AGRICULTURAL LAND AND WATER RESOURCES DEVELOPMENT

GROUP TRAINING COURSE

DESIGN OF NISHIIWASAKI HEADWORKS

1982

Ministry of AgricultUre, Forestry and Fisheries (MAFF)

Japan International Cooperation Agency (JICA)

The Japanese Institute of Irrigation & Drainage (JIID)

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C O N T E N T S

1. Preface

Nishiiwasaki Head Works had been constructed about 90 (1886) years ago, as an irrigation facility of Nasu-sosui Irrigation Project. The Project is one of the most famous project in our country and its benefited area is an alluvial fan of about 40,000 ha.

It has become necessary to improve (rehabilitate) it because it was too old and extended the benefited area.

The construction work was conducted in non-irrigation period from 1974 to 1975 and the cost was about 520 million yen.

2. Profile of the Head Works

(a) Location; Nishiiwasaki, Kroiso-City, Tochigi-Prefecture

(b) Design flood discharge; 1,200 m^3/sec

(c) Catchment area; 147.8 km^2

(d) Type; Concrete floating type

(e) Length; 89 m Fixed weir; 50 m
Scouring sluice; 10 m
Flood sluice; 25 m
Others; 4 m

(f) Height; 3.05 m

(g) Gate;

Item	Type	Length	Number	Height	Winch apparatus
Flood sluice	Steel roller	25.0^m	1	2.4^m	Electric winch
Scouring sluice	-do-	10.0	1	2.9	-do-
Intake Regulating	-do-	2.5	1	1.4	-do-
Intake Regulating	Steel flap	9.0	1	2.5	Oil pressure

(h) Fish way; Right bank: Ladder type Stone (cobble) pitching
Length 29.5 m Width 2 m

Center: Ladder type Stone (cobble) pitching
Length 22.5 m Width 2 m

(i) Riprap; Concrete block, Length 50 m

(j) Revetment works; Reinforced concrete retaining wall, Length 261.57 m
Concrete block lining, Length 350.84 m

(k) Operation bridge; Length 55.75 m Width 1.5 m

(l) Intake facilities

Location : Right bank (12 m upstream)

Regulated intake water level : E.L. 418.00 m
Intake; Width 9.00 m Water depth 1.25 m
Intake water; 8.94 m^3/sec

Intake culvert; Length 2.50 m Height 2.50 m

(m) Operation & Meintenance Office: Reinforced concrete building

3. Geology of the site

The width of Naka River is about 100 m, and the river bed was covered by cobble, gravel and sand.

Geological exploration, borings were conducted 4 points at the center, 1 point at the intake and 1 point at the riprap, total 6 points.

The result of the borings was follows.

Items	Depth	N-value	Coefficient of permeability
Gravel soils included cobble	0 - 3 m	40	10 m/sec
Talus cone	3 - 6	30 --	
Tull(rock) included gravel	6 --	(20 --)	

(E.L. 414 m --)

4. Outline of design

a. Basic design

a-1 Site

It was considered 2 alternatives,

1) 200 m upstream and

2) 25 m downstream from the old Head Works

1) Length 60 m was shorter but approach channel 1,300 m was longer and depth of gravel soil 8 m was deeper than cost would be higher.

2) Length 90 m (fixed weir 50 m, movable weir 40 m) was longer, but approach channel was shorter and depth of gravel soils 3 to 5 m was shallower, than cost would be cheaper.
So site was decided alternative 2), finally decided 10 m downstream of the old one by detail studies.

a-2 Foundation

Minimum N-value is 20, If we planned Beta foundation,

Ultimate beaving capacity q_d = 44.88 ton/m^2

Allowable beaving capacity $q_a = 44.88/3 = 14.96$ ton/m^2
(Safety factor assumed 3)

In earthquake $14.96 \times 1.5 = 22.44$ ton/m^2

Maximum unit weight 11.90 ton/m^2 < 14.96 ton/m^2 OK

In earthquake 19,65 ton/m^2 < 22.44 ton/m^2 OK

So Beta foundation was all right.

a-3 Design flood discharge

1) Calculation of probable rainfall

Itamuro Station; 7 km upstream

43 years maximum daily rainfall data by Iwai method

Probable value	1/5	1/10	1/20	1/50	1/100	1/200
Daily rainfall	151.88	177.18	201.38	232.78	256.56	280.58

2) Calculation of design flood discharge

2-1) Arrival time of flood

l: River length from top of the river to the site 22.5 km

H: Difference of elevation from the river to the site 0.88 km

$W = 72(H/l)^{0.6} = 72(0.88/22.5)^{0.6} = 10.3$ km/hr

$T = l/W = 22.5/10.3 = 2.2$ hr

2-2) Rainfall intensity

γ_{24}; Daily rainfall $n = 0.55$

T; Arraival time of flood

$$\gamma_t = \frac{\gamma_{24}}{24}\left(\frac{24}{T}\right)^n = \frac{232.8}{24}\left(\frac{24}{2.2}\right)^{0.55} = 36.1 \text{ mm}$$

2-3) Flood discharge

$$Q = \frac{1}{3.6} f \cdot \gamma_t \cdot A = \frac{1}{3.6} \times 0.8 \times 36.1 \times 147.8$$

$$= 1{,}186 \text{ m}^3/\text{sec} \doteqdot 1{,}200 \text{ m}^3/\text{sec}$$

a-4 Design water level

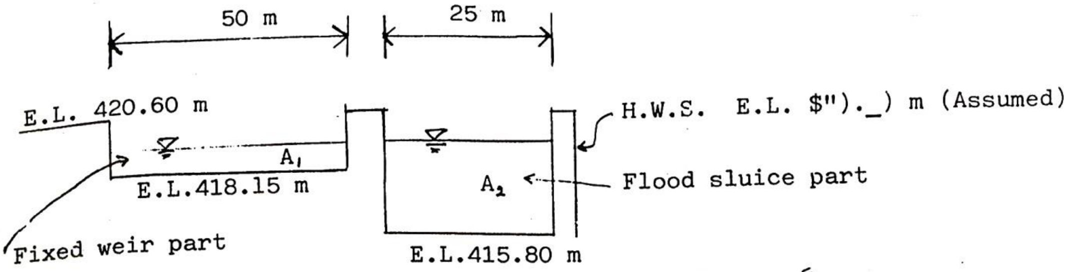
1) Design flood water level

After project, cross section of the site should be bigger than before project to prevent back water.

If design flood water level was assumed E.L. 420.60 m, design flood discharge 1,200 m^3/sec was possible to flow.

So design flood water level was decided E.L. 420.60 m.

2) Discharge after project in flood time



A_1 = 2.45 x 50 - (0.04 x 2.45 x 2) = 122.3 m^2

A_2 = (25 - 2 x 0.04 x 4.80) x 4.80 = 118.2 m^2

2-1) Fixed weir part

Calculation as complete overflow

Q_1 = 1.70 B $H^{3/2}$ = $2.45^{3/2}$ = 3.835

= 1.70 x 50 x $2.45^{3/2}$ = 326.00 m^3/sec

2-2) Flood sluice part

Cross-sectional area of flow A_2 = 118.2 m^2 I = 1/60

$I^{1/2}$ = (1/60) = 0.129

Wetted perimeter P_2 = 4.80 x 2 + 25.0 = 34.60

Hydraulic mean depth $R_2 = A_2/P_2$ = 118.2/34.60 = 3.416

$R^{2/3}$ = 2.699

Velocity of flow $V_2 = \frac{1}{n} R_2^{2/3} I^{1/2} = \frac{1}{0.045}$ x 2.699 x 0.129

= 7.7 m/sec

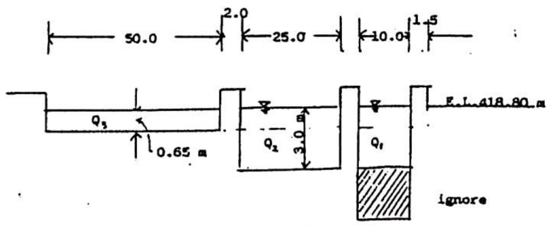
Discharge $Q_2 = A_2 \cdot V_2$ = 118.2 x 7.7 = 910.1 m^3/sec

Total discharge $\Sigma Q = Q_1 + Q_2$ = 326.0 + 910.1 = 1236.1 m^3/sec 1200 m^3/sec OK

(Scouring sluice part was ignpred on f;ppd time by the regulation.)

3) Discharge after project in ordinary time

3-1) Cross-sectional area of flow



3-2) Discharge

Scouring sluice and flood sluice part were natural flow, and Fixed weir part was broad-crested weir flow.

a) Scouring sluice part

A_1 = (10.0 - 2 x 0.04 x 3.0) x 3.0 = 29.28 m^2

P_1 = 10.0 + 2 x 3.0 = 16.0 m

$R_1 = \frac{29.28}{16.0}$ = 1.830 $R^{2/3}$ = 1.496

I_1 = 0.01898 $I^{1/2}$ = 0. 1378

n_1 = 0.045 Coefficient of roughness

$V_1 = \frac{1}{0.045}$ x 1.496 x 0.1378 = 4.58 m/sec · Manning fornula

$Q_1 = 29.28 \times 4.58 = 134.10\ m^3/sec$

b) Flood sluice part

$A_2 = (25.0 - 2 \times 0.04 \times 3.0) \times 3.0 = 74.28\ m^2$

$P_2 = 25.0 + 2 \times 3.0 = 31.0\ m$

$R_2 = 2.396 \qquad R_2^{2/3} = 1.791$

$I_2 = 0.01898 \qquad I_2^{1/2} = 0.1378$

$n_2 = 0.045$

$V_2 = \frac{1}{0.045} \times 1.791 \times 0.1378 \doteq 5.48\ m/sec$

$Q_2 = 74.28 \times 5.48 = 407.58\ m^3/sec$

c) Fixed weir part

Assumed complete flow

$Q_3 = 1.7\ B_3\ H_3^{3/2} = 1.7 \times 50.0 \times 0.65^{3/2} = 44.54\ m^3/sec$

$\Sigma Q = Q_1 + Q_2 + Q_3 = 134.10 + 407.58 + 44.54 = 586\ m^3/sec$

a-5 Bed height

Omission

a-6 Type

1) Type of weir

It was decided flowting type because its cost was cheap.

2) Type of movable weir

2-1) Scouring sluice

The reasons why the length was 10 m.

* The tractive force was around scouring particle size 200 mm.
* It would increase discharge than it kept water route.

2-2) Flooding sluice

The reasons why the length was 25 m.

* After project, cross-sectional area of flow should be larger than before.
* Lower water river part in present condition should be mavable weir.

3) Type of gate

3-1) Scouring sluice and flooding sluice

* Steel roller gate because of simple structure, eary operation and much sediment discharge at flooding.
* Top of elevation was 5 cm higher than fixed weir

because small flooding should flow over fixed weir.

3-2) Intake regulation gate

* Steel flap gate because of easy regulation (over flow), intake surface water and no pier.
* One more steel roller gate was for emergency etc.

a-7 Design of scouring sluice

1) Bed height

E.L. 415.30 m. It was 0.5 m lower than bed height of flooding sluice. Considered effectable scouring.

2) Width and longitudinal slope

Generally, width should be decided by ordinary discharge but in this case it was decided by small flooding discharge because maximum particle size was very big 200 mm.

2-1) Scouring maximum particle size 100 mm.

2-1-1) Scouring velocity requirement

$V_{c1} = 1.5\ c\ \sqrt{d_1} = 1.5 \times 4.3 \times (0.10)^{1/2} = 2.04$ m/sec

C; Coefficient of shape of sand and gravel 4.3

V_{c1}; Scouring velocity requirement

2-1-2) Width

$L_1 = Q_1/q_1 = 8.94/0.866 = 10.3$ m

q_1; Unit discharge requirement Q_1; Scouring standard discharge 8.94 m^3/sec

$q_1 = V_{c1}^3\ /g = 2.04^3/9.8 = 0.866\ m^3/sec/m$

So, width should be 10 m.

2-1-3) Longitudinal slope

It should be move steeper than critical slope and keep the scouring available velocity.

Critical slope $I_c = n^2 g/h_c{}^{1/3} = 0.03 \times 9.8/0.43^{1/3}$

$= 0.0116 = 1/86$

n; Coefficient of roughness 0.03

$q = Q/L = 8.94/10.0 = 0.894\ m^3/sec/m$

Critical depth $h_c = (q^2/g)^{1/3} = (0.894^2/9.8)^{1/3}$

$= 0.43$ m

So, longitudinal slop should be 1/86

2-2) Scouring maximum particle size 200 mm

2-2-1) Scouring velocity requirement

$V_{c2} = 1.5\ c(d_1)^{1/2} = 1.5 \times 4.3 \times (0.20)^{1/2} = 2.88$ m/sec

2-2-2) Discharge requirement

$Q_2 = q_2\ L = 2.44 \times 10.0 = 24.4\ m^3/sec$

$q_2 = V_{c2}^3/g = 2.88^3/9.8 = 2.44\ m^3/sec/m$

2-2-3) Scouring velocity

Discharge $Q_2 = 24.4\ m^3/sec/m$, Width $L = 10.0$ m

Coefficient of roughness $n = 0.03$ Slope $I = 1/80$

above conditions

Tractive velocity 2.95 m/sec

So, natural scouring become possible.

a-8 Design of apron

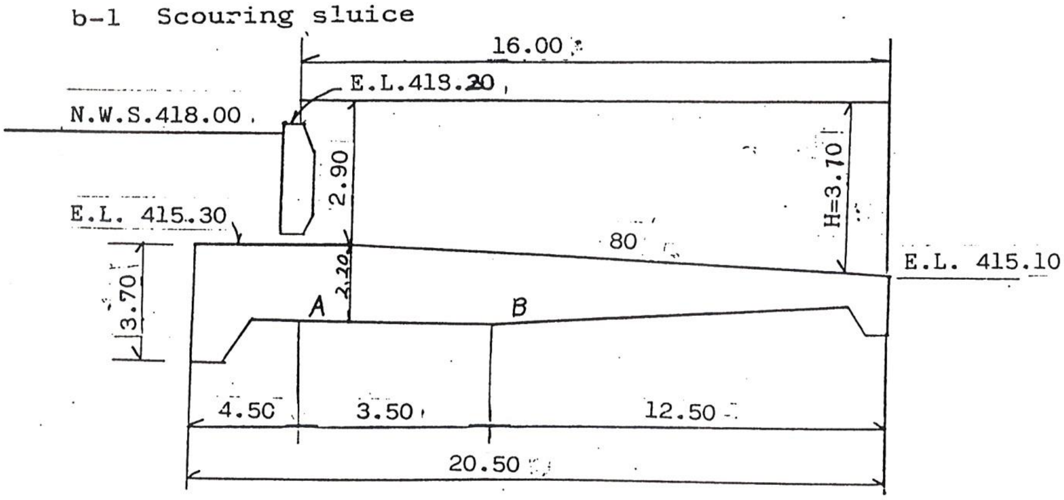
Steel plate pitching method for scouring sluice and flooding sluice part because it would be most effectable against abrasion by avalanche of sand and stone.

a-9 Design of riprap

Omission.

b. Design of weir

b-1 Scouring sluice



b-1-1) Length of apron

Bligh's formula $L_a = 0.9\ C\ (H)^{1/2}$

C; Coefficient of Bligh 7.5 (Gravel included cobble)

H; Maximum water level difference between upstream and downstream

$H = E.L.\ 418.20 - E.L.\ 415.10 = 3.10$ m

$L_a = 0.9 \times 7.5 \times (3.10)^{1/2} = 11.88\ m < 16.00\ m$

Apron length (downstream) was decided 16.00 m

1-2) Piping

(1) Bligh's formula $L = CH$ $C = 7.5$ $H = 3.10$ m

Required creep length $L = 7.5 \times 3.10 = 23.25$ m

Designed creep length $L' = 2.20 + 1.50 \times 2 + 20.50$

$= 25.70 > L$, OK

(2) Lane's formula $L = C'H$

C'; Creep ratio 3.0

Required creep length $L = 3.0 \times 3.10 = 9.30$ m

Designed creep length $L' = 2.20 - 1.5 \times 2 + 1/3 \times 20.25$

$= 12.03$ m $> L = 9.30$ m OK

-1-3) Apron thickness

$$T = \frac{4}{3} \times \frac{H - hf}{\quad - 1}$$

$H = 3.10$ m

hf; Head loss to point A and B.

γ; Unit weight of concrete 2.3 ton/m^3

(1) Point A

$$hf = L_A/L' \times H = \frac{2.20 + 1.50 \times 2 + 4.50}{25.70} \times 3.10 = 1.17 \text{ m}$$

$$T = 4/3 \times \frac{3.10 - 1.17}{2.3 - 1} = 1.98 \text{ m} < 2.20 \text{ m} \quad \text{OK}$$

(2) Point B

$$hf = L_B/ L' \times H = \frac{2.20 + 1.50 \times 2 + 8.00}{25.70} \times 3.10 = 1.59 \text{ m}$$

$$T = 4/3 \times \frac{3.10 - 1.59}{2.3 - 1} = 1.55 \text{ m} < 2.156 \text{ m} \quad \text{OK}$$

-1-4) Length of riprap

Bligh's formula $l = 10\ C\ (Hq)^{1/2}$

$C = 7.5$, $H = 3.10$ m, q; Unit discharge.

$Q = 1{,}200$ m^3/sec, $A = 258.20$ m^2, $D = 4.96$ m

$$q = \frac{1{,}200}{258.2} \times 4.96 = 23.05 \text{ m}^3\text{/sec/m}$$

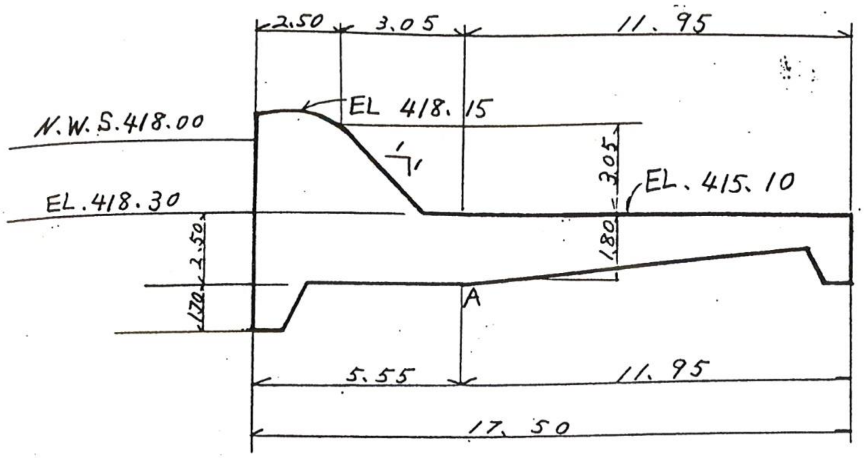
$$l = 10 \times 7.5 \times (3.10 \times 23.05)^{1/2} = 63.4 \text{ m}$$

$$l_R = l - l_a = 63.4 - 16.0 = 47.4 \text{ m}$$

So, riprap length was decided 50 m.

(Flooding sluice was same conditions)

b-2 Fixed weir



b-2-1) Length of apron

Bligh's formula $La = 0.6\ c\ (H)^{1/2}$

$c = 7.5$, H = E.L. 418.15 − E.L. 415.10 = 3.05 m

$L_a = 0.6 \times 7.5 \times (3.05)^{1/2} = 7.86 < 11.95$ m OK

So, length of apron was decided 11.95 m

b-2-2) Piping

(1) Bligh's formula $L = CH$ $C = 7.5$, $H = 3.05$

Required creep length $L = 7.5 \times 3.05 = 22.88$

Designed creep length $L' = 2.50 + 1.20 \times 2 + 17.50$

$= 23.40$ m > 22.88 m OK

(2) Lane's formula $L = C'H$ $C' = 3.0$, $H = 3.05$

Required creep length $L = 3.0 \times 3.05 = 9.15$ m

Designed creep length $L' = 2.50 + 1.70 \times 2 + 1/3 \times 17.50$

$= 11.73 > 9.15$ m OK

b-2-3) Apron thickness

$T = 4/3 \dfrac{H - hf}{\quad - 1}$ $H = 3.05$,

hf; Head loss to point A and B,

$\gamma = 2.3$ ton/m^3

$$hf = \frac{2.50 + 1.70 \times 2 + 5.55}{23.40} \times 3.05 = 1.50 \text{ m}$$

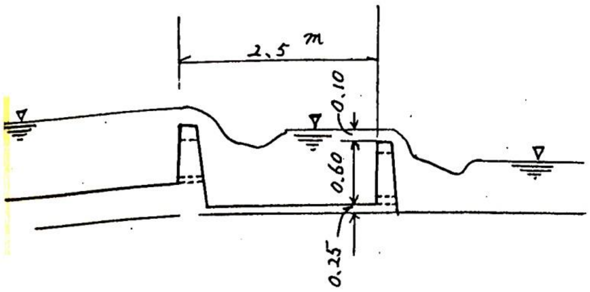
$$T_A = \frac{4}{3} \times \frac{3.05 - 1.50}{2.3 - 1} = 1.50 \text{ m} \quad 1.80 \text{ m} \quad \text{OK}$$

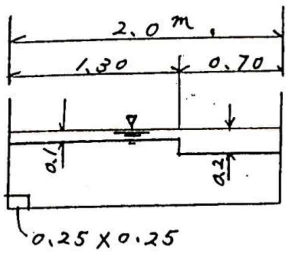
C. Design of fish way

Ladder method, Cobble pitching

Width 2.0 m, Slope 1/10, Overfolw depth 0.1 m

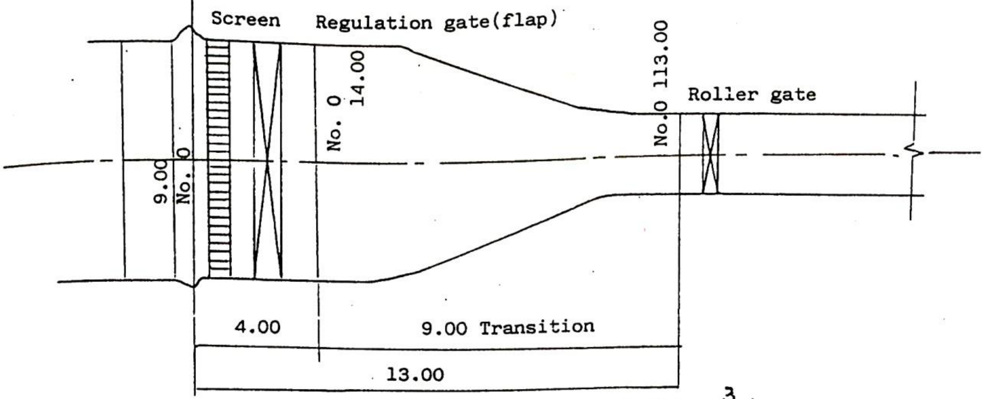
Discharge 0.259 m^3 x 2 = 0.518 m^3/sec


2.5 m
0.10
0.60
0.25


2.0 m
1.30
0.70
0.1
0.2
0.25 X 0.25

d. Design of intake (diversion works)

d-1) Design conditons



Maximum intake water $Q_{max} = 8.94\ m^3/sec$

Intake water depth $H = 1.25\ m$

Intake velocity $V = 0.80\ m/sec$

Intake velocity should be 0.80 m/sec for regulating the amount of intake water.

So, width of intake was calculated 9 m.

$B = Q/HV = 8.94/1.25 \times 0.80$

$= 8.94\ m$

$\doteqdot 9.0\ m$

d-2) Hydraulic calculation

Point N.0.0 + 13.00 water level E.L. 417.747 m

(Canal biginning point)

Head loss calculation was conducted from downstream point E.L. 417.747 m to upstream , then regulated intake water level E.L. 418.00 m was decided.

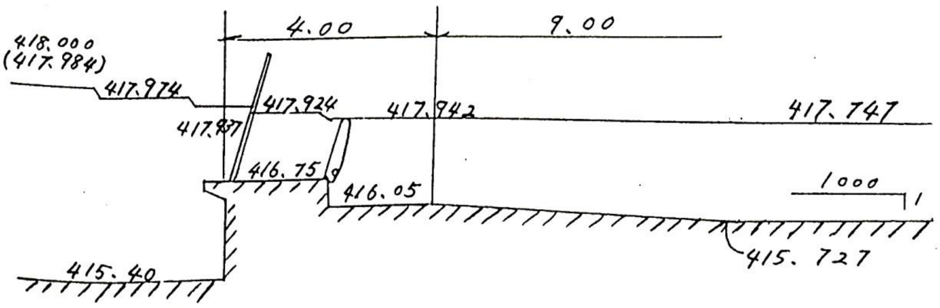
(1) Head loss by transition

(2) -do- by sudden enlargement of section

(3) -do- by screen

(4) -do- by step

(5) -do- by inlet



Annotation

This paper was translated from Japanese to English by K. Miyamoto very roughly. The original was written by H. Handa, irrigation engineer, He had been charged with the design and supervising of construction of Nishiiwasaki Head Works.