not paid to excreta disposal and personal hygiene.

The technology of excreta disposal using simple latrines is relatively straight-forward; but it appears that very few successful latrine programmes exist in this area. This may be from a lack of attention to how people behave.

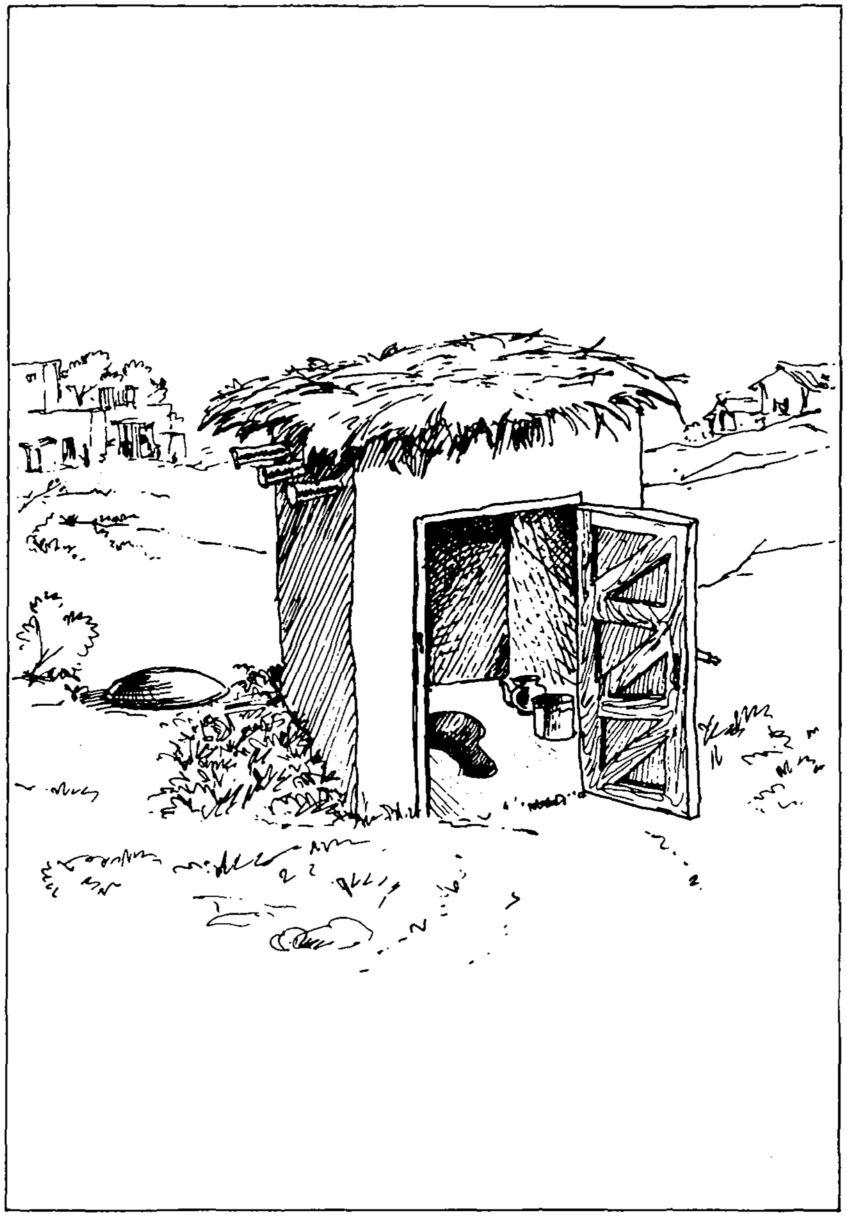
Thus, while this document sets out to be a Design Manual, this section on latrines focuses not only on design, but sociology, health and programme objectives related to latrines.

latrine visit, then some six to eight litres per person per day is required. When this is placed against water requirements for other activities (drinking, cooking, washing and the like), it may constitute an increase of 20% to 33% over what is used already. Where water haul distances are reasonable, this latrine type may thus be appropriate.

Against the VIP type (see Section 11), this latrine suffers far fewer disadvantages. The latrine can be oriented in any direction. The superstructure does not require a roof, if a roof is not wanted. No problems exist with pvc vent pipes degrading in the sun. There is no smell if it is flushed. Flies very rarely are a nuisance. And, it costs just as little. Providing users know about, accept and use the water required (which is little enough), then it provides a very much more satisfactory arrangement than the VIP. It also is unlikely ever to collapse, for the pan is not sited over the pit.

10.1.1 *The Superstructure* can be made from a variety of materials. Generally, it is made from clay—either built up gradually or using unbaked bricks which are finished with a clay and straw mortar. A roof is desirable (but not absolutely essential as is the case with the VIP). The roof can be made from corrugated iron, wood and clay, or simply domed brick/clay. The superstructure should be large enough so that one can defecate/urinate in comfort; yet should also provide privacy during the process.

10.1.2 *The Pan and Water Requirement* have already been mentioned. The pan is designed to accommodate not only adults but also small children by the keyhole form giving a narrow section suitable for small people. The pan has a lip to stop urine-splash. Earlier models did not have this lip, and were wider and shallower. Urine-splash became a major problem leading to non-utilisation of the majority of latrines using such a pan. Besides, Muslims are enjoined when splashed by urine or excreta, to bath before praying; and as *namaz* (prayers) occur five times a day, this can cause adverse reactions as have been seen, particularly where bathing requires quite a large volume of water. The pan is very carefully laid out geometrically to give an exit pipe on a slope of around 30°. This allows for flushing water to leave the pan with very little turbulence, making for efficient flushing with a small volume of water. Earlier models had a vertical exit pipe; leading to a P-trap, where considerable turbulence occurred because of the geometry. This meant that some four litres were required per flush—a volume acceptable where running water is available, but less acceptable where water has to be hauled. Such pans required some 20 to 25 litres per person per day for satisfactory flushing.



THE TWIN INDIRECT-PIT, POUR-FLUSH LATRINE

10. SANITATION: THE POUR-FLUSH LATRINE (P.F.)

The pour-flush latrine is perhaps the best known of all simple latrines. It comes in many forms; with direct or indirect pit (or pits), with different pan types made with different materials, and with a variety of superstructures. Pits vary from the shallow small diameter to large deep ones. The design can be adapted to suit almost any environment.

These notes have been prepared to suggest what might be an appropriate type for Baluchistan which is an arid area. It may well be appropriate for other areas, too. The notes benefit from experience to date in the province, where the Baluchistan Integrated Area Development (BIAD) Programme has been responsible for placing many hundreds of latrines. Some of the earlier models suffered some technical and social deficiencies; and these are noted lest agencies active in placing latrines make the same mistakes.

10.1 **Technology of the pour-flush latrine**: The immediate reaction to a suggestion that a water seal (pour flush) latrine should be used in a semi desert area is that there is inadequate water to allow it to function properly. In fact, this is rarely the case.

People cannot survive without water. If a community exists, this implies that an adequate quantity of water is available. True, the quality may be poor. True, water collectors may have to travel long distances and have to wait while a trickle fills their containers, but without the security of an "adequate" supply, the community either dies of thirst or moves to where sufficient water does exist.

This begs the question as to what constitutes a supply adequate for survival. While governments and individuals will strive to define this "adequate" amount, no two groups appear able to agree. What can be said is that the supply will always be greater than the minimum to sustain life; and usually is a function of how much value is placed on it – which will determine how hard water collectors are willing to work to obtain their "requirement".

The pour-flush latrine requires water for flushing. Depending on the design of the pan, one flush may require between 1 and 4 litres of water. Obviously, in an arid area, the lower amount is appropriate; which is why the pan design is crucial. If the design is correct, there is every chance the latrine will work satisfactorily – but if it requires a large amount of water for flushing each time, people will not be inclined to use it properly. Assuming that only one litre is required per flush, and that one litre is required for ablution per

Some development extension workers have expressed concern at the "greatly" increased cost of latrines employing pans, where the pans are ceramic. In Pakistan, the pan which is described here can be purchased for around Rs. 100 (less than $6) per unit. This can hardly be deemed to be a large increase in cost where the cost of the whole latrine may vary between Rs. 600 and Rs. 1,500 depending on how the various parts are valued, and depending on the standard of the latrine.

10.1.3 *The Slab* is usually made from a lean mortar mix, and is so arranged that water poured onto any part of it flows into the pan to allow for easy washing down of the area surrounding the pan. This slab may (or may not) have raised foot pieces cast into it at the appropriate places, depending on the wishes of the user. The slab/pan arrangement is always raised above natural ground level to avoid any flooding during rain. The slab is only ever laid after proper compaction of the underlying clay/sand, to avoid cracking later.

10.1.4 *The S-bend* is fixed onto the exit pipe from the pan; and provides a water seal which excludes flies and odours. Usually, 4" dia pvc pipe laid at a slope of 1:50 to 1:20 is led away from the S-bend towards the Y-piece. Concrete pipes must not be used, for they are expensive, heavy to handle, and are rough inside. Flow is poor in such pipes.

10.1.5 *The Y-piece* is the place where the pipe bifurcates (splits into two 4" dia pipes). The top of the Y-piece is covered by a small, purpose built slab which can be removed and replaced when one of the pits is full, and switching of the direction of flow must take place. The leg of the Y-piece not being used is usually plugged with clay or a weak cement mortar.

10.1.6 *The Pits and Leaching*: Two small pits are used for storing the latrine effluent. They are used alternatively; when one pit is full, it is closed and left to compost for about one year; after which it is emptied. Because of an understandable reluctance for latrine owners to enter the pits to "sweep" them, they are so designed that they can be emptied by a person standing at ground level using a long handled spade such as is widely available in Baluchistan. It is noted that the composted excreta is odourless after six months, and has the appearance of peat.

Effluent entering a pit is a mix of faeces, urine and flushing water. The fluids are "leached" from the pit, which means that they must pass from the pit into the surrounding soils. Obviously, the walls of the pit must be arranged to allow these fluids to flow out of the pit. Gases generated in the pit are absorbed by the soil, too.

Depending on the soil type where the pit is excavated, the pit may or may not require to have supporting walls. In general, it is felt adviseable for this type of latrine to use a honeycombed brick lining because the pit requires to be covered. Almost any brick is suitable providing it is baked.

The cover—usually made from ferrocement—is light and inexpensive but surprisingly strong. Covers made from reinforced concrete are not recommended for they are extremely heavy and unnecessarily expensive. It is noted that the Baldia Soakpit Project in Karachi recently discovered the technique of ferrocement from Baluchistan and has now converted to making pit covers from this material, it is understood.

Some soils in Baluchistan are fossiliferous—or tightly bound clays. Leaching in such soils can be slow which is why a latrine pan using small amounts to flush is also preferred. Experience to date has shown that slow leaching proves no problem, however.

In some of the low lying areas of the province fed by canals from the Indus, general flooding can be a problem. This flooding in no way affects a pour flush latrine. When effluent enters the pit, the solids *always* drop to the bottom of the pit. Fresh faeces sometimes float which is a function of diet; but this is because of entrapped gases, which slowly dissolve, allowing the denser faeces to sink. Fluids in the pit cannot flow out through the pan, for water will be at the same level above the pan and pit (assuming flooding becomes this serious). As water can only flow downhill, there can be no flow between pit and pan. However, where grease or oil may find its way into pits, this could float out.

10.2 **Sociology of Latrines**: This Section examines some aspects of how people behave with latrines.

10.2.1 *Pan Performance*: Some of the sociological aspects of latrine use and behaviour are also referred to in the later section on the VIP Latrine. This section deals with aspects either peculiar to the pour-flush latrine, or which are omitted in the Section on the VIP latrine. It is noted that a pan, *per se*, is probably necessary if children are not to feel threatened by a dark hole through which they may fall. Sociological research in Bangladesh confirms this. For the pour-flush latrine where a pan is obligatory, it has been noted that the present design requires less water to flush satisfactorily than other somewhat similar pans. This is an important factor related to behaviour patterns in Baluchistan and elsewhere. Where water is not easy to secure or inaccessible in large quantities, if a pan is introduced which requires more water than can be comfortably arranged, the likelihood of obtaining a clean latrine is slight. As the objective of the use of the latrine is improved cleanliness and hygiene, a dirty latrine is a waste of everyone's time and money. Thus, on introduction of a pour-flush latrine, the intended recipients must have it made absolutely clear what the water requirements are, and what the rôle of the user is. This has been done very successfully so far in a number of areas already where BIAD (The Baluchistan Integrated Area Development Project) is promoting the model of latrine described here.

10.2.2 *Latrine Possession vs Use*: It is well known that for a programme of placing latrines to be successful, it is necessary for the recipients to place a value on their latrines. Sometimes, the status of owning a latrine can have the same effect. Some reports have been received of status being attached to having a VIP vent projecting above the walls of the compound, not unlike the status of having a TV antenna on the roof. However, pure possession of a latrine does not imply either use nor satisfactory use of the latrine. It may be useful to suggest for any latrine programme that a limited survey (which may even require limited observation) could indicate whether the objective of latrine use is being met; and if not, why not. A recent visit to the Chagai area of Baluchistan where some considerable numbers of latrines have been (and are still being) installed showed that despite these latrines, some people still defecated away from their latrines. Could this be because latrines were smelly, or so small as to be confining? Could it be because children perceive the hole or dark smelly latrine as a threat? As different groups behave differently, latrine needs and behaviour may vary between groups. Latrines require to meet those varying needs satisfactorily before they can hope to be successful.

10.2.3 *Placing of the Latrine*: Some groups require that men and women use separate toilet facilities. For men, the latrine may be sited outside the compound to provide for guests. For women *and children*, the latrine would be sited inside the compound. For the pour-flush latrine, orientation is not a technical problem; but correct siting is normally crucial if the latrine is to provide the privacy which women require *and* the "defensible space" needed for children. Children require, until they are fully independent – somewhat before or at pubery – security. In some societies, this security is being able to see or touch the mother or siblings. Unless the latrine can be sited to provide this security – possibly with a view of a familiar part of the compound – then it is unlikely that small children will use the latrine alone on a regular basis.

10.2.4 *Sweeping of Pits*: A later section notes that pits have to be emptied periodically. An average person will produce some 38–40 litres of dry compacted material in a pit per year. This material does not compost to a fluid–it remains solid and must be removed. In a Muslim society there is some reluctance to handle excreta from humans. The task of sweeping was, in earlier days, normally left to lower castes of other groups. Now that the area is substantially only Muslim, the opportunity to obtain the services of a sweeper is limited, and may be non-existent in many rural areas. Each user must arrange his own sweeping. This must be made known to intended recipients *before encouraging them to accept a latrine.* As the technique of composting the contents of a pit for around one year is used in this type of latrine; and as the material is odourless after such a period, this sweeping becomes feasible. Perhaps it would be useful to suggest that extension workers promoting latrines should, themselves, participate in emptying a composted pit early in the programme as a means of desensitising the community. The excavated material should be used as a fertiliser. In parts of India, a thriving-business has been developed buying effluent from pits for fertiliser. As the one year composting period results in the decay and destruction of *all* pathogens in the excreta, there are no public health problems associated with such a business.

10.2.5 *Two Latrines per Family*? Earlier, it was noted that some groups require separate facilities for men and women. Some do not. On what basis do development agencies agree to give two subsidised latrines per family in some areas, and only one in others? This can hardly be deemed to be an equitable distribution of resources. While no quick answer to this problem may exist, it may be helpful to suggest that if this problem does arise where the question is posed, the second latrine may possibly be required to be paid for at full cost. As these simple latrines are relatively inexpensive, this is probably feasible. After all, if a man wants to have two cars – one for himself and one for his wife – and if he obtains one subsidised by his firm for business purposes, can he really expect his firm or company to provide the second car free of charge?

10.2.6 *Ways to "Sell" Latrines*: Most public health extension workers when encouraging intended recipients to build a latrine, will stress the health aspects associated with it. Experience tends to suggest that this may not be the best way to go about encouraging people to participate in the programme. While the health aspects are the reason for the programme originally, people value latrines for reasons other than this. Thus it is probably better to "sell" participation based on what they value in the latrines. This will include;

- Absence of smell,
- Safety for children,
- Privacy,
- Convenience,
- Durability/safety of construction, and
- Absence of flies (?)

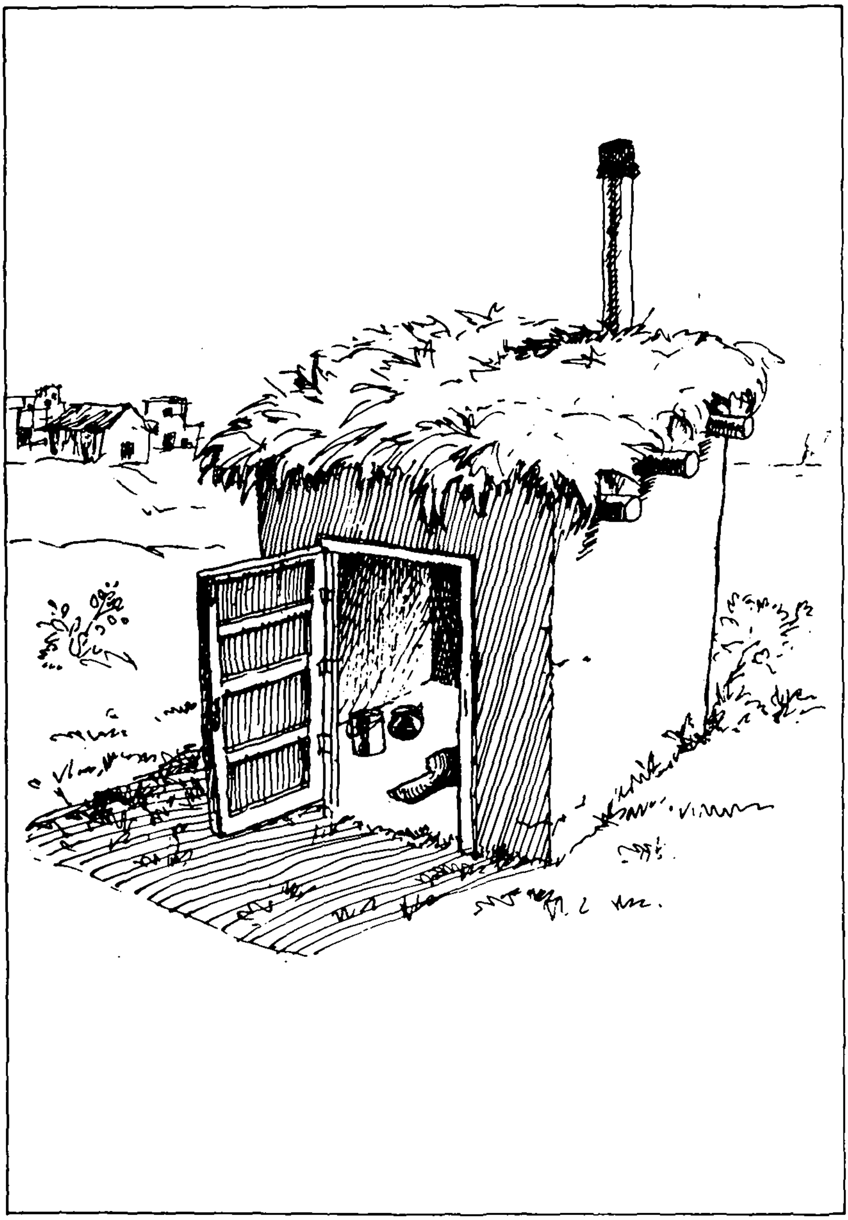
Reference is made to the Rahamaterpara study listed in the bibliography which details the rôle of a number of these factors.

10.3 Health Aspects:

10.3.1 *Pour Flush vs V.I.P. Latrine*: Experience has shown that the pour flush latrine, in general, performs better from a health view point. Fly nuisance, so frequently associated with VIPs because of no roof, poor orientation, a short or broken vent, etc., is rarely a factor with correctly flushed pour flush (PF) latrines. Mosquitos cannot breed in a PF latrine since it is flushed too frequently; but they can breed in a VIP where slow leaching takes place, or where the water table is high such as in Naseerabad. VIP latrines cannot be swept in their present configuration, while PF Latrines are designed to be swept. Even if the VIP were so designed, the sweeping could include a layer of fresh and smelly—and hazardous—excreta. Because water is associated with the PF type, there is often a greater chance that water for hand washing can be obtained; while the stress in the VIP is on a minimal amount of water. This is not to say that VIP latrines cannot be constructed successfully—but the technical requirements of that type are far greater, it is believed, than the PF type. To date, in Baluchistan, far more successful PF latrines have been seen than VIPs.

10.3.2 *Composting in Pits*: Several earlier sections have referred to this matter. Some concern must be expressed here about the composting process in VIP latrines. Because it cannot be handled as in the PF latrine, the VIP must be abandoned when full, and the slab—if it is to be reused, must be removed. The sight and odour in the pit will not be pleasant unless the latrine is left for at least six months. If this is done, what will users do in the mean-while? Another issue which has been raised concerning both VIP and PF latrines, is the possibility of cross contamination between pits and nearby water sources. This is dealt with in considerable detail in a recent publication concerning the risks of pollution from on-site santitation. Suffice it to say that in tight clays, contamination is not a problem, for pathogens are very quickly filtered out. In sandy soils, contamination can be a real hazard. Generally, however, in Baluchistan, soils are of clay except in river courses. Each site should be assessed on this count. As a general rule, latrines should be sited (hydraulically) downstream of water sources wherever possible. Alternatively, latrines should be sited as far from wells as practically possible, and commensurate with the social needs of the users. For instance, it is not feasible to suggest that women leave the home to use the latrine. Where a well exists in a compound, then, the latrine should not generally be sited near to it.

10.4 **Conclusions**: The pour flush latrine has been demonstrated in much of the sub-continent as a low cost feasible solution to excreta disposal. It has considerable advantages over the VIP Latrine–and one or two disadvantages. It is, naturally, up to each agency to decide which type is suitable in any environment. It may be appropriate to suggest, however, that users be given the choice wherever possible, for it is they who will be using them–not the extension workers.



THE VENTILATED IMPROVED PIT LATRINE

11. SANITATION: THE VENTILATED IMPROVED PIT LATRINE (V.I.P.)

The ventilated, improved pit latrine (VIP) as it is known today, was the brain-child of Dr. Peter Morgan of the Blair Institute in Harare, Zimbabwe. True, he may have taken the idea from latrines using vents in South Africa known as far back as the 1940s, but he made an art form of the idea. Dr. Morgan began the promotion of the VIP in the early 1970s in Zimbabwe. Since then, because of a very active health programme in Zimbabwe and the considerable help provided by the Technology Advisory Group (TAG) of the World Bank in promoting this and other types, it has become relatively well known.

11.1 **Technology of the V.I.P** : To start, it may be useful to examine the mechanisms which make the VIP latrine work. Obtaining a firm grasp of these allows the engineer/health worker to understand what can—and cannot—be done to vary the design to suit local situations. As there are *many* facets of the VIP, they will be dealt with individually:

> 11.1.1 *The Vent*: The vent is supposed to remove air upwards from the pit, so dispelling odours and trapping flies. The principle upon which it does this is TWO-Fold:
>
> > First, because the vent pipe is placed outside the latrine superstructure, is often dark in colour *and* faces the sun (for preference), it warms up, so warming the air in the pipe, reducing its density. Slightly cooler air from the latrine superstructure which faces away from the sun, displaces the lighter air in the vent, so creating a gentle draft. The superstructure should be relatively dark to maintain a temperature difference between the "hot" vent and air in the pit. Usually a small temperature difference of ± 2°C should suffice to obtain an air current.
> >
> > Secondly, and *far* more importantly, the vent acts like a pitot-tube. This mechanism "sucks" the air from a pipe when air flows over the top of the pipe. Tests in Zimbabwe have shown that this mechanism is far more powerful than the convection from heating the vent pipe. However, the vent pipe *must* project into the air stream for this mechanism to operate. If the vent is too short, turbulence may (and does) force air *down* the vent. The light at the top of the vent attracts flies, which then die because they cannot get out through the screen at the top of the pipe.

11.1.2 *The Superstructure*: In Zimbabwe, this is typically in spiral form. This has been found advantageous in that no door is required in this configuration; but it is *NOT* necessary to the functioning of the latrine. What *is* necessary, is that the latrine should provide an environment which remains cooler and darker than the vent pipe. If it does not, then convection cannot take place nor will flies be attracted to the top of the pipe and be trapped; and money invested in the vent will have been wasted. To keep the pit cooler and darker, the superstructure *must* have a roof. Perhaps as important–if not more so–is that it provides an environment suitable to the user (see later sections).

11.1.3 *The Slab*: This provides a number of functions. It covers the pit, so excluding smells (except those expelled through the vent). It – hopefully – supports the superstructure/pan/vent pipe from collapsing into the pit. It provides a place for the pan, and for the vent pipe hole; both directly over the pit. This is important for excreta and urine must pass easily into the pit without eroding the side of the pit; *and* the pipe must have its top (and thus the light) visible to flies which enter the pit. They, after feeding on the excreta, always fly towards the nearest light source. This may explain the need for mesh on the top of the vent pipe–to trap the flies.

11.1.4 *The Pan*: The design of the pan is *crucial*. It must provide a conduit for excreta and urine naturally; it must not splash urine on the user, particularly in a Muslim society where a person so splashed is enjoined to wash self and clothes before "namaz" (prayers). As prayers occur five times per day, a splashing pan can prove very tiresome. The pan must not present any threat to children–and must be so designed that it can be used without a child "doing the splits"; or the child will defecate directly on the slab–a common sight in S–E Asia. It must be easily cleaned, or excreta stuck on the pan will attract flies which can escape easily. A simple hole in the slab is rarely successful for it not only threatens children, but is difficult to aim at, particularly for women. The satisfactory functioning of an otherwise well designed latrine may be entirely destroyed for want of a simple, cheap and well designed pan.

11.1.5 *The Pit*: This is no more nor less than a hole in the ground. It should not crumble nor collapse; which mechanism is sometimes referred to as a "pitfall"! It should be big enough to store effluent from a family for sufficiently long to avoid having to rebuild or desludge too often. It should not be so big as to require major effort to make.

Pit Walls: Depending on the soil type, some stabilisation may be required on the walls, best done with ferro cement. Stabilisation should not go too far down in the pit, since the pit is a leach pit. Fluids "leach" into the ground; and this is difficult if concrete bars the way. In soft ground, honeycombed brick is a useful alternative, providing both support to the slab and a water path for fluids.

Emptying of Pits: Excreta–sadly for us–does not digest itself when placed in a pit. Yes, it does undergo some substantial change; but the result is some solid material which occupies space. Thus, after some time, the pit requires to be emptied. This must be considered at the time of designing the latrine. What, for instance, is to be done about pit emptying? Who will do it? Are sweepers available? If they are not, does the programme address the problem of informing people that they will be obliged to empty their own pits at some date in the future? Have they been told what they are likely to see in the pit? How long is the pit to be left to digest? Do we ourselves know what we are going to meet in the pit? What implements are to be used to sweep the pit? Is the pit designed to use the implements that are currently used in the community?

Pit Sizing: Depending on the objectives of the programme, so pit sizes may vary. For instance, if the programme is for a settled population, but it is thought that they will be unwilling to sweep the latrine pit when it is full (implying that they will abandon it and build another later), then the pit size should be large to avoid moving the latrine frequently. If, on the other hand, the latrines are for a mobile population (such as nomads), then it may not be worthwhile to make the pits large as they will be abandoned quite soon. The programmer must make his best estimate. However, whatever the choice, the reason should be shared with the recipient so that he understands when the pit is to be abandoned or swept. If you do not, then the attempts at latrine promotion may fail as a direct result. If so, why waste all the money in latrine promotion in the first instance? Also, it should be noted that people who have been the recipients of an unsuccessful latrine programme build up a resistance to later attempts to do the same thing. It is not thought to be the function of the Government to invest in peoples' future resistance to sanitation.

Future Pits: In Africa, it is the custom to build very deep latrine pits. The basis is that desludging never takes place. They simply abandon the old latrine, and build a new one somewhere else. They have the space available. Is this the case in the area in which this type of latrine is to be built?

11.1.6 *Summary Check List of Important Parameters which make the VIP Work:*

So far, technical functions have been noted which possibly may best be reinforced by a check list of “dos” and don’ts” for the vent, the most important single element in the VIP.

The vent pipe	*Reason why*
DO	
– put it *outside* the superstructure	Because it needs to get hot.
– put it facing the sun (south facing).	Because it needs to get hot.
– make it 4″ dia or greater	To minimise obstructions to the air flow; and to let as much light in as possible to attract flies.
– make it as high as possible	To ensure that the top of the pipe is in the airstream.
– place it directly over the pit	If you don’t, then no light can reach the pit to attract flies to the vent.
– cover it with stainless steel gauze	To make the mesh last as long as possible.
DON’T	
– put it inside	It gets in the way of the pan.
– keep it cool	Convection can’t work.
– have holes/cracks in it	It’s like trying to smoke a cigarette with holes; and flies can enter and leave without being trapped.
– cut it short	Turbulence will ensure that the latrine smells foul.
– cover it with anything more than the mesh	It excludes light and stops the airflow.
– use too small a diam of pipe	It reduces airflow, and does not admit enough light to attract flies.

(If you can’t meet the desirable criteria for the vent which, after all, is the mechanism which makes it work; then, have you selected the right latrine type?).

11.2 The Sociology of Latrines:

11.2.1 *The Importance of Latrine Behaviour*: We perhaps too often ensure that a design or machine works and expect people to adapt to the quirks of the particular design. Unfortunately, people "count"–a fact bitterly remembered by car manufacturers who have gone out of business because they were unable to provide machines which satisfied users. Latrine programmes, surprisingly, work in a very similar manner. Success is usually associated with designs which provide an environment which is private or safe or pleasant–or whatever it is that people need when performing their routine bodily functions. It may thus be best to start with people, and make the technology fit them, rather than the other way around. What, then, do people want from latrines?

11.2.2 *The Rahamaterpara Study*: A study in Bangladesh, in many respects not too unlike Pakistan, showed some surprises about latrine use. It very clearly showed that, despite what people said, privacy was the prime determinant of latrine use. However, this applied only to adults. Children, at least before puberty, displayed a need for security or perceived safety, and would not use latrines that provided privacy, since they felt insecure. Smell was rated as being important, but adults used smelly latrines nevertheless – where they provided complete privacy. Based on that study, it can be said that for a latrine programme to succeed, the latrine should:

Provide privacy, particularly to women;
Provide convenience;
Provide "security" (defensible space) to children;
Be odour-free;
Be made of durable material; and
Should not splash (urine splash or back splash).

Also, the children had some difficulty with certain pan designs, and were frightened of large, smelly black holes.

If the objective is that latrines be used, these factors are Paramount.

11.2.3 *The Importance of a Value being placed on the Latrine*: Too often, development agencies feel morally obliged to provide their inputs entirely free-of-charge. The basis is simple, and sometimes is partially valid; that communities to be served are poor and cannot afford to pay. However, it has been shown time and again that if a service is provided free, it rarely is valued by the recipient, with the consequence that considerable investment may be entirely wasted. This may be viewed as one of the greatest discomforts of development or assistance; that for success in transferring assistance *usefully* to recipients, we almost always have to charge for the service, even at highly subsidised rates.

Today, however, this view about giving development inputs away free-of-charge appears gradually to be changing. The World Bank will not consider financing a water supply unless cost recovery is implemented no matter the socio-economic group to be served. The World Food Programme (WFP) will not simply provide food, people must work for it. The "Food for Work" programme is well known. UNFPA's social marketing of condoms makes for a high value to be placed on those devices. Even the Government now demands that *cash* be paid for latrines (around 30% of the materials' cost) in Bangladesh. All of these different development inputs have been priced—so that a VALUE is placed on them. If people don't want to pay the price, that is their affair. Rarely are people so poor that they cannot afford a *correctly* subsidised latrine. If they are that poor, it seems reasonable to suppose that their priorities lie in food, clothing, shelter and water before sanitation; and who are we to tell *THEM* what they must think? Besides, a latrine programme cannot provide 100% coverage immediately, so it may be better to start providing first to those keen (and able to afford) to participate who should be so handled that they place a value on the latrine.

11.3 Some Health Aspects of the V.I.P. Latrine

11.3.1 *The Excreta of Children*: The excreta of children is highly dangerous. The contamination of the stools of adults and children suffering the same infection is substantially different. Childrens' stomachs are not well developed, and are, as a result, less able to destroy infective organisms as compared to adults. Which, in laymens' terms, means that the excreta of infants and children suffering an infection, should be treated with great respect. The excreta should be disposed of with great care. This also means that children should be encouraged to use latrines at the earliest possible opportunity. AND that latrines be designed so that children find them easy to use. (See the earlier notes about "defensible space" and pan design).

11.3.2 *Insects and the VIP Latrine*: Much has been said about flies and how they can be carriers of disease. The design of the VIP latrine takes this into account by providing a highly effective fly-trap. Tests have shown that, providing that the stainless steel screen is intact, some

90% of the flies entering the latrine pit, are caught under the mesh and die. The same is not true for mosquitos. Mosquitos are active at night when no light is visible at the top of the vent pipe. Tests have shown that only 30% or less of the mosquitos entering a pit will be caught. However, this in itself does not provide a health hazard. The Anopheline group of mosquitos which transmit malaria (and sometimes filariasis) prefer clean water in which to breed. However, the mosquito *Culex pipiens fatigans* which transmits both filariasis and Bancroftian filariasis, normally breeds in polluted water, and has been recorded frequently with latrines. Thus, where the water table is high, or where leaching takes place so slowly as to leave standing water in the pit for a sufficient length of time for breeding of this mosquito, this must be considered. It may even be necessary to discard the VIP latrine as being inappropriate in such areas; and replace it with a simple pour-flush type which uses little water, but which does not allow for passage of the mosquito between latrine and pit.

11.4 **Programme Objectives**: In planning a latrine programme, the objectives should be clearly defined. If the objective is *promotion of sanitation*, then it is probably best to construct a few latrines, but make them well and ensure that they are *used* and kept clean. In the case of nomads where they are not permanently resident, it may be appropriate to suggest that one of the objectives should be that the knowledge of effective and inexpensive methods of excreta disposal should become known, so that when they move, they will be able to perform the construction of their own latrines successfully; and *use* them at their new location.

Promoting latrines that don't work is unlikely to achieve the objective; and is likely to put people off the idea.

If, on the other hand, the objective is to clean the environment by isolating as much excreta as possible, then a totally different strategy may be necessary. It is not sufficient simply to choose a latrine type arbitrarily, and expect it to succeed, no matter what the objective is.

11.5 **Conclusions**: The foregoing notes do not presume to be exhaustive, but simply to highlight what are considered to be some of the most important parameters related to the V.I.P latrine relevant to Baluchistan.

As will have been noted, with the exception of the technical performance of the vent pipe, no weightings have been given of the importance of different aspects of the latrine programme. That is left to the programmers. It will also have been noted that no mention has been made of costs. Only the programmers who have an intimate knowledge of the programme will be able to say what can—and what cannot—be afforded both by the executive and support agencies; and the recipients.

ANNEX I

PUMP AND MOTOR EFFICIENCIES

PUMP EFFICIENCIES[1]

Horizontal Centrifugal	Medium size 80% to 82%, perhaps 85% large size. Even higher with special construction but at higher price.
Top driven turbine	Tending towards about 3% less than the horizontal centrifugal.
Submersible	75% to 81% and can be lower to about 70% for small sizes. Generally about 3% less again than the vertical spindle pump, the reason being that the pump is restricted in diameter.

ELECTRIC MOTORS[1]

For horizontal pumps	93% to 95%. Fixed speed a.c. induction.
For vertical pumps	90% to 94%. Fixed speed a.c. induction.
For submersibles	85% to 89%. Less than above on account of the restrictions imposed on the design.
Variable speed	About 3–5% less than with a squirrel cage a.c. motor.

DIESEL MOTORS:

For small sizes	50% to 60% (see individual manufacturers' data, and Annex III which gives adjustment to be applied for both temperature and altitude).

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1. This is based on Twort, Hoather and Law, "Water Supply", Second Edition (Metric Units), ISBN: 0 7131 3318 X, page 385.

CENTRIFUGAL PUMP PERFORMANCE: TABLES, CURVES AND WORKED EXAMPLE

TABLE 1: GUIDELINES FOR PUMP SELECTION BASED ON TYPE A AND TYPE B CURVES

Present Population (Range) 0 – 0	Design Flow (Range) (1/s)	Pump Type and Speed (rpm)	Pipeline Diameters (mm)		Headloss Coefficients[2] 'k'		Head losses[5] (m)	
			Suction	Delivery[1]	Suction[3] only	Whole p[4] station	Suction[6]	Whole p[7] station
Upto 1110	0.0 – 4.2	Type A/1450	65	82	5.0	20	0.86	1.09
1110 – 1950	4.2 – 7.4	Type A/1450	65	105	5.0	48	1.81	2.34
1950 – 3160	7.4 – 12.0	Type B/1450	82	155	5.0	94	1.89	2.51
3160 – 4210	12.0 – 16.0	Type B/2900	82	155	5.0	94	2.98	4.09
4210 – 5800	16.0 – 22.0	Type B/2900	105[8]	155	6.0	94	2.50	7.04

Notes:
1. Actual delivery pipe sizes determined from BRANCH Programme.
2. k = (Headloss) x $^{2g}/_{v^2}$.
3. Expressed in terms of suction pipe diameter.
4. Expressed in terms of delivery pipe diameter.
5. Calculated for top end of flow range only.
6. Suction loss = NPSH = lift (0.4 m) + p/1 friction + local losses.
7. Includes lift (0.4) = p/1 friction + local losses.
8. Revised suction pipe arrangement including expansion taper (82/105) close to pump inlet.

This Table was provided by Messrs Halcrow-ULG under contract to UNICEF.

TABLE 2: IMPELLER DIAMETERS AND COMBINATIONS FOR TYPE A PUMPS:

Single Stage Pumps:

a = 1 impeller 175 mm max dia.] Single
b = 1 impeller with vane tips cut back to 164 mm dia.] stage
c = 1 impeller trimmed to 153 mm dia.] pumps

Multi-Stage Pumps:

a = x impellers 175 mm dia. (N^{O} of stages = N^{O} of impellers x)
b = x impellers 175 mm dia. + 1 implr. cut to 164 mm dia. (last)
c = x impellers 175 mm dia. + 1 implr. cut to 153 mm dia. (last)
d = x impellers 175 mm dia. + 1 implr. cut to 164 mm dia. (2nd last)
+ 1 implr. cut to 153 mm dia. (last)
e = x impellers 175 mm dia. + 2 implr. cut to 153 mm dia. (last two)

TABLE 3: IMPFLLER DIAMETERS AND COMBINATIONS FOR TYPE B PUMPS:

Single Stage Pumps:

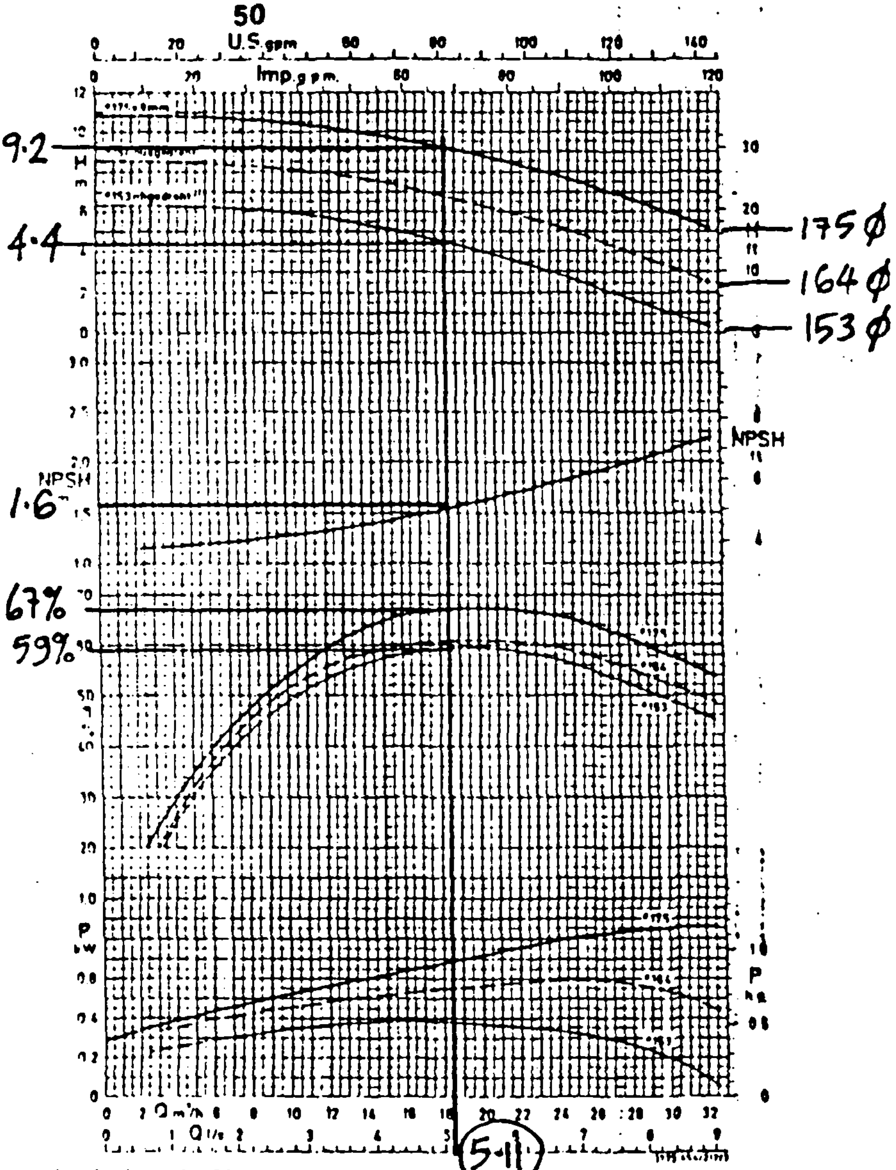
a = 1 impeller 203 mm max dia.] Single
b = 1 impeller with vane tips cut back to 185 mm dia.] stage
c = 1 impeller trimmed to 172 mm dia.] pumps

Multi-Stage Pumps:

a = x impellers 203 mm dia. (N^{O} of stages = Total N^{O} of impellers)
b = x impellers 203 mm dia. + 1 implr. cut to 185 mm dia. (last)
c = x impellers 203 mm dia. + 1 implr. cut to 172 mm dia. (last)
d = x impellers 203 mm dia. + 1 implr. cut to 185 mm dia. (2nd last)
+ 1 implr. cut to 172 mm dia. (last)
e = x impellers 203 mm dia. + 2 implr. cut to 172 mm dia. (last two)

Note: The above Tables should be read in conjunction with the relevant pump combination charts.

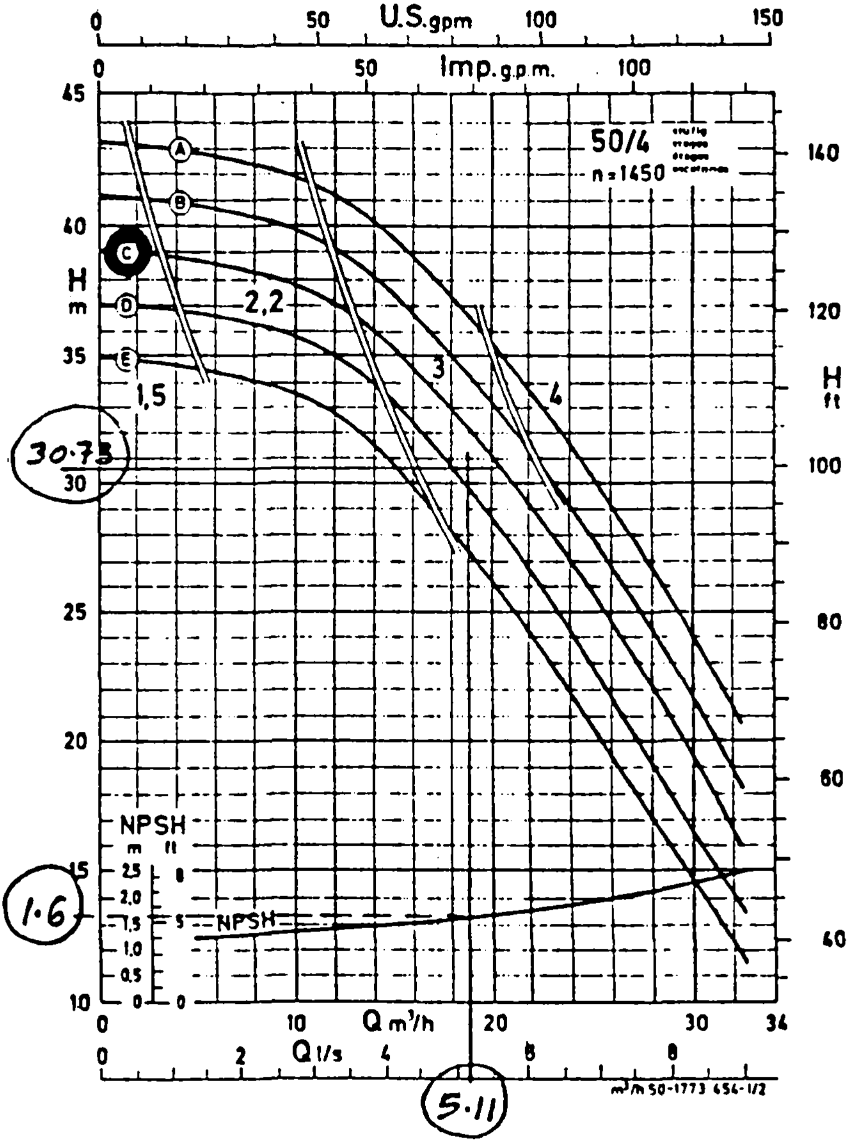
TYPE A PUMP PERFORMANCE CURVES (1450 rpm)



Laufrad	175-153 mm Ø.	Laufradaustrittsbreite	9 mm
Impeller	175-153 mm Ø.	Impeller outlet width	9 mm
Roue	175-153 mm Ø.	Largeur à la sortie de la roue	9 mm
Rodete	175-153 mm Ø.	Anchura de salida del rodete	9 mm

Note: Only the pump performance curves for Type A pumps at the requisite speed (1450 rpm) are presented, although curves are available for each of the different operating speeds. Consult the manufacturer for the full set of curves.

TYPE A: 4-STAGE IMPELLER COMBINATION CHART



Note: Only one chart is shown, although charts exist for single and multi-stage combinations and for the different operating speeds. Consult the manufacturer for the full set of the required charts.

WORKED EXAMPLE FOR CLUSTER "M" WATER SUPPLY SCHEME:

Data:

Present Population, P_p = 1,345

Design Flow, Q_d = 5.11 litres/sec
= 5.11×10^{-3} m^3/sec

Required Head H_r = 29.87 m

From Table 1, Annex II, Pump Type = Type A/1450
Suction dia = 65 mm
Delivery dia = 105 mm
$k_{suction}$ = 5.0
$k_{station}$ = 48

$$\text{Delivery Velocity} = \frac{\text{Flow}}{\text{pipe X-sectnl area}} = \frac{5.11 \times 10^{-3} \times 4}{\pi \times 0.105^2}$$

$$\text{Velocity Head} = \frac{v^2}{2g} = \frac{0.59^2}{2 \times 9.81} = 0.018 \text{ m}$$

$$\text{Headloss through pumping station} = k_{station} \times \frac{v^2}{2g}$$

$$= 48 \times 0.018$$
$$= 0.86 \text{ metres}$$

$$\text{Total Pumping Head (H)} = 29.87 + 0.86$$
$$= 30.73 \text{ metres}$$

From Impeller Combination Charts for Type A Pumps, 4 N° stages are required, employing combination "c". From Table 2, it will be seen that this implies 3 N° impellers @ 175 mm dia., and 1 N° impeller @ 153 mm. Should the exact duty point be required, this may be arranged with the manufacturer, who is able to provide the precise impeller size to reach that duty point. However, given that the hydraulics of the system are very likely to be varied, this matching is not useful, and will not be undertaken.

FROM THE PUMP CHARACTERISTICS GRAPH

Impeller Dia (mm)	N^O #	Head per impeller	N^O x Head	Efficiency (%)	NPSH (m)
175	3	9.2	$27.6 = H_1$	$67 = \%_1$	1.6
153	1	4.4	$4.4 = H_2$	$59 = \%_2$	1.6

(The above table gives $H_1 + H_2 = 32.0$ m. Design Head = 30.73 m.)

Power reqt. of the Pump = $Q \times g \times (H_1/\%_1 + H_2/\%_2)$
= $5.11 \times 10^{-3} \times 9.81 \times (27.6/0.67 + 4.4/0.59)$
= $0.05 \times (41.19 + 7.46)$
= 2.43 kw

Power requ^t. of pump/motor combination
(assuming electric motor is 90% effict) = 2.43/0.9
= 2.7 kw or 3.6 horsepower

Check the suction head:

$$\text{Suction velocity} = \frac{5.11 \times 10^{-3} \times 4}{\pi \times (0.065)^2} = 1.54 \text{ m/s}$$

$k_{suction}$ = 5.0 and $v^2/2g = 0.121$ m

Suction loss in pipe fittings & valves = $k.v^2/2g$
= 5×0.121
= 0.61 m

From drawing, vertical suction lift = 0.40 m

Friction loss in 1.9 m of suction pipe
(From headloss tables $h_f = 40/1000$)
Thus, suction pipe headloss $1.9 \times 40/1000$ = 0.08 m

Total Suction Head = 1.09 m

This is < 1.6 metres (allowable NPSH), and is thus satisfactory

ANNEX III

DIESEL ENGINE PERFORMANCE FALLOFF DUE TO TEMPERATURE AND ALTITUDE

The following table provides the "Falloff Factor" applicable to internal combustion engines (for petrol and diesel), due to both temperature and altitude.

Elevation above Mean Sea-Level metres	TEMPERATURE IN DEGREES CELSIUS							
	20°C	25°C	30°C	35°C	40°C	45°C	50°C	55°C
0	1.007	0.989	0.971	0.953	0.935	0.917	0.899	0.881
250	0.974	0.956	0.938	0.920	0.902	0.884	0.866	0.848
500	0.941	0.923	0.905	0.887	0.869	0.852	0.834	0.816
750	0.909	0.891	0.873	0.855	0.837	0.819	0.801	0.783
1,000	0.876	0.858	0.840	0.822	0.804	0.786	0.768	0.750
1,250	0.843	0.825	0.807	0.789	0.771	0.753	0.735	0.717
1,500	0.810	0.792	0.774	0.756	0.738	0.720	0.702	0.684
1,750	0.777	0.759	0.741	0.723	0.705	0.687	0.669	–
2,000	0.744	0.726	0.708	0.691	0.673	0.655	0.637	–
2,250	0.712	0.694	0.676	0.658	0.640	0.622	0.604	–
2,500	0.679	0.661	0.643	0.625	0.607	0.589	–	–
2,750	0.646	0.628	0.610	0.592	0.574	0.556	–	–
3,000	0.613	0.595	0.577	0.559	0.541	–	–	–

ANNEX IV

A PERSPECTIVE ON BERNOUILLI'S EQUATION FLOW AT THE STREET TAP

Designers of water supply systems will generally be familiar with Bernouilli's Equation. What he was expressing in mathematical terms was that the total energy of water always remains the same under certain specific conditions. The three forms of energy are kinetic, pressure and potential. The equation can be expressed as:

$$\frac{u^2}{2g} + p + z = \text{A constant}$$

[Please note that this applies only to steady flow in a "streamline"; and does not apply when energy is lost through friction.]

This is entirely relevant in the present context, since the computer provides answers to external node conditions which include both kinetic and pressure energy. Unfortunately, taps have this nasty habit of misunderstanding what the computer calculations show, and do not behave as they should. They tend to provide too high a flow and too little residual head, but still keeping the total energy essentially the same, similar to the way that water behaves as described in Bernouilli's equation. We are thus obliged to provide a device at the tap which will regulate the flow so that the residual energy conditions are met. This is done with a globe valve which is able to destroy some of the head in a variable manner; so that at a later stage, when the pipeline pressure falls, it can be opened slightly to restore the original flow. Naturally, it has to be locked so that it cannot be adjusted by someone who does not know what opening the valve fully will do to the rest of the system.

There is another, simpler method of destroying head at the tap. This is by means of an orifice which is placed in the pipeline. This, too, must be so installed to ensure that it cannot be removed. A tack-weld usually suffices. Orifice sizing is dealt with on the following pages.

Either system must be used; for if it is not, the overall system will not behave as it was designed, which results in some taps getting more than they should, and others, too little. Knowing that communities are likely to tamper with the system no matter what one does, it is up to the designer to ensure—to the extent possible—that all external nodes have an equal (and as low as practical) residual pressure.

THE SIZING OF ORIFICES FOR DESTROYING HEAD AT THE TAP OR COMMUNITY TANK

The Standard Formula for flow through a Sharp Edged Orifice as shown in "The Manual of British Water Engineering Practice" is as follows:–

$$Q = C \times a \times \frac{1}{(1 - (d/D)^4)^{0.5}} \times (2gH)^{0.5}$$

Where Q = Discharge, cusecs
a = Area of orifice, ft^2
d = Diam of orifice, ft
D = Diam of pipe, ft
H = Head loss, ft
C = Coefficient

(Please note that the units have been left Imperial
as reflected in the reference)

The Coefficient, C, depends both on the ratio $^d/D$ and on the Reynolds Number. The Manual of British Water Engineering Practice presents a set of curves to determine the value of C for $^d/D$ values from 0.2 to 0.7, and Reynolds Numbers from 1×10^3 to 1×10^6. These curves are based on the work of Johansen, Barnes and Engel.

For the purposes of this Manual, only orifices in galvanised ¾", 1" and 1½" diameter pipes are considered. Please note that orifices in smaller diameter pipes, unless made with very great accuracy, will rarely perform as predicted; because the effects of small variations in geometry play a profound rôle in determining performance.

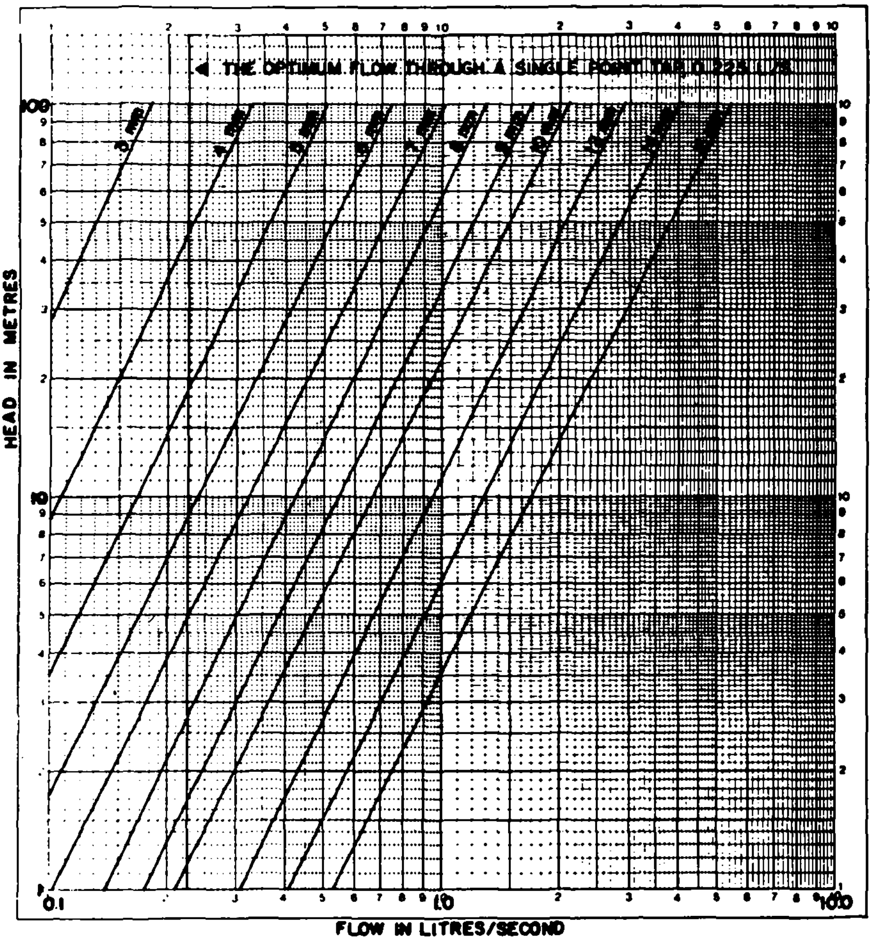
The following page presents:

Orifices of 2 mm to 16 mm in ¾", 1" & 1½"ϕ GI pipes.

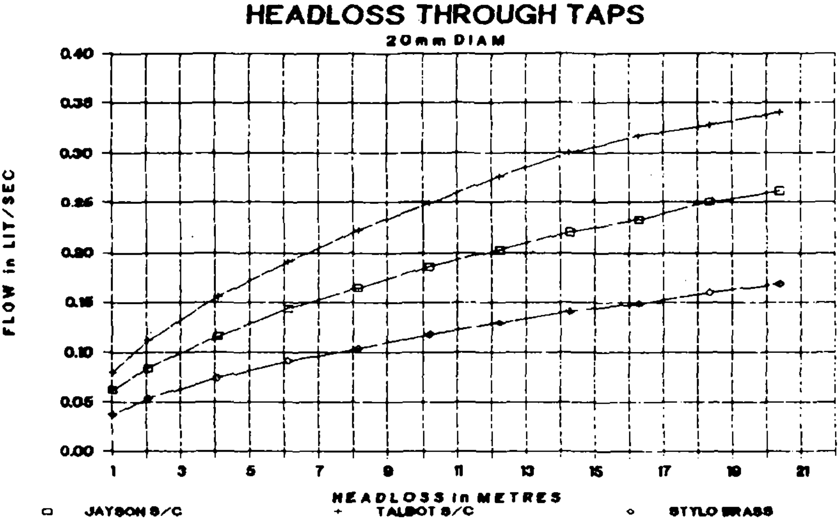
It may surprise readers that the headloss through orifices in varying sizes of pipes are presented on the same graph; but if they calculate the data themselves, they will note that the differences are unable to be plotted.

THE SIZING OF ORIFICES FOR DESTROYING HEAD AT THE TAP OR COMMUNITY TANK

(For orifices 2-16mm in ¾", 1" & 1½" ϕ GI Pipes)



A peculiar problem is faced with self-closing taps. While elsewhere the designer normally seeks to destroy head, with self-closing taps one finds that they tend to destroy rather more head than is available to provide the flow which communities wish. For that reason, curves are presented which show Q vs H for two different self-closing taps together with one, standard, brass bib-cock.



The "Jayson" and "Stylo" taps are available locally; while the "Talbot" must be imported. It will be seen that, to achieve the 'optimum' flow of 0.225 litres/sec, considerable head is required. This may affect what taps may be used on community tanks.

These curves are the result of testing by WEDC of Loughborough University for Halcrow-ULG, for the Modules.

ANNEX V

HAZEN-WILLIAMS FORMULA

The formula can be expressed as:

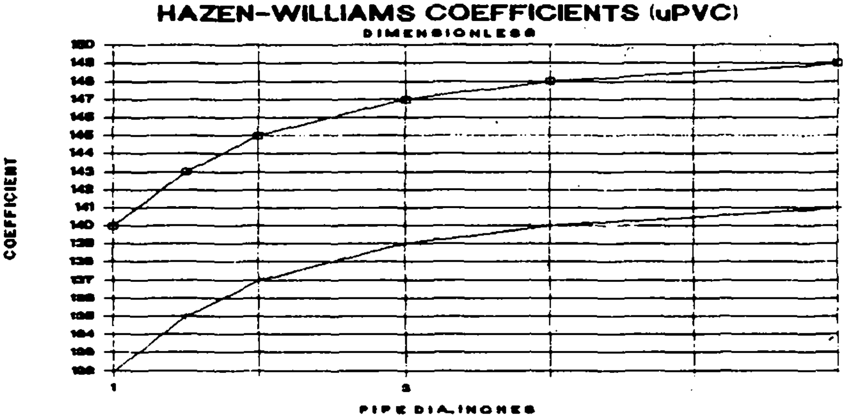
$$V = 0.355 \text{ x } C \text{ x } d^{0.63} \text{ x } \left\{\frac{H}{L}\right\}^{0.54}$$

Where V = Flow velocity in pipe, m/s.
H = Head loss in metres.
L = Pipe length in metres.
d = Pipe diameter, metres.
C = Hazen-Williams Coefficient, dim' less.

The formula is popular because

(i) It is well documented and easy to use;
(ii) It has been used for sufficiently long to establish fairly reliable values of C to take in most circumstances; and
(iii) It is reasonably accurate over a wide range of pipeline sizes and flows which are those normally experienced.

This is based on Twort, Hoather and Law, "Water Supply", Second Edition (Metric Units), ISBN: 0 7131 3318 X, page 316.



(1) Suggested design values are for a non-aggressive and non-sliming water.

(2) The curves apply to 1 m/s flow rate. For double this velocity reduce C values by 5% below 100: 3% below 130: and 1% below 140.

However, please note (see Section 2.11), the HWC (Hazen-Williams Coefficient) that is used with the computer programme "BRANCH", is slightly conservative as per the following table:–

Diameter	Actual HWC	HWC to be used
1 "	140	132
1½ "	143	135
2 "	145	137
3 "	147	139
4 "	148	140
6 "	149	141

---

1. This is based on Twort, Hoather and Law, "Water Supply", Second Edition (Metric Units), ISBN: 0 7131 3318 X, page 318.

ANNEX VI

BS 3505: DIMENSIONS FOR uPVC PIPES

TABLE : PIPE DIMENSIONS FOR CLASSES B, C, D AND E.

Nomin π ins	Mean Outside diameter		Wall thickness											
			Class B 6.0 bar			Class C 9.0 bar			Class D 12.0 bar			Class E 15.0 bar		
			averaged value	individual value		averaged value	individual value		averaged value	individual value		averaged value	individual value	
	min.	max.	max.	min.	max.	max.	min.	max	max.	min.	max	max.	min.	max.
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
1	33.4	33.7	–	–	–	–	–	–	–	–	–	2.7	2.2	2.7
1½	48.1	48.4	–	–	–	–	–	–	3.0	2.5	3.0	3.7	3.1	3.7
2	60.2	60.5	–	–	–	3.0	2.5	3.0	3.7	3.1	3.7	4.5	3.9	4.5
3	88.7	89.1	3.4	2.9	3.4	4.1	3.5	4.1	5.3	4.6	5.3	6.5	5.7	6.6
4	114.1	114.5	4.0	3.4	4.0	5.2	4.5	5.2	6.8	6.0	6.9	8.3	7.3	8.4
6	168.0	168.5	5.2	4.5	5.2	7.5	6.6	7.6	9.9	8.8	10.2	12.1	10.8	12.5

This table is extracted from British Standard 3505 : 1968;
"Table 1: Pipe Dimensions for Classes B, C, D and E".

EXAMPLE
"BRANCH" COMPUTER PROGRAMME OUTPUT

```
TITLE            :  NILLI
NO. OF LINKS     :  23
NO. OF NODES     :  24
PEAK FACTOR      :   1
MIN HL/KM        :  .2
MAX HL/KM        :  25
RESIDUAL HEAD    :   4

AVAILABLE PIPES  :

-------------------------------
  DIAM                  UNIT
  (MM)       HWC        COST
-------------------------------
   29        132        23.34
   43        135        30.05
   55        137        35.73
   82        139        42.19
  105        140        71.95
  155        141       111.30
```

EXAMPLE: "BRANCH" COMPUTER PROGRAMME OUTPUT (Contd.)

PIPE No.	NODE		FLOW (lps)	DIAM (mm)	HWC	HLOSS (m)	HL/KM (m)	LENGTH (m)	COST
	from	to							
1	1	2	2.850	105	140	0.51	1.30	390.00	27,709.50
2	2	3	0.800	43	135	9.35	10.27	910.00	27,345.50
3	3	4	0.250	29	132	0.42	8.48	50.00	1,167.00
4	3	5	0.550	43	135	0.51	5.14	100.00	3,005.00
5	5	6	0.300	29	132	0.89	11.88	75.00	1,750.50
6	5	7	0.250	29	132	1.27	8.48	150.00	3,501.00
7	2	8	2.050	82	139	0.04	2.39	18.12	764.33
8	8	9	1.000	43	135	16.15	15.52	1,040.00	31,252.00
9	9	10	0.250	29	132	1.27	8.48	150.00	3,501.00
10	9	11	0.750	43	135	2.74	9.12	300.00	9,015.00
11	11	12	0.500	43	135	0.22	4.31	50.00	1,502.50
12	12	13	0.250	29	132	0.55	8.48	65.00	1,517.10
13	12	14	0.250	29	132	0.17	8.48	20.00	466.80
14	11	15	0.250	29	132	1.06	8.48	125.00	2,917.50
16	16	17	0.650	43	135	4.02	7.00	575.00	17,278.75
17	17	18	0.200	29	132	0.06	5.61	10.00	233.40
18	17	19	0.450	43	135	0.02	3.54	5.00	150.25
19	19	20	0.150	29	132	0.03	3.30	10.00	233.40
20	19	21	0.300	29	132	0.36	11.88	30.00	700.20
22	22	23	0.200	29	132	0.06	5.61	10.00	233.40
23	22	24	0.200	43	135	0.14	0.79	175.00	5,258.75
								TOTAL =	139,502.89

EXAMPLE
"BRANCH" COMPUTER PROGRAMME OUTPUT (Contd.)

```
NODE     FLOW        ELEV        H G L       PRESSURE
NO.      (LPS)       (M)         (M)         (M)

 1S      2.850       498.0       498.0        0.0
 2       0.000       473.1       497.5       24.4
 3       0.000       466.5       488.2       21.7
 4      -0.250       466.5       487.8       21.3*
 5       0.000       464.8       487.7       22.9
 6      -0.300       463.4       486.8       23.4*
 7      -0.250       462.9       486.4       23.5*
 8       0.000       487.4       497.5       10.1
 9       0.000       467.5       481.4       13.9
10      -0.250       468.1       480.1       12.0*
11       0.000       467.9       478.6       10.7
12       0.000       467.6       478.4       10.8
13      -0.250       466.5       477.9       11.4*
14      -0.250       467.5       478.2       10.7*
15      -0.250       468.5       477.6        9.0
16       0.000       485.0       497.5       12.5
17       0.000       471.5       493.5       22.0
18      -0.200       471.5       493.4       21.9*
19       0.000       471.5       493.5       22.0
20      -0.150       471.5       493.4       22.0*
21      -0.300       471.3       493.1       21.8*
22       0.000       485.3       497.5       12.2
23      -0.200       486.0       497.4       11.4*
24      -0.200       493.4       497.4        4.0
```

*These residual pressures are too high and must be reduced.

SUMMARY

```
  DIAM       LENGTH
  (MM)        (M)        COST

   29         695.0    16,221.30
   43       3,155.0    94,807.75
   82          18.1       764.33
  105         390.0    27,709.50
                      ------------
          T O T A L = 139,502.88
                      ------------
```

EXAMPLE

DETAILS OF PIPES REQUIRED FOR PHASE II SCHEMES

Name of Districts	Name of Schemes	Present Popula-	Lengths of required pipes in metres							Total length of pipe lines	Length of pipe per capita (m)
			1"	1½"	2"	3"	4"	6"	8"		
Pishin	1. S. Alizai	5,295	0	0	0	2,660	3,300	1,200	0	7,160	1.35
	2. Tore Shah	2,560	0	0	1,030	1,170	456	1,084	0	3,740	1.46
	3. Hajizai	6,110	0	960	550	3,977	2,051	2,196	0	9,734	1.59
	4. Muchan	1,345	0	200	0	3,478	593	0	0	4,271	3.18
	5. Segi	3,895	155	4,105	2,320	4,580	720	302	0	12,182	3.13
	Sub Total (1)	19,205	155	5,265	3,900	15,865	7,120	4,782	0	37,087	1.93
	6. Murgha	1,135	0	2,427	3,604	2,800	0	0	0	8,831	7.78
	7. Nilli	750	0	855	2,620	1,080	835	0	0	5,390	7.19
	Sub Total (2)	1,885	0	3,282	6,224	3,880	835	0	0	14,221	7.54
Lasbela	8. Pirwala	1,555	0	0	4,340	1,750	5	0	0	6,095	3.92
	9. Raza Mohd	935	0	1,217	1,040	982	0	0	0	3,239	3.46
	10. Sh. Mangia	785	1,090	180	2,613	200	0	0	0	4,083	5.20
	11. Ismailani	3,120	0	50	0	4,876	343	783	0	6,052	1.94
	12. Musiani	620	1,041	0	800	163	0	0	0	2,004	3.23
	13. Godarl	720	509	6,241	4,680	2,520	0	0	0	13,950	19.38
	Sub Total (3)	7,735	2,640	7,688	13,473	10,491	348	783	0	35,423	4.58

DETAILS OF PIPES REQUIRED FOR PHASE II SCHEMES (Contd.)

Name of Districts	Name of Schemes	Present Population	Lengths of required pipes in metres							Total length of pipe lines	Length of pipe per capita (m)
			1"	1½"	2"	3"	4"	6"	8"		
Turbat	14. Nasirabad	6,095	0	980	4,000	3,000	4,000	2,050	0	14,030	2.30
	15. Baleecha	2,380	0	1,400	1,640	3,386	3,352	437	0	10,215	4.29
	16. Shahrak	1,350	0	1,462	550	860	793	0	0	3,665	2.71
	17. Heronk	1,145	244	0	1,605	762	0	0	0	2,611	2.28
	18. Gumazai	4,330	0	2,680	1,302	3,248	3,173	2,317	0	12,720	2.94
	19. Ginna	3,585	0	1,100	2,740	4,536	1,013	600	0	9,989	2.79
	20. Kalag	1,825	0	500	3,282	4,634	2,030	0	0	10,446	5.72
	21. Sami	1,225	0	0	1,218	504	427	0	0	2,149	1.75
	Sub Total (4)	21,935	244	8,122	16,337	20,930	14,788	5,404	0	65,825	3.00
Kachhi	22. Theri	2,580	0	500	630	400	750	11,814	0	14,094	5.46
	23. Mir Bagh[2]	980	937	4,758	2,151	2,631	2,025	0	0	12,502	12.76
	24. Naghari	460	350	1,713	501	180	0	0	0	2,744	5.97
	25. Chhotai	580	210	2,050	200	154	0	0	0	2,614	4.51
	26. Mushkaf	725	0	1,596	390	260	0	0	0	2,246	3.10
	27. Shoran	1,775	0	840	400	900	2,800	0	0	4,940	2.78
	28. Kotra	2,345	0	680	620	5,332	1,930	1	0	8,563	3.65
	29. Taib	4,505	0	1,450	3,938	2,631	3,643	3,961	50	15,673	3.48
	Sub Total (5)	13,950	1,497	13,587	8,830	12,488	11,148	15,776	50	63,376	4.54
	Total	64,710	4,536	37,944	48,764	63,654	34,239	26,745	50	215,932	3.34
	Add 5% extra for each Pipe		227	1,897	2,438	3,183	1,712	1,337	3	10,797	
	Grand Total		4,763	39,841	51,202	66,837	35,951	28,082	53	226,729	
	Available with BIAD (Present stock)		8,274	120	34,122	9,948	16,986	23,190	8,184	100,824	
	Net quantity required		(3,511)	39,721	17,080	56,889	18,965	4,892	(8,132)	125,905	

1, 2 Godar and Mir Bagh may require revision. Lengths of pipe per capita appear excessive and those communities may not be able to maintain such lengths.

For phasing of schemes, see ANNEX VIII, page 121.

ANNEX VIII (Contd.)

WORK PLAN – 1988–1990

Name of Districts	Name of Schemes	1988				1989				1990			
		1	2	3	4	1	2	3	4	1	2	3	4
Pishin	1. S'Alizai[1]			—			?????				?????		
	2. Tore Shah			—									
	3. Hajizai			—									
	4. Muchan			—									
	5. Segi			—									
	6. Murgha						—						
	7. Nilli						—						
Lasbela	8. Pirwala				—								
	9. Raza Mohd				—								
	10. Sh. Mangia				—								
	11. Ismailani								—				
	12. Musiani								—				
	13. Godar								—				
Turbat	14. Nasirabad[2]				—								
	15. Baleecha				—								
	16. Shahrak				—								
	17. Heronk								—				
	18. Gumazai								—				
	19. Ginna								—				
	20. Kalag								—				
	21. Sami								—				
Kachhi	22. Theri				—								
	23. Mir Bagh				—								
	24. Naghari				—								
	25. Chhotai								—				
	26. Mushkaf								—				
	27. Shoran								—				
	28. Kotra								—				
	29. Taib								—				

Note: This workplan was prepared to avoid the Holy Month of Ramzan, and to conform to weather patterns in the Province.

1. The workplan requires constant updating. For instance, Simzai Alizai's (#1) water source suffered a sudden lowering in yield, and the scheme had to be held over till a secure source could be established. This naturally affected phasing of supplies.

2. Nasirabad (#14) elected to stop their water supply scheme on the basis that a borehole would dry up their karezes. A revised scheme might be based on the karezes.

ANNEX VIII
(Contd.)

EXAMPLE

DETAILS OF PIPES REQUIRED FOR PHASE II SCHEMES

Name of Districts	Name of Schemes	Lengths of required pipes in metres							Total length of pipe lines
		1"	1½"	2"	3"	4"	6"	8"	
Pishin	1. S'Alizai	0	0	0	2,660	3,300	1,200	0	7,160
(6/88)	2. Tore Shah	0	0	1,030	1,170	456	1,084	0	3,740
	3. Hajizai	0	960	550	3,977	2,051	2,196	0	9,734
	4. Muchan	0	200	0	3,478	593	0	0	4,271
	5. Segi	155	4,105	2,320	4,580	720	302	0	12,182
(5/89)	6. Murgha	0	2,427	3,604	2,800	0	0	0	8,831
	7. Nilli	0	855	2,620	1,080	835	0	0	5,390
Lasbela	8. Pirwala	0	0	4,340	1,750	5	0	0	6,095
(10/88)	9. Raza Mohd	0	1,217	1,040	982	0	0	0	3,239
	10. Sh. Mangia	1,090	180	2,613	200	0	0	0	4,083
(10/89)	11. Ismailani	0	50	0	4,876	343	783	0	6,052
	12. Musiani	1,041	0	800	163	0	0	0	2,004
	13. Godar	509	6,241	4,680	2,520	0	0	0	13,950

ANNEX VIII (Contd.)

DETAILS OF PIPES REQUIRED FOR PHASE II SCHEMES

Name of Districts	Name of Schemes	Lengths of required pipes in metres							Total length of pipe lines
		1"	1½"	2"	3"	4"	6"	8"	
Turbat	14. Nasirabad	0	980	4,000	3,000	4,000	2,050	0	14,030
(10/88)	15. Baleecha	0	1,400	1,640	3,386	3,352	437	0	10,215
	16. Shahrak	0	1,462	550	860	793	0	0	3,665
(10/89)	17. Heronk	244	0	1,605	762	0	0	0	2,611
	18. Gumazai	0	2,680	1,302	3,248	3,173	2,317	0	12,720
	19. Ginna	0	1,100	2,740	4,536	1,013	600	0	9,989
	20. Kalag	0	500	3,282	4,634	2,030	0	0	10,446
	21. Sami	0	0	1,218	504	427	0	0	2,149
Kachhi	22. Theri	0	500	630	400	750	11,814	0	14,094
(10/88)	23. Mir Bagh	937	4,758	2,151	2,631	2,025	0	0	12,502
	24. Naghari	350	1,713	501	180	0	0	0	2,744
(10/89)	25. Chhotai	210	2,050	200	154	0	0	0	2,614
	26. Mushkaf	0	1,596	390	260	0	0	0	2,246
	27. Shoran	0	840	400	900	2,800	0	0	4,940
	28. Kotra	0	680	620	5,332	1,930	1	0	8,563
	29. Taib	0	1,450	3,938	2,631	3,643	3,961	50	15,673

Note: These sheets are not mandatory but are a useful summary.

ANNEX VIII
(Contd.)

EXAMPLE

DETAILS OF PIPES REQUIRED FOR PHASE II SCHEMES

(All figures are in metres unless otherwise specified)

Size of Pipes	Stock on 4/88	Five Pishin Schemes by 6/88	Balance at 7/88	Needed by 6/88	Nine Schemes K/L/T by 10/88	Balance at 11/88	Needed by 10/88	Two Pishin Scheme by 5/89	Balance at 6/89	Needed by 5/89	13 Schemes K/L/T by 10/89	Balance at 11/89	Needed by 10/89
1"	8,274	155	8,119	0	2,377	5,742	0	0	5,742	0	2,004	3,738	0
1½"	120	5,265	(5,145)	5,145	12,210	(17,355)	12,210	3,282	(20,637)	3,282	17,187	(37,824)	17,187
2"	34,122	3,900	30,222	0	17,465	12,757	0	6,224	6,533	0	21,175	(14,642)	14,642
3"	9,948	15,865	(5,917)	5,917	13,389	(19,306)	13,389	3,880	(23,186)	3,880	30,520	(53,706)	30,520
4"	16,986	7,120	9,866	0	10,925	(1,059)	1,059	835	(1,894)	835	15,359	(17,253)	15,359
6"	23,190	4,782	18,408	0	14,301	4,107	0	0	4,107	0	7,662	(3,555)	3,555
8"	8,184	0	8,184	0	0	8,184	0	0	8,184	0	50	8,134	0
	100,824	37,087	63,737	11,062	70,667	(6,930)	26,658	14,221	(21,151)	7,997	93,957	(115,108)	81,263

Comulative pipe requirements = 215,932
Add: Excess pipe 1" as of 11/89 = 3,738
Excess pipe 8" as of 11/89 = 8,134
227,804

Stock available as of 4/88 100,824
Actual pipe required as of 11/89 126,980
227,804

ANNEX IX

BALUCHISTAN: 6TH 5-YEAR PLAN (83–88)
EXTRACTS

Pg 214, §2: Introduction:

Of all the Provinces of the country, the problem of providing drinking water to the people of Baluchistan is the most acute and poses great difficulties. Whereas in other Provinces, the source of supply is readily available adjacent to population clusters, in Baluchistan strenuous efforts are required to establish a dependable source and then convey water through long distances in difficult terrain. This results in very high unit costs, whether measured per 1000 gallons supplied or per 1000 people served.

Pg 214, §4: Fifth Plan:

The gap between availability and requirement was also very large even if modest standards were adopted. Realizing its importance, the provision of a safe water supply, particularly in rural areas, was laid down as one of the major objectives of the Fifth Five Year Plan. It was decided that in order to cover maximum population with the available limited resources, *high standards and sophisticated technology be avoided.*

Pg 215, § 3: Fifth Plan (Rural):

For rural water supplies the target should be to cover at least the cost of operation and maintenance.

Pg 216, §2: Implementation of the Fifth Plan Programme:

Many of the issues which were highlighted in the Fifth Plan still remain unresolved. Regarding tariff for Quetta town the situation has slightly improved but it is still far from satisfactory. The water rates for Quetta town have been increased but still the revenues are only a fraction of what is spent on maintenance and repairs. Similarly is the case of rural· water supply schemes. Leaving aside some rural towns where a part of O & M cost is recovered as water tax, all others are entirely subsidized.

Pg 216, §5: Present Position (Rural):

Depending upon the yield of the source daily amount of water supplied per capita varies from 5 to 15 gallons.

Pg 216 §7: Present Position (Urban)–Quetta:

Water availability per capita per day varies from 5 gallons to 14 gallons. In most parts of the city, specially central areas, although piped water is available, yet it is not safe.

Pg 217 §4: Criteria:

Regarding selection of new schemes for rural areas the following criteria have been laid down:

i. Distance and difficulty of access from the existing sources.
ii. High rate of water borne disease.
iii. Availability of low cost reliable source of supply.
iv. Size of the village.
v. Unit cost per capita.
vi. Willingness of the community to pay operation and maintenance cost.

Pg 218 §4: Revenues (Rural):

In the small rural villages water is supplied through community tanks and stand pipes and no water rate is charged from the consumers.

Pg 218 §5:

In the small and medium towns house connections are provided and Rs. 500 is charged as connection fee. The monthly water rates for domestic and commercial connections are Rs. 20 and 40 per month. Statistics on revenues collected from water rates are not readily available. However there is not the least doubt that incomes are only a minor fraction of the expenditures.

Pg 219 §2:

During the Sixth Plan where high proiority has been assigned to the water supply sub-sector, the expenditure on O & M of of water supply schemes would increase considerably.

Pg 219 §3:

Being conscious of the situation, the Government of Baluchistan has constituted a working group to submit its recommendation along the following terms:

(a) Examine the advisability of levying a land betterment tax in Baluchistan.

(b) Examine ways and means of charging water rates in the urban and rural water supply supply schemes of the Province with a view to recovering the capital and/or maintenance costs of these projects.

Pg 227 §1: Institutional Arrangements:

Only a year ago some water supply schemes in rural towns like Loralai, Pishin, Chaman and some on the periphery of Quetta have been transferred to respective Town Committees or the Municipal Corporation. These water supplies are almost fully subsidised and the maintenance continues to be charged to the revenue budget of the Irrigation Department. In future more local bodies are going to take over the maintenance responsibilities of rural water supply schemes. But the arrangement of operation and maintenance of water supply schemes which have been handed over to local authorities is not satisfactory as they are lacking in technical know-how. The new policy envisages that with the passage of time the local authorities would improve their income and technical know-how to the extent that they could run the water supply schemes without depending upon the Government.

* * * * *

ANNEX X

LISTING OF "MODULES"

The following is a listing of the standard drawings (Modules) which form part of this Design-and-Construction system. Please note that it will only be correct at the time of going to press, since it seems likely that users will add to the set, drawings which they deem to be required in addition. However, it is this set that will be available from UNICEF.

Drawing		**Title of Drawing**
		Ground-water Schemes
001		Pump House Details, Sectional Elevations
002		Pump House Details, Plan and Roofing
003		Pump House Details, Power Board and Misc Details
004		Pumping Plant Assembly, Detail: Deep Well Turbine
005		Pumping Plant Assembly, Detail: Single Stage Centrifugal
006		Booster Pump Assembly Details: Multi-Stage Centrifugal
007		Ground Level Storage Tank/Booster Pump Bay: Gen Arrgt
008	*	11.4 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
009	*	11.4 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
010	*	22.8 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
011	*	22.8 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
012	*	34.1 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
013	*	34.1 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
014	*	45.5 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
015	*	45.5 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
016	*	68.3 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
017	*	68.3 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
018	*	91.0 m^3 Ground Level Storage Tank, Reinfrcmt Details 1/2
019	*	91.0 m^3 Ground Level Storage Tank, Reinfrcmt Details 2/2
020		Groundwater Supply: General Arrangement
		Surface-water Based Schemes:
021		Canal Intake, Surface Water Supply: General Arrangement
022		Infiltration Gallery^A
023		Canal Intake Structures
024		horizontal Roughing Filter: General Arrangement
025		Pre-Sedimentation Basin: Plan and Sectional Views
026		Pre-Sedimentation Basin: Sects. and Inlet/Outlet Pipe Detail

LISTING OF MODULES

027		Slow Sand Filter: General Arrangement: 25 m^2 Plan Area
028		Slow Sand Filter: General Arrangement: 50 m^2 Plan Area
029		Slow Sand Filter: General Details and Information
030	*	25 m^2 Slow Sand Filter: Reinforcement Details, Sheet 1/2
031	*	25 m^2 Slow Sand Filter: Reinforcement Details Sheet 2/2
032	*	50 m^2 Slow Sand Filter: Reinforcement Details, Sheet 1/2
033	*	50 m^2 Slow Sand Filter: Reinforcement Details, Sheet 2/2
034		Clearwell/Booster Pump Bay: General Arrangement
035	*	11.4 m^3 Clear Well: Reinforcement Details, Sheet 1/2
036	*	11.4 m^3 Clear Well: Reinforcement Details, Sheet 2/2
037	*	22.8 m^3 Clear Well: Reinforcement Details, Sheet 1/2
038	*	22.8 m^3 Clear Well: Reinforcement Details, Sheet 2/2
039	*	34.1 m^3 Clear Well: Reinforcement Details, Sheet 1/2
040	*	34.1 m^3 Clear Well: Reinforcement Details, Sheet 2/2
041	*	45.5 m^3 Clear Well: Reinforcement Details, Sheet 1/2
042	*	45.5 m^3 Clear Well: Reinforcement Details, Sheet 2/2
043	*	68.3 m^3 Clear Well: Reinforcement Details, Sheet 1/2
044	*	68.3 m^3 Clear Well: Reinforcement Details, Sheet 2/2
045	*	91.0 m^3 Clear Well: Reinforcement Details, Sheet 1/2
046	*	91.0 m^3 Clear Well: Reinforcement Details, Sheet 2/2

General Drawings:

047	Community Tank and Standpost Detail
048	Pipe Trenching Details
049	Valve and Chamber Details
050	Improved Dug Well: Twin Handpump Assembly
051	Improved Dug Well: Single Handpump Assembly
052	Spring Capping Detail
053	Break-Pressure Tank Detail

Sanitation:

054	Sanitation: Ventilated Improved Pit Latrine
055	Sanitation: Twin Pit Pour-Flush Latrine

Notes: 1. All Dwgs marked with an * have associated bending schedules.

2. The Dwg of the Infiltration Gallery[A] will be renumbered. It does not belong with surface-water based schemes.

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ANNEX XII

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Please note that a very large bibliography exists for the pour-flush type of latrine; but, to avoid complication, none is presented. UNICEF, Islamabad or Quetta may be contacted for further assistance.

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Because the Manual is not for sale, copyright appears not to be a problem. However, it is noted that certain standard texts were used extensively. They are noted in the bibliography and, where appropriate, in the text as well. Their authors will be pleased to note that many copies of their publications appear to have been ordered as a result.

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K. Gibbs
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