



WATER MALAYSIA '92

8TH ASPAC - IWSA REGIONAL WATER SUPPLY CONFERENCE & EXHIBITION

PUTRA WORLD TRADE CENTRE
KUALA LUMPUR, MALAYSIA
26 - 30 OCTOBER 1992

TECHNICAL PAPERS VOLUME 4

CONTAINING PAPERS TO BE PRESENTED ON
30 OCTOBER 1992

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AIDE



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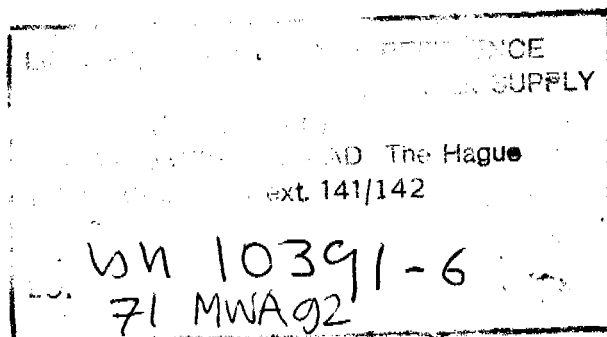
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FOR SESSIONS

11A	11B	11C
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TECHNICAL SESSION 11A
SALINATION AND DESALINATION

The Risk of Saltwater Intrusion at the Tanjung Mas Well Fields, Kelantan

Desalination for Supplemental Water Source Development

Solar and Wind Powered Desalination for Remote Communities

THE RISK OF SALTWATER INTRUSION AT THE TANJUNG MAS WELLFIELDS, KELANTAN

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ABSTRACT: In situations of increased and continuous pumping to meet escalating demand for quality groundwater in Kota Bharu, the problems of saltwater intrusion from the sea into the multi-layered aquifer system at Tanjung Mas wellfields become imminent. An analysis of the extent of such migration risks is presented. Field observations show that there was significant vertical leakage from the 2nd aquifer with comparatively little or no vertical leakage from the underlying 4th aquifer while data analysis based on a computer modelling predicts that horizontal saltwater intrusion was occurring extremely slowly in the third aquifer. The results of the study can be used as an early warning to more serious problems of saltwater intrusion that could result from extensive future withdrawals.

INTRODUCTION

Malaysia's most important wellfields are located in coastal alluvia where the threat of saltwater encroachment always exists. The largest wellfields are located in North Kelantan where a 4-layered aquifer system has been recognised (Chong et al, 1977). Among these aquifers, at Tanjung Mas, the 3rd aquifer has been identified as one having a potential of producing the largest water quantity (with acceptable quality) due to its enormous length and thickness. Recent production rates had reached 12740 m³/day without evidence of saltwater contamination. However, with continuous and increased pumping operations, the pressure of groundwater around the pumping position will be lowered and consequently saltwater may gradually intrude towards the pumping well from the sea. This low pressure phenomenon around the wells may in turn induce leakage of groundwater from the upper or lower aquifers through semipermeable layers. Any presence of brackish or saline water from these neighbouring aquifers can alter the water quality in the third aquifer.

The study aims to investigate the saltwater movement in the multilayered aquifers from the sea and the leakage induced by pumping stress in the 3rd aquifer using data from field study and monitoring programs which are being carried out by the Geological Survey Department. UTM researchers, in part, collaboratively monitor and analyse the relevant data for aquifer performance of groundwater quality. Also will be presented in the paper are the results of risk analyses on the extent of saltwater encroachment in the multilayered aquifer system at Tanjung Mas based on the Galerkin finite element simulation of a two-dimensional fresh-saltwater interface model.

STUDY AREA

Tanjung Mas wellfields are located about 5.0 km inland from the South China Sea coastlines (Figure 1). They are surrounded by Sg. Pengkalan Chepa and Sg. Kelantan on the west, respectively 1 km and 4 km at the nearest point; a creek flowing just behind the wellfields area joins the Sg. Pengkalan Chepa,

and Sg. Pengkalan Datu 4 km to the east. The study area is partly urbanised with commercial activities, housing estates and small scale industries. Some traditional agricultural practices are still in progress in the area. The town centre of Kota Bharu is located 3 km away from the sites. An abandoned waste dumping site, which once served the entire Kota Bharu Town, is about 2.0 km to the east.

The average annual local rainfall is about 2700 mm, 60% of which fall in the months of October to January under the influence of the north-easterly wet monsoon. January - May can at times be very dry while October - December can have the most consistent rainfalls. The average annual evaporation is 1700 mm, making the area conducive to vertical infiltration and associated leachates. The coastal plain, towards north of Pengkalan Chepa and Sabak is predominantly flat with drainage northwards to the sea. Most of the area is drained by small rivers leading to the main river and eventually to the coast. Most rivers are continually fed by the shallow groundwater aquifers especially in dry seasons. Past river flows and salinity measurements have shown that saline water can extend up to 15.4 km and 6 km of Sg. Kelantan and Sg. Pengkalan Datu respectively during the worst low river flows in the drought months of March - May.

HYDROGEOLOGY OF THE SITE

In general the shallow groundwater is fresh and of suitable quality for domestic use. Near the coast and the estuarine lower reaches of the river where the groundwater is saline, the usual fresh/salt water interface would be found. Recharge usually occurs by local rainfall infiltration and its condition is enhanced by high rainfalls, relatively porous surface soils, low flat topography and ponding of water. Watertable lies at 1.0m - 6.0 m below groundwater surface over most of the area. The piezometric levels in both the 2nd and 3rd aquifers indicate that flows are towards the South China Sea with gradients 1: 3200 and 1: 6500 respectively as shown by contours from the data observed by Chong et al, 1977. The water level is generally higher in the 2nd aquifer in inland areas.

The quaternary alluvium of Tanjung Mas, 152.0 m thick, consists of unconsolidated gravel, sand, silt and clay. It is discontinuous over the areas either further inland or towards the coast. The nature and thickness of the aquifers vary from place to place. Aquifer 3 indicates a potential of producing the largest water quantity due to its length and thickness. Aquifers 3 and 4 are suspected to be hydraulically connected to the offshore deep ocean bed. The salt-freshwater interface usually extends over a much greater distance inland as the aquifer becomes deeper unless in places where aquifer ocean bed boundary is far off the coast. The 1st aquifer is semi-confined, having a static water level of about 2.8 m above MSL. The 2nd and 3rd aquifers are confined with piezometric levels of 1.5 m and 1.8 m above MSL respectively. The piezometric level of the 4th aquifer has not been established (Chong et al, 1977).

The vertical cross section along line A-B-C in Figure 1 is schematically shown in Figure 2. It illustrates the multilayered aquifer system, referred to as the 1st, 2nd, 3rd and 4th aquifers from the top. The 2nd, 3rd and 4th are confined aquifers bounded by semipermeable layers or bedrock. There is evidence to show that the water in the 2nd and 4th aquifers is brackish and saline, whereas that in the 3rd is fresh. Measured chloride ion concentrations in the 2nd aquifer were 1200 mg/L and 640 mg/L at the distance 2.5 km and 6.5 km inland from the coast, respectively. The chloride content of the groundwater in the 1st aquifer is around 17.8 mg/L and is in the region of 10-65 mg/L in the 3rd (Chong et al, 1977).

The Tg. Mas wellfields are withdrawing water from the 3rd aquifer from depths 50 to 130 m where groundwater abstraction began after its first exploration in 1976. From 1980 to 1988 the water production at Tg. Mas increased to the order of 110 - 130 m³/h. The wellfields presently have nine 12" wells installed at depths from 50 to 95m. The six older wells with depths ranging from 50 - 60m initially yielded 80 - 90 m³/hr with an average drawdown of 28.0 m. However, the production from these wells has since deteriorated and 3 new wells with depths from 92 to 95 m were constructed in early 1985 to augment the yield, each with a production capacity of 90 m³/hr and drawdowns between 10 and 24 m. The transmissivity in this part of the 3rd aquifer was computed to be 58 m²/h. Recent city demand had actuated pumping of fresh water from the 3rd aquifer to some 11000 m³/day, a rate that could possibly invite saltwater contamination in the aquifer by virtue of leakage through the semi-permeable layers

possibly from the over-lying and under-lying aquifers.

INTRUSION ASSESSMENT

Groundwater quality can be adversely altered by a large number of pollution sources, of which saltwater intrusion from the sea is a most difficult problem to solve. Since the water from the 3rd aquifer is drawn at 95 m depth which is in close proximity (5km) to the coast, major problems posed by saltwater intrusion can be expected. Intrusion of saline water normally occurs where saline water displaces or mixes with fresh water in the aquifer. In the deep 4th aquifer the saline water of geologic origin could possibly be present. In shallow parts of the aquifer, saline water may also commonly be formed from water concentrated by evaporation in tidal lagoon, return flows to streams from irrigated lands, and saline waste. In coastal regions, both shallow and deep aquifers are usually intruded by seawater.

To investigate the impact of saltwater migration in the multilayered aquifers due to natural or forced hydraulic gradients, it is essential to monitor and model the groundwater system. The monitoring scheme can be used as an early warning to more serious problems, principally serving the needs of groundwater protection measures for the 3rd aquifer through observation of the water quality and piezometric head conditions. The saltwater intrusion risk assessment presented herein is based on field study and computer modelling.

Field Study and Monitoring

At present, none of the operating wellfields in North Kelantan is in any immediate danger of saline water intrusion; but the situation can reverse without management skills. So far, only the general settings of the fresh/saltwater conditions are known and to protect these wellfields a monitoring program was introduced and implemented in 1989 by the Geological Survey Department (GSD), Malaysia. The program is to establish long-term information and to check any changes occurring in the environmental impacts as a result of changing boundary conditions and stresses. The system is so designed as to permit sampling and measurement of hydraulic parameters at horizons corresponding to the 2nd aquifer monitoring wells to check sources, degree of pollution and pressure head of possible leakage contribution to the 3rd aquifer. The 3rd aquifer monitoring wells are capable of measuring current degree of saltwater pollution from all possible pathways.

The monitoring wells are operated at three different sites within the district of Kota Bharu namely at the Tanjung Mas wellfields, Pengkalan Chepa and Jalan Merbau (Figure 1). Multiple wells systems with varying penetrations were used for monitoring different aquifer layers (Figure 3). To ensure water samples are representative of the formation water, sampling was accomplished through the use of bladder pumps by evacuating stagnant water before samples were taken at a pumping rate of 4 L/min. This was achieved by pumping to waste approximately four times the volume of water in each well.

Along the section parallel to the shore, the groundwater is found saline in shallow places of the 2nd aquifer where there exists freshwater. However, freshwater throughout the whole section exists in the 3rd aquifer. Generally, the influence of the sea is restricted to less than 6.4 km inland from the shore (Flathe, 1975). Its seasonal changes are not known but the salt-freshwater interface will generally move further inland. An approximate salt freshwater boundary in the 2nd aquifer based on the data presented by Flathe (1975) is shown in Figure 1.

The first series (Feb., 1989) of groundwater monitoring analyses was interpreted in order to attempt a provisional assessment of the contamination risk for the Tg. Mas wellfields by Sofner (1989). Figure 4 shows the lateral and vertical distribution of chemical types of groundwater along the cross-section running from Pengkalan Chepa through Tg. Mas to Jln. Merbau. It indicates a mobilisation of brackish water into the depression cone of the production wells at Tg. Mas. The figure illustrates, in the increasing order of aquifer depth (1) a section of low groundwater mineralization down to about 15 m showing chloride < 30 mg/L (2) an intermediate section with chloride concentrations between 100 mg/L and

> 1500 mg/L, indicating the water is normally composed of Na, Mg, and Ca-Cl, and (3) a deep section of a very low to low groundwater mineralisation with chloride concentrations between 3 and 6 mg/L indicating a Na-HCO₃ type of water in the south, and a chloride concentration of 25-30 mg/L indicating a Na/Mg/Ca-Cl/HCO₃ type of water in the north, deduced from consideration of an intermediate position between the HCO₃ type and the overlying Cl type. The mixing process of the different water types becomes possible as a consequence of varying groundwater gradients towards the production wells.

At the Tg. Mas monitoring site for the multiple wells (Figure 3), the chloride concentration of 46 mg/L in the upper part of the production horizon of the 3rd aquifer (well B2) indicates a mobilization of brackish water into the fresh water domain from the northern intake area of the wellfields. The chloride concentration of 176 mg/L at Pengkalan Chepa for the same aquifer (B2) is formed possibly due to natural hydraulic gradients. Temporal variations of TDS and Cl at Tg. Mas (Figure 5), as deduced from multiple monitoring wells, show positively that the problem of the brackish water intrusion does exist at Tg. Mas wellfields and is continuous over the period 1989 - 1991. However, the slight increase of chloride concentration from 30 to 45 mg/L so far in the raw water produced at the treatment plant does not as yet endanger the drinking water quality.

Water quality data observed from pumping well water samples (Figure 6) show not only temporal variations for TDS and Cl at Pengkalan Chepa and Tg. Mas but also depict seasonal changes which could be due to varying hydraulic interactions of the aquifer, tidal effects and outcrop recharge zones upstream or local recharge. However, no saltwater movement due to pumping stress can be traced.

An estimation of leakage of brackish and saline water due to pumping stresses in the multilayered aquifer was carried out. Using the 1985 pumping operation data at Tg. Mas (At $Q_w = 270 \text{ m}^3/\text{h}$, the drawdown was 20m below MSL) and static conditions data (piezometric head = 2.25m for the 2nd aquifer and 2.16m for the third aquifer, after Chong et al, 1977) and assuming soil permeability, $k = 10^{-5} \text{ m/s}$ and aquifer thickness = 6m for the semiconfining layer, vertical recharge rates were estimated at 352 m³/day for an area of 100 m². This 2nd aquifer is known to have brackish and saline water with a tendency to pollute the fresh water in the 3rd aquifer through leakage in the semi-confining layers as a result of piezometric gradients set up as explained above due to the natural and/or forced hydraulic gradients. The field observation on the vertical recharge is in good agreement with the theory as described by Nawang (1986) in which leakage is maximum at the pumping axis and gradually approaches zero towards the boundary of no hydraulic gradient.

Computer Simulation

With changing boundary conditions in the 3rd aquifer, there is an apparent need for monitoring in order to permit an estimation of the extent of saltwater migration resulting from continuous and increased withdrawals to meet future water demand. A practical tool to better define and predict this migration problem is to employ mathematical modelling.

To analyze the saltwater movement in the aquifer, approximate equations based a fresh-saltwater interface model are used (Nawang and Kishi, 1990). Model solution requires an assumption of a constant thickness throughout the aquifer and a fixed side boundary, in this case 5km to the east and 5km to the west from Tg. Mas. The area as defined by these boundaries is divided into 960 triangular elements with a total of 527 nodal points. A Galerkin finite element method is used to obtain the numerical solutions.

In order to determine the spatial distribution of fresh-saltwater interface and piezometric head of groundwater in steady state, it is necessary to know the magnitude of hydraulic conductivity, k , of the aquifer and inflow rate of groundwater, Q , from the upstream boundary conditions. The hydraulic conductivity of the 3rd aquifer approximated by pumping test has been shown to be 0.0184 cm/s (15.9 m/d) (Chong et al, 1977). A similar coefficient value is assumed for the 2nd aquifer.

The heights of fresh-saltwater interface along $y=0$ line and positions of the toe of the interface are shown in Figure 7 for the 3rd aquifer. Without pumping, the position of the toe of the interface is located very

near to the outlet of the groundwater far off the coast. At a pumping rate of $Q_w = 11000 \text{ m}^3/\text{d}$, $x = 7 \text{ km}$ and $y = 0$ (Tg. Mas wellfields), the position of the toe of the interface considerably intrudes inland. The degree of intrusion gets much larger when Q decreases; At $Q = 12300 \text{ m}^3/\text{d}$, the interface extends inland beyond the pumping station and the upconing of saltwater occurs around this position.

Time dependency of groundwater level and fresh-saltwater interface is closely related to the magnitude of factors, S/kd and n/qkd , respectively. Since the storage coefficient, S , is small, usually in the range of $10^{-3} - 10^{-4}$, and $q = 0.025$, S/kd is much less than n/qkd . Therefore, the groundwater level may vary much more quickly compared with the fresh-saltwater interface. Figure 8 shows the variations of groundwater level in the 3rd aquifer after pumping of $11000 \text{ m}^3/\text{d}$ is initiated, with recharge $Q = 12300 \text{ m}^3/\text{d}$, $S = 0.001$ and porosity, $n = 0.04$. Groundwater level is rapidly lowered and approaches unity in the final steady state for $t > 400$ day. Figure 9 shows the movement of the toe of the interface in the 3rd aquifer after pumping initiation, under the same conditions as in the case shown in Figure 8. It also shows that the interface moves extremely slowly and that the toe of the interface moves only by the distance 250 - 500 m during a period of 30 years.

When pumping is commenced in the 3rd aquifer, the drawdown of groundwater level rapidly takes place (Figure 8). The lowering of piezometric head in the 3rd aquifer may have caused the leakage of groundwater from neighbouring 2nd and 4th aquifers through semipermeable layers. Fresh water (from the 3rd aquifer) or saltwater (from the 2nd or 4th) will flow nearly vertically in a semipermeable layer with directions defined by the resulting hydraulic gradients of the respective aquifers. Therefore, the phenomenon of saline groundwater leakage from the neighbouring 2nd and 4th aquifers plays an important role in predicting the quality problem in the 3rd aquifer.

FUTURE MONITORING AND MANAGEMENT

To protect the groundwater resource against saltwater pollution due to more increased pumping stresses, vertical leakage, interface and upconing, monitoring programs based on more strategically located wells for continuous measurements, observation and evaluation are necessary. Monitoring methods may include not only sampling and analyses of groundwater quality, but also determination of groundwater levels for the multilayered system and flow directions through multiple monitoring wells such as Waterloo System (Pianosi and Belshaw, 1990). As for future planning, prediction requires a relatively more detailed data input. Unfortunately, the present field studies cannot provide sufficient data to run the model (for instance, data on the boundary of the ocean bed and multilayered aquifer interconnection; and location of the interface between freshwater and saltwater in those aquifers near the coast). A field hydrogeological study is thus warranted to derive the associated geometries and parameters. In order to minimise the risk of contamination and increase the supply of groundwater, the aquifer may be managed and remediated through artificial recharge and optimal pumping programs. Design and implementation of these programs may vary widely depending on the source of the saline water, the extent of intrusion, local geology, water use, and economic factors. One possible alternative would be using abundant flood waters collected in the area for artificial recharge purposes. Acceptable water quality standards, however, have to be assured for such programs although self purification is normally achieved through porous media flow. All these can be planned with intensive computer modelling based groundwater management.

CONCLUSION AND RECOMMENDATIONS

The computer model predicts somewhat slow horizontal intrusion of saltwater from the sea bottom to the 3rd aquifer even at increased pumping. Field observations show the existence of a mobilization of brackish water from the 2nd aquifer into the upper part of the 3rd aquifer due to both natural and forced hydraulic gradients. However, no occurrence of significant contamination has been identified in the 3rd aquifer at Tg. Mas. The results of the study can be used to assist, in particular, in solving the immediate problem of saltwater intrusion ensuing from extensive future withdrawals and in the overall management of the Tg. Mas wellfields.

ACKNOWLEDGEMENT

The authors wish to thank the Geological Survey Department, Malaysia (Hydrogeology Section), Public Works Department, Malaysia (Water Supply Division) for their continued supports of this project. Thanks are also due to Abd. Rashid Othman from the Environmental Engineering Department, UTM, Skudai for his comments on water quality and the general checking on the manuscript. The computational facilities provided by the Computer Science Center, Ehime University, Japan are greatly appreciated.

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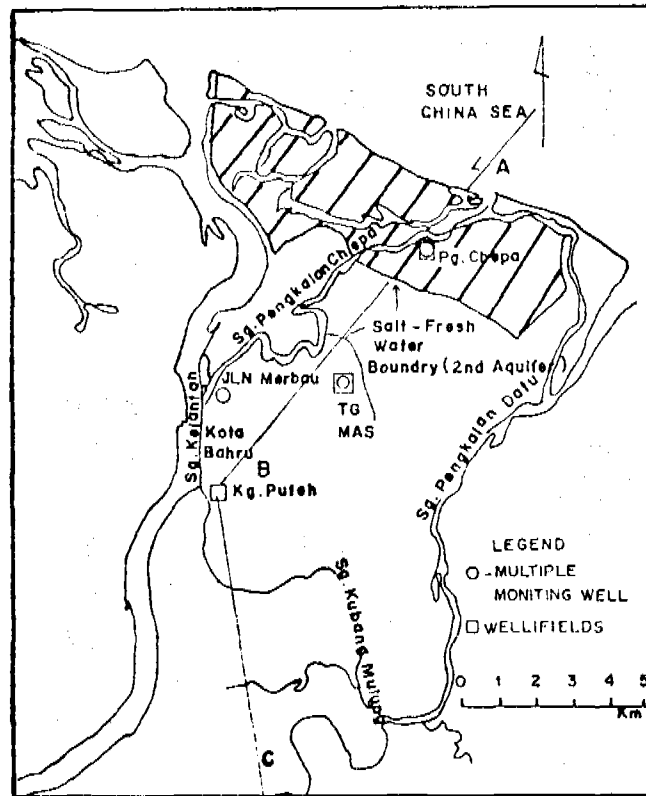


Figure 1 The Study Area

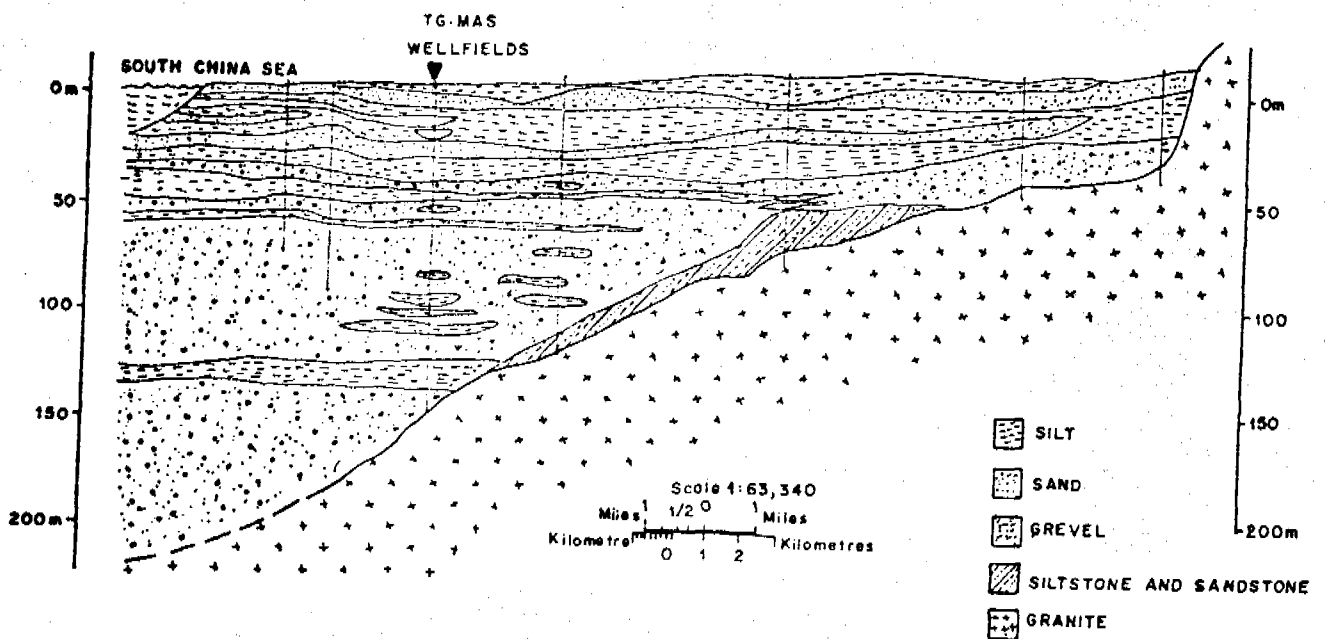


Figure 2 Schematic view of vertical cross section A-B-C of the multilayered aquifer system in Figure 1 (Source: Chong et al, 1977)

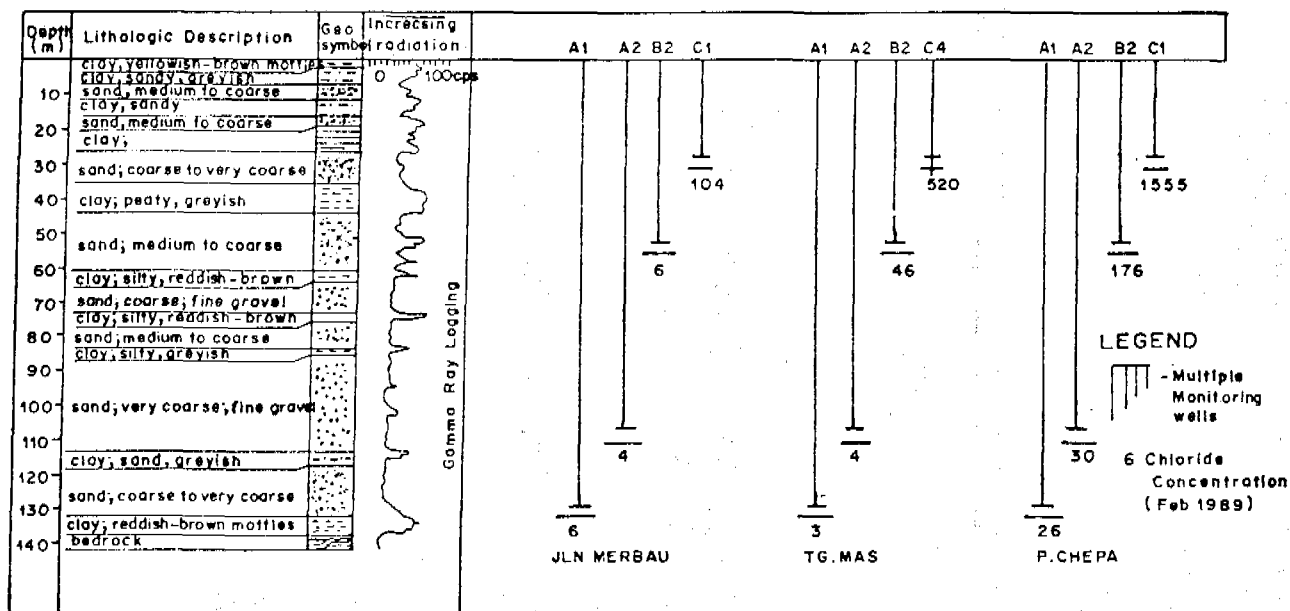


Figure 3 Geologic log and monitoring wells at Jln. Merbau, Tg. Mas and Pengkalan Chepa

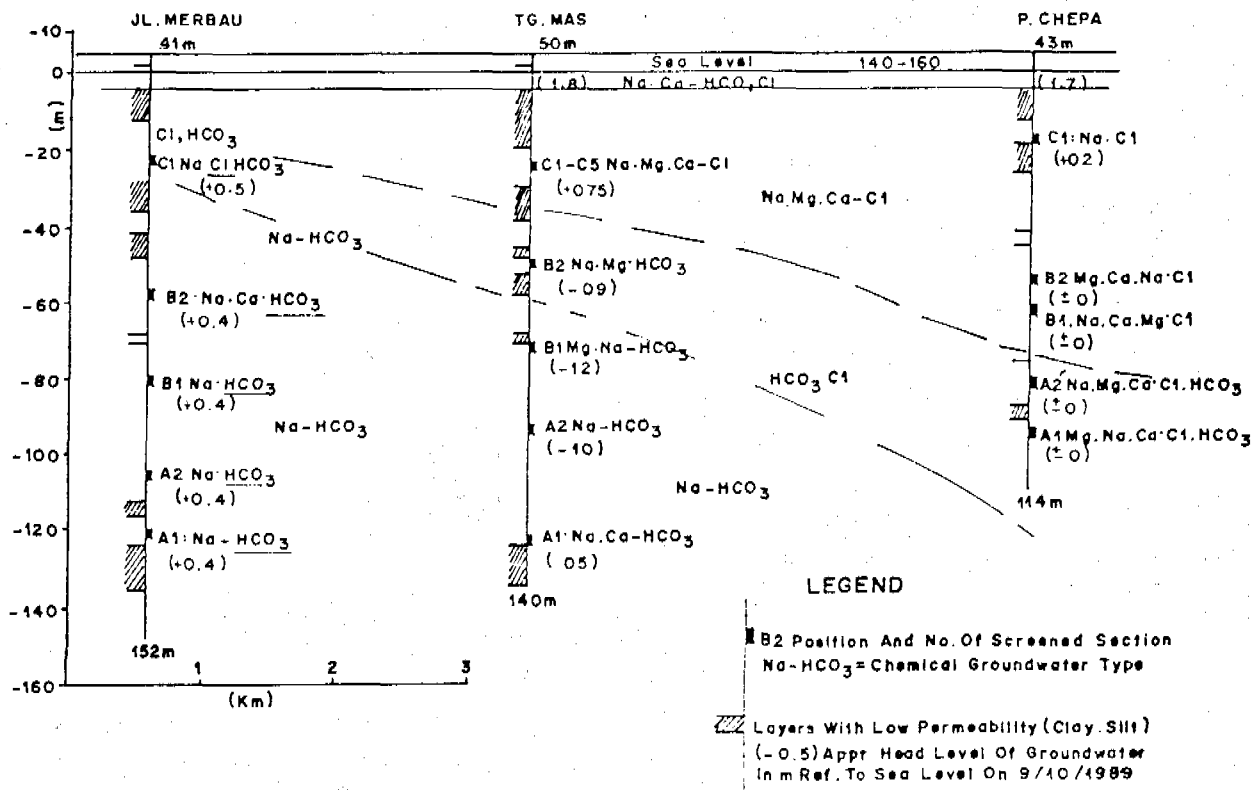
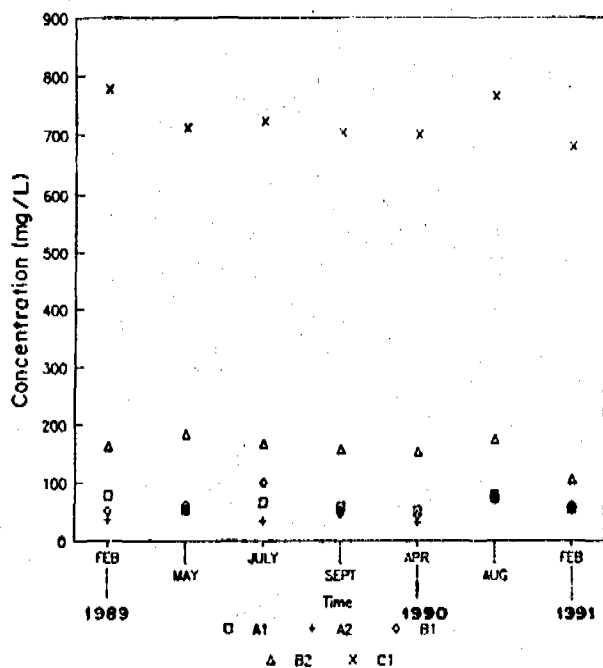
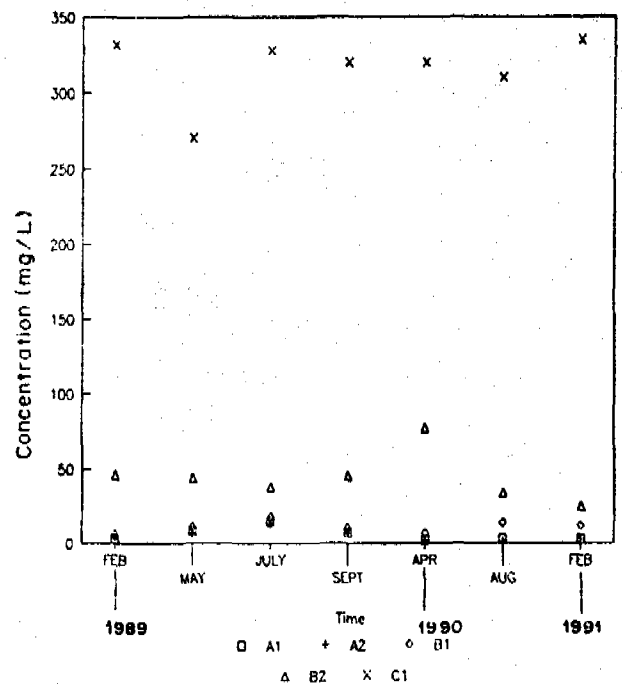


Figure 4 Hydrochemical cross-section of the Kota Bharu groundwater between Jln. Merbau and Pengkalan Chepa (Source: Sofner, 1989)



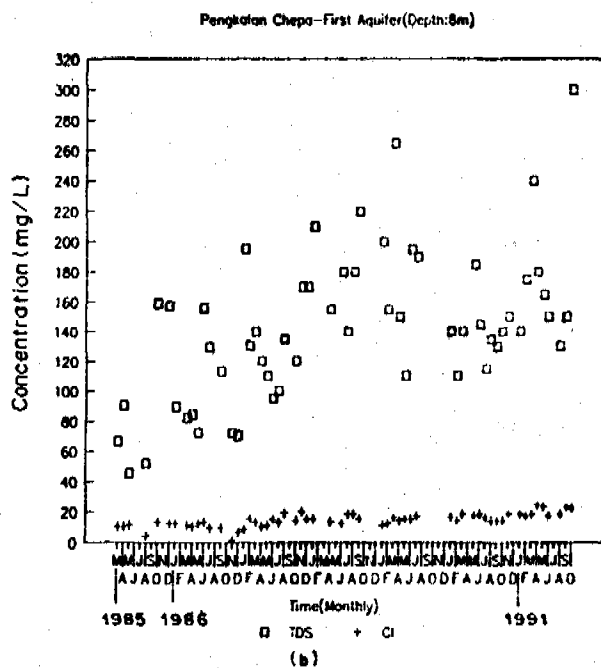
(A) Total Dissolved Solids



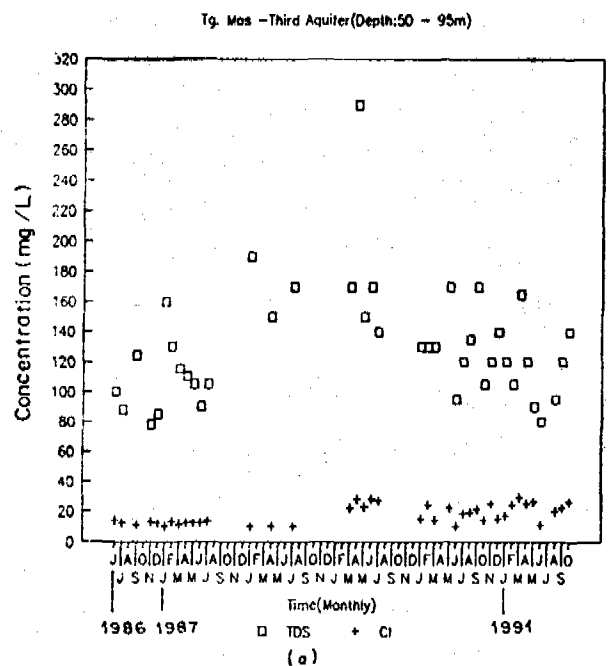
(B) Chloride

Figure 5

Temporal variations of TDS and Cl at Tg. Mas Monitoring wells (Source of data: GSD, Ipoh)



(b)



(a)

Figure 6

Water Quality Analyses for Cl and TDS at (a) Tg. Mas Wellfields and (b) Pengkalan Chepa Wellfields (Source of data: Public Works Department, Malaysia-Water Supply Division)

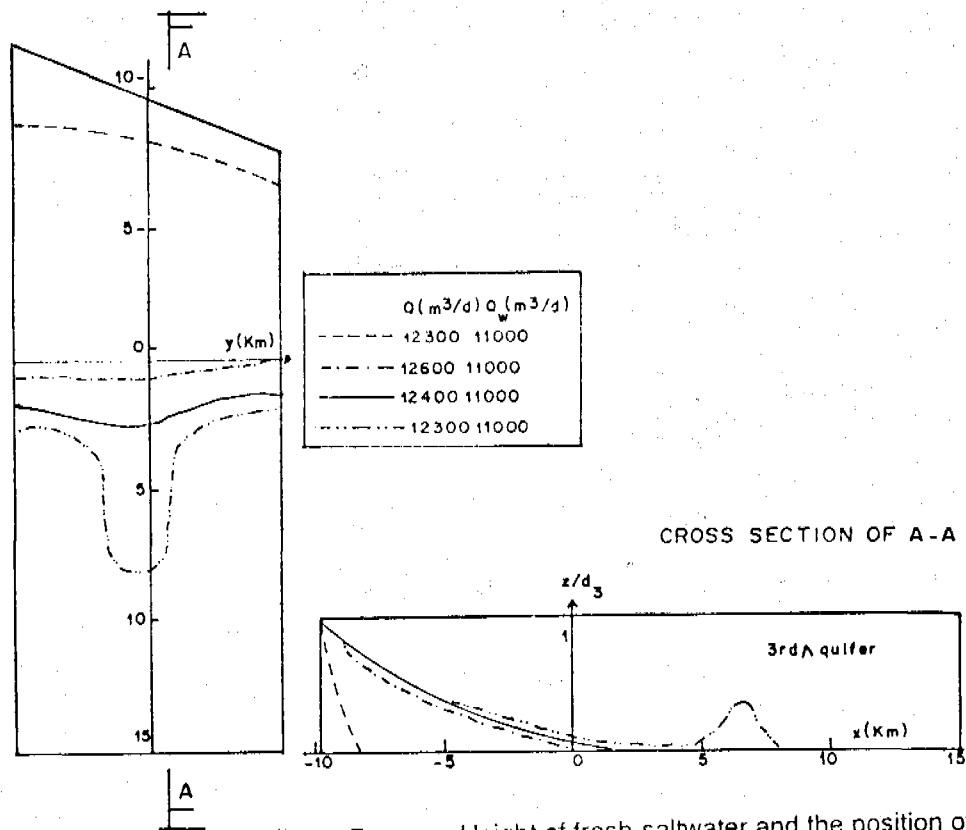


Figure 7

Height of fresh-saltwater and the position of toe of interface in steady state for the 3rd aquifer.

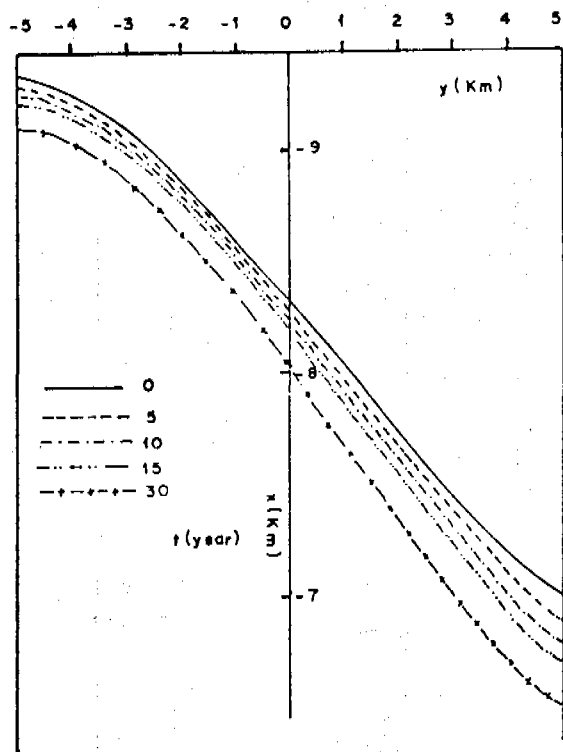


Figure 9

Movement of toe of interface induced by pumping rates of $Q_w = 11000 m^3/d$ and $Q = 12300 m^3/d$ in the 3rd aquifer.

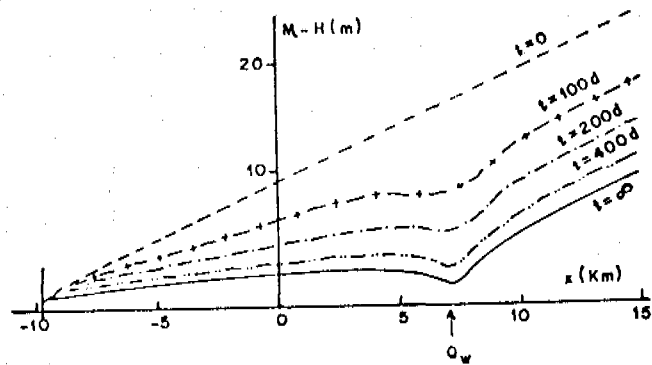


Figure 8

Time variations of groundwater level in the 3rd aquifer induced by pumping rates of $Q_w = 11000 m^3/d$ and $Q = 12300 m^3/d$

DESALINATION FOR SUPPLEMENTAL WATER SOURCE DEVELOPMENT

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ABSTRACT Water supply systems in Japan are increasingly dependent on impounded reservoirs as their raw water sources. As a result, it becomes difficult to develop new reservoirs, requiring a long period of construction, causing social and environmental problems. Due to the established priority system of water right in Japan, new water users or utilities are not necessarily entitled to extract raw water during its minimum flow of the river(Fuantei-shusui). This situation accordingly makes the utilities very keen and sensitive to drought. This paper attempts to depict duration and magnitude of drought-origin water supply suspension, practiced in this country in the past few years.

It is the fact that many utilities currently endeavor to minimize water losses and to develop water resources even under the restricted condition. In some drought-sensitive areas, however, construction of desalination plants are recommended as a supplemental water source to meet the increasing water demand.

The Government has been providing financial assistance to utilities in a form of equity for water resources development. It is the Government's new policy that construction of desalination plants may have a similar privilege as well. This regulation is to be enforced and will become effective from 1992.

This paper also highlights the outline of desalination plant planned to be constructed as means of new water source development in Okinawa island where surface water source is very limited.

1. Introduction

Water supplies in Japan recorded its service-coverage (population-served) ratio at 94.7% of the total population in all over the country, and the annual amount of water supply totaled to 16,000 million m³/d, in FY 1990.

Currently, rapid increase of water supply, such as experienced in 1960-1970, has not been observed. However, water demand is increasing in recent years. Therefore, the intake amount of raw water has also been increasing, see Table-1.

The extracted raw water amount in all over the country with its percentage, by respective water sources, are presented in Fig. 1. The figure shows that the surface water, including river water, dam impounding water and lake water, gets approximately to 70% of the total. While the ground water shares the rest 26% of the above total raw water amount.

However, the availability of the most portion of dam water must comply with the current water right, which prescribes its priority of water usage and reveals that the dam water has weakness or shortcoming in droughts.

The required raw water extraction for municipal water (domestic and industrial water) use, under the above unstable availability of water sources, is estimated to total 3,200 million m³/d, in FY 1990.

And in some area/locally, it proves that the unstable portions amount more than 20% of the total water demands in that area. Therefore, it is required urgently to recover and stabilize this unstable situation of water sources. Further, in recent years, the water supply restriction or rationing is practiced almost every year in some area in the country, and forcing hardship or inconvenience to people's life.

WATER SOURCES 1989

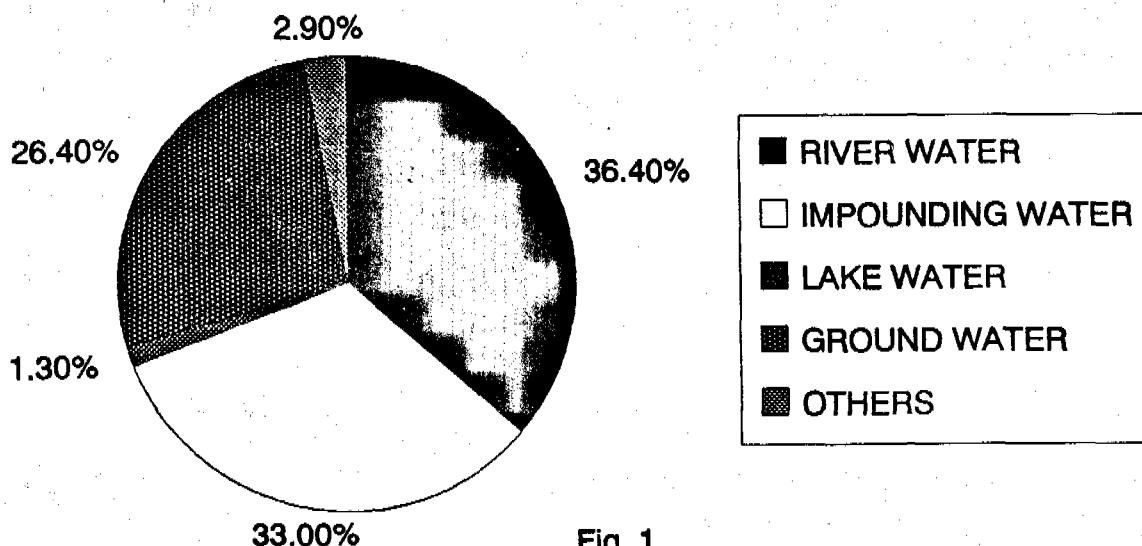


Fig. 1

(Unit million m³)

Year	1965	1970	1975	1980	1985	1988	1989
Water Sources							
River	3,581	4,745	5,804	5,400	4,873	5,785	5,793
(Natural flow)							
Dam Reservoir	756	1,904	2,774	3,645	5,287	4,951	5,250
Lake/Pond	56	88	117	179	190	187	212
Under Flow							
Water	947	1,041	1,072	954	738	672	691
Well	935	1,830	2,494	2,844	3,307	3,431	3,504
Others	378	244	281	473	488	450	463
Total	6,653	9,851	12,542	13,495	14,884	15,477	15,913

Table - 1 Records of the Extracted Water Amounts by Water Sources
in Japan

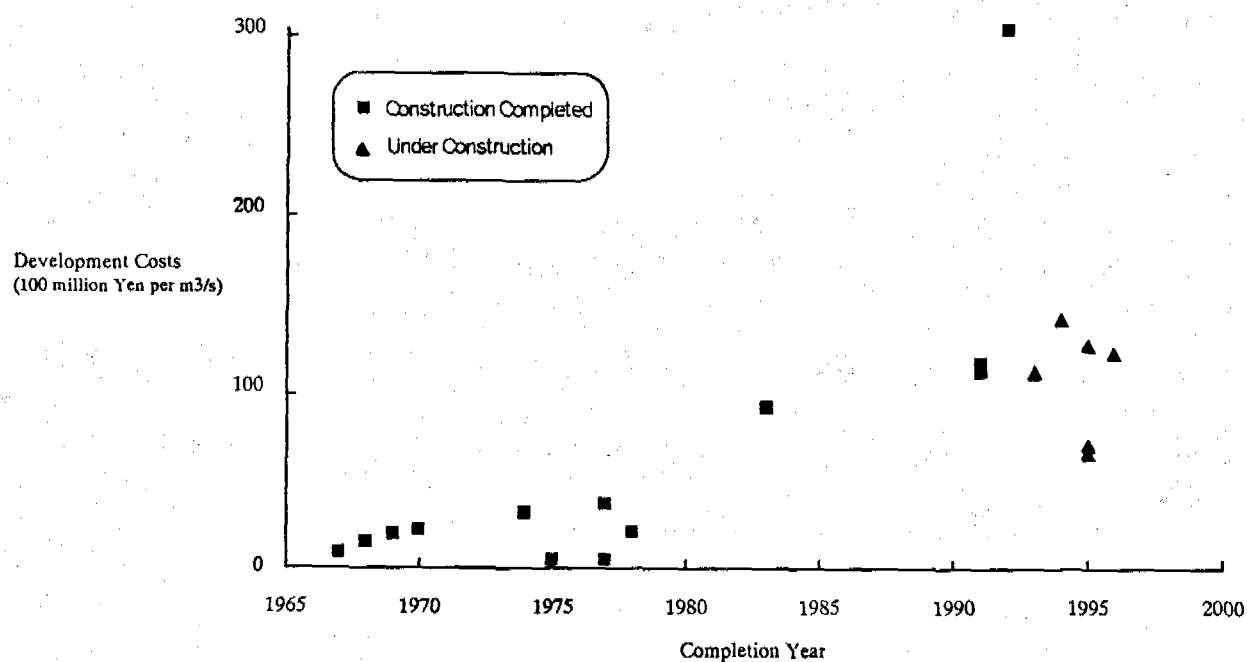


Fig. 2 Dam-Reservoir Development Costs by Completion Year
(Dams Constructed by Water Resources Development Corporation)

2. Measures for Securing Water Sources

For Stable/permanent water supply, appropriate water sources are required to secure and meet the water demands. In Japan, following measures for water sources development have been taken for practice, they are: (1) additional dam reservoir construction, (2) diversion of the present sources usage to water supply, (3) diversification of water sources such as a planning of sea water desalination, and (4) furthermore to increase the amount of effective water by implementing wastage and leakage control.

1) Development/promotion of dam construction

There are two ways to construct a dam for water supply, one is to construct a dam reservoir by the water enterprise or water supply company itself; and, the other is to join a multi-purpose dam system constructed by the river administrator and/or the Water Resources Development Public Corporation. In Japan, the latter case is the majority of the water resources development.

Facilities for water supply are normally constructed with the water tariff revenue. However, because a dam construction usually requires a huge expenditure to a water supply enterprise, the government subsidies are applied under the specific conditions in order to ease the burden. The portion of the government subsidies is, principally, one half or one third of the total construction costs. And the current budgets prepared for F/Y 1992 total approximately to 72 billion Yen.

2) Measures for leakage control

The ratio of effectively used water to the total distributed water is defined as the ratio-of-effective-supply. In Japan, that of the average ratio is achieved to 89%, in 1989. This suggests, on the contrary, that the water amount about 2 billion cubic meters is wasted in a year. And, the most part of the wastage is considered/estimated as leakage. Therefore, the effects which finally create water production increase through the leakage control, are considered as the great accomplishment.

3. Introduction of Sea Water Desalination

In Japan, the sea water desalination plants have been applied, until now, mainly to small-scale water sources for remote islands, and a part of industrial water supplies. As for desalination-system for water sources in remote islands, distillation had been adopted. However, in recent years reverse-osmosis system has increased its installation cases.

Table-2 shows the desalination facilities which are installed at remote islands in Japan. For these facilities, the government subsidies were

Table-2 Sea Water Desalination Facilities in Japan
(Ministry of H & W Subsidies Portion)

1990 end at present

Prefecture	Designated Area	Island	City/Town	Capacity of Desalination System			Commissioning Year
				Raw water	Production Amount	Method	
TOKYO	Izu Is.	Oshima	Oshima	Brackish water	m3/day 1,300	Electro dialysis	47, 49, 57
	-ditto-	-ditto-	-ditto-	-ditto-	2,500	-ditto-	1990
EHIME	Ochi Is.	Hakata	Hakata	Sea water	300	Reverse Osmosis	56
	Kutsuna Is.	Nokutsuna	Nakajima	-ditto-	20	-ditto-	60
	-ditto-	Tsuwaji	Nakajima	Brackish water	75	-ditto-	54
FUKUOKA	Chikuzen Is.	Ainoshima	Kitakyushu	Sea water	50	Reverse Osmosis	60
NAGASAKI	Hirado Is.	Mushima	Ojika	Sea water	30	Reverse Osmosis	54
	-ditto-	Ojika	-ditto-	Brackish water	585	-ditto-	55
OKINAWA		Kitadaito	Kitadaito	Sea water	240	Reverse Osmosis	59
		Minamidaito	Minamidaito	Brackish water	150	Electro dialysis	49
		-ditto-	-ditto-	-ditto-	150	Ion-exchange	53
		-ditto-	-ditto-	Sea water	300	Reverse Osmosis	1990
		Tonaki	Tonaki	-ditto-	240	-ditto-	62
		Aguni	Aguni	Brackish water	400	-ditto-	62
		Hateruma	Taketomi	-ditto-	240	-ditto-	1989

(Note) Stopped & Closed System are not included

applied as a part of small-scale community water supplies. Therefore, the government subsidies are applied to its construction works of all facilities, including the intake works of water sources to distribution pipelines.

In recent years, a need for desalination plant/facilities installation for large-scale water supplies in Japan has increased, in addition to remote islands.

To cope with the above situations, it is necessary to consider and study the introduction of a desalination system which has the following advantages:

- (1) no threat of droughts exists.
- (2) requires rather short period of construction.
- (3) requires less construction site areas, and
- (4) requires less construction costs for water transmission facilities, in case that the water demand area is close to the sea coast.

On the one hand, a reverse-osmosis-method system has become more advantageous for sea water desalination, since it requires lesser energy cost. Therefore, desalination system application is being realized now in Japan for medium and large-scale water sources development, since Japan does not have oil resources.

Further, regarding a reverse-osmosis system itself, following technical developments are achieved or observed now:

- (1) improvement of the rate of pure-water production and/or collection per unit area of the membrane, by increasing the amount of transmitting water through the membrane, and
- (2) increasing of the life of membranes, by improving its washing method.

It can be said that the total project cost, initial and running costs, for dam-reservoir water production and that of sea water desalination have become almost the same level. It can be analyzed from the above backgrounds, which are reflecting and depending on the situations of the local or regional conditions.

The unit water production cost of a sea water desalination system, it is approximately calculated at 260 Yen/m³, for the under-planning plants/facilities of the Public Works of Okinawa Prefecture, whose details are in the succeeding section.

While, that of dam water is calculated, experimentally under the following conditions, at 210 Yen/m³ which shows about 80 % of a sea water desalination system:

- Development cost of a dam, about 19 billion Yen/m³/sec,
Construction cost of water transmission and treatment facilities, about 25 billion Yen/m³/sec, and
- Depreciation term, 80 years.

The above costs comparison is made under an estimation of the dam construction costs in Honshu Island in Japan. On the other hand, in case of Okinawa Prefecture which has a tendency of much higher dam construction costs. Therefore, a sea water desalination facilities is considered to be viable, from the points of view of its developing costs in Okinawa.

From the above backgrounds, sea water desalination-system facilities can be incorporated as the facilities for water resources development. Therefore, it is also identified as a project to which the government subsidies could be applied, from the fiscal year of 1992, as well as the dam construction project.

4. Necessity of Introduction of the Sea Water Desalination-System Facilities in Okinawa Prefecture and its Project Plan

In Okinawa Main Island, the current water demands are in a tendency of increasing more and more, due to the population increase, improvement of its economy, improvement of the level of living standard, and increase of the tourists.

The average annual rainfall in this Prefecture is estimated approximately at 2,040mm. The rain water flows out to the sea instantaneously, due to the topographical conditions; the catchment area in Okinawa is small, and the length of river flow basin is short with rather steep slope.

Further, if there is no rainfall even for a short period, it becomes difficult to secure the stable water extraction. Since, it becomes easily in drought condition if there is no rainfall in the catchment area, and the annual rainfall concentrates in summer. Further, the yearly variation of rain fall is also large.

Further, location of water resources are concentrated to the northern part of the Island. Therefore, it is situated under a fairly unfavorable natural conditions to secure the water, or the water must be transmitted for a long distance to the city of Naha and other populated areas, which are located in the central and southern part of the Island.

In Okinawa, from the above reason, droughts have been frequently experienced. In the past 20 years, water supply rationing with restricted supplies were practiced in 13 years(times). Its total period of rationing were so far totaled to 1,099 days. Recently, in 1991, the water supply was restricted for the duration of 64 days from June to September.

Under such backgrounds, in Okinawa, the urgent measures are required to be taken to get the stable water sources. And, therefore, it is planned to install the sea water desalination facility with a capacity of 40,000m³/d as a part of the multi-purpose water sources development program, aiming at stable water supply.

1) Position of the sea water desalination facilities

For the water supply in Okinawa, as of the present actual status in 1991, the water served ratio or water supply coverage ratio (served population / total population) reached to 99.6 percent, or the served population totaled to 1,226,536, with the daily average water supply amount of 487,565 m³/d (or max. day supply of 562,865 m³/d).

As a water supply enterprise, the Public Works of Okinawa is supplying water to 30 cities and towns in the Main Island, out of the total 53 cities/towns and villages. In 1991, its average daily supply totaled to 399,000m³/d (or max. day supply of 459,400m³/d) which covers 80% of the total demands in Okinawa Prefecture.

Regarding the water extraction amount of each water source, 29.4% of them was river water, 58.4% the dam reservoir water, and 12.2% ground water, respectively.

And at present, an approximately 30% of the raw water is relying on the unstable river water, out of the total amount. In the long-term plan of the water supply enterprise, the daily max. water supply amount of 583,000 m³/d (which requires the water sources amount of 606,000 m³/d) is estimated/ planned, targeting the year 2001.

Following this plan, a multi-purpose dam construction program, with the river water and ground water development plan is under planning. However, the shortage of water amount at water sources is anticipated. Therefore, the sea water desalination-system development, with a capacity of 40,000m³/d, is concurrently planned as a part of the multi-purpose water sources development program.

2) Planned Project (See Flow Chart in Fig. 3)

- | | |
|--|---|
| a) Capacity of the facilities
(Production base) | : 40,000 m ³ /d |
| b) Method of desalination | : Reverse osmosis |
| c) Total project costs | : approx. ¥20billion |
| d) Water production costs | : approx. ¥260/m ³ |
| e) Location of installation | : Chatan water treatment plant,
Miyagi, Chatan Town,
Okinawa Prefecture |

Note : 1. The desalination facilities are constructed/installed next to the surface water/dam water facilities and ground water facilities
 2. Production amounts:
 surface water and ground water: 214,300 m³/d
 sea water: 40,000 m³/d
 total/mixed supply: 218,300 m³/d

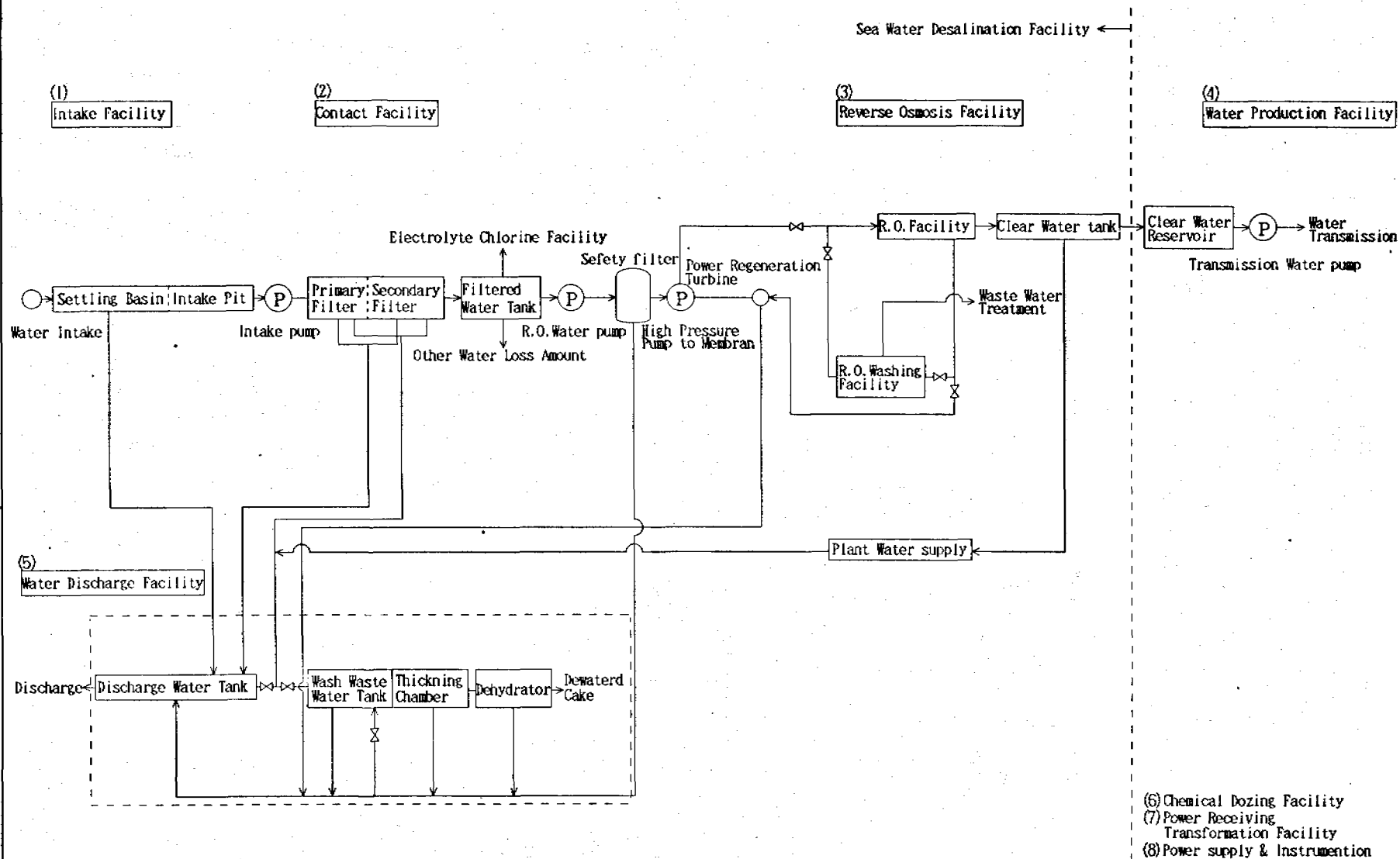
f) Facilities development plan

<u>Year</u>	<u>1992</u>	<u>1993</u>	<u>1994</u>	<u>1995</u>	<u>1996</u>	<u>1997</u>
<u>Description</u>						
1. Design & Construction	Detailed Design	Const- ruction	ditto-	ditto-	* Construction completion	
2. Production <u>Amount(m³/d)</u>						
Commissioning				10,000	15,000	15,000
<u>Total</u>				10,000	25,000	40,000

5. Conclusion

Introduction of the large-scale desalination system in Japan is the first trial, and compared with small-scale systems in remote islands, several problems are anticipated. In the future, guidelines for installation and maintenance of a desalination system will be compiled through design of experiences the specific facilities so that other city water supplies can utilize it.

Fig.3 Flow Chart of the Sea Water Desalination(Reverse Osmosis Method)Facility of Okinawa Prefecture



SOLAR AND WIND POWERED DESALINATION FOR REMOTE COMMUNITIES

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ABSTRACT The Remote Area Developments Group of Murdoch University has been investigating the technology of Reverse Osmosis (RO), powered by both photovoltaic panels and windmills, as a means of desalinating drinking water for remote communities. While energy recovery systems are seen as the key to successful adaptation of RO to solar power, the key to wind powered systems has been the use of pressure accumulators. An overview of the design, construction, field testing and performance analysis of prototype reverse osmosis desalination systems for remote areas is discussed.

1. INTRODUCTION

Many inland community water sources have salinities in the range of 3000-6000 ppm, making it unsuitable for drinking water supply. Nitrate and fluoride are also often found in concentrations higher than the recommended standard. In places where surface water sources are not available or are limited, rainfall is low and groundwater salinity is high, the only possible supply of drinking water is by transportation or by desalination.

Distillation (solar and multi-stage-flash) was, until recently, the world's leading desalination technology. Its appropriateness for remote community water supply however is questionable. Reverse osmosis desalination has now superseded distillation and has been applied in the provision of drinking water worldwide.

Recently, low pressure membranes have been developed which operate at 10 atmospheres or less and, though not suited to seawater desalination, are ideal for the treatment of brackish water allowing a high production rate and a lower energy demand. The process has, however, remained a highly technical one requiring skilled supervision and ongoing maintenance. Accordingly, its application in remote communities has not generally been successful (Feron, 1985; James, 1988). Further, its energy demand, even with low pressure membranes, has made its adaptation to alternative energy sources a difficult and expensive option.

The Remote Area Developments Group of Murdoch University has been investigating the process with a view to adapting it to alternative energy sources for remote applications. The criteria for successful adaptation are seen as,

1. Simplicity
2. Low cost
3. Low maintenance
4. Robustness.

A trade-off of system efficiency for reliability likewise improves the applicability of this technology for remote area use (Swinton, 1985).

This paper describes the development of two desalination systems using reverse osmosis powered by solar and wind energy respectively which meet the above criteria.

2. DEVELOPMENT STRATEGY

The amount of water treated per membrane is restricted to 10 - 15% as higher recovery ratios lead to rapid membrane deterioration. Large plants use several banks of membranes connected in series to achieve recovery ratios of 50 - 90%, leaving a highly concentrated brine which may be very

difficult to dispose of. Additionally, the last few membranes in series are treating quite concentrated salt solutions. Without chemical pretreatment (complexing agents and pH control) many sparingly soluble salts commonly found in groundwater (eg. CaSO_4) can exceed their solubility and precipitate out of solution as a scale which fouls the membrane (Heitmann, 1990). Post treatment dosing is also required to adjust the pH back to drinking water standard.

Very small units, typically run from generators, may use only one or two membranes and so achieve recovery ratios of only 10 -20%. They require a much lower degree of pre and post treatment and brine disposal is much less of a problem (Keefer *et al*, 1985). The brine is only slightly more concentrated than the feed but there is a lot of it, as 80 -90% of the feed is rejected. Accordingly, for every litre of fresh water produced, up to 10 litres of feed water has to be pumped up to the system pressure. Since only a small proportion of the water consumption of a community is for drinking and cooking this does not pose a problem if a dual supply system is installed.

3. SOLAR POWERED DESALINATION

3.1 Introduction

Two factors have combined to make solar powered RO unattractive in remote applications. One is the high capital cost of the solar array needed to power a pump capable of producing the high pressures required especially when ten time more water is pumped up to pressure than is produced as fresh water. The second is the inability to cope with a varying power supply such that a battery bank is usually used to keep constant power. Thus the pumping rate is kept constant but the duration of pumping is dramatically reduced. The batteries also consume 30% of the power from the array so the array has to be oversized.

3.2. Energy Recovery

In terms of the hydraulic energy requirements, the loss of 90% of the pressurised water in the reject is quite wasteful and makes adaptation to solar power a very expensive option. Also, since the system pressure and recovery ratio are determined by a throttle valve on the reject line, regular supervision and adjustment would be required to cope with the varying supply of power from the solar array.

Great savings in energy can be gained by using this still pressurised waste stream to assist in the pumping of the feedwater. The three main methods of utilising this energy include the use of turbines or pelton wheels which can be attached to the pump or a generator (Salaspini, 1985), flow work exchangers which act directly on the feed water (Gilbert, 1971), or systems which return the waste stream directly to the back of the piston in a positive displacement pump (Keefer *et al*, 1985). For small scale applications in remote areas, the latter method is probably preferable. It effectively fixes the recovery ratio as the ratio of piston-rod diameter to cylinder diameter. This is a particularly valuable property where variable power from renewable energy is used. The unit becomes self tending to a large extent. Also, the high efficiencies of the positive displacement pumps are valuable when the high capital costs of solar panels are considered.

The RADG group has been developing a simple pumping system using double acting pumps to deliver an even flow, and energy recovery to reduce the power requirement. Our goal is to produce up to 1000L per day of fresh water from as little as two solar panels. To assist us in this goal, we have employed low cost pump optimisers and solar trackers to extract the maximum power available from the panels.

A Perth company, Solar Track Pty Ltd, markets a robust and reliable solar tracker which has only two moving parts - the two bearings. The solar tracker operates purely on the weight difference between two half-shaded copper cylinders, each partly filled with liquid Freon. When one cylinder heats up more than the other, more Freon evaporates which forces the liquid Freon below it into the other cylinder.

The pump optimiser keeps the voltage output from the solar panel up near its peak so that power output is always near the maximum possible regardless of insolation levels. It then delivers the required current to the motor of the pump at whatever voltage is available from the relationship

$$V_{in} \times I_{in} = V_{out} \times I_{out}$$

As a consequence, sufficient starting torque is provided to ensure an early start up in the mornings while the pumps keep operating through periods of light cloud (albeit slowly) that would otherwise stop it. The optimiser is an inexpensive alternative to a Maximum Power Point Tracker (or Maximiser) and though less efficient, is more cost effective for small solar arrays.

Unlike most larger scale RO plants, it is anticipated that no other control mechanisms will be required. It should be stressed that every control mechanism requires some power, is a source of failure and adds to both the capital and maintenance costs. In keeping with this philosophy, the pump is connected through the optimiser directly to the solar array and not through a battery bank. Batteries are only 70% efficient, so 30% of the power from a solar array is lost if batteries are used (Van Overstraeten and Mertens, 1986). Further, since only a fixed amount of solar energy is available per day, it is considered more efficient to store that energy as water than to store it as chemical/electrical energy.

3.3. Experimental Work

While the development work on the prototype was in progress, a small commercially available desalinator was tested for its adaptability to solar power. The 'Powersurvivor' is a 12V unit designed for the boating market with a 5 L/h capacity and a high pressure membrane suitable for desalinating seawater. The unit was installed in a test facility on Murdoch campus and connected to a 55W solar panel via a solar pump optimiser. The panel was mounted on a solar tracker and both the tracker and the optimiser could be disarmed to evaluate their performance.

3.4. Results

A typical summer production pattern of the 'Powersurvivor' coupled to the solar panel is shown in Figure 1.

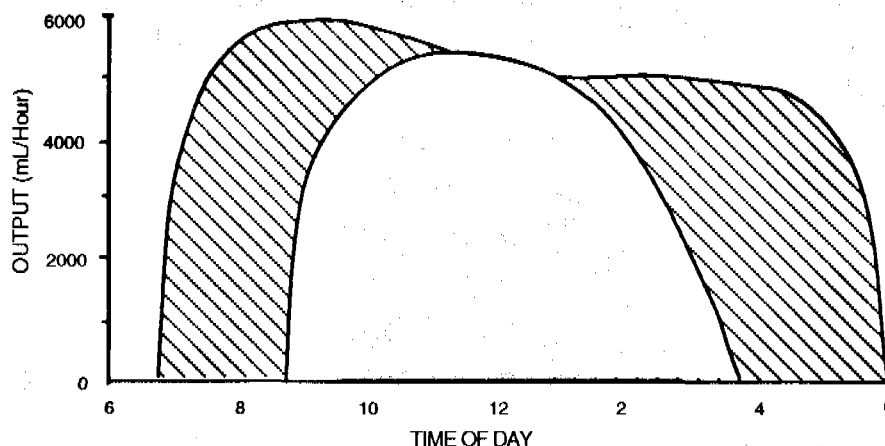


Figure 1 The hatched area represents the increased production of fresh water resulting from the use of solar tracking and pump optimisation.

The tests demonstrated the benefits of the solar tracker and the optimiser on production rates. The tracker in particular produced a significant increase in production while allowing a rapid startup in the morning. The optimiser showed its benefits in keeping the unit working well through marginal conditions such as early morning, late evening and, as earlier tests showed, through periods of light cloud. The power provided by the panel in the mornings is also delivered in a high current, low voltage form by the optimiser and so gives the pump the required starting torque. The tracker and optimiser together nearly doubled the production of fresh water. From these results, it is likely that the maximum daily duration of production in Perth will be 11 to 12 hours. The maximum production rate from the Powersurvivor is approximately 65 litres from a day which recorded 12.7 hours of bright sunshine. Using the system adopted by Legge (1987) we can assume a production rate of 55 litres from an average Perth summers day of 10.7 hour of bright sunshine. We can also predict the production rates for other regions using this simple system

3.5. Computer Modelling of Proposed Unit

A computer model which takes into account energy recovery has been constructed and by using the flux rates of a commercially available low pressure membrane, a maximum production rate of 83.7 L/h is expected for the unit we are developing. The experiments conducted on the 'Powersurvivor' demonstrated the extremely long duration of pumping that can be achieved using a solar tracker and solar optimiser. Twelve hours of pumping in Perth would therefore produce 1000 L/day from just two solar panels.

4. WINDPOWERED REVERSE OSMOSIS DESALINATION

4.1. Introduction

With the use of low pressure membranes, the power produced by a windmill in coastal areas is usually adequate to power a reverse osmosis plant capable of up to 1000 L/day. The major problem with this concept is the rapid fluctuations that occur in water pressure resulting from the turbulent nature of the wind regime. Shocks of this nature can rapidly wear and compress the membrane making the system uneconomic.

4.2. Energy Storage

By linking RO to the standard windmill pump via a pressure vessel, a prototype has been designed which combines renewable energy with a well-proven pumping technology for the purpose of groundwater desalination.

The prototype windpowered reverse osmosis plant is illustrated in Figure 2. The system operates by charging up pressure accumulating vessels using low volume input from a windmill pump under varying windspeed conditions. The water accumulates until it reaches a critical pressure, triggering the pressure switch which closes the circuit to a 12V solenoid valve, opening to send pressurised water to the reverse osmosis membrane at the required flow and pressure. When the pressure falls below a minimum value, the switch opens the circuit, recommencing the

pump-up phase. In this way the low, variable flow from the windmill can be effectively matched to the constant flow/pressure requirements of a RO membrane. It has been shown that the system is capable of producing from 130 litres/day to over 1000 litres/day of desalinated water depending upon wind conditions. The RO membrane removes on average 96% of sodium chloride, the predominant contributing ions to salinity, and nitrate and fluoride rejection rates averaged 65% and 78% respectively (Robinson, 1987).

4.3. Experimental Work

Field testing of the prototype consisted of connecting the unit to a 3 metre diameter Aeromotor windmill pump near Murdoch University, and running through samples of simulated groundwater feed of various salinities. The prototype commenced operation on July 18th, 1988 and operated continuously since then until its de-commissioning on the 16th of August, 1989, a period of 13 months. During that time some 220 days or 5280 hours of raw data was logged and stored for analysis, producing 3348 hours of usable windspeed, direction input, and product water output data (Robinson, 1990).

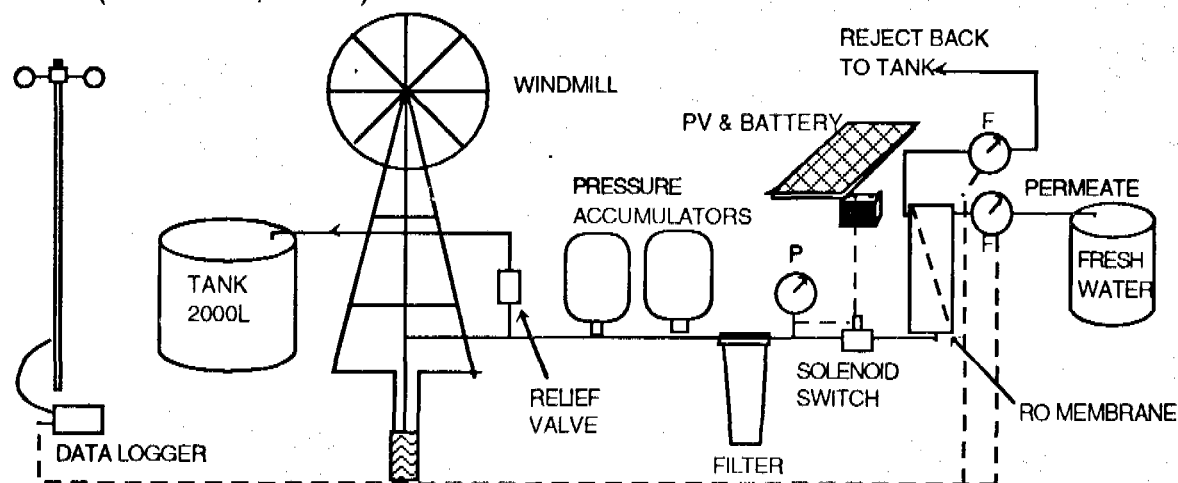


Figure 2- Prototype Windpowered Desalination System

4.4. Results

Correlation was made between wind speed, input salinity and permeate production rates at the test site and used in the production spreadsheet described below. Permeate quality ranged between 900 and 1150 ppm, well within recommended standards for drinking water. Permeate quantity and quality are both significantly improved at lower feed salinities.

4.5. Production Spreadsheet

It is useful to translate the test performance curves into production curves of predictive value. This is achieved by combining wind profile data characterised by the site frequency distribution curve with the production curve of the prototype derived from the field trials. A software spreadsheet package was used to allow the prediction of system performance given the groundwater characteristics from log bores and average windspeed at various potential locations in Western Australia.

An example of the spreadsheet production prediction is shown in Table 1.

TABLE 1: Performance Prediction
Ngangganawili Community, near Wiluna, W.A. :

Feed Salinities (ppm).....	2610
Membrane age (months).....	12
Recover ratio (%).....	15
Weibull Parameters: C:.....	3.4
K.....	1.51

Windspeed (m/s)	Modelled performance Flow (L/h)				WindFreq (h/year)	Projected Flow	
	Qm	Qr	Qs	Qt		Qp (L/day)	+ / -
0.5	39.1	5.87	3.35	2.56	1385	10	7.9
1.5	58.1	8.71	6.18	4.73	1917	25	34.8
2.5	95.8	14.38	11.85	9.07	1774	44	55.7
3.5	199.3	29.89	27.36	20.94	1389	80	46.4
4.5	329.1	49.36	46.83	35.84	975	96	20.0
5.5	364.9	54.73	52.20	39.95	629	69	8.8
6.5	363.5	54.52	51.99	39.79	379	41	4.6
7.5	386.1	57.91	55.38	42.38	214	25	2.1

System delivery average	411	+ / -	182 L/day
For an average of	365	days per year	
Average permeate salinity	200	+ / -	40 ppm

The input variables and output estimates for the spreadsheet model are described, in order of presentation on the spreadsheet model.

(i) Feed/permeate salinity - The osmotic pressure required for reverse osmosis is directly proportional to the feed salinity - hence higher feed salinity produces a greater power requirement per unit production of permeate.

(ii) RO Membrane age - With increasing age the overall tendency of the RO membrane is for a decline in both the salt rejection and permeate flux. An equation for performance decline is entered into the spreadsheet programme, based on field and other research results.

(iii) Recovery Ratio - The recovery ratio is the most influential of the operator-controlled input variables, and is best maximized. Permeate salinity is improved under a higher recovery ratio, as salt flux is constant, but permeate flow improved. Thus increasing recovery ratio is a common strategy for improving the salt rejection. The ceiling to recovery ratio increase is the maximum allowable concentration of salts in the reject stream. Once ionic concentrations exceed scaling values, the increased cost and complexity of pretreatment must be offset against the increased efficiency and product quality due to a higher recovery ratio.

(iv) Weibull parameters - The wind characteristics of a site can be resolved into two factors describing the frequency vs windspeed curve given by the equation:

$$f(V) = (K/C)(V/C)^{K-1} \exp[-(V/C)^K]$$

where V is the windspeed (m/s);
 K is a dimensionless shape parameter and
 C a scaling parameter (Lyons, 1989).

The K value determines the 'skewness' of the frequency distribution curve in relation to the y-axis. Thus a K value of 2 describes a 'Raleigh' distribution curve; that of 3 a normal distribution curve. The C parameter scales the value with respect to the x-axis- a C value above 5 gives higher wind frequency values for a given windspeed category. These concepts are illustrated below.

The effects of changes in wind characteristics are large, understandable since this is the power source for driving the desalination process. The production at windspeeds below approximately 2.5 m/sec are negligible, hence a windpattern having a higher K value translates windspeed into production more effectively.

(v) Modelled performance flow - The scaling factor Q_m is the modelled performance curve for the RO prototype, determined by the field data obtained which has been calculated for time= 0 months, salinity= 0 p.p.m. and recovery ratio= 100% using equations derived in analysis (Robinson 1990). Values for the performance for these variable levels determined by a particular site water quality and design parameters can then be calculated from the modelled flow using these same equations. This value (Q_t) is then multiplied with the particular windspeed frequency value for that windspeed bin, to derive the final performance estimate, Projected Flow in litres per year, which is converted an average litres per day estimate.

5. CONCLUSION

Recent advances in RO technology have facilitated its adaptation to renewable energy sources. The new low pressure membranes operate in a pressure range within the capabilities of both solar and wind power. The key features of both systems are energy recovery for the solar units and the use of pressure vessels in the wind powered units. Energy recovery allows small RO units to operate at a fraction of their power requirements thereby reducing the capital cost of the solar array. Trackers and optimisers greatly extend the pumping abilities of the RO units. The pressure vessel of the wind powered unit stores the pressurised water delivered to it by the windmill, then releases it in a controlled and steady flow to the RO membrane, thus eliminating the enormous fluctuations in water flow that windmills produce. Both systems offer the potential for cost effective desalination of drinking water in remote communities. Field testing of both systems will continue.

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TECHNICAL SESSION 11B
SEISMIC EFFECTS AND
MANAGEMENT IN WATER SUPPLY

Effect of Earthquake and Volcanic Activity on Groundwater

Seismic Evaluation on Renovation Priority of Distribution Pipes

An Approach to Seismic Management for Reliable Water Supply

SEISMIC EVALUATION ON RENOVATION PRIORITY OF DISTRIBUTION PIPES

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ABSTRACT:The pipeline facilities in a water supply system are important to ensure a reliable water supply service, not only in normal circumstances, but also in earthquake emergencies. In Osaka City, approximately 5,000km pipeline have been laid during a period of than 90 years since the water supply system started in 1885. So, it takes a long period to construct an earthquake-proof distribution system. The most effective method for earthquake preparedness is to establish the renovation priority of pipelines.

This paper presents the practical guideline considering such seismic factors as liquefaction, ground tectonic line and earthquake damages in the past. The priority evaluation of pipe renovation and the pipe renovation methods are also introduced.

1. INTRODUCTION

The water supply system in Osaka city was established in 1895 as the fourth modern water supply system in Japan. At that time, the water was supplied to 610 thousand citizen by a gravity flow system, and the length of pipeline was 325 km. The pipe material was mainly cast iron. Since then expansion has multiplied the system 9 times such that the present length of pipeline is 5000 km. On the other hand, problems of low water supply pressure, red water, etc. began to occur, as pipes were getting older. In order to eliminate these problems, the 1st Distribution Pipe Improvement Program started from 1965, and lately the goal was reached. However, for water supply systems, higher service than before is expected, and further improvements of functions and reliability is essential for social life. Accordingly, the necessity to secure a stabilized water supply even in the time of emergency such as earthquake, etc., has surfaced. So, the 4th Distribution Pipe Improvement Program has been carried out since 1990 to make earthquake-proof pipe network system.

Particularly, the pipelines to the evacuation area where

citizens take refuge during a large scaled disaster, are important , so they are laid with earthquake resistant joints to secure emergency water supply.

However, under the present circumstances in which many aged pipe exists, it is necessary to establish the renovation priority of main pipes in order to carry out renovation effectively for the earthquake-proof pipe network system.

From this viewpoint, in 1988, Osaka Municipal Waterworks has established their own guideline for earthquake proof design of the pipelines. This report introduces the priority evaluation of pipe renovation and pipe renovation method which are described in the guideline.

2.THE BASIC THINKING FOR THE GUIDELINE

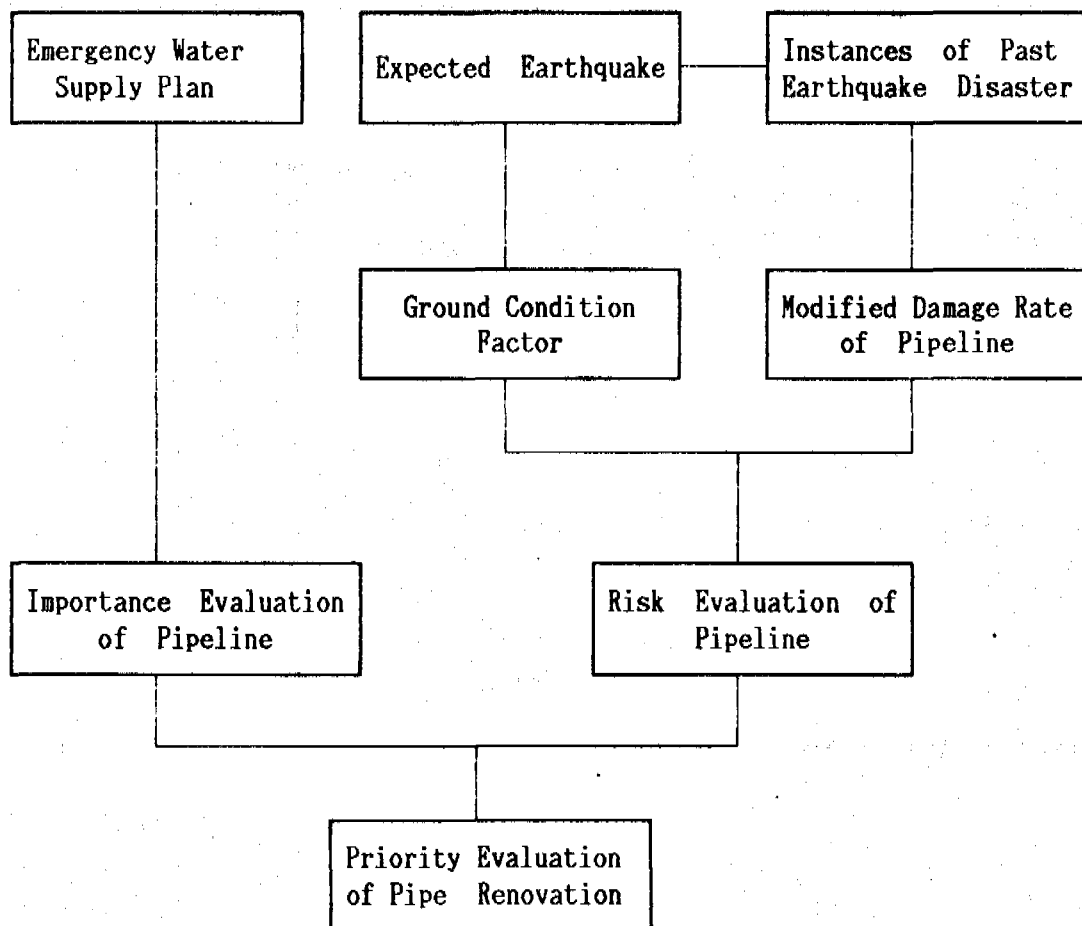


Fig.1 Procedure of Priority Evaluation of Pipe Renovation

The basis for the priority evaluation of pipe renovation is as follows. At first, the average damage rate of pipeline for each material and joint type are obtained from the occurrence of the past earthquakes. Next ground conditions are determined from behavior of ground at the time of the expected earthquake. The priority evaluation of pipe renovation is

carried out using the average damage rate of pipeline and ground condition.

On the other hand, the importance evaluation of pipeline is carried out, assuming trunk pipelines, which are necessary to emergency water supply plan at the time of earthquake disaster, important lines, etc. Judging the result above generally, the priority evaluation of pipe renovation is carried out, and the pipe renovation method is determined as shown in Fig.1.

3.THE RISK EVALUATION OF PIPELINE

[1] Earthquake Damages in the Past

For the risk evaluation of pipeline, it is necessary to investigate the earthquake damages in the past. The earthquakes in the past which suffered serious damages on pipelines are as follows. Those are Kanto Earthquake (1923), Fukui Earthquake (1954), Niigata Earthquake (1964), Tokachi Offshore Earthquake (1968), Miyagi Prefecture offshore Earthquake (1978), Urakawa Offshore Earthquake (1982), and Central Japan Sea Earthquake in Japan, and San Francisco Earthquake (1906), San Fernando Earthquake (1971) and Mexico Earthquake (1985) in overseas. Many investigations and researches have been carried out. As a result, the findings are as follows.

(1) Materials of pipeline and type of joins

A) Reinforced Concrete Pipe

Cracks on pipe and damages to joints

B) Cast Iron Pipe

Damages of cast iron pipe show these tendencies.

Pipes with large diameter(>400mm):cracks in axial direction

Pipes with small diameter(<350mm):circumferential breaks

Pipe fitting: shear failure, movement of bent pipes and valve trunk breaks

C) Steel Pipe

Cracks at field-welded joints

D) Asbestos Cement Pipe

Failure of pipe, movement of joints

(2) Ground and Topographic Features

A) The softer ground where the pipes are laid, the larger the number of damages occurred

B) Damages to pipeline in stable hill areas are very slight.

C) Damages to pipeline laid in embankment and fill ground are particularly large.

D) Damages to pipeline across ground tectonic line are large.

E) Damages to pipeline which are laid in the sloping ground such as scarp are large.

F) Damages to pipeline such as movement of joints in sandy ground with underground water, are frequent because of sand liquefaction.

[2] The Expected Earthquakes

To determine the expected earthquakes, the records of the Research Institute of Civil Engineering (Ministry of Construction) are used. Based on magnitude and distances from epicenters of the earthquakes which broke out during 416-1981 in the area about 200 km around Osaka, maximum acceleration are obtained. The expected earthquakes are determined as in Table I.

Table I Expected Earthquake

Expected Earthquake	Classification	Distance of Epicenter	Magnitude	Maximum Ground Acceleration
Earthquake of Short Distance	Inland type	15 [km]	7.2	1.75 [m/sec ²]
Earthquake of Medium Distance	Inland type	70 [km]	7.6	0.84 [m/sec ²]
Earthquake of Long Distance	Ocean type	130 [km]	8.3	0.81 [m/sec ²]

[3] The Risk Evaluation for Modified Damage Rate

Osaka Municipal Waterworks uses the following guidelines as the influencing factors on damage to pipeline and accordingly carries out the risk evaluation of pipeline.

- 1) Material of pipe
- 2) Type of joint
- 3) Ground strain at the time of earthquake
- 4) Liquefaction
- 5) Ground tectonic line
- 6) Trace of old river, artificial banking, etc.
- 7) Transition line between hard ground and soft ground

The procedure of risk evaluation of pipeline is as follows. Average damage rates are obtained by referring to Miyagi Offshore Earthquake and Central Japan Sea Earthquake which corresponds to the expected earthquake of Osaka City in scale, and the damage conditions of them are relatively well listed.

First, average damage rates (per 1 km) of pipeline for each material and joint type, are obtained.

Besides, Osaka City area is divided with meshes of 500 m square. Ground condition factors are fixed quantitatively from the behavior of ground at the time of the expected

earthquakes. And, applying these factors, average damage rates of pipeline for respective mesh of the area, are modified. And the risk evaluation of pipeline is carried out, classifying into 3 ranks.

Ground condition factors are explained as follows.

1) Ground classification factor (G1): Forming simulated wave forms which satisfies the characteristics of the expected earthquakes, and the dynamic analyses are carried out, using these simulated wave forms as input earthquake wave. Each mesh of Osaka City is classified by displacement and natural period of surface ground. And the factor in each mesh is obtained from the strain and apparent wave length at the time of the expected earthquakes. 2) Liquefaction Factor (G2) : Obtained by assuming that the damage to pipeline in the liquefaction area becomes 2 times of the average ground. 3) Ground Tectonic Line Factor (G3) : Obtained by assuming that the factor is proportional to the extension of tectonic line in mesh. 4) Artificial Ground Factor (G4) : Obtained by assuming that the factor is proportional to the area of artificial modification ground in mesh. 5) Transition Factor of Ground Condition (G5) : Obtained by assuming that the factor is proportional to the extension of transition line between hard ground and soft ground in mesh.

Ground condition factors are the sum of the factors.

$$\text{Ground Condition Factor} = G1 + G2 + G3 + G4 + G5 \quad \text{Eqn(1)}$$

Modified Damage Rate is obtained by following equation.

$$\text{Modified Damage Rate} = \text{Average Damage Rate} \times (1 + \text{Ground Condition Factor}) \quad \text{Eqn(2)}$$

Table II Risk Evaluation of Pipeline

Risk	Modified Damage Rate per km
Risk I (Large)	> 0.23
Risk II (Medium)	> 0.16, < 0.23
Risk III (Small)	< 0.16

4. THE IMPORTANCE EVALUATION OF PIPELINE

In the case of determining the importance of pipeline, the most important pipelines are basically the emergency water supply route to evacuation area at the time of large scaled earthquake disaster.

In addition, Osaka City Waterworks maintains water pressure and flow rate by dividing the city into 17 blocks.

The trunk pipelines which are distributing to these blocks are also placed as the most important pipelines from the viewpoint of proper information control at the time of emergency. The importance evaluation of pipeline is carried out as shown in Table III.

Table III Importance Evaluation of Pipeline

Importance	Ranking of Pipeline
Importance I (Large)	Network of Key Pipeline at time of Earthquake
Importance II (Medium)	Generally Important Pipeline
Importance III (Small)	Other General Pipeline

5. THE PRIORITY EVALUATION OF PIPE RENOVATION

The priority evaluation of pipe renovation is carried out using the results of the risk evaluation and the importance evaluation of pipeline which are described earlier.

Table IV Priority Evaluation of Pipe Renovation

Risk \ Importance	I	II	III
I	I	II	III
II	II	II	III
III	III	III	III

6. PIPE RENOVATION METHOD

The renovation methods of the most important pipelines are basically replacement and inner pipe insertion.

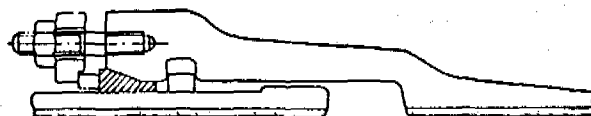
But, for other pipelines, hose lining is sometimes adopted. Because it can secure water distribution even if some damages to pipes has broken out at the time of earthquake, and

the pipe restoration in short period can be expected after the occurrence of damages.

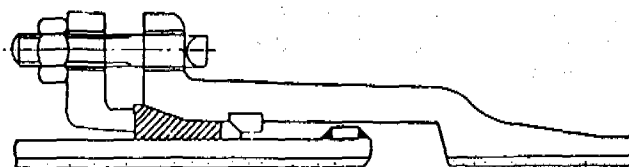
Features and earthquake resisting properties of respective pipe renovation methods are as follows.

[1] Replacement Method

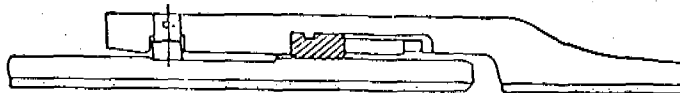
Free selections of diameter and type of joint are possible and steel and ductile-iron pipes are both available. Particularly, by applying anti-escarpment joint in fig. 2, it is possible to provide remarkable seismic property to pipeline. However, there are problems of high construction cost, long construction period etc., because excavation along pipeline is required.



S Type of Joint ($\phi 500 \sim \phi 2600$)



SH Type of Joint ($\phi 100 \sim \phi 450$)



PU Type of Joint ($\phi 700 \sim \phi 1350$)

Fig. 2 Type of Joint

[2] Inner Pipe Insertion Method

This method is to insert new pipes into existing pipes from the vertical shafts where a valve or T-pipe is installed. Similar to the replacement method, Steel and ductile-iron pipes are both available. And it is possible to select type of joint freely. Accordingly, earthquake resisting property is expected and compared with the replacement method, big reductions in construction cost and construction period are also expected. However, the diameter of the new pipe will be smaller by approximately 100 mm, since new pipes are inserted into existing pipes.

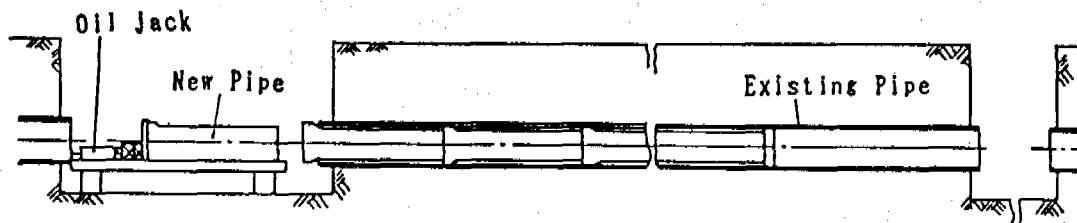


Fig.3 Inner Pipe Insertion Method

[3] Hose Lining Method

This is one of regeneration methods applying lining material which was knitted with resin coated fiber. Lining material is inserted to inner surface of existing pipes by using pneumatic pressure and fixed by thermosetting adhesive. Earthquake resistance and water tightness of lining material have been proved by experiments.

However, improvement in strength of pipe can not be expected because aged existing pipes are used in this method. So, this method must not be used for the improvement of the more important pipelines.

This method is economical when compared with the replacement method or inner pipe insertion method.

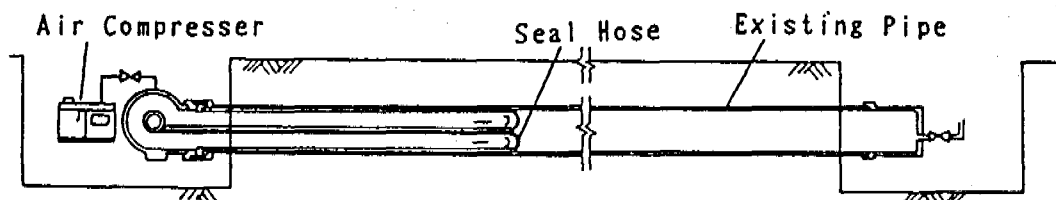


Fig.4 Hose Lining Method

7. CONCLUSION

In the guideline introduced in this paper, the risk evaluation of pipeline is dependent on the data of earthquake disasters in the past. This is because elucidation of mechanism of damage to pipeline is insufficient yet and complete theoretical evaluation is too difficult now. So, results of investigation and research in these fields are expected in future.

It is most desirable to improve all pipelines using joints with high earthquake resistance for constructing earthquake-proof pipe network system. In future, it is necessary to investigate selection of renovation method taking economics into consideration. Because expected earthquakes are classified into 2 ranks, medium scaled earthquake and large scaled earthquake, and the possibility of occurrence of large scaled earthquake is rare. In addition, for the pipelines which connect the main pipelines, it is necessary to review the renovation priority by raising their degree of importance.

The disaster prevention in the city area is not possible to realize in short period, so, the planned execution of disaster prevention measures is required from viewpoint of long range. Particularly, for providing earthquake resistance to complicated facilities such as pipelines of water supply system, the effects are influenced by the priority of renovation or selection of method. The methods which are introduced in this paper are expected to offer proper reference.

AN APPROACH TO SEISMIC MANAGEMENT FOR RELIABLE WATER SUPPLY

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Abstract - Water supply in Japan has been developed throughout the country and is currently regarded as one of the important lifeline systems. Water supply suspension implies complete stoppage of all human and socio-economic activities. It is of vital importance for water utilities to maintain constant and reliable water supply to the public even in case of emergency. This paper attempts to present a general approach to attaining reliable water supply, focusing the attention on the seismic risk management and planning.

Keywords - Seismic risk management, reliable water supply, design criteria, emergency water supply, restoration from seismic damages,

1. INTRODUCTION

Water supply in Japan has been developed since 1887 when a modernized water supply system was first introduced in Yokohama. During the past century, a service ratio in the country has increased tremendously to 94.7% as of March, 1991. The water supply is now regarded as one of the fundamental lifeline systems. If once the water supply stops its operation, entire areas of vulnerable human and socio-economic activities would be seriously affected. In this sense, it is of primary importance for water utilities to realize a goal of stable and reliable water supply even in case of an emergency.

Water supply system developed so far differs much from area to area reflecting the geographical features of the service area and the technologies existing at the times of expansion. Experimental evidences from past earthquake, although limited in numbers, show vulnerability of the systems in wide varieties. These situations may have made utilities difficult to take comprehensive measures against a loss of water frequently occurred during such disasters.

This paper summarizes an interim report prepared by the study team in JPRC, in an attempt to provide information on approach method for seismic risk control and management.

2. CONCEPT OF RELIABLE WATER SUPPLY

Measures to minimize seismic damages are generally considered similar to those for maintaining stable and reliable supply of water. For discussion on an approach method for seismic

risk control and management, this paper first explains words of “system input”, “output” and “water supply system”.

Major components of the inputs to the water supply system are:

- 1) Raw water characteristics such as raw water availability, intake rate, quality, raw water extraction method, potentials of intake facilities for restoring from seismic damages,
- 2) Disturbance by natural disasters such as droughts, earthquakes, floods and typhoons, and
- 3) Water production targets enumerated in operation manuals, on the basis of which the water supply system is operated to meet the daily and hourly fluctuation of water requirements.

The water supply system contains facilities for raw water extraction, transmission, sedimentation, filtration, storage, distribution and mechanical/electrical control system with their appurtenances, which have also the following aspects that may be decisive factors in seismic management:

- 1) Provisions for disasters, design criteria adopted, age of facilities in addition to the general function of the water supply facilities, and
- 2) Operational and institutional aspects of water supply for rapid recovery and restoration from seismic damages.

The system outputs have generally the following three components:

- 1) An amount of water distribution or water production to be supplied to the consumers,
- 2) Supply pressure, and
- 3) Quality of water supplied.

Concept of the reliable water supply and relationship among the water supply system, inputs and outputs are presented on Fig-1. For simplification purpose, only quantitative aspects of the system output are considered herein. As outputs, there will be several sets of allowable ranges. It can be said that the reliability of water supply system is a potential of the system to restore from seismic damages or maintain normal operation to supply water to the consumers, while earthquakes are undesired inputs that disturb normal operation of the system. It is general procedures that allowable limits of outputs should be determined first. Then, options how to supply water to the consumers should follow in consideration of magnitude of water losses.

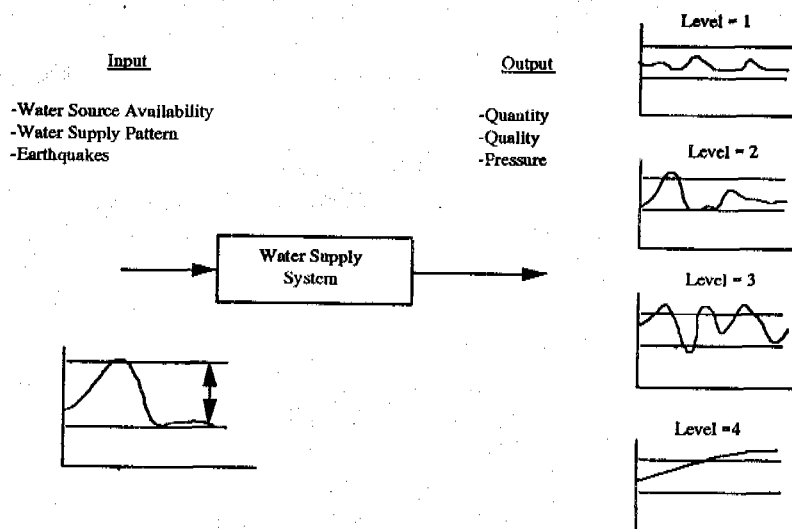


Fig-1 SEISMIC CONTROL AND WATER SUPPLY SYSTEM

3. PROCEDURES TO PREPARE FOR EARTHQUAKES

Water utilities are agencies responsible for minimizing seismic damages on water supply by preparing a comprehensive disaster protection program. Scale and area of predicted seismic disasters or damages will be precious information to determine priority options and to clarify their effects and relationship between options. Succeeding discussions will be based on overview of seismic options.

3.1 Options for Protecting from Earthquake Damages

To select possible options against earthquake damages, a schematic diagram of the disaster process was prepared as shown on Fig-2.

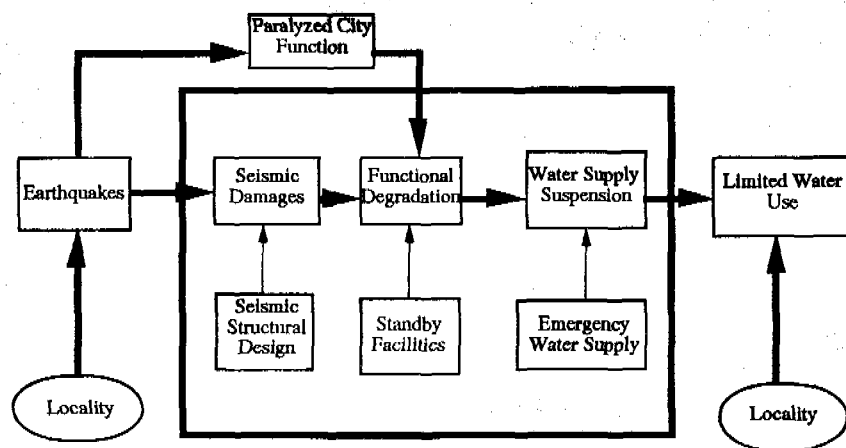


Fig-2 WATER SUPPLY IN CASE OF EMERGENCY

From this figure, it can be understood that when earthquakes occur, water supply facilities are damaged physically with substantial reduction in their water production due to the subsequent power failure. This may cause a critical water shortage for the consumers in service areas. Water loss or shortage by earthquakes thus takes place as system outputs. Possible options of the system are design/construction of facilities to withstand the earthquake shaking, to rapidly recover from damages and to restore water supply services.

To clarify further the relationship between options, expected damages are examined from the following three aspects:

- 1) Reliability: Frequency or probability of water shortage occurrence,
- 2) Resiliency: Capability of water supply facilities to restore water supply services, which can be represented as a period required for restoration.
- 3) Vulnerability: Intensity of damages, for example, an amount of water shortage due to seismic damages.

From these, damages could be measured as products of resiliency (period) and vulnerability (losses of water). The above relations are also shown on Fig-3. The figure explains that seismic risk management and control has two stages of options: 1) structural strengthening of the facilities at design and construction stages and 2) establishment of emergency water supply after the earthquake. This implies options we can select are the option 1), 2) independently or both in combination. Particularly, development of structural design method to ensure safety of facilities,

free from seismic damages, and organization of emergency water supply are of vital importance. To set up targets and approach method, expected seismic damages and their options are outlined below.

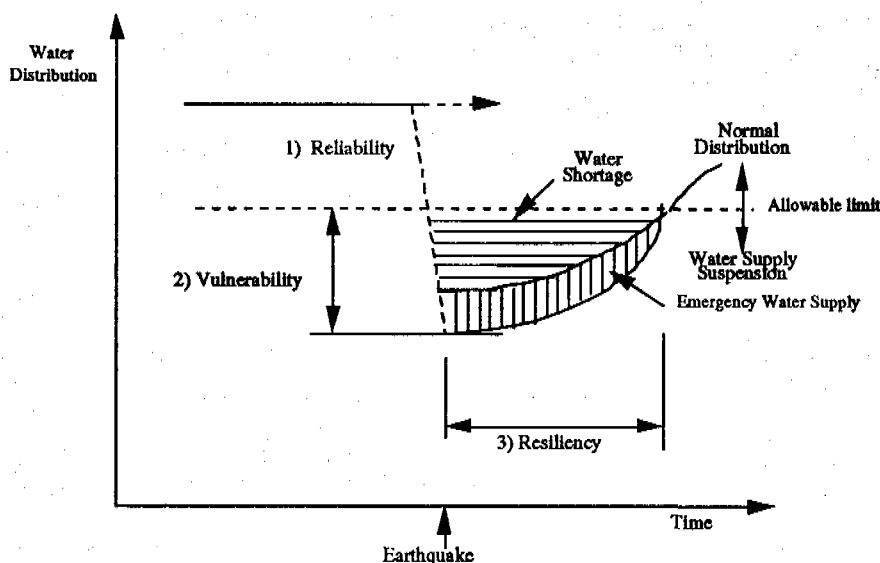


Fig-3 WATER DISTRIBUTION AND EARTHQUAKE

3.2 Proposed Planning Procedures for Earthquake Damages

Possible options may differ in effectiveness depending on their characteristics as schematically illustrated on Fig-4.

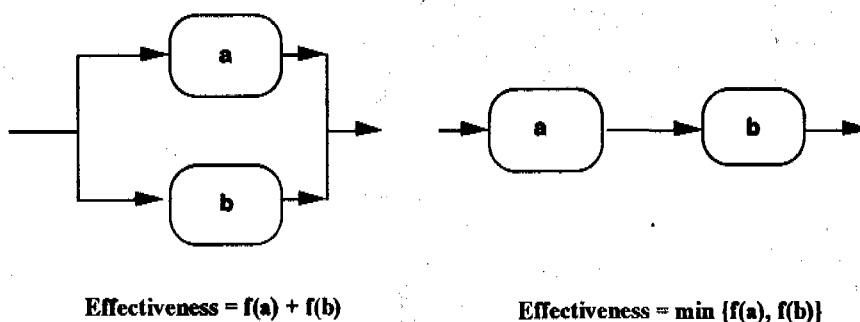


Fig-4 EFFECTIVENESS OF OPTIONS

When both options (a) and (b) are adopted in combination, their effectiveness to reduce seismic damages are accelerated. This is shown on Case (1) of the figure. In Case(2), total effectiveness is completely dependent on an option which have minimum effectiveness in reducing damages. If the service pipelines are more vulnerable than distribution trunk mains, any measures to strengthen the trunk mains are meaningless. This also implies that planning procedures may be derived from review of such aspects of options.

Many options are available. The water supply system is recognized as an important lifeline system to supply water permanently to the public, while earthquakes occur almost suddenly without any indications. The seismic damages may be immediately restored to restart water supply services. Hence more emphasis should be placed on seismic design and construction of facilities which are rather vulnerable and critical. Such design and construction options should have a priority. Measures for emergency water supply should have the second priority as they are considered supplemental. Facilities, equipment and pipes do not necessarily have similar adequacy to seismic damages reflecting their ages and technology available at the age of installation. The seismic design standards have been improved year by year. This resulted in difficulty for water utilities to take comprehensive measures for earthquakes. Seismic reconnaissance surveys are required to evaluate vulnerability of water supply facilities. One recommended method might be evaluation of the facilities and equipment based on their ages. In case of water supply system, effectiveness of priority options generally follow characteristics of the Case (2) stated above. Options should be according to the water flow direction from intake to distribution facilities. In this paper, emergency pipelines and power generators are considered as one of the options.

Apart from the considerations during design and construction, emergency water supply to save the public from water shortage are generally grouped into two: 1) measures to restart normal water supply service after earthquakes and 2) measures for emergency water supply.

As measures 1), following options are considered effective:

- (1) Immediate evaluation of equipment, pipe routes, and areas including valve operation and control to minimize damages,
- (2) Purchase and stockpiling of emergency supply and establishment of delivery source under emergency,
- (3) Rehabilitation and restoration for immediate repair of damaged facilities, and
- (4) Establishment of organization, emergency operation manuals, training, communication for restoring from seismic damages.

As measures 2) for emergency water supply, following options are considered effective:

- (1) Raw water extraction from other sources, establishment of emergency water supply points, supply through water tank or storage, storage as bottled water,
- (2) Water treatment by mini-filtration, and desalination plants,
- (3) Expansion of water transmission facilities under emergency, and
- (4) Institutional strengthening for emergency water supply.

All the above are largely dependent to the daily activities of water utilities for earthquake preparation. Many options available can be classified through the said procedures. Fig-5 shows planning procedures for seismic damages.

First, adequacy of water supply facilities for seismic damages are evaluated through seismic reconnaissance surveys and/or information collection. Then, vulnerable area or facilities should be identified for further seismic considerations. For vulnerable areas due to transmission and distribution pipelines rupture, back-up supply sources from the neighboring water utilities or rerouting of pipelines are to be established. For facilities supposed to be damaged by earthquakes, more strict design criteria should be applied to minimize the seismic damages of the facilities. Once the areas are found most vulnerable and obliged to stop water supply during a long period, it is fundamental to evaluate water loss amount in advance. Emergency water supply should be based on such analyses.

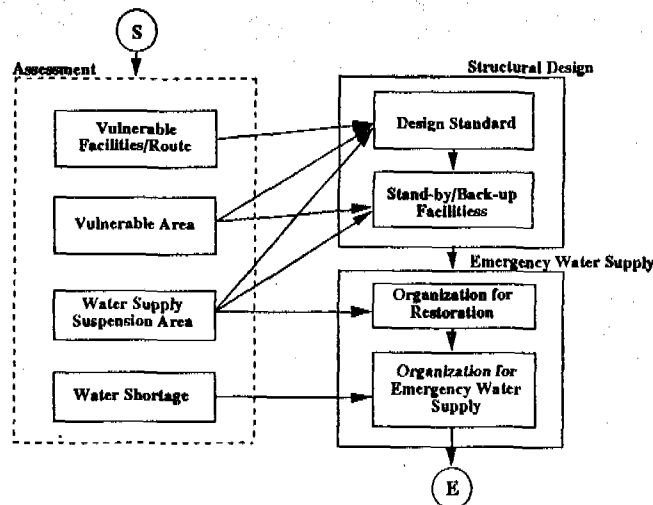


Fig-5 PLANNING PROCEDURES

4. CONCLUSION AND ACKNOWLEDGEMENT

This paper described tentative results of the study team with an emphasis on planning procedures for saving the water supply facilities from the seismic damages. Followings are considered important in the course of the study:

- (1) In this paper, a methodology how to assess the vulnerable area and facilities to the earthquakes is not necessarily in detail. It is recommended to make more precise review of data and information on earthquake damages. This might enable water supply system planners to establish simple methods for the assessment.
- (2) The options listed above are not sufficient. More options practical and applicable to each water utility should be studied further.
- (3) Effects of earthquakes are tentatively discussed in this paper. Much attention should be paid to comprehensive measures to ensure reliability of water supply under floods, typhoon and droughts as well as earthquakes.

We would like to express our heartfelt acknowledgment to Messrs. Yoshio Yagi and Kazuaki Mori, Ministry of Health and Welfare who kindly gave us valuable advice and comments on the whole area of the study team activities. In preparing the present paper, all advisory committee members and particularly the Chairman, Prof. Masaru Hoshiya have kindly made a review of the draft Report with many useful comments. We also appreciate all efforts to collect relevant data, made by Messrs. Kazunori Mayama and Kazuhiro Itou, Nihon Suido Consultants, Co., Ltd.

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TECHNICAL SESSION 11C
PIPELINE MATERIALS AND SURGE ANALYSIS

Why Far East Utilities are Turning to MDPE/PE80

**Estimating Dynamic Loads on Pipe Bends by Passage of
Water Hammer Waves**

Surge Analysis of a Sewerage Pumping Main

WHY FAR EAST UTILITIES ARE TURNING TO MDPE/PE80

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Wavin Industrial Products Ltd.
Great Britain

Overview

Polyethylene is not a new material to the Water Industry in many parts of the World, where both LDPE and HDPE have been used for domestic services for many years.

In the UK the preferred material grade is medium density polyethylene (MDPE/PE80), originally developed for use in gas mains where long life and high integrity are essential for safe operation.

Now over 5000km of MDPE/PE80 is installed by UK Water Companies each year, in sizes up to 710mm.

In the Far East MDPE has added attractions: ease of installation, less reliant on skilled labour, lower long-term costs, adaptability to local conditions. The material also offers attractions in terms of technology transfer and production of MDPE/PE80 pipes has started in 1992 near Kuala Lumpur, under licence from a British Company.

This paper discusses the technical and practical aspects involved in the use of MDPE/PE80 for water engineers in the Far East.

The Material

Polyethylene is a plastic produced by the polymerisation of ethylene under various conditions using a range of catalysts. The resulting products differ considerably in molecular structure and hence in properties.

Molecular weight, for example, is critically dependent on polymerisation temperature. There are thus hundreds of polyethylenes. Parameters which control their physical properties (and hence application) include molecular weight, molecular weight distribution and volume fraction crystallinity.

Molecular weight distribution strongly influences such properties as the melt flow characteristics of the material and its resistance to brittle fracture. The relative number of small molecules affects fracture characteristics since they form weak layers in the material.

Generally, the higher the molecular weight, the tougher the polyethylene.

Design Factors

The most important property of a buried pipeline is its long term reliability under service loads. Failure can be expensive and disruptive.

It is the water engineer's responsibility to choose materials that minimise the risk of failure. With ferrous materials reliability is reduced by corrosion. MDPE/PE80 will not corrode and has very high resistance to chemical and bacteriological attack. Modern additives also prevent deterioration due to heat or light.

This protection used to be provided by carbon black, but alternatives have been developed so that colour coded pipe is now the norm. (In Britain, for example, all MDPE/PE80 pipe for drinking water are light blue in colour). These additives can be formulated to meet even the most stringent toxicological requirements for potable water distribution and do not cause taste and odour problems.

Since corrosion can be ignored, the strength of the pipeline may be tailored to meet the operating pressures and secondary loads likely to be imposed.

Polyethylene creeps under load, its apparent strength reducing with time. This phenomenon can be analysed by pressure tests at 20 degrees Celsius carried through to failure.

The system designer usually extrapolates these 20 degree Celsius tests to a suitable service life - typically 50 years. A maximum design stress can then be calculated by dividing the extrapolated 50 year stress by a suitable safety factor. Maximum design stresses for LDPE, HDPE and MDPE are shown in Appendix I.

Close comparison between stress rupture characteristics of HDPE and MDPE indicates another very important difference in their behaviour. For HDPE at long test lives at 20 degree Celsius (or at shorter lives at higher temperatures) there is a significant increase in the slope of the stress rupture characteristics: the longterm performance of MDPE/PE80 is far superior to that of HDPE (Appendix II).

This sudden change in slope co-incides with the transition from ductile failure to a brittle failure mode. With MDPE/PE80 the onset of brittle failure is delayed beyond the 50 year design life by the higher resistance of the material to show crack growth. This is particularly important for a buried pipeline, since failure can be caused by slow crack growth initiated at defects or stress concentrations.

It is well known that pressure transients in pumped water mains can lead to premature failure. Recent tests on polyethylene systems under cyclic pressure loading at elevated temperatures indicate that they can be made resistant to fatigue loading conditions.

The failure mode is similar to that produced under static pressure

loading at elevated temperature: cracks develop at stress concentrations and grow through the pipe wall. Tests also show that the higher a material's resistance to stress crack growth the better its fatigue performance.

Under impact loading conditions some materials will shatter, others may have their protective coating damaged leading to eventual failure. However, polyethylene can tolerate such loads without any loss of performance. If the nature of the impact is such that it does damage polyethylene pipe (for example, a mechanical digger) then the damage will be localised, easily identifiable and easily repairable immediately. Under certain conditions, impact damage can lead to the development of a crack that propagates at high speed over a considerable distance. Full scale testing in Britain of MDPE/PE80 pipe filled with water, at 3 degrees Celsius and at twice the rated pressure, has shown that such cracks are arrested a very short distance from their initiation points. There are no recorded instances of an MDPE/PE80 water pipe failing in the UK in this way: in other pipeline materials this problem is more of an everyday occurrence.

Installation Factors.

The main installation factors for any buried pipeline are:

- * Ease of handling
- * Flexibility
- * Suitability for various jointing techniques

In all cases MDPE/PE80 offers major advantages over more traditional materials. For instance, obstacles in the ground can be avoided by using pipe flexibility alone (MDPE/PE80 can be bent to a radius of up to 15 times the pipe diameter). This avoids the need for special fittings. pipes can be jointed above ground, enabling trench widths to be kept to virtually the width of the pipe and any imported backfill necessary to be greatly reduced.

Pipes of up to 180mm diameter can be supplied in coils or on drums. This reduces the number of joints to be made, improving integrity and speeding up installation as well as reducing costs. Straight pipes can be supplied in lengths up to 30m. Even larger savings can be achieved where system renovation can be done by insertion of the MDPE/PE80 pipe into the old main instead of replacement. This not only reduces costs; it also eliminates disruption in congested urban areas. Where the capacity of the old main has to be maintained pipe bursting techniques can be used, to break up the old pipe in-situ and pull in the new MDPE/PE80 pipe behind.

More recent developments include the 'roll-down' technique: a thin-walled MDPE/PE80 pipe is reduced in size prior to insertion into the old main.

Joints made by fusion welding are attractive to the water engineer because they maintain homogeneity within the system, effectively eliminating one of the major areas of weakness in other materials - the joints themselves. Laboratory testing of MDPE/PE80 pipe to pipe

and pipe to fitting fusion welds continually proves fusion joints to be at least as strong as the pipes themselves. Such joints also provide full end load resistance, negating the need for anchorages to prevent joint withdrawal.

Fusion welding uses a combination of controlled heat and pressure for a specified time to ensure 100% joint security. The two most common techniques are:

- * Electrofusion - a wire coil within an MDPE/PE80 fitting is energised by a power source. Controlled melting of the fitting and pipe takes place, creating the fusion weld.

Operation of the entire process is kept at a safe, basic level by the use of simply-operated pipe preparation tools and an automatic fusion cycle started by the power source when the operator pushes a single start button.

- * Butt fusion - the pipe ends are trimmed square and heated by a flat plate. On removal of the plate the pipes are brought together and left cool.

Security of the joint is determined on cooling, by dimensional checks of the bead created during the heating cycle. In critical applications the internal bead can be removed to give even better flow rates. A Data Retrieval System is available, giving a print-out of every butt fusion joint made and offering additional evidence that the joint has been made correctly.

Whilst using a combination of these two techniques covers all pipe sizes available in MDPE/PE80, there are still instances where mechanical fittings may be needed - for example, at valves or at transitions to an existing system in a different material.

One British company interviewed in the early 1980's a large cross-section of water authority people, ranging from Managers to service layers and mains layers, regarding the features they wanted in a mechanical fitting for MDPE/PE80 pipe. They quickly identified the following:

- * quick and easy to assemble;
- * no loose components to fall out or fit the wrong way around;
- * unaffected by over or under tightening of nuts etc.;
- * tolerant of pipe which has not been cut squarely;
- * same type of fittings for both services and mains pipes.

Wavin can offer a range of flanged sockets for mains pipes, in nylon coated aluminium, which are lightweight, corrosion resistant and provide a water tight seal designed for a minimum design life of 50 years - the same as the MDPE/PE80 pipes and fusion weld fittings. The joint between the pipe and the socket is end load resistant, so again no anchor blocks are required.

Some British manufacturers have extended the range of their 'pushfit' fittings into service sizes also. Appendix III shows diagrammatically how the principles of the 'pushfit' concept work.

Installation Costs

The installation cost of any buried pipeline system involves many factors, including the cost of materials, excavation and reinstatement, investment in ancillary equipment and the cost and availability of trained labour. Materials costs can usually be qualified easily, though variations in feedstock and energy prices can lead to fluctuations.

However, as energy prices rise, plastics show an improved cost index relative to alternatives which require higher energy input. An indication of current costs for alternative materials is given in Appendix IV. Up to 180mm diameter MDPE/PE80 is competitive. Above this diameter it is less competitive metre for metre ex-works, but remember that this can be significantly improved through the lower cost of fittings and installation.

Typically for a 125mm main, materials cost 15-25% of the total cost of the project. By far the largest cost is excavation and reinstatement which can absorb 70% of system costs. However, MDPE/PE80 can reduce this since above ground jointing leads to the use of narrower trenches, while material flexibility allows problem areas to be avoided with ease.

This means that, even in sizes over 180mm, MDPE/PE80 can usually show a saving in total installed cost to the water engineer - and I have not even included the extra savings that you will achieve through a minimum 50 years maintenance free life. The use of trenchless techniques, as originally practised in land drainage, is possible on a large scale. That cuts costs even more, despite the special machinery, and may well put MDPE/PE80 beyond the reach of alternative material less adaptable to new technology. This installation method has been used with great success in Britain where water engineers are now able to cost-efficiently connect smaller rural villages to mains water supplies for the first time- thanks to MDPE/PE80.

Conclusions

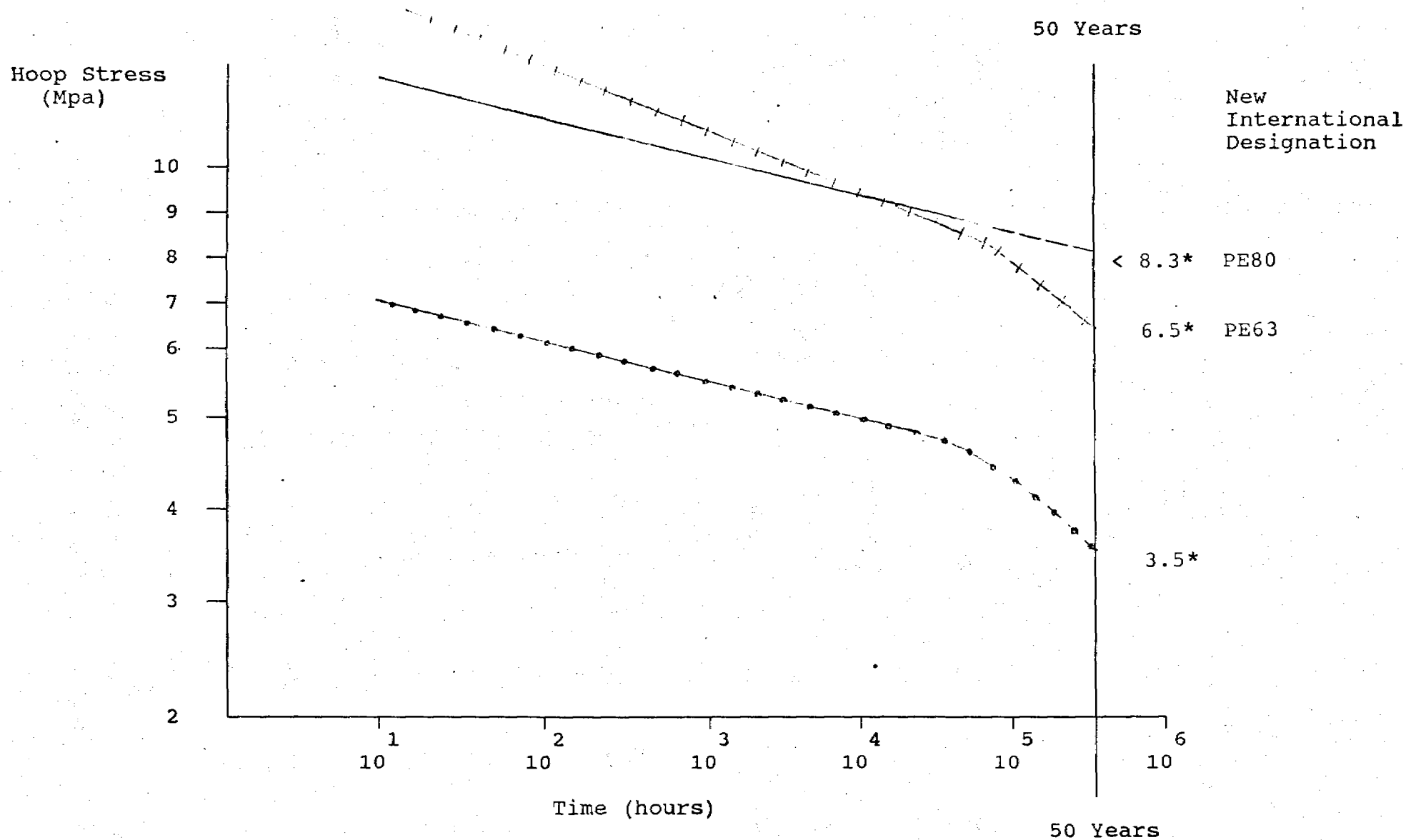
MDPE/PE80 pipe systems for drinking water are now well established in Britain and mainland Europe because they provide the water engineer with solutions to problems affecting existing systems and opportunities to provide cost effective new systems. MDPE/PE80 systems are corrosion resistant and flexible, and offer 100% secure, leak-free jointing techniques. Together with the wide range of installation techniques possible, MDPE/PE80 has all of the ingredients necessary for its immediate success in the Malaysian market.

Appendix I : Design Stresses of Polyethylene Water Systems - UK

PE Type	Design Stress (MPa)	Safety Factor
Low Density (LDPE)	3.2	1.1
High Density (HDPE - PE63)	5.0	1.3
Medium Density (MDPE - PE80)	6.3	1.3

EJO/AH/APPI

APPENDIX II : STRESS RUPTURE CHARACTERISTICS (AT 20°C)

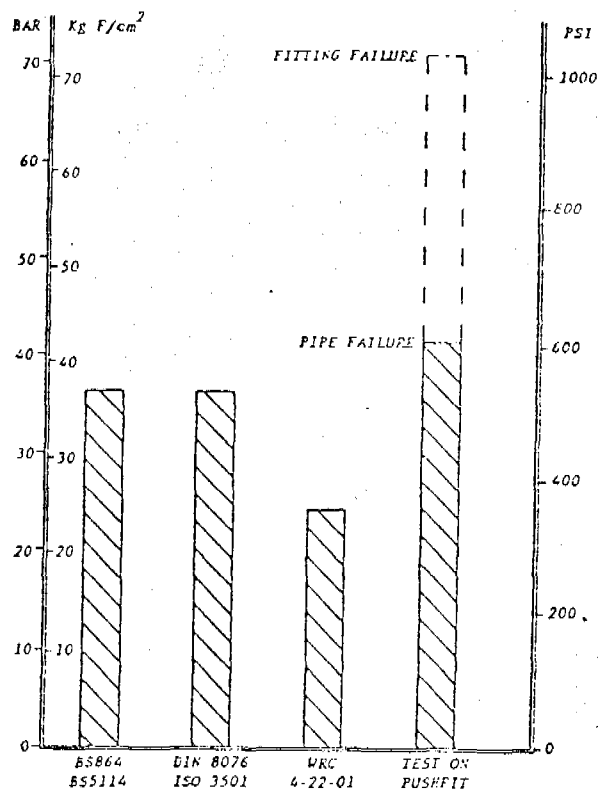


* Predicted failure stress

The Pushfit joint performs well in excess of the requirements of these test standards as can be seen from the following;

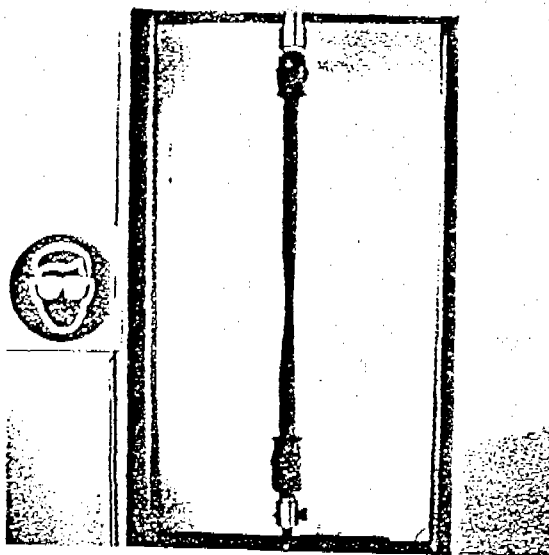
In pressure tests to destruction the pipe fails before the joint does.

PRESSURE TEST PERFORMANCE

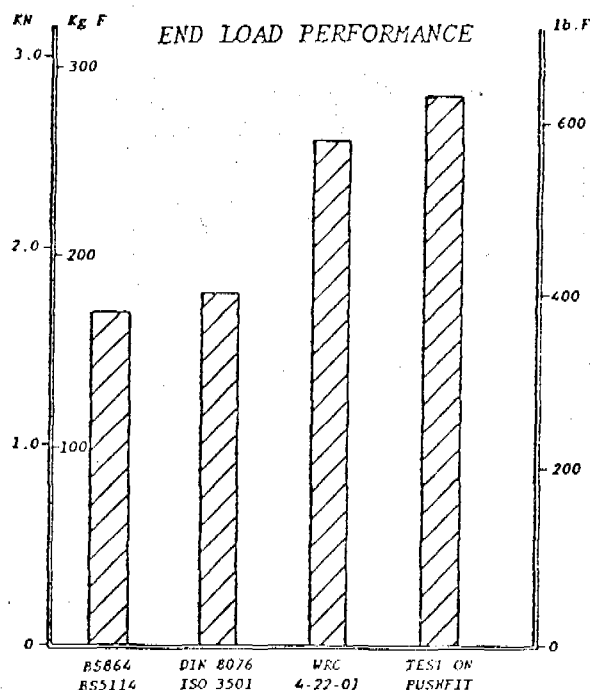


PUSHFIT FITTINGS FOR P.E. PIPE
COMPARISON AGAINST INTERNATIONAL
TEST STANDARDS

In tensile or pull tests to destruction the pipe fails before the joint does.



END LOAD PERFORMANCE

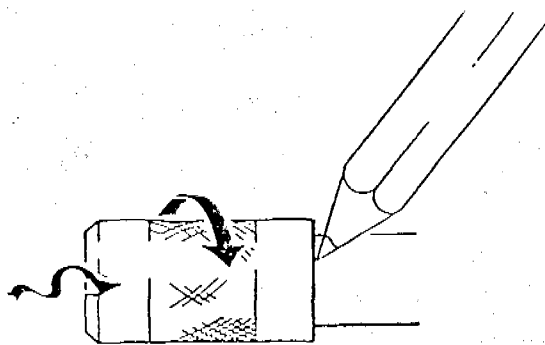


PUSHFIT FITTINGS FOR P.E. PIPE
COMPARISON AGAINST INTERNATIONAL
TEST STANDARDS

As can be seen from the foregoing, the Pushfit joint performs substantially better than the various tests require. In fact the joint is stronger than the pipe itself!

Assembling The Joint

It is recommended that the pipe is chamfered using the purpose designed bevelling tool. This bevelling tool also serves to measure the length of insertion into the fitting.

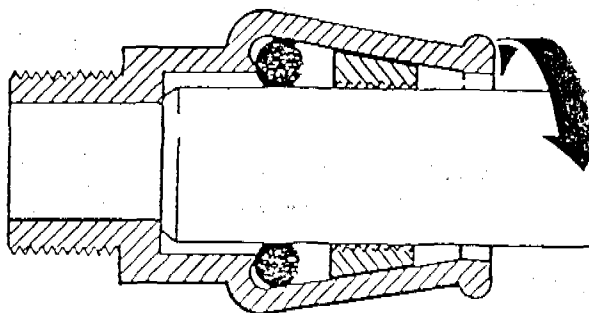


- 1 Bevel Pipe
- 2 Mark Insertion Point

Note; For the majority of applications Pushfit Fittings are used without liners. However, for some specifications of pipe the performance of the joint may be enhanced by using purpose designed nose cone liners.

Push pipe fully home onto fitting with a slight twist.

It is very important to ensure that the pipe passes the two points of resistance of the grip ring and the 'O' ring and is fully seated.



Dismantling The Joint

In general, it is very rarely that a joint is required to be dismantled. Under normal conditions of use the fitting is assembled onto the pipe and the pipe is then buried with no subsequent requirement for maintenance. However, on the rare occasion that a joint needs to be dismantled for example the line needs to be extended or a new valve installed, then the following procedures should be carried out.

Dismantling		4. EXTRACT 'O' RING Inspect, and replace if damaged.	
1. If possible, first ROTATE fitting and PUSH towards pipe.		5. INSERT NEW GRIP RING by forming figure '6' and pressing in.	
2. PUSH EXTRACTORS home and withdraw pipe.		6. Grip ring must be THIS WAY ROUND.	
3. EXTRACT GRIP RING with screwdriver. DO NOT RE-USE grip ring.			

Performance of the Pushfit Joint

As well as being easy to use in what are often cold, wet and dirty conditions, any joint for polyethylene must comply with the various tests that are defined within the ISO DIN and other International Standards to which they will be subjected. These include tests for leak proofness under pressure, resistance to pull out and so on.

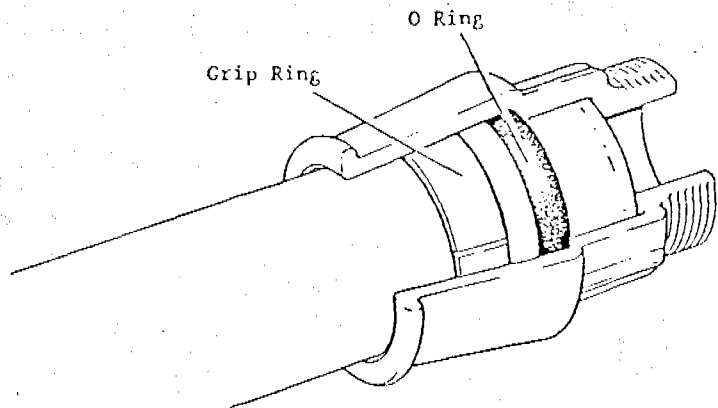
Application of Pushfit Fittings

As can be seen from the foregoing, the decision to use the pushfit joint concept, as the basis of the design for a new range of fittings for polyethylene, came very easily from the results of the comprehensive market research exercise. Basically because it was the only joint which met all the stated requirements of end users.

However, potential end users were asking the question, "How does the joint work?" To those used to having to tighten a nut to make a secure joint it seemed unlikely that such a simple fitting as Pushfit could work effectively.

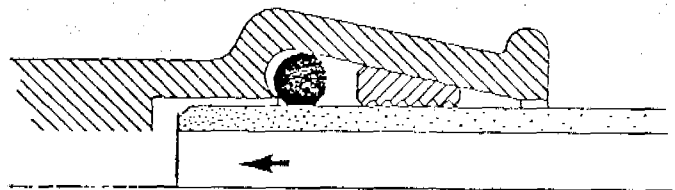
So How Does The Joint Work?

The polyethylene pipe is pushed into the joint firstly past the grip ring, which is designed to stop the pipe pulling out of the fitting under end load or water pressure, and secondly past the 'O' ring which provides the water seal.

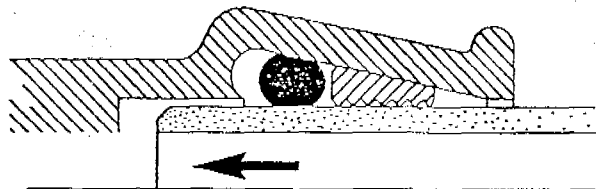


How It Works

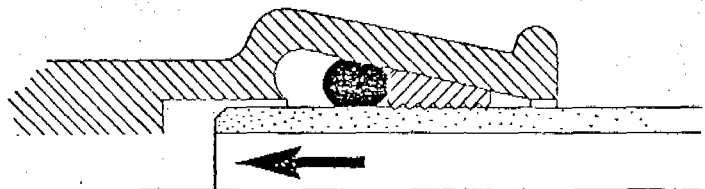
- 1 Immediately after insertion of pipe, with components at rest, 'O' ring provides water seal, grip ring is ready to resist pull-out.



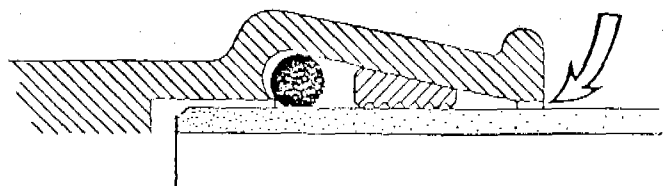
- 2 As water pressure rises, 'O' ring is forced down the constricting body towards grip ring, thus increasing the sealing pressure against pipe and body.



- 3 At high water pressure, 'O' ring is pressed against grip ring and material of 'O' ring flows around inner edge of grip ring to create excellent pressure seal.

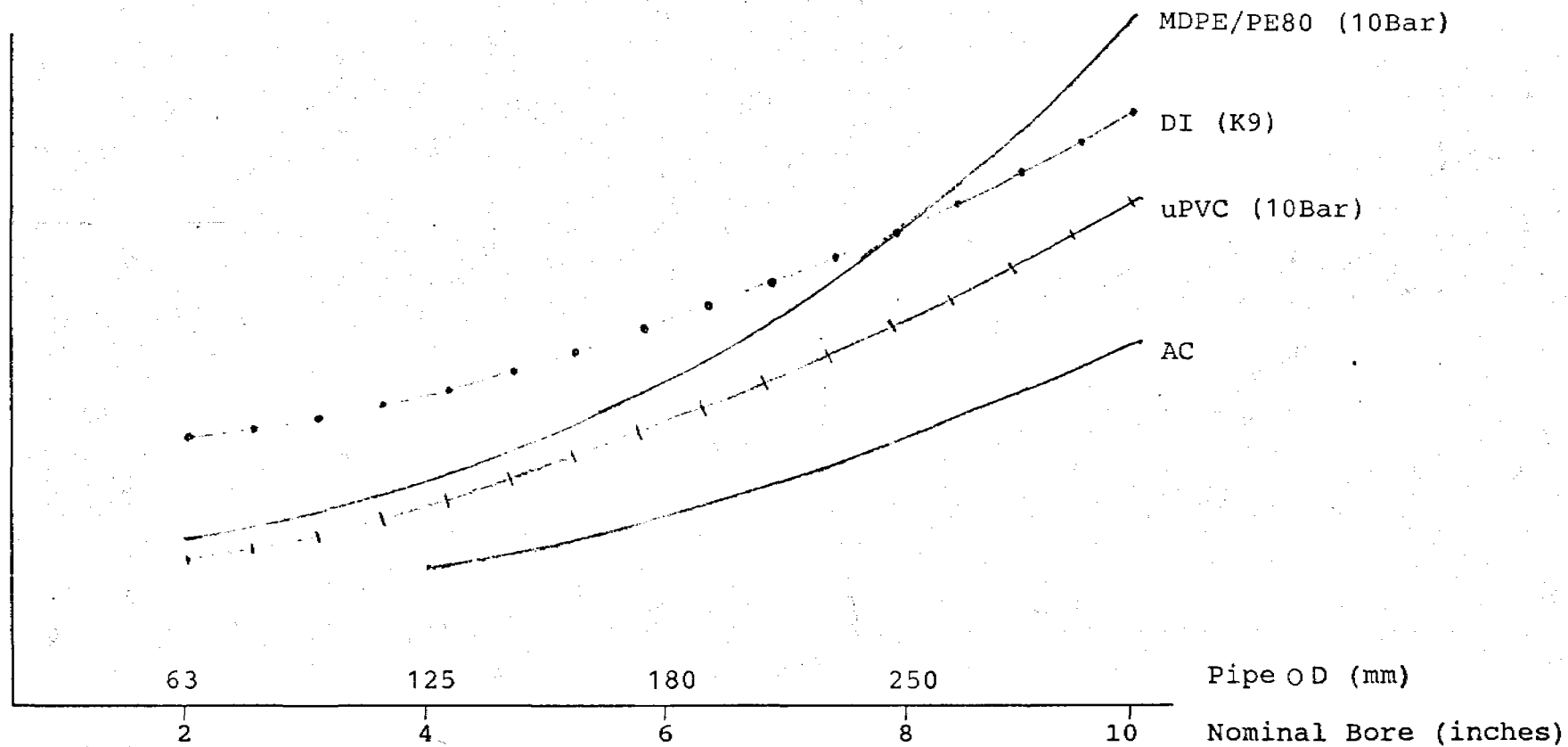


- 4 Under negative pressure conditions the 'O' ring remains in its original seating position and provides an effective vacuum seal.



APPENDIX IV : COST COMPARISON OF ALTERNATIVE PIPELINE MATERIALS

Price/Metre
ex-works



ESTIMATING DYNAMIC LOADS ON PIPE BENDS BY PASSAGE OF WATER HAMMER WAVES

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ABSTRACT : Unbalanced thrusts exerted on a pipe bend by passage of water hammer waves are estimated using one-dimensional momentum equations. Laboratory measurements showed that the peak force experienced by an elbow restraint during passage of water hammer waves was considerably lower than (less than one quarter) that estimated using the static momentum equation. The dynamic loading on the restraint was observed to oscillate at a different (much higher) frequency from that of the excitation transient pressure at the bend. The results show that the static momentum equation, which does not account for fluid-pipe structure interaction effects, is unable to simulate the dynamic response of pipe bends to transient pressure waves, although it could provide a conservative estimate for the peak loading on the bend.

INTRODUCTION

Piping systems are subjected to a wide variety of loadings broadly classified as sustained loads (self weight and pressure), occasional loads (such as wind, seismic, vibration, postulated plant accidents, fluid transients) and expansion loads (including thermal expansion, differential anchor displacements) [5]. At a pipe bend, unbalanced thrusts could be induced by internal fluid flow as a result of change in flow momentum around the bend, as well as action of internal fluid pressure on the bend [1]. Under normal circumstances, the momentum force component is small compared to fluid pressure forces. The pressure forces could arise from steady fluid flow pressure, as well as transient water hammer pressures associated with sudden changes in flow velocity.

Momentary unbalanced forces in piping systems in response to passage of water hammer waves could be very severe, setting the pipes into motion, and inducing reaction forces at pipe supports and restraints. Pipe vibrations and fluid transients have in fact been identified as significant factors affecting overall pipe system maintenance and safety. Analysis of dynamic loadings of a piping system resulting from hydraulic transients is important for evaluating the integrity of the system.

The methods used to determine pipe support loadings under various operating conditions could vary from simple manual calculations to sophisticated computer solutions, depending on the complexity of the

system and criticalness of the pipeline. Computer aided analysis techniques have been developed which enable rigorous dynamic simulation of piping systems' response to predicted water hammer transients, and computation of unbalanced reactions at pipe sections where flow direction changes.

The method of characteristics [6] widely used for analysis of water hammer transients is generally accurate when the piping system is rigidly restrained or massive, such that its motion has little impact on the pressure wave propagation. To account for fluid acoustical and fluid structure interaction effects, it is necessary to simulate the manner in which fluid transients interact with pipe structural motion. Techniques for numerical analysis of fluid transients coupled with the dynamics of pipe motion have been developed [4]. Alternatively, dynamic interaction of the liquid and piping may be modelled by separate analyses of the liquid and solid components, followed by synthesis of the component solutions [3].

Dynamic time series analysis, although accurate, is expensive and time consuming. When a high level of accuracy is not required, static analysis may be used. Simplified procedures for approximate manual calculation of momentary internal unbalanced forces due to water hammer, to enable design of restraints for the affected pipe, have been proposed, which give conservative but reasonable results [2].

In this paper, the reaction forces exerted on a pipe bend by passage of water hammer pressure waves are estimated using one-dimensional momentum equations. Laboratory measurements of the dynamic response of a pipe bend to passage of water hammer waves were carried out and compared with the approximately computed values.

APPROXIMATE COMPUTATION OF REACTION FORCES AT A BEND

The dynamic thrusts exerted on a pipe bend caused by sudden closure of a downstream valve along the pipeline shown in Figure 1 can be computed using the one-dimensional momentum equations. The following assumptions are made :

1. the bend lies in a horizontal plane and is rigidly restrained in the x direction at its corner
2. all vertical loads (self-weight, etc.) are carried by flexible hangers which do not provide any restraint in the horizontal plane
3. all flow induced thrusts at the bend must necessarily be in the horizontal directions and are resisted by the support at the bend
4. flow is initially steady at velocity V_0 , and valve closes instantaneously and completely at time $t = 0$
5. pipe flows full all the time, and flow is frictionless and one-dimensional

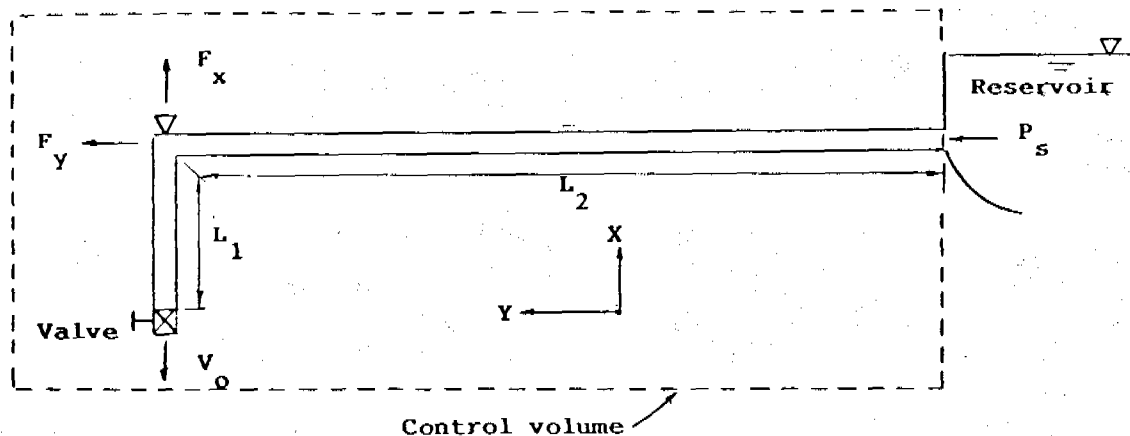


Figure 1. Definition diagram

The components of flow-induced reactions on the bend support in the x and y directions, F_x and F_y , following sudden valve closure, can be computed by applying the momentum equation to the body of fluid within the pipeline as follows :

a. For $t < 0$

The solenoid valve is fully open and flow along pipe is steady at velocity V_0 . Applying momentum equation to the fluid within the control volume (volume of fluid within the pipeline) in the x-direction :

$$\begin{aligned}
 F_x' &= \text{resultant force exerted on fluid body in the control} \\
 &\quad \text{volume by pipeline in x direction} \\
 &= \text{change in momentum of fluid in control volume in x-direction} \\
 &\quad + \text{rate of momentum outflow from the C.V. in x direction} \\
 &\quad - \text{rate of momentum inflow into the C.V. in x direction} \\
 &= M_x + M_{x0} - M_{xi} = -\rho A V_0^2
 \end{aligned}$$

where A = cross-sectional area of pipe.

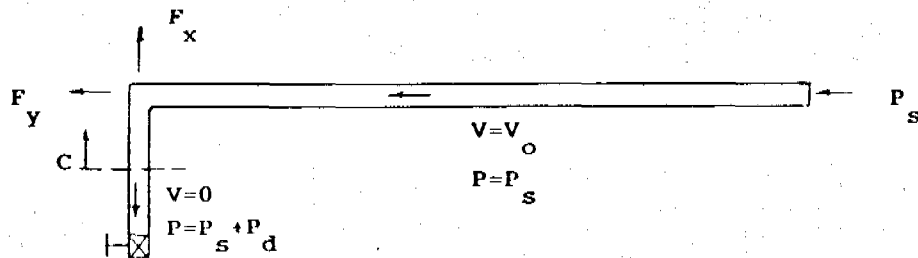
Applying the momentum equation in the y direction :

$$\begin{aligned}
 F_y' + P_s A &= M_y + M_{y0} - M_{yi} = -\rho A V_0^2 \\
 F_y' &= -P_s A - \rho A V_0^2
 \end{aligned}$$

where F_y' = force exerted on fluid in the C.V. by the pipeline in the y direction, and P_s = initial steady fluid pressure at the reservoir end of pipeline.

The corresponding reaction forces exerted on the bend support by the fluid are $F_x = -F_x' = \rho A V_0^2$, and $F_y = -F_y' = \rho A V_0^2 + P_s A$, respectively.

b. For $0 < t < t_1$



At $t = 0$, the valve is closed instantaneously, resulting in generation of a high pressure wave at the valve which propagates upstream at a celerity C . Fluid upstream of the wave front continues to flow towards the valve at V_0 until it is stopped by the advancing wave front. The wave front reaches the bend at time $t_1 = L_1/C$, where L_1 is the length of first (shorter) leg of the bend. The dynamic pressure rise P_d resulting from the flow stoppage is given by :

$$P_d = \rho C V_0$$

Applying momentum equation to the C.V. in the x-direction :

$$F_x' = M_x + M_{x0} - M_{xi} = \rho A V_0 C$$

($M_x = \rho A V_0 C$ as a mass of fluid $\rho A C$ with velocity V_0 is brought to rest every second by propagation of the wave front).

Applying the momentum equation to the C.V. in the y-direction :

$$F_y' + P_s \cdot A = M_y + M_{y0} - M_{yi} = -\rho A V_0^2$$

$$F_y' = -\rho A V_0^2 - P_s \cdot A$$

The corresponding reaction forces on the bend are :

$$F_x = -\rho A V_0 C$$

$$F_y = \rho A V_0^2 + P_s \cdot A$$

c. For $t_1 < t < (t_1 + t_2)$

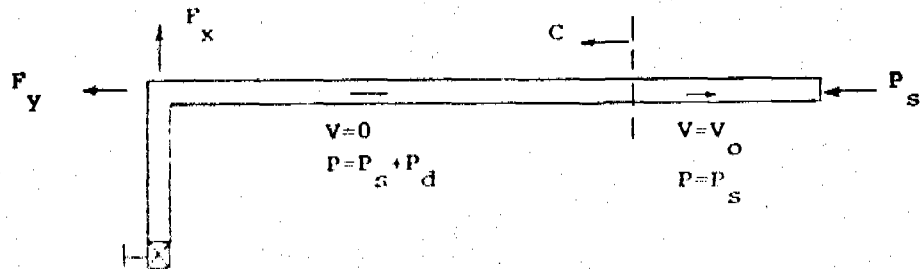
The pressure wave front now travels along the second (longer) leg of the bend (L_2). The reaction forces acting on the bend support can be computed as follows :

$$F_x = 0$$

$$F_y = \rho A V_0^2 + P_s \cdot A + \rho A V_0 C$$

d. For $(t_1 + t_2) < t < (t_1 + 2 t_2)$

At $t = (t_1 + t_2)$, the pressure wave front reaches the reservoir. The pressure head difference at the reservoir causes the fluid to flow from the pipe back into the reservoir at velocity V_0 . A negative surge propagates back to the valve reducing the pressure at the reservoir end to the original steady flow pressure P_s . Upon passage of the wave front, pressure returns to normal (original), while velocity is reversed ($-V_0$), as illustrated below :



The reaction forces at the bend can be computed as :

$$F_x = 0 \text{ and } F_y = \rho A V_0^2 + P_s \cdot A + \rho A V_0 C$$

e. For $(t_1 + 2 t_2) < t < (2 t_1 + 2 t_2)$

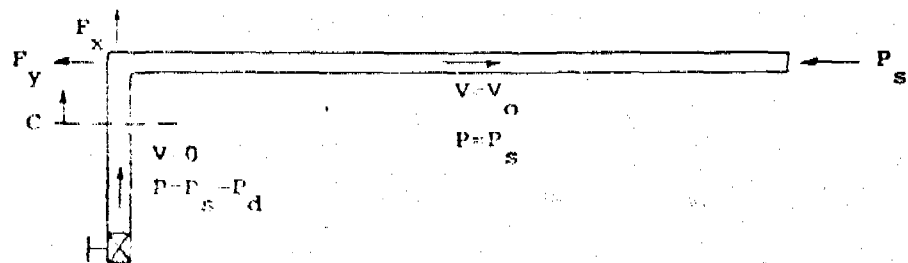
The wave front now travels along the first leg of the bend and the reaction forces are give by :

$$F_x = - \rho A V_0 C$$

$$F_y = \rho A V_0^2 + P_s \cdot A$$

f. For $(2 t_1 + 2 t_2) < t < (2 t_2 + 3 t_1)$

At $t = 2 (t_1 + t_2)$, the wave front reaches the valve. As there is no fluid available to maintain the upstream flow at the valve, the normal pressure is reduced by the dynamic pressure change P_d to bring the fluid to rest. This wave of reduced pressure propagates back toward the reservoir. Upon passage of the wave front, the fluid comes to rest, and pressure drops to P_d below normal, as illustrated below :



The reaction forces on the bend are calculated as follows :

$$F_x = \rho A V_0 C$$

$$F_y = \rho A V_0^2 + P_s \cdot A$$

g. For $(2 t_2 + 3 t_1) < t < (3 t_1 + 3 t_2)$

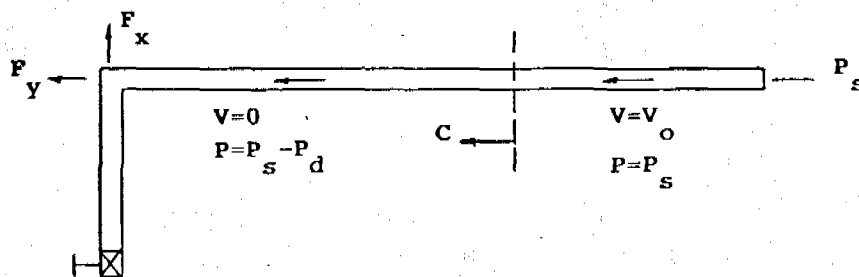
The wave front now moves along the second leg, and the reaction forces are given by :

$$F_x = 0$$

$$F_y = \rho A V_0^2 + P_s \cdot A + \rho A V_0 C$$

h. For $(3 t_1 + 3 t_2) < t < (3 t_1 + 4 t_2)$

At $t = 3(t_1 + t_2)$, unbalanced pressure at the reservoir causes the fluid to flow back into the pipe at velocity V_0 . A pressure wave at the original pressure propagates downstream towards the valve as illustrated below :



The reaction forces can be calculated as follows :

$$F_x = 0$$

$$F_y = \rho A V_0^2 + P_s \cdot A - \rho A V_0 C$$

i. For $(3 t_1 + 4 t_2) < t < (4 t_1 + 4 t_2)$

The wave front now travels along the first leg, and the reaction forces can be calculated as follows :

$$F_x = \rho A V_0 C$$

$$F_y = \rho A V_0^2 + P_s \cdot A$$

The above cycle continues to repeat periodically every $4(t_1 + t_2)$ time period. In a real pipe system, the water hammer waves would be dampened out by frictional dissipation.

In general, the force component due to flow direction change, $\rho A V_0^2$ is small compared to the water hammer force on the pipe segment, $\rho A V_0 C$. The variation of pressure at the valve P_v , and the F_x and F_y reaction forces during one cycle period are as shown in Figure 2.

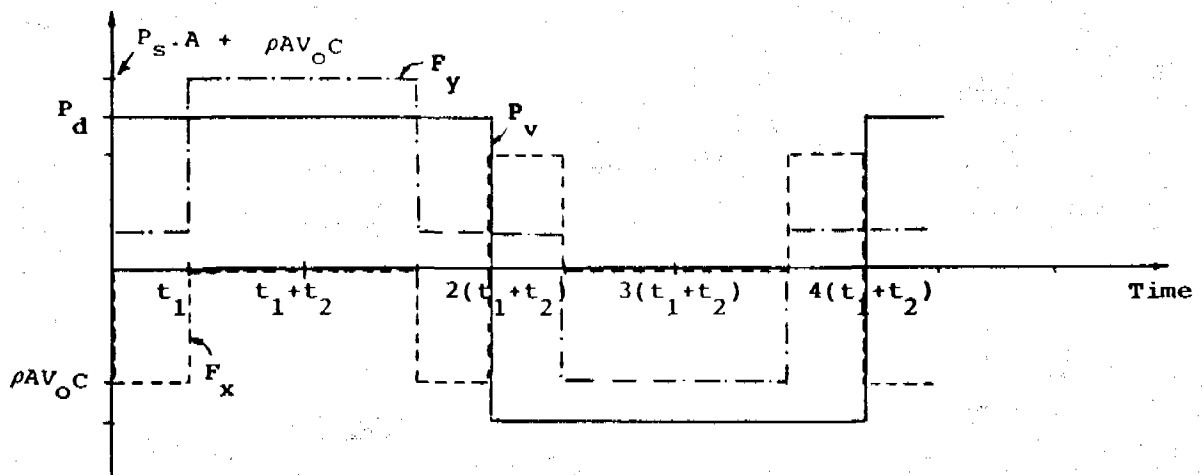


Figure 2. Variation of pressure and reaction forces at bend

EXPERIMENTAL MEASUREMENT OF SUPPORT DYNAMIC RESPONSE

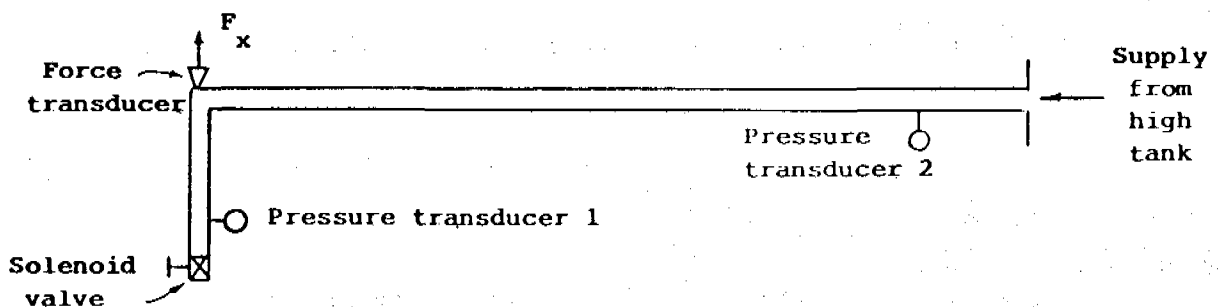


Figure 3. Schematic layout of experimental pipeline

An experimental pipeline similar to the system considered above was set up in the laboratory to permit measurement of the dynamic response of the bend to passage of water hammer waves. A schematic layout of the experimental pipeline is as shown in Figure 3. The pipe was constructed of a 15 mm internal diameter copper tubing, with a total length of about 40 m. Two pressure transducers (PCB Piezotronics PW-10 with a nominal range of between 0 to 100 m of gauge pressure head) were installed for measurement of transient pressures at the valve, and at 35 m upstream of the valve. The F_x component of the bend reaction was measured using a force transducer (PCB Piezotronics Model 208 B with a range of up to 4.2 Kg in both tension and compression) fixed to the pipe bend, which served as a rigid restraint in the x-direction for the bend. Outputs from the transducers were acquired directly onto a Personal Computer at a rate of between 300-400 Hz, with the assistance of a data acquisition software ASYSTANT+. During each test run, the transient reaction force F_x and pressures arising from the sudden closure of the solenoid valve were measured for up to about 4 seconds, after which the transients were observed to have been substantially dissipated. The F_y component was not measured as it was resisted by axial rigidity of the pipeline. A typical output from the three transducers is as shown in Figure 4.

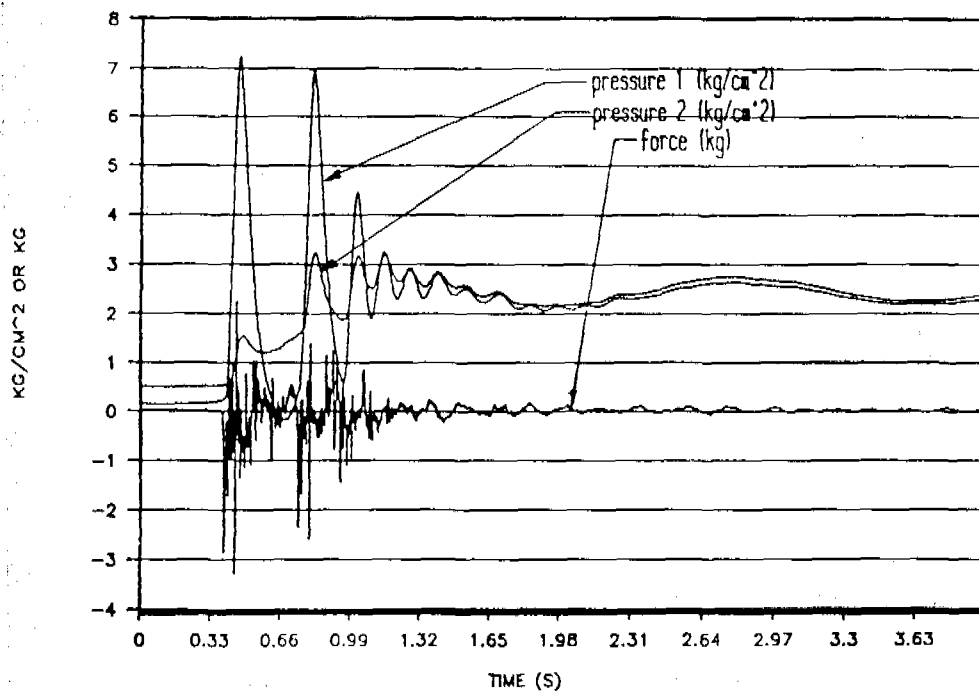


Figure 4. Typical output of transducers (Test 5)

DISCUSSION OF RESULTS

Measurement of wave celerity C

The celerity of the water hammer waves was determined from the time lag in the response of the two transducers to the passage of the first surge. From the results of fifty measurements, the average value of wave celerity C was determined to be 1301 m/s. The value of C was also be computed from known physical properties of the pipe-fluid system using the following equation :

$$C^2 = \frac{K/\rho}{1 + C_1 \frac{KD}{Ee}}$$

where K = bulk modulus of elasticity of water estimated at 2.23×10^9 N/m², E = modulus of elasticity of copper estimated at 1.2×10^{11} N/m², D = internal pipe diameter = 0.015 m, e = pipe wall thickness = 0.0007 m and $C_1 = 1 - \mu/2$ for pipe anchored at its upstream end, where μ = Poisson's ratio for copper = 0.3. The calculated C value of 1290 m/s is very close to the average experimentally measured value.

The Darcy-Weisbach friction factor of the pipe was also estimated from the pressure readings during the initial steady flows before valve closure, using the equation :

$$f = \frac{2D}{\rho LV^2} \Delta P$$

where L = spacing, and ΔP the pressure drop, between the two transducers. The average f value from 11 measurements at a Reynolds number of 30,000 was 0.028. This gives a relative roughness for the pipe of about 0.0002.

Measurement of Peak F_x

The peak F_x experienced at the support was associated with the passage of the first surge after closure of the solenoid valve. The measured peak value of F_x from 12 test runs involving bend included angles of 90° , 135° and 150° are summarised in Table 1 below.

Table 1
Comparison of measured and computed Peak F_x

Test	θ (degree)	Δh (m)	ΔV (m/s)	Peak F_x measured (N)	Computed F_x (N)	$\frac{F_x \text{ measured}}{F_x \text{ computed}}$
1	90	118	0.89	43	205	0.21
2	90	130	0.98	43	225	0.19
3	90	133	1.00	40	231	0.17
4	90	136	1.03	43	236	0.18
5	90	75.5	0.57	33	130	0.25
6	135	117	0.88	38	203	0.19
7	135	120	0.91	49	208	0.24
8	150	72	0.54	28	124	0.23
9	150	87	0.66	33	151	0.22
10	150	77	0.58	30	134	0.22
11	150	74	0.56	34	128	0.27
12	150	58	0.44	21	101	0.21

In Table 1, Δh is the maximum dynamic pressure head rise measured at the valve, while ΔV is the effective flow velocity change required to generate the dynamic pressure head rise of Δh computed from :

$$\Delta V = \frac{g}{C} \Delta h$$

The computed value of F_x is obtained from :

$$F_x = \rho A V_0 C$$

assuming V_0 is $\ll C$, and value of P_s at the valve is negligible.

The measured peak F_x was found to be significantly lower than (average about 0.21) that calculated using the static momentum equation, for all the bend included angles tested. The loading experienced by the force transducer was also observed to oscillate at much higher frequencies compared to the frequency of the excitation pressure at the valve (Figure 4). The relatively low value of measured F_x could indicate that the total thrust developed at the bend was probably not fully transmitted onto the force transducer, but could have been partially resisted by the flexural stiffness of the pipeline. The high frequencies of the force response show that the bend and pipeline could oscillate at frequencies different from that of the excitation pressure. The results show that the static momentum equation is not able to predict the dynamic response of the bend, although it gives a very conservative estimate for peak support reaction.

CONCLUSIONS

The reaction forces exerted on a pipe bend by water hammer waves computed using the static momentum equation were found to be significantly higher (more than 4 times) than laboratory measured values. The loading response of the bend was also found to oscillate at much higher frequencies compared to that of the excitation water hammer pressure waves. The deviations show that the static momentum equation, which does not account for fluid-structure interaction effects, is unable to simulate the dynamic response of the bend. The unbalanced thrust developed at the bend was likely to have been partially resisted by the flexural stiffness of the pipeline.

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SURGE ANALYSIS OF A SEWERAGE PUMPING MAIN

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ABSTRACT: This paper describes the surge analysis performed for a 3 km stretch of a sewerage pumping main, to ascertain if the design required any changes arising from adverse water hammer effects. The surge analysis resulted in refinements to the original design. Airvalves were required and they had to be strategically located to keep the pressures within permissible levels. Without any surge protection devices, column separations and rejoins were predicted, with the resulting pressures exceeding the minimum and maximum levels permitted. As the raw sewage is expected to contain dissolved gases, the effects of various amounts of air released in the sewage were also studied. The ameliorating effect of air release is more pronounced in reducing the extreme pressure peaks in the unprotected pipeline than in the pipeline protected with airvalves. A field measurement point has been selected along the pipeline so that comparison between field measurements and predicted values can be made after commissioning.

INTRODUCTION

The sewerage pumping station studied is equipped with three radial flow centrifugal pumps discharging into an existing 500 mm diameter pumping main. An additional pumping main consisting of 600 mm diameter cement lined ductile iron pipes is to be added to the existing main. The pipeline, which extends about 3 km from the pumping station, generally follows the existing ground surface topography, Fig. 1.

Three existing parallel pumps will discharge the sewage either through the existing 500 mm diameter main or the new 600 mm diameter main. Each pump is fitted with a

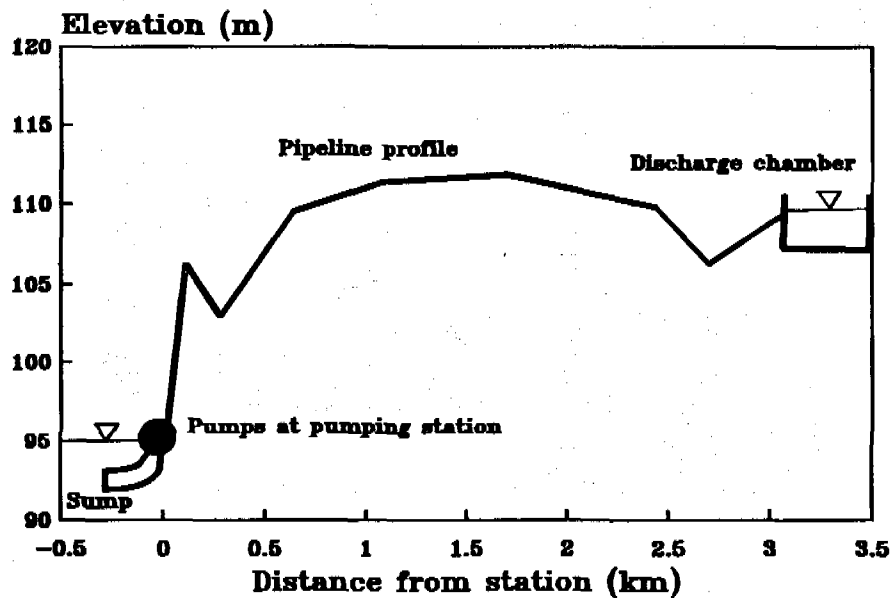


Figure 1 : Longitudinal profile of proposed pumping main

non-return reflux valve on its delivery line. Under normal operating conditions, two of the pumps will be in operation with the third serving as a standby pump. Sluice gates are automatically operated such that no more than two pumps would discharge simultaneously into each main.

A surge investigation was carried out to predict the maximum and minimum pressure envelopes likely to be experienced along the pumping main under all possible operating conditions, and recommend the types and locations of surge control devices.

The following operating conditions along the pipeline are expected to initiate adverse surge conditions and are analysed: pump power start-up; pump power failure in the absence of surge control devices; and pump power failure with proper surge control devices installed. The effects of strategic siting of airvalves and the consideration of air release from the sewage are considered.

Assumptions and Parameters

The pump suction pond (wet well) is simulated as a reservoir at an elevation of 95 m, corresponding to the cut-out level of the first duty pump. Similarly, a reservoir at an elevation of 110.0 m is used to model the discharge chamber.

A number of access and wash-out chambers are present along the pipeline. However, these fittings are only expected to contribute to the additional local minor losses, without any significant effect on the surge propagation. The additional minor losses are accounted for by assuming a slightly higher effective value for the pipe friction factor.

The specified permissible minimum pressure along the pipeline is 0.5 bar or 5.2 m of water below atmospheric pressure, while the specified permissible maximum pressure is 10 bars or 103.4 m of water above atmospheric pressure. Saturation vapour pressure is taken to be 10.0 m below atmospheric pressure.

Because raw sewage contains dissolved and free gases, it would be unrealistic to assume no air release during pressure surges. A minimum amount of 0.0001 % was assumed and this value is expected to be much less than the actual amount of gases present in the raw sewage.

ANALYSIS METHODOLOGY

The surge analysis is carried out using a general purpose computer program for water hammer analysis in complex piping systems. The computer program is based on a finite difference method, namely the method of characteristics. Its formulation follows mainly Wylie and Streeter (1983) in which air release can be taken into account, and Chaudhry (1979). The governing equations are given by:

$$g \frac{\sqrt{C_3 + H_a^2}}{a H_a} \frac{dH_a}{dt} + \frac{dV}{dt} + g \sin \alpha + \frac{fV|V|}{2D} = 0$$

$$\frac{dx}{dt} = a \frac{H_a}{\sqrt{C_3 + H_a^2}}$$

$$-g \frac{\sqrt{C_3 + H_a^2}}{a H_a} \frac{dH_a}{dt} + \frac{dV}{dt} + g \sin \alpha + \frac{fV|V|}{2D} = 0$$

$$\frac{dx}{dt} = -a \frac{H_a}{\sqrt{C_3 + H_a^2}}$$

where

$$C_3 = C_2 m / \gamma^2$$

$$C_2 = \left(\frac{RTK_l}{1 + \frac{K_l}{Ee}} \right)^{\frac{1}{2}}$$

The computer program supports the simulation of a wide range of internal and external boundary conditions, including pump start-up, pump failure, constant speed pump operation, internal and discharge valves, reservoirs, pipe junctions, water column separation and rejoin, surge control devices such as airvalves, surge tanks, pressure regulating valves, etc. When the air release option is activated in the program, the amount of air released in the fluid can be varied using different rates of air release.

RESULTS

Case of No Surge Devices along Pipeline

An analysis was carried out on the pipeline before the inclusion of any surge control devices to determine the potential surges that can occur and their effects. The most severe case of surges is during power failure when two parallel pumps are in operation. Other scenarios were also considered.

Figure 2 shows the predicted pressure envelopes from the surge analysis carried out without air release. It can be seen that low sub-atmospheric pressures during the initial downsurge resulted in column separation occurring practically along the entire length of the pipeline. The pressure envelopes were not acceptable because the minimum pressure dropped to the saturation vapour pressure of about -10.0 m below atmospheric, which is beyond the minimum permissible value of -5.2 m. The subsequent rejoin of the water columns generated pressures as high as 99.2 m above atmospheric.

The analysis of the pipeline was also carried out using different amounts of released air. Figure 3 shows the pressure envelopes obtained when the amount of released air is 0.01%, noting that the maximum and minimum values are 26.6 m and -6.1 m relative to atmospheric. Typically, the small amounts of released air in the liquid reduced the pressure surges considerably.

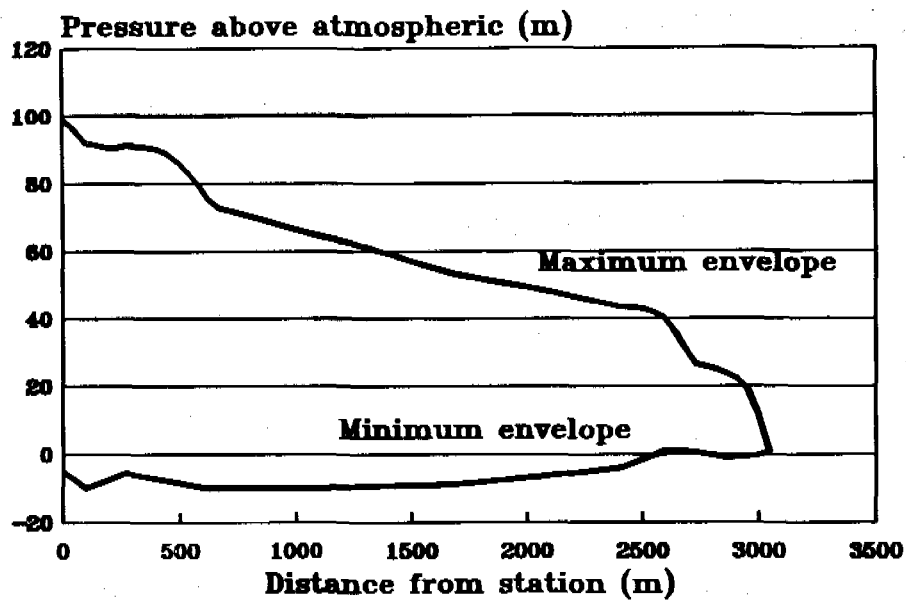


Figure 2 : Pressure envelopes for the unprotected pipeline

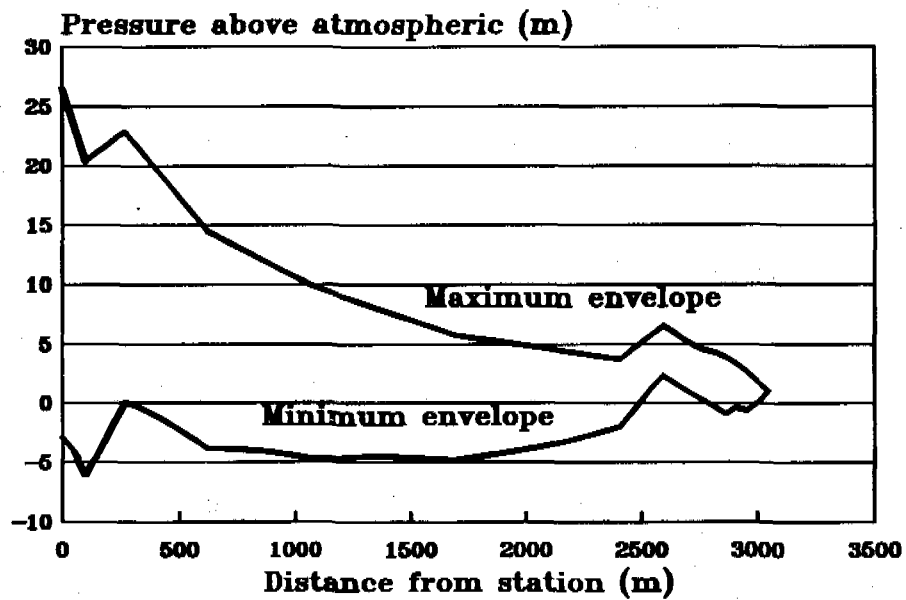


Figure 3 : Effect of 0.01 % air release on pressure envelopes for unprotected pipeline

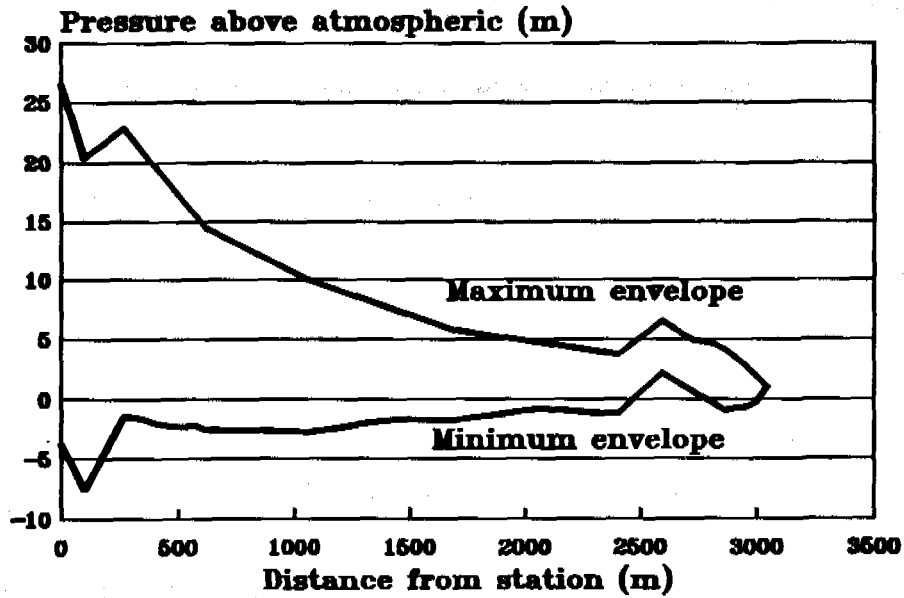


Figure 4 : Pressure envelopes for pipeline with airvalves at chainages 0 m, 620 m and 1690 m

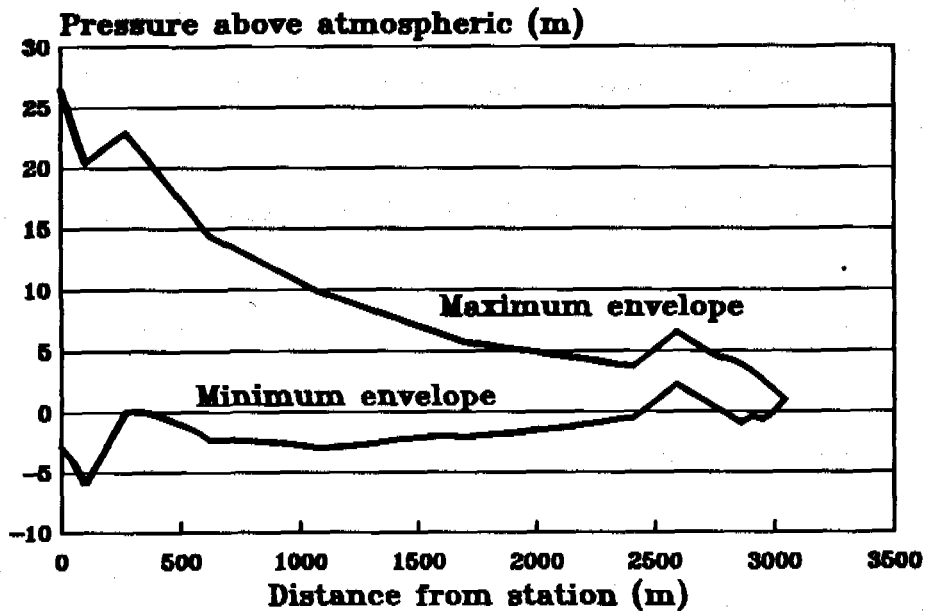


Figure 5 : Effect of 0.01 % air release on pressure envelopes for pipeline with airvalves at chainages 0 m, 620 m and 1690 m

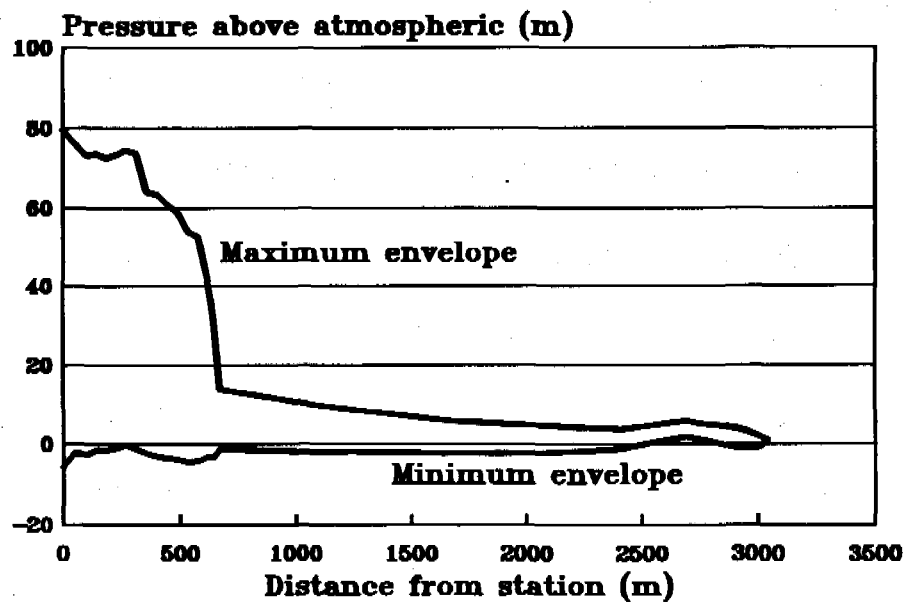


Figure 6 : Pressure envelopes for pipeline with airvalves at chainages 0 m, 671 m and 1280m

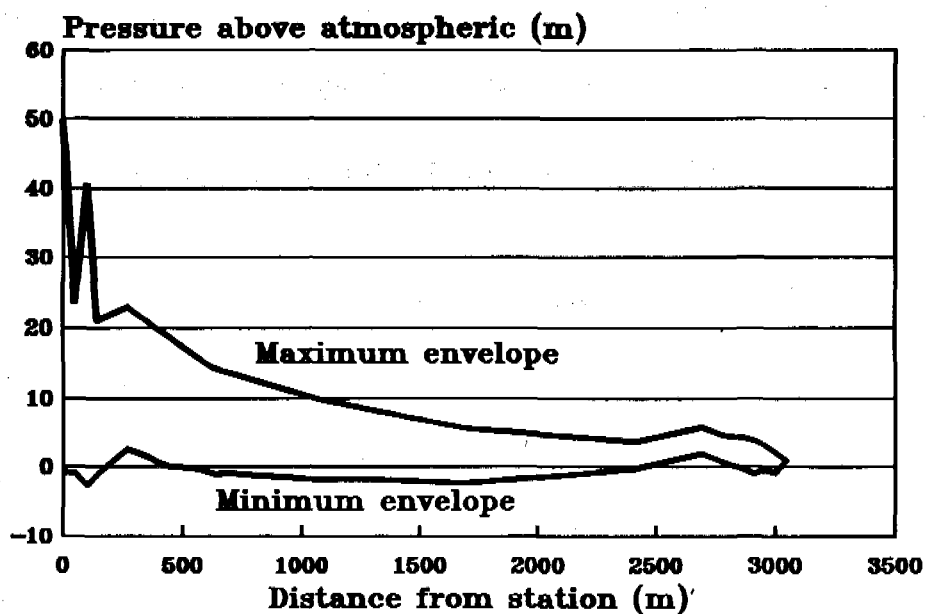


Figure 7 : Effect of 0.01 % air release on pressure envelopes for pipeline with airvalves at chainages 0 m, 671 m and 1280 m

Case of Airvalves Located at Peaks

The original locations of the airvalves are at Chainages 0 m, 620 m and 1690 m. Power failure is assumed to occur while the two new parallel pumps are operating.

From Fig. 4, it can be observed that the installation of the airvalves moderates the pressure fluctuations along the pipeline to within a range of -7.6 to 26.6 m. The minimum pressure is below the minimum permissible of -5.2 m relative to atmospheric.

With an assumed amount of released air of 0.01%, the maximum and minimum pressure peaks are about 26.6 and -6.0m respectively, as can be seen from Fig. 5. For the other rates of air release considered, it is similarly observed that the ameliorating effects of released air are small. The minimum pressure is now closer to but still lower than the permissible value.

Case of Airvalves Relocated

In this case, power failure occurs when the two new parallel pumps are in operation, with the airvalves located at Chainages 0 m, 671 m and 1280 m.

Figure 6 shows that the inclusion of a third airvalve at Chainage 0 and the re-siting of the other two airvalves now moderate the surge pressures to within a range of between -5.7 m to 79.6 m, which are practically within the permissible limits. If an air release amount of 0.01 % is assumed then the pressures range from -2.8 m to 49.8 m relative to atmospheric, well within the permissible range.

DISCUSSION

From the above results, it can be seen that locating the airvalves at the maximum peak points does not necessarily produce the optimum surge mitigation. This is particularly so in using airvalves to protect the pipeline from excessive low pressures.

Sewage invariably contains free and dissolved gases. For realistic analyses, it is therefore essential to consider the effects of air release in any surge analysis involving raw sewage as the transported medium. As can be seen from the results above, the reduction in the transient pressure extremes as a result of air release is more marked in the unprotected pipeline. For example, the increase in the amount of released air to 0.01 % reduced the maximum pressures in the unprotected pipeline by up to 73 %,

while the maximum pressures in the unprotected pipeline are reduced by up to 38 %.

As the surge analysis was carried out prior to the construction of the sewage pumping main, a field instrumentation point was selected along the pipeline. The monitoring of the instrumentation point during pressure surges will enable the predicted pressures to be compared with actual field-measured pressures. At the time of writing, the pumping main is still under construction and therefore no field readings are as yet available.

CONCLUSION

The results of the surge analysis carried out show that the proposed pumping main can be adequately protected against excessive adverse pressure surges through the installation of three airvalves at locations not at the high points along the main, but upstream of them. Consideration of air release of varying amounts has resulted in less severe pressure peaks as compared to the pressures from assuming no air release.

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NOTATIONS

a	wave speed
D	pipe diameter
E	modulus of elasticity
e	thickness of pipe
f	Darcy-Weisbach friction factor
H _a	absolute pressure head
K _l	bulk modulus of elasticity of liquid
R	gas constant
T	temperature
t	time
V	instantaneous velocity of liquid

x distance along pipe
 α angle of inclination of pipe

TECHNICAL SESSION 12C
SPECIAL PIPELAYING TECHNIQUES

**The Application of the Automatic Pipe Driving Method in the Pam Jaya
Distribution System**

**Swamp and Tidal Flats Pipeline Installation for the Labuan Water Supply
Privatisation Project**

THE APPLICATION OF THE AUTOMATIC PIPE DRIVING METHOD IN THE PAM JAYA DISTRIBUTION SYSTEM

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ABSTRACT : In conjunction with the increasing development utilization of the advanced technology becoming important solution for the fast improvement of the capability to overcome the physical problems.

The said method is no longer new for the established country but for the developing country such as Indonesia with the Jakarta as it's capital city, the utilization of technology will be very advantageous. The Jakarta Municipality is improving all sectors of infrastructures such as the widening of the roads, construction of highway, fly over bridges etc. On the other hand the rehabilitation as well as construction of new utilities such as telecommunication, gas, electricity and water mains network are facing many problems.

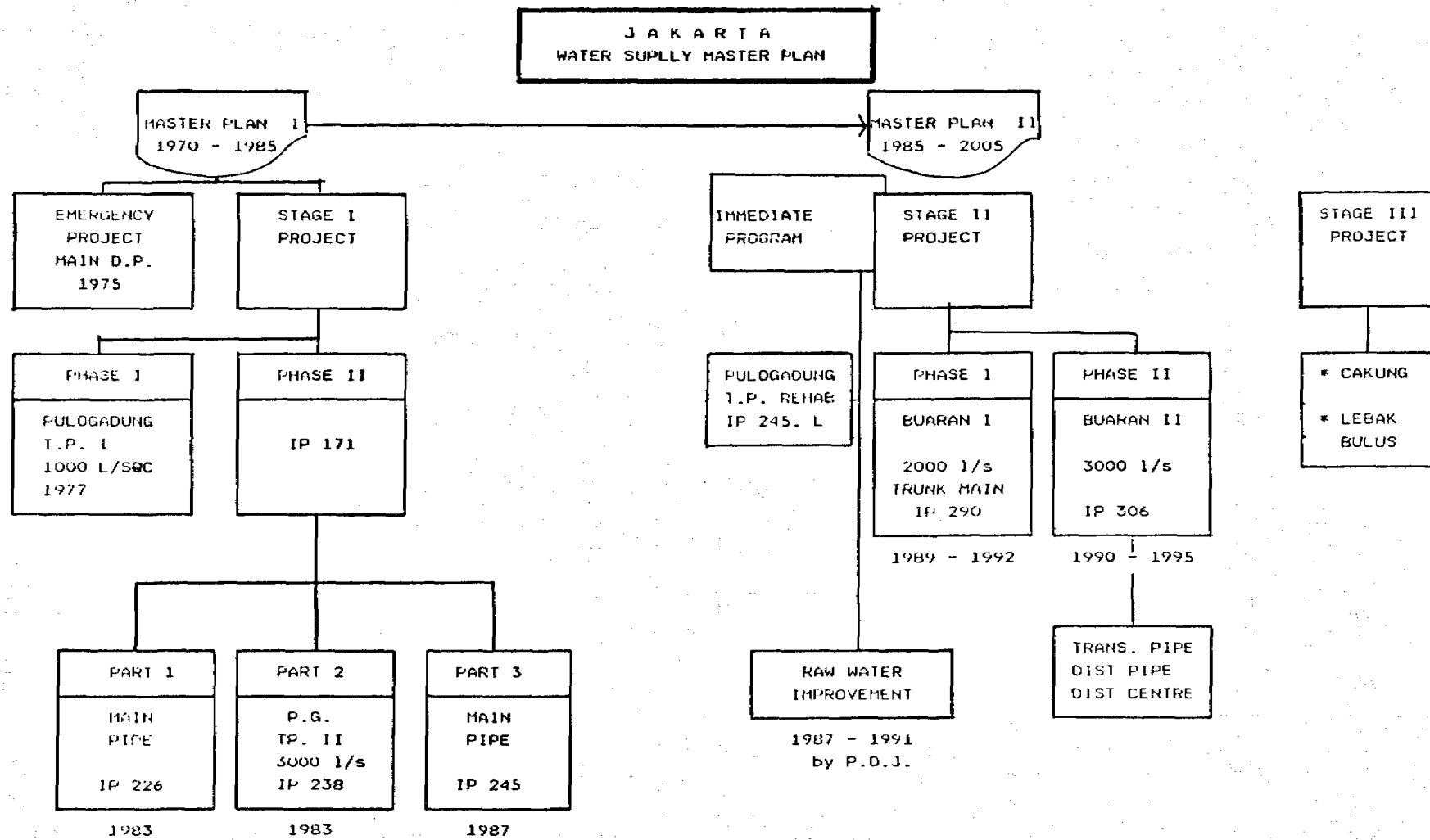
In order to minimize the hindrance to the city transportation and to reduce the physical problems the Government adopted the feasible method for the installation of distribution trunk main by means of Automatic or Conventional Pipe Driving.

In Jakarta the automatically driven Pipeline of about 4.5 km has been completed, of which a large diameter of pipes has been driven underground along side the roads and crossing the rivers (canal) railway including the highway crossing.

This method was introduced by KUBOTA JAPAN as the manufacture of the equipment and implemented by the local contractor/consultant as joint operation under IP-245 Project. The technical information regarding this method is presented in this paper.

1. BACKGROUND

The city of Jakarta is one of the largest and most rapidly developing city is large, the presently served population accounts for only about 50% of the total population. The water supply system in 1988 barely covers 44% of the Metropolitan area. Also, the future population growth is anticipated to be very rapid and to exceed 10 million by 1996.



The water supply Master Plan of the city of Jakarta completed in 1985 by the Japan International Cooperation Agency (JICA) proposed a long term development program for 20 years up to the years 2005 to be implemented in stages.

The first Stage of the development program with financial assistance from Overseas Economic Cooperation Fund (OECF) of Japan already underway at the time of the Master Plan preparation. Each of the subsequent Second and Third stages was recommended to be implemented in two phases to avoid heavy investment and to consider the overall construction period of the project.

PDAM DKI Jaya is the agency responsible to operate and maintain the Jakarta Water Supply System which was initiated from the early 1920' utilizing spring water from Ciburial, Bogor at the foot of Mount Salak. Spring water was originally transmitted by twin pipeline from Ciburial for about 60 km to Jakarta and is still functioning. The system has undergone expansion several times since the 1950's including construction of the Pejompongan Treatment Plants I and II, the Pulogadung Treatment Plant, several mini-plants and many distribution pipelines. The expansion work is still continuing at present to cope with the rapid growth and increasing demand of the city.

The road system in Jakarta is quite complex, while the construction of the new ring road, toll road, fly over, as well as the improvement of artery roads are growing faster than the development of water supply networks. Especially for the installation of larger diameter of pipeline, the open excavation on the road will be very inconvenience. This conventional open trench system therefore shall be executed night time when the traffic load usually low, i.e. for road crossing work. Regarding this matter and as also the increasing number of Vehicles, the traffic conditions becoming very busy and hence the automatic pipe driving had been tried to be implemented.

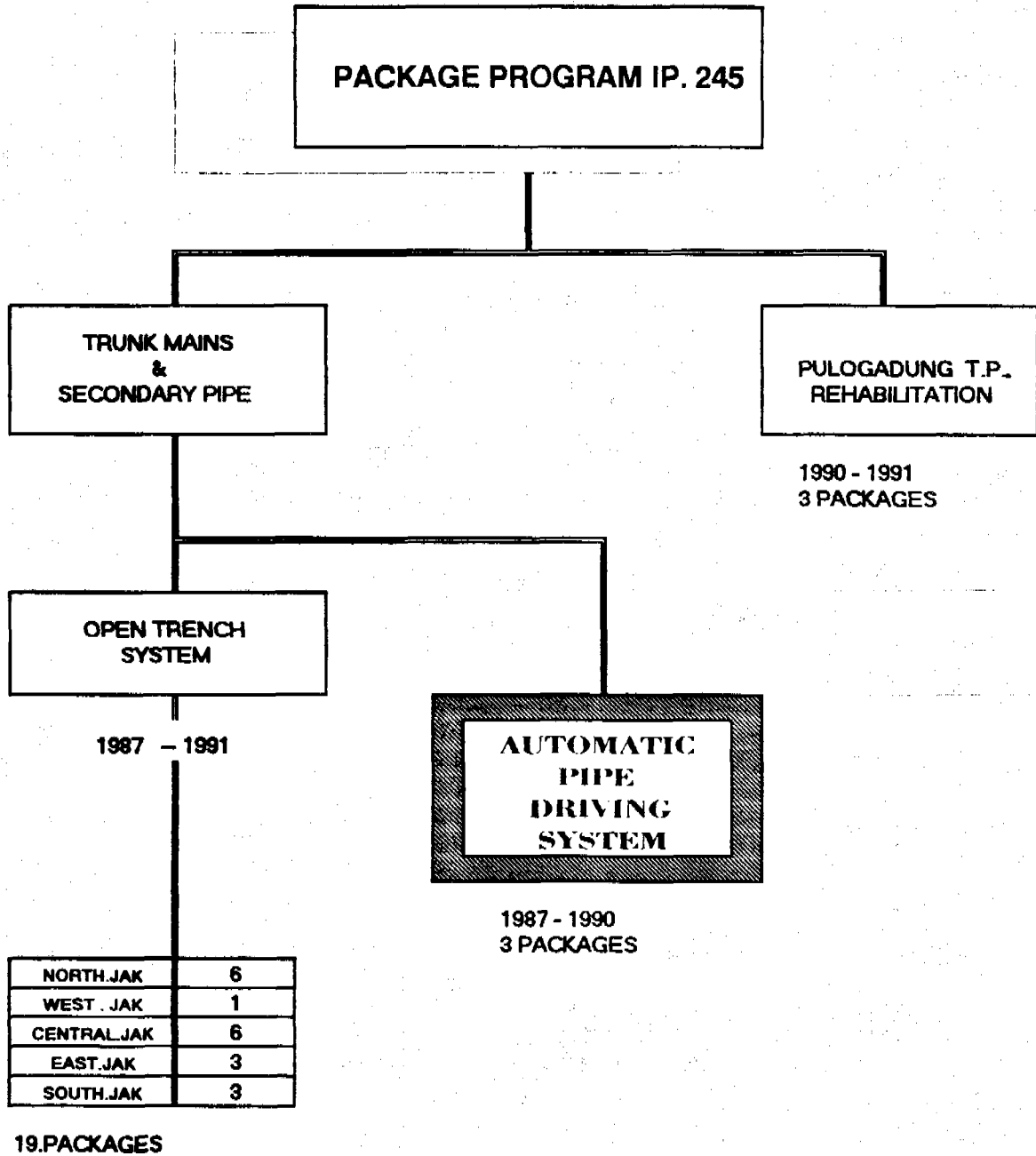
2. PACKAGE PROGRAM IP-245

IP-245 Program consisted the Trunk Mains and secondary pipe, which scheduled in part 3 of phase II, Stage I is now has been completed. Part of this pipeline installation is using Automatic Pipe Driving Methods.

The main consideration are as the followings:

- 2.1. The installation of the pipeline by this method will be advantageous in the City of Jakarta where traffic are rapidly increasing, so that the traffic disturbance can be minimized. Therefore, the open trench method is not always feasible and in this case the pipe driving method shall be used to prevent the pipe laying work from impeding the traffic flow on the road.
- 2.2. The pipes 600 mm and 800 mm in diameter will be installed at intervals of 100 m or more in length. It is difficult to use the conventional manual pipe driving works considering the safety of the workmen inside pipes of these sizes and lengths.
- 2.3. Long distances from driving pits to arriving pits will be required to minimize the disturbance to traffic loads. The ductile iron pipe with high axial strength has been selected to meet the difficult conditions.

GENERAL SCHEME



- 2.4. The purpose of this method is not only to minimize traffic disturbance but also to obtain a reliable performance and safety of the works.
- 2.5. The pipes shall be installed by the driving method at sufficient depths to achieve clearance from existing underground utilities and structures. The exact alignment of the installed pipes is important for the future maintenance of the pipelines. Consequently, the Automatic pipe Driving method shall be selected to achieve a high construction quality in works and maintain accurate control of the driving operation with the continuous output of performance data.

3. PIPE JACKING EQUIPMENT

- 3.1. Equipment for jacking method manufactured by KUBOTA is called DUCMOLE and tried for the first time in Japan in 1983. In Jakarta this method has been applied in 1989. This equipment is of TD type, and capable for Jack Ductile Iron Pipe of diameter 400 mm - 800 mm.

- Soil test standard < 50 (N Value)
- Maximum Jacking Distance + 100 m.

(but we have copied opisated the longer distance 144 M)

- Accuracy + 50 mm.
- Equiped with lasser transit and laser direction detector.

- 3.2. Computer Supporting Unit

A Computer Supporting Unit is used to control the accuracy of this jacking equipment, which could detected:

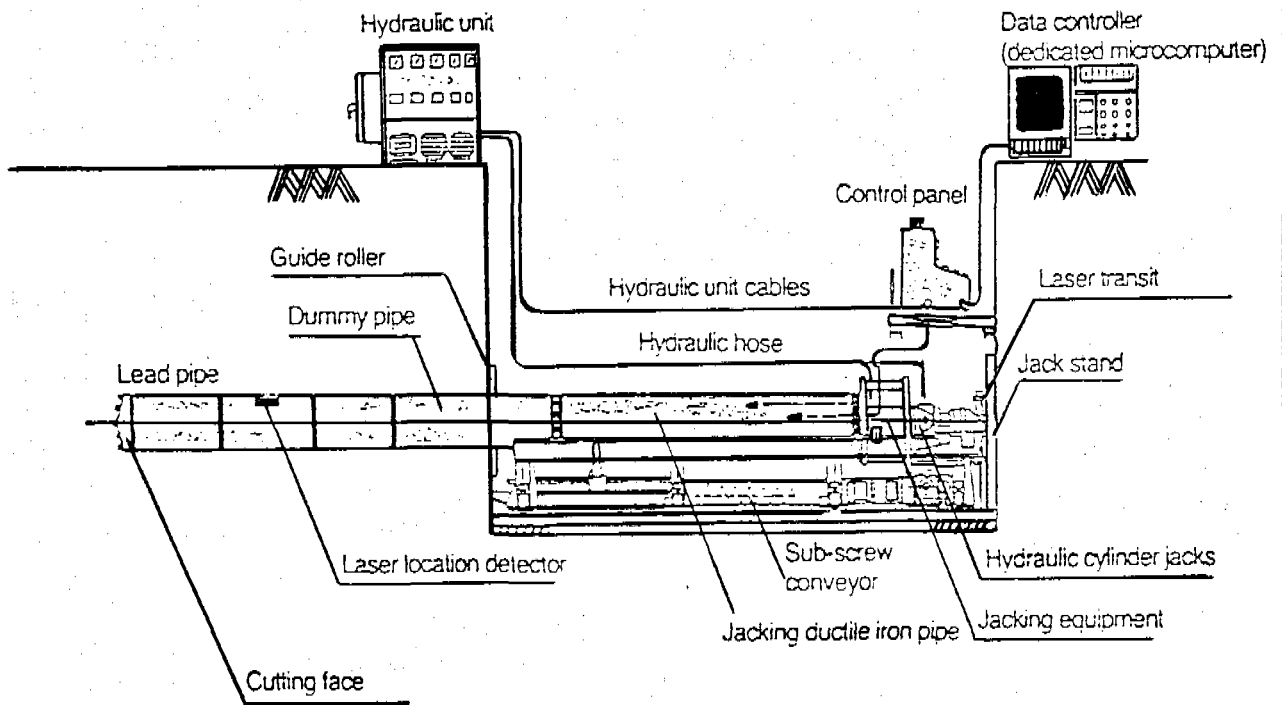
- The position of lead pipe
- The direction of lead pipe
- The jacking force
- The forque of cutting face.

Those data could be directly obtained through the computer print out.

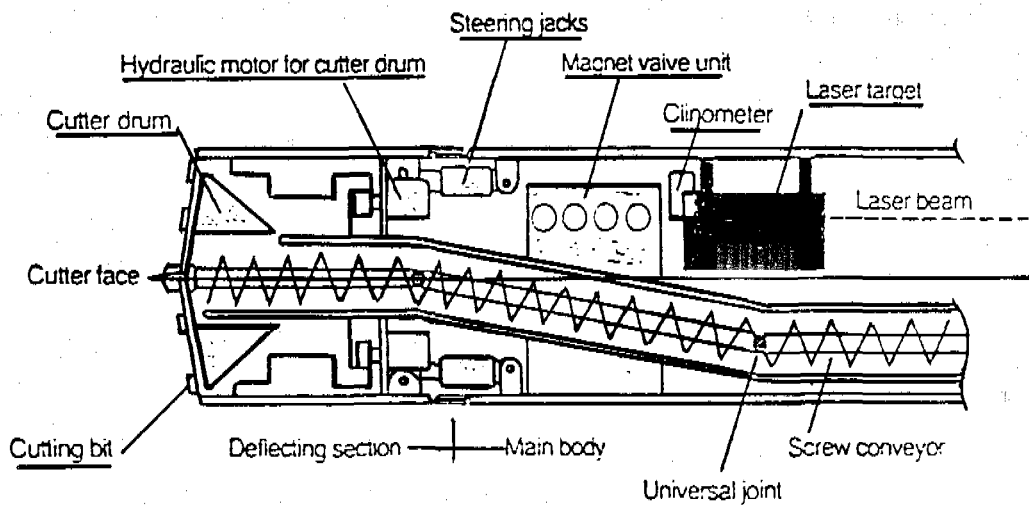
- 3.3. The hydraulic system is used to minimize the vibration as well as noise.
- 3.4. The accuracy of the pipe direction can be controlled by the adjustment of the jack at the lead pipe , i.e. to avoid the deviation.
- 3.5. This equipment is operable by manual as well as automatic.

Jacking Machine

Jacking Machine Configuration



Structure of the Lead Pipe



Specifications

Applicable Pipe	• TD-type jacking ductile iron pipe, 400mm to 800mm diameter (standard laying length: 4m).
Applicable Soil	• Wide range of soil conditions with N-value less than 50; maximum diameter of conglomerate to be disposed by the screw conveyor is approx. 40mm.
Jacking Distance	• Greater than 100 meters, as a result of the high resistance of ductile iron pipe to jacking force.
Excavation	• Excavation unit is driven independently of the soil removal unit to increase excavation efficiency. • Ultra-hard alloy cutting bits can break up conglomerates. • Soil amount taken in can be adjusted by changing the opening ratio of the cutting face plate to reflect soil conditions. • Hydraulic drive cutter can rotate clockwise or counterclockwise; consequently, it is easy to compensate for rolling in the lead pipe as well as to cut through conglomerate material with minimal noise and vibration.
Soil Removal	• Excavated soil is removed by the screw conveyor. • Amount of soil removal can be adjusted by changing the rotational speed of the screw conveyor. • Hydraulic drive motor can reverse the rotation of the cutter face to prevent blockage of cutter opening sections. • The cutter drum is equipped with water adding equipment to facilitate excavation of sticky soil. • The screw conveyor casing is fitted with rubber casters on the bottom to avoid damaging the internal surface of the pipe.
Location Check	• Position of the lead pipe is automatically detected by laser transit and target. • Lead pipes are automatically measured in three directions: vertical, horizontal, and circumferential.
Operation	• Operation can be conducted automatically or under manual control. • Direction adjustment, rolling correction and jacking speed can be controlled automatically. • Execution data can be analyzed by the dedicated microcomputer. • Warning display for values outside allowable range as well as overload protection mechanisms are provided.
Direction Adjustment	• Adjustments in the direction of the lead pipe are accomplished by coordinated action of the four steering jacks. • Required stroke for each steering jack is determined automatically.
Jacking Device	• Max. jacking force: 400 tons/t • Max. jacking speed: 16.2 cm/min • Jacking stroke: 400 mm • Under automatic control, jacking machinery runs by itself. • Required electric capacity: 125 kVA (including that for lead pipe)

4. PREPARATION FOR IMPLEMENTATION

Considering the pipe line alignment the pipe laying work required a formal permit from BKJS (Coordination Body for Underground Utility Network) DKI. since is should be coordinated by several government authorities such as DTK (Tower Panning Services), DPU (Public Works), DLLAJR (Jakarta Traffic and Transportation Services), Police and also some other underground utilities such as PLN, Telkom and Gas.

Pits shall be constructed to place the jacking equipment i.e. driving pit, with different dimensions. The distance between pits was decided based on the site survey or depend on the width of the road crossed by the pipeline.

The site Investigation to be undertaken before implementation are :

- 4.1. The underground survey which is necessary to identify the soil layer condition and the position of the existing infrastucture such as drainages pipes, cables or gas pipes.
 - The average depth of the jacking pipe is about 5m - 6 m.
 - The special pipe DIP stored in the stockyard and transported to worksite just before installation .
- 4.2. Soil Investigation, of which the soil sampling is necessary to be analysed in the laboratory for it's properties such as Natural Water Content, Liquid Limit, plasticity etc. The soil test results of some selected locations are given in the following page.

5. THE IMPLEMENTATION

- The work is executed as soon as the necessary permit issued by the authorities agencies obsanies.
- The Pipe Jacking mentioned above has been implemented in Jakarta and the followings is high light of the project location

- | | | |
|------|-----------------------|---|
| 5.1. | Tanah Tinggi Barat | 1.14 km x ϕ 600 mm |
| 5.2. | Merdeka Timur-Veteran | 1.2 km x ϕ 800 mm |
| 5.3. | Juanda - Harmoni | 1.34 km x ϕ 600 mm |
| 5.4. | Special case : | <ul style="list-style-type: none">- Railway Crossing- River Crossing- Highway CrossingGatot Soebroto 2 x crossing(112 m x 120 m) x ϕ 600 mm |

6. CONSTRUCTION CONTROL

As an example, the pipeline installation by jacking method in Jalan Gatot Soebroto Jakarta is presented in the followings.

6.1. Shop Drawings

The pipe driving work has been executed based on the approved shop drawings.

6.2. The period of the construction

6.2.1. Pit

Start 2nd July 1988
Finish 15th July 1988

6.2.2. Driving

Installation of Machine Start 6th July 1988
 Finish 20th July 1988

Driving Start 21st July 1988
 Finish 29th July 1988

Taking the support of machine and so on a part
 Start 30th July 1988
 Finish 1st August 1988

6.3. Time schedule a day

From 08.00 am - 17.00 pm
From 20.00 pm - 05.00 am

6.4. The Data of Driving

6.4.1 The condition of soil

Clayey Silt

N-Value more than 45

Water-level GL - 1.5 m -3.5 m

Note:

1. The soil condition is very stiff, but the cohesion is very low.

2. The natural water content was not so much different from liquid limit.

6.4.2. The speed of the driving per day

Driving only 14 m per day

Driving Nett (including preparation and so on) 7 m per day

6.4.3. Thrust force (calculated) by computer

6.4.4. Position (X-Y) (calculated) by computer

6.4.5. Screw Torque (calculated) by computer

6.5. Consideration

6.5.1. Accuracy of the driving and speed the actual center line of pipe by the execution of driving was almost same with the plan. We could execute the driving at the speed of 14 m/day without the damage to road structure.

6.5.2. The soil was very stiff, and suitable for Jacking Machine. But we could execute driving completely, because the natural water content of the soil was not so different from liquid limit. If the natural water content was different from liquid limit, our execution would be difficult.

6.5.3. As for pit construction, the Korean Embassy which is very close to work site strongly demanded KOJ (the contractor) not to use the Vibro Machine, because it was interfere their building. We could not hit the sheet piles into the earth and so the sheet piles were only attached the wall of soil in the pits. We worried about the strength of soil against the driving force. But, the actual thrust force was not high. As a result, we were fortunate to be able to execute this driving work at these bad conditions until its completion.

7. CALCULATION EXAMPLE

In some cases the excavation for the construction of driving as well as arriving pits is very deep and therefore the stability and the structure strength is important to ensure the safety. For this purpose the steel sheet piles has been used as the retaining wall of the pit sonstruction. The following are the calculation example of the sheet pile structures for Driving & Arriving pits.

We shall use sheet piles for pit construction. There are several types of sheet pile structures. In this project Multi-Strut Type has been used as a temporary retaining wall for excavation. Multi-Strut Type is generally designed based on the following procedure:

- (1) DETERMINATION OF DESIGN CONDITIONS
- (2) ASSUMPTION OF WALE ATTACHMENT LEVEL
- (3) INVESTIGATION OF HEAVING
- (4) CALCULATION OF EXTERNAL FORCES

- (5) CALCULATION OF REQUIRED PENETRATION DEPTH
- (6) COMPUTATION OF MAXIMUM BENDING MOMENT OF SHEET PILE
- (7) DETERMINATION OF SECTION OF MAIN SHEET PILE
- (8) DESIGN OF WALES AND STRUTS

7.1. DETERMINATION OF DESIGN CONDITIONS

Soil conditions at each location are based on the Geotechnical Information attached in the Contract Document Volume III and the check boring result executed from 24 June through 22 July by KOJ. The angle of internal friction in sandy soil and the cohesion in cohesive soil are derived from N-value of the standard penetration test.

$$\theta = \sqrt{15N + 15}$$

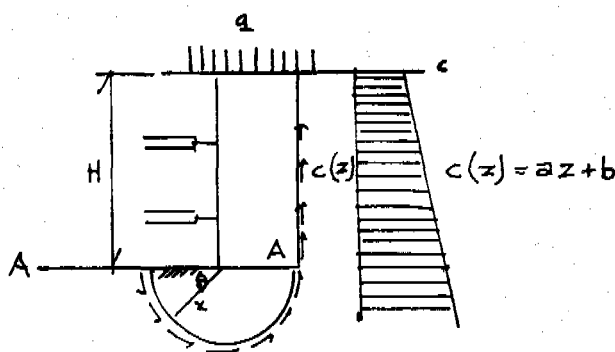
$$C = qu/2 \quad (qu = N/8)$$

7.2. ASSUMPTION OF WALE ATTACHMENT LEVEL

The attachment level of wales should be determined, considering the pit construction process and automatic driving work.

7.3. INVESTIGATION OF HEAVING

For Construction in soft clay, heave at the bottom of the excavation may occur in settlement of the surrounding ground surface. The conventional method of analysis for investigation heave was developed by Nihon Douro Kyokai and is illustrated in the following drawing.



In this case, the vertical of soil along the sheeting is assumed to exert a pressure on the horizontal plane A-A. When the pressure exerted by this soil column exceeds the bearing capacity of the soil

beneath the sheeting a bearing type failure will occur, resulting in heave of the bottom of the excavation and settlement of the surrounding ground surface. Based on this failure model the depth of excavation at which heave will occur and a factor of safety are as follows :

$$C(Z) = a \times Z + b$$

$$X_o = \sqrt{\frac{a h^2 + 2 b h}{4 a}}$$

$$F_s = \frac{2}{\delta h + q} \left((a h + b) \pi + 2 \sqrt{(a h)^2 + 2 a b h} \right)$$

Z	=	Depth from the surface	m
C	=	Cohesion of Cohesive Soil	t/m ²
a	=	Constant number	
b	=	Constant number	
X _o	=	Depth of surface sliding	m
h	=	Height of excavation	
F _s	=	Factor of safety	
δ	=	Unit weight of soil	t/m ³
q	=	Surcharge	

A Factor of safety of 1.2 applied to the soil cohesive shear strength is normally recommended

7.4. CALCULATION OF EXTERNAL FORCES

Active and passive earth pressure can be obtained from Rankine-Resal equation.

P _a	=	(q + δ h) tan ² (45° - θ/2) - 2 C tan (45° - θ/2)	
P _p	=	(q + δ h) tan ² (45° + θ/2) + 2 C tan (45° + θ/2)	
P _w	=	δ w x h w	
P _a	=	active earth pressure	t/m ²
P _p	=	passive earth pressure	t/m ²
P _w	=	residual water pressure	t/m ²
q	=	surcharge	t/m ²
δ	=	unit weight of soil	t/m ³
δ w	=	unit weight of water	t/m ³
h	=	height of soil layer	m
h w	=	height of water	m
θ	=	angle of internal friction of soil.	deg
c	=	cohesion of cohesive soil	t/m ²

7.5. CALCULATION OF REQUIRED PENETRATION DEPTH

The required sheet pile penetration depth is calculated by the Free Earth Support Method, that is, the moment due to passive earth pressure which is equal to the moment due to active earth pressure and residual water pressure about the wale point. The formula is as follows:

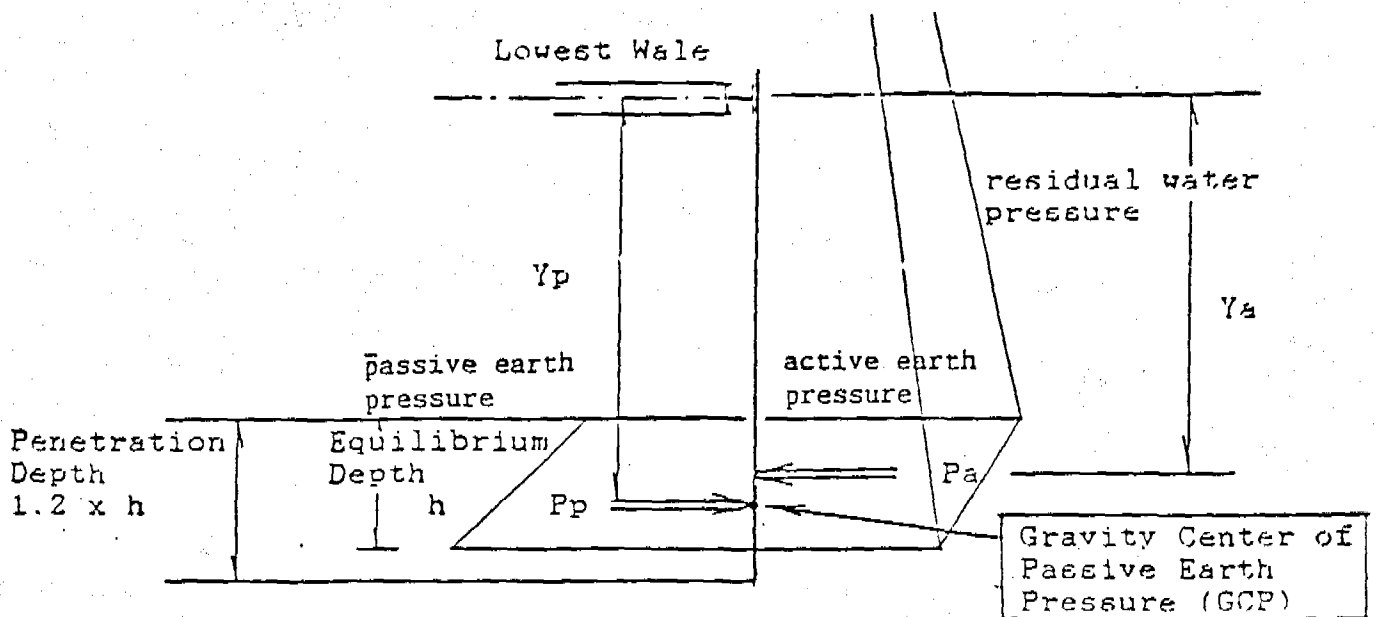
$$M_p = M_a$$

$$M_p = P_p \times Y_p$$

$$M_a = P_a \times Y_a$$

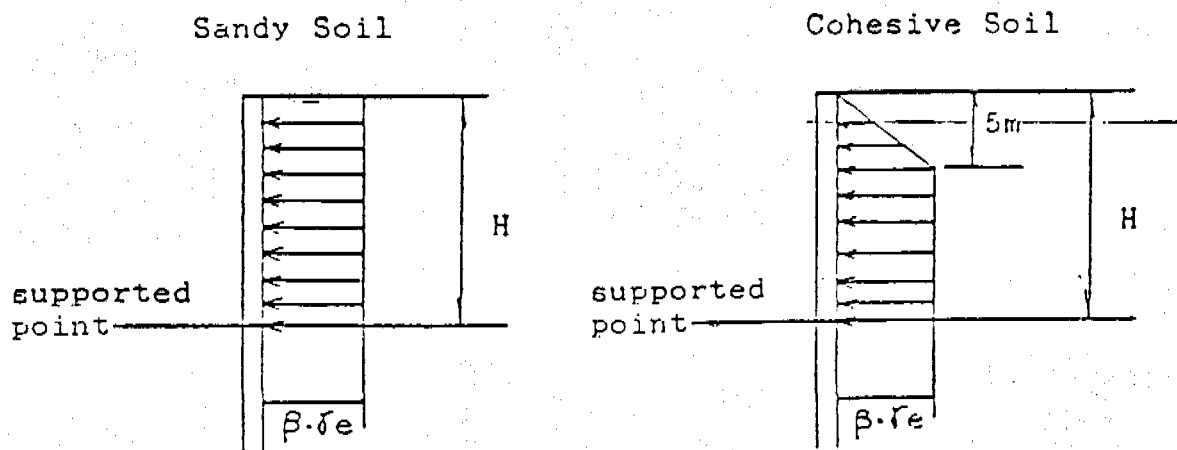
Where:

- M_p = moment due of passive earth pressure about the wale point
- M_a = moment due to active earth pressure and residual water pressure about the wale point.
- P_p = passive earth pressure t/m
- Y_p = arm length between P_p and wale point m
- P_a = active earth and residual water pressure t/m
- Y_a = arm length between P_a and wale point m

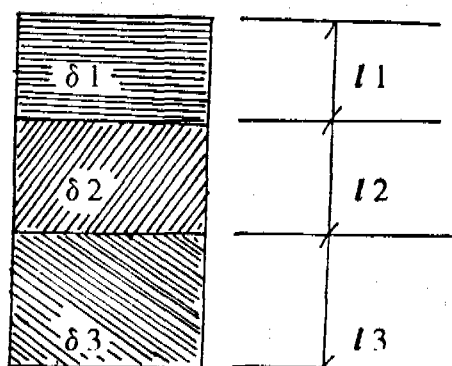


7.6. COMPUTATION OF MAXIMUM BENDING MOMENT OF SHEET PILE

The maximum bending moment is calculated on the assumption that the sheet pile wall is a simple beam supported at the lowest wale point and the gravity center of passive earth pressure (GCP), and external forces act on the beam. The distribution of external forces is different due to the soil condition. A trapezoidal distribution of earth and residual water pressure can be seen in the following drawings.



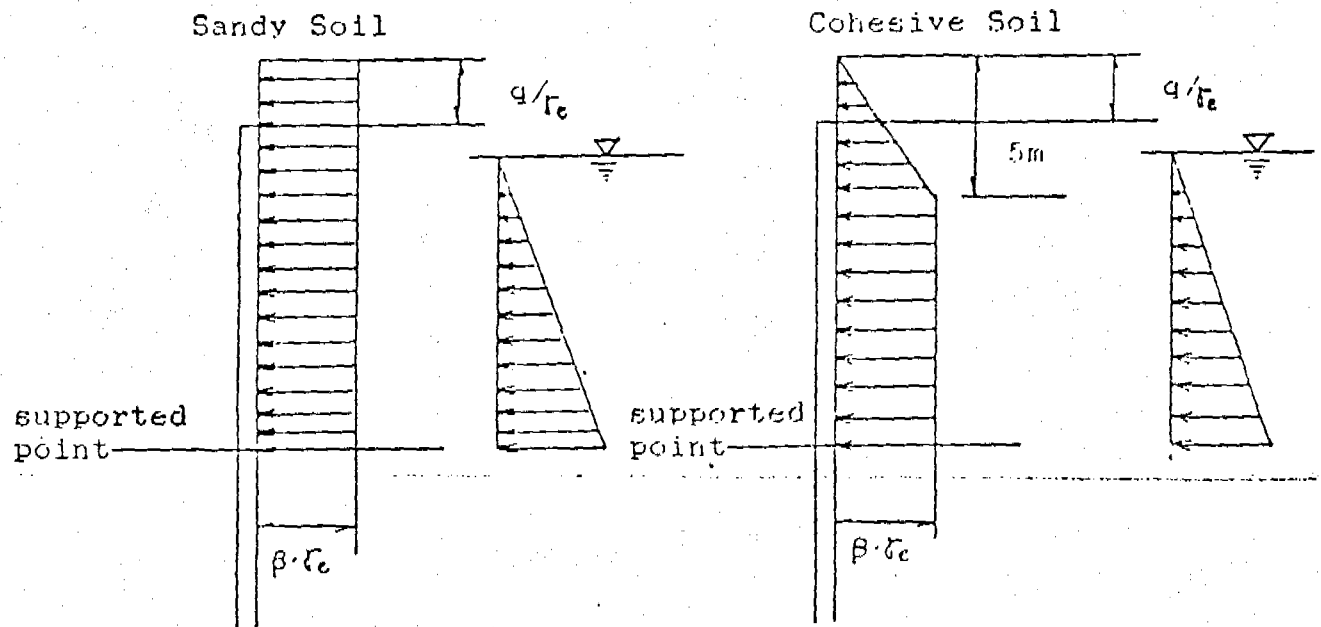
δ_e = average effective unit weight, t/m^3
 β = coefficient due to soil



$$\delta_e = \frac{\delta_1 \cdot l_1 + \delta_2 \cdot l_2 + \delta_3 \cdot l_3}{l_1 + l_2 + l_3}$$

Sandy soil	Cohesive soil	
2	$N > 5$	4
	$N \leq 5$	6

In case of surcharge and underground water.



7.7. DETERMINATION OF SECTION OF MAIN SHEET PILE

The required section modulus of sheet pile can be obtained from the following equation.

$$Z = \frac{M_{\max}}{\tau_a} \times 10^5$$

Where :

Z	=	required section modulus of sheet pile,	cm^3/m
$M_{\max}$	=	maximum bending moment of sheet pile wall	t-m/m
τ_a	=	allowable stress of sheet pile	2700 kg/cm^2

7.8. DESIGN OF WALES AND STRUTS

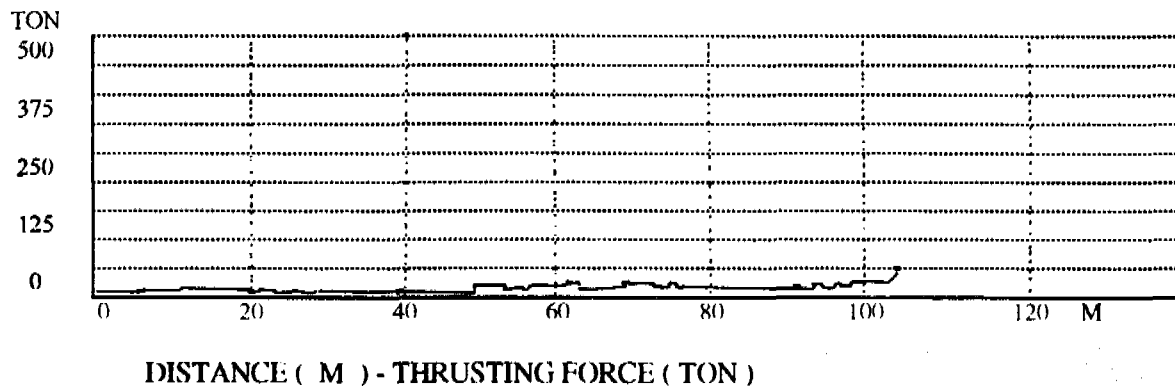
The wales are designed to resist the horizontal reaction from the sheeting. The wales are taken as a continuous beam supported by struts.

In regard to external forces, the earth and residual water pressure can be calculated from the aforementioned drawing. For this calculation lower supported point is considered to be excavated level.

PIPE DRIVING RECORD

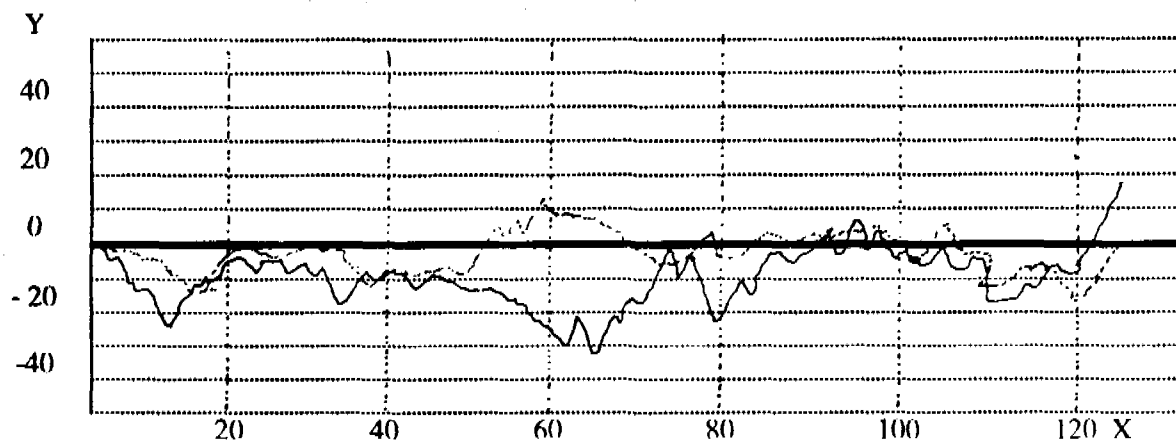
Location of Construction		Merdeka (Veteran I)	PIT NO.6' NO.7A
ITEMS		Ø 800 x 108 m	RAILWAY CROSSING
1. Period of Construction			
1.1. PIT	start	14 November 1988	
	finish	19 Desember 1988	
1.2. Driving	start	12 Desember 1988	
	finish	22 Desember 1988	
1.3. Taking out the equipment	start	23 Desember 1988	
	finish	24 Desember 1988	
2. Working time schedule per day		24 Hours	
3. Datas of driving			
3.1. Conditions of Soil			
☐	What of Soil	Silt and Clay	
☐	N - Value	18	
☐	Water Level	GL - 2.85 m	
☐	Consideration	The Soil condition is Not so easy for driving because of cohesive clay	
3.2. Speed of drivong per day			
#	Driving Only	15 m / a day	
#	Driving Nett	8 m / a day	

4 Thrust Force

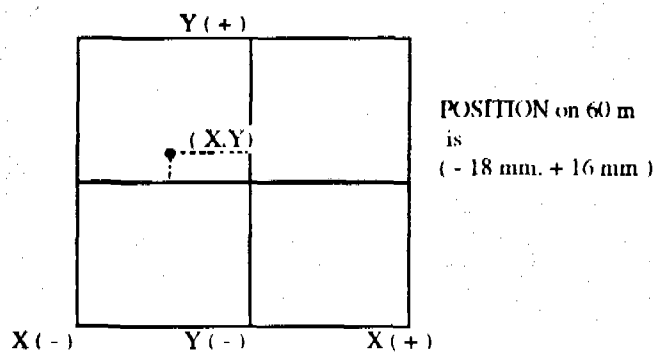


Location of Construction ITEMS	Merdeka (Veteran I) PIT NO.6' NO.7A Ø 800 x 108 m RAILWAY CROSSING
--	--

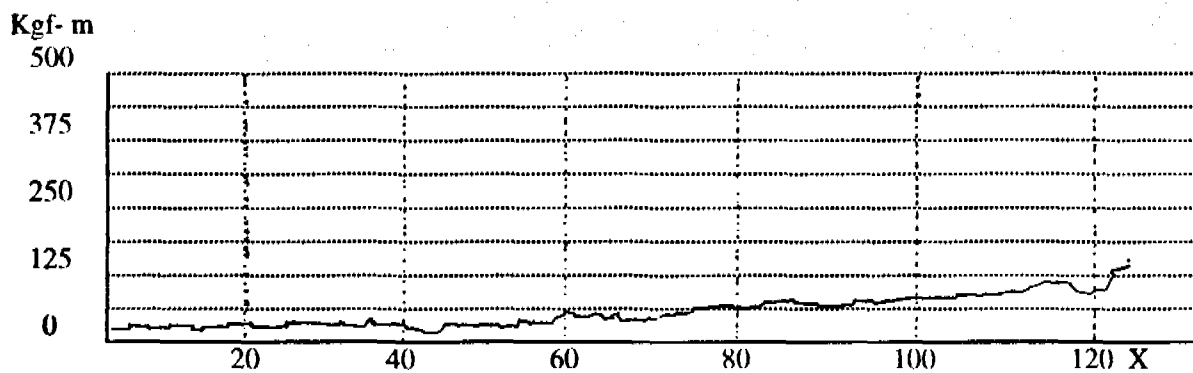
5. Position



Distance (m) - Position (mm) (..... X, Y)



6 Screw Torque

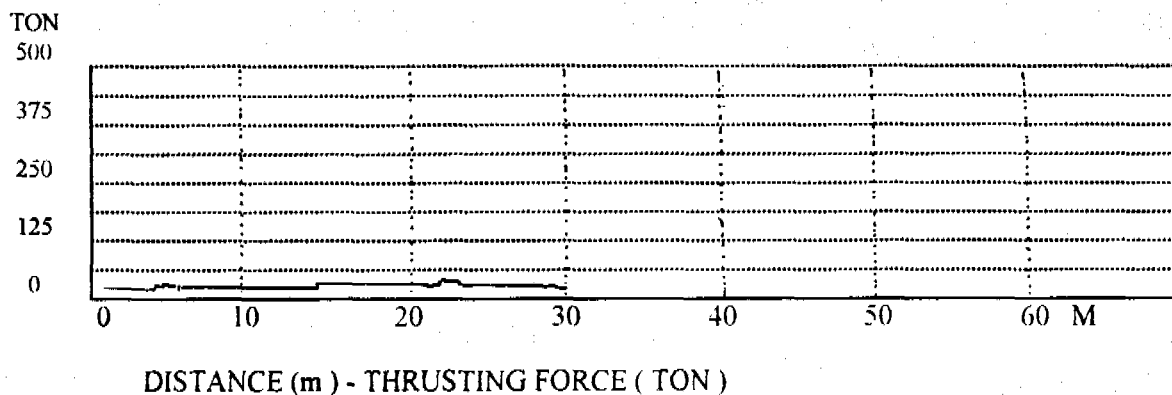


DISTANCE (m) - Screw Torque (Kgf - m)

PIT NO.2' NO.3

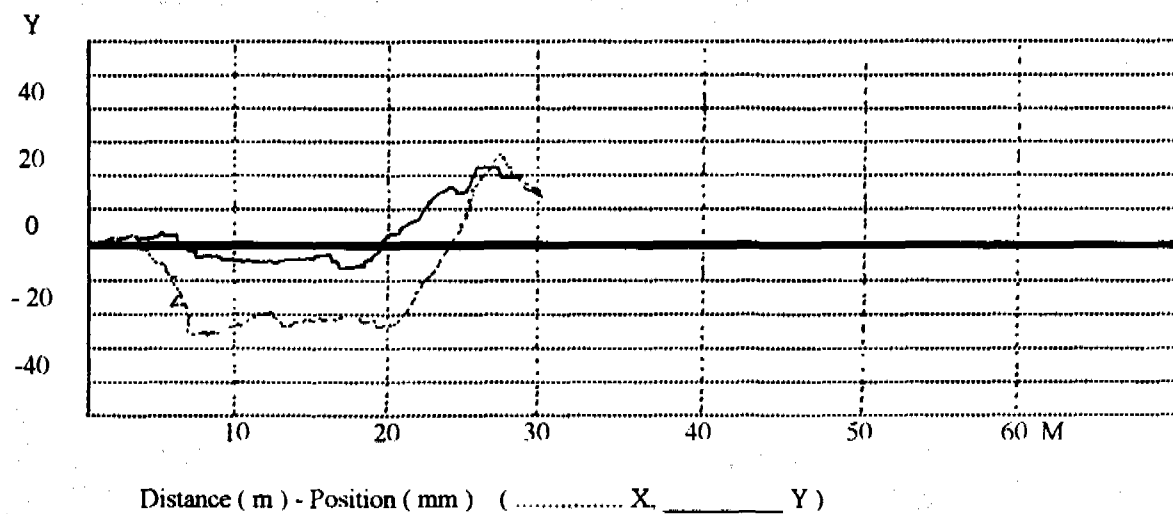
Location of Construction		Juanda Ø 600 x 32 m	PIT NO.2' NO.3 CANAL CROSSING
ITEMS			
1. Period of Construction			
1.1. PIT	start finish	14 Oktober 1988 4 November 1988	
1.2. Driving	start finish	28 November 1988 3 Desember 1988	
1.3. Taking out the equipment	start finish	4 Desember 1988 6 Desember 1988	
2. Working time schedule per day		24 Hours	
3. Datas of driving			
3.1. Conditions of Soil			
☐	What of Soil	Silty Clay	
☐	N - Value	4	
☐	Water Level	GL - 2.68 m	
☐	Consideration	The Soil condition is No Problem for Driving Work	
3.2. Speed of drivong per day			
#	Driving Only	8 m / a day	
#	Driving Nett	3.5 m / a day	

4 Thrust Force

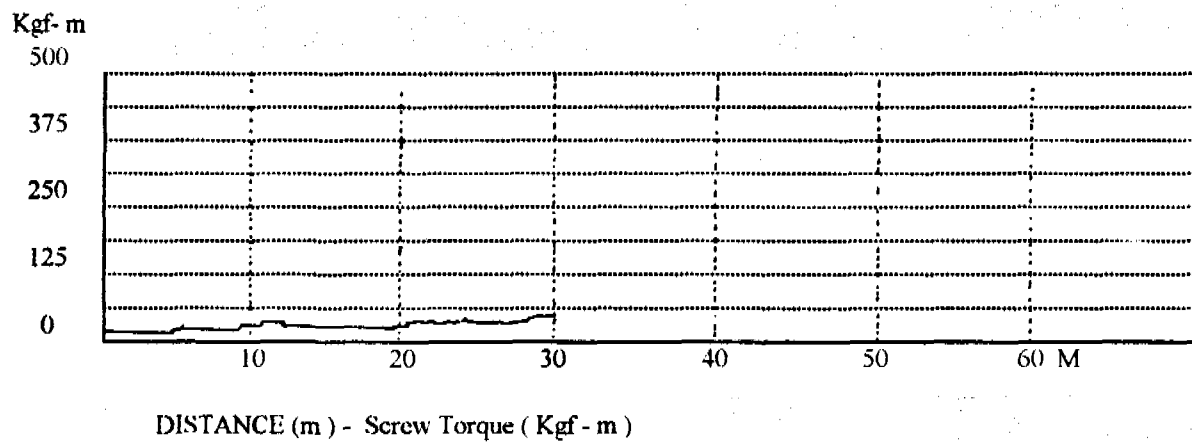


Location of Construction ITEMS	Merdeka (Veteran I) PIT NO.2' NO.3 Ø 600 x 32 m CANAL CROSSING
--	--

5. Position



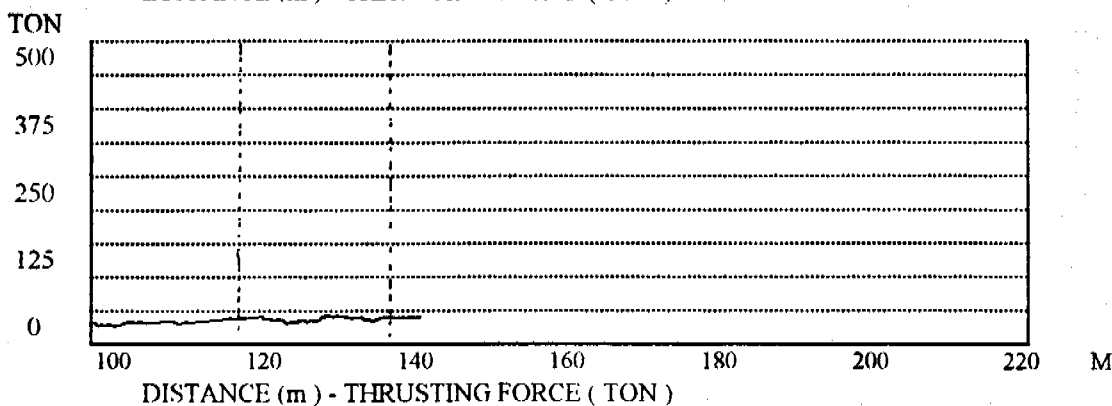
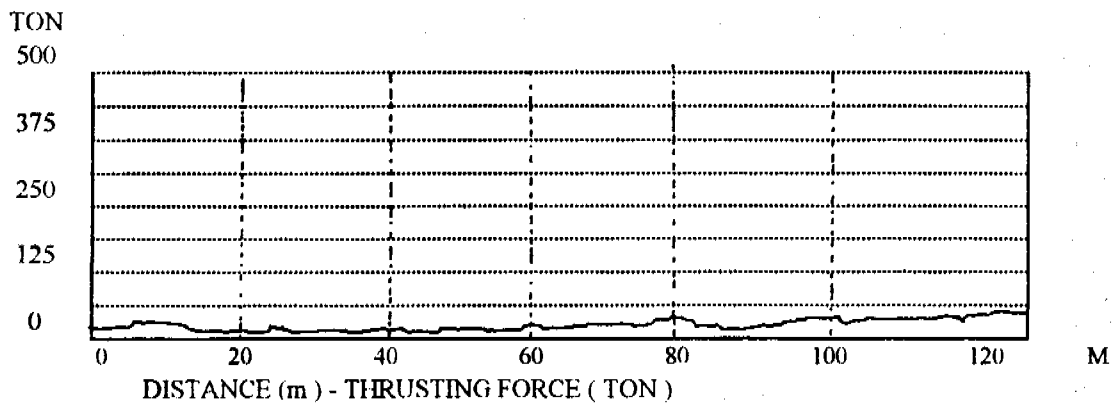
6 Screw Torque



PIPE DRIVING RECORD

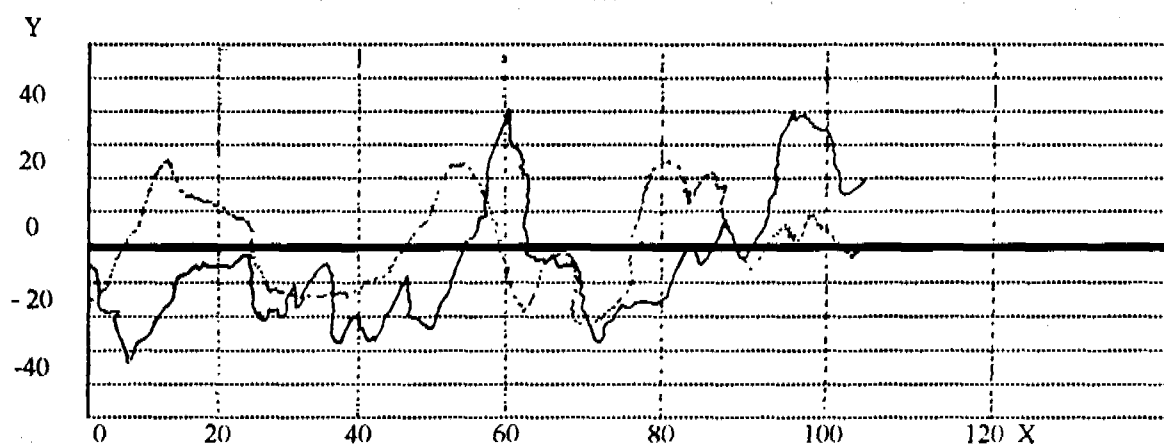
PIT NO.15' NO.13

Location of Construction	Juanda Ø 800 x 144 m	PIT NO.15' NO.13 (THE LONGEST SPAN)
ITEMS		
1. Period of Construction		
1.1. PIT	start	4 September 1988
	finish	27 September 1988
1.2. Driving	start	27 September 1988
	finish	18 October 1988
2. Working time schedule per day	24 Hours	
3. Datas of driving		
3.1. Conditions of Soil		
☐ What of Soil	Clayey Silt	
☐ N - Value	7	
☐ Water Level	GL - 3.84 m	
☐ Consideration	It is not so easy for us to do drivong work because of high cohesive clay	
3.2. Speed of drivong per day		
# Driving Only	10 m / @ day	
# Driving Nett	8 m / @ day	

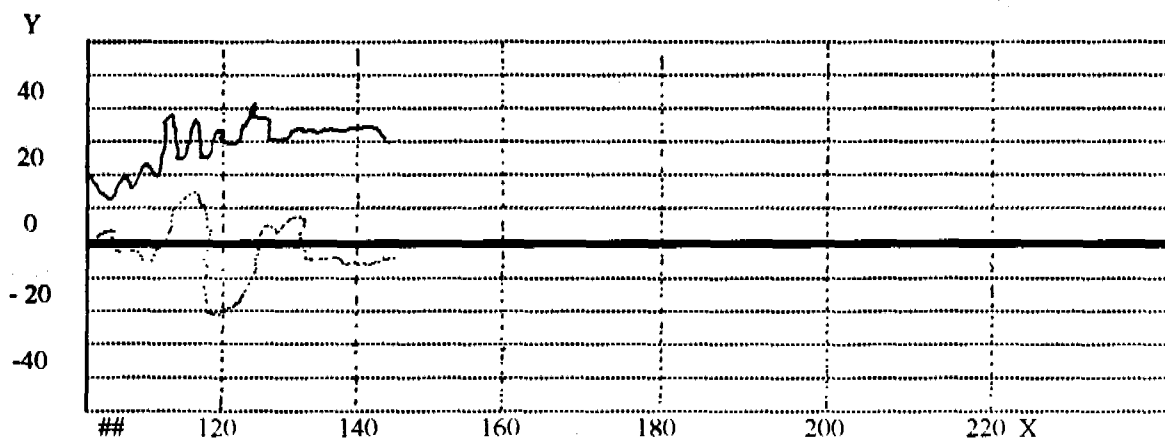


Location of Construction ITEMS	Merdeka (Veteran I) PIT NO.15' NO.13 O 800 x 144 m (THE LONGEST SPAN
--	---

4 Position



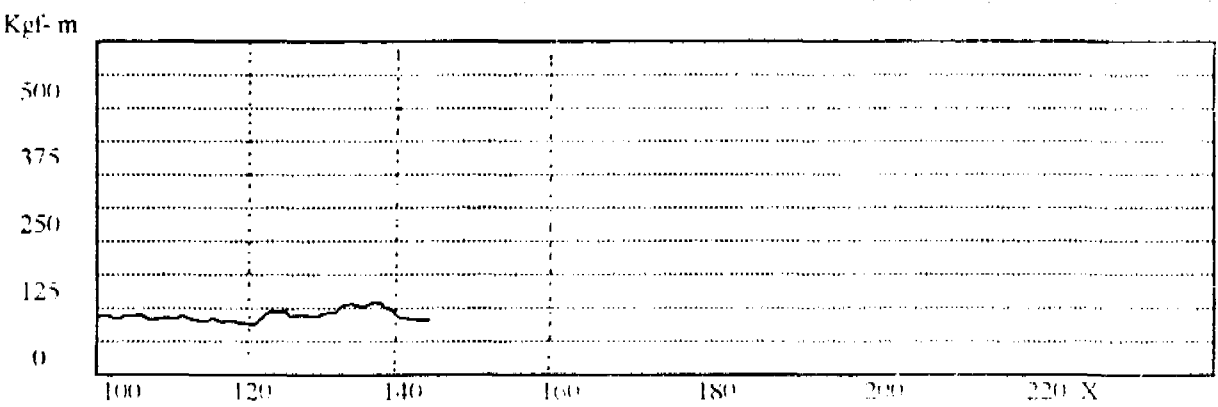
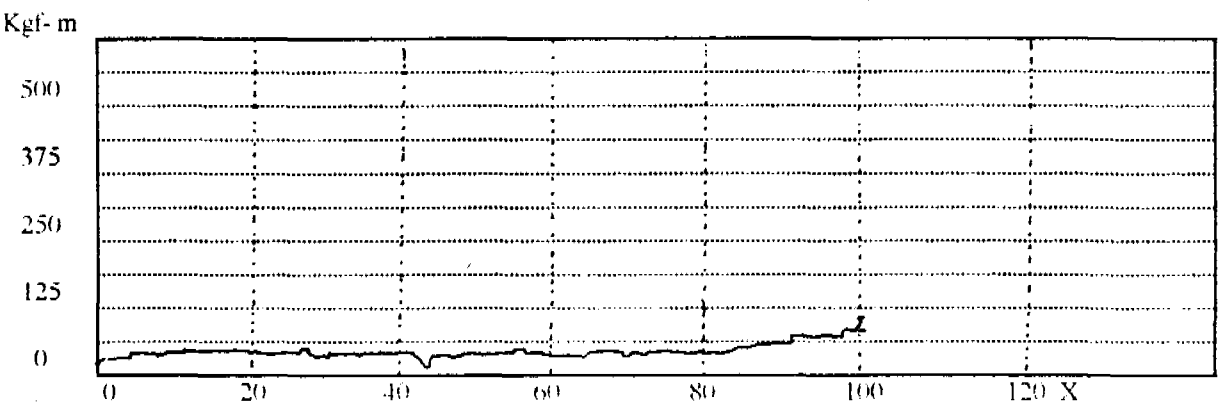
Distance (m) - Position (mm) (..... X, Y)



Distance (m) - Position (mm) (..... X, Y)

Location of Construction ITEMS	Merdeka (Veteran I) PIT NO.15' NO.13 O 800 x 144 m (THE LONGEST SPAN)
--	---

5 Screw Torque



SWAMP AND TIDAL FLATS PIPELINE INSTALLATION FOR THE LABUAN WATER SUPPLY PRIVATISATION PROJECT

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ABSTRACT

Pipeline installation in swamp and tidal flats must be engineered in combination with a well developed construction plan to ensure successful completion.

It is possible to use the unique aspects of swamp and tidal environments to assist rather than hinder pipeline construction.

This paper presents the concept, engineering and construction of the swamp and tidal flats pipeline installation for the Labuan Water Supply Privatisation Project. The design and construction features of the swamp and tidal pipeline demonstrates how it is possible to turn seemingly adverse site conditions to the benefit of pipeline installation works.

INTRODUCTION

GENERAL

On 10 July 1987, the Labuan Water Supply Privatisation contract was signed between the Government of Malaysia and Labuan Water Supply Sdn Bhd. The Contract called for the construction of a Water Treatment Plant on mainland Sabah adjacent to the Padas River and a 610mm diameter pipeline across 47.3 kilometres of land, swamp, tidal, and subsea environs to the Island of Labuan. Two 1 million gallon capacity concrete storage reservoirs provided terminal storage on Labuan Island. The key plan is shown in Figure 1.

IPCO Constructors Sdn Bhd in a joint venture with Pernas Construction Sdn Bhd entered into a turnkey contract with Labuan Water Supply Sdn Bhd to design and construct the Water Supply Facility including the Raw Water Intake, Water Treatment Plant, Treated Water Pipeline and Storage Tanks in Labuan.

ENGINEERING CONCEPT

The original proposed for supplying water from Sabah to solve Labuan Islands perennial water shortage called for a dam and a storage reservoir and conventional water pipeline following a land route.

The IPCO solution was to use oil and gas technology to design and install a high pressure pipeline taking water directly from the Treatment Works adjacent the Padas River to storage on Labuan Island.

IPCO selected a route through mainly Government owned swamp utilising swamp pipeline installation techniques. This resulted in significant savings in the pipeline route length the eliminated expensive land acquisition.

SCOPE OF WORKS

The scope of works for the project comprised the following:

- Water Treatment Facility at mainland Sabah including water intake works for 8.3 MGD (38 million litres per day).
- 610mm diameter transmission cement mortar lined steel pipeline from the Water Treatment Facility to the Storage Tanks in Labuan Island.

The Transmission Pipeline comprised the following sections:

- Mainland Sabah Land Pipeline	10.6 km
- Mainland Sabah Swamp Pipeline	10.4 km
- Tidal Flats Pipeline at Sabah Coast	3.7 km
- Subsea Pipeline between Sabah and Labuan	22.0 km
- Labuan Land Pipeline	0.6 km
Total	47.3 km

- Storage Tanks at Labuan Island for the storage of 2 million gallons of treated water complete with necessary pipework to JKR pumping station.

BACKGROUND FOR THE SELECTION OF PIPELINE ROUTE

The eventual selection of the pipeline route through the variable terrain was selected for the following reasons:

- A shorter route through the swamp reduced the pumping head resulting in reductions in both energy costs and pipe wall thickness.
- The route reduced the cost and time associated with land acquisition as most of the swampy land is owned by the State Government.
- IPCO rapid installation techniques in pipeline construction were ideal for construction for the swamp route.
- The position of the intake was set by the maximum upstream limit of saline intrusion in the Padas River as determined by salinity studies. These salinity studies undertaken demonstrated the suitability for the raw water intake location and consequent pipeline route through the swamp.

ENGINEERING OF PIPELINE

PRELIMINARY ENGINEERING

Preliminary engineering studies showed that the concept of utilising higher pipeline pressures than normally used for water pipelines would prove practical and economic.

A number of alternative routes and pipeline materials were considered. Also, intensive studies were made to determine the optimum location of the raw water intake and hence pipeline route and length.

DETAILED ENGINEERING

The detailed engineering comprised all facets of pipeline design, and of greater importance, construction engineering. The construction engineering interfacing with the design was maintained at all times during this process to ensure that considerations with respect to installation stresses and other installation criteria were fully realised.

The Swamp Pipeline comprised the section of the pipeline from a point adjacent to Bangkalalak (Kilometre Station 10.6) to the mouth of the Sungei Burong at the Sabah Shoreline (Kilometre Station 21.0) as shown in Figure 1. The Swamp Pipeline was installed in an open and flooded trench which was not backfilled.

Pipeline Wall thickness and Grade

The pipe nominal wall thickness was based on the internal pressure and minimum wall thickness requirements together with rigorous installation stress analysis.

A 610mm pipeline manufactured in accordance with API Specification 5L Grade X52 was utilised for the Swamp Pipeline.

Although design resulted in a minimum wall thickness of 4.1mm. A wall thickness of 6.35mm was selected to satisfy code requirements.

Pipeline Stability

The pipeline in the swamp must have sufficiently mass to ensure negative buoyancy. Stability was provided by a concrete weight coating giving the pipeline a specific gravity of 1.04 (assuming fresh water of density 1000 kg/m^3). A concrete coating thickness of 40mm was used.

Pipeline Expansion/Contraction

The swamp pipeline was conservatively assumed to be unrestrained. The magnitude of longitudinal expansion/contraction of the Sabah Land, Swamp and Subsea Pipeline due to uniform temperature and pressure differentials involved an intense detailed analysis.

Limiting Installation Criteria

For an unsupported span length, L, the permissible horizontal and vertical radii of curvature of the pipeline during installation was based on the steel strain criterion defined in the design parameters.

Pipe Handling Length

The allowable pipe handling span length, based on the steel strain, was determined to be 32.5 metres.

Unsupported Span Length

The pipeline span length limitations in the swamp areas were based on two criteria as defined below:

- Critical Span Length (L_{cr}) due to vortex shedding limitations. This is the adverse response to flow induced cyclic loads generated as a result of currents acting on pipeline free spans.
- Maximum Permissible Span Length (L_{max}) due to static stresses.

Steady state currents were used in the vortex shedding analysis. In the static stress analysis, loads due to differential pressures and self weight were considered.

The maximum allowable unsupported span length for the swamp pipeline was determined to be 28.7 metres.

PLANNING AND PROGRAMME

GENERAL

Intense construction planning was carried out in conjunction with engineering and procurement activities.

The planning was undertaken by the project management team comprising engineers and managers with experience from civil, mechanical, electrical and instrumentation disciplines. In this way, it was possible to ensure all facets of construction and operation were considered for the pipeline installation.

PROGRAMME

A fast track construction programme was required especially for the pipeline construction which was on the critical path. IPCO's rapid installation construction techniques were adopted for the 47.3km pipeline route resulting in the actual period for completion of pipeline construction being less than 6 months. This was an extremely short time considering that the pipe route had to traverse mainland Sabah through land, swamps, tidal mud flats, cross the sea to Labuan Island.

CONSTRUCTION OF PIPELINE IN SWAMP AND TIDAL FLATS

CONCEPT

With the help of aerial photographs, pipeline route survey, and site inspection, the following installation methods were selected:

- Swamp pipeline - surface push-pull in wet trench
- Tidal flats - bottom pull

For both methods jointing of pipes was undertaken in a fabrication yard. The pipeline was then installed progressively by pushing/pulling into a predredged trench until the desired length was achieved.

For the swamp areas, the pipe was maintained at positive buoyancy to minimise the pulling force. For the tidal flats area, the pipe was maintained at a slightly negative condition (just touching the bottom) to minimise environmental risks.

As the swamp and tidal flats pipeline sections were undertaken as one operation the same installation crews was used for both activities.

GENERAL

The swamp and tidal flat pipeline route is shown in Figure 1.

Pipe stringing was undertaken at the Fabrication Yard, designated RDX4. The location is shown in Figure 1 and details of the yard are shown schematically in Figure 2.

The route through the swamp was traversed by several rivers and streams which further complicated pipeline installation.

EQUIPMENT

Base Barge "Jin Lin"

A self contained accommodation/work barge, 45m x 15m x 3m (the "Jin Lin") was used to carry out the swamp and tidal flats dredging. This was to be the only barge on site and was used for swamp excavation, tidal flat dredging, surface-pull and bottom pull operations as well as a carrier barge for all equipment to be mobilised.

Pontoons

These were similar to "flexi float" pontoons but utilised I-beams as the interlocking members. 5 units of 24m x 4.9m x 1.5m rectangular shaped and 2 wedged shaped pontoons were built in IPCO's workshop. The pontoons were used as work platforms at various stage of the work.

Dredging Crane - American 7260 Crane

This crane was sized to be able to handle a 2.5m³ bucket as well as lift 18 tonnes excavators.

Excavators - Komatsu

Excavators with 0.7m³ buckets.

Despite the low ground pressure of the tracks, timber mats were necessary for the entire swamp excavation. The timber mats were sized at 2.4m wide by 3.6m long and were made with 25mm wire rope holding the timber together. 3 mats were provided for each machine.

The excavators were able to "walk" on the swamp by progressively laying mats ahead of the path to be taken.

Other Equipment

- Bottom Pull Winch
- 330HP Tug Boat
- 70 Tonnes Pipe Offloading Crane
- 28 Tonnes Capacity Side Boom
- Pipe Rollers
- Push-Pull Winches
- Anchoring Systems

DREDGING

Swamp Excavations

To install the pipe using the push-pull method, a continuous trench was required with sufficient depth of water to float the pipe. The design of the trench profile and invert was required to satisfy the following:

- . 1 metre cover.
- . Max 1:60 forward slope gradient at all stream crossings.
- . 500 metres vertical radius for pipeline coming out of stream crossings.
- . Sufficient water in trench to float pipes.

Using the above criteria, the invert level of the trenches except for river crossings and approaches varied from -1.8m MSL in the land section to -2.2 metre MSL towards Sungei Burong. This allowed the installation of the pipe to be undertaken continuously except at spring low tides.

The terrain and ground conditions varied from the land section at Fabrication Yard RDX4 to Sungei Burong. There were four river and several stream crossings.

Two types of machines were used for the excavation of the trenches in the swamps:

- **Excavators**
Excavators proved to be more efficient, economical and productive. However, maximum digging depth was only 4 metres for operating in the swamp with minimum risk.
- **Clamshell**
In areas where the dredging depth exceeded 4 metres, ie river/stream crossings and approaches, clamshell dredging was used.

Tidal Flats Dredging

A trench of 2.3km was needed in the tidal flats area. Initially, the tidal flats dredging was to be carried out by the excavators working on springs low tide.

After attempting dredging with the excavator and later with the clamshell, it was decided that an alternative dredger would be more productive for the Tidal Flats dredging. The dredger use to

prepare the shore approaches for the submarine pipeline was mobilised to undertake this work. This dredger worked in conjunction with the the "Jin Lin".

The dredger working in a 24-hour day operation, undertook most of the dredging working from the sea towards Sungei Burong.

The "Jin Lin" worked from Sungei Burong to the sea, undertaking the shoreward extent of the tidal flats dredging.

PIPELINE INSTALLATION

General

The specifications for the pipelines were as follows:

- Tidal Flats - 610mm diameter API 5L X 52 pipe of 8.74mm wall thickness with 60mm weight coat and 10mm internal mortar lining.
- Swamp - 610mm diameter API 5L X 52 pipe of 6.35mm wall thickness with 40mm weight coat and 10mm internal mortar lining.

Offloading yard

The offloading yard was situated by the river, close to the existing road and only 2km from the Fabrication yard RDX4.

Fabrication Yard

The Fabrication yard, RDX4, had an area of 150m x 50m. The yard layout is shown in Figure 2.

Jointing of Pipes

Pipes in double random lengths were welded on rollers and advanced progressively into a trench and pulled to Sungei Burong in a floating condition. Two continuous lengths were fabricated as follows:

- An approximate length of 4,120 metres consisting of 313 Tidal Flats and 29 Swamp pipe joints.
- An approximate length of 10,960 metres consisting of 839 Swamp and 81 Land pipe joints.

The tie in point was in a sheltered area in Sungei Burong.

The production of the pipe strings comprised welding, NDT, repairs and coat and wrap.

Push-pull Operation

A signalling system was set up in which each respective station signalled completion of work. On completion of work at the last station, an alarm would sound to start the pulling operation. The pipe was advanced 12 metres at a time.

Initially, only the pulling winch in the yard was used. Subsequently a pull barge was used to complete the pipe pull.

Bottom Pull

The entire tidal flats pipeline was installed by the bottom pull method with a submerged weight of approximately 25kg/m. The pull barge was repositioned on several locations during the pipe installation work, depending on the length of wire ropes to the pulling winch.

The pull was actually accomplished with a combination of surface and bottom pulls. The Tidal Flats and Swamp Pipeline was terminated at the -13m ACD contour mark to meet the submarine pipeline. The pipe was pulled with buoyancy tanks up till the exit of the dredged trench.

The bottom pull section for the pipe was approximately about 1270 metres.

Flooding

With the "Jin Lin" stationed in the mouth of Sungei Burong at the tie in point, the ends of the Swamp and Tidal Flats pipeline were attached to the "port" side of the barge. Both ends had test heads attached and were flooded to ensure that the pipe moved into the centreline of the trench.

Correction of Unsupported Spans

Prior to hydrotesting, remedial work was carried out on all unsupported spans exceeding 30 metres in length.

Sand bags were used to support spans at appropriate positions and the high spots were removed by jetting.

Pigging and Hydrotest

Pigging

The line was pigged using a 610mm bi-directional Pig, fitted with nylon brushes for cleaning and a gauging plate.

The swamp and land sections were pigged with a pig sent from the Water Treatment Facility to the tie-in point in Sungei Burong.

The tidal flats section was pigged, prior to commencement of the submarine pipeline installation. After the tie-in to the submarine line, another pig was sent from the tie-in area in Sungei Burong to verify that there was no damage during the tie-in and lowering operation.

Hydrotest

Two separate tests were carried out:

- For the land and swamp pipeline from the Water Treatment Facility to tie-in point in Sungei Burong.
- For tidal flats and submarine pipeline from tie in point in Sungei Burong to Labuan.

A final hydrotest was carried out on completion of the entire 47.3km line.

Tie-In

On completion of hydrotest of both lines, the final tie-in was completed in Sungei Burong. This location was preferred as it had the following advantages:

- Sheltered from waves and cross currents.
- Sufficient water to prevent grounding of "Jin Lin" and both pipelines.
- Need to only handle limited lifting loads from the pipeline.

Reinstatement

The reinstatement of the swamps required:

- The settlement of the pipes to their designed level.
- Plugging of all streams, banks and filling of trenches of all stream crossings.
- Placing of pipe line markers and test points along the pipeline route.

The rest of the trench was left open and spoil left where deposited during excavation. No sand bagging of the banks was required.

Due to the high siltation rate in the area and the method of lowering the pipe, the level of the pipe was found to be unacceptable especially in the following areas:

- Approaches/exits of stream crossings.
- Stream crossings where pipes were hung up on the banks instead of resting in the invert.

A long time was needed for the removal of surplus material in nearly all the stream crossings and approaches to ensure that the pipe settled to the designed/acceptable levels.

Several methods were used including air jetting, water jetting and pumping. The methods employed were restricted to locally/readily available equipment suitable for working in the swamp.

No backfill operation was carried out in the tidal flat but from soundings carried out a month after installation most of the pipeline was found to be covered to top of pipe.

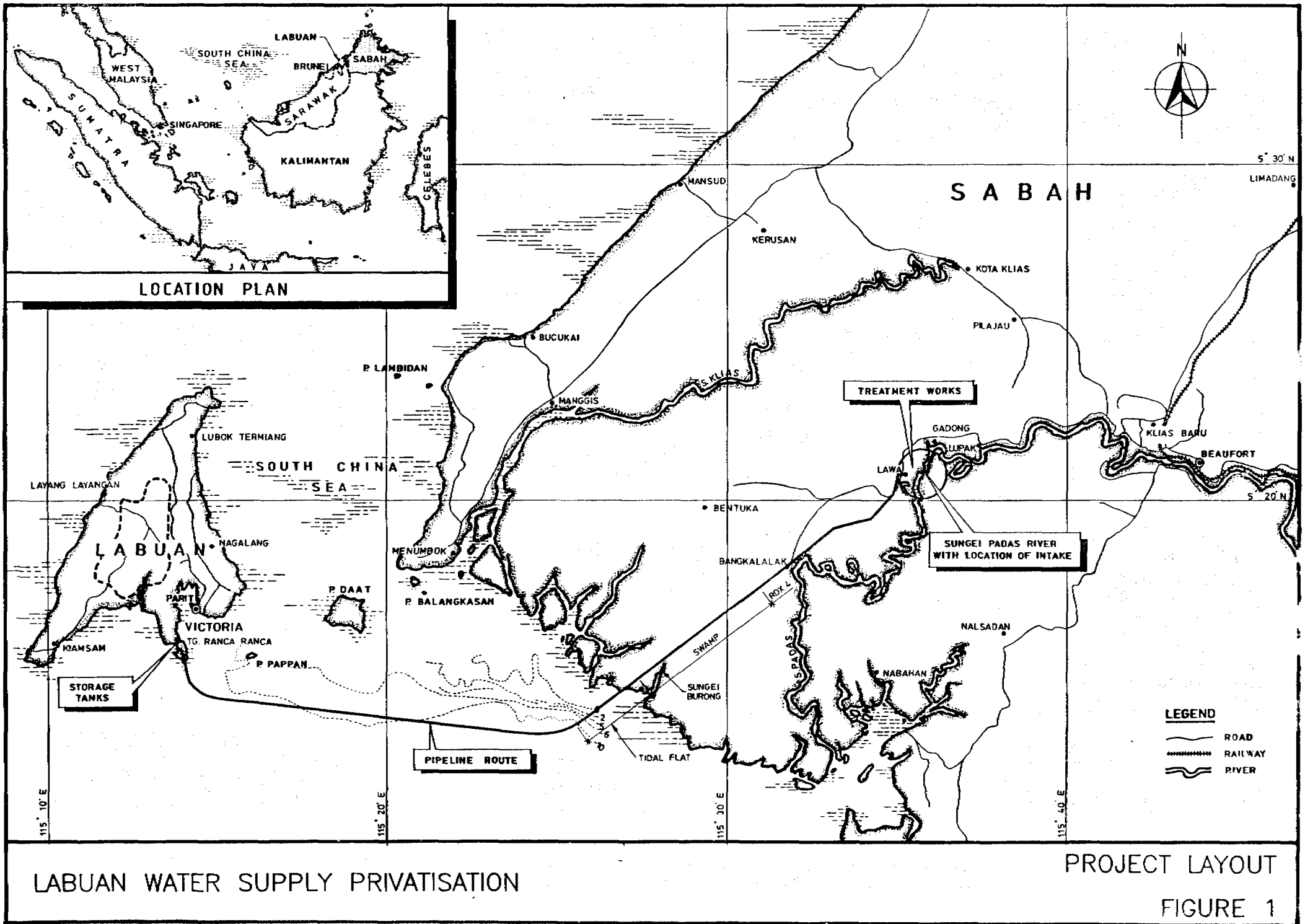
CONCLUSION

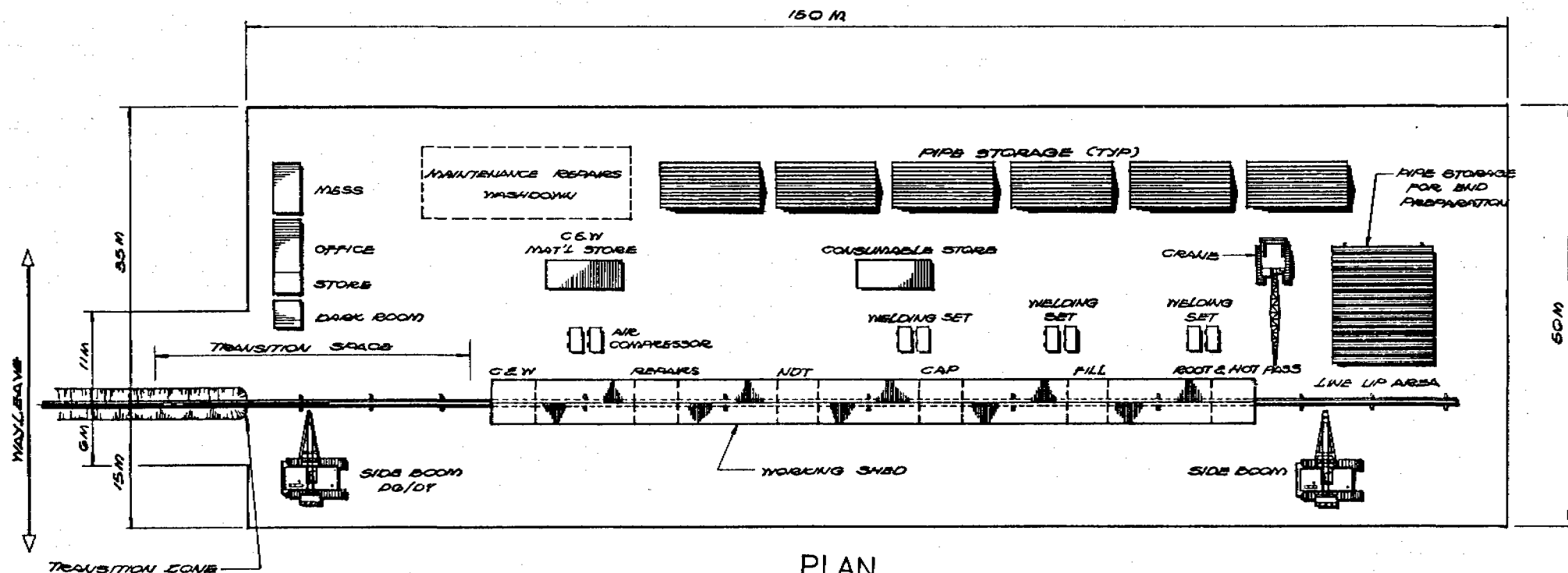
The installation of the swamp and tidal flats pipes can be easily accomplished if construction engineering is fully integrated with the pipeline design at an early stage in project planning.

The experience from this project showed that the swamps and terrain could be used advantageously for the installation.

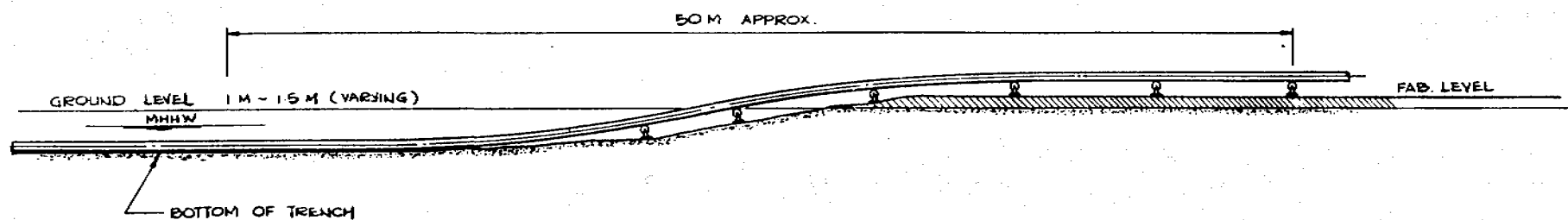
The bottom pull and push-pull operation although normally used for offshore projects was adopted favourably for the swamp and tidal flats pipeline installation in this instance.

It can be concluded that with proper planning, innovation, engineering, and the correct resources pipeline installation in swamp and tidal flats can be completed to a fast track programme in a cost effective manner.





PLAN

PUSH-PULL LAUNCH METHOD
TRANSITION

FABRICATION YARD LAYOUT

FIGURE 2

OTHER PAPERS

Nitrate Contamination Control in the Waterworks of Budapest

Silver Ion as Disinfectant for Swimming Pool Application

Water Supply Program in Uttar Pradesh (India)

- Centennial Panorama (1890 - 1990)

NITRATE CONTAMINATION CONTROL IN THE WATERWORKS OF BUDAPEST

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ABSTRACT

The Waterworks of Budapest is the largest system in Hungary with a capacity of 1,300,000 m³/day. 770 different wells are located along the gravel aquifer adjacent to the Danube river. There are two main wellfields supplying the capital, one on the Szentendre island of 55 km² north of Budapest and another on the Csepel island of 265 km² south of the city.

These wellfields provide water for 2 million of inhabitants in Budapest and 20 small towns. The gravel aquifer can be contaminated with nitrate as a consequence of intensive agriculture, sewage water and Danube itself. Groundwater quality is monitored in 250 observation wells. Specific sources of nitrate contamination are detected and engineering and administration interventions are made to solve the problem.

During the last 10 years nitrate concentration of drinking water has increased from 3 - 4 mg/litres to 10 - 15 and this trend is continuing. These values are still lower than the present threshold of 40 mg/litres.

According to economic analyses the prevention of nitrate contamination is far less expensive in this system than nitrate removal from drinking water. It is shown how supervising and monitoring activity results in minimizing nitrate contamination hazard.

INTRODUCTION

Nitrates as one of the major problems in public Water supplies first appeared in the early sixties, mainly in countries of well developed agricultural production.

In Hungary studies were initiated in the seventies on the sources, origin and fate of nitrates in natural waters [Fekete, 1984; Csaki and Endredi, 1981; Alföldi, 1982; Kelemen, and Tarjan, 1981]. Since the drinking water resources in Hungary are based mainly on subsurface water - 39 % is bank-filtered and 21 % is taken from the near surface - the growing NO_3 content in groundwaters has raised concern and drew the attention of water suppliers and governmental organisations toward possible solutions to the prevention of continuously increasing nitrate contamination.

A nation-wide project which was begun in the late seventies covered the following:

- collecting information about the degrees and risks nitrate contamination in Hungary,
- developing new nitrate removal technology or to adopt an appropriate technical solution to the removal of excess nitrate,
- developing adequate control and legislation to prevent the contamination of water resources.

The Budapest Waterworks is the largest drinking water supply system in the country and has a capacity of $1.300.000 \text{ m}^3/\text{day}$, representing 37 % of the country's total drinking water supply, and their experts played a significant role in the project.

The water supply of the Capital is based on the water resources of the Danube - mostly bank-filtered water - but in peak demand periods a direct intake from the Danube, treating the river water by conventional technology, provides supplementary supply. The bank-filtered well system consists of 770 different wells located along the gravel aquifer adjacent to the Danube river. Two main wellfields can be found: on Szentendre Island 55 km^2 north of Budapest, and Csepel Island 265 km^2 , south of the capital.

Drinking-water quality is determined by the River Danube itself, by the filter media and by the non-point source pollution of the background.

THE WATER QUALITY OF THE DANUBE

The Danube passing through 8 countries is second in size in Europe only to the Volga. The water quality of the Danube can be characterized by Fig.1 showing seasonal changes of special components within a year, depending on the discharge rate and climatic conditions.

Table 1. contains a 10 year series of water quality parameters along the Austrian/Czecho-Slovakian and Hungarian stretch of the Danube indicating the percentage changes of the parameters that the nitrate ion concentration is continuously increasing [Endredi, 1988].

Trends of Water Quality changes in the Danube (1978-1985) based on yearly data							
Section	Component						
	COD	BOD ₅	Dissolved O ₂	Ammonium	Nitrate	Ortho-phosphate	Conductivity
Felsen-Hütt	-2.5	+1.0	+0.4	+4.4	+1.1	+0.2	+0.2
Linz	-3.9	+3.1	+0.6	+0.5	+1.4	+0.2	+0.1
Ybbs-Pesenbeug	-2.6	+2.6	+0.6	+3.6	+0.4	+0.1	+0.3
Wien-Nussdorf	-3.1	+5.9	+0.9	+4.4	+1.9	+2.5	+0.4
Wolfsthal	-1.6	+3.4	+1.7	+3.9	+0.1	+1.7	+0.4
Rajka	-1.9	-2.6	-0.1	+0.6	+2.2	-0.4	+0.4
Komarom	-0.5	-1.9	-0.2	-0.1	+1.5	-1.0	+0.2
Almasneszmely	-0.1	-0.6	-0.2	-0.1	+1.6	-0.6	+0.0
Szob	-0.7	+0.1	+0.5	-0.7	+2.3	+0.0	+0.4
Budapest	-1.7	+1.3	+0.8	-3.4	+2.3	+1.7	+0.1

In Figg. 1. and 2. 20 year series are presented showing that the average NO₃ concentration and the range between the maximum and minimum values is growing. [Homonnay, 1984]

GRUNDWATER QUALITY

The same increasing rate is observed, in most of the bank-filtered wells, consequently also in the supplied drinking water.

Although the maximum values are still lower than the present drinking water standard of 40 mg/l, this increase has driven attention to the urgent need for remedial actions.

In addition, the temporary quality control of the wells has shown that in some cases and some areas, the concentration of nitrates was growing faster or was essentially higher than was measured in the Danube water. To understand the phenomena and to control the contamination 250 observation wells were located on both islands and continuously monitored. Figg. 3. and 4. show a typical picture of the different NO₃ levels measured in the observation wells.

The maps in Figg 3. and 4. indicate the pollution patterns of the Csepel and Szentendre Islands. The groundwater is significantly less polluted on Szentendre Island in comparison

to Csepel Island and on the latter, nitrate contamination is becoming a real danger. Figg. 5. represents the change in nitrate contamination in the cross section A at Szentendre Island varying in time. According to the definition of bank-well filtration movement, the flow of surface water from the river to the wells is the predominant flow, however a portion of highly polluted groundwater can reduce the quality of bank filtered water.

Figg. 6. shows an example measured in a collector well (Raney), where the nitrate concentration of the ground water entering the well mostly stems from the background.

NITRATE CONTROL - GRUNDWATER CONTROL

Monitoring the quality of the observation wells, together with the hydrological data, resulted in a programme for improving or at least retaining the quality of the drinking water sources by decreasing the nitrate load of water resources [Horvath and Lacko, 1987]. Within the frame work of this project it was recommended that the problem should be investigated to find an appropriate technical solution, permitting the use of contaminated water by appropriate nitrate removal techniques. However, as water resources are becoming scarce, emphasis should be placed on prevention of this kind of pollution.

For this purpose, investigation was begun into:

- supervising land-use on both islands, taking into account all possible sources of contamination and with special regard to the three main types of pollution:
- waste water infiltration in areas with no main drainage,
- waste deposits - mostly illegal. agricultural land use,
- determining the nitrate load from cultivated areas.

For this we used a model developed for and adapted to our circumstances [Bogardi and Bardossy, 1983].

NITRATE REMOVAL POSSIBILITIES

Although it may be obvious to give priority to the protection of drinking water resources by means of establishing protection zones, because of the lack of legislation and the present financial situation, it is necessary, to gather information about the alternative methods of removing the excess NO_3 by the technological means available in various countries.

The WHO Regional Office for Europe in co-operation with its Hungarian collaborating centre, the Water Resources Development Research Centre /VITUKI/, therefore decided to organize a working group to discuss the alternative methods of reducing and preventing excessive nitrate concentrations in drinking-water available in the countries of the European Region.

Twenty-four experts from 13 countries, including representatives of seven international institutes and organizations, attended the meeting, and some presented national reports or reviewed some specific aspect of the topic.

As a conclusion of that meeting held, in 1986 a list of installations operated in those countries was presented. The list indicated that only a limited number of plants have been in operation, and for relatively short periods. For long-term, full-scale application no simple, reliable and cheap process has been found and further research should be encouraged [WHO Working Group, 1988].

In Hungary reverse osmosis, electrodialysis, chemical nitrate reduction, ion exchange and various biological de-nitri- fication processes were evaluated, together with possible methods of source control.

At present, ion exchange and bacterial denitrification are applied at water treatment plants producing drinking-water. Even these methods, although in use, have disadvantages. For the time being the following three processes seem to be practicable [Princz, 1986]:

1.

A simple photochemical method for the elimination of nitrates in drinking water was developed. The method is based upon the photo-reduction of nitrate to nitrogen or di-nitrogenoxide in the presence of organic compounds. The process of nitrogen elimination is a multi-stage reaction which generally results in intermediates and derivatives NO_2 , NH_4 , $\text{H}_2\text{N}_2\text{O}_2$. If the irradiation of the water was carried out at $\text{pH}=8$ in the presence of 20-50 % surplus organic material, the formation of derivatives could be avoided. The main advantage of the method is that no special chemicals or micro-organisms are needed. Acetic acid, ethyl alcohol and other food industry products can be applied as organic compounds and bicarbonate of soda can be used for pH adjustment. Under the effect of UV light, besides denitrification, the sterilization of the water is also accomplished. Due to its simplicity the process also renders the construction of a simple device for domestic use possible. The present capacity of the process is 3-4 literes nitrogen free water/kWh in the range of 100-200 mg. influent nitrate concentration.

2.

A new approach to biological nitrate removal involves the use of lower fungi which present no health hazard to human consumers [Kiss, 1986]. The principle underlying the method-named AZOTEX-B - is that, in their biological activity, fungi use nitrates, reducing them within their cell substance. Aerobic conditions are essential for the metabolism of fungi. Since the organic content of drinking-water is low, organic matter must be added to the untreated water to achieve effective nitrate removal. The unused part of this organic matter is then removed from the treated water in stages during the secondary treatment that is always unavoidable when using biological methods. A pilot plant with a capacity of 50 l/h was built and operated with promising results.

3.

The Nitredox - a trademark of Vyredometer - is the in-situ removal of nitrates, using bacteriological processes. A typical Nitredox plant comprises a number of injection-pumping wells located on the circumference of two concentric circles and a pumping well in the centre through which water, partly free of nitrates is produced. Water with the carbon nutrient is injected through the wells located on the outer circle play the role of the injection wells. The Nitredox process is associated with the formation of nitrogen, which is removed through the wells located on the inner circle. Nitredox processes include flow phenomena, transport, chemical reactions and bacteriological processes. These phenomena - chemical reactions and bacteriological processes - are described and formulated mathematically as first step in the mathematical modeling of such processes.

CONCLUSIONS

Although the researchers have made several promising attempts to remove nitrates from drinking water, preventive measures need to be adapted not only because it is important to protect the environment but also because a suitable treatment process which affords technical reliability, low costs and long-term, full-scale application have not been developed.

Assessment of the public health risk and evaluation of the possible or real consequences of contamination can only be made if the water supplier is properly informed of the processes going on in the water sources. The need for detailed information about the quality of the water resources and its geographical and hydrological backgrounds urged the water-suppliers to create and develop a monitoring system. To preserve the quality of water resources for the future, an effort must be made to reduce the nitrate pollution in strict co-operation with the various authorities involved; agriculture, water management industry, environmental and public health.

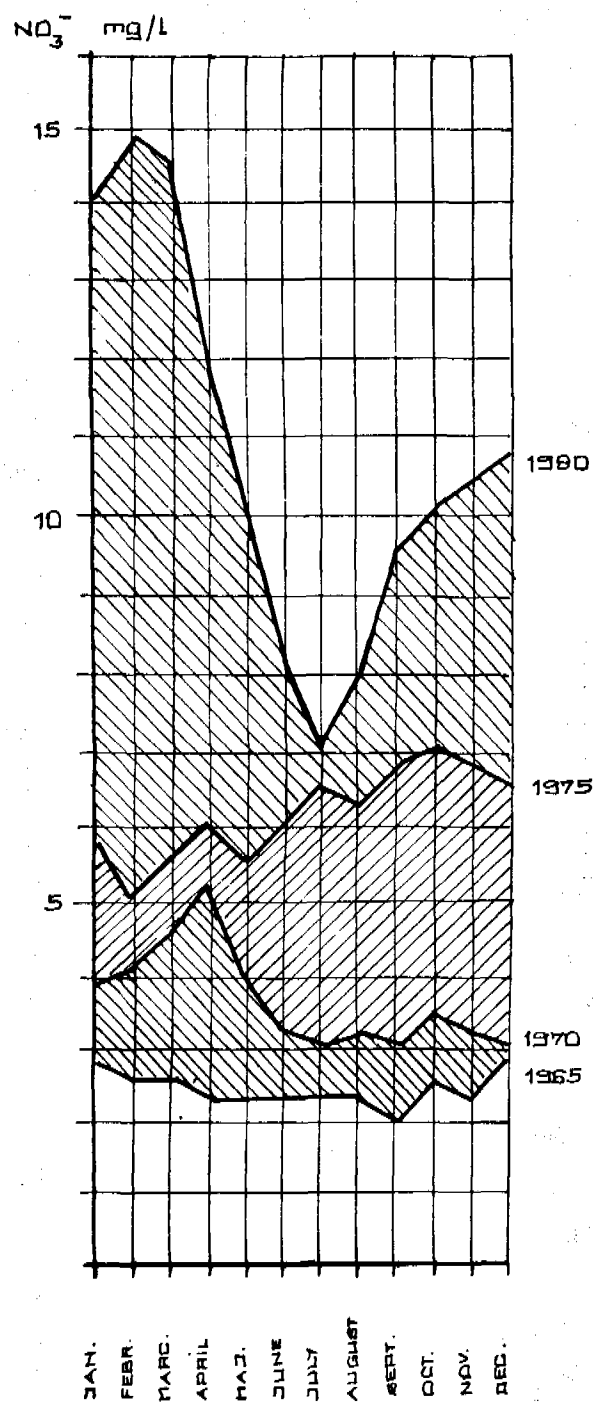


Figure 1.
Danube nitrate concentration at Budapest based on monthly average values

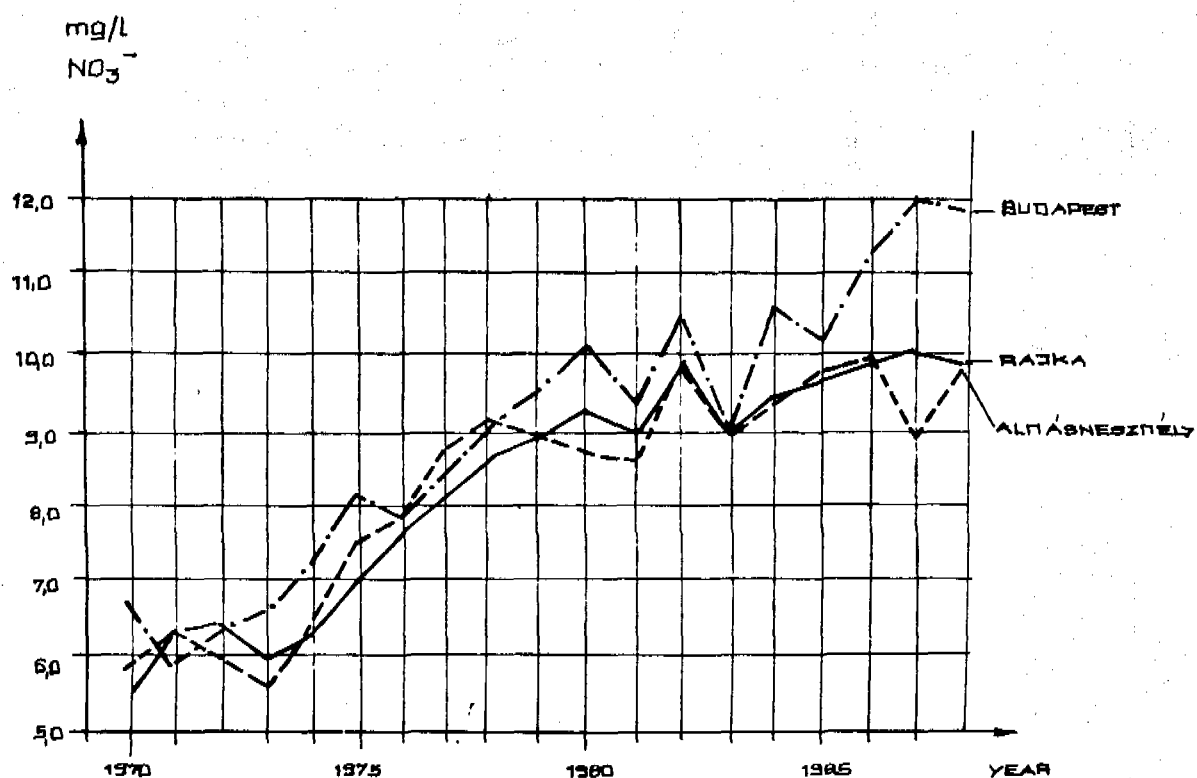


Figure 2.
Danube nitrate concentration at three location upstream Budapest in the Hungarian section between 1970-1990.

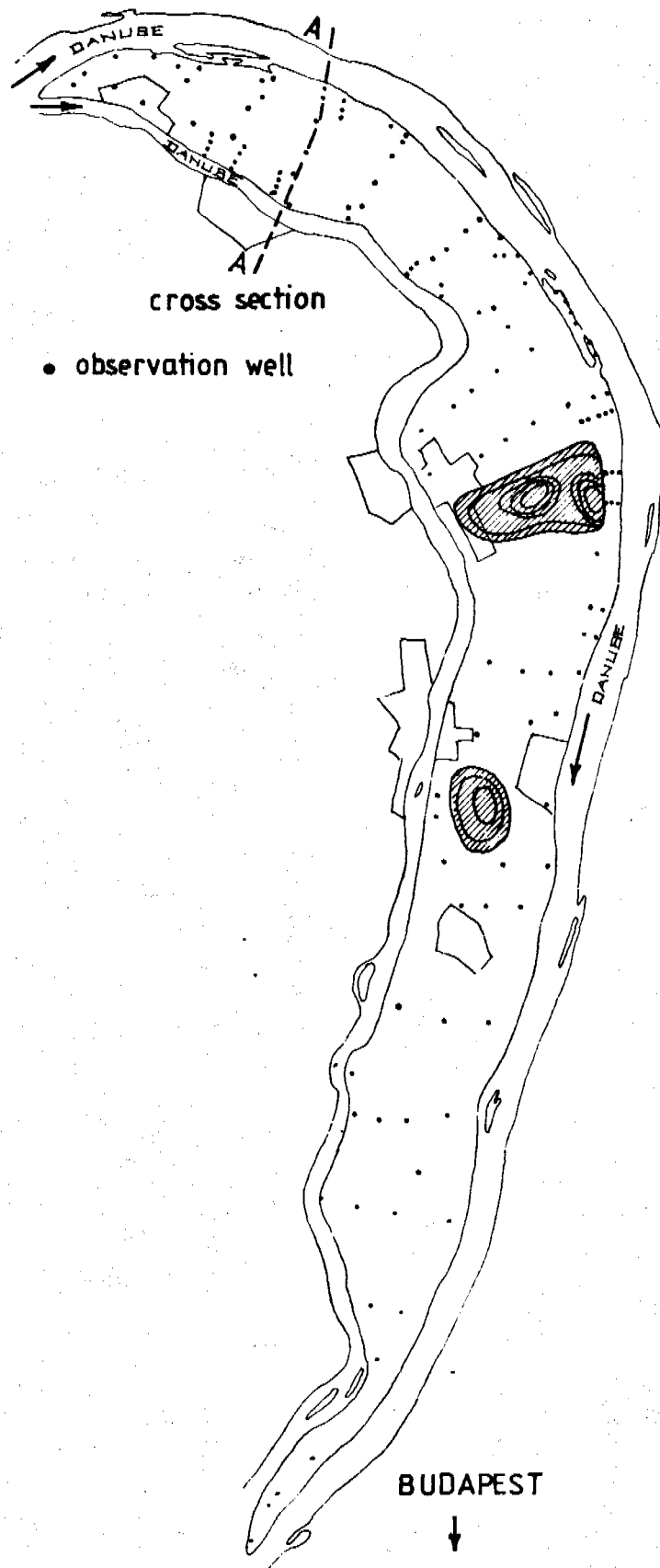


Figure 3.
Nitrate-ion concentration (>40 mg/l) in observation wells in the Szentendre Island.

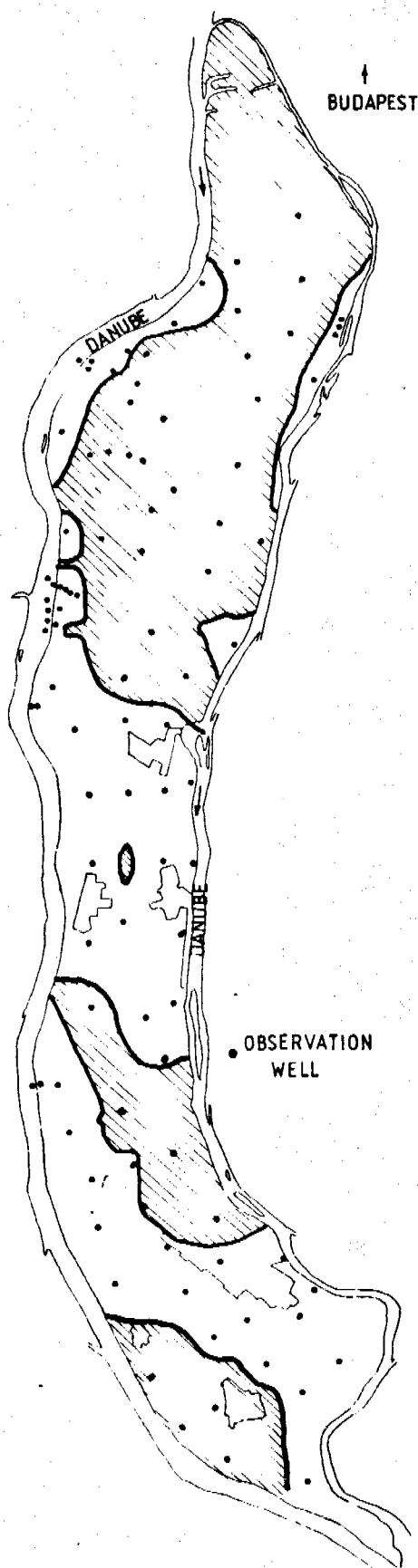
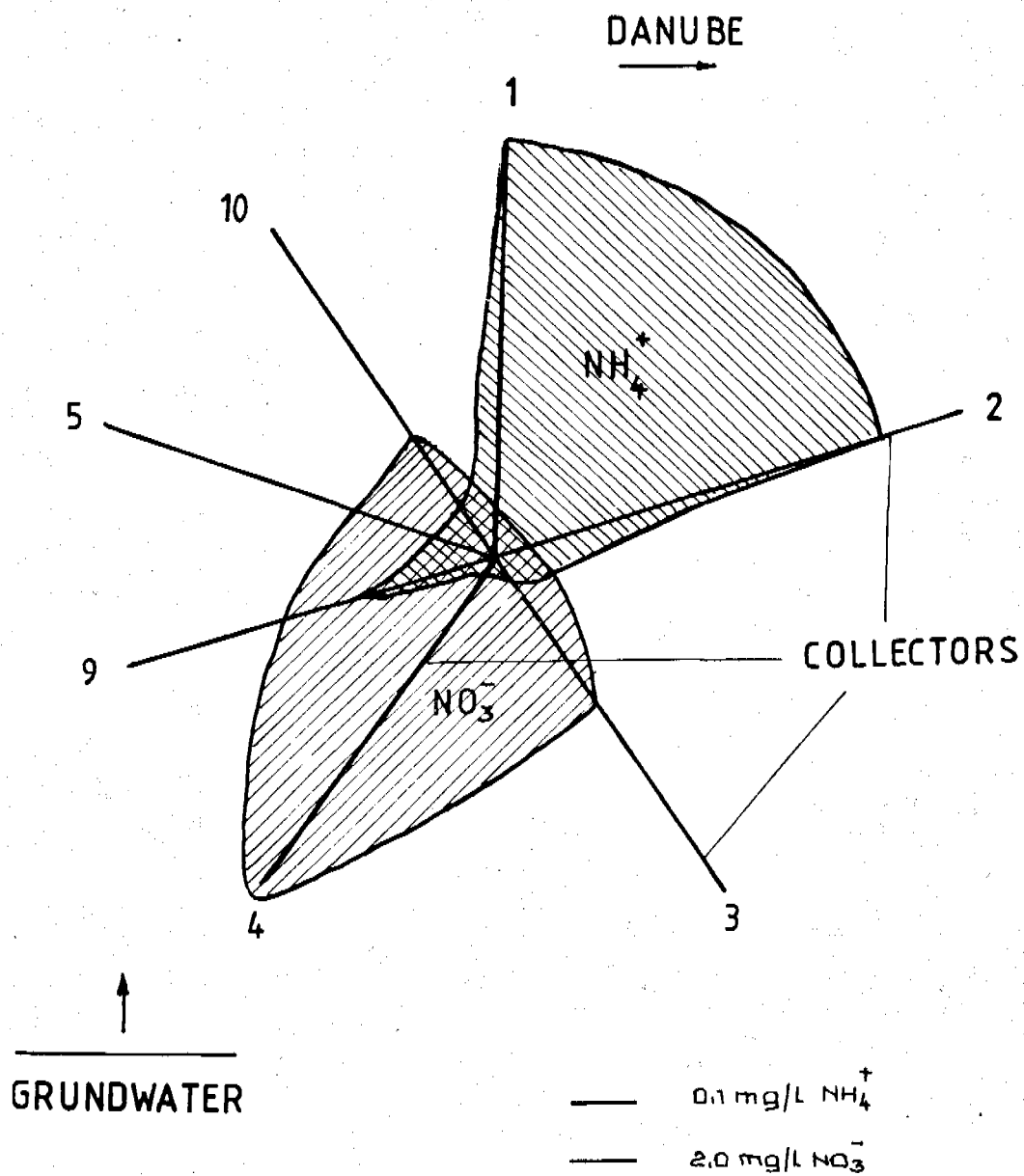


Figure 4.
Nitrate-ion concentration (> 40 mg/l) in observation wells in the Csepel Island.



KISOROSZI N°- 12. COLLECTOR WELL (Raney)

Figure 6.
Nitrate contamination in different sections of collector well.

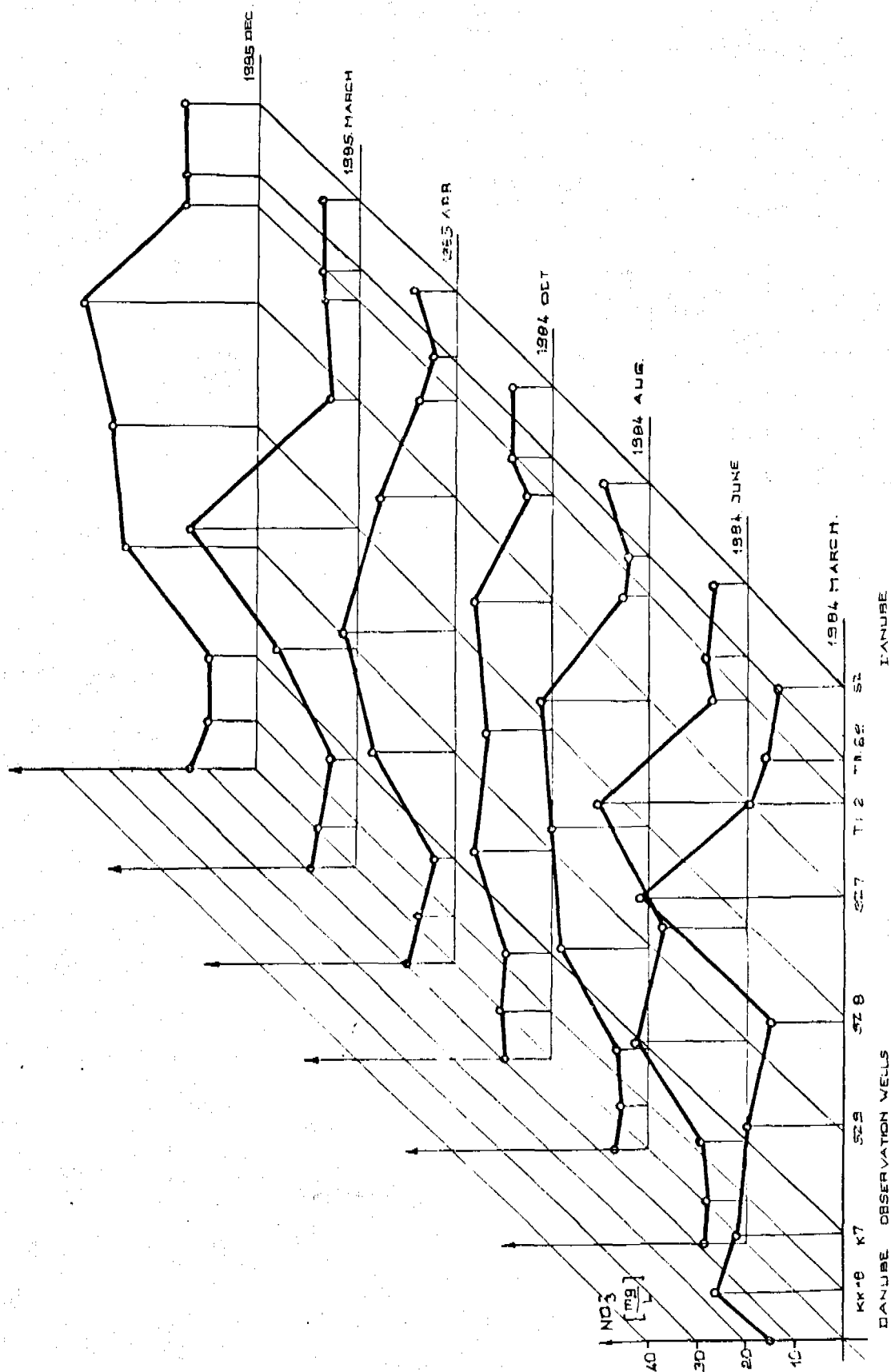


Figure 5.
Nitrate contamination in the northern part of Szentendre Island.

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- Summary reports on WHO Working Group. 1988 Removal of nitrates from drinking water. WHO Koppenhagen Trends of Water Quality changes in the Danube (1978-1985) based on yearly data

SILVER ION AS DISINFECTANT FOR SWIMMING POOL APPLICATION

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ABSTRACT: This paper presents the laboratory studies of using silver ion as disinfectant for swimming pool water. Silver ion is emitted from a newly invented instrument, Crystal Clear Purifier, based on the electrolytic decomposition process of silver metal. Silver ion exhibited an excellent bactericidal property. Bacteria were eliminated even at a silver ion concentration of 6 mg/l. The bactericidal efficiency increased with the increase of silver ion concentration. Higher bactericidal efficiency of silver ion was achieved at alkaline condition.

INTRODUCTION

Water used for swimming pool must be constantly filtered and treated with disinfectant in order to ensure the health of swimmers. Since swimmers may ingest some of the water from the pool, the quality of water in swimming pools must be equal to or exceed the bacteriological standards prescribed for drinking water. Disinfection of swimming pool water is required to inactivate harmful microorganisms to ensure the safety of swimmers.

Chemicals are commonly used for pool water disinfection. The chemicals that can be used as effective disinfectants are chlorine, bromine, iodine and ozone. Chlorine is the most commonly used chemical for swimming pool water disinfection. It is mainly due to cost advantage and its residual disinfection effect. However, chlorine treatment for swimming pool water has been known to cause eye and skin irritations, bleaching of hair and swimwear, and produces strong unpleasant smell.

Metal ions such as silver, copper, mercury, cobalt and nickel exhibit significant bactericidal properties with minute concentration. The term "Oligodynamic" or "forces of trifles" has been used to describe the sterilizing action of minute quantities of metal ions. Silver is the only metal ion used for water disinfection, particularly for swimming pool water.

Silver has been reported as an effective disinfectant since 1869 (Raulin 1969). Researchers from various countries have demonstrated that concentrations of silver ion

as low as 10 ug/L (10 ppb) are sufficient to destroy microorganisms (Shapiro & Hale 1937, Newton & Jones 1949, Chambers et al 1953, Chambers 1962). Silver is not only has an excellent bactericidal properties, it also provides powerful and long-lasting residual effect for continuing protection. Silver ion for water treatment does not produce taste, odor, color or form addition products that might do so. Another advantage is that silver ion is nontoxic to humans at concentration of 500 ug/L or below. Drinking Water Standards of the United States Public Health Service (US Public Health Service 1962, US EPA 1976) set an upper limit of 50 ug/L mainly to avoid the development in an individual who regularly drank the water of the condition known as "argyria". This is a permanent blue-gray discoloration of skin, eyes, and mucous membrane.

Even though silver is a better disinfectant for water treatment as compare to chlorine, it has not been used in large scale application. In a report issued by the US Public Health Service in 1962 (US Public Health Service 1962), it was mentioned that because of its relative high cost, long reaction time, and no satisfactory method for the quantitative determination of silver in low concentration, silver is not a very useful disinfectant in large applications. However, the report noted that silver might be useful in specific, small applications such as swimming pools. With the advancement of science and technology, instruments for accurate and fast determination of silver concentration at ppb (ug/L) level are available.

Silver can be applied as a solution of its salts or by desorption from filter beds of silver-coated sand, carbon, fabric, or other coated silver-releasing materials. Other form of silver application can be achieved by electrolytic decomposition of the metallic element. With proper design and instrumentation the electrolytic decomposition of silver for water disinfection can be cheaper than chlorine.

Silver can be used as an effective bactericide for swimming pool. Crystal Clear Purifier designed by Inovatech consists of a pair of electrodes with a blends of copper and silver elements. By a process of electrolysis, various proportions of metal ions are released into the water. The emission rates of metal ions are directly related to the amount of electrical current applied. The School of Civil & Structural Engineering of Nanyang Technological University was commissioned to conduct the study of the bactericidal effectiveness of silver ion liberated by the Crystal Clear Purifier. This report presents the results of laboratory investigations of the bactericidal effectiveness of silver ion.

METHODOLOGY

Series of tests were conducted to study the bactericidal effectiveness of silver ion. Various silver ion concentrations in the water were generated by the Crystal Clear Purifier. Water samples contained various amounts of silver ions were collected after 24 hours. Silver ion concentrations for various water samples were determined by the Atomic Absorption Spectrophotometer with graphite furnace. Altogether, eight water samples with different silver ion concentrations ranged from 6 ug/L to 110 ug/L were prepared. The silver ion concentrations for the eight water samples were 5 ug/L, 10 ug/L, 25 ug/L, 35 ug/L, 45 ug/L, 70 ug/L, 90 ug/L and 110 ug/L.

Samples of screened and degritted wastewater from the Jurong Sewage Treatment Works were collected to provide bacteria population for the bactericidal tests. Based on previous

investigations conducted at the Nanyang Technological University, only 10 ml of fresh wastewater samples were required to provide sufficient bacteria densities for the water samples. The tests were conducted in a rectangular tank with dimensions of 700 mm x 300 mm x 350 mm, and the volume of water sample was 70 litres. After wastewater samples were added to the water in the tank, flow was continuously circulated in the tank by a constant-flow pump to create water movement inside the tank throughout the experiment.

Samples were collected at various time intervals, 0, 10 minutes, 20 minutes, 30 minutes, 1 hour, 1 1/2 hours, 3 hours, 6 hours, 8 hours, 24 hours and 48 hours. Two different types of bacteria, Total Coliforms bacteria and Escherichia Coli (E. Coli) were measured. These two types of bacteria are commonly used as indicators for the bacteriological quality of potable water. Membrane filter technique with media recommended by the American Public Health Association, American Water Works Association and Water Pollution Control Federation were used. The bacteria densities were evaluated in accordance with the Standard Methods for Water and Wastewater Examination (APHA, AWWA, WPCF 1985).

DISCUSSION

Bactericidal Effectiveness Of Silver Ion

For every water samples collected from the laboratory studies, three dilutions were prepared for the bacteria determinations. The average for the three dilutions were used as the bacteria density for that sample. The average initial bacterial densities were 10,000 and 6,000 per ml of sample for the Total Coliform bacteria and E. Coli respectively. Table I presented the percent remaining of Total Coliform bacteria densities at different time intervals for various silver ion concentrations. Table II showed the bactericidal effectiveness of various silver ion concentration on E. Coli at different time intervals. Bacteria were completely eliminated after 8 hours of exposure to silver ion, therefore Tables I and II did not include information beyond 8 hours.

As shown in Tables I and II the bactericidal efficiency increased with the increase of silver ion concentration. Shorter duration was required for the elimination of bacteria at higher silver ion concentration. For lower silver ion concentration of 6 ug/L and 10 ug/L, the bacteria were eliminated after 8 hrs. For silver ions concentration of 25 ug/L, 100% removal of bacteria was obtained at 3 hrs. At medium silver ion concentrations of 35 ug/L and 45 ug/L, bacteria were removed at 1 1/2 hours. At higher silver ion concentrations of 70 ug/L to 110 ug/L bacteria were eliminated at 30 minutes.

Tables I and II showed that the destruction or inactivation of bacteria is not instantaneous. The process occurred gradually, in a manner similar to many physiochemical processes. The rate of destruction of bacteria can be expressed by a first-order kinetic equation usually referred to as Chick's law. Chick's law simply states that the rate of bacteria destruction is directly proportional to the number of living bacteria remaining at any specified time. The law can be expressed by the following equation.

$$N = N_0 e^{-kt}$$

where N = the number of bacteria per unit volume surviving at time t

N_0 = the initial number of bacteria per unit volume

k = rate constant

Based on the above equation and information from Tables I and II the times required for 99% (t_{99}) destruction of bacteria can be estimated. The value of t_{99} is commonly used as parameters for bactericidal effectiveness of disinfectants. Figure 1 presented the t_{99} values for various silver ion concentrations on the destruction of Total Coliform bacteria and E. Coli.

Figure 1 showed that the time required for the destruction of Total Coliform bacteria is markedly reduced with higher silver ion concentration. At silver ion concentration of 6 ug/L, t_{99} was 242 minutes and decreased to 19 minutes at silver ion concentration of 110 ug/L and below, a rise of 10 ug/L silver ion concentration decreased the destruction time t_{99} by 95 minutes. For silver ion concentrations of 35 ug/L to 110 ug/L, a rise of 10 ug/L reduced t_{99} by 3.8 minutes.

The time required for the destruction of E. Coli was also markedly reduced with the increase of silver ion concentration as shown in Figure 1. The time required for the elimination of E. Coli was shorter than the time required for the destruction of Total Coliform bacteria. The t_{99} values ranged from 16 minutes to 177 minutes for silver ion concentrations of 110 ug/L and 6 ug/L respectively.

For silver ion concentration of 25 ug/L or below, t_{99} value for the destruction of E. Coli decreased by 64 minutes for an increase of 10 ug of silver ion concentration. For silver ion concentrations of 35 ug/L, t_{99} decreased by 4.1 minutes for a rise of 10 ug/L silver ion concentration.

Effect of pH on Bactericidal Effectiveness

A series of tests were carried out to investigate the effect of pH on the bactericidal effectiveness of silver ion. Three silver ion concentrations at low, medium and high ranges were selected. The silver ion concentrations selected were 10 ug/L, 45 ug/L and 70 ug/L. For each concentration, pH values of the samples were adjusted at the range of 6 to 8 by the addition of HCl or NaOH. The pH values for the tests were 6, 6.5, 7, 7.5 and 8.0.

Tables III and IV showed that the bactericidal efficiency was higher at higher pH value. It required shorter time to eliminate the bacteria at higher pH. For silver ion concentration of 10 ug/L, the bacteria were eliminated at 6 hours for pH of 7 to 8. However, at pH of 6 and 6.5, complete destruction of bacteria were achieved at 8 hours. The results were more significant for higher silver ion concentration. For silver ion concentration of 45 ug/L, elimination of bacteria were obtained at 6 hours at pH of 6 and decreased to 1 hour at pH of 8. Similar results were obtained for higher concentration

of silver ion. At silver ion concentration of 75 ug/L bacteria were eliminated at 1 hour for pH of 8 and increased to 3 hours at pH of 6.

The time required for 99% (t_{99}) destruction at various pH were estimated from Tables III and IV. The effects of pH on the bactericidal effectiveness of silver ion for Total Coliform bacteria were shown in Table III and Figure 2. The results indicated that bactericidal efficiency increased with higher pH value. For silver ion concentration of 10 ug/L, the time required for 99% destruction, t_{99} of Total Coliform bacteria was 105 minutes at pH of 8 and increased to 418 minutes at pH of 6.

For silver ion concentration of 45 ug/L, t_{99} increased from 30 minutes to 78 minutes at pH of 8 and 6 respectively. For silver ion concentration of 70 ug/L, t_{99} increased from 24 minutes at pH of 8 to 63 minutes at pH of 6.

For water samples with pH value below 7 (acidic condition), a reduction of pH by 1 unit, the times required for 99% destruction of Total Coliform bacteria increased by a factor of 1.95. For water samples with pH above 7 (alkaline condition), a rise in pH by 1 unit, t_{99} decreased by a factor of 1.46. For pH values of 7 and 7.5, pH range for domestic water supply, the bactericidal effectiveness of silver ion were similar.

The results for the destruction of E. Coli at various pH values for different silver ion concentrations were presented in Table IV and Figure 3. The bactericidal efficiency of E. Coli increased with the increase of pH value. At low silver ion concentration of 10 ug/L, t_{99} increased from 94 minutes at pH of 8 to 419 minutes at pH of 6. For silver concentrations of 45 ug/L and 70 ug/L, t_{99} were 21 minutes and 19 minutes respectively at pH of 8.0 and increased to 72 minutes and 68 minutes respectively at pH of 6.0.

For water with pH below 7, the times required for 99% destruction of E. Coli increased by a factor of 2.43 for every unit drop in pH. For pH above 7, t_{99} decreased by a factor of 1.59 for every unit rise in pH. At pH of 7 and 7.5, the bactericidal efficiency for silver ion were about the same.

CONCLUSION

Silver ion exhibited an excellent bactericidal properties. Bacteria can be eliminated even at a silver concentration of 6 ug/L. However, to be used as an effective disinfectant and provide long lasting residual protection, silver ion concentration in the swimming pool should be maintained at a level of 10 ug/L to 30 ug/L.

The time required for the destruction of bacteria is markedly reduced with higher silver ion concentration. For silver ion concentration of 25 ug/L and below, a rise of 10 ug/L decreased the times required for 99% (t_{99}) destruction of Total Coliform bacteria by 95 minutes. The time required for the elimination of E. Coli, t_{99} decreased by 64 minutes for an increase of 10 ug/L silver ion concentration.

For silver ion concentrations of 35 ug/L to 110 ug/L, t_{99} values for Total Coliform bacteria and E. Coli decreased by 3.8 minutes and 5.9 minutes respectively for an increase of 10 ug/L silver ion concentration.

The higher the pH value of the water, the better the bactericidal efficiency. For water with pH value below 7, the times required for 99% destruction of Total Coliform bacteria increased by the factors of 1.95 for a drop in pH of one unit. For water with pH value above 7, t_{99} values decreased by a factor of 1.46 and 1.59 for Total Coliform bacteria and E. Coli respectively for an increase in pH of one unit. For pH values of 7 and 7.5, the bactericidal effectiveness of silver ion were similar.

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**TABLE I EFFECT OF SILVER ION ON BACTERICIDAL EFFICIENCY
OF TOTAL COLIFORM BACTERIA**

Silver Concentration ug/L	Percent of Total Coliform Bacteria Surviving After							
	10 min	20 min	30 min	1 hr	1 1/2 hr	3 hr	6 hr	8 hr
6	95	75	40	15	8	3	1.5	-
10	95	70	30	10	4	2	1	-
25	32.5	20	10	5	0.8	-	-	-
35	30	12	5	2	-	-	-	-
45	26	8	3.5	1	-	-	-	-
70	20	5	1	-	-	-	-	-
90	15	1	-	-	-	-	-	-
110	10	0.8	-	-	-	-	-	-

**TABLE II EFFECT OF SILVER ION ON BACTERICIDAL EFFICIENCY
OF E. COLI**

Silver Concentration ug/L	Percent of E. Coli Surviving After							
	10 min	20 min	30 min	1 hr	1 1/2 hr	3 hr	6 hr	8 hr
6	95	75	35	10	1	0.8	0.6	-
10	90	70	25	9	0.7	-	-	-
25	30	12	8	3.1	1.1	-	-	-
35	25	11	6	1.3	-	-	-	-
45	20	8	2.5	0.7	-	-	-	-
70	18	3	0.5	-	-	-	-	-
90	15	0.6	-	-	-	-	-	-
110	10	0.3	-	-	-	-	-	-

TABLE III EFFECT OF pH ON REMOVAL OF TOTAL COLIFORM BACTERIA

Silver Concentration ug/L	pH	Percent of Bacteria Surviving After							
		10 min	20 min	30 min	1 hr	1 1/2 hr	3 hr	6 hr	8 hr
70	6	30	20	9	3	1	0.5	-	-
	6.5	20	10	2	0.8	-	-	-	-
	7.0	18	4	0.8	-	-	-	-	-
	7.5	18	3	0.5	-	-	-	-	-
	8.0	10	0.5	-	-	-	-	-	-
45	6	35	25	11	2.3	1.5	0.8	-	-
	6.5	25	15	6	1.5	-	-	-	-
	7.0	20	10	2.5	0.8	-	-	-	-
	7.5	20	8	2.5	0.7	-	-	-	-
	8.0	14	10	-	-	-	-	-	-
10	6	100	95	50	35	20	17	0.5	-
	6.5	90	80	40	22	12	2	-	-
	7.0	90	70	26	10	0.7	-	-	-
	7.5	90	70	25	9	0.7	-	-	-
	8.0	70	48	15	5	0.5	-	-	-

TABLE IV EFFECT OF pH ON REMOVAL OF E. COLI

Silver Concentration ug/L	pH	Percent of Bacteria Surviving After							
		10 min	20 min	30 min	1 hr	1 1/2 hr	3 hr	6 hr	8 hr
70	6	35	20	12	3	1.0	-	-	-
	6.5	25	10	7	1.9	0.7	-	-	-
	7.0	20	5	2	0.9	-	-	-	-
	7.5	20	5	1	-	-	-	-	-
	8.0	10	3	0.5	-	-	-	-	-
45	6	45	30	17	8	1	0.5	-	-
	6.5	30	15	8	2	0.9	-	-	-
	7.0	25	8	4	1	-	-	-	-
	7.5	26	8	3.5	1	-	-	-	-
	8.0	20	4	2	-	-	-	-	-
10	6	100	95	55	35	14	8	1	-
	6.5	95	85	40	25	10	2	0.5	-
	7.0	95	72	29	13	5	1	-	-
	7.5	95	70	25	10	4	1	-	-
	8.0	79	54	18	8	1	0.5	-	-

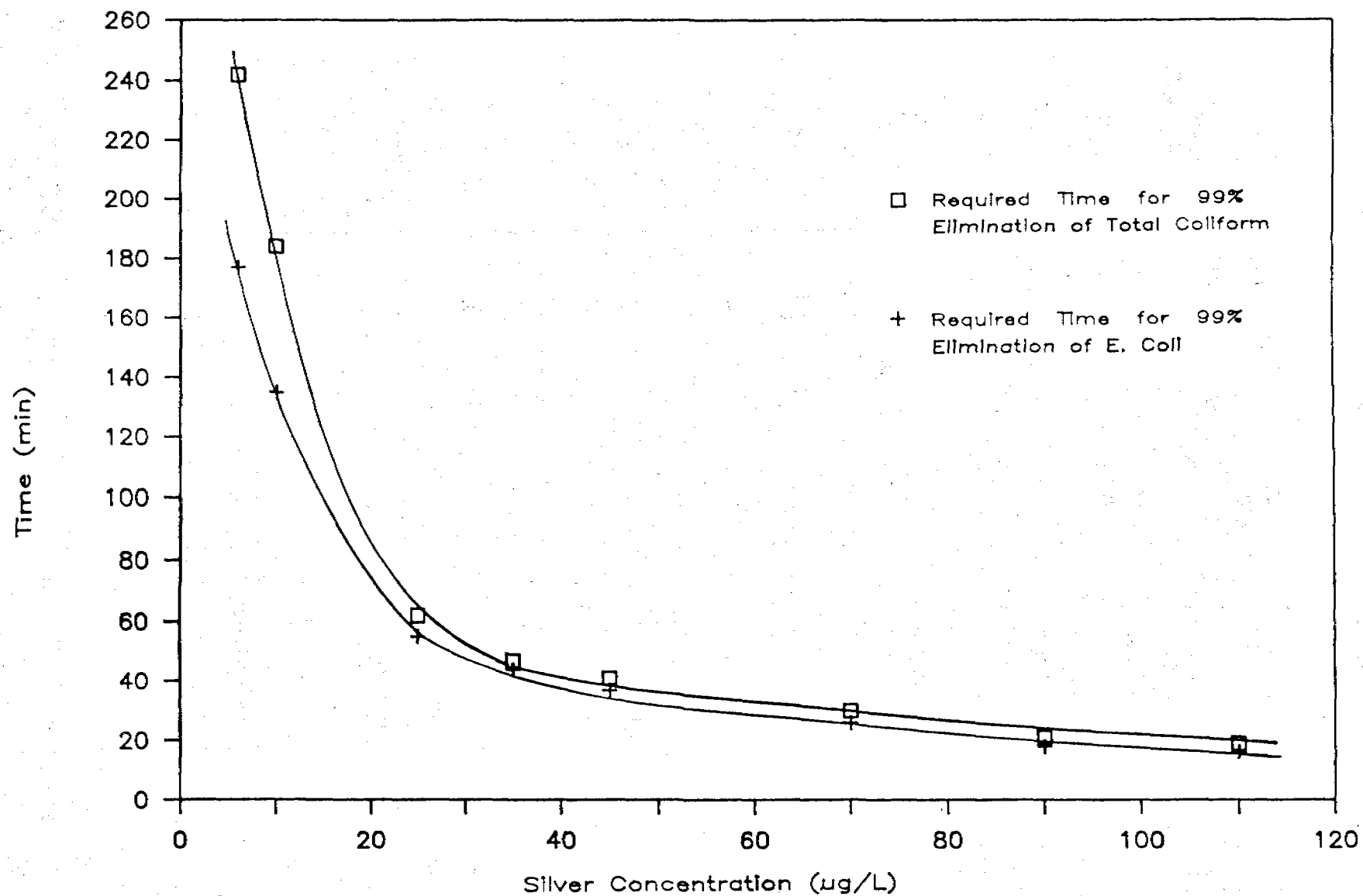


Fig. 1 Bactericidal effectiveness of silver ion

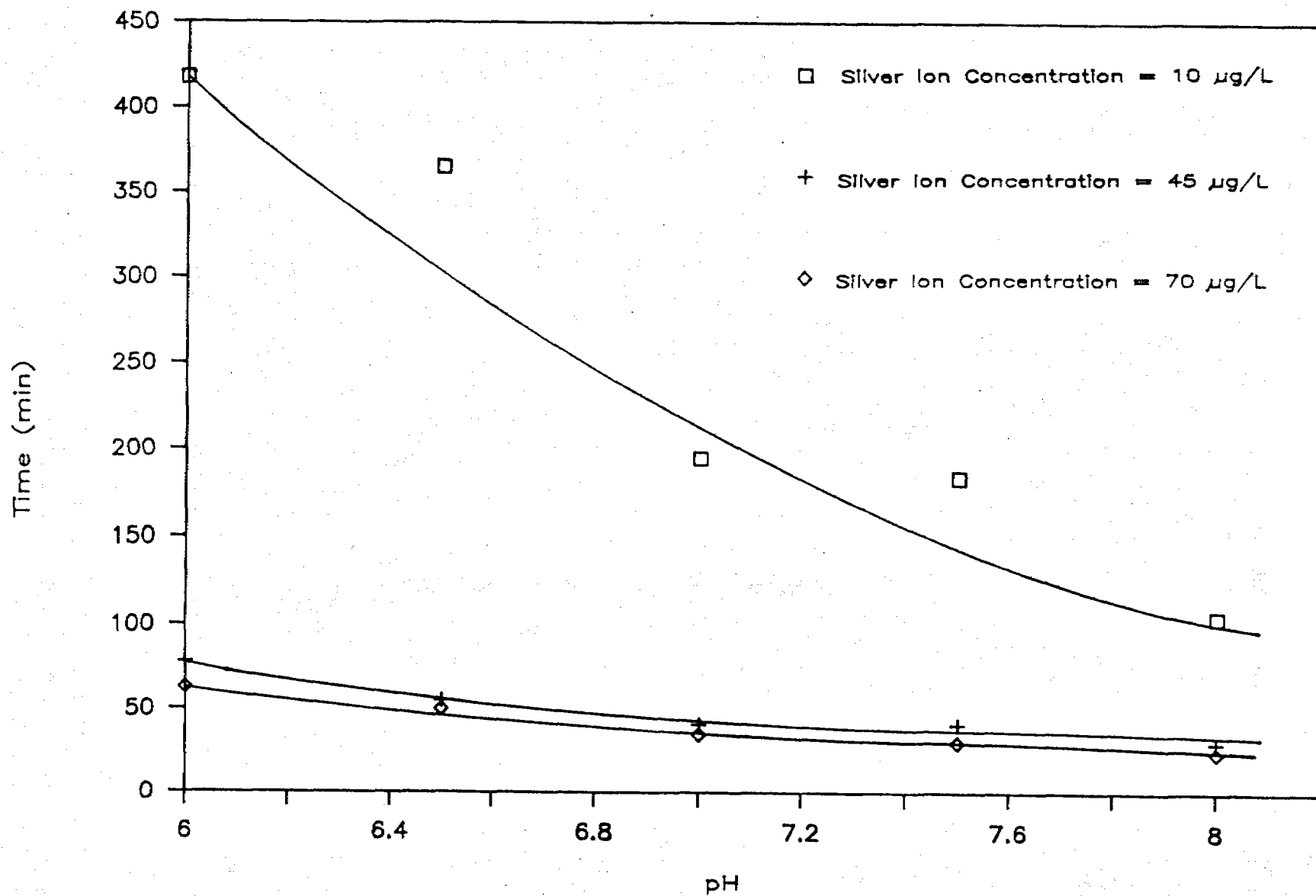


Fig. 2 Effect of pH on t_{99} for Total Coliform Bacteria

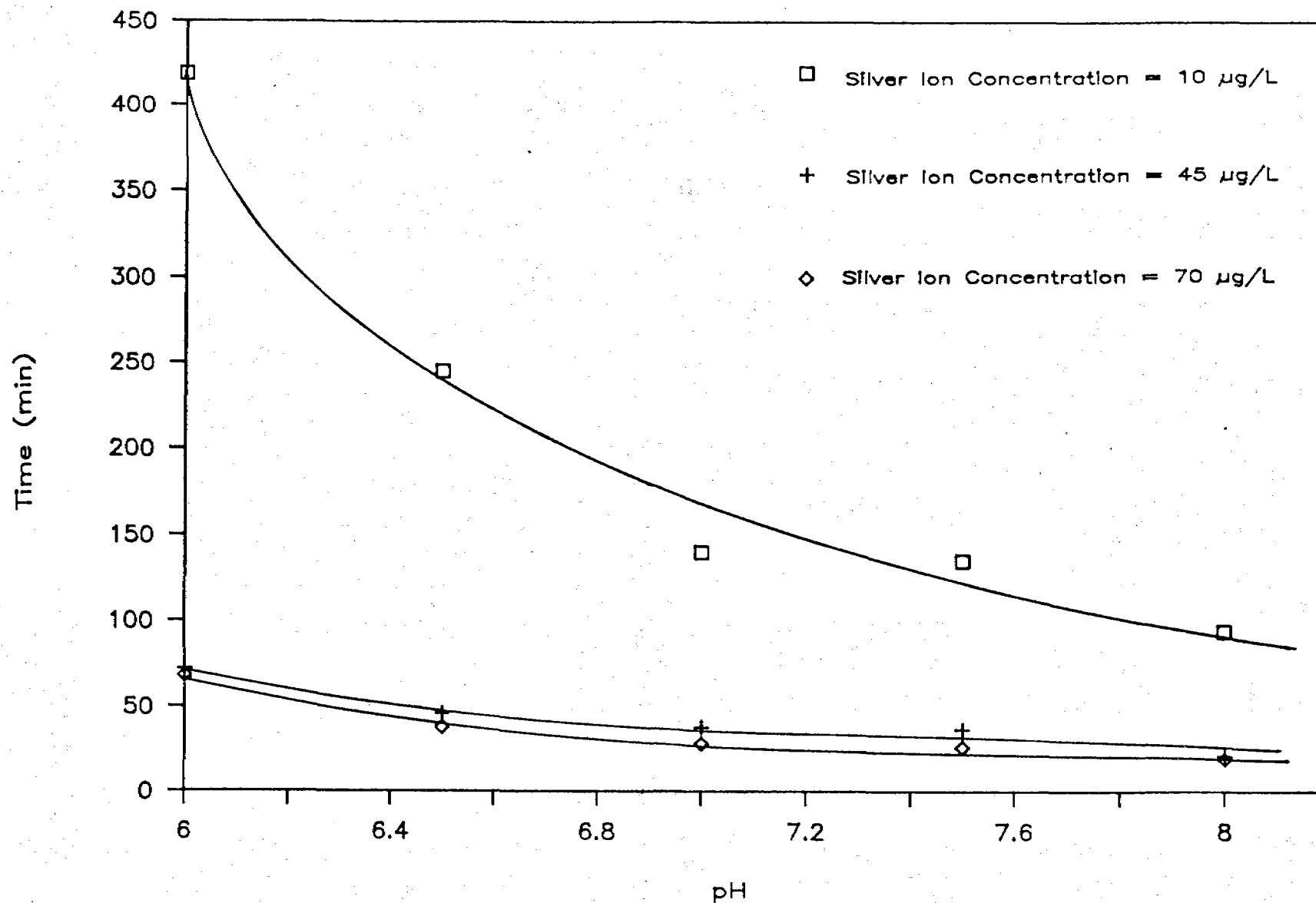


Fig. 3 Effect of pH on t_{99} for E. Coli

WATER SUPPLY PROGRAMME IN UTTAR PRADESH (INDIA) - CENTENNIAL PANORAMA (1890 - 1990)

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ABSTRACT : The history of water supply service in Uttar Pradesh (U P) is now just a century old, when at Agra - the TAJ MAHAL city, the first water-works was inaugurated in 1890. Till 1947 only 27 major towns could be benefited by piped water supply service. On the recommendations of the National Water Supply & Sanitation Commission (1954), water supply sector was included in the First Five Year Plan (1955-60). Though the allocation was kept very meagre, but, the program gained some momentum and by the end of the Plan the total number of towns covered under water supply rose to 73. The rural area could only be included for the first time in Second Five Year Plan (1960-65), during which only 37 villages could be provided with piped water supply.

Gradually in subsequent Plans, the water supply was extended to more areas, but the pace was too slow and till March 1980 only 10056 (9 %) villages and 409 (63.5 %) towns/cities could be covered. To cover the remaining areas an expenditure of Rs 18830 million was estimated on 1980 price index, which was not possible for U P Govt to spare. Year 1981-90 was declared as Water Supply & Sanitation Decade under the aegis of U N World Water Conference (Argentina, 1977), simultaneously in Year 1985-86 Govt of India launched 'Water Technology Mission', in which it was committed that by the end of 1990 'Safe Water' shall be provided to 100 % population.

The water demand in rural areas are found to be quite low due to the prevailing Socio-Economic conditions. This fact led to an idea that the installation of an sturdy hand pump viz India Mark II Deep Bore Hand Pumps in each village may solve the problem appropriately. Therefore, the orientation of the program was altered and instead of executing Piped Water Supply Schemes, it was decided to install IM II HP @ 250-300 users. This alternate provision contained many additional benefits besides curtailing the expenditure & execution time drastically, the system has simple design, easy installation, swift operation, less maintenance and above them all it can be easily repaired locally by the users themselves. This program is considered equally feasible for small towns/cities also.

To achieve the target, it was felt necessary to associate various National & International Agencies including World Bank to overcome the financial burden as well as to obtain supporting

infrastructure in the form of equipments and expertise. Now almost every village & town of U P is having IM II HP besides piped water supply.

A panoramic review of the water supply progress in U P along with the methodologies applied in implementation of it and achievements gained from the year 1890 to 1990 are detailed in this paper systematically.

INTRODUCTION :

The general population normally has no freedom of choice regarding drinking water. Therefore, an assurance that the water available for drinking is of the highest possible quality is necessary. The concept of what is acceptable changes with place and time. The final decision or choice of what is drinking water and what is not depends on the prevailing environment & economical situations. WHO estimate indicates that 80 % of sickness and diseases in the world are associated with water.

Since Independence, food production has increased manifold in India, this is a paradox, as per the statistics more people might be drinking contaminated water than they did before. Because it was not possible to provide safe water even to the increased population leaving aside the backlog. Yet India has been emphasising the use of uncontaminated water since time immemorial. It is mentioned in the "Oisruta Sanghita" a medical book in sanskrit of 2000 BC that "... it is good to keep water in copper vessel, to expose it to sunlight and filter through charcoal ...". Mahatma Gandhi in his article "Key to Health" has emphasised the importance of pure water for drinking and advised that none should take water from a well the quality of which is not known. Gurudev Rabindra Nath Tagore writing under the title "Crisis of Civilization" wondered how could a country be proud of its civilization when even safe water for drinking could not be provided to the people. In villages, the traditional sources of water for the poor lot are rivers, streams, lakes, ponds, gadheras, shallow dug wells etc and for those who can afford are ordinary hand pumps, tube wells etc.

Millions of open dug wells in U P are by and large grossly polluted. Successive investigations carried out by various Research Institutes have conclusively proved that there are hardly any public dug well, the water of which could be considered bacteriologically safe as per the prescribed standard. Yet these wells are the main source of water in mostly all the villages of U P . It was not an easy task to provide safe water to the entire population of U P in a short period, while till late forties, progress of water supply even in the urban areas was slow and spasmodic. But the concerted efforts of U P Govt, Govt of India with the assistance of National & International Organizations has now inverted the statistics, and at present almost each village besides towns/cities of U P has been provided with atleast one source of safe water. The programs are

not yet completed and within a few years the last person will be catered with SAFE DRINKING WATER and that is too at his/her doorstep.

AMEN

UTTAR PRADESH (U P) : Uttar Pradesh, which means in Hindi language as Northern Province or State, the name is self explanatory that it is located in the northern region of India. China & Nepal share its North-Eastern border where as Himachal Pradesh, Haryana, Bihar, Rajasthan, Madhya Pradesh States surrounds it from inside of India. Population-wise it is the largest State, and area-wise it stands fourth in India having land constituting 9% coverage of whole Nation. U P is divided into three distinctive geographical regions viz (I) the Northern area is a part of Himalayan Mountain range, below it there is marshy Tarai area, (II) the Southern Plateau & (III) the Gangetic plains. For administrative purposes U P has been sub-divided into zones, commissioners, districts, cities/towns, tehsils, blocks, villages etc. Ganges, the longest river of India originates from here and besides this many other important rivers flow through U P. The weather of U P variates from very cold in winter to very hot in summer. There is heavy rainfall between July & September with some scattered rain in winter too. The dense evergreen forests of pine and firs encircles the north, the soil in the plains is very fertile and good for agriculture which is the main occupation in U P. Besides many dialects, the 'franca lingua' are Hindi & Urdu. Lucknow - the world reknown city for its Nawabi culture and historical monuments is the capital of U P, Agra - proud city of TAJ MAHAL, Allahabad - Divine 'Amrit' sprinkled city of SANGAM, personally adored by Akbar The Great, Nainital/Mussoorie - hill resorts etc to name a few, because every city/town of U P posses its own tales of glorious past and expedious present.

WATER SUPPLY ACTIVITIES IN U P

Piped water supply (PWS) was first introduced in the State in the year 1890 in five major towns namely Kanpur, Agra, Varanasi, Allahabad & Lucknow, collectively called as KAVAL Towns. Rather with slow progress, till 1947 only 27 towns could be covered with PWS, but the rural area remained practically untouched in this respect. An awareness of the need to ensure adequate and wholesome water supply had prompted Govt of India (GOI) to appoint a committee known as 'Bhore Committee' in 1944. The Committee made a number of recommendations and suggested that the target should be to provide safe water supply for drinking purposes to the entire rural and urban population within a period of 35 years. This was followed by the Environmental Hygiene Committee in 1949, which recommended the comprehensive provision of water supply and sanitation facilities on a priority basis to 90 % of population within a period of 40 years, it also suggested the formation of a Public Health Engineering Services. The National Water Supply & Sanitation Committee assesses the magnitude of the existing water supply problem in the country and suggested significant procedural, organizational and financial reforms. On the recommendations of the committee the National Water Supply & Sanitation Program (NWSSP) was launched by GOI in year 1954, and for the first time a meagre sum was allocated for rural water supply programs (RWSP) alongwith

urban water supply programs (UWSP). The Drinking Water Board was set up in 1963 and which ceased to function after submitting its report, recommended that the highest priority be given to the provision of water supply in difficult and scarcity areas and emphasised that the completion of this activity within a maximum period of 5 years.

In 1962, the Union Ministry of Health, GOI undertook an assessment of the rural water supply problems to have the right perspective for launching a program to meet the needs of the Problem/Scarcity areas on a priority basis. The survey was completed in 1972. The identification of Problem/Scarcity Villages (PV) was based on the following criteria :

- A. Where source of water is 1.6 Km away from the village in plain areas or 0.8 Km in case of hilly areas.
- B. Where source is 100 M up or down from the village in hilly areas or water table is 20 M below the ground level in plains.
- C. Where the source of water supply are susceptible to water borne diseases like Cholera, Typhoid, Guinea-worm infestation etc.
- D. Where water sources contain excessive salinity, Iron/Fluoride contents beyond permissive limit etc.

The above survey identified a total of 35,506 villages as PV out of a total of 1,12,561 villages. Therefore, it may be noted that comprehensive planning and large scale financial allocations were necessary for the vigorous implementation of RWSP. The development under various program did not show particular progress because of financial limitations. It was observed that only 1,437 villages and 144 cities/towns were provided with piped water supply (PWS) till 1969. The emphasis on the rural water supply underwent a radical change with the commencement of the Fourth Five Year Plan (1969-74). On the basis of the survey report of 1972, rural water supply was taken to be one of the major component of the Minimum Need Program (MNP) in the Fifth Five Year Plan (1974-79). During this Plan attempts were made to provide Safe Water Supply to the PV identified in 1972. The allocations made under various Plans and coverage till 1980 for water supply in U P is shown in Table I . The table reveals that the investments for RWSP started in a big way from the Fourth Five Year Plan onwards. Prior to that the investments, specially that during first three plan periods were negligible. Subsequently greater emphasis was given to RWSP and its portion in the allocation increased from a mere 0.15% in First Five Year Plan to 64.18% in Fourth Five Year Plan.

The non-availability of required finance in lump-sum, a policy was outlined to adopt austere standards, by providing the cheapest mode of water supply to meet the minimum needs to the optimum population in the difficult villages and emphasis was laid on tapping local ground water sources, piped water supply schemes were to be adopted only if a less expensive system such as hand-pump supply was not feasible. The GOI launched an Accelerated Rural Water Supply Program (ARWSP) as a Centrally sponsored program during 1977-78 with a view to cover all the PV by 1981-82. This program envisages the sanctioning of 100% Grant-in-Aid to the State for providing

**Table I Allocations For Water Supply Under Various Plans
And Achievements During the Plan Period**

<u>Sl No</u>	<u>Developmental Plans</u>	<u>Total Allocation for U.P.</u>	<u>Coverage till Plan</u>				
		<u>Alloc</u>	<u>Total</u>	<u>Urban</u>	<u>Rural</u>	<u>Villages</u>	<u>Cities/Town</u>
01	First Five Year Plan	110.0	66.6	66.5	0.1	nil	73
02	Second Five Year Plan	1140.0	50.4	48.9	1.5	37	1
03	Third Five Year Plan	1160.0	116.2	95.8	20.4	127	58
04	Annual Plan (66-67, 67-68, & 68-69)	1060.0	105.9	44.7	61.2	1273	12
05	Fourth Five Year Plan	5740.0	260.8	94.5	166.3	2387	44
06	Fifth Five Year Plan	7500.0	632.1	325.3	306.8	2846	188
07	Annual Plans (78-79 & 79-80)	N.A.	805.4	288.5	516.9	3386	33
		TOTAL	2037.4	964.2	1073.2	10056	409

Note : Allocations are in Million Rupees, N.A. = Figure Not Available, Coverage of cities in First Plan includes number of cities covered from 1890 to 1956.

**Table II Status of Water Supply Program Achievements At
A Glance As On 1.4.91**

RURAL WATER SUPPLY

Number of Villages	:	112566	(97522-Plain	15044-Hill
Total number of Problem Villages	:	78050	(66408	" 11642 ")
Benefited Problem Villages	:	77477	(66408	" 11069 ")
Benefited Non-Problem Villages	:	23751	(20784	" 2967 ")
Benefited Total Villages	:	101228	(86893	" 14335 ")
Total IM-II-HP Installed	:	282700	(282687	" 13 ")

Target For 1991-92

Water Supply to Problem Villages	:	205	(nil	205 ")
Water Supply to Non-Problem Vill	:	2940	(2935	" 5 ")
Installation of IM-II-HP	:	23227	(23227	" nil)

URBAN WATER SUPPLY :

<u>Total Cities/Towns</u>	<u>Category I</u>	<u>II</u>	<u>III</u>	<u>IV</u>	<u>V</u>	<u>VI</u>	<u>Total</u>
Number	30	37	89	192	229	112	689
Benefited	30	37	89	178	196	83	613
Balance	00	00	00	14	33	29	76

Category : I = Population above 100,000 ; II = Between 50,000 to 99999 ; III = Between 20,000 to 49,999 ; IV = Between 10,000 to 19,999 ; V = Between 5,000 to 9,999 & VI = Less than 5,000.

water to PV. Till 1980 only 10,056 villages (7,001 PV & 3,055 Non-PV) out of total 1,12,566 villages ie about 9% could be covered under different water supply programs. The table also shows that besides 91% villages about 38% of urban population was also yet to be provided with safe and adequate potable water. Therefore, it was resolved to cover all the remaining areas with PWS during the coming decade. The cost estimation to complete this task gave a staggering figure of Rs 18830 million at the 1980 price index. This work was supposed to be carried out in phased manner spread in a 10 year span provided that the sufficient finance could be arranged in the meantime.

But the biggest question before the U P Govt was that from where to raise this much of finance ? Besides the money, it was also not an easy job to arrange required paraphernalia in bulk quantity. Therefore, till 1982 very little could be achieved in this sector. To solve this, it was considered appropriate to explore the possibility of obtaining assistance from International Agencies viz World Bank, UNICEF, foreign Govts etc to accelerate the pace of this program. Negotiations in this respect were first started with World Bank and the Bank came forward initially with an offer of US \$ 40 million.

The World Bank financial assistance gave a boost to water supply activity in U P. Simultaneously GOI and other Financial Institutions also extended their support in this great job and many water supply schemes under various sponsored projects were started, some of them are listed as;

- A. Minimum Need Program (MNP) - Sponsored by U P Govt.
- B. Accelerated Rural Water Supply Program (ARWSP) - Sponsored by Govt of India.
- C. Rural Development Department Program - U P Govt Program.
- D. Water Supply & Sanitation Decade (1981-90) - The 31st U N General Assembly endorsed the recommendations of U N World Water Conference (Argentina, 1977), India being a participant country declared 1981-90 as Water Supply & Sanitation Decade. In this Decade it was decided to provide safe water to 100% population (rural & urban both) till the end of 1990.
- E. Water Technology Mission - Sponsored by Govt of India to counter the drinking water problem in rural areas specially where qualitatively water is not fit for drinking purpose on a war footing. GOI has set up the Technology Mission on Drinking Water in villages & Related Water Management with the Department of Rural Development (GOI) as the nodal agency. The objective of the Water Technology Mission is to cover residual acute problematic villages by the end of 1990.

With the passage of time, massive industrial developments, environmental/ecological deterioration, improved socio-economic conditions increased the water demand with the result that more villages came under the PV category. On the instructions of GOI to all District Authorities of U P , 42544 villages were additionally identified as PV in 1985 based on the same criteria as was followed in 1972 survey report with slight modification. Thus, now the total number of PV increased from 35,506 in 1972 to 78,050 in 1985.

Observing the pace of work, increased number of PV, financial burden and with a view to provide at least one source of safe drinking water in each village the authorities were under pressure to find out an alternative economical approach. The water demand in the rural areas is found to be quite low as compared with the recommended demand due to the prevailing poor socio-economic condition of the general mass, and this fact led to an idea that if a good handpump be installed in each village then this may solve the problem appropriately. Therefore, it was decided to install a sturdy community hand-pump @ one hand-pump for every 250-300 persons with a maximum of two hand-pumps in each village initially, out of these one hand-pump was to be installed in Harijan and/or Scheduled Caste/Tribe settlement of the village. This socio-economic condition related orientation in the policy yielded bilateral benefits, on one hand it drastically reduced the water supply project cost which could be utilized in covering more & more villages and on the other hand it extremely cut-shortened the execution/completion time of the project from years to weeks.

The selection of a suitable deep well hand-pump was not an easy job in the beginning of the eighties, because at that time there was no any economical deep well hand-pump of proven performance track was available. Some of them were Vergnet (France), Mono/Climax/Godwin/Consalieu (all England), ABI (Ivory Coast), GSW (Canada), Jalna/Jalwad/Sholapur (all India) etc. Actually the deep well hand-pump program was taken up in a small way by many States of India other than U P in 1966-67. The UNICEF also evinced interest in this program by supplying the inputs viz hand-pumps, drilling rigs, support vehicles, spares etc. However, the expectations were shortlived as it was found that 75% of the pumps were remains out of order at any point of time. The huge rate of failure dampened the enthusiasm of the executing agencies. They all agreed that there is no use of providing defective pumps as these will be counter-productive. Because the high rate of failure made the hand-pumps a very unreliable source of water in the eyes of the users and they preferred to switch back to their usual conventional source of water. As per the statistics available the pumps in those days were working for not more than 60 days a year.

Under the compelling circumstances, it was felt necessary to develop indigenously an economical & improved deep well hand-pump, and a program for rejuvenation of the defunct scheme. To initiate it GOI, WHO & UNICEF organised a workshop on hand-pump, senior representatives of M/S Richardson & Cruddas (1972) Ltd (a GOI enterprise) were special invitee to the workshop. After the workshop M/S Richardson & Cruddas (1972) Ltd in association with Mechanical Engineering Research & Development Organization (MERADO) and UNICEF started working on the development of a new pump based on SHOLAPUR HAND-PUMP. This was the beginning of the world famous INDIA MARK II HAND-PUMP (IM II HP). The group within a record period of 18 months accomplished the assignment and we got IM II HP to convert impossible task into a possible one.

This potent tool ie IM II HP took over the entire rural water supply projects except those of hill region and in regions where

Table III. Details of Water Supply Position in Uttar Pradesh

Sl No	District	Area Sq Km	Population			Villages			City	Vill Covered by			No of IM-II-HP	
			Total	R%	U%	Total	PV	NonPV		PWS	IMHP	Total	Reqd.	Installed
01	Uttarkashi	8016	190948	93	07	669	429	240	02	422	-	422	-	-
02	Chamoli	9125	364346	92	08	1516	1152	364	07	1052	-	1052	-	-
03	Tehri-Garhwal	4421	497710	96	04	1953	1609	344	05	1312	-	1312	-	-
04	Dehradun	3088	761668	51	49	743	595	148	04	540	-	540	-	-
05	Garhwal	5440	637877	90	10	3237	2242	995	05	1607	-	1607	-	-
06	Pithoragarh	8856	489267	95	05	2174	1569	605	05	1362	-	1362	-	-
07	Almora	5385	757373	94	06	3019	2398	621	04	2146	-	2146	-	-
08	Nainital	6794	1136523	73	27	1806	1648	158	21	1624	3	1627	13	13
09	Saharanpur	5595	2673561	73	27	1700	191	1509	17	122	66	188	9747	2350
10	Muzaffarnagar	4176	2274487	78	22	927	61	866	16	1	59	60	8902	2029
11	Bijnor	4848	1939261	69	31	2154	661	1493	17	32	564	596	7292	2137
12	Meerut	3911	2767246	66	34	920	200	720	16	37	149	186	9916	2612
13	Ghaziabad	2590	1843130	81	19	704	271	433	13	12	239	251	6070	2039
14	Bulandshahar	4352	2358270	75	25	1365	1052	313	13	1	1034	1035	9511	3710
15	Moradabad	5967	3149406	73	27	2473	1345	1128	18	22	1315	1337	11504	4845
16	Rampur	2367	1178621	73	27	1092	588	504	08	5	583	588	4317	1935
17	Badaun	5168	1971946	84	16	1785	1326	459	20	127	1192	1319	8269	3827
18	Bareilly	4120	2273030	71	29	1901	920	981	48	51	837	888	8070	3878
19	Pilibhit	3499	1008312	84	16	1198	1193	05	06	65	1118	1183	4224	2428
20	Shahjahanpur	4575	1647664	81	19	2124	1887	237	09	42	1690	1732	6642	4708
21	Aligarh	5013	2574925	77	23	1704	1288	416	19	70	1131	1201	9914	4720
22	Mathura	3811	1560446	79	21	867	867	0	18	487	367	854	6159	3240
23	Agra	4805	2852942	62	38	1174	948	226	14	406	508	914	8830	7968
24	Etah	4446	1858692	85	15	1510	730	780	18	34	635	669	7853	3052
25	Mainpuri	4343	1726202	89	11	1371	1352	19	09	25	1314	1339	7674	4454
26	Farrukhabad	4274	1949137	84	16	1577	957	620	11	70	854	924	8171	5485
27	Itawah	4326	1742651	85	15	1462	1057	405	12	4	1049	1053	7425	9409
28	Kanpur	6176	3742223	54	46	1885	512	1373	12	58	417	475	10044	5187
29	Fatehpur	4152	1572421	91	09	1349	1272	77	06	47	1009	1056	7155	5873
30	Allahabad	7261	3797033	80	20	3514	2822	692	16	1085	1495	2580	15117	9524
31	Jalaun	4565	986238	80	20	939	769	170	10	279	484	763	3949	3903
32	Jhansi	5024	1137031	62	38	759	752	7	11	175	574	749	3528	3680
33	Lalitpur	5039	577648	87	13	683	497	186	04	69	428	497	2503	2769

Continued...

Table continued

Sl No	District	Area Sq Km	Population		Villages			City	Vill Covered by			No of IM-II-HP	
			Total	R% U%	Total	PV	NonPV		PWS	IMHP	Total	Reqd.	Installed
34	Hamirpur	7165	1194168	83 17	917	668	249	13	136	530	666	4979	4285
35	Banda	7624	1533990	88 12	1207	1001	206	11	450	546	996	6764	6257
36	Kheri	7680	1952680	90 10	1699	1699	0	09	9	1687	1696	8826	8370
37	Sitapur	5743	2337284	90 10	2330	1408	922	10	24	1317	1341	10484	6612
38	Hardoi	5986	2274929	89 11	1881	835	1046	12	63	770	833	10117	4726
39	Unnao	4558	1822591	88 12	1687	731	956	17	31	628	659	8031	4965
40	Lucknow	2528	2014574	47 53	899	31	868	09	0	31	31	4774	1535
41	Rae-bareilly	4609	1886940	93 07	1731	1569	162	09	591	954	1545	8739	6329
42	Bahraich	6877	2216245	89 11	1884	2032	852	05	142	1875	2017	10300	9915
43	Gonda	7352	2834562	97 03	2809	2647	162	10	17	2150	2167	13135	9092
44	Barabanki	4401	1992074	93 07	2043	2031	12	09	1	1776	1777	9070	5680
45	Faizabad	4511	2382515	93 07	2645	1285	1360	06	3	1271	1274	10606	6089
46	Sultanpur	4436	2042778	91 09	2492	2370	122	06	123	2183	2306	9877	11913
47	Pratapgarh	3717	1801049	95 05	2185	632	1553	07	47	555	602	8551	2302
48	Basti	7228	3578069	95 05	6929	4767	2162	07	408	4320	4728	17030	16273
49	Gorakhpur	6272	3795701	89 11	4110	2406	1704	09	142	2251	2393	16969	12488
50	Deoria	5445	3496564	93 07	3538	2204	1334	16	170	2033	2203	16323	10090
51	Azamgarh	5740	3544130	91 09	4935	4718	217	19	60	3841	3901	16657	10251
52	Jaunpur	4038	2532734	93 07	3245	1588	1657	07	89	1098	1187	11819	5948
53	Ballia	3189	1945376	91 09	1920	1118	802	09	65	1053	1118	8842	7959
54	Ghazipur	3377	1944669	92 08	2540	1386	1154	09	357	1028	1385	8952	5146
55	Varanasi	5091	3701006	73 27	3662	2806	856	16	882	1830	2712	13531	10146
56	Mirzapur	11310	2039149	87 13	3024	2893	131	11	931	1918	2849	8857	10553

NOTE : The population of each districts are as per the Census Report of India 1981 . The remaining datas are given as per the position as on 1.4.91.

Abbreviation used : R% = Percentage of Rural Population ; U% = Percentage of Urban Population
 PV = Problem/Scarcity Villages ; NonPV = Non Problem/Scarcity Villages ; PWS = Piped Water Supply ; IMHP & IM-II-HP = India Mark II Hand Pumps ; Reqd. = Total number of hand-pumps required for coverage of the rural population by water supply as per the Census of 1981 @ 250 users per hand-pump.; Installed = Total number of India-Mark-II-Hand Pumps installed as per the compilation as on 1.4.91.

the quality of water is not found satisfactory due to the presence of excessive chemicals like Fluoride, Iron etc or bacteriologically/biologically contaminated/infested, because piped water supply is the only remedy for these regions. The hand-pump based rural water supply program took a great leap in progress and in less than ten years almost entire State has been covered and remaining portion is scheduled to be covered within a few years

CONCLUSION :

Technically speaking the commitment of the Govt to provide safe water to the rural as well as to the urban population of U P has been fulfilled to a greater extent. Due to the change in methodology the rate of progress increased manifold. It can be seen that till 1980 only 10,056 villages and 409 cities/towns were benefited by water supply programs which constitutes a mere 9% of total villages and 63.5% of total cities/towns. But between 1981-90 the number of villages covered under water supply programs are 90,172 (excluding those villages covered till 1980) and number of cities/towns benefited by WSP rose from 409 to 689. Thus on an average the rate of progress of water supply programs can be easily calculated as covering 9,017 villages and 20 cities/towns per year. Now the remaining coverage of villages & cities/towns left to the tune of 10% & 11% respectively, which may be easily covered within a span of a year or two from now. The major role played by IM II HP in rapid execution of rural water supply projects is equally commendable besides the zeal of committed personnels.

ACKNOWLEDGEMENT

Author is sincerely thankful to the U P Jal Nigam Administration for the encouragement and assistance in preparing this status paper.

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