Report on the seismic reinforcement work of Sagamihara Sedimentation Basin

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Abstract

Sagamihara Sedimentation Basin is a structure of the earth-fill dam type for removing suspended solids in raw water via deposition. In FY 2011, the bank was inspected for seismic resistance. The inspection showed that the main bank lacked the required seismic resistance on the side facing the basin (referred to as the "upstream face").

By considering constraints on construction, etc., the basin was decided to be seismically retrofitted by replacing the outside part (referred to as the "downstream face") of the bank with reinforced embankment of soil cement.

1. INTRODUCTION

Yokohama Water Works Bureau supplies a daily average of approximately 1.15 million m^3 of drinking water to approximately 3.7 million citizens.

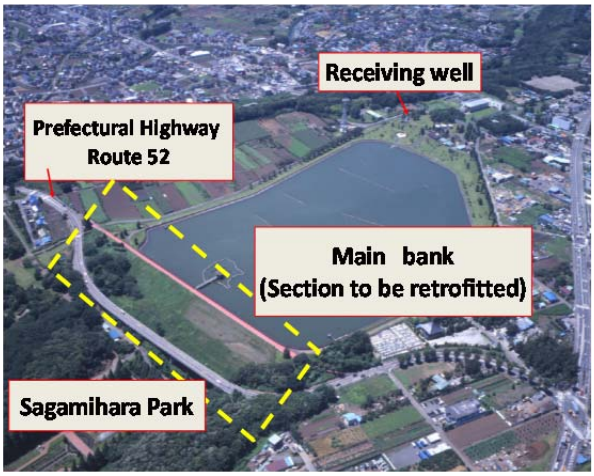


Fig. 1.1 An entire view of Sagamihara Sedimentation Basin

Since the establishment of waterworks in 1887, the Bureau has expanded its facilities eight times in order to respond to population growth and the expansion of the urban areas. Today, the facilities are being improved on seismic resistance and water treatment functions based on the "Long-term vision and 10-year plan", which was formulated in 2006.

This paper is a report on the seismic reinforcement work on the main bank of Sagamihara Sedimentation Basin.

2. OVERVIEW OF SAGAMIHARA SEDIMENTATION BASIN

Sagamihara Sedimentation Basin was constructed in 1954 by making use of the natural landform. This basin is located between Lake Sagami, which is one of water resources of Yokohama City, and Nishiya water purification plant (Fig. 2.1). Its structure is the earth-fill dam type. (Fig.2.2 and 2.3) It can store 883,000 m3 of water as emergency water storage and remove suspended solids in raw water via deposition. In case that the turbidity of the raw water becomes high such as during a typhoon or storm, the sedimentation is accelerated by adding PAC (Poly aluminum chloride).

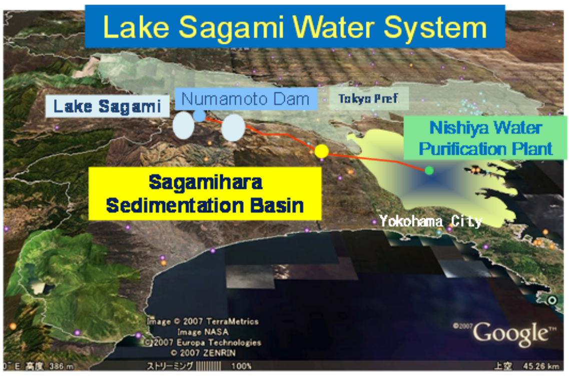


Fig. 2.1 Locality map of Sagamihara Sedimentation Basin

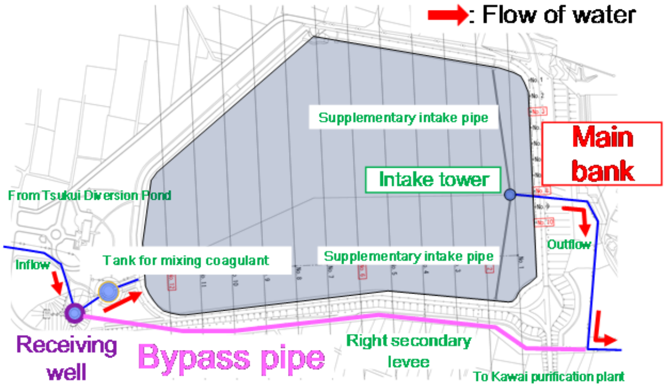


Fig. 2.2 Plan of Sagamihara Sedimentation Basin

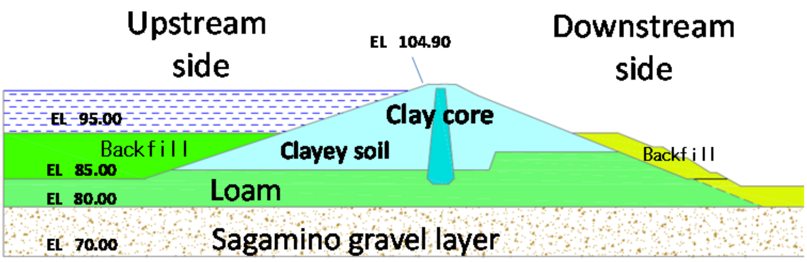


Fig. 2.3 Representative sectional view of Sagamihara Sedimentation Basin

3. CIRCUMSTANCES OF THE SEISMIC RESISTANCE INVESTIGATION

Yokohama Water Works Bureau examined the seismic resistance of Sagamihara Sedimentation Basin in 1982 and discovered that the slip surface of the downstream face of the main bank lacked seismic resistance. Therefore, the main bank was retrofitted by cutting and installing counterweight fill along the downstream face of the bank (Fig. 3.1). Later, in 1997 and 2009, the Seismic Design Guideline for Water Works Facilities (hereinafter referred to as the “Guidelines”) was revised requiring for a raised seismic resistance level. Because there are a university and Sagamihara Park, which is a governmentally designated evacuation site at the time of disaster, near the basin, collapse of the bank was feared to lead to secondary damage and affect people and properties. A dam which had a structure similar to that of this basin collapsed in the Great East Japan Earthquake. Therefore, the seismic resistance of the basin was checked based on the Guidelines (2009). The main bank was found to not have the required seismic resistance, and retrofit work started in 2013.

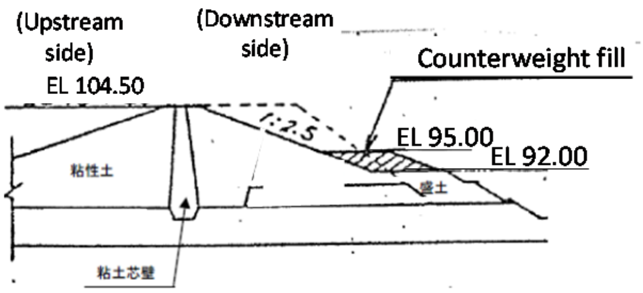


Fig. 3.1 Outline map of the retrofit work

4. VERIFICATION OF SEISMIC RESISTANCE

The main bank of Sagamihara Sedimentation Basin is an earth-fill dam structure. Therefore, the seismic resistance of the main bank was evaluated by referring to the standards for dams.

The basin is an important facility for lowering the turbidity of water as well as storing water for emergencies. Therefore, the stability of the main bank against Level 1 earthquake motions was analyzed by using the modified seismic coefficient method, which is a strict method for evaluating seismic resistance. The standards to conform to were those of the “Draft of Guidelines for Seismic Design of Embankment Dams” (1991, Japan Institute of Country-ology and Engineering), which refers to the inspection of seismic resistance of dams by using the modified seismic coefficient method.

According to the Guidelines (2009), the safety against Level 2 earthquake motions was analyzed by following the “Guidelines for Seismic Performance Evaluation of Dams During Large Earthquakes (Draft) and Explanation” (2005, Ministry of Land, Infrastructure, Transport and Tourism). An overview and the results of the evaluation are shown in Table 1.

Table 1 Overview of seismic diagnosis of the main bank and results

Item	Level 1 earthquake motion*		Level 2 earthquake motion**	
Inspection guideline	Draft of Guidelines for Seismic Design of Embankment Dam		Guideline for the Seismic Performance Evaluation of Dams against Large Earthquakes (draft)	
Inspection standards	Safety factor of the slip circle		Residual settlement, elevation difference	
Analytical standards	Fs≧１．２		No overflow, secured water storage function (settlement not exceeding 1.0m)	
Method of analysis	Slip circle method by the modified seismic coefficient method		Dynamic analysis by the equivalent linearization method, etc.	
Analytical model	Combined model of the dam and ground		Two-dimensional finite element model	
Target water level	Normal water level		Normal water level	
Inspection result	Upstream slip plane	Downstream slip plane	Upstream slip plane	Downstream slip plane
	✗	○	○	○

***Level 1 earthquake motion**…Out of the earthquakes that are assumed to happen in the area where the applicable facility is located, the earthquakes with the highest probability of occurring during the facility's in-service period.
****Level 2 earthquake motion…**The largest magnitude earthquake that is assumed to happen in the area where the applicable facility is located.

The seismic diagnosis showed that the main bank did not have the required seismic resistance against Level 1 earthquake motion at the slip surface on the upstream face that passes through the downstream slope (Fig. 4.1).

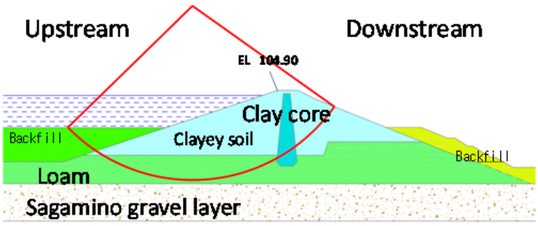


Fig. 4.1 Representative slip surface on the upstream face

5. INVESTIGATION OF RETROFITTING METHODS

The constraints on construction were organized based on the site conditions and survey results as described below in (1); methods for retrofitting the upstream and downstream side of the main bank were selected.

(1) Constraints on construction

①*Water supply control and management*. The raw water supplied from Lake Sagami accounted for at least 20% of the total water supply of Yokohama City. Therefore, it was difficult to cut off the water supply from Lake Sagami during the retrofit work.

②*Water quality*. It was possible to send the raw water directly from Lake Sagami to Nishiya water purification plant so as to bypass the basin by using only the bypass pipeline shown in Fig. 2.3. However, there are no sedimentation facility between Nishiya Water Purification Plant and Sagamihara Sedimentation basin. This would lower the performance of water treatment in the event that the turbidity of the raw water increased, for instance during a typhoon or a storm. Therefore, water could not be sent solely through the bypass pipeline over a long period of time.

③*Ground conditions, etc*. When the basin was completed approximately 60 years ago, cracks developed on the bottom surface of the basin. When the bottom surface was dried cracks were highly likely to develop again and cause water leakage. Furthermore, construction of temporary structures that were needed for the retrofit work had risks of inducing cracks and water leakage.

(2) Plans of retrofitting the upstream face

A common method for retrofitting a slip surface on the upstream face is to retrofit the edge of the slip surface. Because of the constraints on water supply control and management mentioned in ①, raw water transmission could not be halted. Therefore the three methods shown in Table 2 were investigated for retrofitting the upstream side.

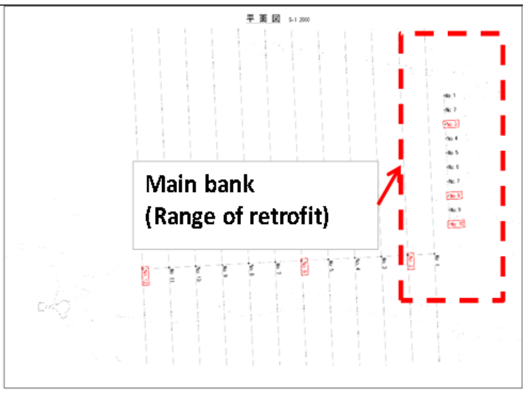
Plan A involves emptying the basin and retrofitting the upstream side. As described above regarding constraints on ground conditions (③), the bottom surface of the basin was suspected to develop cracks and leak water. To empty the basin, it was necessary to solely use the bypass pipeline, but water could not be sent solely through the bypass pipeline over a long period of time as described in the constraints on water quality (②). Therefore, Plan A was rejected.

Plan B involves closing the area to retrofit by installing a temporary structure such as steel sheet piles and retrofitting the upstream side. However, as mentioned in the constraints on ground conditions (③), installation of a temporary structure was suspected to cause cracks on the bottom surface of the basin. Therefore, Plan B was rejected.

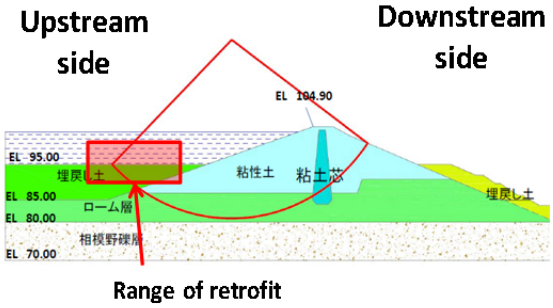
Plan C involves separating the area to retrofit by installing an underwater curtain and retrofitting the upstream side. However, insertion of the retrofitting materials was suspected to stir up sediments on the bottom of the basin and deteriorate the quality of water. Because the plan does not meet the constraints on water quality (②), Plan C was rejected.

It was thus judged very difficult to retrofit the upstream face.

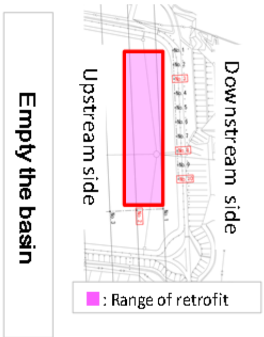
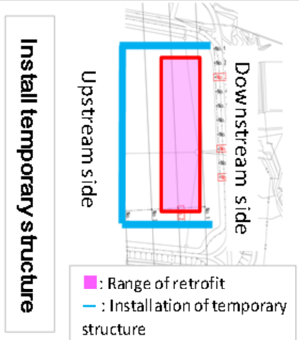
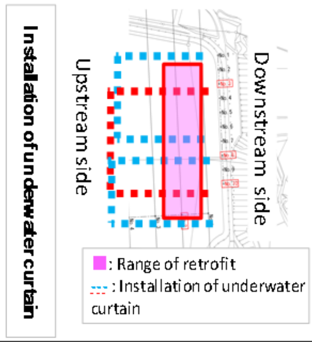
Table 2 Plans of retrofitting the upstream face



Plan of the range of retrofit



Cross section of the range of retrofit

	Plan A	Plan B	Plan C
Retrofit plan			
Overview	The basin is emptied by using the bypass pipe. The embankment is then retrofitted by placing counterweight fill on the edge of the upstream slip plane.	The section to be retrofitted is separated from the basin with a temporary structure and emptied. The embankment is retrofitted by counterweight fill, etc.	The range to be retrofitted is separated with underwater curtain. The bank is retrofitted by counterweight fill, riprap work, etc. in the separated range.
Retrofitting method	・ Counterweight fill ・ Riprap work	・ Counterweight fill ・ Riprap work	・ Counterweight fill ・ Riprap work
Over-all judgment	✘	✘	✘
	・ Emptying the basin may cause cracks to develop on the bottom surface . ・ The bypass cannot be used for a long period of time.	・ Installing the temporary structure may cause cracks to develop on the bottom surface, creating a water leakage hazard. ・ Emptying the zone inside the temporary structure may cause cracks to develop on the bottom surface and result in water leakage.	・ Sediments flung up during the works may deteriorate the quality of water.

(3) Plans of retrofitting the downstream face

It was judged difficult to retrofit the slip plane (Fig. 4.1) that did not have the required safety factor on the upstream side (Plans A, B, and C).

Therefore, a method was investigated that involved increasing the resistance of the slip surface on the downstream face to ensure the required safety factor (Fig. 5.1).

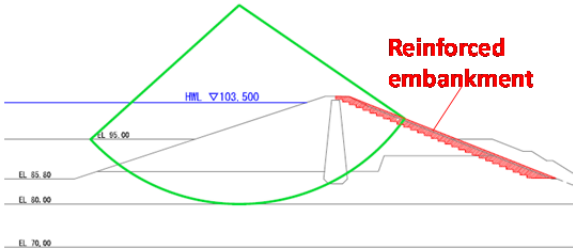


Fig. 5.1 Schematic sectional view of the retrofit plan

This method improves the downstream slope where the slip surface passes through, increases the cohesion and thus enhances the resistance along the slip surface.

The downstream slope was decided to be improved through reinforced embankment by replacement. The method involves replacing the surface soil of the downstream slope with reinforced embankment. The uniaxial compressive strength of the reinforced embankment was increased by mixing cement to the excavated surface soil. The method has the following advantages among others:

* Work is performed only on the downstream side of the main bank outside the basin; and thus there are no limitations on water transmission method, such as having to solely use the bypass pipe, and no deterioration of water quality.
* The depth of improvement is 1 to 2m from the dam surface. Installation of temporary structures is not required and the materials of the existing bank can be used. Therefore, the method is economical.

As described, the reinforced embankment by replacement was judged to satisfy the constraints and to be economical and was thus adopted.

6. RETROFITTING WORKS

(1) Overview of the works

The working area was approximately 10,000m^2 on the downstream slope of the main bank of Sagamihara Sedimentation Basin. The uniaxial compressive strength

Counterweight fill	14,759m^3
Reinforced embankment	16,759m^3
Amount of cement added (Total)	3,965t
Area to be retrofitted	10,337m^2 (235m×23～52m)

Table 3 Major construction quantities of the retrofit work

levels required from the reinforced embankment in the three sections shown in Figs. 6.1 and 6.2 were set at 1,060kN/m^2, 920 kN/m^2 and 1,120 kN/m^2.

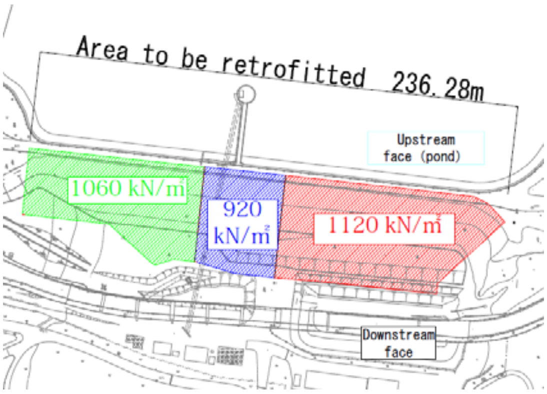


Fig. 6.1 Plan of the range of the bank to be retrofitted

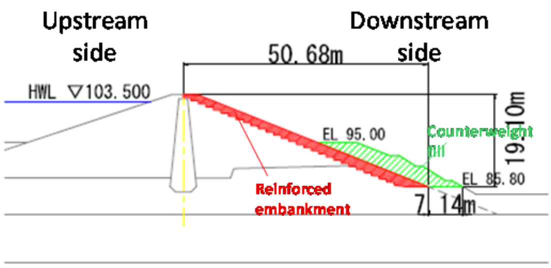


Fig. 6.2 Representative sectional view of the retrofit work

(2) Preparation of reinforced soil for embankment

The mixture proportions of the reinforced embankment were decided based on the soil test results of the soil samples taken at the site. Cement can be mixed into soil either by using a backhoe to improve subgrade soil or by using a plant. The amount of cement that was to be added in this project was approximately 4 times the amount used for improving the subgrade soil in an ordinary road improvement project. Because the soil of the main bank was clayey, it was difficult for a backhoe to stir and uniformly mix the soil and cement. Moreover, the site was adjacent to a residential area, park, etc., and thus dust needed to be minimized as much as possible. Upon considering these conditions, a plant shown in Fig. 6.3 was assembled at the site.

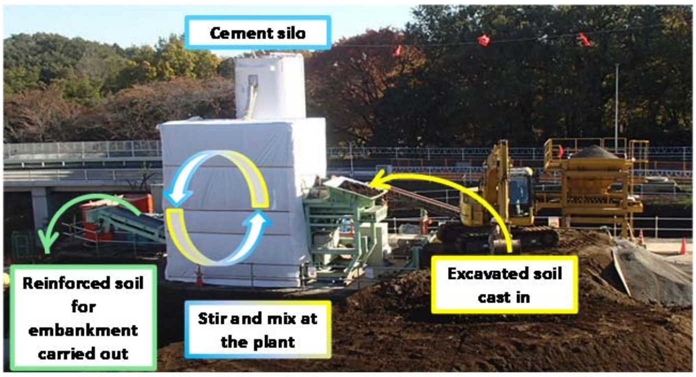


Fig. 6.3 Cement mixing plant

(3) Workflow

The work involved removing the counterweight fill from the area to be retrofitted (Fig. 6.4 and the green section in Fig. 6.2) and excavating part of the bank within the range (red section in Fig. 6.2). To increase the stability of the main bank, the main bank was bench cut as shown in Fig. 6.5, and then chipping was performed.

The excavated soil was stirred and mixed with cement and prepared into reinforced soil for embankment. The reinforced soil and counterweight fill were banked on the main bank, and the slope was formed. The workflow is shown in Fig. 6.6.

Fig. 6.4 Excavating the bank

Fig. 6.5 Bench cutting

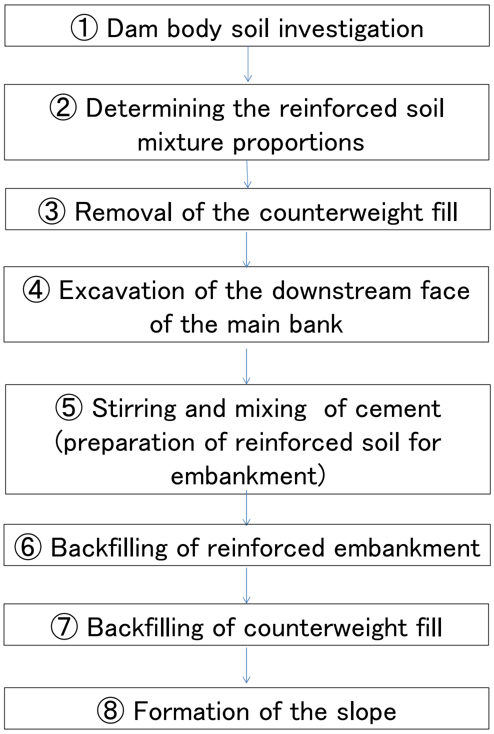


Fig. 6.6 Flow of the works

(4) Work control

① *Quality control of reinforced embankment*. The weight of the cement to be added was always monitored at the control room of the plant to ensure that the added material was consistent with the mixture design. A specimen was sampled for each 500m^3, and was subjected to an unconfined compression test.

② *Checking rolling and compaction*. The reinforced soil for embankment was banked on the main bank where the soil was excavated. Soil cement cannot manifest the target strength unless it is sufficiently compacted [1)]. Therefore, the soil cement was roll-compacted during backfilling at every 30cm in depth to ensure sufficient compaction. After completion of the backfill, the degree of compaction was properly checked by the RI method.

③ *Controlling the time from preparation of reinforced soil for embankment until backfilling*. Prepared reinforced soil for embankment starts hardening soon after preparation [1]. Therefore, the construction quantity per hour was adjusted so that the prepared reinforced soil was backfilled and roll-compacted within 6 hours of the preparation.

7. CONCLUSION

The retrofit work of the main bank of Sagamihara Sedimentation Basin is scheduled to be complete in March 2016. Holding up the goal of constructing an earthquake-resistant and reliable lifeline in the 10-year long-term vision formulated in 2006, Yokohama Water Works Bureau has promoted seismic retrofitting of its facilities.

This project is one of seismic retrofitting projects of its water conveyance facilities. The Bureau will continue this and other projects aiming at 100% earthquake-resistant facilities.

References

1) Fukutani, W. and T. Sakakibara: Study on the use of mortar mixed backfill soil for sewer installation, Technical Note of National Institute for Land and Infrastructure Management, No. 531, 2009.