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EXPERIMENTAL STUDY ON SIPHON DREDGING SYSTEM AT WONOGIRI MULTIPURPOSE DAM RESERVOIR (1ST REPORT)

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BACKGROUND OF DEVELOPMENT

In Japan, more than 50 years have passed since many dams were constructed, and depending on the dam, sediments exceed the planned amounts. Because dam life can be extended semi-permanently if the sediments which accumulate year by year are dredged, the development of sediment dredging/removal techniques has become important in recent years. At present, the main dredging/removal methods are sand flushing and sand bypass, which employ the energy of flowing water to remove sediments. Because removing by these methods must be implemented in a short period during the flood season, when the sand-carrying capacity of the downstream river is comparatively large, preservation of the downstream river environment becomes an issue. With the sand bypass method, the cost of constructing and maintaining the bypass channel tends to be large. As other methods, excavation/dredging methods by backhoe + pump transport and pump dredging are conceivable, but energy consumption for excavation/dredging and conveying is large, and numerous restrictions limit the quality of soil and dredging locations where these methods can be applied.

In this project, a siphon dredging system which employs the difference in the water level in a dam impoundment and downstream river to excavate/dredge and convey sediments was proposed. This system can be operated continuously, not limited to the flood season, because of comparatively environment-friendly, small energy consumption and controllable the concentration of the discharged sediment. In this siphon dredging system, the difference in water level causes a water flow in the dredging pipe by the siphon principle, and earth and sand are sucked up and transported by this flow.

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This paper reports the results of a dredging demonstration test at the Wonogiri Multipurpose Dam Reservoir in Indonesia, which was performed to confirm the optimum conditions of soil quality, dredging depth, and conveying distance using this principle, as well as differences in performance and operation with pump dredging in tests, and to collect various data for practical application.

WONOGIRI MULTIPURPOSE DAM RESERVOIR

The Wonogiri Multipurpose Dam is a fill-type multipurpose dam (Fig. 1) that was completed in 1982 for flood control, water supply for irrigation, domestic use and power generation. It is the sole large-scale reservoir in the Bengawan Solo river basin, which is the largest river on Java Island in Indonesia. The reservoir has an area of 90km^2. In particular, inflowing sediments from the Kudowan River causes blockage in front of the power generation water intake and obstructs the power generating function.

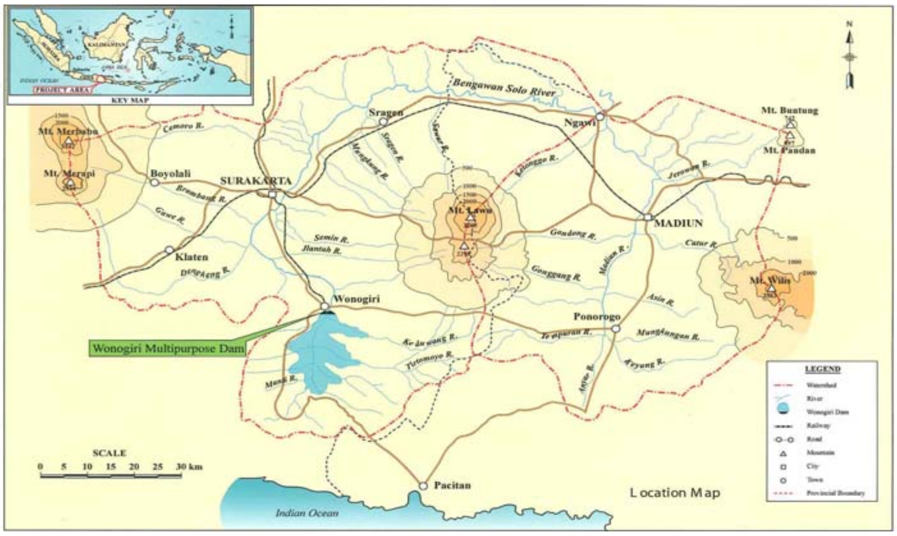

Fig.1 Location map

The sediment consists of clayey soil, silt, sandy clayey silt, and sandy silt. The surface layer (0-1m) has porosity of more than 60%, but at depths greater than 1m, porosity is less than 60%. Porosity decreases with depth, and a tendency to consolidation (porosity: 53%) was found at a depth of 5.5m

DREDGING SYSTEM

The test dredging system comprises intake, conveying, receiving, and return sections. Fig.2 shows the arrangement of the dredging system; Fig. 3 shows a flowchart of the dredging operation.

The receiving tank is installed at a level where the specified difference in water level relative to the reservoir water level can be obtained. Polyethylene pipe is used. The pipe is supported by floats in the reservoir and laid along the spillway passing over the crest of the floodgate when the gate is opened during the test period.

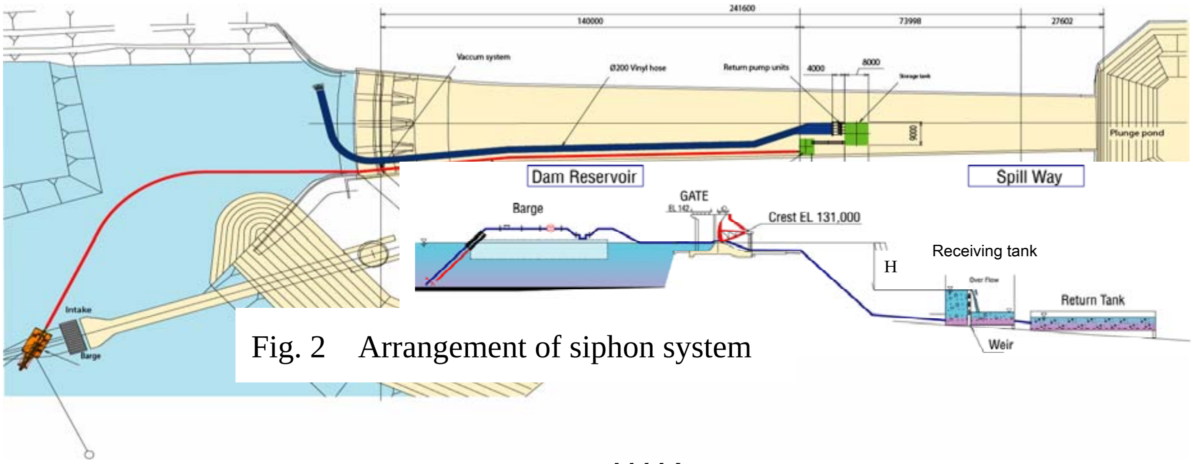

Fig. 2　Arrangement of siphon system

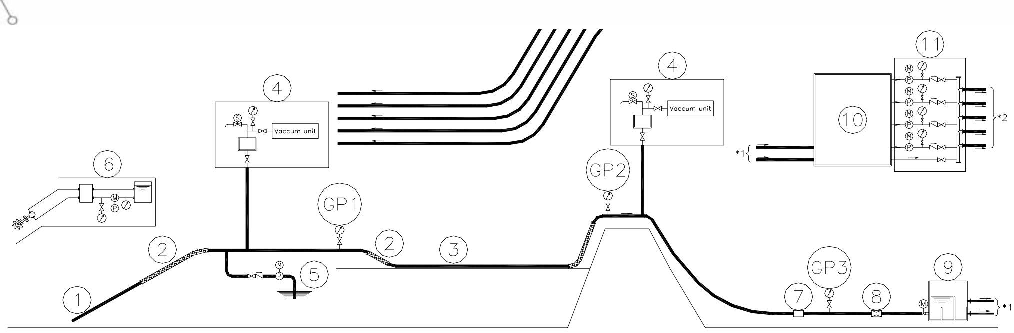

Fig. 3　Flowchart

① Suction pipe	④ Vacuum unit	⑦ Densitometer	⑩ Return tank
② Flexible tube	⑤ Water pump unit	⑧ Flowmeter	⑪ Return pump units

Intake section: An intake pipe with an inner diameter of 400mm is installed on a steel barge, as shown in the barge in Photo 1. A side rotary-type excavator is mounted on the end of the intake pipe. The excavator has a structure in which rotor blades on its two sides cut and break up trash and consolidated soil, which are then sucked into holes on the two sides.

Photo 1　Barge

Conveying section: The dredging pipe has an inner diameter of 400mm. A high density polyethylene pipe is used in the lake section. Steel pipes are used in the siphon at the floodgate and in part of the line which discharges into the receiving tank. The line reaches the area downstream of the dam by passing over the top of one opened floodgate.

Receiving section: A receiving tank (4mL x 5mW x 4mH) is installed on the pipe route and serves as a dam, making it possible to stabilize the difference in water levels.

METHOD AND CONDITIONS

Tests were performed by varying the flow rate by adjusting the opening of the start/stop valve at the terminus of the line, varying the dredging depth in the range of 1-4m, and varying the conveying distance by changing the length of the lake section of the line. Dredging was performed by artificially adjusting the distance between the intake pipe and the lake bottom so that sediment would not accumulate in the conveying pipe. In the dredging tests, effective dredging was possible after various dredging tests were carried out to eliminate the influence of silting in the pipeline.

As measurement items, flow rate and density were measured by an electromagnetic flowmeter and γ-ray densitometer installed in the line near the receiving tank, as shown in the flowchart in Fig. 3. Pressure in the pipe was measured at three points, namely, on the barge, at the siphon at the crest of the floodgate, and before the receiving tank. Power consumption limited to the excavator installed on the intake pipe and total power consumption by the winch, vacuum devices, and pipeline filling pump were measured with two watt-hour meters, respectively. The amount of dredged sediment was calculated by surveying the lake-bottom topography before/after the test.

RESULTS AND DISCUSSION

Confirmation of Operation

Work efficiency and operability did not differ greatly from that in pump dredging. Because the siphon system is not power-driven, the working environment was good, with no mechanical vibration or noise.

① The time required for initialization until a siphon formed was approximately 20 minutes. Starting and stopping were performed by opening and closing the start/stop valve, and could be performed easily with no vibration of the piping or other parts.

② Although depending on the condition at the suction holes, operating performance was stable. In particular, there were no problems at the siphon section passing over the gate crest, even when instantaneous blockage occurred.

③ The practical advantage of using lightweight, high density polyethylene pipe in the pipeline was confirmed, in that silting in the line can be checked easily by visual observation of floating/sinking of the pipe, which is sensitive to changing conditions.

Pressure Loss in Pipeline

The pressure loss in the piping system was obtained from the measured values of PG1, PG2, and PG3 shown in Fig. 3 by varying the flow rate by adjusting the valve opening during dredging. The pressure loss coefficient was obtained from the pressure loss during water conveying using the Darcy-Weisbach equation.

$$\lambda = \frac{H \times D}{\frac{v^2}{2g} \times L} \qquad [1]$$

where, H :Head loss due to friction in pipe (m)

λ :Pressure loss coefficient with clear water

L :Length of pipe route (m)、 D :Diameter of pipe (m)

Pressure loss in the pipe is shown in Fig. 4.

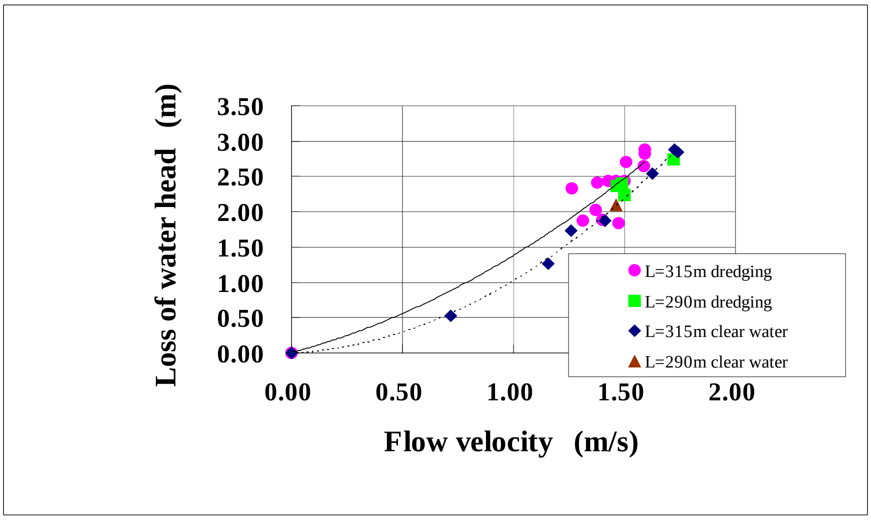

Fig. 4 Pipeline head loss

The average value of the pressure loss coefficient was 0.0258. Although the pressure loss coefficient of the high density polyethylene pipe was expected to be 0.018, the actual value was larger than planned. As the reason for this, the flange connections in the high density polyethylene pipes are concave, and joints between pipes are not smooth, but rather, have protruding parts due to bonding. As this test system included 6 flange connections and 11 bonded joints, it would appear that these parts affected the pressure loss coefficient.

The pressure loss coefficient during dredging was calculated based on a simplified equation proposed by Hasegawa, Yagi, and Tokunaga (Port and Airport Research Institute, 1958). As regards the frictional resistance of the pipe when conveying mud, the frictional resistance of the pipe when conveying water is proportional to the mud content of sediment-bearing water, and the increment of resistance is thought to differ depending on the soil.

$$H = \alpha \cdot \lambda \frac{L}{D} \cdot \frac{v^2}{2g} \qquad [2]$$

$$\alpha = 1 + \beta(\gamma - 1) \qquad [3]$$

where, α :Coefficient of increment of pipe friction during dredging

β :Soil coefficient (shown in Table 1) ,

γ :Density of muddy water (measured value)

Table 1 Soil coefficient

Soil property	β
Clay・ Silt	2
Fine sand・ Normal sand	3
Coarse sand・ Gravel mixed sand	4
Gravel	5

Table 2 Soil coefficient (Average)

Dredging depth		0～1	1～2	2～3
Soil	Clay	1.5	-	-
	Sandy silt	-	2.5	4.0

The coefficients of the increment of pipe friction during dredging and soil coefficients (shown in Table 1), were obtained from the average pressure loss coefficient during water conveying and pressure loss during dredging. The results are shown in Table 2.

Large differences can be seen in the soil coefficient, depending not only on the type of soil but also on the dredging depth (0-2m, 2-3m).

・ At a dredging depth of 0-1m, the soil is clay and the average soil coefficient was 1.5;

however, at 1-2m, the soil is sandy silt, and the average coefficient was 2.5. The test value in the case of clay is small in comparison with the value indicated by Hasegawa et al. Because the sandy silt at Wonogiri consists of approximately 50% fine sand, its coefficient is assumed to be an intermediate value between “clay silt” and “fine sand normal sand.” Based on this, the 2.5 test value for sandy silt is considered to agree with the value proposed by Hasegawa et al. Accordingly, the measured values of the soil coefficient in the dredging depth range of 1-2m are considered to be in rough agreement with the soil coefficient proposed by Hasegawa et al.

On the other hand, although the soil at dredging depths of 2-3m is sandy soil, the average soil coefficient was 4.0. The sandy silt at Wonogiri is considered to have a coefficient on the order of 2.5. Thus, a large difference could be seen in the soil coefficient. The following factors are thought have influenced the soil coefficient:

Factor 1: Intake pipe vertical head: $(\gamma - 1) \times h$, γ: specific weight during dredging (kg/m^3), h: dredging depth (m).

Factor 2: Increment of pressure loss due to change of angle around intake pipe.

From this, it appears to be necessary to consider not only the soil type, but also the dredging depth in the increment of pressure loss during dredging. Further study is necessary, including collection of data on the relationship between the soil coefficient and dredging depth.

Relationship between Pipe Flow Velocity and Density

The test data on the flow rate and density during dredging are shown in Fig. 5. A decreasing tendency in the flow rate can be seen under the operating conditions indicated by figure ①, ②, ③, and ④. These phenomena shall be caused by the flow decreasing with sediment accumulation. According, in order to prevent the sediment accumulation increasing, the flow of water only was made through the line. At this condition, the dredging water density was 1.03-1.05. This suggests that sediment may begin if dredging is performed under these operating conditions. In other words, these are unstable conditions which indicate the approximately critical velocity in the pipe.

On the other hand, operating condition ⑤ and ⑥ shows a peak density, but because the flow rate is comparatively stable before and after this peak, and furthermore, the density is relatively low in comparison with that under other operating conditions with the same flow rate, it is considered that dredging is being performed without sediment accumulation.

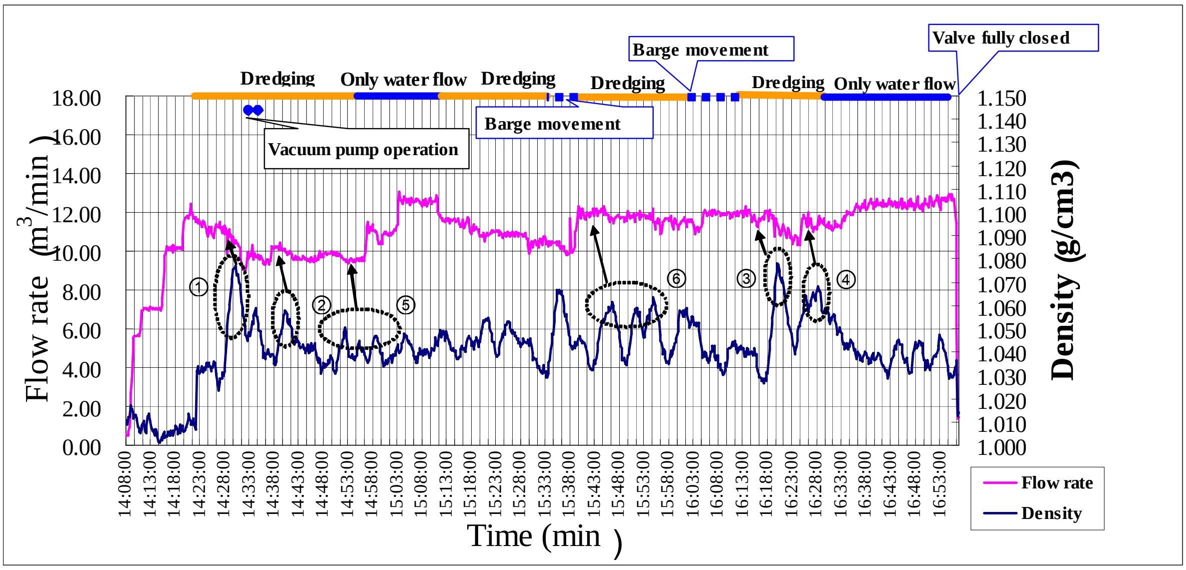

Fig. 5　Relationship between flow rate and density

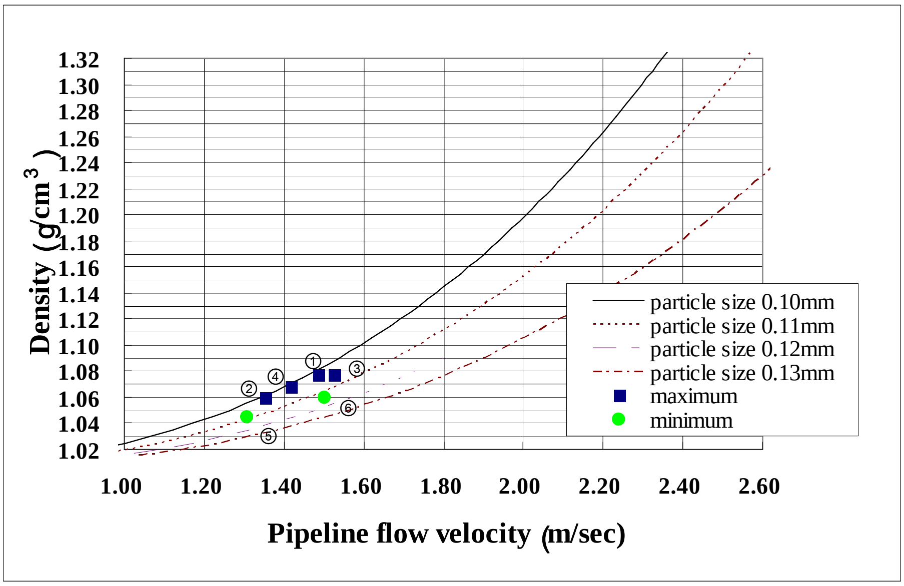

Fig. 6　Pipeline flow velocity and density

Fig. 6 shows a comparison of the above measured data and an experimental equation for the critical flow velocity proposed by Yagi, Okuide, Miyazaki, and Koreishi (Port and Airport Research Institute, 1979). The representative particle size of the mixed sediments adopted here is 0.093mm, which is the average value of the particle size in a soil particle size test by the 60% mass screening method. In the range where the flow velocity is comparatively slow, the pipeline is basically horizontal, excluding an inclination of about 15° in the siphon section. This shows good agreement with the limit flow velocity for a representative grain size of 0.1mm in the experimental equation.

$$V_C = 5 \times C^{1/3} \times D^{1/2} \times \left(4.5 - \frac{1}{(d_S)^{1/2}} \right)^{5/6} \qquad [4]$$

where, Vc : Critical flow velocity (m/s), C : True volumetric density
D : Pipe diameter , ds : Representative diameter of mixed sediments (mm)

Energy Consumption in Dredging

With this demonstration test system, dredging was performed at a maximum flow velocity of 1.7m/sec while adjusting the intake pipe operation to approximately 50% of the rated speed of the rotary excavator on the intake end. The maximum dredging depth was 4.0m. (Photo 2)

Photo 2 Accumulated sediments in return tank

Table 3 shows the dredging rate per hour and the dredging rate per unit of power consumption, as obtained from total sediments dredged with the rotary excavator, dredging time, and dredging equipment power consumption. As reference, the table also shows the specifications of a small-scale pump dredging barge (E200PS) with roughly the same capacity as this test system.

Table 3 Test data for side rotary method

Type of sedimental excavator	Flow rate	Volumetric concentration (Average)	Dredging volume	Operation time	Dredging volume per hour	Electricity consumption	Dredging rate per unit of
	(m^3/min)	(%)	(m^3)	(h)	(m^3/h)	(kwh)	(m^3/kwh)
Side rotary method	9～12	6	122.1	4.06	30.1	15	8.14
Hydraulic dredge	15.4m^3/min× 31m×147kw	10	92.0	1	92.0	147	0.63

The dredging rate per unit of power consumption is 8.14m^3/kWh, or more than 12 times higher than that in pump dredging (0.625m3/kWh; pump dredging power = pump drive power x 0.8). The siphon dredging system is clearly an extremely effective energy saving system in comparison with the pump dredging barge and other power dredging methods. In this test, the volumetric concentration was limited to about 6%, but this can be increased by increasing the flow velocity.

Trash Passing Performance

Photo 3 shows examples of the trash found in the receiving tank and return tank in this test. The kinds of trash and their maximum dimensions are shown in Table 4. Stalks and wood debris with lengths exceeding the pipe diameter passed through the line. Because there are no obstacles to the passage of trash through the line in the siphon system, virtually no blockage occurs in the system if trash enters the intake hole. Thus, the system has excellent trash passing performance.

Table 4 Types of trash

Type	Dimensions
Pebbles	Maximum diameter: 130mm
Bamboo, stalks	Maximum length: 600mm x width: 50mm
Vinyl	Scraps; approx. 150mm x 150mm

Photo 3 Discharged pebbles, trash, and bamboo

CONCLUSION

A siphon dredging system which excavates/dredges and conveys sediments using the difference in the water level of a dam reservoir and the downstream river was proposed. Operation of this system is continuously and is not limited to the flood season. It is also considered to be a comparatively environment-friendly system, as energy consumption is low and it is possible to control the concentration of discharged sediments. This report describes a demonstration test and evaluation of the system at the Wonogiri Multipurpose Dam Reservoir.

① Workability and operability are similar to those in pump dredging, but because the siphon system is not power-driven, mechanical noise and vibration are minimal. Thus, this is an energy saving, environment-friendly system.

② It is possible to adjust the concentration of discharged sediments by adjusting the opening of the start-stop valve.

③ The increment of pressure loss in the pipeline during dredging shows comparatively good agreement with a simplified equation proposed by Hasegawa et al. However, in order to improve accuracy, it is necessary to consider the dredging depth (soil consolidation).

④ In the range where the flow velocity in the pipeline is comparatively slow (1m-1.7m/sec) and the range where the total conveying distance is short (315m), the relationship between the sediment particle size, critical flow velocity, and volumetric concentration show good agreement with an experimental equation proposed by Yagi et al.

In the future, while continuing to accumulate actual results, the authors hope to clarify the conditions for maximizing the effectiveness of the siphon system by further tests of the sediment particle size, dredging depth, and conveying distance.

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