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BEHAVIOR OF A LARGE-SCALE ROCKFILL DAM CONSTRUCTED ON SAND AND GRAVEL FOUNDATION DURING FIRST FILLING OF RESERVOIR *

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1. INTRODUCTION

Chubetsu Dam is a multi-purpose dam 86m in height and with a total storage capacity of 93,000,000 m^3, It is constructed across the Chubetsu River that runs in the central part of Hokkaido, Japan. It is a combined type of a concrete

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* *Comportement d'un grand barrage en remblai construit sur une fondation de gravier sablonneux lors de la première mise en eau de la retenue*

gravity dam and a rockfill dam with a central earth core and the foundation for the rockfill dam is composed of sand and gravel layers.

Chubetsu Dam is Japan's largest combined type dam. The earth core base protection using bituminous mixture for the rockfill dam is the world's first foundation treatment. In this respect, approximately 800 units of 25 kinds of instruments were installed for construction and safety management.

The construction of Chubetsu Dam started in August 1995 and completed in March 2007, after first filling of reservoir, in which applied techniques were examined and the safety of the dam was demonstrated.

This paper reports the behavior of the dam and its foundation, which was proven to be safe through first filling of reservoir, focusing on the rockfill dam and the contact between the rockfill dam and the concrete gravity dam.

2. OVERVIEW OF DAM DESIGN

2.1. DESIGN OF THE DAM BODY AND ITS FOUNDATION

The site of the dam contains pyroxene andesites formed by the Neogene and middle-to-upper Miocene volcanism, which are covered with the late Quaternary consolidated sediment.

The riverbed sediment is up to 40 m deep, and divided into the slightly loose Holocene upper sand and gravel layer (ua-layer) and the more consolidated Pleistocene lower sand and gravel layer (la-layer) with the border of the weathered andesite layer. In consideration of geological characteristics, a concrete gravity dam was constructed along the left riverside and a rockfill dam on the sand and gravel layer (Fig.1).

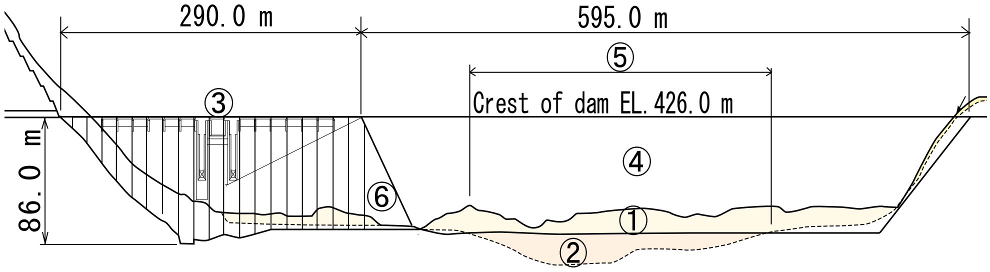

Fig.1
Longitudinal cross-section of Chubetsu Dam
Coupe longitudinale du barrage

1	Upper sand-gravel layer (ua-layer)	*1*	*Couche supérieure de graviers (Ua)*
2	Lower sand-gravel layer (la-layer)	*2*	*Couche inférieure de graviers* (ℓa)
3	Concrete gravity dam	*3*	*Barrage-poids en béton*
4	Rockfill dam	*4*	*Barrage en enrochement*
5	Sand-gravel foundation for the rockfill dam	*5*	*Partie du barrage en remblai fondée sur des graviers*
6	Contact of two type of dams	*6*	*Zone de contact entre les deux barrages*

The rockfill dam has a central earth core. The basement of the core and the filter zones was composed of the la-layer since, with its relatively law hydraulic conductivity, the la-layer had high resistance to seepage flows. To avoid the differential settlement of the concrete inspection gallery and the core as completely as possible, the elevation of the excavation base was set at 347.5 m, with its modulus of elasticity over 500 MPa. For the rock foundation, the weathered rock was excavated because it could have trouble in grouting because of its too high hydraulic conductivity.

Fig.2 shows the cross-section of the rockfill dam, which meets the required safety standards, as based on stability analysis using the sliding plane method, as well as the circular sliding method using seismic coefficients and modified seismic coefficients.

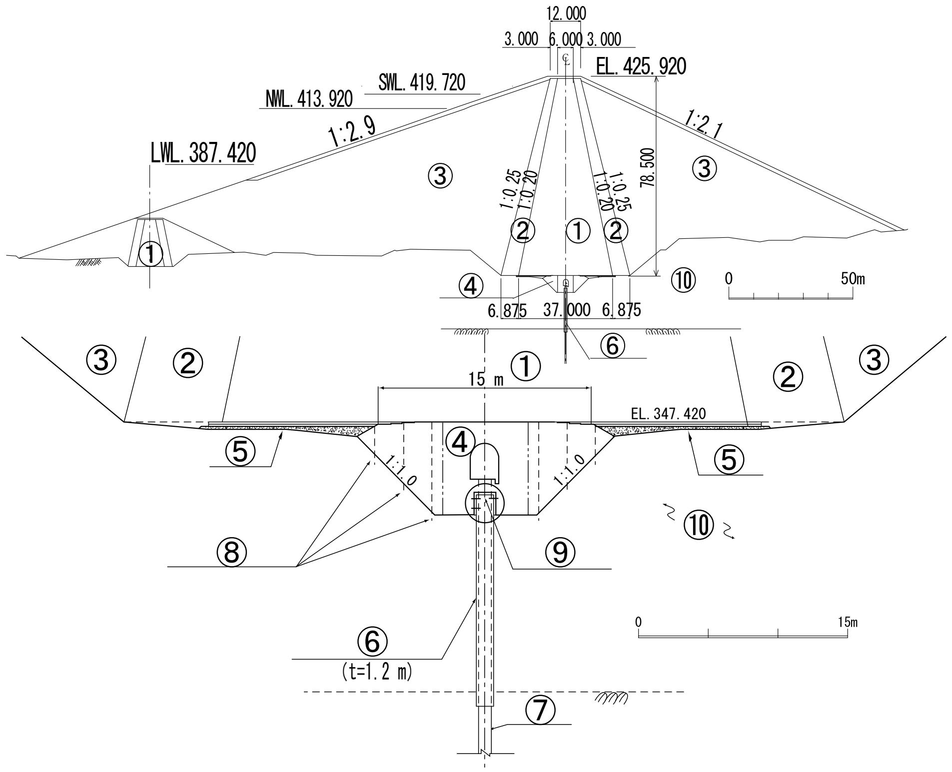

Fig. 2

Cross-section of the rockfill dam and its sand-gravel foundation

Traitement des fondations de la partie du barrage en remblai fondée sur des graviers

1	Core	*1*	*Noyau*
2	Filter	*2*	*Filtre*
3	Rock	*3*	*Roche*
4	Concrete inspection gallery	*4*	*Galerie de visite*
5	Core base protection	*5*	*Protection du noyau*
6	Diaphragm wall (reinforced concrete)	*6*	*Paroi étanche (béton armé)*
7	Curtain grouting	*7*	*Voile d'injection*
8	Contact grouting	*8*	*Injection de collage*
9	Joint between the inspection gallery and the diaphragm wall	*9*	*Zone de contact entre la galerie et la paroi étanche*
10	Lower sand-gravel layer	*10*	*Couche inférieure de graviers sablonneux*

2.2. FOUNDATION TREATMENT FOR THE ROCKFILL DAM

For the sand and gravel foundation, a diaphragm wall, a concrete inspection gallery and core base protections were constructed as foundation treatment to ensure the hydrological safety of the dam (Fig.2).In addition, the impermeability of the rock foundation below the sand-gravel foundation was improved by curtain grouting.

2.3. CONTACT BETWEEN THE ROCKFILL DAM AND THE CONCRETE GRAVITY DAM

For the contact between the concrete dam and the rockfill dam, stress analysis was conducted using the finite element method, considering parameters such as the gradient of the contact surface and the position of the core foundation in the contact. Based on the analysis, the following two points were demonstrated:

1) When the contact surface gradient becomes steeper, the local safety factor in the core is over 1.2. There was no significant decrease in safety factor.
2) When the contact surface gradient becomes steeper, the minimum principal stress decreased but was greater than the pore water pressure, with no possibility of hydraulic fracturing.

With reference to the example of another combined type dam (53 m in height; 1 to 0.65 in contact surface gradient), the gradient of the contact surface of the two types of dams of Chubetsu Dam is 1 to 0.7. The height of the contact is as high as 76 m. The core was directly connected to the concrete dam and the shell of the rockfill dam was embanked around the concrete dam (Fig.3).

2.4. MEASUREMENT FOR THE ROCKFILL DAM

The measurement of the behavior of Chubetsu Dam focused on ensuring that it is dynamically and hydrogeologically safe. This included the measurement of deformation in the sand-gravel foundation; the deformation and seepage in the core base protection, a concrete inspection gallery, a diaphragm wall and joints, and the deformation in the contact between the dams.

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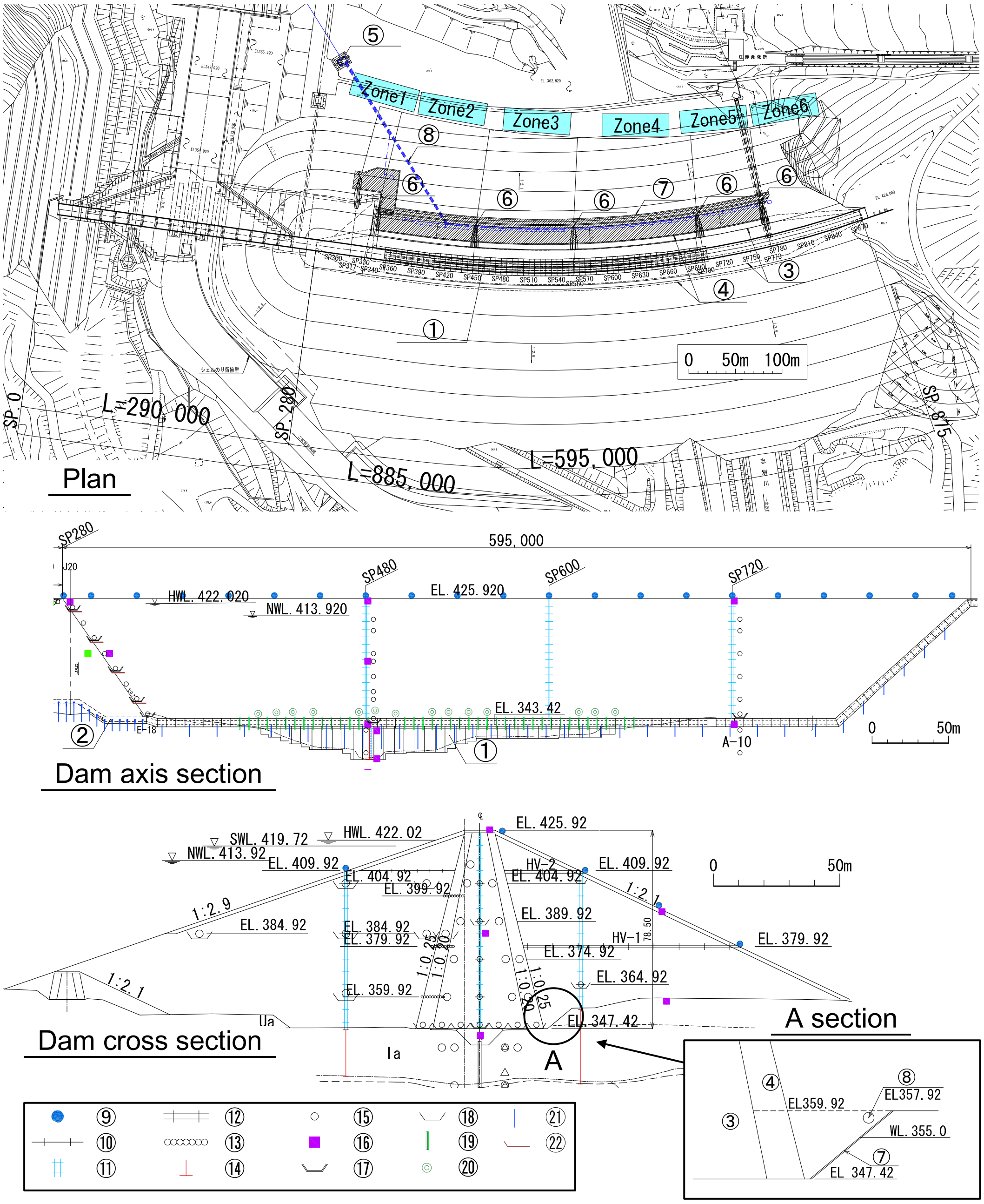

Plan
Zone1
Zone2
Zone3
Zone4
Zone5
Zone6
L=290, 000
L=885, 000
L=595, 000
SP. 0
SP. 280
SP. 875
0 50m 100m
Dam axis section
595, 000
SP280
SP480
SP600
SP720
HWL. 422. 020
NWL. 413. 920
EL. 425. 920
EL. 343. 42
0 50m
Dam cross section
SWL. 419. 72
HWL. 422. 02
NWL. 413. 92
EL. 425. 92
EL. 409. 92
EL. 404. 92
EL. 399. 92
EL. 384. 92
EL. 379. 92
EL. 359. 92
EL. 389. 92
EL. 374. 92
EL. 364. 92
EL. 347. 42
1:2. 9
1:2. 1
1:0. 25
1:0. 20
0 50m
A
A section
EL359. 92
EL357. 92
WL. 355. 0
EL 347. 42

3. BEHAVIOR OF THE ROCKFILL DAM DURING FIRST FILLNG OF RESERVOIR

3.1. Overview of first filling of reservoir

First filling of reservoir at Chubetsu Dam started in March 30, 2006. The water reached its low water level of EL.387.42 m on April 25th and its normal water level of EL.413.92 m on June 24th. The normal water level was maintained during the flooding period until October 31st. Then, on November 1st, the water level was again raised. On November 21st, the water reached its surcharge level of EL. 419.72 m, which was maintained for 24 hours to ensure the safety of the dam. Then, the water level was lowered until it reached its low water level on January 5th, 2007. On March 28th, the first filling of reservoir was finally completed (Fig. 4).

Fig. 3
Arrangement of instrumentation
Répartition des équipements de mesure

1	Diaphragm wall	*1*	*Paroi étanche*
2	Inspection gallery	*2*	*Galerie*
3	Core	*3*	*Noyau*
4	Filter	*4*	*Filtre*
5	Seepage observation station	*5*	*Salle de mesure du débit des fuites d'eau*
6	Dividing wall	*6*	*Cloisons de séparation des écoulements d'eau*
7	Impervious sheet	*7*	*Feuille étanche*
8	Seepage collection line	*8*	*Canal de collecte des percolations*
9	Surface displacement gauge	*9*	*Capteur des déplacements en surface*
10	Relative horizontal displacement gauge	*10*	*Mesure des déplacements horizontaux relatifs*
11	Multi-layer settlement gauge	*11*	*Tassomètres (par couches)*
12	Relative horizontal/vertical displacement gauge	*12*	*Mesure des déplacements horizontaux / verticaux relatifs*
13	Shear displacement gauge for contact surface	*13*	*Mesure de la résistance au cisaillement de la surface de contact*
14	Combined probe extensometer	*14*	*Mesure des déplacements du bedrock*
15	Pore water pressure gauge	*15*	*Piézomètre*
16	Strong-motion seismograph	*16*	*Sismographe pour secousses fortes*
17	Five-facet earth pressure gauge	*17*	*Mesure de la pression des terres (cinq côtés)*
18	Three-facet earth pressure gauge	*18*	*Cellule de pression des terres (trois côtés)*
19	Inspection gallery joint gauge	*19*	*Mesure de l'ouverture des joints de la galerie*
20	Pore water pressure gauge for monitoring core base protections	*20*	*Piézomètre pour la surveillance du revêtement du noyau*
21	Observation hole for seepage flows (pressure)	*21*	*Forage de contrôle des débits (et pressions) de percolation dans la zone de contact*
22	Shear displacement gauge for treated joint	*22*	*Mesure des déplacements de cisaillement au droit du joint traité*

3.2. SEEPAGE BEHAVIOR IN THE ROCKFILL DAM

Seepage monitoring in the rockfill dam was conducted by inspection to measure seepage and deformation in the dam body and the foundation as well as to check any abnormalities such as fissures in the inspection gallery and contamination of seepage water. Based on the behavior measurement, the safety of the dam was evaluated.

3.2.1. Seepage flows in the dam body and the foundation

Seepage flows in the rockfill dam were measured in six zones in which the area between the filter and the rock in the downstream side of the core is divided by walls (dividing walls made up of the core materials 3 to 8 m in width; Fig.3).

Impervious sheets were placed on the trench slope in the downstream side of the core so that the measurement would not be affected by water levels. Seepage flows were collected in a line (seepage collection line; diameter of 200 mm) installed at EL.357.92 m, conducted down the line into an observation station, and automatically measured in a V-notch weir (Fig.3). Monitoring focused on ensuring that there is no rapid change in the amount of seepage flows compared with change in reservoir water levels.

In Zones 1 and 2 on the left-bank side, the volume of seepage flows from the foundation was too small for measurement. In Zones 3, 4, 5 and 6, substantial volumes of seepage flows were observed when the snow thawed or when it rained, even before first filling of reservoir. To ensure that seepage flows were accurately measured; those from the reservoir were used for the measurement so that these will not be affected by a thaw or rainfall. Therefore, seepage flows due to rainfall was estimated when there was no thaw and when it did not snow, using a highly-reproducible tank model, while seepage flows due to a thaw was estimated using a temperature-rainfall correlation equation. [2].

Fig.4 shows seepage flows based on the above estimation that is not affected by a thaw or rainfall. In each Zone, there is a difference in seepage flows between, until and when the water reaches its normal water level, however, they are generally in linear correlation with the reservoir water levels. In Zone 3, particularly, which has a diaphragm wall for water interception, the volume of seepage flows is less than in other Zones. Moreover, there was no muddiness in the observed seepage.

Based on the correlation between the above estimated seepage flows and reservoir water levels, it was found that the seepage flows directly coming from the reservoir are relatively less and highly correlated with the reservoir water levels. This behavior was considered safe.

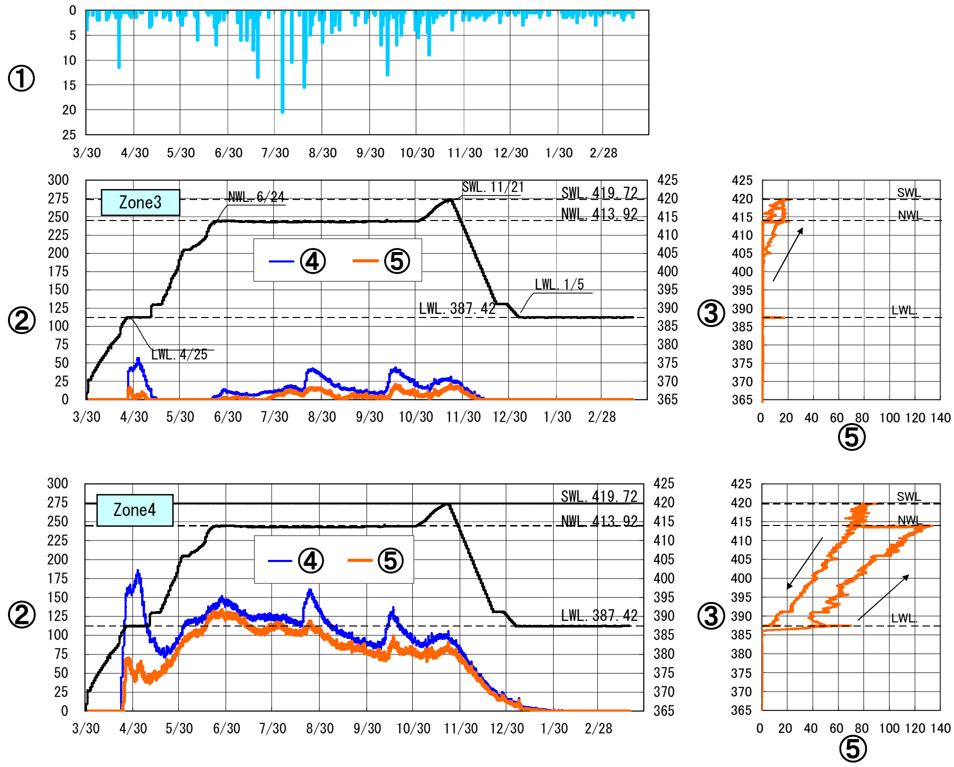


Fig. 4
Change in seepage flows in the rockfill dam
Diagramme des variations de la percolation dans le barrage en remblai

1 Daily rainfall (mm)
2 Seepage (L/m)
3 Reservoir water level (EL; m)
4 Observed seepage (L/m)
5 Seepage without the influence of a thaw or rainfall (L/m)

1 Précipitations journalières (mm)
2 Percolation (ℓ/m)
3 Cote du plan d'eau EL (EL; m)
4 Percolation observée (ℓ/m)
5 Percolation après élimination de l'incidence des précipitations de pluie et de la fonte des neiges (ℓ/m)

3.2.2. Seepage pressure in the foundation

Observation holes were excavated in the foundation from the inspection gallery of the rockfill dam body to manually measure the seepage pressure.

Perforated pipes were installed within the observation holes at intervals of 12 m in the sand-gravel foundation and at intervals of 18 m in the rock foundation. The length of the pipe for the sand-gravel foundation varies from 7 to 15 m depending on the depth, while that for the rock foundation is 7 m (Fig. 5). Monitoring focused on the seepage pressure and any rapid change in it.

As the longitudinal cross-section of the rockfill dam with the distribution of seepage pressure (Fig. 5) shows, seepage pressure in the rock foundation was higher than that in the sand-gravel foundation. Especially, seepage pressure in Zone 3 in the central part of the sand-gravel foundation was almost constant while the reservoir water level changed, whereas seepage pressure in Zones other than Zone 3 increased when reservoir water levels were higher (Fig.5).

Seepage flows in the rock foundation have correlation with reservoir water levels, while those in the sand-gravel foundation were considered to be intercepted by the diaphragm wall. The behavior was considered to be ascribable to the dam foundation condition and the foundation treatment.

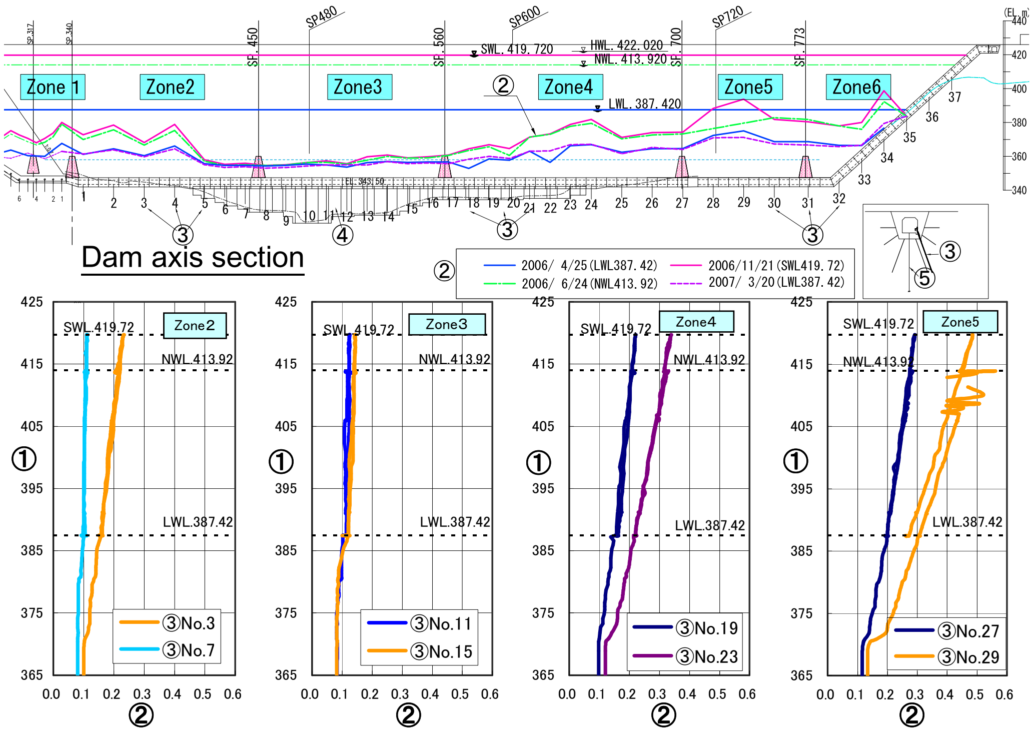


Fig.5

Seepage pressure in the foundation for the rockfill dam

Pression des percolations dans la fondation du barrage en enrochement

1	Reservoir water level (EL; m)	*1*	*Cote du plan d'eau EL (EL; m)*
2	Seepage pressure (MPa)	*2*	*Pression des percolations (MPa)*
3	Seepage observation hole	*3*	*Forages de contrôle des percolations*
4	Diaphragm wall	*4*	*Paroi étanche*
5	Curtain grouting	*5*	*Voile d'injection*

3.2.3. Pore water pressure in the earth core

Pore water pressure was automatically measured along three sections, namely, in SP280 (dam contact), SP480 (sand-gravel foundation) and SP720 (rock foundation) using pore water pressure gauges installed in the core, the filter and the shell as well as in the foundation (sand-gravel foundation and rock foundation). Monitoring as to seepage behavior focused on ensuring that there is no rapid change in pore water pressure and that pore water pressure on the downstream side is lower and more constant than that on the upstream side. In this paper, the distribution of pore water pressure in the monitored cross-section will be discussed.

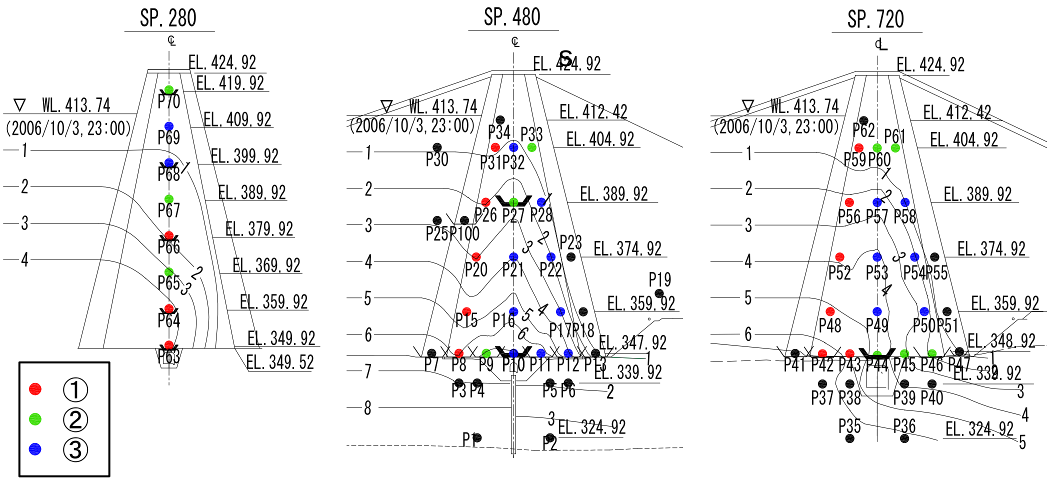


Fig.6

Distribution of ratio of pore water pressure to reservoir water pressure (as of October 3, 2006)

Distribution des pressions interstitielles en fonction de la pression hydrostatique du plan d'eau (3-10-2006)

1 The ratio decreases and then increases until the water reaches NWL.	*1 La valeur du rapport baisse, puis augmente en fonction du niveau de la retenue pour atteindre son maximum lorsque la retenue s'établit.*
2 The ratio decreases and becomes almost constant until the water reaches NWL.	*2 La valeur du rapport baisse pour devenir quasiment constant lorsque la retenue s'établit*
3 The ratio decreases until the water reaches NWL and continues to decrease while NWL is maintained.	*3 La valeur du rapport baisse progressivement en fonction de la montée du niveau amont, et continue à baiser lorsque la retenue est maintenu à ce palier (NWL)*

Pore water pressure along each line in the core was as follows:

SP280 (dam contact): Pore water pressure was almost constant while reservoir water level changed, although P63 and P64 at lower elevations quite slightly increased at EL. 405 m and above.

SP480 and SP720: Pore water pressure on the upstream side and in the central part increased almost in proportion to the increase in reservoir water level, but there was no rapid increase. On the other hand, pore water pressure (SP480: P8, SP720: P46) on the downstream side was almost constant. In the distribution of pore water pressure, there was difference between SP480 and SP720 especially around the core base and the pressure that remained in SP480.

To understand the overall behavior of the dam with regards to the pore water pressure, the ratio of pore water pressure to reservoir water pressure, as shown in Fig. 6 (above), was calculated respectively in SP280, SP480 and SP720. The results show that there is no abnormality in the distribution of pore water pressure.

3.3. DEFORMATION BEHAVIOR OF THE DAM BODY

To monitor deformation behavior of the dam body, the external and internal deformations were measured. In this paper, only the external deformation will be discussed.

Monitoring the deformation of the rockfill dam included visual inspection and geodimeter measurement, with targets set at intervals of 30 m the upstream and downstream sides, and on top of the dam (including five traverse target lines of SP280, SP360, SP480, SP600 and SP720).

Although there were different degrees of deformation, the following points were observed (Fig. 7):

- As the reservoir water level rose, horizontal displacement was observed on the downstream side. It was approximately 2.4 cm at its maximum on top of the dam (SP570 in the central part of the rockfill dam) when the water reached its surcharge water level.
- As the reservoir water level dropped, a slight displacement was observed on the upstream side and its behavior was almost elastic.
- Vertical displacement (settlement) increased with time. Especially, when reservoir water level dropped from the surcharge water level, settlement increased and was approximately 2.5 cm at its maximum on top of the dam and approximately 4 cm in the downstream side.

The exterior deformation was typically ascribable to an increase in the reservoir water level, although there was no rapid change in deformation. Although, the central part of the dam body has sand-gravel foundation, it was concluded that said foundation does not contribute to any significant deformation or settlement.

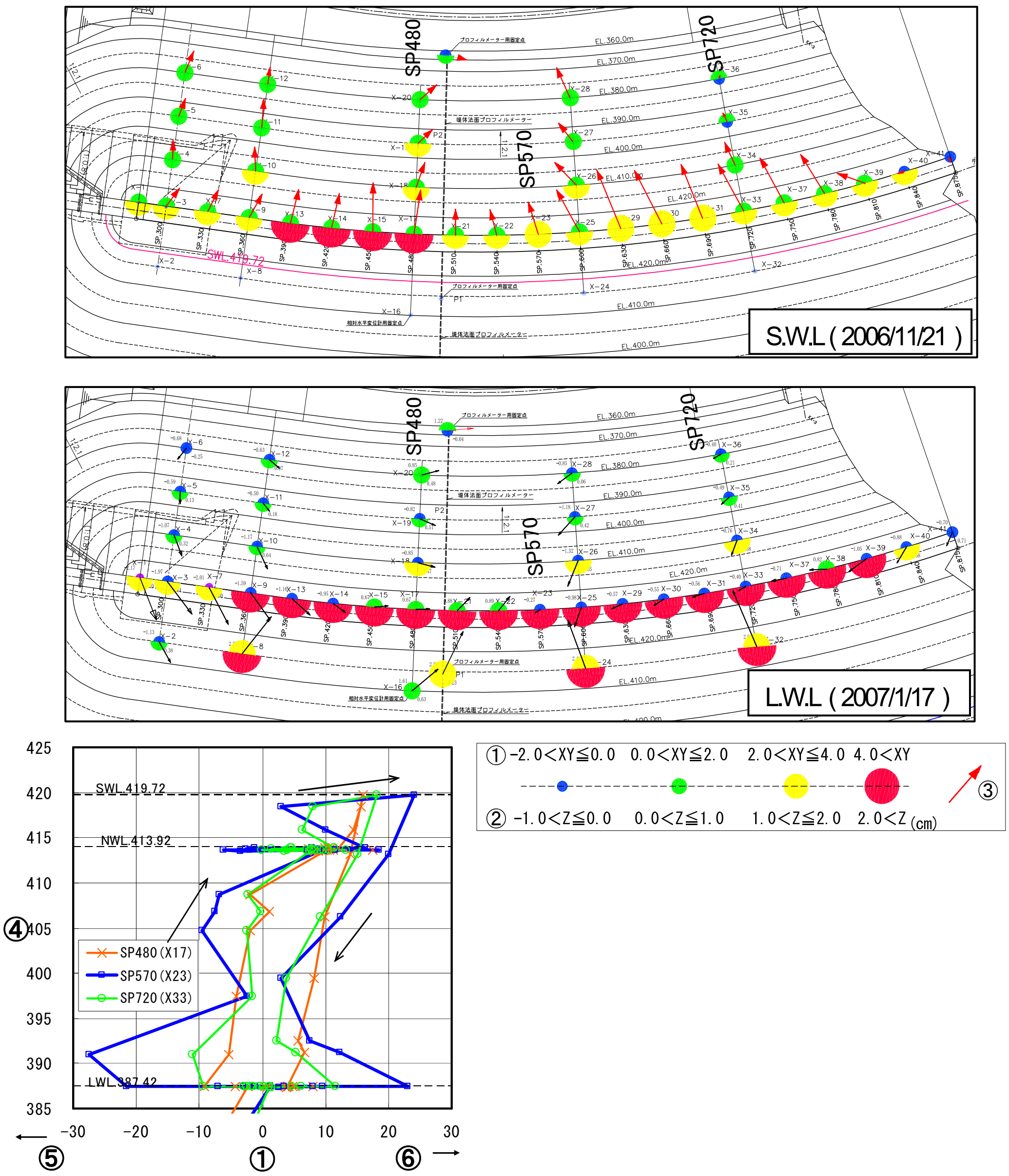


Fig. 7

Dam surface displacement distribution

Diagramme de classification des déplacements du barrage

1	Horizontal displacement (cm)	*1*	*Déplacements horizontaux (cm)*
2	Settlement (cm)	*2*	*Tassement (cm)*
3	Horizontal displacement direction	*3*	*Direction des déplacements horizontaux*
4	Reservoir water level (EL; m)	*4*	*Cote du plan d'eau EL (EL ; m)*
5	Upstream side (-)	*5*	*Amont (-)*
6	Downstream side (+)	*6*	*Aval (+)*
7	Sand-gravel foundation	*7*	*Partie en graviers de la fondation*

3.4. BEHAVIOR OF CORE BASE PROTECTIONS

Since cement-based grouting is not effective for the sand-gravel core foundation, core base protections were constructed using bituminous mixture to reduce seepage flows in the boundary between the core and the sand-gravel foundation.

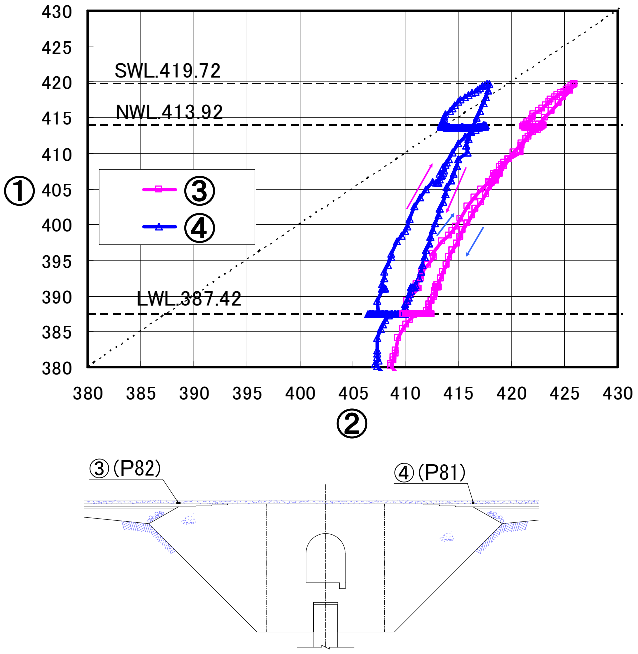
To monitor the core base protections, pore water pressure was automatically measured using a circulation-type pore water pressure gauge in the inspection gallery joints on the upstream and downstream sides where distortion was at its maximum.

Since any damage to the core base protections causes a rapid change in pore water pressure, monitoring was focused on this.

Pore water pressure in the core base protections changed more slowly than the reservoir water level (Fig.8). As the reservoir water level rose, the difference in water level between the upstream and downstream sides became greater, which showed that seepage in the core base protections was well controlled.

Pore water pressure was generally high because it was difficult to disperse due to the extremely low-permeable compacted fine-grain material of the core and impervious core base protections.

In summary, no rapid change in pore water pressure was observed and consequently the core base protections were found to be impervious.



3.5. BEHAVIOR OF THE INSPECTION GALLERY AND THE DIAPHRAGM WALL

3.5.1. Displacement in the inspection gallery

Since the inspection gallery has sand-gravel foundation, joints were set longitudinally at intervals of 6 m (longitudinal joints). To secure an extension of core base protections, a concrete inspection gallery was constructed with its width of 15 m being equal to 20 % of the maximum water depth. Joints were also set to reduce stress (stress joints) in the upper and downstream side. Monitoring of the inspection gallery for the safety of the dam focused on joint displacement and differential displacement.

Displacement measurement was automatically taken using displacement gauges set in the longitudinal and stress joints on top of the concrete inspection gallery.

In both longitudinal and stress joints, there was no displacement during first filling of reservoir (Fig.9). Joint displacement was mostly observed in the embankment and converged before first filling of reservoir.

Longitudinal joints were displaced where the depth of the sand-gravel layer changed and the displacement was 4 to 6 mm at its maximum.

The displacement of stress joints was in proportion to the depth of the sand-gravel layer with a maximum of approximately 20 mm in the central part of the thickest sand-gravel layer. On both sides where the layer is thinner, displacement was also smaller. On the right of the central part, a difference in the displacement was observed between the upper and downstream sides: approximately 15 to 20 mm displacement in the joint in the upper reach and approximately 10 mm in the downstream side, for a maximum difference of approximately 10 mm.

Fig. 8
Correlation between pore water pressure in core base protections
and reservoir water level (SP486)
Rapport entre la pression interstitielle dans la protection du noyau
et la cote du plan d'eau (SP486)

1	Reservoir water level (EL; m)	*1*	*Cote du plan d'eau (EL ; m)*
2	Pore water level (EL; m)	*2*	*Niveau de l'eau interstitielle (EL ; m)*
3	Pore water pressure gauge on the upstream side	*3*	*Piézomètre amont*
4	Pore water pressure gauge on the downstream side	*4*	*Piézomètre aval*

Joint grouting was conducted for the longitudinal joints with over 10 mm displacement in the central part before first filling of reservoir.

Generally, deformation was considered to depend on the depth of the sand-gravel layer, as estimated.

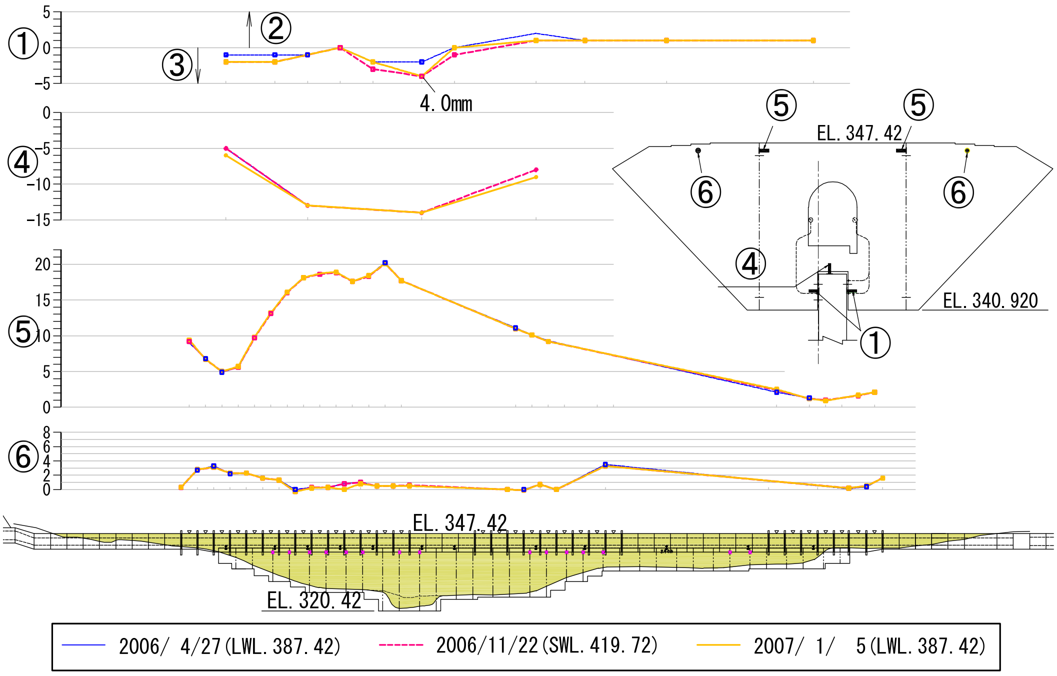


Fig.9

Seepage pressure in the joint between the inspection gallery and the diaphragm wall & longitudinal cross-section of displacement measurement

Pression de percolation dans la zone de contact entre la galerie et la paroi étanche - Coupe longitudinale et diagramme des déplacements

1	Horizontal displacement in the gallery-wall joint (mm)	*1*	*Déplacements horizontaux de la zone de contact galerie/paroi (mm)*
2	Tension	*2*	*Variations dans la direction de la dilatation*
3	Compression	*3*	*Variations dans la direction de la compression*
4	Vertical displacement in the gallery-wall joint (Compression; mm)	*4*	*Déplacements verticaux de la zone de contact galerie/paroi (compression) (mm)*
5	Displacement of stress joints in the inspection gallery (mm)	*5*	*Déplacement des joints transversaux fortement sollicités de la galerie (mm)*
6	Displacement of longitudinal joints in the inspection gallery (mm)	*6*	*Déplacement des joints longitudinaux de la galerie (mm)*

3.5.2. Displacement in the diaphragm wall

Seepage monitoring in the diaphragm wall is indispensable for the safety of the dam. Normal seepage behavior was ensured in the observation holes for seepage pressure measurement (Fig.5). Displacement in the diaphragm wall is discussed hereunder in terms of its behavior.

Displacement in the diaphragm wall was measured using a tiltmeter fixed to the B-6 Element, the longest element of the wall.

Fig. 10 shows horizontal displacement measured using the tiltmeter during first filling of reservoir. It was more significant in the upper parts. There was no displacement in the lower parts, because the diaphragm wall is embedded in the rock. As the reservoir water level rose, there was horizontal displacement toward the downstream side at around the joint with the inspection gallery in the upper parts. The displacement was approximately 6 mm when the water reached its surcharge water level. As the water level dropped, the horizontal displacement is reduced.

On the other hand, the displacement gauge in the joint between the inspection gallery and the diaphragm wall showed approximately 2 mm horizontal compression on the downstream side. It was therefore considered that the inspection gallery itself moved approximately 4 mm (in a range between 2 to 6 mm) toward the downstream side.

The stress gauge for reinforcing bar showed approximately 130 MPa tensile stress as its maximum, and it was considered that the behavior is elastic as estimated.

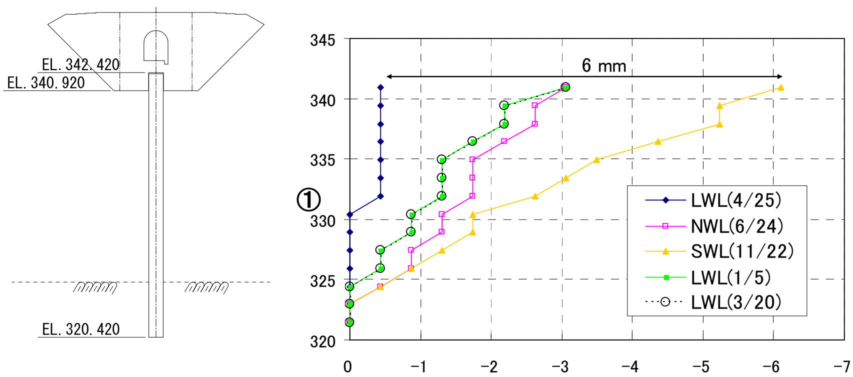


Fig. 10

Horizontal displacement in the diaphragm wall (tiltmeter)

Déplacements horizontaux de la paroi étanche (Inclinomètre)

1	Elevation (EL; m)	*1*	*Hauteur ((EL; m)*
2	Horizontal displacement during first filling of reservoir	*2*	*Déplacements horizontaux à partir du déclenchement de l'essai d'immersion (mm)*

3.5.3. Joint between the inspection gallery and the diaphragm wall

The inspection gallery was separated from the diaphragm wall about 3 cm on the upstream side, 5 cm on the downstream side, and 10 cm at the top between them so that any deformation of one would not affect the other. Seepage control within the gap is maintained using specially designed rubber waterstops as well as asphalt fillings and bentonite sealants applied to the top of the joint.

To ensure the safety of the gallery-wall joint, the width of the gap between the inspection gallery and the diaphragm wall and the seepage needed to be monitored. Displacement (compression and tension) in the gap was automatically measured using deformation gauges installed at the top and sides of the diaphragm wall. Furthermore, in the observation hole for seepage pressure measurement, sensors were set for purpose of automatic measurement, taking into consideration the location of the waterstops.

1) Displacement in the gallery-wall joint

Vertical displacement of the gallery-wall joint was found to be proportional to the depth of the sand-gravel layer, with the maximum of approximately 14 mm compression. Displacement during reservoir impounding was insignificant (Fig.9).

Horizontal displacement was not observed on the upstream side but on the downstream side; it was approximately 2 mm at its maximum (compression) where the depth of the sand-gravel layer is the greatest. The compression increased by approximately 2 mm during first filling of reservoir. This means that there was a decrease of approximately 4 mm at its maximum in the width of the gap.

There was no rapid change in displacement in the gallery-wall joint and the ratio of displacement to the width of the gap was small. This behavior of gallery-wall joint was considered to indicate stability.

2) Seepage monitoring

Seepage pressure on the upstream side of the gallery-wall joint was correlated with the reservoir water levels, while that in the lower part of the water stop on the downstream side was not and was almost constant (Fig.11). Therefore, the impermeability of the gallery-wall joint was considered to be ensured by the water stop on the downstream side.

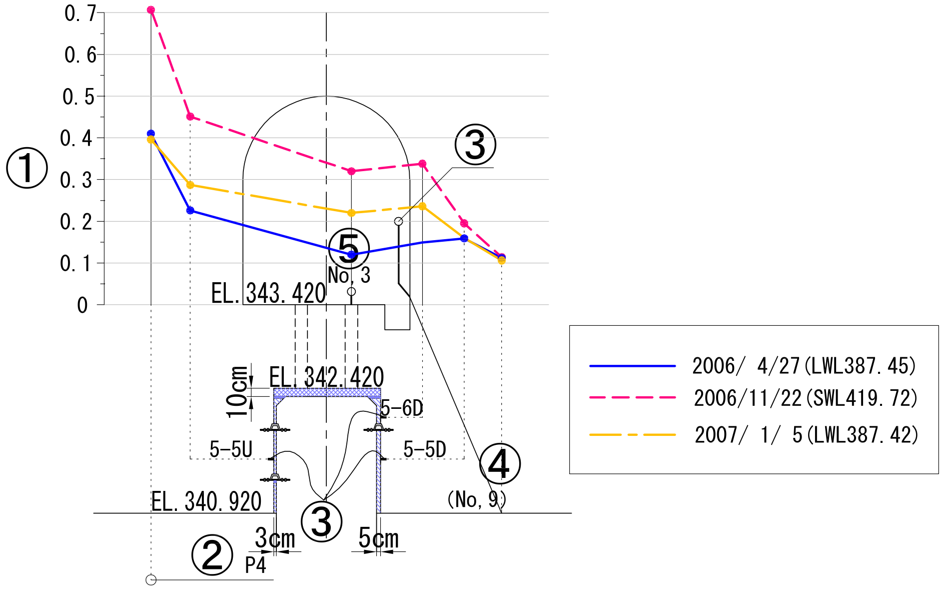


Fig.11

Seepage pressure in the joint between the inspection gallery and the diaphragm wall & cross-section distribution (BL23)

Pression de percolation dans la zone de contact entre la galerie et la paroi étanche et coupe (BL23)

1	Seepage pressure (MPa)	*1*	*Pression de percolation (MPa)*
2	Pore water pressure gauge P4	*2*	*Piézomètre dans la fondation P4*
3	Seepage pressure observation hole in the gallery-wall joint	*3*	*Forage de contrôle de la pression de percolation dans la zone de contact galerie/paroi*
4	Seepage pressure observation hole on the downstream side No.9	*4*	*Forage de contrôle de la pression de percolation dans la fondation en aval No.9*
5	Uplift gauge No.3	*5*	*Jauge de sous-pression No.3*

CONCLUSIONS

Seepage pressure measured in the downstream side of the dam was generally in linear correlation with the reservoir water levels and did not change significantly. It was therefore demonstrated that the dam body and its foundation are water tight.

- The water tightness of the diaphragm wall was high while having no significant correlation with seepage pressure on the downstream side and reservoir water levels.
- The behavior of horizontal displacement of the dam body was elastic and in accordance to reservoir water levels, while vertical displacement is

small and stable. The density and stability of the dam body were therefore demonstrated as acceptable.

- In the joint between the inspection gallery and the diaphragm wall, there was no significant displacement during the first filling of reservoir and the joint remains stable. The sand-gravel foundation was considered to have been solidly constructed.

Deeply rooted in the earth, Chubetsu Dam and its original beautiful landscape give its community more life and well-being.

REFERENCES

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SUMMARY

The safety management measurement at Chubetsu Dam was described and correctly explained in this paper and, by identifying their correlation with other related measurements, the safety of the dam was demonstrated. The behavior of the rockfill dam during first filling of reservoir is summarized as follows:

The overall behavior of the dam body and the foundation was demonstrated by monitoring seepage flows. Seepage flows in the downstream side of the dam were generally in linear correlation with reservoir water levels and showed stable water tightness behavior.

On the other hand, seepage in the rock foundation and the sand-gravel foundation was in correlation with the foundation treatment. Since the sand-gravel foundation has a diaphragm wall, seepage pressure in the downstream side of the diaphragm wall and pore water pressure in the foundation were carefully correlated with reservoir water levels. The sand-gravel foundation generally indicates water levels on the downstream side. This indicates the excellent watertightness of the diaphragm wall.

Horizontal displacement in the dam body changed in correlation with reservoir water levels. The overall behavior was elastic. Exterior deformation on top of the dam moved toward the downstream side by approximately 3 cm at its maximum, when the water reached its surcharge water level. It shifted again towards the upstream side when the reservoir water level dropped. The behavior however remains elastic. Vertical displacement (settlement) increased with time but the settlement due to first filling of reservoir was small. The embankment was considered to be dense and solid. There was no significant difference in deformation between the sand-gravel foundation and the rock foundation.

With regards to the displacement in the gap between the inspection gallery and the diaphragm wall, settlement was almost not visible during the construction of the embankment. With regards to horizontal displacement, the diaphragm wall was displaced by approximately 6 mm towards the downstream side during first filling of reservoir. This displacement due to first filling of reservoir was considered small and the structure's behavior remains stable.

RÉSUMÉ

La campagne de mesures effectuée au barrage de Chubetsu fut décrite et correctement expliquée dans ce document, et en identifiant sa corrélation avec d'autres mesures pertinentes, la sûreté du barrage fut ainsi démontrée. Le comportement du barrage en remblai rocheux lors de la première mise en eau est résumé comme suit :

Le comportement global du corps du barrage et de la fondation fut démontré par mesure du débit de percolation. Le débit de percolation détecté en aval du barrage était généralement en corrélation linéaire avec les niveaux d'eau du réservoir.

Par contre, la percolation dans la fondation (rocher et gravier sablonneux) était en corrélation avec le traitement de celle-ci. Puisque la fondation en gravier sablonneux comporte une paroi moulée, la pression de percolation à l'aval de la paroi moulée et la pression interstitielle dans la fondation furent soigneusement corrélées avec les niveaux d'eau du réservoir. La fondation en gravier sablonneux généralement reflète les niveaux d'eau aval. Cette situation indique une excellent étanchéité de la paroi moulée..

Le déplacement horizontal du corps du barrage s'est modifié en corrélation avec les niveaux d'eau du réservoir. La loi de comportement globale était du type élastique. Les déformations extérieures de la partie supérieure du barrage correspondaient à un déplacement de 3 cm maximum ~~en~~ vers l'aval lorsque l'eau dans le réservoir atteignait le niveau des plus hautes eaux, pour revenir à nouveau vers l'amont lorsque le niveau du réservoir baisse. Ainsi, le

comportement extérieur demeurait élastique. Un déplacement vertical (tassement) a augmenté avec le temps mais le tassement causé par la mise en eau du réservoir fut peu important. Le remblai fut jugé dense et solide. Il n'y avait pas de différence significative entre les déformations de la fondation en gravier sablonneux et celles de la fondation rocheux.

Concernant le déplacement dans l'espace entre la galerie de contrôle et la paroi moulée, le tassement n'était presque pas visible lors de la construction du remblai. Quant aux déplacements horizontaux, la paroi moulée fut déplacée d'environ 6 mm vers l'aval lors de la mise en eaux. Ce déplacement causé par le remplissage du réservoir fut jugé minime et le comportement de l'ouvrage reste constant.