# Appendix 5 Present Status of Water Supply

# 1 Present Status of Water Supply in the Project Area

## 1.1 Present Status of the Existing Facilities

### 1.1.1 Outline

Water supply facility of Hai Duong City, which consisted of Cam Thuong system with source of surface water, has been in use since 1930s. Present production capacity of this facility is estimated approximately 14,000 $m^3$/day. In view of the loss in the treatment plant and the leakage at distribution pipes, the effective supply capacity may be as small as approximately 9,000 $m^3$/day. The City's water demand is increasing gradually due to a rapid urbanization in recent years. Water demand in 1996 and 2000 were estimated approximately 30,000 and 37,000 $m^3$/day respectively.

In order to improve the lack of water supply capacity, extension and rehabilitation of the facility were implemented. However, those were not able to meet the increase of water demand. Then, an additional water treatment facility was constructed in 1991 for supplying a part of Hai Tanh area, southern part of the city, that takes groundwater.

Present water supply capacity still can not meet the water demand of whole city. Water supply areas are obliged to be divided into two areas so that each area is supplied water every two days. Nevertheless, distributed water capacity and water pressure seem still poor. As such, in some parts of the service area, temporal water supply is now conducted by using booster pumps.

### 1.1.2 Water Supply Facility of Groundwater Source System

(1) Present status of the existing facility

For the purpose of water supply to a part of the southern area of Hai Duong, Hai Hung construction service constructed a water supply system in 1992 that is composed of two deep wells and Hai Tanh water treatment plant. The construction was funded by the governmental budget.

A. Intake facility

At present, two production wells exist along the lakeside in the south of the city. The depth of the wells are about 30 m and the diameter of it is 150 mm. Operation hours are 10 – 15 hours/day. Every well is kept in a pump house.

B. Hai Tanh Treatment Plant

As raw water contains high iron concentration, the iron in raw water is removed at the filter bed. Water is disinfected by chlorine. Treated water is contained in the clean water reservoir, the capacity of which is 300 m3, and distributed by distribution pumps to the service area. The distribution capacity is 750 m3/day. Operation hours are : 5:30 – 7:00, 9:00 – 12:00 and 16:00 – 19:00 ; in total 7.5 hours per day.

For iron removal, aeration tower has equipped. Raw water is sprayed through the perforated pipes and three intermittent floors and transmitted to the following contact basin. Since oxidized iron easily sticks to the intermittent floors, the floors are replaced every three or four months. Thickness of filtration of the sand filter is approx. 1m. For washing the sand filter, back-washing using the clean water in the reservoir is carried out for 50 – 60 minutes every three days.

(2) Operation and Maintenance (O&M)

O&M works are conducted by the Hai Duong Water Supply Company (HDWSC) under Hai Duong province construction service. Number of personnel of the Hai Tanh treatment plant for O&M works are nine. Among that, each two persons, in total four persons, are assigned for operation of the intake facilities. Maintenance works including small repairing are conducted by the O&M staffs. As for large scale repair works, they are supported by the O&M staffs of Cam Thuong treatment plant which has 49 O&M staffs. Ordinary repair works has been conducted by the own O&M staffs without a private firms.

### 1.1.3 Water Supply Facility of Surface Water Intake System

#### (1) Intake Facility

| | |
|---|---|
| Specifications | Q = 500 $m^3$/hr x 3 units (including standby pump 1 unit) |
| Year of Installation | 1978 |
| Conditions | Superannuated |

#### (2) Transmission Pipeline

| | |
|---|---|
| Diameter | 500mm |

#### (3) Treatment Facility

1) Receiving Tank

| | |
|---|---|
| Detention period | little |
| Rapid mixing | mixing in pipeline |
| Aluminum sulfate dosage | Max. 20 ppm/min<br>Ave. 10 ppm/min |

2) Sedimentation Basin

| | Basin A | Basin B | Basin C |
|---|---|---|---|
| Year of construction | 1936 | 1963 | 1963 |
| Capacity | 1,000 $m^3$ | 5,000 $m^3$ | 15,000 $m^3$ |
| Number of basins | 1 basin | 1 basin | 4 basins |
| Type | Sludge Blanket type | | |
| Conditions | Operated effectively, floc are sometime over-carried. | | |

3) Rapid sand filter basin

| | Filter A | Filter B |
|---|---|---|
| Year of construction | 1963 | 1978 |
| Dimensions | 3.7 m x 3.4 m = 12.58 $m^2$ | 4.8 m x 4.3 m = 20.64 $m^2$ |
| Number of filter basins | 3 basins | 5 basins |
| Treatment Capacity | 5,600 $m^3$/day(3 basins, 150 m/d) | 15,400 $m^3$/day(5basins, 150 m/d) |
| Backwash Frequency | | Once a day, Air scouring - 5 minutes, Water - 12 minutes |
| Backwash type | | Air-water combined |
| Specifications of the backwash pumps | Backwash pump: 1,200$m^3$/hr x 18m x 1 unit<br>Backwash blower: 1,200$m^3$/hr x 0.3kg/$m^2$ x 18.5kW x 1unit | |
| Conditions | Backwash pump, blower and valves were replaced under the OECF commodity loan. Problems caused by mal- function of backwash were solved, and the filtration is now working at the near-design capacity. | |

4) Clean Water Reservoir

| | Reservoir A | Reservoir B | Reservoir C |
|---|---|---|---|
| Year of construction | 1936 | 1963 | 1978 |
| Design Capacity | 200 $m^3$ | 300 $m^3$ | 1,050 $m^3$ (Called 1,500 $m^3$) |
| Type | Round type, Reinforced Concrete | | |
| Dimensions | Inner Diameter : 23.4m<br>Height : 2.45 m | | |
| Capacity | 1,050 $m^3$ + ( 200 $m^3$ + 300 $m^3$ ) = 1,550 $m^3$ | | |
| Detention Period | 1,500 $m^3$ / 11.81 $m^3$/min = 1.27 hr | | |
| Conditions | Insufficient detention period of 1.27 hours, where at least 4.0 hours of the treatment capacity is required for reservoir. | | |

5) Distribution Pumps

| | |
|---|---|
| Specifications | Q = 540 $m^3$/hr x 60 m x 3 units (including standby 1 unit )<br>Replaced under the OECF loan in 1997. |
| Operation Hours | One pump for 24 hours continuous operation<br>One pump for 6 hours operation (morning / daytime / evening ; 2 hours each operation) |
| Conditions | Distribution capacity of 15,000 m3/day is calculated from the operation hours. However, taking into account of the superannuated pumps and damaged water meters, actual capacity is estimated to be much lower. |

6) Distribution Water Meter

| | |
|---|---|
| Type | Orifice meter |
| Number of meters | 2 units (including one broken-down meter) |
| Conditions | Determined from the fact that the meter indicated 245 m3/hr even when no water was distributed, both meters are regarded as out of order. |

7) Chemical Dosing Equipment

| | |
|---|---|
| Chemical used | Liquid Chlorine |
| Container | 60 kg and 400 kg containers |
| Injection point | Clean water reservoir |
| Dosage | 0.6 - 0.7 ppm/min |
| Conditions | This equipment was replaced under the OECF commodity loan in 1997. |

Considerations

Rapid sand filter (Filter A: 5,600 $m^3$/day and Filter B: 15,400 $m^3$/day) was not properly working. However, filtration capacity was recovered nearly to the design capacity, since the facility was replaced under the OECF commodity loan.

(4) Distribution / Water Supply Facility

1) Current Status of Water Supply

In Hai Duong, there are two water treatment plants. One is Cam Thuong water treatment plant which takes surface water from Thai Binh river. The other is Hai Tanh water treatment plant which takes groundwater. Their design capacity are 20,000 $m^3$/day and 750 $m^3$/day respectively. Service area is ward I to VI and some parts of ward VIII to X of Hai Duong. Population served is estimated to be approx. 64,000.

Since Cam Thuong water treatment plant was rehabilitated under the OECF commodity loan in 1997, its treatment capacity (daily maximum) at present is estimated approximately 14,000 $m^3$/day. In addition, it is reported that the 35 % leakage exists in water distribution and water supply facilities. Hence, actual water supply capacity is estimated at about 9,000 $m^3$/day.

In order to overcome the situation that water supply capacity is unable to meet the water demand, the service area is divided into two areas so that each area is supplied water every two days. Nevertheless, distributed water capacity and water pressure seem still poor. As such, in some parts of the service area, temporal water supply is now conducted by using booster pumps.

2) Distribution Pipelines (Type / Diameter / Length / Year of installation)

Total length of the distribution pipelines more than Dia. 75 mm is approx. 30 km. Type of distribution pipes is ductile iron pipe. In the case that diameter is less than 600 mm, Vietnamese pipes and valves are installed. Pressure resistance of pipes is 6 kgf/$cm^2$.

As for pipes installed in the beginning of water supply system in 1936, the exact locations are unknown. And valves, most of them are deteriorated, are avoided to be open/close.

Table 1-1 Existing Pipelines by Diameter / Installation period

(Unit : m)

| Install Dia. | 1936 - 1975 | 1976 - 1985 | 1986 - 1995 | Total |
|---|---|---|---|---|
| φ600 | | 860 | | 860 |
| φ400 | | 930 | | 930 |
| φ300 | 2,100 | 3,090 | | 5,190 |
| φ250 | | 230 | 230 | 460 |
| φ200 | 880 | 1,320 | 1,350 | 3,350 |
| φ150 | 2,560 | 5,670 | 1,860 | 10,090 |
| φ100 | 700 | 1,860 | 3,360 | 5,920 |
| φ75 | 1,620 | | 900 | 2,520 |
| Total | 7,860 | 13,960 | 7,700 | 29,520 |

3) Water Supply Equipment

Number of connections is 12,300 for domestic use and 210 for offices and public utilities. Among them, about a half number of water meters are installed. Most connections are branched from less than Dia. 200 mm pipes by corporation stop with saddle. Some connections are branched from Dia. 300 mm pipes.

Water supply equipment for domestic use consists of steel pipe (Dia. 20 mm), stop valve, water meter (13 mm), box for water meter and indoor piping. At present, no water supply equipment can be seen for higher than 2nd floor. Every house is equipped with concrete water reservoir (capacity : 100 – 200 liters) on/under the floor. In addition, some houses are equipped with the booster pumps and elevated tanks on the roof. To reserve water inside house is regarded not only as self-preparation for emergency but also as old custom.

4) Internal Water Pressure of Pipeline

The results of water pressure survey for existing distribution pipeline is tabulated in Table 1-2. Only at the point near the water treatment plant, more than 2 $kgf/cm^2$ is measured. But at most points, water pressure is poor.

Table 1-2 Result of water pressure survey (August 31, 1996)

| Measuring point | Measuring Time | Water Pressure (kgf/cm2) | Remarks |
|---|---|---|---|
| 1 | 9:00 | >2.0 | |
| 2 | 9:10 | 1.15 | |
| 3 | 9:30 | 0.0 | Water shutdown |
| 4 | 9:25 | 0.3 | Public tap |
| 5 | 9:40 | 0.1 | Public tap |
| 6 | 10:10 | 0.15 | |
| 7 | 10:10 | 0.25 | Public tap |
| 8 | 10:20 | ≒0.0 | Poor water supply |
| 9 | 10:30 | <0.1 | |
| 10 | 10:35 | <0.1 | |
| 11 | 10:50 | 0.45 | |
| 12 | 11:00 | 0.2 | |
| 13 | 11:05 | 0.3 | |
| 14 | 11:10 | 0.35 | |
| 15 | 11:20 | 0.55 | |
| 16 | 11:30 | <0.1 | Poor water supply |

5) Fire Hydrants

No fire hydrants are installed in the existing system.

6) Necessity to improve the existing water distribution/supply facilities

Current status of water distribution and water supply are summarized as Table 1-3. As most people suffer from poor water quantity/pressure, any countermeasures are required. Water supply conditions seem to be serious especially in Ward I of urban, Ward VI and Ward VIII of semi-urban. Urgent necessity for improvement is high compared to the other.

Table 1-3 Necessity for improvement

| Ward | Outline of the area | Present conditions of water supply | Priority |
|---|---|---|---|
| I | Urban area<br>Governmental offices<br>Population density : 237 persons/ha | Population served : approx. 90%<br>Water supply : every two days<br>Service conditions : permanently poor water quantity/pressure | Priority 1st<br><br>Improvement of this area contributes to water supply to ward VIII |
| II | Urban area<br>Ho Lake occupies 30% area<br>Population density : 158 persons/ha | Population served : approx. 90% (exclude Ho Lake)<br>Water supply : every two days<br>Service conditions : extremely poor water supply | Priority 2nd |
| III | Urban area<br>Center of Hai Duong<br>Population density : 215 persons/ha | Population served : 100%<br>Water supply : every two days<br>Service conditions : generally acceptable | Priority 3rd |
| IV | Urban area<br>Commercial zone<br>Population density : 187persons/ha | Population served : approx. 85%<br>Water supply : every two days<br>Service conditions : using booster pump in some area; low water pressure along road No. 5. | Priority 2nd |
| V | Urban area<br>Commercial zone<br>Population density : 211 persons/ha | Population served : approx. 70%<br>Water supply : every two days<br>Service conditions : permanently poor water quantity/pressure in service area of Cam Thuong system. | Priority 2nd |
| VI | Semi-urban area<br>Rice fields, Medical facilities<br>Population density : 29 persons/ha | Population served : approx. 10%<br>Water supply : every two days<br>Service conditions : permanently poor water quantity/pressure | Priority 1st<br><br>High urgent necessity on account of many medical facilities |
| VIII | Semi-urban area<br>Rice fields<br>Population density : 22 persons/ha | Population served : approx. 15%<br>Water supply : every two days<br>Service conditions : seriously poor water quantity/pressure; temporal water supply (for one hour) | Priority 1st<br><br>Most serious service conditions among the existing service area |
| IX | Semi-urban area<br>Population density : 39 persons/ha | Population served : approx. 50%<br>Water supply : every two days<br>Service conditions : low water pressure along road No. 5. | Priority 2nd |
| X | Semi-urban area<br>Rice fields<br>Population density : 21 persons/ha | Population served : approx. 30%<br>Service conditions : generally acceptable | Priority 3rd |

Priority 1st : Urgent necessity is High
Priority 2nd : Improvement of existing conditions is required
Priority 3rd : Present situation seems acceptable

## 2 Water Demand Forecast

### 2.1 Land Use Plan and Population Forecast

#### 2.1.1 Population Forecast

Total population of Hai Duong Town in 1985 was 130,481. Seeing past ten years trend of population movement, natural increasing rate of 1985 was 1.7% and that of 1995 was 1.3% which decreased little year by year.

On the other hand, population growth due to social factor showed 1% around from 1985 to 1991. It turned up gradually later on. This is mainly because of a movement of population in agricultural area stipulated by today's urbanized/developing circumstances, whereas family plan policy is going to success.

Table 2-1 Population trend of Hai Duong city

| Year | Population | Natural growth | | Social growth | | Population growth | |
|---|---|---|---|---|---|---|---|
| | (persons) | (%) | (persons) | (%) | (persons) | (%) | (persons) |
| 1970 | 86,980 | 13.50% | 11,742 | 1.10% | 958 | 14.60% | 12,700 |
| 1975 | 99,680 | 13.00% | 12,958 | 2.32% | 2,312 | 15.32% | 15,270 |
| 1980 | 114,951 | 10.00% | 11,495 | 3.51% | 4,036 | 13.51% | 15,531 |
| 1985 | 130,481 | 1.70% | 2,219 | 1.01% | 1,318 | 2.71% | 3,537 |
| 1986 | 134,018 | 1.50% | 2,010 | 1.00% | 1,341 | 2.50% | 3,351 |
| 1987 | 137,369 | 1.50% | 2,060 | 3.30% | 4,534 | 4.80% | 6,594 |
| 1988 | 143,963 | 1.50% | 2,159 | 1.30% | 1,870 | 2.80% | 4,029 |
| 1989 | 147,992 | 1.50% | 2,219 | 0.95% | 1,410 | 2.45% | 3,629 |
| 1990 | 151,621 | 1.50% | 2,274 | 0.50% | 759 | 2.00% | 3,033 |
| 1991 | 154,654 | 1.30% | 2,010 | 1.00% | 1,546 | 2.30% | 3,556 |
| 1992 | 158,210 | 1.30% | 2,057 | 1.76% | 2,784 | 3.06% | 4,841 |
| 1993 | 163,051 | 1.30% | 2,120 | 2.04% | 3,324 | 3.34% | 5,444 |
| 1994 | 168,495 | 1.30% | 2,190 | 2.74% | 4,625 | 4.04% | 6,815 |
| 1995 | 175,310 | 1.30% | 2,279 | 4.56% | 7,995 | 5.86% | 10,274 |
| 1996 | 185,584 | | | | | | |

Population forecast by wards are given in Table 2-2, taking into account of the above mentioned the trend of population movement and the land use plan. Population growth rate in urban area ranges 0.5% to 5.3% (3.8% in average) by year 2000, considering the current population movement from rural area. It becomes declined to 2.3% in average from year 2000 to 2010.

Table 2-2 Population growth rate

| Ward | | Acreage (ha) | Population (persons) | | | Growth rate | | |
|---|---|---|---|---|---|---|---|---|
| | | | 1996 | 2000 | 2010 | 1996 ~2010 | 1996 ~2000 | 2000 ~2010 |
| I | Phuong Tran Hung Dao | 40.60 | 9,618 | 11,200 | 14,000 | 2.7% | 3.9% | 2.3% |
| II | Phuong Quang Trung | 86.35 | 13,686 | 16,800 | 21,000 | 3.1% | 5.3% | 2.3% |
| III | Phuong Nguyen Trai | 47.75 | 10,244 | 12,040 | 15,050 | 2.8% | 4.1% | 2.3% |
| IV | Phuong Pham Ngu Lao | 64.75 | 12,089 | 14,000 | 17,500 | 2.7% | 3.7% | 2.3% |
| V | Phuong Tran Phu | 88.00 | 18,566 | 22,400 | 28,000 | 3.0% | 4.8% | 2.3% |
| VI | Phuong Thanh Binh | 565.20 | 16,573 | 19,600 | 24,500 | 2.8% | 4.3% | 2.3% |
| VII | Phuong Hai Tan | 255.00 | 8,139 | 8,400 | 10,500 | 1.8% | 0.8% | 2.3% |
| VIII | Phuong Ngoc Chau | 654.60 | 14,409 | 17,080 | 21,350 | 2.8% | 4.3% | 2.3% |
| IX | Phuong Binh Han | 307.70 | 12,100 | 12,320 | 15,400 | 1.7% | 0.5% | 2.3% |
| X | Phuong Cam Thuong | 250.00 | 5,300 | 6,160 | 7,700 | 2.7% | 3.8% | 2.3% |
| Subtotal | | 2,359.95 | 120,724 | 140,000 | 175,000 | 2.7% | 3.8% | 2.3% |
| Commune | | | | | | | | |
| 1 | Xa Ai Quoc | 650.00 | 8,754 | 9,104 | 11,380 | 1.9% | 1.0% | 2.3% |
| 2 | Xa An Chau | 400.00 | 5,657 | 6,120 | 6,400 | 0.9% | 2.0% | 0.4% |
| 3 | Xa Nam Dong | 893.00 | 13,382 | 14,100 | 15,000 | 0.8% | 1.3% | 0.6% |
| 4 | Xa Tan Hung | 610.00 | 6,160 | 6,404 | 8008 | 1.9% | 1.0% | 2.3% |
| 5 | Xa Thach Khoi | 552.00 | 7,736 | 8180 | 10,100 | 1.9% | 1.4% | 2.1% |
| 6 | Xa Tu Minh | 628.70 | 12,583 | 13,200 | 16,500 | 2.0% | 1.2% | 2.3% |
| 7 | Xa Viet Hoa | 613.30 | 10,588 | 11,800 | 13,500 | 1.8% | 2.7% | 1.4% |
| Subtotal | | 4,347.00 | 64,860 | 68,908 | 80,888 | 1.6% | 1.5% | 1.6% |
| | | | | | | | | |
| Total | | 6,706.95 | 185,584 | 208,908 | 255,888 | 2.3% | 3.0% | 2.0% |

Future population of Hai Duong is forecast in Table 2-3.

Table 2-3 Population Forecast of Hai Duong

(persons)

| | 1996 | 2000 | 2010 |
|---|---|---|---|
| Urban area | 120,724 | 140,000 | 175,000 |
| Semi-urban area | 64,860 | 68,908 | 80,888 |
| Total | 185,584 | 208,908 | 255,888 |

(Note)

Urban area : Ward I - X

Semi-urban area : Commune 1 - 7

### 2.1.2 Land Use Plan

Land use plan is formulated in accordance with the future population growth and urban development plan. Land use is categorized into the following seven zones :

- Residential zone
- Office zone
- Public utility zone
- Warehouse zone
- Industrial zone
- Park / Green land zone
- Others

According to the land use plan, about 20% of urban area is given for residential zone. Industrial zone, about 10% of city is planned, is decided to be moved gradually from the central area to suburban area. Park / Green land zone is given relatively rich space of 37% so as to secure wealthy living environment. Table 2-4 summarizes the land use plan.

Table 2-4 Acreage by Planned Land Use

(ha)

| Year | Whole city | Residential zone | Industrial zone | Park / Green land |
|---|---|---|---|---|
| 1996 | 3,601 | 590 | 260 | 1,682 |
| 2000 | 6,700 | 794 | 674 | 882 |
| 2010 | 6,700 | 862 | 883 | 874 |

Table 2-5 gives population density by ward/commune. Comparing population density in 2000, that of urban area ranges 25 to 276 persons/ha and that of semi-urban area ranges 10 to 21 persons/ha. It can be said that there are a lot of rooms to accept population in semi-urban area.

Seeing population density by residential zone, that of urban area ranges 142 to 368 persons/ha. It implies that about a half of wards comes almost to saturation of population and the rest is still capable to accept the population.

From aforementioned, land use plan and population forecast as well as its distribution, which are stated in the Master Plan, are considered to be reasonable.

Table 2-5a Population density by ward

| Ward | | Acreage (ha) | Population 1996 | Population 2000 | Population 2010 | Population Density 1996 | Population Density 2000 | Population Density 2010 |
|---|---|---|---|---|---|---|---|---|
| I | Phuong Tran Hung Dao | 40.60 | 9,618 | 11,200 | 14,000 | 237 | 276 | 345 |
| II | Phuong Quang Trung | 86.35 | 13,686 | 16,800 | 21,000 | 158 | 195 | 243 |
| III | Phuong Nguyen Trai | 47.75 | 10,244 | 12,040 | 15,050 | 215 | 252 | 315 |
| IV | Phuong Pham Ngu Lao | 64.75 | 12,089 | 14,000 | 17,500 | 187 | 216 | 270 |
| V | Phuong Tran Phu | 88.00 | 18,566 | 22,400 | 28,000 | 211 | 255 | 318 |
| VI | Phuong Thanh Binh | 565.20 | 16,573 | 19,600 | 24,500 | 29 | 35 | 43 |
| VII | Phuong Hai Tan | 255.00 | 8,139 | 8,400 | 10,500 | 32 | 33 | 41 |
| VIII | Phuong Ngoc Chau | 654.60 | 14,409 | 17,080 | 21,350 | 22 | 26 | 33 |
| IX | Phuong Binh Han | 307.70 | 12,100 | 12,320 | 15,400 | 39 | 40 | 50 |
| X | Phuong Cam Thuong | 250.00 | 5,300 | 6,160 | 7,700 | 21 | 25 | 31 |
| Sub total | | 2,359.95 | 120,724 | 140,000 | 175,000 | 51 | 59 | 74 |
| Commune | | | | | | | | |
| 1 | Xa Ai Quoc | 650.00 | 8,754 | 9,104 | 11,380 | 13 | 14 | 18 |
| 2 | Xa An Chau | 400.00 | 5,657 | 6,120 | 6,400 | 14 | 15 | 16 |
| 3 | Xa Nam Dong | 893.00 | 13,382 | 14,100 | 15,000 | 15 | 16 | 17 |
| 4 | Xa Tan Hung | 610.00 | 6,160 | 6,404 | 8008 | 10 | 10 | 13 |
| 5 | Xa Thach Khoi | 552.00 | 7,736 | 8180 | 10,100 | 14 | 15 | 18 |
| 6 | Xa Tu Minh | 628.70 | 12,583 | 13,200 | 16,500 | 20 | 21 | 26 |
| 7 | Xa Viet Hoa | 613.30 | 10,588 | 11,800 | 13,500 | 17 | 19 | 22 |
| Sub total | | 4,347.00 | 64,860 | 68,908 | 80,888 | 15 | 16 | 19 |
| Total | | 6,706.95 | 185,584 | 208,908 | 255,888 | 28 | 31 | 38 |

Table 2-5b Population density by residential zone

| Ward | | Residential Zone (ha) 1996 | Residential Zone (ha) 2000 | Residential Zone (ha) 2010 | Population (Persons) 1996 | Population (Persons) 2000 | Population (Persons) 2010 | Population Density (Persons/ha) 1996 | Population Density (Persons/ha) 2000 | Population Density (Persons/ha) 2010 |
|---|---|---|---|---|---|---|---|---|---|---|
| I | Phuong Tran Hung Dao | 32.10 | 32.10 | 32.10 | 9,618 | 11,200 | 14,000 | 300 | 349 | 436 |
| II | Phuong Quang Trung | 45.71 | 45.71 | 45.71 | 13,686 | 16,800 | 21,000 | 299 | 368 | 459 |
| III | Phuong Nguyen Trai | 34.21 | 34.21 | 34.21 | 10,244 | 12,040 | 15,050 | 299 | 352 | 440 |
| IV | Phuong Pham Ngu Lao | 40.37 | 40.37 | 40.37 | 12,089 | 14,000 | 17,500 | 299 | 347 | 433 |
| V | Phuong Tran Phu | 62.01 | 62.01 | 62.01 | 18,566 | 22,400 | 28,000 | 299 | 361 | 452 |
| VI | Phuong Thanh Binh | 43.10 | 68.60 | 85.75 | 16,573 | 19,600 | 24,500 | 385 | 286 | 286 |
| VII | Phuong Hai Tan | 20.00 | 29.50 | 36.75 | 8,139 | 8,400 | 10,500 | 407 | 285 | 286 |
| VIII | Phuong Ngoc Chau | 120.25 | 120.50 | 120.50 | 14,409 | 17,080 | 21,350 | 120 | 142 | 177 |
| IX | Phuong Binh Han | 43.48 | 45.48 | 53.90 | 12,100 | 12,320 | 15,400 | 278 | 271 | 286 |
| X | Phuong Cam Thuong | 21.60 | 25.00 | 26.95 | 5,300 | 6,160 | 7,700 | 245 | 246 | 286 |
| Subtotal | | 462.83 | 503.48 | 538.25 | 120,724 | 140,000 | 175,000 | 261 | 278 | 325 |
| Commune | | | | | | | | | | |
| 1 | Xa Ai Quoc | 29.70 | 31.80 | 39.80 | 8,754 | 9,104 | 11,380 | 295 | 286 | 286 |
| 2 | Xa An Chau | 13.50 | 21.00 | 22.00 | 5,657 | 6,120 | 6,400 | 419 | 291 | 291 |
| 3 | Xa Nam Dong | 44.60 | 49.00 | 52.00 | 13,382 | 14,100 | 15,000 | 300 | 288 | 288 |
| 4 | Xa Tan Hung | 20.90 | 22.40 | 28.00 | 6,160 | 6,404 | 8008 | 295 | 286 | 286 |
| 5 | Xa Thach Khoi | 25.80 | 28.00 | 35.00 | 7,736 | 8180 | 10,100 | 300 | 292 | 289 |
| 6 | Xa Tu Minh | 100.00 | 100.00 | 100.00 | 12,583 | 13,200 | 16,500 | 126 | 132 | 165 |
| 7 | Xa Viet Hoa | 27.90 | 39.00 | 47.25 | 10,588 | 11,800 | 13,500 | 379 | 303 | 286 |
| Subtotal | | 262.40 | 291.20 | 324.05 | 64,860 | 68,908 | 80,888 | 247 | 237 | 250 |
| Total | | 725.23 | 794.68 | 862.30 | 185,584 | 208,908 | 255,888 | 256 | 263 | 297 |

## 2.2 Water Demand Forecast

### 2.2.1 Water Use Category

In Vietnam, type of water demand is categorized into the following four categories :

| | | |
|---|---|---|
| 1 | Domestic Use | |
| 2 | Commercial Use | Offices, Schools, Hospitals, Hotels, Restaurants, Stores, Small factories, etc. |
| 3 | Industrial Use | |
| 4 | Public Use | Park, Road washing, etc. |

### 2.2.2 Domestic Use

As domestic water consumption data of Hai Duong city has not measured, it is to be estimated based on the relating information/data. Although the design capacity of the existing water treatment plant was 21,000 $m^3$/day, it was superannuated and deteriorated. In 1997, the existing plant was partially rehabilitated and the superannuated equipment was replaced under the OECF commodity loan. Determined by meter reading of the distribution pumps, however, actual treatment capacity is estimated to be about 14,000 $m^3$/day. Besides, deterioration of the pipelines must cause a lot of amount of physical loss. Distributed water that is available in houses is estimated to be 60 to 120 l/c/d. Insufficient water to be used is supplemented from rain water, hand pumps and etc. It is difficult that we clearly understand actual water consumption of domestic use.

According to the design standard prepared by the Quality Control Department of the Ministry of Construction, water consumption for domestic use in Viet Nam should be taken referred to the standard summarized in Table 2-6.

Table 2-6 Unit water consumption

(l/cap/day)

| Design standard (MOC) | Water consumption |
|---|---|
| 1) Public tap | 40 - 60 |
| 2) Yard tap | 80 - 100 |
| 3) House connection + Sewerage | 150 - 180 |
| 4) 3) + Flash toilet + Bathtub | 180 - 250 |
| 5) 4) + Hot water heater | 250 - 300 |

Although unit water consumption of domestic use corresponds to 3) in Table 2-6 since the center of Hai Duong is high densely populated area. However taking into account that Hai Duong is a local city, the unit consumption in the First Year Basic Design Study was assumed at 135 l/c/d as proposed in the City's Master Plan. In the Second Year Study, however, it was determined at 100 l/c/d to reduce the Project scale on the basis of a policy of the Japanese Government.

### 2.2.3 Commercial Use

Water for commercial use includes commercial purposes, offices, hotels, restaurants, small factories and so on. As it is difficult to estimate water consumption of such purposes, usually past record of similar case or relating base data/information are to be taken reference for estimation. In Hai Duong City, as existing data for them are not available. The City Master Plan estimated 15 % of total domestic demand. This figure may be reasonable because Hai Duong is a capital city of Hai Duong Province and take an important role in the northern Viet Nam region.

### 2.2.4 Industrial Use

Industrial use water in Vietnamese water supply does not mean process water for production but means domestic use for staff/labors of factories. In most case, factories have their own water supply systems. Groundwater, their water source, contains iron so that water treatment is difficult to meet the drinking water criteria. Therefore, public water supply system is to be supplied to such factories for their domestic purpose.

In the water supply master plan of Hai Duong, 30% of domestic water is estimated for industrial use. However, this may mislead an excessive water demand estimation in densely inhabited area. To avoid such estimation, Hanoi water supply master plan has estimated 30 $m^3$/ha in 2000 and 35 $m^3$/ha in 2010. Therefore, the consumption of industrial use in the Project employs 25$m^3$/ha in 2000, that corresponds to 12 % of the domestic water.

In addition, as rate of land use of industrial zone is assumed to be increasing year by year, water demand is calculated in stepwise, that is, Ward I – V, center of city and already urbanized, is 100% ; Ward VI – X, surroundings of the city and being developing, is 30% in 2000.

### 2.2.5 Physical Loss, Other Use, etc.

Since the existing water supply facilities of Hai Duong are superannuated, physical loss is assumed to be high. Turbidity of surface water from river is extremely high so that a lot of back wash water is required. Moreover, recycle system of back wash water is not applied to the existing system. Considering these facts, back wash water used in treatment process may reach to about 15% of the production capacity.

Effective water estimated from water ledger of the water supply company is about 9,000 $m^3$/day. On the other hand, distribution capacity recorded by water meter is about 14,000 $m^3$/day. Consequently, physical loss is calculated at around 35 %.

In the water supply master plan of Hai Duong, the physical losses and other use is 25-35% of total water supply capacity. However, it is selected as 25% for calculation of the water demand. This project plans rehabilitation of the existing system and expansion for new service area. Even after rehabilitation, it seems reasonable to estimate physical loss at 35% or so, whereas 15% is feasible for the new system. As for plant loss, 5 % is assumed for new plant, considering water quality and treatment system.

### 2.2.6 Daily Maximum Factor / Hourly Peak Factor

Water supply master plan of Hai Duong has been applied 1.3 for daily maximum factor, whereas 1.4 in the water supply master plan of Hanoi (JICA, 1997). Hence, 1.3 is considered to be appropriate value in the Project. As for hourly peak factor, referring to the value of 1.35 in the master plans of Hanoi and Hai Duong, 1.35 is applied for it.

### 2.2.7 Water Demand

Service area of Hai Duong is divided into three areas, taking into consideration urban developing formation and natural conditions, namely, northern area, central area and southern area. Water demand by these area is forecast in Table 2-7.

Table 2-7 Water demand of Hai Duong (Average daily water demand)

| | Year 1996 | | Year 2000 | | Year 2005 | | Year 2010 | |
|---|---|---|---|---|---|---|---|---|
| | Persons | $m^3$/day | persons | $m^3$/day | persons | $m^3$/day | persons | $m^3$/day |
| Northern area | | | | | | | | |
| Urban area | - | - | - | - | - | - | - | - |
| Semi-urban area | 27,793 | 1,567 | 29,324 | 4,911 | 31,016 | 6,485 | 32,780 | 8,059 |
| Subtotal | 27,793 | 1,567 | 29,324 | 4,911 | 31,016 | 6,485 | 32,780 | 8,059 |
| Central area | | | | | | | | |
| Urban area | 64,203 | 13,310 | 131,600 | 21,739 | 148,050 | 31,125 | 164,500 | 40,504 |
| Semi-urban area | 71,553 | 12,012 | 25,000 | 4,208 | 27,500 | 5,484 | 30,000 | 6,761 |
| Subtotal | 135,756 | 25,323 | 156,600 | 25,946 | 175,550 | 36,609 | 194,500 | 47,265 |
| Southern area | | | | | | | | |
| Urban area | | | 8,400 | 1,414 | 9,450 | 2,191 | 10,500 | 2,965 |
| Semi-urban area | 22,035 | 2,806 | 14,548 | 2,029 | 16,346 | 2,907 | 18,108 | 3,786 |
| Subtotal | 22,035 | 2,806 | 22,984 | 3,446 | 25,769 | 5,098 | 28,608 | 6,750 |
| Total | 185,584 | 29,695 | 208,908 | 37,050 | 323,368 | 48,192 | 255,888 | 62,072 |

(Note) Northern area : Commune 1-3
Central area : Ward I – VI, VII – X, Commune 6,7
Southern area : Ward VII, Commune 4,5.

# Appendix 6 Hydrogeology

Ground Water Condition of Water Source Development Area

1 Water Source Development Area

The water source development area is a part of the Cam Giang district and has a width of about 3.0km in an east to west direction. It is located 19km west of Hai Duong city. In order to investigate the geological and ground water condition of the area, a total of six test wells were drilled. Of these test wells, the LK6 and LK8 were constructed by the Vietnamese side, and other wells by the Study Team. The details of the structures of the test wells are indicated in Table.1-1. The locations of test wells are shown in Fig.1-1.

Table 1-1 Details of Wells in Development Area.

| Well No | Construction | | Well Depth (m) | Depth Screen Installed (m) | Diameter (mm) | | Depth Pump Installed (m) |
|---|---|---|---|---|---|---|---|
| | Start | Completed | | | Casing | Screen | |
| LK1 | July 17, 1996 | 1996<br>August 30, 1996 | 83 | 57-77 | 273-219 | 219 | 29.35 |
| LK6 | June 16, 1996 | August 25, 1996 | 100 | 60-84 | 273-219 | 219 | 30.00 |
| LK8 | 1997 | March 16, 1997 | 95.5 | 54-74 | 130 | 130 | 83.46 |
| LK11 | July 11, 1996 | August 3, 1996 | 108 | 70-77<br>82-94<br>98-105 | 273-219 | 219 | 29.23 |
| LK15 | November 11, 1996 | December 12, 1996 | 110 | 60-86 | 273-219 | 219 | 30.00 |
| LK17 | December 6, 1996 | December 24, 1996 | 110 | 58-75 | 273-219 | 219 | 30.00 |

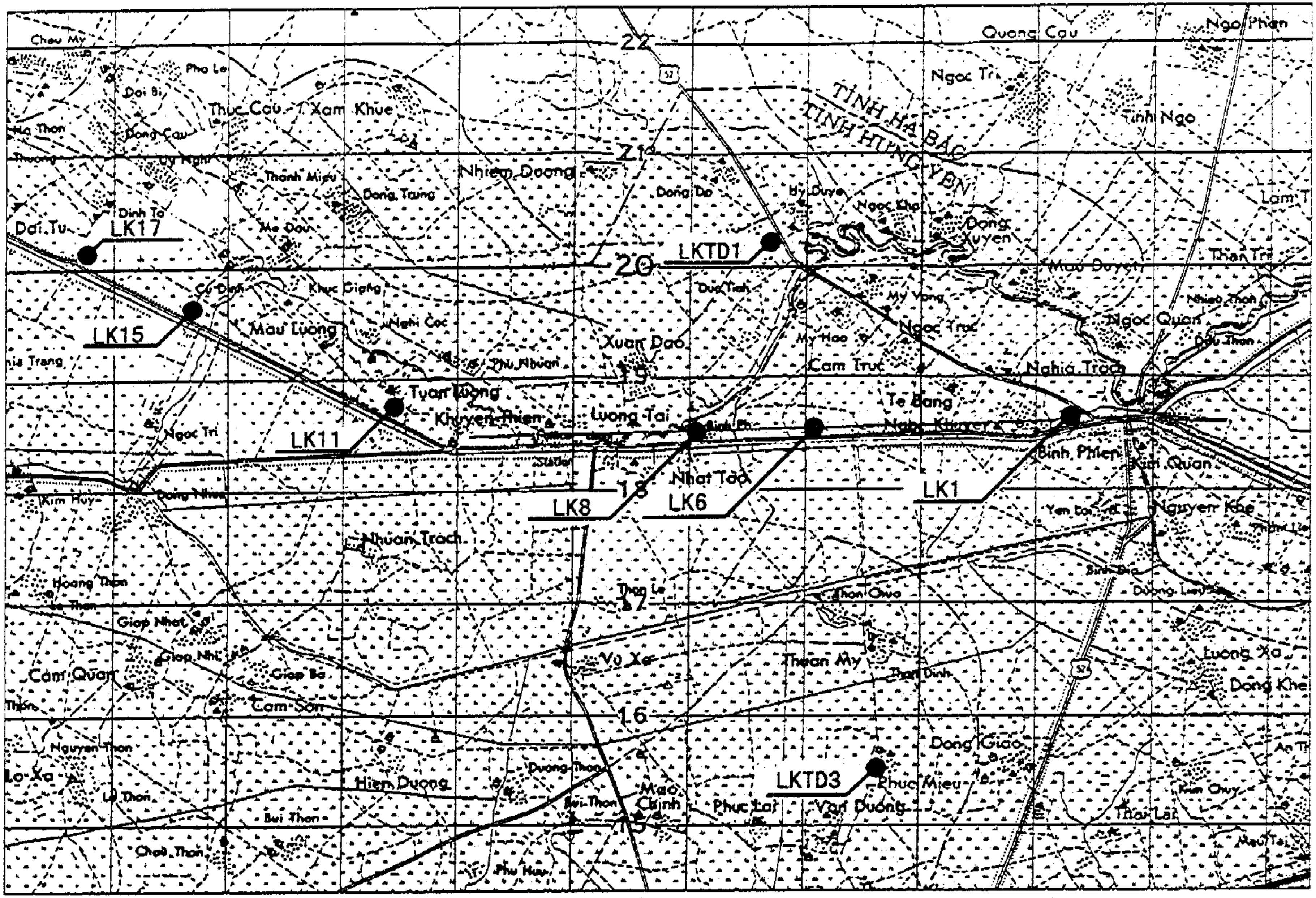



Fig.1-1. The Location Map of Test Wells

## 2 Hydrogeology of Development Area

### 2-1 Geological Structure

The Development area is located in the center of the Bac Bo Delta, which is formed by the Red River and its tributaries. The geology of the development area is almost the same as that of the Hanoi area which is located on the upper reaches of the Red River. The outline of the geological stratigraphy and hydrogeological characteristics of the development area are shown in Fig.2-1. Also, a sectional drawing based on the columnar section of the wells is indicated in Fig.2-2. The stratigraphy in the development area is classified as follows:

1) Hai Hung Formation

Correspond to the lower to middle Holocene, Quaternary.

《Upper part ($Q_{IV}^{1-2}hh2$)》

Formation Thickness : 2m to 6.5m.( Average 3.4m).

Geology : Silt, clay, including decomposed plants in clay layer.

《Lower part ($Q_{IV}^{1-2}hh1$)》 **【qh aquifer】**

Formation Thickness : 6m to 15m. (Average 11.0m).

Geology : Fine sand layer.

2) Vinhphuc Formation

Correspond to the upper Pleistocene, Quaternary.

《Upper part ($Q_{III}vp2$)》

Formation Thickness : 3m to 21.5m. (Average 8.7m).

Geology : Clay, including decomposed plants in clay layer.

《Lower part ($Q_{III}vp1$)》 **【qp2 aquifer】**

Formation Thickness : 5.5m to 22.5m. (Average 15.3m).

Geology : From fine to coarse sand layer.

3) Hanoi Formation

Correspond to the middle Pleistocene, Quaternary.

《Upper part ($Q_{II\,III}hn2$)》

Formation Thickness : 6m to 19m. (Average 8.8m).

Geology : Clay layer.

《Lower part ($Q_{II\,III}hn1$)》 **【qp1 aquifer】**

Formation Thickness : 27m to 46m. (Average 33.8m).

Geology : Sand and gravel, layer in most upper parts consist of fine and coarse sand.

4) Lechi Formation ($Q_1 lc$)

Correspond to the lower Pleistocene, Quaternary.

Formation Thickness : 6m to 23m. (Average 14.4m).

Geology : Sand and gravel with silt layer.

5) Vinhbao Formation ($N_2 vb$)

Correspond to the Upper Pliocene, Neogene

Formation Thickness : More than 6m. According to existing LK58-4 data, 2km far from LK17 in a westerly direction, the formation thickness was verified to be more than 100m.

Geology : Weakly consolidated gravel, sand, mud stone.

## Fig.2-1. Stratigraphy and Hydrogeological Characteristics

| Geological age | | | Formation Name | Symbol | Column | Thickness (m) | Hydrogeologic Characteristic |
|---|---|---|---|---|---|---|---|
| Period | Epoch | Stage | | | | | |
| Quaternary | Holocene | Middle-Lower | Hai hung | $Q_{IV}^{1-2}hh2$ | | 2.0～6.5 | Silt ,Clay layer<br>Aquiclude |
| | | | | $Q_{IV}^{1-2}hh1$ | | 6.0～15 | Sand layer<br>qh confined aquifer |
| | Pleistocene | Upper | Vinhphuc | $Q_{III}vp2$ | | 3.0～21.5 | Thin clay layer<br>Aquiclude |
| | | | | $Q_{III}vp1$ | | 5.5～22.5 | Sand layer<br>Lower part:Sand,gravel<br>qp2 confined aquifer |
| | | Middle | Hanoi | $Q_{II-III}hn2$ | | 6.0～19.0 | Mainly clay layer<br>Aquiclude |
| | | | | $Q_{II-III}hn2$ | | 27.0～46.0 | Most upper part :<br>Sand layer<br>Mainly gravel layer<br>qp1 confined aquifer |
| | | Lower | Lechi | $Q_{I}lc$ | | 6.0～23.0 | Clayey sand & gravel or sand & gravel layer |
| Neogene | Pliocene | Upper | Vinhbao | Nvb | | 6.0< | Conglomerate, sand, clay<br>Weakly consolidated |

The development aquifer which is the lower part of the Hanoi Formation($Q_{II-III}hn1$:qp1 aquifer) is distributed throughout entire area of the Bac Bo plain and the average thickness is 34m. This aquifer corresponds to sand and gravel and has the highest potential for groundwater development in the development area. The ground water of the Hanoi Formation is in a confined condition because of the existence of impervious layers of the clay layer at Upper Hanoi Formation and others. According to the test well's data, the static water level in the development area is about 0.8 to 1.65m below ground.

However, groundwater with high chloride concentrations, ranging from 3,000 mg/l to about 6,000 mg/l, is in some limited areas of the Bac Bo Plain because of connate water accumulated with sea water. Based on the data of the Geological Survey in Vietnam (K2) of the Vietnamese Ministry of Industry, the distribution condition in groundwater with high concentrations of chloride is shown in Fig.2-3. According to this distribution map, groundwater with high chloride concentrations of one to three g/l (TDS value) is distributed in the location about 5km east of the proposed groundwater development area. TDS represents the total amount of dissolved materials. The proposed water source development area is located near or in the area with lower concentrations of TDS, 0.5 g/l or less. As Hai Duong city is located on the outskirts of the area with chloride concentration, the chloride concentration is generally high. Thus, the Cam Giang area was selected for water source development.

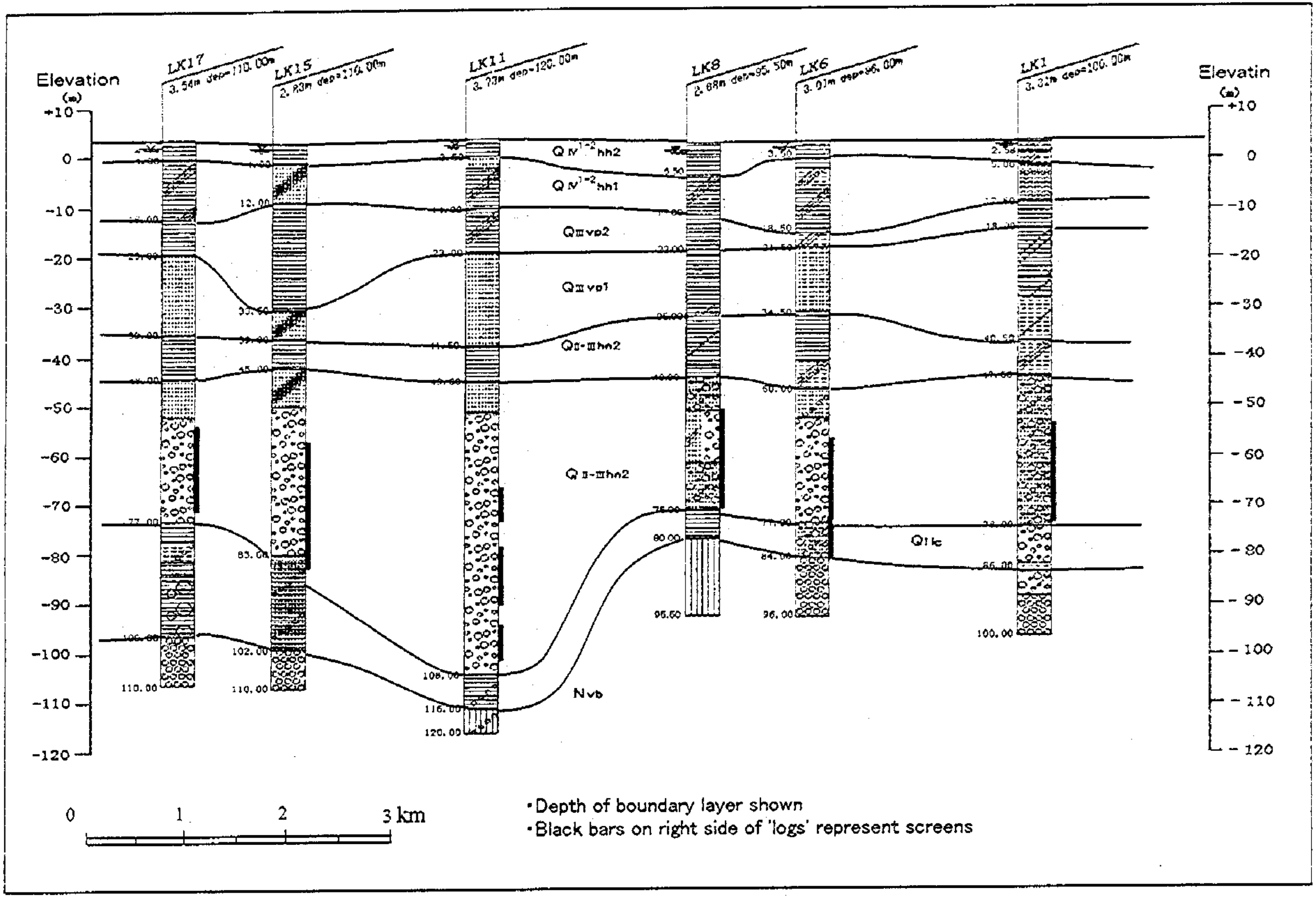



Fig.2-2. Geological Section of Pumping Area

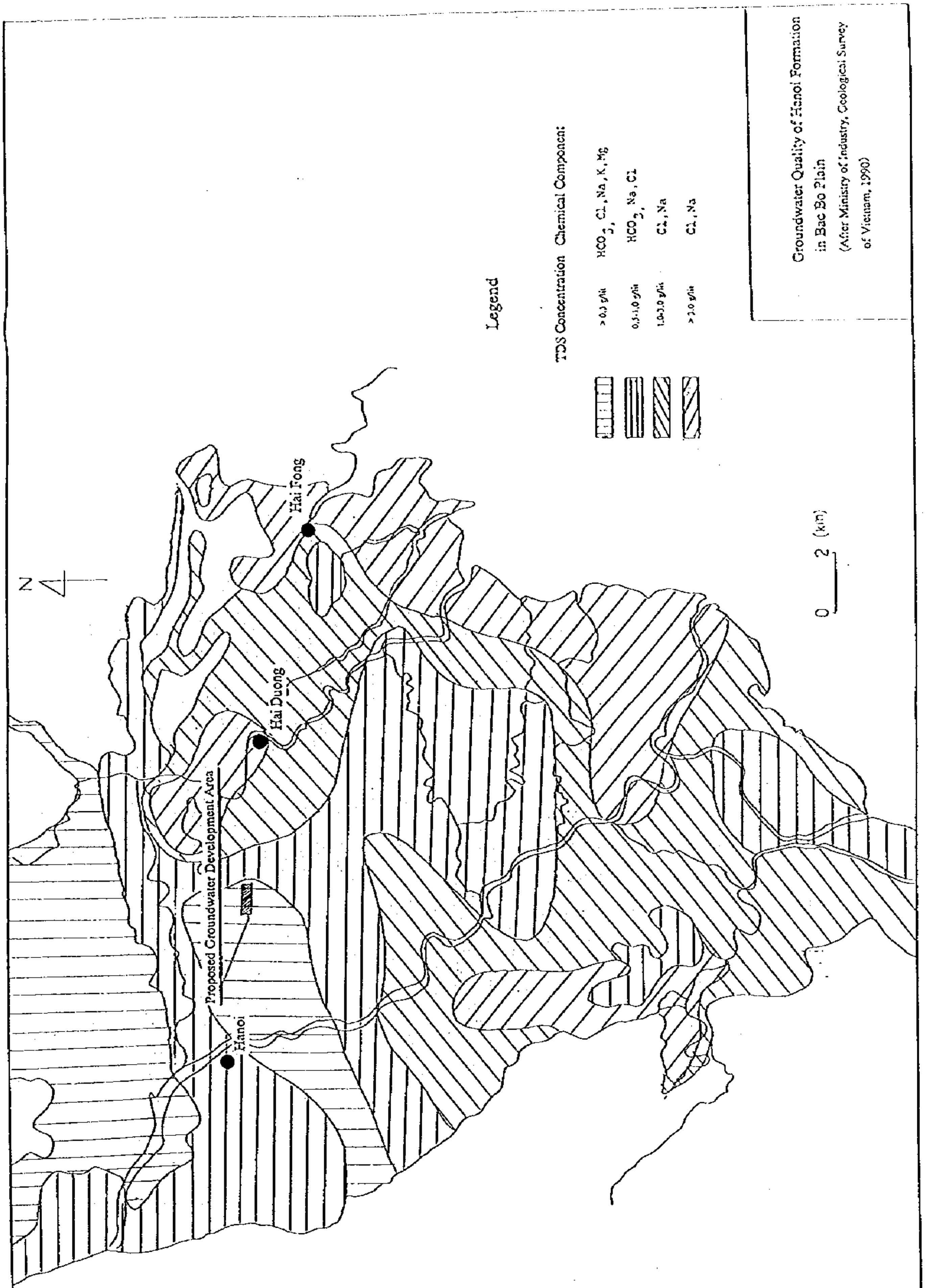



Fig.2-3 Groundwater Quality of Hanoi Formation in Bac Bo Plane

## 2-2 Results of Pumping Tests of Test Wells

### 1) Results of Step Drawdown Pumping Tests

According to the Japanese Design Standard of Water Supply Facilities, the Critical yield was analyzed based on the results of the step drawdown pumping tests. Of the critical yield, 70% was determined as the safe yield. The results are shown below:

Results of Step Drawdown Tests

| Well | Critical Yield (l/s) | Safe Yield (l/s) |
|---|---|---|
| LK 1 | 50.5 | 35.4 |
| LK 6 | 44.1 | 30.9 |
| LK11 | 60.0 | 42.0 |
| LK17 | 56.0 | 39.2 |
| Average | 52.7 | 36.9 |

The average of the safe yield is Q = 36.9 l/s. The designed operation hours of submersible pumps is 20, the production rate per planned water source well is Q = 132.84m$^3$/hours = 2,656 m$^3$/day.

### 2) Results of continuous pumping test

The continuous pumping test was carried out under the Critical Yield after the finish of the step drawdown test in each test well. The results of the pumping tests were analyzed using the Jacob Method, and the recovery test by Theis's Recovery Method, the Transmissivity (T) and the Coefficient of Permeability (K) was calculated as follows:

Analysis Results of Continuous Pumping Tests

| Well No. | Transmissivity (T ; m$^2$/d) | | | Coefficient of Permeability (K : cm/sec) |
|---|---|---|---|---|
| | Jacob Method | Theis's Recovery Method | Average | |
| LK 1 | 4,763 | 4,763 | 4,763 | 1.22×10-1 |
| LK 6 | 3,394 | 2,036 | 2,715 | 9.14×10-2 |
| LK11 | | 3,176 | 3,176 | 7.98×10-2 |
| LK15 | 2,149 | | 2,149 | 6.55×10-2 |
| LK17 | 2,386 | 2,938 | 2,662 | 1.06×10-1 |
| | | Average | 3,093 | 9.29×10-2 |

As a result, the average Transmissivity $T=3,000m^2/d$ in the development area is adopted. The average Coefficient of Permeability is $K=9.0 \times 10^{-2}cm/sec$. Some other groundwater hydrologic calculations were also performed, such as the examination of the potential for groundwater development, the estimation of influence radius and the maximum drawdown of groundwater level in the case of the allocation of numerous production wells.

## 2-3 Groundwater Quality

### 1) Results of Water Quality Analysis

Water quality analysis in test wells was carried out on groundwater obtained by continuous pumping tests. Three or four samples were analyzed per well. The main results are as follows:

Main Results of Groundwater Quality Analysis

| Well, Sample No. | | pH | $Cl^-$ (mg/l) | $NH_4^+$ (mg/l) | $Fe^{++}$ (mg/l) | $Mn^{++}$ (mg/l) | Number of Authority |
|---|---|---|---|---|---|---|---|
| Vietnam Standard | | 6.5～8.5 | 250 | 3.0 | 0.3 | 0.1 | 1 |
| LK1 | 1/1 | | 297.4 | 2.5 | 38.9 | 1.70 | 2 |
| | 1/2 | | 318.6 | 2.1 | 45.5 | 2.03 | 2 |
| | 1/3 | 6.0 | 315.1 | 2.3 | 45.4 | 1.90 | 2,3 |
| | | 5.62 | 309.6 | 1.9 | 42.0 | 1.81 | 4 |
| | Ave | 5.81 | 310.2 | 2.2 | 43.0 | 1.86 | |
| LK6 | 6/1 | 6.8 | 272.6 | 2.20 | 28.0 | 3.00 | 2,3 |
| | 6/2 | | 168.0 | 1.02 | 29.8 | 1.81 | 2 |
| | 6/3 | | 136.3 | 2.20 | 27.2 | 1.81 | 2 |
| | | 5.5 | 304.8 | 1.90 | 38.0 | 2.02 | 4 |
| | Ave | 6.15 | 220.4 | 1.83 | 30.9 | 1.87 | |
| LK8 | 8/1 | | | | 38.4 | 1.40 | 2 |
| | 8/2 | | | | 37.3 | 1.40 | 2 |
| | 8/3 | | 209.6 | 1.20 | 30.5 | 1.30 | 2 |
| | Ave | | 209.6 | 1.20 | 35.4 | 1.37 | |
| LK11 | 1 | | 157.1 | 3.6 | 41.2 | 1.67 | 2 |
| | 2 | | 145.1 | 3.8 | 42.0 | 2.60 | 2 |
| | 3 | 6.11 | 145.1 | 8.4 | 40.9 | 2.50 | 2,3 |
| | | 5.7 | 155.8 | 1.29 | 44.88 | 1.989 | 5 |
| | Ave | 5.9 | 150.8 | 4.27 | 42.3 | 2.19 | |
| LK15 | 1 | 6.01 | 109.7 | 0.8 | 5.1 | 1.40 | 2,3 |
| | 2 | | | | 5.3 | 1.45 | 2 |
| | 3 | 6.0 | 109.7 | 1.01 | 5.3 | 1.40 | 2,3 |
| | Ave | 6.0 | 109.7 | 0.91 | 5.23 | 1.42 | |
| LK17 | 1 | 6.3 | 28.3 | 0.64 | 2.2 | 0.50 | 2,3,5 |
| | 2 | 6.2 | 31.9 | 0.5 | 2.4 | 0.70 | 2,3,5 |
| | 3 | | 30.5 | 0.79 | 1.8 | 0.50 | 2,3 |
| | | 6.7 | 24.8 | 0.84 | 1.95 | 0.51 | 2,3,5 |
| | Ave | 6.4 | 28.9 | 0.69 | 2.09 | 0.55 | |

Authority: 1.Vietnam Standard for Drinking Water

2.'Final Report on Results of Detail Ground Water Exploration Step in Camgiang'

3.Results of laboratory test (Nucleus Science and Technology Institute) K2

4.Results of laboratory test (VIWASE's Laboratory)

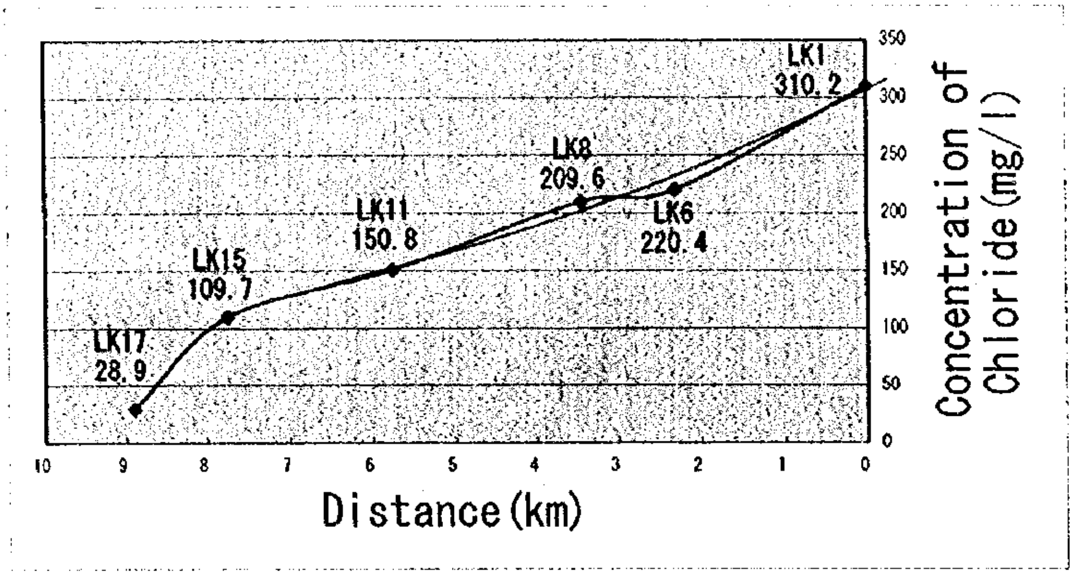
5.Results of laboratory test (QUATEST's Laboratory)

The results of water quality analysis show generally low pH levels, considerably over the Vietnamese drinking water standard in $Fe^{2+}$ and $Mn^{2+}$, a little over the standard in some samples of $NH_4^+$. The $Cl^-$ concentration in LK1, LK6 on the Hai Duong city (east) side is over the Vietnamese standard.

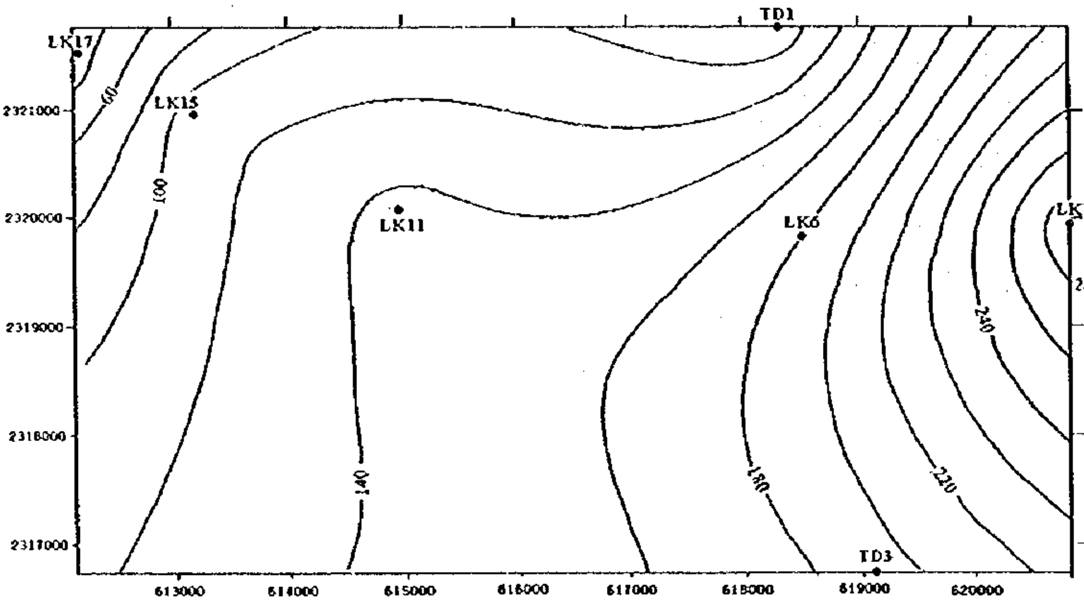
The most difficult item to treat is $Cl^-$, therefore it is desirable that the development area be appointed in a low concentration area of $Cl^-$.

2) Distribution of $Cl^-$ concentration

From results of the water quality tests, distribution of $Cl^-$ average concentration in east and west direction of water source area is shown in figure below.



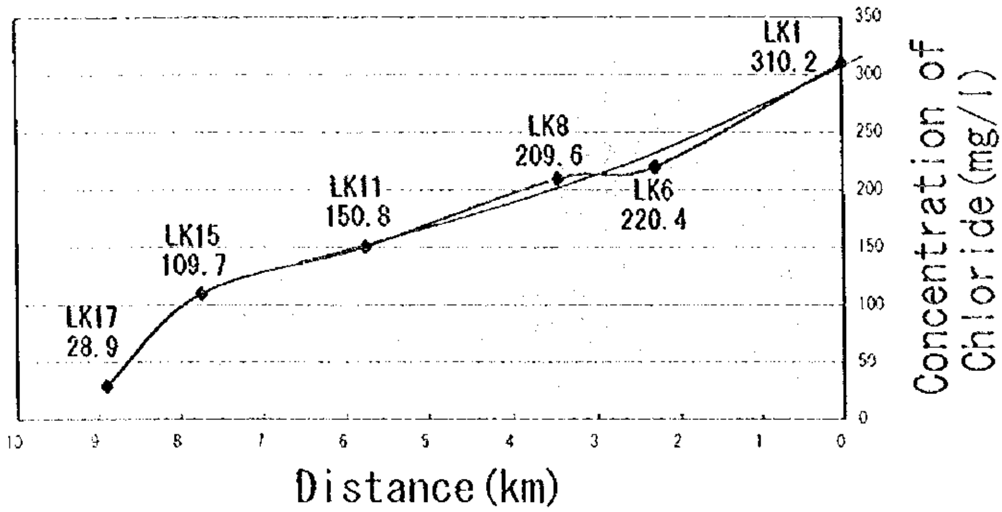

The area over the standard value in $Cl^-$ concentration is read as 500m to 600m east of LK6 on this graph. $Cl^-$ concentration of north and south direction tends to the same or less values as indicated in contour map below.



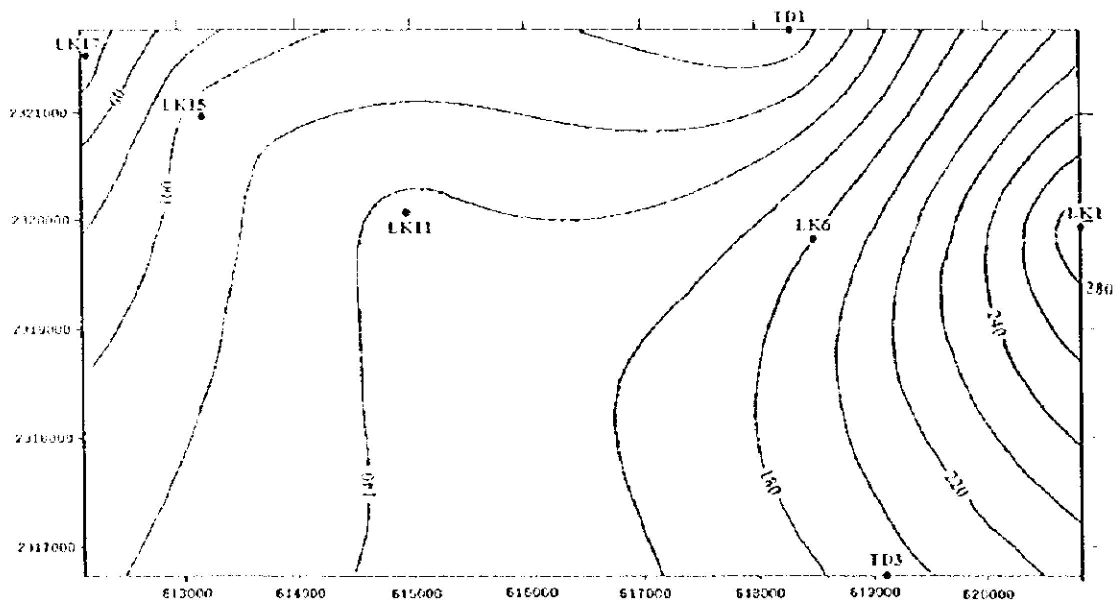

Contour Map of $Cl^-$ Concentration

2) Distribution of Cl⁻ concentration

From results of the water quality tests, distribution of Cl⁻ average concentration in east and west direction of water source area is shown in figure below.




The area over the standard value in Cl⁻ concentration is read as 500m to 600m east of LK6 on this graph. Cl⁻ concentration of north and south direction tends to the same or less values as indicated in contour map below.




Contour Map of Cl⁻ Concentration

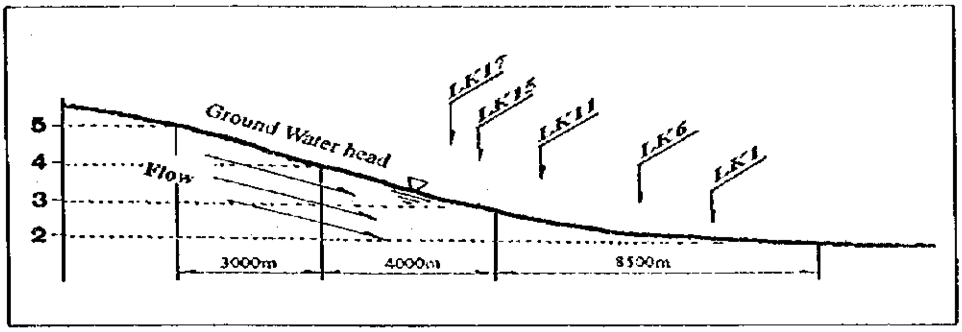
3) Consideration of salinization

The most important factor in deciding the site of a pumping well is the $Cl^-$ concentration in the water development area.

The groundwater west of LK6 is less than the value of the Vietnamese drinking water standard. Due to water pumping, the high $Cl^-$ concentration water in the eastern area is pulled, thereby possibly raising the concentration of $Cl^-$.

① Normal Groundwater Flow

Seeing the Groundwater Head Contour from 'The $Q_{\mathrm{II}\,\mathrm{III}}$ aquifer map of Bac Bo plain', the groundwater head incline to southeast is about one four thousandth of a meter to one six thousandth of a meter. Refer to the figure below.




According to Darcy's Law natural groundwater flow per 1km width is as follows:

Difference of groundwater head : $\Delta h = 1$ (m)

Distance of groundwater head falling 1m : $\Delta x = 4000$m

Thickness of aquifer : $b = 34$ (m)

Calculated width : $L = 1000$ (m)

Coefficient of permeability : $k = 77.8$ (m/day)

Groundwater flux : $Q = L\ b\ k \times \Delta h / \Delta x = 34 \times 1000 \times 77.8 \times 1 / 4000 = 661.3$ ($m^3/d$)

If the water development area length is supposed to be 3.5km, and an angle between groundwater flow and pumping well line is supposed to be 45 degrees, then groundwater flow width into pumping area is $3.5\text{km} \times \sin 45^\circ = 2.47$km therefore groundwater flow from the northwest is calculated as follows:

$$Q = 661.3 \times 2.47 = 1633 \ (m^3/d)$$

This is about 16% of the planned intake of water volume. (Refer to the following figure.)

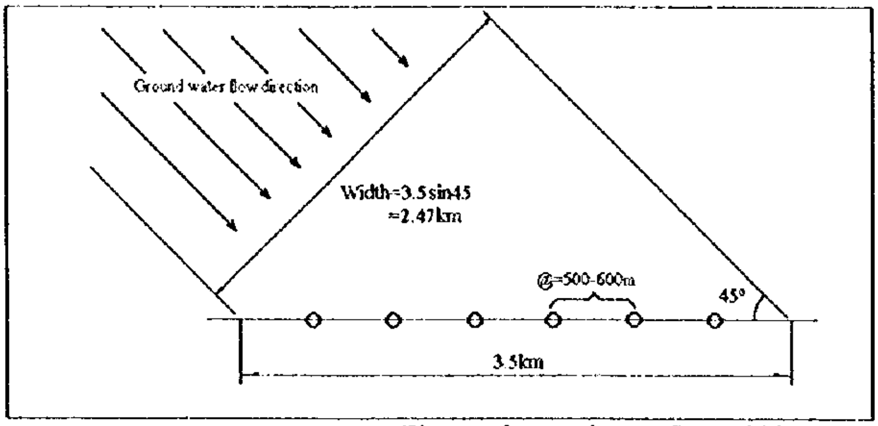



Figure of groundwater flow width

② Result of continuous pumping tests

The results on the table on 'Main Results of Groundwater Quality Analysis' were from water quality samples obtained from continuous pumping tests. Excluding LK6, the transition of chloride concentration is constant. The LK6 chloride concentration decreases 298.4----168.0----136.3(mg/l).

The chloride in the water development area, which is sedimented with connate water, is different from the seashore area. The groundwater seems to be inclined towards dechlorination.

For these reasons, it was concluded that chloride concentrations won't rise. Thus water development area was selected in the west of LK6.

## 3 The Plan of the Intake Facility

### (1) Required Number of Wells

The safe yield of a single well at the proposed wellfield is estimated by pump tests at 2,200m$^3$/d and the design capacity of the water source is 10,200m$^3$/d. Therefore the total number of wells required is six(6) including one(1) standby well for maintenance and emergencies.

Number of Planned Wells : 6 Wells(including one standby well)

### (2) Influence Radius of Well and Location of Pumping Wells

The influence radius of a pumping well is calculated using the Modified Cooper-Jacob Equation (20 years of pumping duration, 0.8m of drawdown).

Modified Cooper-Jacob equation

$$s = 0.183\frac{Q}{T}\cdot\log\frac{2.25Tt}{r^2S}$$

$s$ : Drawdown(m) $= 0.8$ m

$Q$ : Pumping Volume(m$^3$/d) $= 2200$ m$^3$/d

$T$ : Transmissivity(m$^2$/d) $= 3000$ m$^2$/d

$t$ : Continuous pumping time(day) $= 7300$ days(20 year)

$r$ : Influence radius(m)

$S$ : Storativity $= 1.0\times10^{-3}$

Therefore the influence radius is calculated to be 232m. Supposing the influence radius to be 250m, then the distance between each well is not less than 500m.

Distance between planning wells : more than 500m

Therefore based on water quality and site conditions, the locations of wells were appointed as per Fig.3-1to 3-7.

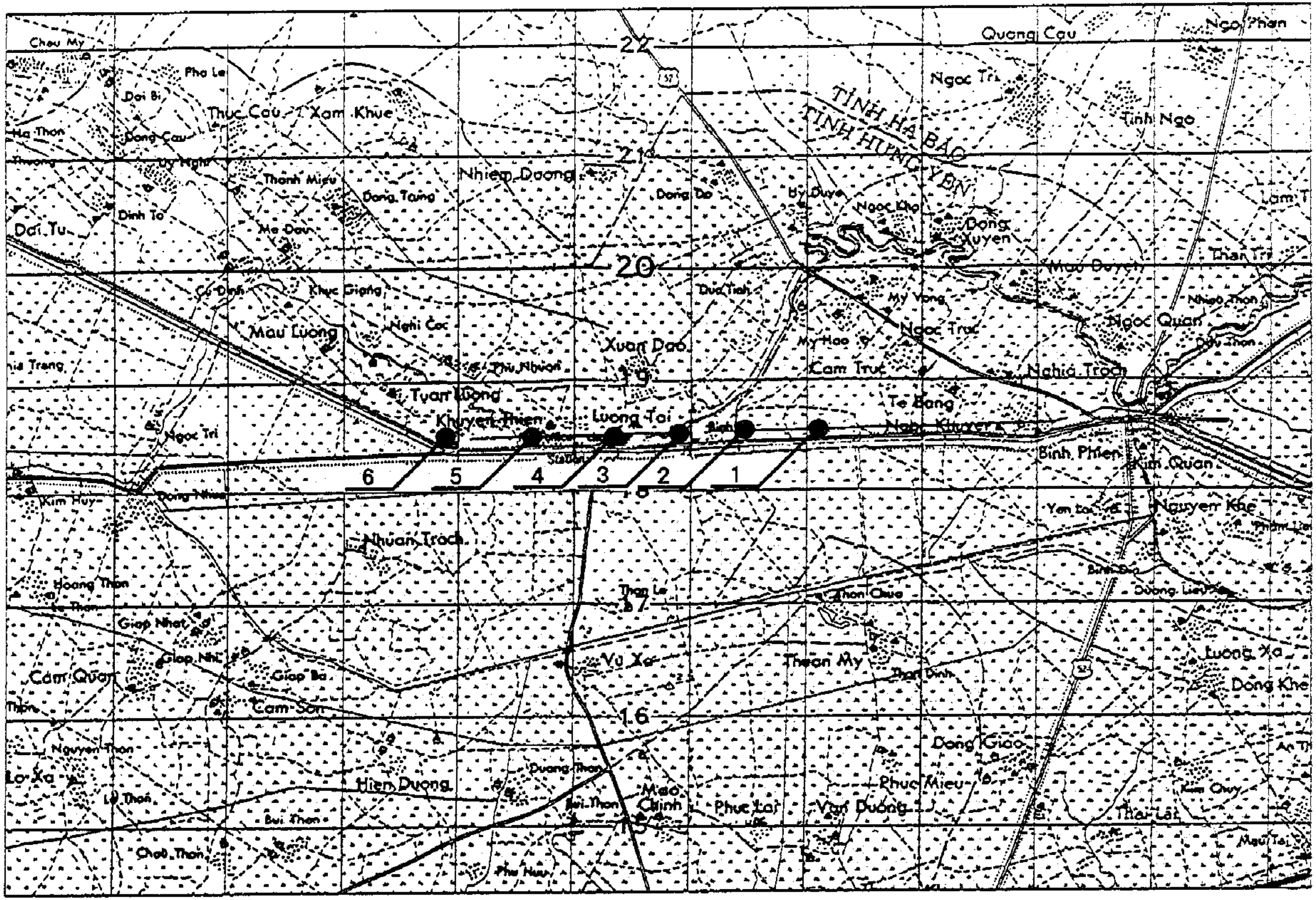



Fig.3-1 Location of Planning Wells

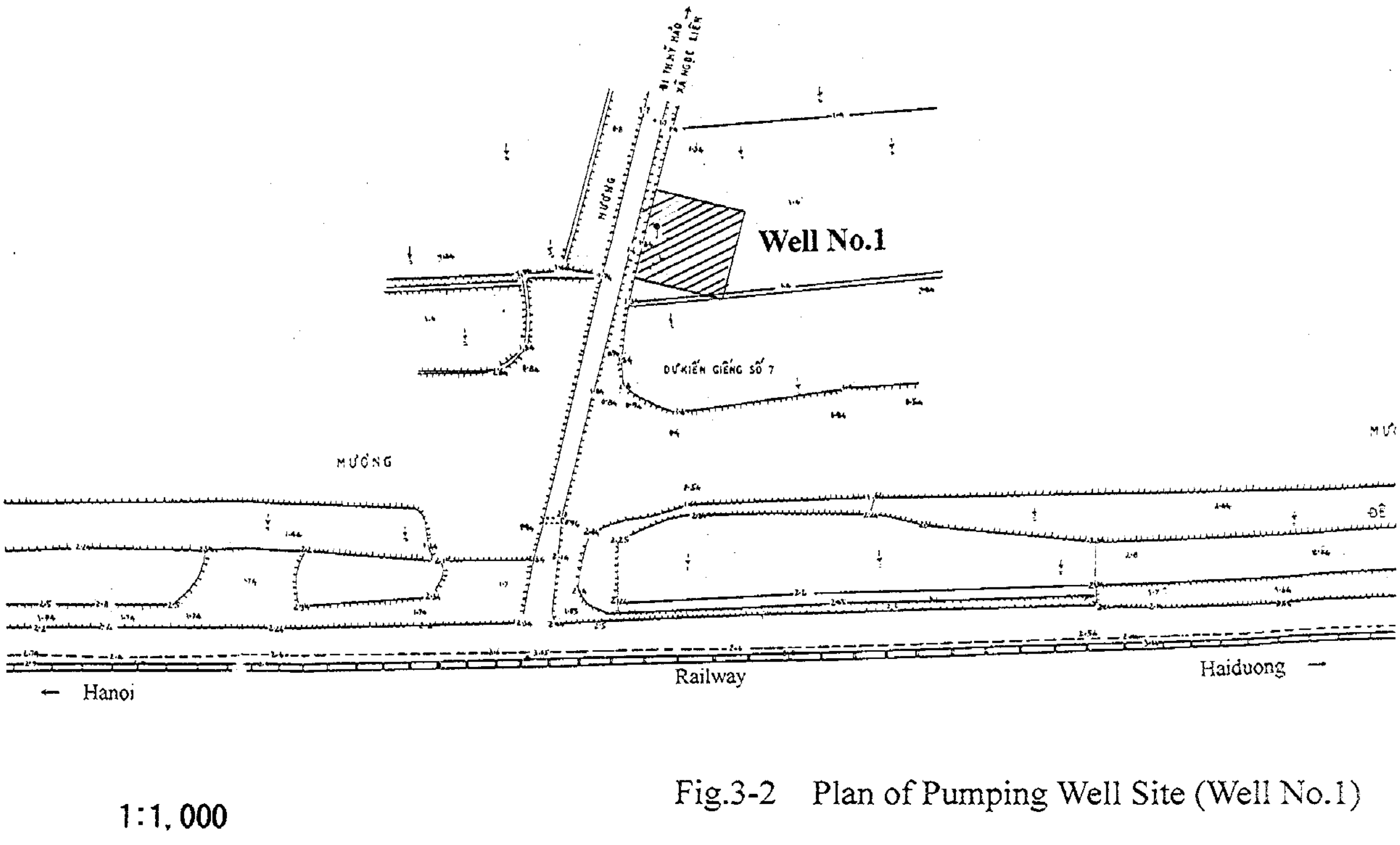
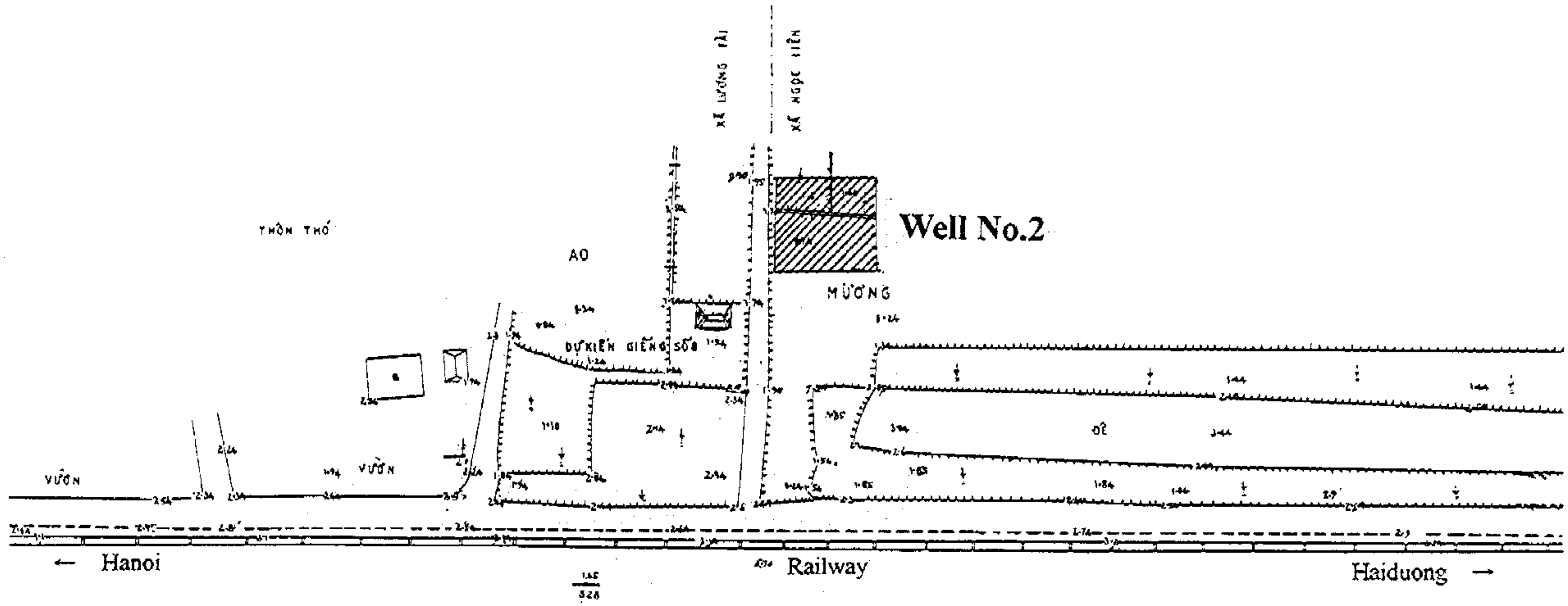
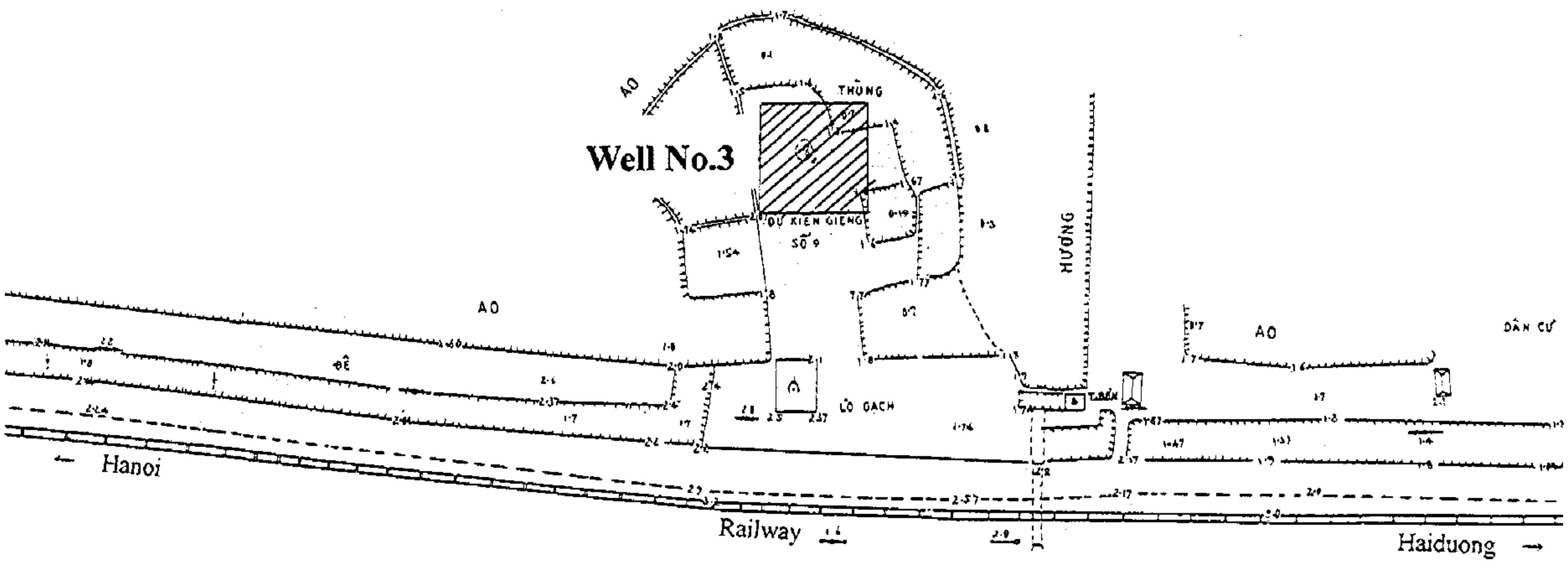



Fig.3-2 Plan of Pumping Well Site (Well No.1)




1:1,000

Fig.3-3 Plan of Pumping Well Site (Well No.2)




1:1,000

Fig.3-4 Plan of Pumping Well Site (Well No.3)

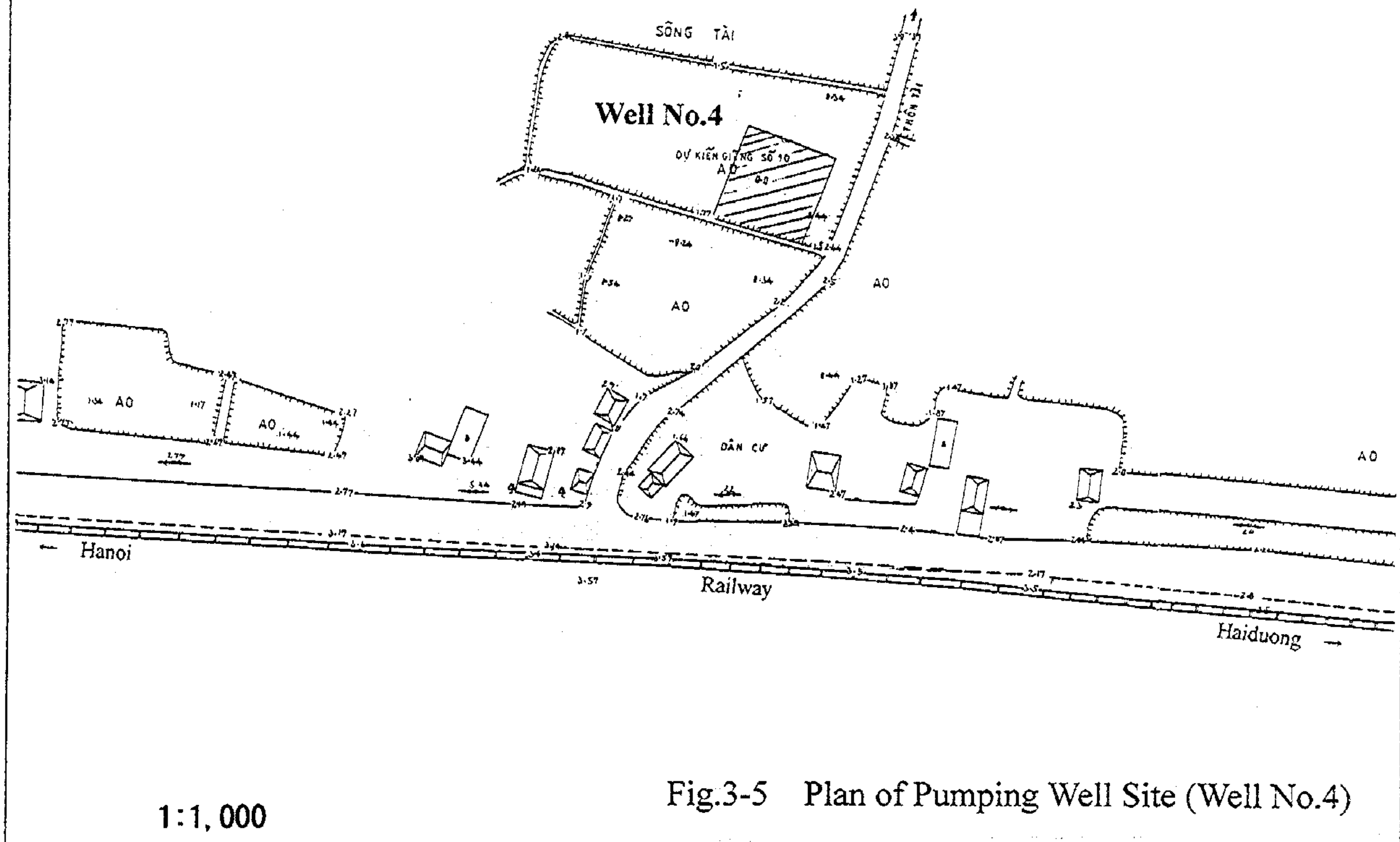
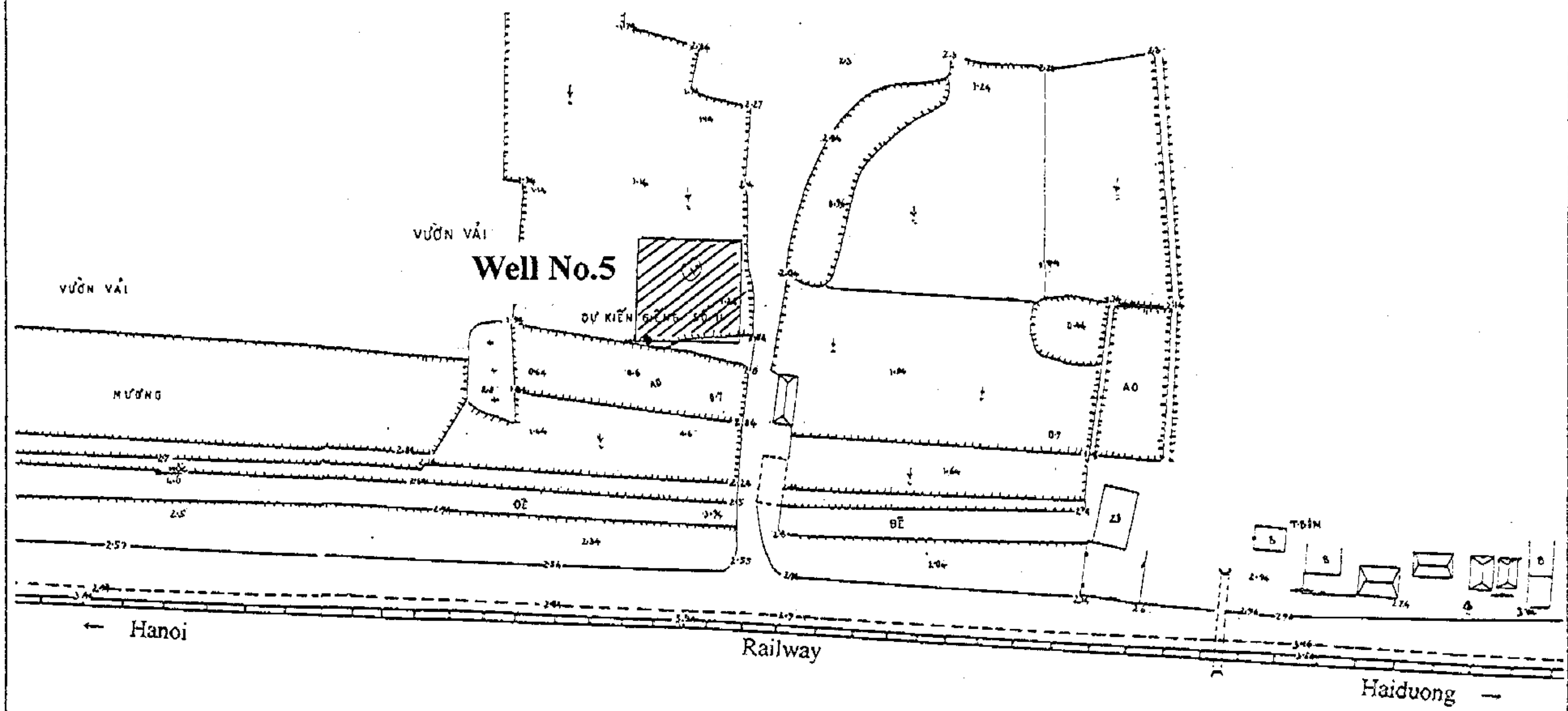
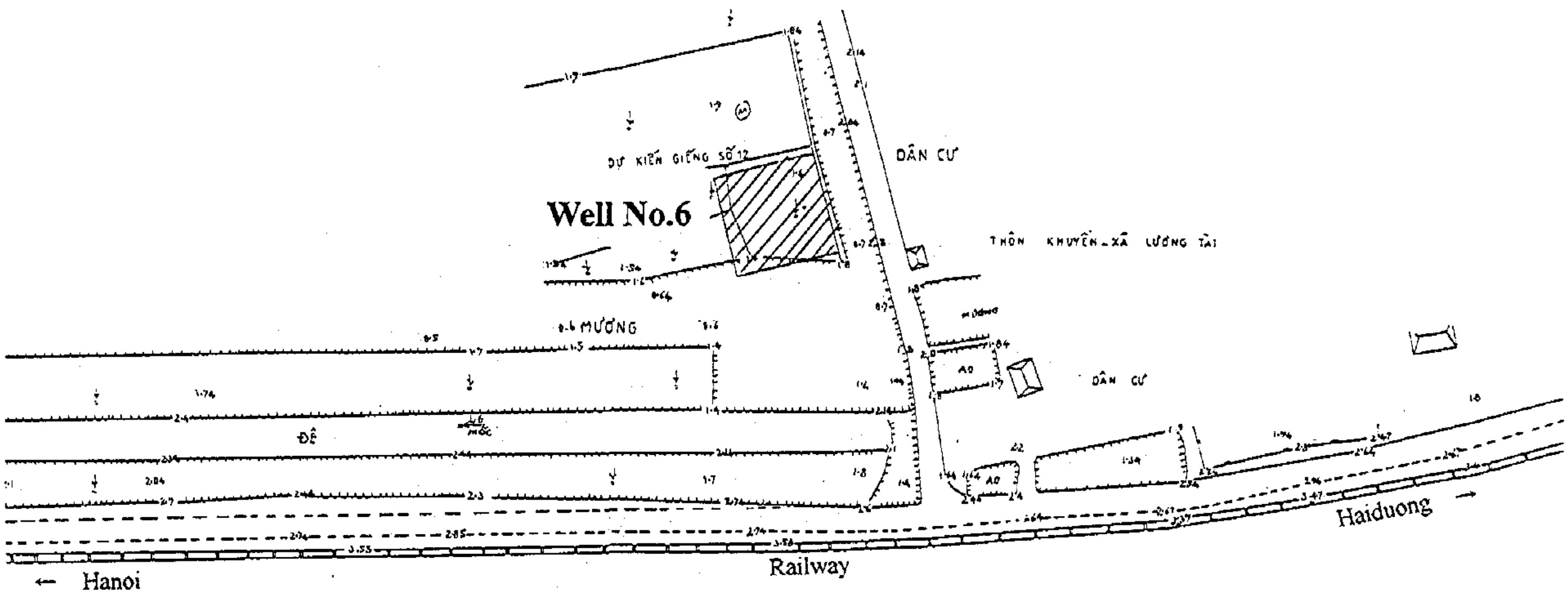


Fig.3-5 Plan of Pumping Well Site (Well No.4)




1:1,000

Fig.3-6 Plan of Pumping Well Site (Well No.5)




1:1, 000

Fig.3-7 Plan of Pumping Well Site (Well No.6)

(3) Estimation of Drawdown for Group Well

In case of wells in the same well field being drawn at the same time, the groundwater level of all wells is affected. Therefore the drawdown of any one point is the sum of each well's influence. Six wells were constructed in approximately a straight line, but in the near future an additional three (3) wells would be constructed so that 9 wells (7 wells and 2 standby) in total would be operated. The drawdown in the area of the 7 wells is calculated thus:

|←600m→|←680m→|←650m→|←590m→|←590m→|←620m→|

O O O O O O O

Well.7 Well.6 Well.5 Well.4 Well.3 Well.2 Well.1

*) Distance form well 1～5 are taken from topographical map. Well 6-7 is estimated.

In the case of all 7 wells being used, maximum drawdown appears at the center of Group Well LK4. The influence of each well at the Well4 is calculated using the aforementioned Modified Cooper-Jacob Equation, and total drawdown is the sum of these influences and the drawdown in Well4.

Drawdown of Well4

| Well. Number | R (m) | T ($m^2$/d) | t (Day) | Q ($m^3$/d) | S | $s_n$ (m) | Well.4 Drawdown $s_0$(m) | Total $\Sigma s_n+s_0$ (m) |
|---|---|---|---|---|---|---|---|---|
| 1 | 1800 | | | | | 0.56 | | |
| 2 | 1180 | | | | | 0.61 | | |
| 3 | 590 | | | | | 0.69 | | |
| 4 | 0 | 3000 | 7300 | 2200 | 0.001 | | 4.58 | 8.27 |
| 5 | 650 | | | | | 0.68 | | |
| 6 | 1330 | | | | | 0.60 | | |
| 7 | 1930 | | | | | 0.55 | | |

*) $s_0$ equal observation value in LK11 which is maximum value by all test wells.

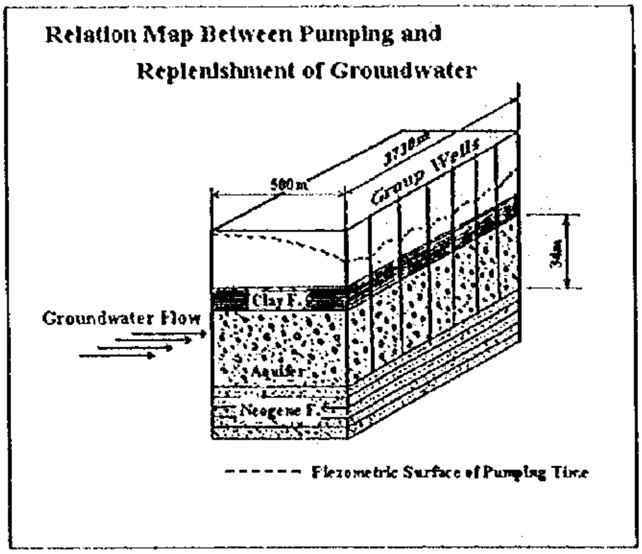
Maximum drawdown in 7 wells using : 8.27m

The depth of a deep well pump must be deeper than the sum of the static water level, the annual water level change and the calculated value (8.27m) expressed above.

(3) Groundwater Development Potential of Development Area

The possibility of the intake of the planned water volume, a 13,900$m^3$/d (using 7 wells in the future) from the water development area, has been investigated.

When the case of pumping the above quantity of water from 7 group wells, as water level draws down, the groundwater flow will be at a right angle to the group wells, as shown in the right sketch figure.




The piezometric groundwater surface lowers because of the Hanoi formation($Q_{II-III}hn1$) which is an object of water development in confined condition. Taking into account of a water influence area of the groundwater basin with a width of 500m, the groundwater flux at pumping time is roughly estimated using Darcy's Law:

$Q=A\cdot k\cdot(h2-h1)/L$ -----Darcy's Law

Q : Groundwater flux

A : Sectional area (Width × Thickness)--3730m × 34m

K : Coefficient of Permeability-----------9.0 × 10-2 cm/sec=77.8m/day

h1: Initial water level

h2: Drawdown at group wells

L : Horizontal distance--------------------500m

The drawdown in the well excluding loss at well is calculated as follows:

Drawdown of group wells : $s = 8.27 - CQ^2 = 8.27 - 1.18 = 7.09$(m)

$CQ^2$ : Well loss

$C=1.31\times10^{-3}$(from result of step drawdown test in LK11)

Q : 30 l/s

Thus the groundwater flux to the pumping area is:

$Q=3730\times34\times77.8\times7.09/500=139,900m^3/d$

When groundwater is supplied on both sides, then the total flux is two times this value, namely 279,800 $m^3/d$. The planned intake water volume (13,700$m^3/d$) is less than 5% of this calculation. Therefore this aquifer has the planned intake water volume capacity.

(5) Quality of Pumping Water

Since water source wells are to be constructed between LK6 to LK11, the design concentrations for the five wells are planned by forecasting concentration for the following items which are over the Drinking Water Standard.

| Item | Well.1 | Well.2 | Well.3 | Well.4 | Well.5 | Well.6 | Using 5 well | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Range | Average |
| PH | 5.6 | | | | | | 5.5~5.7 | 5.6 |
| Cl- mg/l | 230 | 215 | 200 | 185 | 170 | 160 | 186~200 | 193 |
| NH4 mg/l | 2.6 | | | | | | 2.6 | |
| Fe mg/l | 28.0 | 30.8 | 33.6 | 36.4 | 39.2 | 42.0 | 33.6~36.4 | 35.0 |
| MN mg/l | 1.95 | | | | | | 1.95 | |

Because still more well construction is planned by the Vietnamese in the western where the water quality is better, in the future we can expect better overall water quality results than are shown in the above chart.

(6) Consideration of Land Subsidence

Land subsidence due to drawing up is a phenomenon of consolidation of cohesive soil caused by the increase of effective stress due to drawdown.

The groundwater levels in the observation well during the pumping test in LK6 (qp1) and LK6b (qp2) are shown in the following table.

Pumping test at LK6 (qp1 aquifer)

| Well.No. | 6 | 6-1 | 6-2 | 6b | 6b-1 | 6b-2 | 6a | 6a-1 |
|---|---|---|---|---|---|---|---|---|
| Distance (m) | Pumped Well | 20.46 | 40.13 | 3.01 | 21.46 | 41.06 | 57.66 | 62.71 |
| Aquifer | qp1 | qp1 | qp1 | qp2 | qp2 | Qp2 | qh | Qh |
| Drawdown (m) | 5.17 | 1.51 | 0.99 | 0.58 | 0.56 | 0.52 | 0.00 | 0.00 |

Pumping test at LK6b (qp2 aquifer)

| Well.NO. | 6 | 6b | 6b-1 | 6b-2 | 6a |
|---|---|---|---|---|---|
| Distance (m) | 3.01 | Pumped Well | 18.45 | 37.12 | 38.05 |
| Aquifer | qp1 | qp2 | qp2 | qp2 | qh |
| Drawdown (m) | 0.14 | 8.09 | 1.29 | 0.82 | 0.16 |

*Taken from the 'Final Report on Results of Detail Ground Water Exploration Step in Camgiang'

In the case of the pumping test at well LK6, the drawdown of the pumping well is 5.17m, and the drawdown of the qp2 aquifer at LK6b (3.01m far from LK6) is 0.58m. In the same way, in the case of the pumping test at well LK6b, the drawdown of the qp2 aquifer at LK6b-2 is 0.82 and the drawdown of the qh aquifer at LK6a (0.93m far from LK6-2) is 0.16m.

As a result, when the water is drawn up from the qp1 aquifer we can expect the drawdown of the qh aquifer (surface groundwater) to be less than 20cm. Therefore, the increase of effective stress is little and land subsidence will not occur.

# Appendix 7 Operation and Maintenance Cost

## Appendix – 7　Operation and Maintenance Cost

### O&M Cost per Annum

Operation and Maintenance (O&M) cost for water treatment plant and intake/transmission facilities is composed of personnel cost, chemical cost, electric power cost, sludge disposal cost, fuel cost and repair cost. Annual O&M cost is summarized as below:

(× 1000 VND/annum)

| O&M Cost | | | | | | |
|---|---|---|---|---|---|---|
| A<br>Personnel | B<br>Chemical | C<br>Electric Power | D<br>Sludge Disposal | E<br>Fuel | F<br>Repair | Total |
| 156,000<br>4.9 % | 758,671<br>23.9 % | 1,669,883<br>52.7 % | 124,465<br>3.9 % | 48,355<br>1.5 % | 413,606<br>13.1 % | 3,170,980<br>100.0 % |

A. Personnel Cost

[Number of personnel for O&M] × [Average personnel expense per person※1]
= 20persons × 600,000VND/month × 13months※2
= 156,000,000VND

※1 Hai Duong Water Supply Company, record of year 1998
※2 Monthly Salary × 12 + Bonus Salary × 1

B. Chemical Cost

B 1　Chlorine
[Treatment capacity] × ([Dosage for manganese oxidization] + [Dosage for disinfection])
× [Unit price of Chlorine]
= 7,850m$^3$/day × 365days × (2.6mg/l + 2.0mg/l) × 10$^{-3}$kg/g × 7,000VND/kg
= 92,261,050VND

B 2　Lime
[Treatment capacity] × [Lime dosage] × [Unit price of lime]
= 7,850m$^3$/day × 365days × 45mg/l × 10$^{-3}$kg/g × 900VND/kg
= 116,042,625VND

B 3　PAC
([Treatment capacity] × [Dosage for sedimentation]
+ [Waste water capacity] × [Dosage for sludge coagulation])
× [Unit price of PAC]
= (7,850m$^3$/day × 365days × 10mg/l + 1,259m$^3$/day※3 × 365days × 50mg/l) × 10$^{-3}$kg/g
× 10,660VND/kg
= 550,367,805VND

B 1 + B 2 + B 3
= 758,671,480VND

*3 [Discharged sludge from sedimentation] + [Backwashed water]
= ([Dry sludge:919.7kg/day] / [Sludge concentration:0.12%=1.2kg/$m^3$])
+ ([Back washing water per filter bed:82.2$m^3$/day] × 6beds)
= 766.4$m^3$/day + 493.2$m^3$/day
= 1,259$m^3$/day

C. Electric Power Cost

The following formula is applied for calculation:

Σ([Pump/motor output] × 0.8*4 × [Number of units in operation] × [Operation period] × [Unit price per kW])

*4 0.8 : Power coefficient of pump/motor

C1 Intake pump
37kW × 0.8 × 5unit/1.3(Daily average) × 24hr × 365days × 760VND/kWh
= 821,104,000VND

C2 Distribution pump
(55kW × 0.8 × 3unit/1.3(Daily average) + 30kW × 0.8 × 1unit/1.3(Daily average))
× 24hr × 365days × 760VND/kWh
= 798,912,000VND

C3 Sludge transfer pump
11kW × 0.8 × 1unit/1.3(Daily average) × 5hr × 365days × 760VND/kWh
= 9,388,923VND

C4 Others (miscellaneous pumps/motors, Total output:7.6kW)
7.6kW × 0.8 × 24hr × 365days × 760VND/kWh
= 40,478,208VND

C1 + C2 + C3 + C4
= 1,669,883,131VND

D. Sludge disposal cost

[Sludge amount(Solid content:20%)] × [Sludge disposal fee]
= 6.2t/day × 365days × 55,000VND/t
= 124,465,000VND

E. Fuel cost

[Fuel consumption of truck] × [Fuel price]
= 29.44lit/day*5 × 365days × 4,500VND/lit
= 48,355,200VND

*5 On assumption that 4-tons truck drive for four hours per day (2-round trips, 20km distance), fuel consumption are calculated referring to the "Guideline for calculation on construction machinery, 1998".

F. Repair cost (15% of other O&M cost)

(A + B + C + D + E) × 15%
= 413,606,221VND

O&M Cost per Unit Amount of Water

Based on the calculation result of O&M cost for the water treatment facilities, O&M cost per unit amount of water is calculated as below:

| | Production (Distribution) | Accounted-for Water |
|---|---|---|
| Amount of Water per Annum | 2,708,300 $m^3$ | 2,303,150 $m^3$ |
| O&M Cost per Annum | 3,170,980,000 VND | |
| O&M Cost per Unit Amount of Water | 1,170 VND/$m^3$ | 1,377 VND/$m^3$ |

[Production Water per Annum] = [Daily Average Distribution] × 365days
= 7,420 $m^3$/day × 365days
= 2,708,300 $m^3$

[Accounted-for Water per Annum] = [Daily Accounted-for Water (Daily Average Water Demand)] × 365days
= 6,310 $m^3$/day × 365days
= 2,303,150 $m^3$

# Appendix 8 Financial Statement of the Hai Duong Water Supply Company

## Financial Statemtnt of Hai Duong Water Supply Company

### Income Statements

(Unit : Million VND)

| | 1994 | 1995 | 1996 | 1997 |
|---|---|---|---|---|
| **Revenue** | | | | |
| Sales of Domestic Water | 2,746 | 3,556 | 3,775 | 4,646 |
| Sales of Industrial Water | 400 | 495 | 500 | 550 |
| Sales of Public Water | 64 | 135 | 165 | 387 |
| Subsidies | 0 | 0 | 0 | 0 |
| Other Revenue | 0 | 0 | 0 | 0 |
| Total Revenue | 3,210 | 4,186 | 4,440 | 5,583 |
| | | | | |
| **Cost** | | | | |
| Depreciations | 413 | 625 | 667 | 700 |
| Personnel Expense | 390 | 492 | 531 | 1,000 |
| Chemicals | 720 | 836 | 635 | 763 |
| Electricity | 1,118 | 1,350 | 1,405 | 1,770 |
| Repair | 160 | 270 | 394 | 500 |
| Others | 365 | 535 | 584 | 674 |
| Total Cost | 3,166 | 4,108 | 4,216 | 5,407 |
| | | | | |
| **Profit** | 44 | 78 | 224 | 176 |

(Source: Hai Duong Water Supply Company, Finance Department)

# Appendix 9 Calculation on Capacities of the Water Treatment Facilities

## Appendix – 9 Calculation on the Capacities of Facilities

### 1 Intake Facilities

Six intake wells are to be constructed. Intake capacity is 10,200 $m^3/d$ which is discharged from five wells among the six wells where the rest one is standby. Operation hours are to be 24 hours for the five wells.

| | |
|---|---|
| Discharging capacity | : 0.0236 $m^3/s$ (1.417 $m^3/min$) |
| Number of pumps | : 6 units (including one unit for standby) |

### 2 Aeration Facility

Aeration facility is to oxidize the iron (ferrous ion) of the raw water so as to catch the oxidized iron (ferric oxide) by the succeeding process, namely sedimentation and filtration. Oxidation process of iron is as follows:

$$2Fe(HCO_3)_2 + 1/2O_2 + H_2O \rightarrow 2Fe(OH)_3 + 4CO_2$$

Spray nozzle type aeration is to be employed in the Project for the following reasons:

a . Simple mechanism that is easy for maintenance
b . Abundant experience in Viet Nam
c . By using atmospheric air, no extra air supply is required.

Aeration area of the spray nozzle equipment is designed to be more than 0.4$m^2$ per 1$m^3/h$ of water inflow. Water pressure at the outlet of the nozzle is to be 5m.

In addition to oxidation of iron, removal of free carbonic acid in raw water is expected by the aeration process.

Since oxidation of ferrous ion does react not promptly but after a certain period of detention time in the contact basin. The detention time depends on the raw water quality. It ranges in general from 5 minutes to 150 minutes. In the case of this Project, the high content of silica reacts with the iron that makes colloidal floc so that longer retention time is required. This system employs the alkali dosing equipment after aeration that effects to enhance the oxidation reaction by increasing the pH value.

At the bottom of the aerator, gravel layer is to be embedded for removal of ammonia.

## 2 Coagulation and Sedimentation Tank

Since iron contents in the raw water is extremely high, load to filtration process would become quite high when aerated raw water comes in directly to the filter beds. Coagulation / sedimentation tank is to reduce the load to filter beds. The sedimentation process is decided to be up-flow type sedimentation type from the result of the experiment on the water treatment process.

| | |
|---|---|
| Treatment Capacity | : 10,200m$^3$/d (7.1m$^3$/min) |
| Separation Area | : W5m×L21m×2tanks=210m$^2$ |
| Surface Load | : 7.1m$^3$/min／210m$^2$=33.8mm/min |

## 4 Filtration

Since it was found by the model test that approximately 90 % of iron was removed through coagulation and sedimentation process, dual-layer sand filter: iron removal and manganese removal is proposed. Iron will be filtered by anthracite layer of 30 cm in depth, and manganese be filtered by the manganese sand sublayer of 70 cm in depth.

| | |
|---|---|
| Treatment Capacity | : 10,200m$^3$/d (7.1m$^3$/min) |
| Filtration Area (One bed) | : 3.8mW×3.8mL=14.44m$^2$ |
| Total Filtration Area | : 14.44m$^2$×6beds=86.6 m$^2$ |
| Filtration Rate | : 10,200 m$^3$/d／86.6 m$^2$=117.8 m/d |
| Filtration Rate in Washing a Filter | : 117.8 m/d×6／5=141.4 m/d |
| Backwashing Water Volume | : 14.44 m$^2$×0.6m/min=8.7m$^3$/min |
| Surface Washing Volume | : 14.44 m$^2$×0.07m/min=1.01m$^3$/min |

## 5 Distribution Tank

Distribution tank is to be divided into two by the section wall.

| | |
|---|---|
| Dimension | : 24.9mW×30.3mL×4.7mH |
| Effective Capacity | : 2,400m$^3$ |
| Detention Time | : Approx. 5.6 hours |
| Number of Tank | : 1tank |

## 6 Distribution Pumps

Four distribution pumps are to be installed. Main specification is as follows:

| | |
|---|---|
| Pump Capacity | : 3.55 m$^3$/min |
| Number of Pumps | : 3 units (including 1unit standby) |
| Pump Capacity | : 1.95 m$^3$/min |
| Number of Pumps | : 1unit |

## 7 Chlorinator

Chlorine is used for removal of manganese. Injection point is at the inlet of the filtration facility. In addition, chlorine is also used for disinfection. Injection point is at the outlet of the filtration facility. Dosing rate is calculated as follows:

Removal of Manganese : 10,200$m^3$/d × 2.6mg/l = 26.52kg/d
Disinfection : 10,200$m^3$/d × 2.0mg/l = 20.4kg/d

Total Dosage : 47 kg/d

Capacity of the Chlorinator : 2000 g/h
Number of the Chlorinator : 2 units (including 1unit standby)

## 8 Chemical Dosing Equipment

Lime dosing equipment for pH control and PAC dosing equipment for coagulation process are installed.

### (1) Lime Dosing Equipment

Dosage : 45mg/l
Lime Quantity : 10,200 $m^3$/d × 45mg/l = 459 kg/d (790lit as powder)

Injection Method
Measured lime powder is to be put into the mixing chamber. The saturated lime solution is to injected by the injection pump.

Volume of Solution : 450 $m^3$/d
Dimension of Mixing Chamber : 2.0mW × 3.2mL × 1.2mH = 7.6$m^3$/tank (24min)
Number of Tanks : 2 tanks

### (2) PAC Dosing Equipment

Dosage : 10mg/l
PAC Quantity : 10200 $m^3$/d × 10mg/l = 102 kg/d for coagulation/sedimentation
(75kg/d) for Sludge Treatment(mentioned later)

Injection Method
Approx. 10% of PAC solution is prepared by the PAC dissolving tank in which PAC powder is poured and mixed. The PAC solution is injected by the injection pump to the injection point.

Volume of Solution : 1.5 $m^3$/d
Dimension of the Dissolving Tank : 2.0mW × 3.2mL × 1.2mH = 7.6$m^3$/tank (for 5 days)
Number of Tanks : 2 tanks

## 9 Sludge Treatment Facility

Amount of sludge generated at the water treatment process is as follows:

| | |
|---|---|
| Fe | 10,200 $m^3/d \times 35mg/l \times 106.8/55.8 = 683.3kg/d$ |
| $Ca(OH)^2$ | 10,200 $m^3/d \times 45mg/l \times 0.8 \times 100.1/74.1 = 496.0kg/d$ |
| PAC | 10,200 $m^3/d \times 10mg/l \times 0.1 \times 156/102 = 15.6kg/d$ |
| Total | $683.3kg/d \times 496.0kg/d + 15.6kg/d = 1,195kg/d$ |

Drain water is transferred from the followings:

a. Drain water of the coagulation and sedimentation tank

| | | |
|---|---|---|
| Sludge content | : 0.12%(1.2 $kg/m^3$)···assumption | |
| Amount as dry sludge | : 1,195kg/d | |
| Volume of drain water | : 1,195kg/d ÷ 1.2 $kg/m^3$ = 996 $m^3/d$ | ① |

b. Drain water of the filter washing

| | |
|---|---|
| Filter area per bed | : $3.8m \times 3.8m = 14.44m^2$ |
| Number of filters | : 6 beds |
| Drain water per bed : | |
| Detention water in bed | $14.44m^2 \times 1.0m = 14.44m^3$ |
| Washing volume | $14.44m^2 \times 0.67m/min \times 7min = 67.7m^3$ |
| Total drain water | $82.2m^3$ |

Assuming that one filter bed are washed everyday:
Total drain water of filter washing per day : $82.2\ m^3 \times 6 = 493\ m^3$ ②

Total drain water to be treated is:
①+② = 1,489$m^3$ (1500 $m^3$)

### (1) Drain Water Reservoir Tank

One drain water reservoir tank is to be constructed so as to transfer the drain water constantly to the sludge thickener. The capacity of the reservoir tank is to be capable of two-times drain water volume of filter washing.

| | |
|---|---|
| Effective capacity | : 360$m^3$ |
| Transferring volume | : Assuming 1,500 $m^3$ of drain water to the thickener for 20 hours,<br>1,500 $m^3/(20 \times 60)min = 1.25\ m^3/min$ |

### (2) PAC Dosing Equipment (both as raw water coagulation and sludge coagulation)

It is known that iron sludge is difficult to sediment without dosing coagulant such as PAC. PAC dosing equipment is to installed both for aforementioned raw water coagulation process and this sludge coagulation process. The injection point is to be the transfer pipe between drain water reservoir and sludge thickener.

(3) Sludge Thickener

The transferred sludge from the drain water reservoir is thickened in the sludge thickener. The thickened sludge is to transferred to the sludge drying bed. The supernatant is returned to the sedimentation/coagulation tank.

| | |
|---|---|
| Upflow velocity | : 7mm/min |
| Treatment capacity | : 1.25 $m^3$/min |
| Separation area required | : 1.25 $m^3$/min／0.007m/min＝178 $m^2$ |
| Design area of the sludge thickener | : 10m × 10m × 2 tanks＝200 $m^2$ |

(4) Sludge Drying Bed

The thickened sludge transferred from the sludge thickener is to be dried at the sludge drying bed. Assuming the content of the thickened sludge is 1% (10 kg/$m^3$), volume of the thickened sludge per day is calculated as follows:

$\{1{,}195\text{kg/d}+(0.153\times 75\text{kg/d})\}\ \times 1/10\ \text{kg/m}^3=121\ \text{m}^3$

The number of drying beds is to be seven so that capacity per bed is capable of approx. six days of the transferred 1% sludge.

On assumption that surface load is 30kg/ $m^2$, required area per bed is:

$1{,}206\text{kg/d}\times 6\times 1/30\ \text{kg/m}^2=241\ \text{m}^2$

Thickened sludge in the sludge thickener is transferred at every two days. The transferring time is estimated to be approx. five to six hours. After having transferred the six-days sludge to the one sludge drying bed, the next drying bed begins to be transferred.

The drying bed which have been transferred of the six-days sludge is to start drain process, drying process and sludge removal to the disposal site. It is planned to use the drying bed every 36 days since one drying bed is assumed to have completed one cycle of sludge drying and disposal.

(5) Sludge Transfer Pump

Two sludge transfer pumps are to be installed. Sludge amount of two-days which is reserved in the drain water reservoir tank is as follows:

$121\text{m}^3/\text{d}\times 2\text{d}=242\text{m}^3$

This amount of sludge is to be transferred in five (5) hours.

$242\text{m}^3\times 5\text{h}=48.4\text{m}^3/\text{h}$

| | |
|---|---|
| Pump Capacity | : 50$m^3$/ h × 10 m × 11kW |
| Number of Pumps | : 2 units (including 1 unit standby) |

# Appendix 10 Model Experiment for Water Treatment Process

# EXPERIMENT FOR WATER TREATMENT PROCESS

## 1. Purpose of Experiment

The water quality observed in thee (3) test wells drilled in the proposed wellfield indicated that the groundwater in the wellfield contains high concentration of iron (40 - 50 mg/l), manganese (1.5 - 2.5 mg/l), ammonia (1.5 - 3.5 mg/l) and chloride (145 - 320 mg/l).

The purpose of this experiment is to collect the records and data for the study and investigation on the most appropriate treatment process to remove the iron, manganese, and ammonia from the lifted water. The water of test well No. 11 was used for the experiment because the observed water quality was considered to be the most suitable for this experiment.

## 2. Method of Experiment

The following two (2) methods were employed for the experiment; physicochemical and biological treatment processes.

(1) Physicochemical Treatment Process

The results of the coagulation test carried out in August 1997 indicate that sediments of iron hydroxide is found adding coagulant in pH control process and iron concentration was observed below 0.3 mg/l in top of the water. Therefore, the process consisting of pH control, coagulation and rapid filtration is considered as a basic treatment process with the dual-layer filtration of manganese sand for removing manganese. The following cases were considered in the experiment for physicochemical treatment.

| | |
|---|---|
| A-1 | Aeration - Rapid Filtration (Sand) - Filtration (Manganese Sand) |
| A-2 | pH Control - Coagulation - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand) |
| A-3 | pH Control - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand) |
| A-4 | pH Control - Coagulation - Aeration - Separate Filtration (Iron and Manganese) |
| A-5 | Chlorination - Coagulation - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand) |
| RA-2 | Improved Case A-2 |

Since it was found that the records observed in the case A-2 indicated good results in the effect of treatment, the case RA-2 which is considered as a modified case A-2 was taken up to confirm and grasp further details of the effects.

(2) Biological Treatment Process

In addition to the above physicochemical treatment process, the slow filtration process which is considered as an easy operational and chemical free treatment method was also employed for the experiment to confirm the removal effect of the contents. The following cases were considered in the experiment for biological treatment.

| | |
|---|---|
| B-1 | Aeration - Coarse Sand Filter - Slow Filtration |
| MB-1 | Improved Case B-1 Adding Slow Filtration and Adjusting Filtration Rate |
| MB-1(2) | Improved Case MB-1 Changing Location of Aeration |

As a result of the case B-1, it was found that its treatment effect was not enough, the cases MB-1 and MB-2 were taken up improving the process of the case B-1.

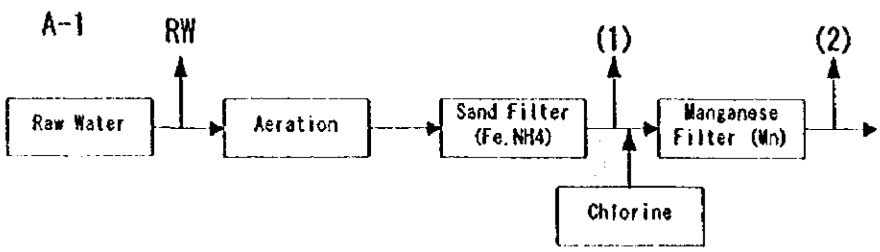
## 3. Period of Experiment

The experiment was carried out from June 1 to September 5, 1997 after the preparatory works in May 1995.

## 4. Results of Experiments

(1) Physicochemical Treatment Process

A-1: Aeration - Rapid Filtration (Sand) - Filtration (Manganese Sand)




| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 52 | 1.91 | 2.88 |
| | Aeration/Filtration (1) | 23 | 1.78 | 0.61 |
| | Manganese Filter (2) | 15 | 1.62 | 0.35 |
| Chlorine Demand | | 20 | 2 | 3 |

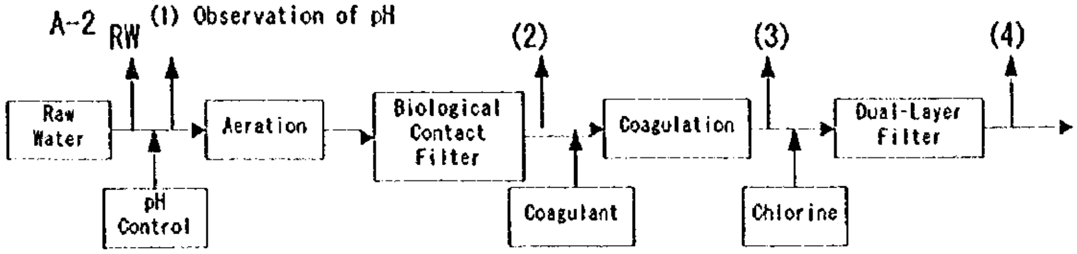
The oxygen dissolved in the raw water was measured in a range from 0.2 to 1.5 mg/l, but it rose to 4. - 5 mg/l after the aeration process. The iron concentration is, on the contrary, increased and it carries over the iron-ammonium filter resulting in the exceeding consumption of chloride for oxidation of manganese. Therefore, inoxidized manganese remains in the filtrated water.

It is necessary to feed 25 mg/l of chloride to remove iron, manganese and ammonium. Since the pH value of the treated water is measured as high as 4.35 though it was reduced from 6.06 of the raw water, alkali agent has to be dosed to further reduce it to the acceptable value in the standard.

The following measures are considered to facilitate the removal effect.

① To increase chloride dosing to 30 mg/l
② To provide aeration enough to oxidize iron of 50 mg/l
③ To add a coagulation process to remove iron

A-2: pH Control – Coagulation - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand)




| | Measured Items | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 54 | 1.91 | 2.90 |
| | Aeration/Biological Contact (2) | 36 | 1.68 | 0.62 |
| | Coagulation (3) | 4.27 | 1.19 | 0.13 |
| | Dual-Layer Filter (4) | 0.22 | 0.01 | 0.00 |

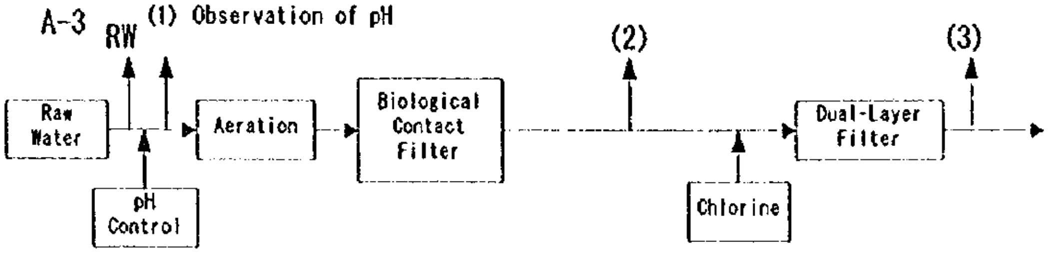
The oxygen dissolved in the raw water is measured in a range between 0.2 and 1.5 mg/l, while in a range between 4 and 5 mg/l after aeration process which is considered to be almost same as the records obtained in the case A-1. About 33 % and 79 % of iron and ammonium were removed in average, respectively, after the biological contact aeration process, but only 12 % of manganese were removed.

About 59 % of iron and 17 % of ammonium were removed in average after the coagulation process which doses PAC after the biological contact aeration process, but only 26 % of manganese was removed. Then, 7.5 % of iron, 62 % of manganese and 5 % of ammonium were removed by the chlorine dosing of 4 mg/l and the dual-layer filtration of manganese sand and anthracite.

Since the specific gravity of iron floc is light, some floc flows into the filter at the initial stage when the blanket has not yet formed. The backwash has to be made as frequent as every eight (8) hours during the initial stage, but when it grows to two (2) m thick the frequency becomes to be 48 hours. Therefore, blanket type of filter is considered suitable for this treatment.

About 4 mg/l of chlorine was consumed for this treatment process, and out of this volume about 2.6 mg/l and 1 mg/l were consumed for oxidation of manganese and ammonium, and iron, respectively, resulting in the residual chlorine of 0.3 - 0.5 mg/l. As a result, this treatment process of this case is considered suitable.

[A-3: pH Control - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand)]

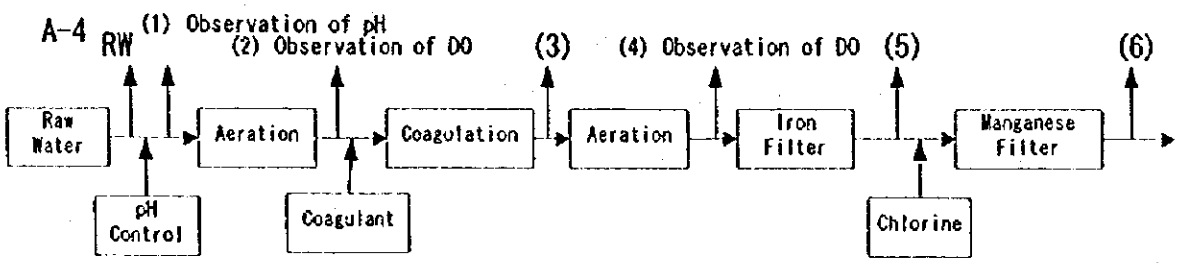



| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 47 | 1.82 | 3.07 |
| | Aeration/Biological Contact (2) | 44 | 1.83 | 0.50 |
| | Dual-Layer Filter (3) | 30 | 0.37 | 0.44 |

In this treatment process, the treatment effect was reduced remarkably, because the coagulation process which contributed 60 % of iron removal effect in the process of case A-2 was omitted. Throughout the whole process of this case, 36 % and 80 % of iron and manganese were removed. It is possible to remove manganese by increasing dosing volume of chlorine, but incapable of removing iron to the acceptable level of water quality.

The backwash with a frequency of 2 - 3 times a day is considered to be necessary in the iron filtration, and it is not proposed to remove iron more than 10 mg/l considering the frequency of necessary backwash.

[A-4: pH Control - Coagulation - Aeration - Separate Filtration (Iron and Manganese)]



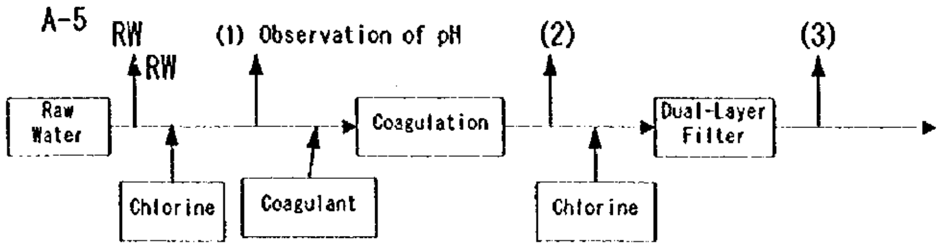

| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 46 | 1.87 | 1.79 |
| | Aeration/Coagulation (3) | 3.22 | 1.09 | 0.21 |
| | Iron Filter (5) | 1.70 | 0.85 | 0.10 |
| | Manganese Filter (6) | 0.07 | 0.02 | 0.01 |

The oxygen dissolved in raw water was measured in a range between 0.6 and 1.5 mg/l, and it rose to a range between 4 and 5 mg/l.

About 91 % of iron and 90 % of ammonium were removed in average by aeration and coagulation process after pH control, while only 42 % of manganese were removed. By the sand filtration process, 3.3 % of iron and 6.1 % of ammonium were removed in average, but the manganese removal is measured as low as 12.8 %. By the chlorine dosing and the manganese sand filtration processes, 3.5 % of iron and 5 % of ammonium were removed. As for the manganese removal, 42 % was removed and is considered acceptable.

As a result, comparing those results obtained in the experiments of cases A-2 and A-4, the effect of iron removal in case A-2 is considered more remarkable than in case A-4.

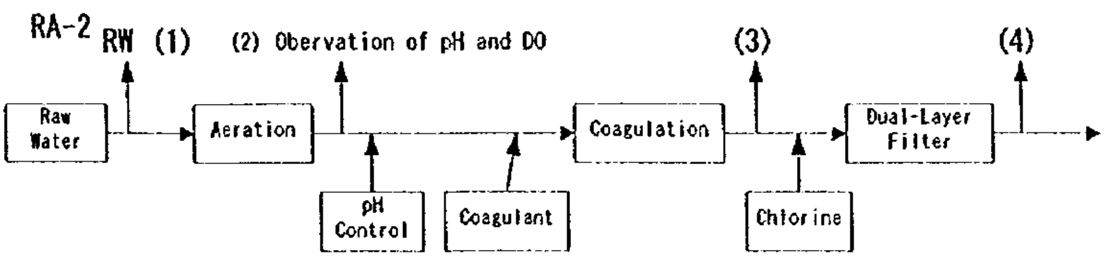
A-5: Chlorination - Coagulation - Rapid Filtration (Dual-Layer of Anthracite and Manganese Sand)




| | Measured Items | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 49 | 1.77 | 1.47 |
| | Aeration/Coagulation (2) | 36 | 1.54 | 0.96 |
| | Dual-Layer Filter (3) | 22 | 1.07 | 0.64 |
| Chlorine Demand | | 29 | 1.4 | 5.1 |

In the treatment process of this case, the aeration process is omitted from that of case A-2 and 40 mg/l of chlorine were fed for pH control process instead of NaOH. About 55 % of iron, 40 % of manganese and 56 % of ammonium were removed in average. To remove these contents to the acceptable level, about 35 mg/l of chlorine dosing is considered to be required. Further, pH value measured to be 6.02 in raw water was reduced to 5.24 in the treated water by this process, but it is necessary to feed alkali agent to reduce it to the level acceptable for drinking water.

RA-2: Improved Case A-2



| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 45 | 1.73 | 2.28 |
| | Aeration/Coagulation (3) | 3.29 | 1.33 | 0.14 |
| | Dual-Layer Filter (4) | 0.08 | 0.02 | 0.01 |

The oxygen dissolved in raw water was measured in a range from 0.1 to 0.7 mg/l and it was increased to a range from 4 to 6 mg/l, which is considered to be same extent of effect as the case A-2. After the pH control, aeration and PAC dosing processes, 91 % of iron and 94 % of ammonium were removed, while only 23 % of manganese were removed. By the 4.3 mg/l of chlorine dosing and the dual-layer filtration of manganese sand and anthracite, 7 % of iron and whole of manganese and ammonium were removed. As same as the case A-2, iron floc flows into the sand filtration because of light specific gravity and frequent backwash is required at the initial stage when the blanket had not yet been formed. However, the time interval of backwash is increased as the blanket grows, and when it grows to two (2) m thick the interval becomes 48 hours. Therefore, the blanket type of sedimentation tank is considered suitable.

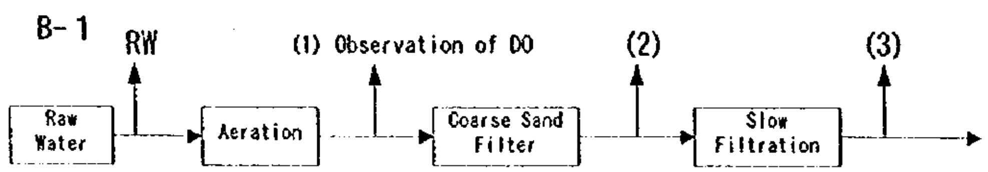
The total volume of chlorine dosed was measured to be about 4.3 mg/l, and out of this volume 2.6 mg/l and 2 mg/l were consumed for the oxidation of manganese and ammonium, and iron, respectively. The residual chlorine was measured to be in a range between 0.3 and 0.5 mg/l. As for manganese dosing more than 5 mg/l of chlorine satisfied removal, the criteria for drinking water standard of 0.10 mg/l.

70 mg/l of NaOH were fed in average of for controlling pH value in this case. It is possible to reduce this volume to 40 mg/l, but the pH value is increased to about 6.0 requiring further addition of dosing to reduce it to the acceptable level for drinking water. 3 - 5 mg/l of PAC is considered enough for the coagulation process if pH control is made properly.

The treatment process experimented in this case is considered suitable.

(2) Biological Treatment Process

B-1: Aeration - Coarse Sand Filter - Slow Filtration




① Filtration Rate: Filter (1) = 100 m/day, Filter (2) = 7.9 m/day

| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 53 | 1.94 | 2.49 |
| | Aeration/Sand Filtration (2) | 24 | 1.70 | 0.23 |
| | Slow Filtration (3) | 3.67 | 2.21 | 0.06 |

The oxygen dissolved in raw water was 0.2 - 1.3 mg/l, and it is improved for 4 - 5 mg/l after the aeration process, which is considered almost same effect as in the cases for the physicochemical treatment process.

By the aeration and coarse sand filtration processes (Filter (1)), 55 % of iron and 91 % of ammonium were removed in average, but most of manganese was not removed. In the slow filtration process (Filter (2)), 38 % of iron and 7 % of ammonium were removed, but manganese was not removed or increased. It is considered impossible to treat iron by this process, because manganese having bivalent ion is prevailing.

The alkali control is required to reduce the pH value of 4.76 measured in the treated water to the acceptable level for drinking water, though it was improved from 6.07 of raw water through this process.

② Filtration Rate: Filter (1) = 70 m/day, Filter (2) = 5.5 m/day

| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 51 | 1.89 | 3.07 |
| | Aeration/Sand Filtration (2) | 17 | 1.72 | 0.49 |
| | Slow Filtration (3) | 7.12 | 1.96 | 0.26 |

The oxygen dissolved in the raw water was measured to be 1.2 - 1.3 mg/l, and it was improved for 4 - 5 mg/l after the aeration process as same as in the cases for the physicochemical treatment process.

By the aeration and coarse sand filtration processes (Filter (1)), 67 % of iron and 84 % of ammonium were removed in average, but no manganese was removed. Through the slow filtration process (Filter (2)), 20 % of iron 7 % of ammonium were removed in average, but most of manganese was left or increased.

The pH value was improved from 6.05 in raw water to 4.8 in treated water, but it is necessary to provide the alkali controlling process to reduce it to the acceptable level for drinking water.

③ Filtration Rate: Filter (1) = 30 m/day, Filter (2) = 2.4 m/day.

| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 47 | 1.95 | 2.55 |
| | Aeration/Sand Filtration (2) | 19 | 1.94 | 0.66 |
| | Slow Filtration (3) | 8.51 | 1.93 | 0.42 |

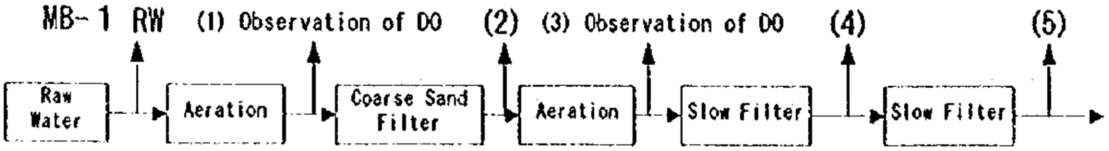
The dissolved oxygen in the raw water was measured in a range from 0.9 to 2.1 mg/l, and was improved to a range from 3 to 5 mg/l after the aeration process as same as in the cases for the physicochemical treatment process.

By the aeration and coarse sand filtration processes (Filter (1)), about 60 % of iron and 91 % of ammonium were removed in average, but no manganese was treated. In the slow filtration process (Filter (2)), about 22 % of iron and 9 % of ammonium were removed, while manganese was not removed or increased.

The pH value was improved from 6.01 in raw water to 4.68 in treated water, but it is necessary to provide the alkali controlling process to reduce it to the acceptable level for drinking water.

Based on the above three (3) trials changing the filtration rate both in Filter (1) and Filter (2), high filtration rate results in better removal effect in Filter (1), and in Filter (2) slower rate results in better effect.

**MB-1: Improved Case B-1 Adding Slow Filtration and Adjusting Filtration Rate**




| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 46 | 1.78 | 2.18 |
| | Aeration/Filter (1) (2) | 16 | 1.77 | 0.62 |
| | Filter (2) (4) | 5.68 | 1.80 | 0.20 |
| | Filter (3) (5) | 1.32 | 1.77 | 0.09 |

A slow filter was added at the last stage and an aeration process was added between the first and the second filters. The filtration rates were set for each filter as follows:

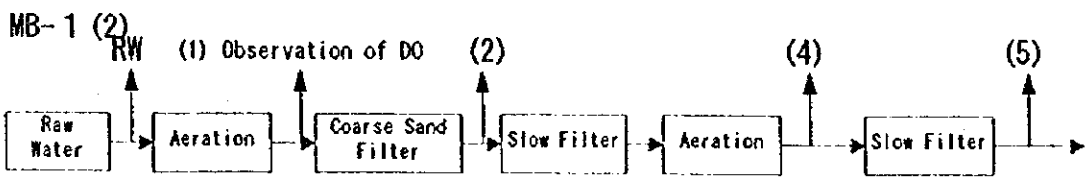
- Filter (1) = 70 m/day
- Filter (2) = 20 m/day
- Filter (3) = 20 m/day

As same as the effects obtained in the cases for the physicochemical treatment process, the dissolved oxygen measured in a range from 0.1 to 0.7 mg/l in the raw water was reduced in a range from 4 to 6 mg/l after the aeration process.

By the aeration and coarse sand filtration processes (Filter (1)), about 65 % of iron and about 72 % of ammonium were removed in average, but no manganese was treated. In the slow filtration process (Filter (2)), 22 % of iron and 19 % of ammonium were removed, but manganese were not removed or increased as same as case B-1. In Filter (3) for slow filtration, 9 % of iron and 5 % of ammonium were removed, but no manganese was treated.

As for the pH values, it was improved from 6.0 in raw water to 3.79 in treated water, but some extent of alkali control is considered to be required to reduce it to the acceptable level for drinking water.

**MB-1(2):Improved Case MB-1 Changing Location of Aeration**



| Measured Items | | Iron | Manganese | Ammonium |
|---|---|---|---|---|
| Sampling | Raw Water (RW) | 47 | 1.80 | 2.30 |
| | Aeration/Filter (1) (2) | 20 | 1.83 | 0.21 |
| | Filter (2) (4) | 10.61 | 1.80 | 0.21 |
| | Filter (3) (5) | 5.34 | 1.84 | 0.03 |

The experiment for this case was carried out adding an aeration process before the 3rd. Filter and setting the filtration rates as follows:

- Filter (1) = 105 m/day
- Filter (2) = 30 m/day
- Filter (3) = 30 m/day

The oxygen dissolved in the raw water was measured in a range between 0.1 and 0.9 mg/l, and it was improved in a range between 3 and 5 mg/l after the aeration process.

By the aeration and the coarse sand filtration processes (Filter (1)), about 57 % of iron and about 91 % of ammonium were removed in average, but no manganese was treated. In the slow filtration process by Filter (2), about 20 % of iron and 6 % of ammonium were removed in average, but manganese was not removed or increased as same as case B-1. In the process of slow filtration by Filter (3), about 11 % of iron and most of ammonium were removed, but no manganese was treated.

The pH value was improved from 5.87 in raw water to 3.59 in treated water, but it is still required to provide alkali controlling process to reduce it to the acceptable level for drinking water.

Comparing the filtration rate and effect of iron removal, the worse the removal effect of iron becomes, the slower the filtration rate in the slow filtration process. The most suitable rate of filtration is considered to be in a range from 7 to 10 m/day comparing the results of each case.

## 5 Conclusion

Considering the results obtained through the experiments mentioned above, the physicochemical treatment process is proposed to be adopted for the project, and the proposed treatment process is discussed below.

| | |
|---|---|
| ① Aeration Process: | Removal of free carbon dioxide and oxidation of ferrous in the water |
| ② pH Control: | Growth of floc |
| ③ Coagulation and Sedimentation Process: | Removal of iron and ammonium |
| ④ Rapid Filtration Process: | Removal of iron and manganese |
| ⑤ Chlorination Process: | Disinfection of water |

The details are discussed below.

(1) Aeration Process

The raw water is considered to include 80 - 120 mg/l of free carbon dioxide and 40 mg/l of silica. Generally, the iron contents react with the silica to form iron suspension by aeration. This phenomenon is remarkable when the content of silica exceeds 40 mg/l, and is confirmed in this experiment too. The settled silica is usually considered to be obstacle to proper operation of filtration process. It is proposed to provide an aeration process in pumping facility from the wellfield to the transmission facility in order to remove free carbon dioxide and to reduce the corrosive nature. The results of experiment indicate that the iron suspended by aeration is easily coagulated by pH control with alkali agent.

(2) pH Control

Generally, the coagulation effect is improved by providing pH control process as a pre-treatment of coagulation process. According to the results of experiment, the coagulation effect is increased when the pH value of the raw water is adjusted at between 6.8 and 7.0.

The effect of pH controlling agent was compared between caustic soda and lime, and as a result, lime is found to facilitate the effect remarkably. The slacked lime is locally available with affordable price.

(3) Coagulation Process

The purpose of coagulation process is to remove iron and manganese, and PAC available in local market with affordable price is proposed to be applied for the Project.

The coagulant dosage is proposed to be 5 - 3 mg/l, and it was found that the effect of the baffling type flocculation basin with about 20 min. of mixing is most suitable. Since the specific gravity of iron is light, vertical flocculation is recommended. The effect of coagulation was measured considering the frequency of required backwash in the rapid filter. The interval period of backwash is as short as 8 hours when the blanket of floc has not yet been formed in the sedimentation tank. The thicker the formed blanket becomes, the longer interval of backwash is required. When the blanket grows 2 m thick, the required interval becomes as long as 48 hours. With this thickness of floc blanket, the absorption effect of floc becomes maximum. The ammonium is removed with this process completely.

(4) Rapid Filtration Process

The rapid filtration process is provided to remove the iron and manganese which could not been treated in the coagulation process. Chloride dosage is made to activate the manganese sand. Since the removal effect of iron by sand filtration is low according to the experiment results, the dual-layer filtration of sand and manganese sand is proposed for the Project. The sand filtration is provided for removing iron, while the manganese sand filtration is for removing manganese. Since most of the iron and ammonium are removed in the coagulation process, it is possible to feed chloride to dual-layer filtration process in the most economical way. The filter media is proposed to be anthracite and manganese sand.

(5) Disinfection

Disinfection by chlorination of treated water is proposed, and its dosage has to be determined so as to satisfy the Vietnamese standard; less than 0.1 mg/l.

# Appendix 11 Cost Estimation Borne by the Recipient Country

## Cost Estimation Borne by the Recipient Country

(Million Yen)

| No | Item | Q'ty | Cost |
|---|---|---|---|
| 1 | Land acquisition, preparation and compensation for the construction sites | 1 set | 90 |
| 2 | Construction of access road to the sites | | 10 |
| 3 | Provision of stock yard for construction materials | | 5 |
| 4 | Fence (Intake house, Water treament plant) | | 10 |
| 5 | Primary power supply works | | 20 |
| 6 | Drainage works | | 5 |
| 7 | House connection | | 70 |
| 1) | Pipe laying works (less than Dia. 50mm) | 1 set | |
| 2) | Procurement of the house connection pipes | 1 set | |
| | Total | | 210 |

(note)
The above cost estimation has been made by the JICA Basic Design Study Team.

# Appendix 12 Calculation on Transmission Pipeline

## Appendix – 12 Calculation on Transmission Pipeline

### 1 General Conditions

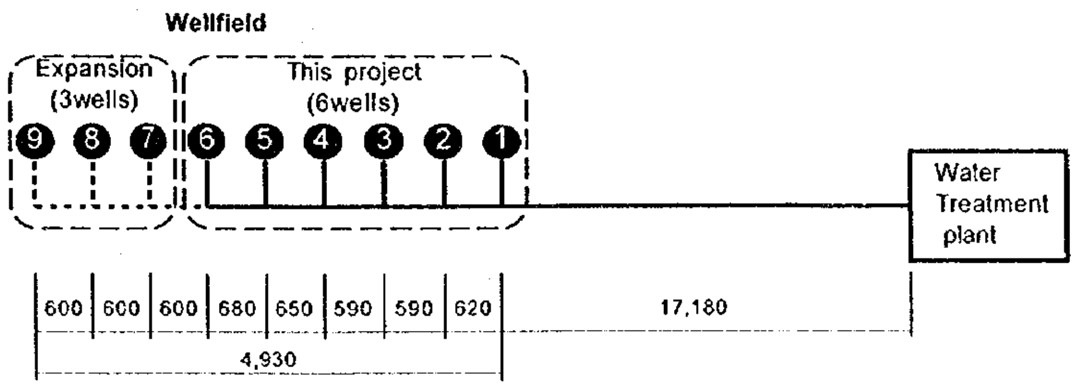
This Project is to construct a water treatment facility whose treatment capacity is 10,200 $m^3$/day. For the sake of future expansion of the treatment capacity of the plant, the transmission pipeline is designed to be capable of 13,800 $m^3$/day. For expansion, it is required to construct additional three wells and corresponding intake pipes.

The capacities of this Project and the expansion plan are outlined as below:

| | This Project | Expansion Plan |
|---|---|---|
| Design Capacity | 10,200$m^3$/day | 13,800$m^3$/day |
| Intake Pumps | Operation: 2,040$m^3$/day × 5<br>Standby: 2,040$m^3$/day × 1 | Operation: 1,970$m^3$/day × 7<br>Standby: 1,970$m^3$/day × 2 |
| Intake/Transmission Pipes | 13,800$m^3$/day | |

note) After expansion, intake pump capacity is to be controlled from 2,040 $m^3$/day to 1,970 $m^3$/day by means of control valves.

Raw water is transmitted from the well field to the water treatment plant. Topological condition is regarded as flat.

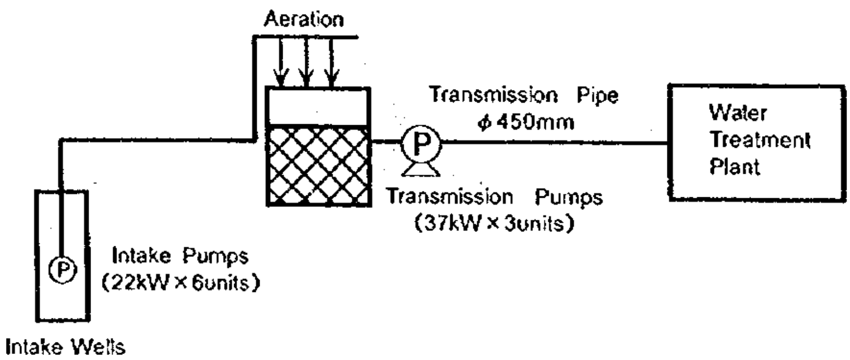


## 2 Comparison on Transmission Method

Since raw water contains corrosive free carbon dioxide and the length of transmission pipeline is approximately 20 km, the following two transmission method are compared:

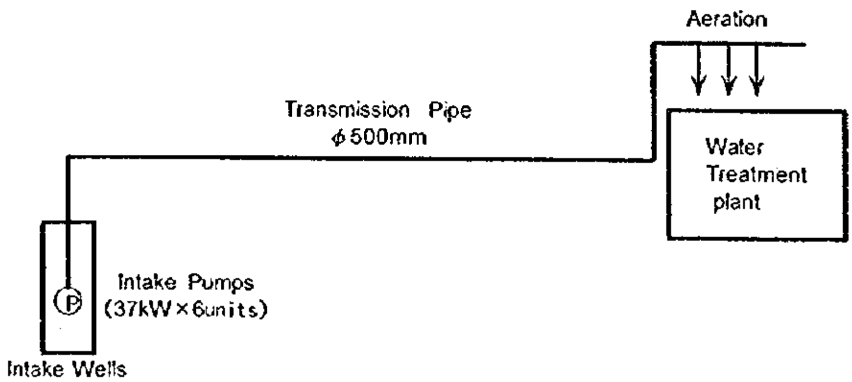
### Alternative 1

Transmission pumping station with aeration facility is constructed to remove the free carbon dioxide by aeration. The aerated water is transmitted by the transmission pumps to the sedimentation tank in the water treatment plant.

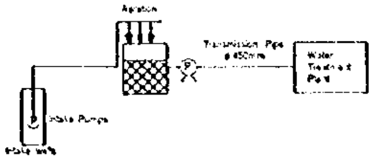
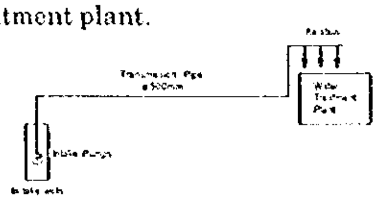



### Alternative 2

Raw water is transmitted directly to the aeration facility in the water treatment plant by the intake pumps.



## Comparison on Transmission Methods

| Item | Alternative 1 | Alternative 2 |
|---|---|---|
| System | Raw water is aerated and transmitted at the transmission pumping station.<br> | Raw water is transmitted directly to the aeration facility in the water treatment plant.<br> |
| Characters of raw water quality | Treatment capacity : $10,200\ m^3/d$<br>Type of raw water : Groundwater (Cam Giang district)<br>pH : 5.5~6.0<br>Free Carbon : 80~120 mg/l<br>Fe : 40~50 mg/l<br>Mn : 1.5~2.5 mg/l<br>$NH_4$ : 1.5~3.5 mg/l<br>$HSiO_3$ : 35~45 mg/l<br>Cl : <200mg/l | - ditto - |
| Treatment process to remove free carbon and iron | Free carbon dioxide is removed by the aeration facility so as to control pH value from acidic to neutralized. In aeration process, ferrous ion is also oxidized which reacts with silica and generates white floc or scale. | Aeration for removal of free carbon dioxide and iron oxidation is taken place in the water treatment plant. |
| Specifications of facilities | 1 Number of wells : 6wells<br>2 Intake pumps : $1.4m^3/m$ x 50m<br>3 Intake pipes : Max. φ450mm : 2,810m<br>4 Aeration facility : 1set<br>5 Pumping station : 1set<br>6 Transmission pipes : φ450 mm : 14,500m<br>7 Treatment facility : 1set<br>8 Distribution facility : 1set | 1 Number of wells : 6wells<br>2 Intake pumps : $1.4m^3/m$ x 80m<br>3 Intake pipes : Max. φ500mm : 3,130m<br>4 Aeration facility : in WTP<br>5 Pumping station : none<br>6 Transmission pipes : 500 mm : 17,180m<br>7 Treatment facility w/ aeration facility : 1set<br>8 Distribution facility : 1set |
| Operation and maintenance | 1 Removal of floc/scale is required in the facilities below:<br>a Water tank below aerator<br>b Transmission pumps and values<br>c Transmission pipes<br>2 Operation and maintenance in transmission pumping station is required | 1 Since no floc nor scale are expected to be generated in the transmission pipeline<br>2 Operation and maintenance as a daily work is not required<br>3 Operation and maintenance to control intake wells is required |

| Item | Alternative 1 | Alternative 2 |
| --- | --- | --- |
| Comparison in cost<br>I. Construction cost<br>(Million Yen) | 1 Intake facility<br>1) Intake well ± 0<br>2)Intake pipe (450mm) ± 0<br>2 Transmission facility<br>1)Pumping station ± 0<br>2)Transmission(450mm) ± 0<br>3 Water treatment plant ± 0<br>4 Distribution ± 0<br>Total ± 0 | 1 Intake facility<br>1) Intake well + 11<br>2)Intake pipe (450mm) + 9<br>2 Transmission facility<br>1)Pumping station - 56<br>2)Transmission(450mm) + 78<br>3 Water treatment plant + 0<br>4 Distribution ± 0<br>Total + 42 |
| II.O&M cost<br>(Million Yen) | 1 Personnel Expense ± 0<br>2 Electric power cost ± 0<br>3 Chemical cost ± 0<br>4 Sludge disposal ± 0<br>5 Repair ± 0<br>Total ± 0 | 1 Personnel Expense − 4<br>2 Electric power cost ± 0<br>3 Chemical cost ± 0<br>4 Sludge disposal − 3<br>5 Repair ± 0<br>Total − 7 |
| Merit | 1 Cheaper construction cost.<br>2 Corrosion protection is required only for intake facility and intake pipes. | 1 No operation and maintenance work for pumping station is required.<br>2 Due to existence of free carbon dioxide, floc nor scale in hardly generated so that operation and maintenance is easier.<br>3 O&M cost are estimated to be cheaper.<br>4 This transmission method has been experienced in Vietnam. Hai Duong has enough skills to operate. |
| Demerit | 1 White colloidal floc/scale of iron with silica is generated in the transmission facility as well as transmission pipes.<br>2 O&M cost for the transmission facility depends on amount of the sludge that may be more costly.<br>3 Sludge disposal work is required.<br>4 It is necessary to prevent intake pumps and transmission pumps from accident by water hammer.<br>5 There's no experience in Vietnam to operate this system. | 1 More costly in terms of construction cost.<br>2 Corrosion protection is required for both intake and transmission facilities.<br>3 It is necessary to operate intake pumps prudently<br>4 It is necessary to prevent intake pumps from accident by water hammer.<br>5 Since the length of transmission pipeline is as 17.5km the diameter of the pipes is bigger in order to reduce friction loss through the pipeline. |
| Countermeasure to electrical power cut off (Countermeasure to Water hammering accident) | 1 Intake pumps<br>Air valves are to be installed at the pumps and important points of the intake pipes. After recovering of electric power supply, one by one at a certain interval so as to discharge the induced air.<br>2 Transmission pumps<br>Flywheels are to be installed at the pumps. | 1 Intake pumps<br>Air values are to be installed at the pumps and important points of intake and transmission pipes. After recovering of electric power supply, one by one at a certain interval so as to discharge the induced air. |

Evaluation

In terms of construction cost, alternative 2 is more expensive since unti-corrosion material are required for whole intake and transmission facilities although no transmission pumping station is required. As for operation and maintenance cost, on the other hand, alternative 2 is more economical because no O&M cost for the pumping station nor sludge disposal cost are required.

As for operation of intake pumps after recovering of electric power supply, pumps are to be restarted one by one so as to discharge air that is introduced into the pipeline from air values installed to prevent water-hammering accident. In the case of alternative 2, since total pump head is rather high, shut-off operation is prohibited to prevent excessive inner pressure of casing pipe.

In order to remove floc/scale, in the case of alternative 1, it is required to discharge, transport and disposal at the receiving tank of transmission pumping station and transmission pipes. On the other hand, in the case of alternative 2, since aeration is conducted in the treatment plant, floc/scale is hardly generated in the pipeline.

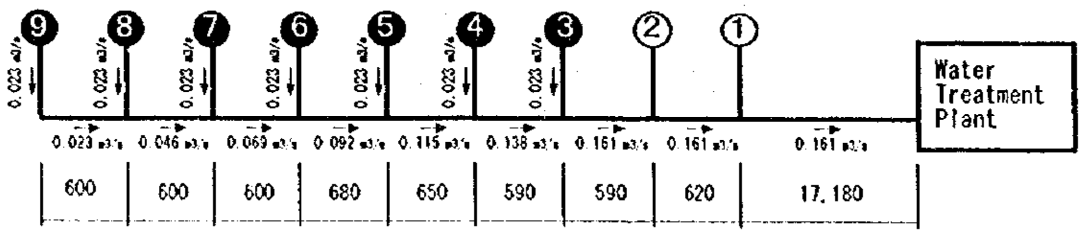
In the experience of Vietnamese water supply system, direct transmission method, namely alternative 2, has been employed as transmission method. The Hai Duong Water Supply Company is skillful enough to operate prudently. Accordingly, alternative 2 is appropriate as transmission method.

## 3 Hydraulics

### (1) Transmission pipeline

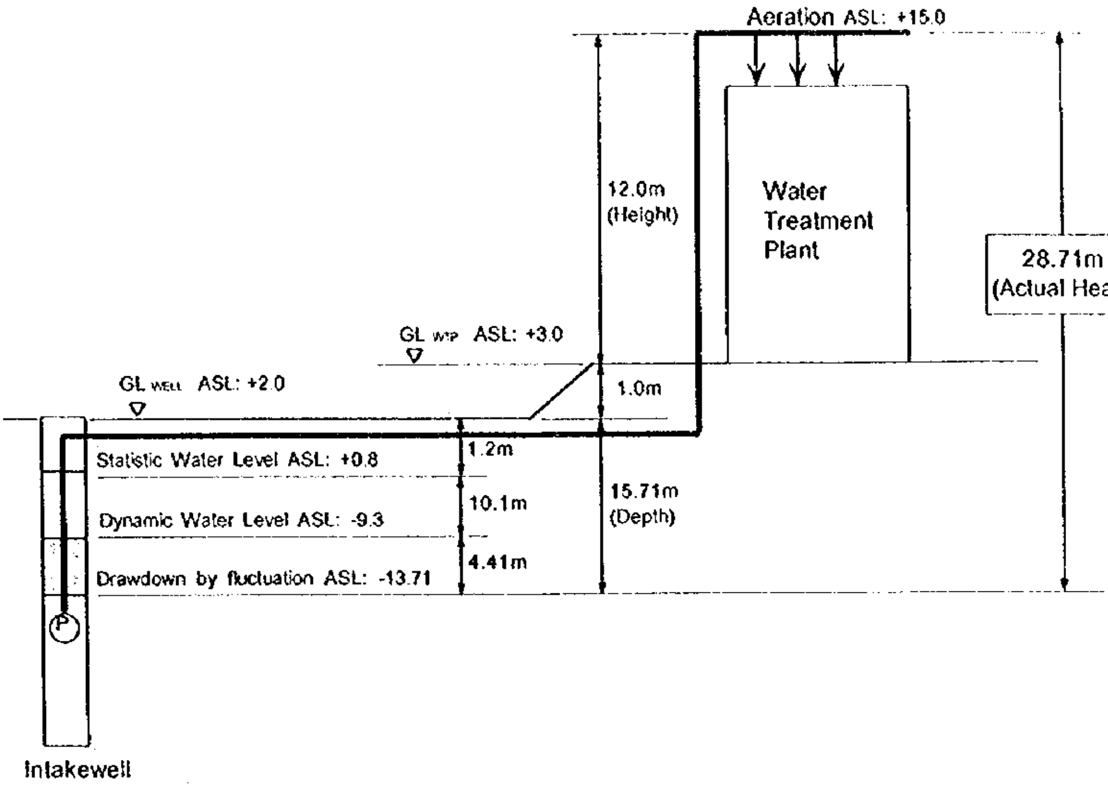
Hydraulics of transmission pipeline is calculated on condition that seven intake pumps, No.3-No.9, are operated.

Intake wells (No.3-9, 7units in operation)




| | Branch Pipe of Well | Well 9 - Well 8 | Well 8 - Well 7 | Well 7 - Well 6 | Well 6 - Well 5 | Well 5 - Well 4 | Well 4 - Well 3 | Well 3 - Well 2 | Well 2 - Well 1 | Well 1 - WTP |
|---|---|---|---|---|---|---|---|---|---|---|
| Flow $Q(m^3/sec)$ | 0.023 | 0.023 | 0.046 | 0.069 | 0.092 | 0.115 | 0.138 | 0.161 | 0.161 | 0.161 |
| Pipe Length L(m) | 50 | 600 | 600 | 600 | 680 | 650 | 590 | 590 | 620 | 17,180 |
| Pipe Drameter D(mm) | 200 | 200 | 250 | 300 | 350 | 400 | 400 | 400 | 400 | 500 |
| Coefficient C | 120 | 120 | 120 | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| Velocity V(m/sec) | 0.73 | 0.73 | 0.94 | 0.98 | 0.96 | 0.92 | 1.10 | 1.28 | 1.28 | 0.82 |
| Hydrawmic Gradient I(m/1000m) | 3.59 | 3.59 | 4.36 | 3.80 | 3.05 | 2.41 | 3.37 | 4.49 | 4.49 | 1.51 |
| Friction Loss h(m) | 0.18 | 2.15 | 2.62 | 2.28 | 2.08 | 1.57 | 1.99 | 2.65 | 2.78 | 26.01 |
| Accumulated Loss $\Sigma h(m)$ | 0.18 | 2.33 | 4.95 | 7.23 | 9.31 | 10.87 | 12.86 | 15.51 | 18.29 | 44.31 |

(2) Actual Head




| Intake Well | Distance | ASL | Actural Head | |
|---|---|---|---|---|
| Intake Well (1-9) (GL WELL) | 0.0m | ASL+2.0m | | |
| Statistic Water Level | -1.2m | ASL+0.8m | 15.71m | |
| Dynamic Water Level | -10.1m | ASL-9.3m | | |
| Drawdown by seasonal fluctuation | -4.41m | ASL-13.71m | | 28.71m |
| Aeration Facility | | | 1m | |
| GL (GL WTP) | 0.0m | ASL+3.0m | 12m | |
| Height of Aeration facility | 12.0m | ASL+15.0m | | |

(3) Total Pump head

| | | |
|---|---|---|
| Friction Loss | 44.31m | |
| Actural Head | 28.71m | |
| Discharging Pressure at Aeration | 5.00m | 80m (Total Pump Head) |
| Loss at Discharging | 1.00m | |
| Minor Losses | 0.98m | |

## 4 Water Hammer Analysis

Sudden change of water flow in pipeline causes big changes of water pressure that is referred to as water hammer. In case transmission pumps lose its driving power due to some accidents such as power cut, water pressure inside pipe decreases suddenly because of sudden loss of water discharge by pumps. When water pressure becomes under –10m, water evaporates and cavity may appear at the point that makes separation of water. After the separation, in turn, wave from upstream and downstream collide to increase extremely high pressure.

As these phenomena may cause serious damage to pipes, it is important to consider some countermeasures against water hammer. It is known that there are some preventive measures such as installation of flywheels, surge tanks or air valves. Since this project is to employ submersible pumps, flywheels is not applicable. As for surge tanks, it is difficult to construct them because total discharge head is as high as 80 m. Therefore, air valves are to be employed.

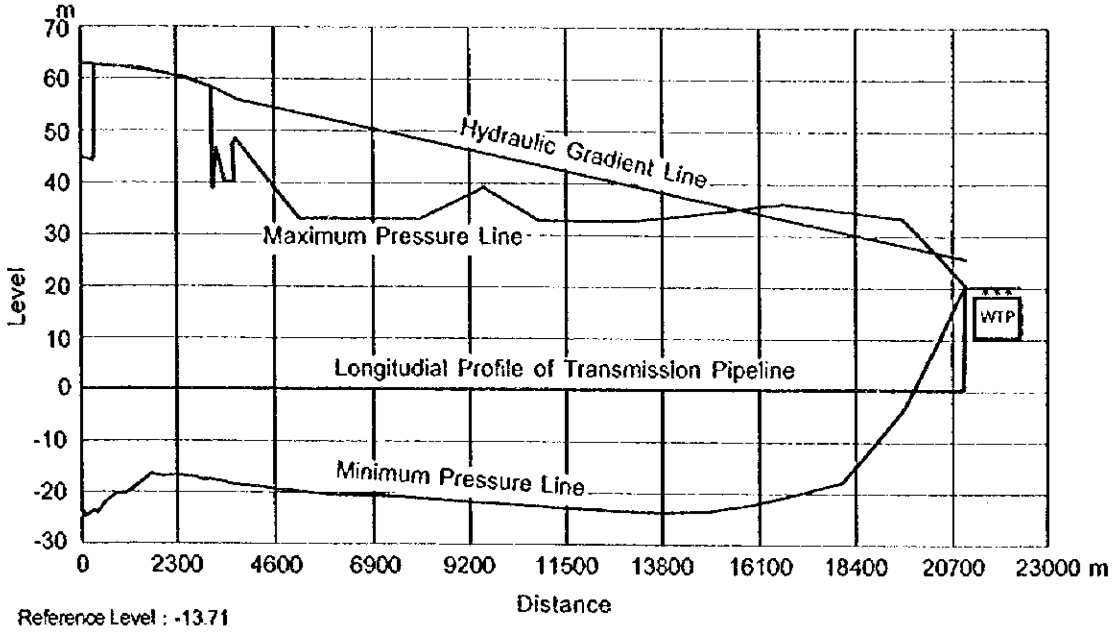
The following page shows the results of water hammer analysis in both case of without countermeasures and air valves installed. Key assumptions for analysis are as follows:

| | |
|---|---|
| Hydraulic Formula | Hazen-Williams Formula |
| Transmission Pipe | Ductile Cast Iron Pipe (Thickness : K=9, C=110) |
| Discharge Head of Pumps | Total Head : 77m, Actual Head : 28.71m |

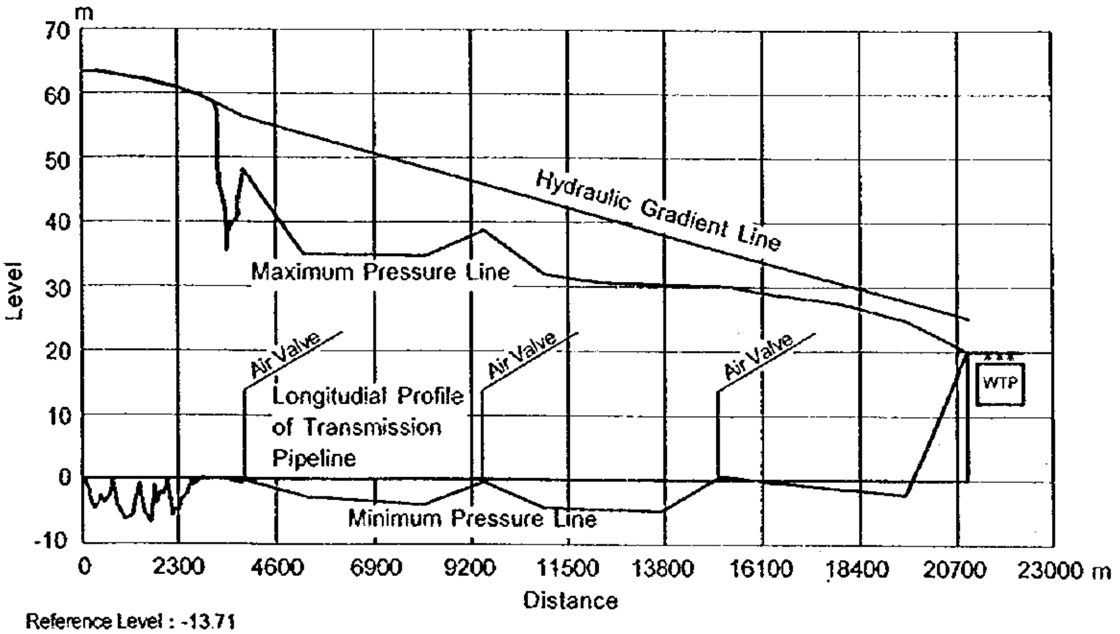
Seeing the results, in the case of no countermeasures, minimum pressure becomes as low as –20m that may cause pipe damage. As for the case of air valve installed, the possibility of damaging pipes is quite low since negative pressure is controlled within –5m.

# Water Hammer Analysis Result

## Case-1 Without Preventive Measures




## Case-2 Air Valves are installed



# Appendix 13 Network Analysis

## Appendix – 13 Network Analysis

### 1 General Conditions

This Project is to construct a water treatment facility of 10,200 $m^3$/day treatment capacity. As for distribution pipes, they are planned to be capable of 13,800 $m^3$/day, taking into account that treatment capacity be expanded to 13,800 $m^3$/day.

The capacities of distribution pipes are to meet hourly peak water demand. The capacities of this project and expansion plan are summarized as below:

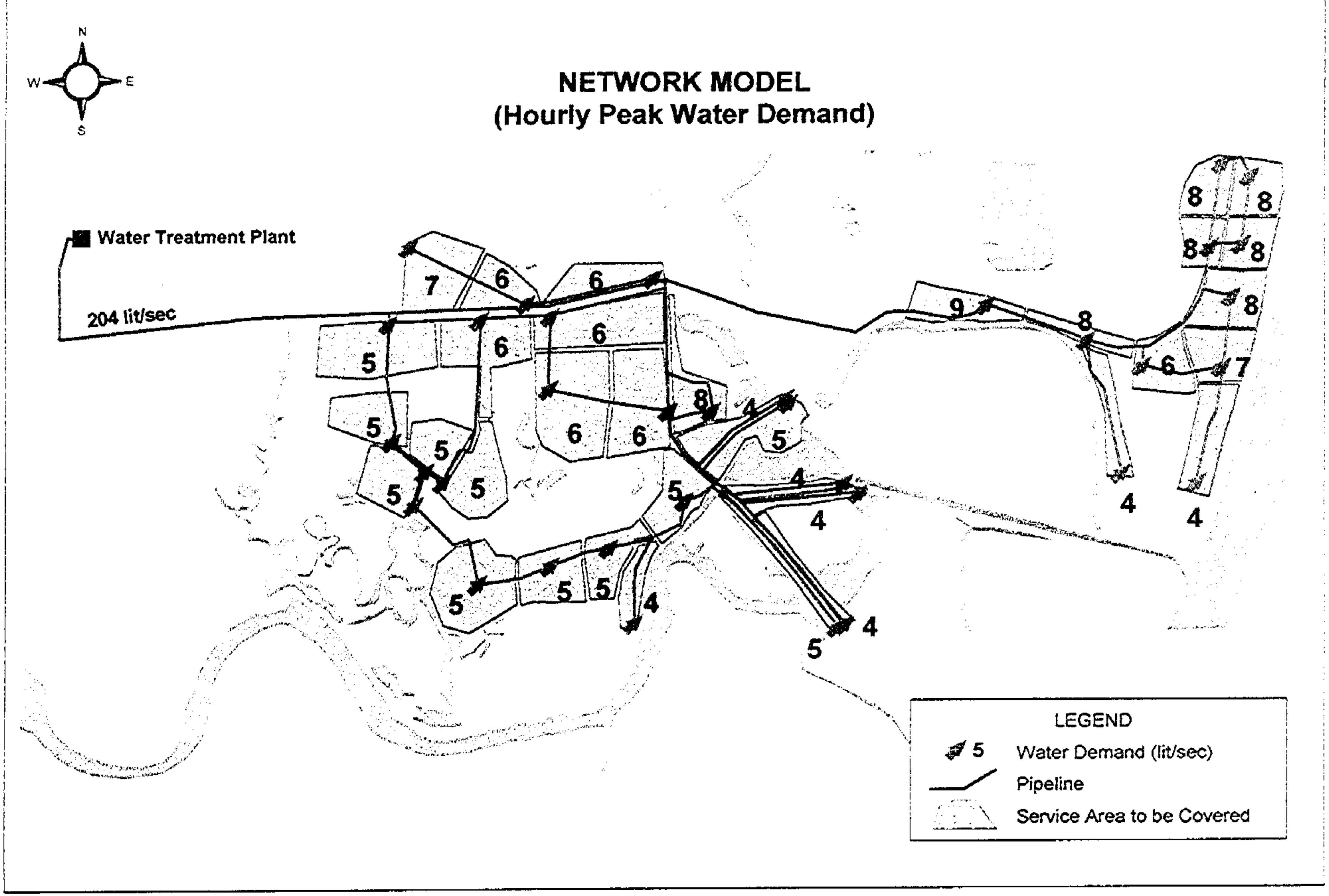
| | This Project | Expansion Plan |
|---|---|---|
| Daily Maximum Treatment Capacity | 10,200 $m^3$/day | 13,800 $m^3$/day |
| Daily Maximum Distribution Capacity (Plant Loss:5%) | 9,650 $m^3$/day | 13,000 $m^3$/day |
| Hourly Peak Distribution Capacity (Peak Factor:1.35) | 13,000 $m^3$/day (150 lit/sec) | 17,600 $m^3$/day (204 lit/sec) |

### 2 Network Model / Analysis Result

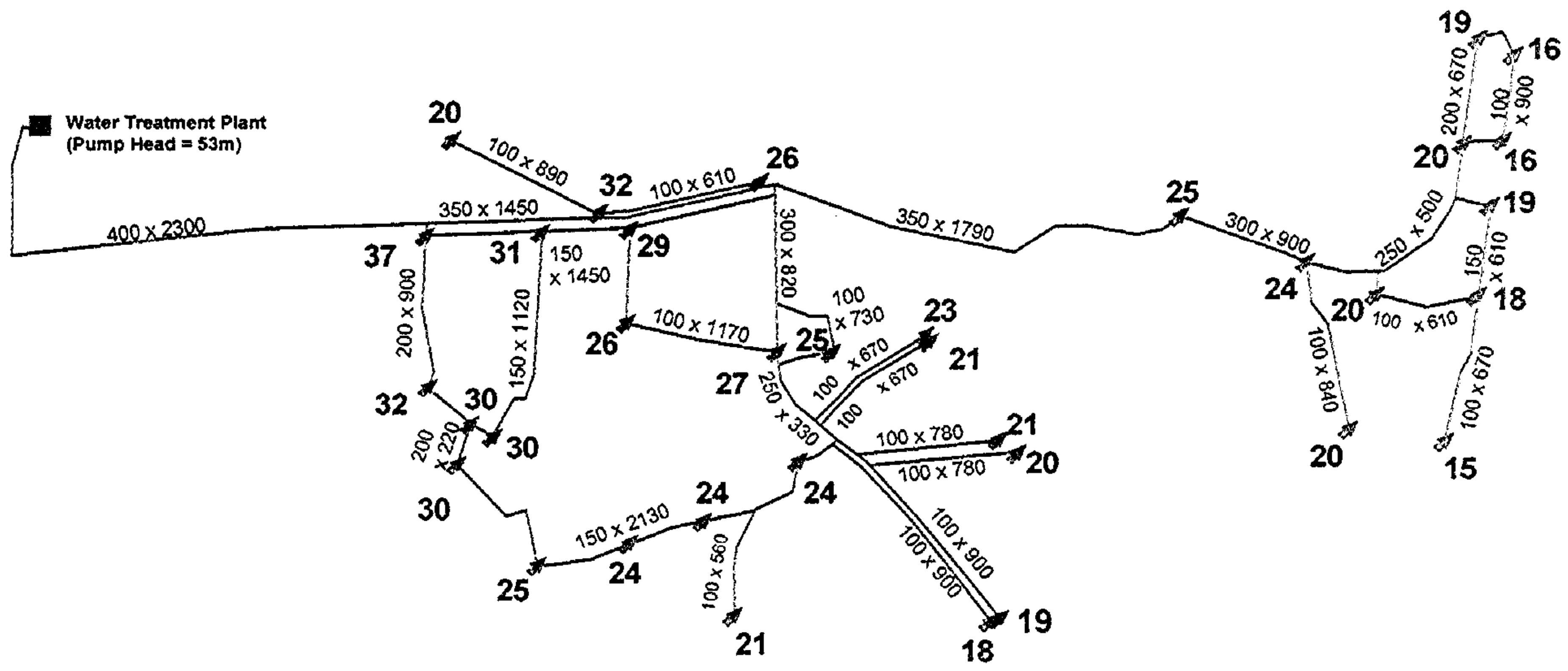
Network model was designed on the basis of the projected population served in year 2000 and field survey results. The network model and the analysis result are shown in the following pages. General information is given as table below:

| | |
|---|---|
| Low Water Level of Distribution Reservoir | +1.0 m |
| Discharge Capacity of Distribution Pumps | 68lit/sec × 3 units = 204 lit/sec |
| Discharge Head of Distribution Pumps | Total Head: 55m, Actual Head: 53m |
| Hydraulic Formula | Hazen-Williams Formula |
| Distribution Pipes | Ductile Cast Iron Pipe (ϕ 100-400mm, C=130 for main, C=110 for branch) |
| Topological Condition of the Service area | Flat (+2.8 m) |
| Water Pressure at Nodes | More than 1.5 kgf/cm² (Pressure Head: 15m) |
| Fire Hydrant | To be ignored |

## NETWORK MODEL
## (Hourly Peak Water Demand)



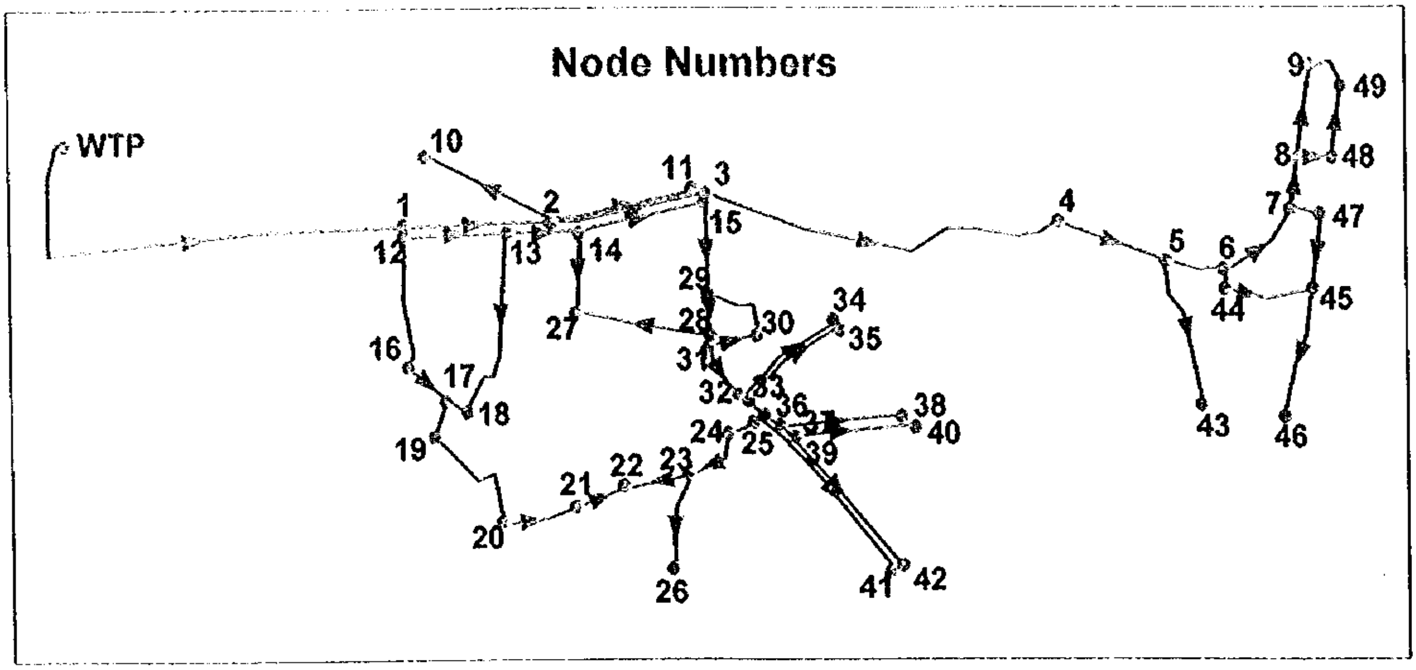
# NETWORK ANALYSIS RESULT
## (Water Pressure / Pipe Diameter x Length)




LEGEND

18 Residual Pressure at Node (m)

150 x 600 Pipe Diameter (mm) x Pipe Length (m)




## Hydraulic Calculation of Distribution Pipelines

| Pipeline | L<br>Length<br>(m) | D<br>Diameter<br>(mm) | C<br>Coefficient | Q<br>Hourly Peak Flow<br>(Lit/s) | V<br>Velocity<br>(m/s) | I<br>Hydraulic Gradient<br>(1/1000) | H<br>Head Loss<br>(m) | TH<br>Total Head Loss<br>(m) | DWL<br>Dynamic Water Level<br>(+m) | GL<br>Ground Elevation<br>(+m) | RP<br>Residual Pressure<br>(+m) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| WTP | - | - | - | - | - | - | - | - | +54.00 | +2.80 | +51.20 |
| WTP-1 | 2300 | 400 | 130 | 204.00 | 1.62 | 5.96 | 13.71 | 13.71 | +40.29 | +2.80 | +37.49 |
| 1-2 | 840 | 350 | 130 | 146.83 | 1.53 | 6.22 | 5.22 | 18.93 | +35.07 | +2.80 | +32.27 |
| 2-3 | 610 | 350 | 130 | 127.83 | 1.33 | 4.81 | 2.93 | 21.87 | +32.13 | +2.80 | +29.33 |
| 3-4 | 1790 | 350 | 130 | 78.00 | 0.81 | 1.93 | 3.45 | 25.32 | +28.68 | +2.80 | +25.88 |
| 4-5 | 560 | 300 | 130 | 69.00 | 0.98 | 3.26 | 1.83 | 27.15 | +26.85 | +2.80 | +24.05 |
| 5-6 | 340 | 300 | 130 | 57.00 | 0.81 | 2.29 | 0.78 | 27.93 | +26.07 | +2.80 | +23.27 |
| 6-7 | 500 | 250 | 130 | 47.45 | 0.97 | 3.96 | 1.98 | 29.91 | +24.09 | +2.80 | +21.29 |
| 7-8 | 220 | 200 | 130 | 32.00 | 1.02 | 5.66 | 1.25 | 31.15 | +22.85 | +2.80 | +20.05 |
| 8-9 | 450 | 200 | 130 | 15.08 | 0.48 | 1.41 | 0.63 | 31.79 | +22.21 | +2.80 | +19.41 |
| 2-10 | 890 | 100 | 110 | 7.00 | 0.89 | 13.56 | 12.07 | 31.00 | +23.00 | +2.80 | +20.20 |
| 2-11 | 610 | 100 | 110 | 6.00 | 0.76 | 10.19 | 6.22 | 25.15 | +28.85 | +2.80 | +26.05 |
| 1-12 | 6 | 250 | 110 | 57.17 | 1.17 | 7.61 | 0.05 | 13.75 | +40.25 | +2.80 | +37.45 |
| 12-13 | 500 | 150 | 110 | 19.74 | 1.12 | 12.81 | 6.41 | 20.16 | +33.84 | +2.80 | +31.04 |
| 13-14 | 330 | 150 | 110 | 11.69 | 0.66 | 4.86 | 1.60 | 21.76 | +32.24 | +2.80 | +29.44 |
| 3-15 | 6 | 300 | 110 | 49.83 | 0.71 | 2.43 | 0.01 | 21.88 | +32.12 | +2.80 | +29.32 |
| 14-15 | 620 | 150 | 110 | 1.47 | 0.08 | 0.11 | 0.07 | 21.95 | +32.05 | +2.80 | +29.25 |
| 12-16 | 670 | 200 | 110 | 32.42 | 1.03 | 7.90 | 5.29 | 19.05 | +34.95 | +2.80 | +32.15 |
| 16-17 | 220 | 200 | 110 | 27.42 | 0.87 | 5.80 | 1.28 | 20.32 | +33.68 | +2.80 | +30.88 |
| 13-18 | 1010 | 150 | 110 | 2.05 | 0.12 | 0.19 | 0.19 | 20.35 | +33.65 | +2.80 | +30.85 |
| 17-18 | 110 | 150 | 110 | 2.95 | 0.17 | 0.38 | 0.04 | 20.36 | +33.64 | +2.80 | +30.84 |
| 17-19 | 220 | 200 | 110 | 19.48 | 0.62 | 3.08 | 0.68 | 21.00 | +33.00 | +2.80 | +30.20 |
| 19-20 | 670 | 150 | 110 | 14.48 | 0.82 | 7.22 | 4.84 | 25.84 | +28.16 | +2.80 | +25.36 |
| 20-21 | 340 | 150 | 110 | 9.48 | 0.54 | 3.30 | 1.12 | 26.96 | +27.04 | +2.80 | +24.24 |
| 21-22 | 220 | 150 | 110 | 4.48 | 0.25 | 0.82 | 0.18 | 27.14 | +26.86 | +2.80 | +24.06 |
| 22-23 | 340 | 150 | 110 | 0.52 | 0.03 | 0.02 | 0.01 | 27.15 | +26.85 | +2.80 | +24.05 |
| 23-24 | 340 | 150 | 110 | -4.52 | 0.26 | -0.84 | -0.29 | 26.86 | +27.14 | +2.80 | +24.34 |
| 24-25 | 220 | 150 | 110 | -9.52 | 0.54 | -3.33 | -0.73 | 26.13 | +27.87 | +2.80 | +25.07 |
| 23-26 | 560 | 100 | 110 | 4.00 | 0.51 | 4.81 | 2.69 | 29.84 | +24.16 | +2.80 | +21.36 |
| 14-27 | 500 | 100 | 110 | 4.22 | 0.54 | 5.31 | 2.66 | 24.42 | +29.58 | +2.80 | +26.78 |
| 27-28 | 670 | 100 | 110 | -1.78 | 0.23 | -1.08 | -0.72 | 23.69 | +30.31 | +2.80 | +27.51 |
| 28-29 | 200 | 300 | 110 | -47.47 | 0.67 | -2.22 | -0.44 | 23.25 | +30.75 | +2.80 | +27.95 |
| 15-29 | 560 | 300 | 110 | 51.31 | 0.73 | 2.56 | 1.43 | 23.38 | +30.62 | +2.80 | +27.82 |
| 29-30 | 450 | 100 | 110 | 3.83 | 0.49 | 4.45 | 2.00 | 25.39 | +28.61 | +2.80 | +25.81 |
| 28-31 | 60 | 300 | 110 | 39.69 | 0.56 | 1.59 | 0.10 | 23.79 | +30.21 | +2.80 | +27.41 |
| 30-31 | 280 | 100 | 110 | -4.17 | 0.53 | -5.19 | -1.45 | 23.93 | +30.07 | +2.80 | +27.27 |
| 31-32 | 330 | 250 | 110 | 35.52 | 0.72 | 3.16 | 1.04 | 24.98 | +29.02 | +2.80 | +26.22 |
| 32-34 | 670 | 100 | 110 | 4.00 | 0.51 | 4.81 | 3.22 | 28.20 | +25.80 | +2.80 | +23.00 |
| 33-35 | 670 | 100 | 110 | 5.00 | 0.64 | 7.27 | 4.87 | 29.85 | +24.15 | +2.80 | +21.35 |
| 37-38 | 780 | 100 | 110 | 4.00 | 0.51 | 4.81 | 3.75 | 29.88 | +24.12 | +2.80 | +21.32 |
| 33-39 | 220 | 200 | 110 | 26.52 | 0.84 | 5.45 | 1.20 | 27.33 | +26.67 | +2.80 | +23.87 |
| 39-40 | 780 | 100 | 110 | 4.00 | 0.51 | 4.81 | 3.75 | 31.08 | +22.92 | +2.80 | +20.12 |
| 33-41 | 900 | 100 | 110 | 5.00 | 0.64 | 7.27 | 6.54 | 32.67 | +21.33 | +2.80 | +18.53 |
| 39-42 | 900 | 100 | 110 | 4.00 | 0.51 | 4.81 | 4.33 | 31.66 | +22.34 | +2.80 | +19.54 |
| 5-43 | 840 | 100 | 110 | 4.00 | 0.51 | 4.81 | 4.04 | 31.19 | +22.81 | +2.80 | +20.01 |
| 6-44 | 110 | 100 | 110 | 9.55 | 1.22 | 24.09 | 2.65 | 30.58 | +23.42 | +2.80 | +20.62 |
| 44-45 | 500 | 100 | 110 | 3.55 | 0.45 | 3.86 | 1.93 | 32.51 | +21.49 | +2.80 | +18.69 |
| 45-46 | 670 | 100 | 110 | 4.00 | 0.51 | 4.81 | 3.22 | 35.73 | +18.27 | +2.80 | +15.47 |
| 45-47 | 390 | 150 | 110 | -7.45 | 0.42 | -2.11 | -0.82 | 31.68 | +22.32 | +2.80 | +19.52 |
| 7-47 | 220 | 150 | 110 | 15.45 | 0.87 | 8.14 | 1.79 | 31.70 | +22.30 | +2.80 | +19.50 |
| 8-48 | 170 | 100 | 110 | 8.92 | 1.14 | 21.24 | 3.61 | 34.76 | +19.24 | +2.80 | +16.44 |
| 48-49 | 500 | 100 | 110 | 0.92 | 0.12 | 0.32 | 0.16 | 34.92 | +19.08 | +2.80 | +16.28 |
| 9-49 | 230 | 100 | 110 | 7.08 | 0.90 | 13.83 | 3.18 | 34.97 | +19.03 | +2.80 | +16.23 |

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