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Desing and Structural Behavior of Nagawado Dam

Le calcul et le comportement structural de Barrage de Nagawado

Entwurf und Gefügeverhalten von Nagawado Talsperre

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1. Preface

Nagawado Dam, located on the Azusa River - the name given to the upper reaches of the Shinano River which flows through the central part of Japan into the Japan Sea - was completed in 1969. It is an arch dam, 155 m high above the bedrock and built from about $66x10^4 m^3$ of concrete.

Three arch dams were constructed across the Azusa River by Tokyo Electric Power Company, Inc. in order to generate 900 MW of electric power, including pumped storage generation, to meet the load demand of the Tokyo metropolitan area.

Nagawado Dam is the main structure of the scheme and is the uppermost of three dams, creating a reservoir with a total storage capacity of $123x10^6 m^3$. This reservoir stores and regulates the flow of river water and also serves as the upper reservoir of the pumped storage power plant.

This paper is to observe and compare the results of theoretical analyses - both trial load and finite element methods were applied -, of model tests using plaster models and of observations of the actual dam after filling, in order to determine guide lines for future structural design.

2. Stress Analysis

2-1 General Features of dam

Nagawado Dam is a double curvature arch dam and the horizontal cross section is formed by a single-centered circular curve on the upstream face, and a three-centered circular curve on the downstream face, provided with fillets at the abutments.

The main dimensions of Nagawado Dam are as follows:

height: 155 m
crest length: 356 m
crest thickness: 5 m (theoretical), 10 m (actual)

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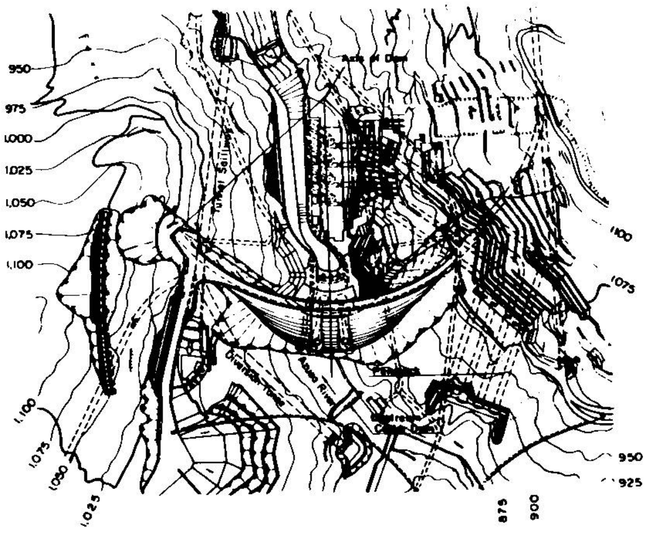
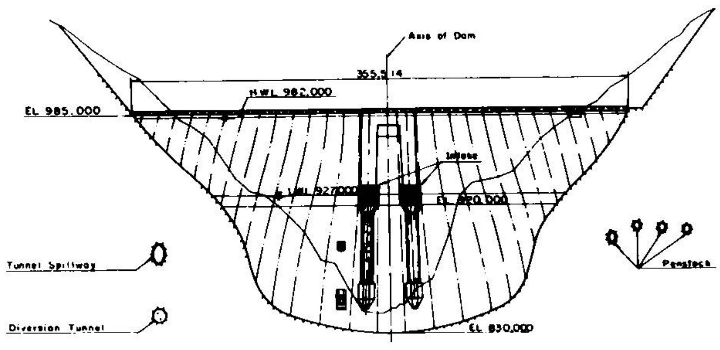
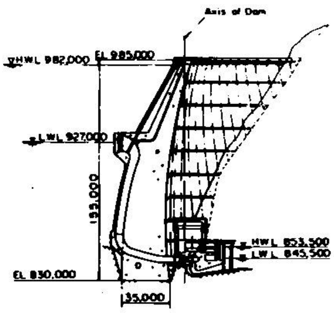


Plan



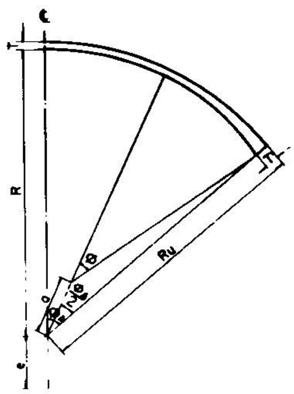
Developed Upstream Elevation



Typical Cross Section

Fig.1 Features of Nagawado Dam

Arch						Fillet			
						Left side		Right side	
EL.	Ru	Tu	T	ϕ_A	e	a	T'	a	T'
985	210.000	0.000	5.000	50°-00'-00"	0.000	42.652	10.000	42.652	10.000
970	190.716	- 8.579	9.240	51°-26'-56"	27.863	35.448	13.566	75.785	22.000
950	162.922	-18.607	14.402	53°-00'-00"	65.685	28.620	18.091	70.672	28.500
930	137.023	-27.023	13.000	54°-06'-39"	100.000	19.006	21.458	54.420	29.970
910	117.678	-33.825	23.033	54°-46'-40"	126.147	6.808	23.852	39.137	30.421
890	108.002	-39.014	26.500	55°-00'-00"	141.012	0.000	26.500	24.928	30.500
870	101.327	-42.591	29.292	55°-00'-00"	151.264	—	29.292	11.704	30.857
850	95.555	-44.555	31.875	49°-30'-00"	159.000	—	31.875	2.775	32.141
830	89.906	-44.906	35.000	00°-00'-00"	165.000	—	35.000	0.000	35.000



Tab.1 Dimensions of Nagawado Dam

base thickness: 35 m
concrete volume: $66\times10^4\ m^3$
high water level: EL.982 m
low water level: EL.927 m
total storage capacity: $123\times10^6\ m^3$
effective storage capacity: $94\times10^6\ m^3$

The foundation rock of the dam consists mainly of biotite granite, and hornfels over it. Dikes of lamprophyre also exist at some places. The foundation rock is generally sound, and the modulus of elasticity (E_R) was assumed to be from 30×10^3 to $100\times10^3\ kg/cm^2$ corresponding to the position from the crest to the bottom of the dam. Because of a gentle slope fault in the right abutment around EL.950 m, an open joint was provided at the upper right side of the dam, so that the arch thrust would not be carried directly to the part above the fault. Therefore, the upper half of the right abutment was made thicker than that of the left part of the dam, so that the stability of the dam and foundation of both banks would be balanced.

Since a group of faults almost parallel with the river course existed in the foundation rock of both banks, foundation treatment work was carried out on a large scale by means of replacing with concrete, grouting the surrounding rocks, and prestressing with steel rods.

General plan, developed upstream elevation, typical cross section, and dimensions of Nagawado Dam are shown in Fig.1 and Tab.1.

2-2 Trial Load Analysis

In analysing by the trial load method, a conventional analysis of an arch dam, five arch elements and eleven cantilever elements were used (see Fig.3); three adjustments of the assymmetrical, three-centered, variable thickness arch dam were carried out by radial displacement, tangential displacement, and rotation about the vertical axis.

The load division to the arch and cantilever elements was not obtained by trial and error method, so called trial load procedure, but by simultaneous linear equations of many unknowns expressing the load distribution.

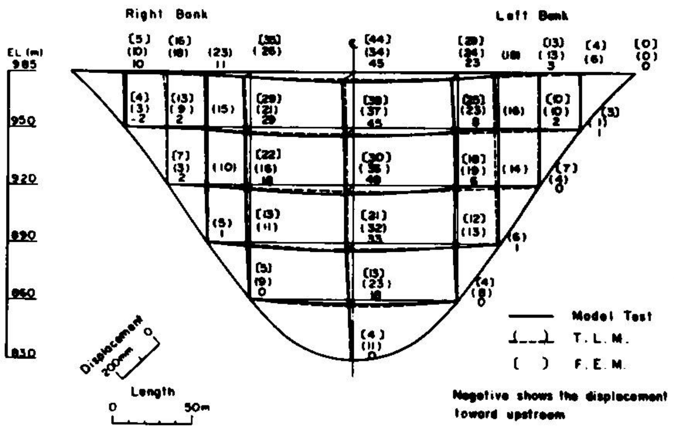


Fig.2 Displacement (Static Load)

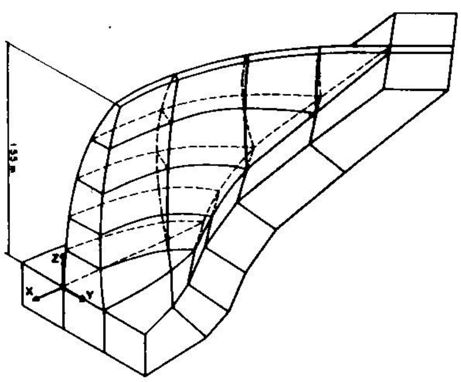


Fig.4 Subdivision of Nagawado Dam by F.E.M.

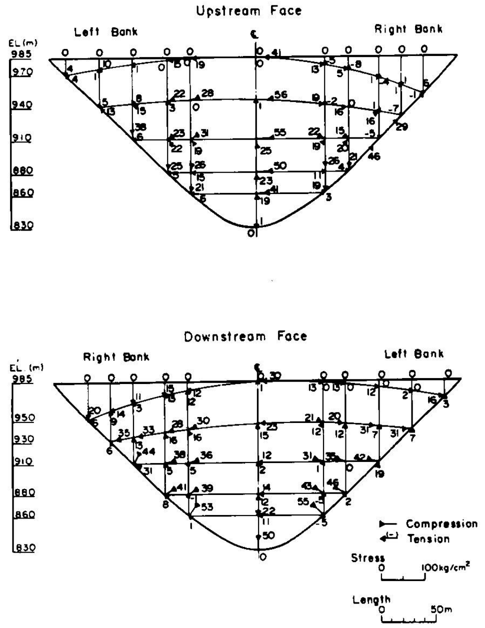


Fig.3 Principal Stress by T.L.M. (Static Load)

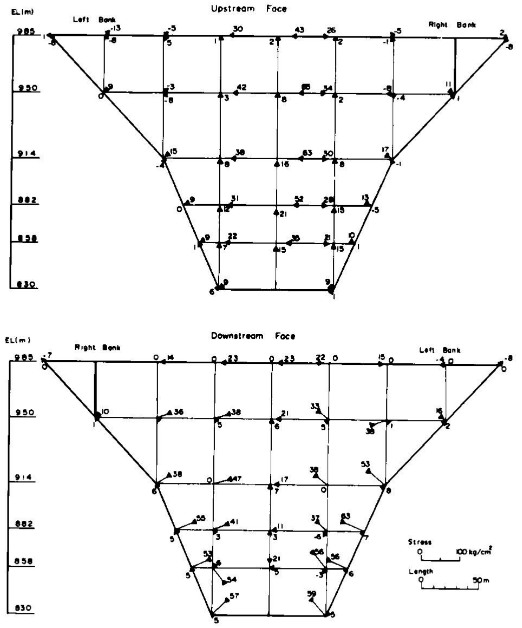


Fig.5 Principal Stress by F.E.M. (Static Load)

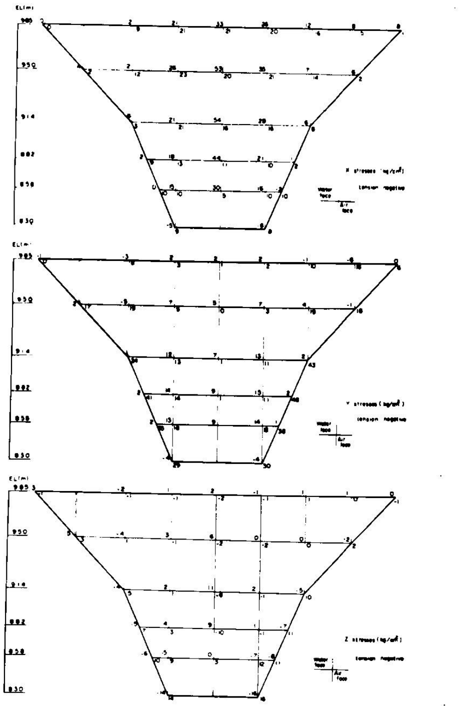


Fig.6 Tri-axial Stress by F.E.M. (Static Load)

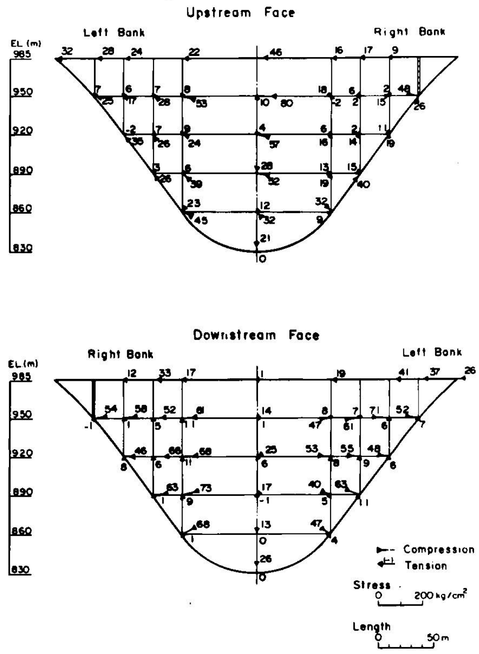


Fig.7 Principal Stress by Model test (Static Load)

The upper arch elements were chosen, taking into consideration the design conditions imposed to avoid carrying arch thrust to the right bank higher than EL.950 m. Radial displacement and principal stress obtained from this analysis, using E_C=250x10^3 kg/cm^2, γ=0.2, are shown in Fig 2,3.

2-3 Finite Element Analysis

In the finite element analysis, two types of isoparametric curved elements were used, a hexahedron and a pentahedron. The number of the elements were 65 - 26 in the dam, 39 in the foundation - as shown in Fig 4.

Radial displacement and principal stress obtained from this analysis based on the above-mentioned design conditions are shown in Fig.2,5.

2-4 Model Test

Model tests using plaster models, scale 1/300, were carried out. The model was made from a mixture of plaster of Paris, diatom earth, and water. The proportion of plaster of Paris, diatom earth, and water was varied to meet the variation of elasticity modulus of foundation rock and the ratio to that of dam concrete and the elasticity modulus of the model was brought to represent one fifth of that in prototype in accordance with laws of similitude.

Wire-strain gauges were used for the strain measurement. Dial-gauges and differential transformer type displacemeter were used for the displacement measurement.

During the design stage of dam, model tests were carried out on eleven models of six shapes. Radial displacement and principal stress obtained from the model tests are shown in Fig.2,7.

2-5 Comparison

Fig.3,5 and 7 indicate the distribution of the principal stresses on the upstream and downstream faces of the dam obtained from the three analysis methods above-mentioned. Comparing these results, the following may be noted:

(1) The maximum principal stress on the upstream face appears at the crown around midheight approx. EL.950 m, and the direction is nearly horizontal. The maximum values are 56 kg/cm^2 in the trial load analysis, 65 kg/cm^2 in the finite element analysis and 80 kg/cm^2 in the model tests.

(2) The maximum principal stress on the downstream face appears at the abutment of the lower part of the dam and the direction is nearly at right angles to the abutment. The maximum values are 55 kg/cm^2 in the trial load analysis, 63 kg/cm^2 in the finite element analysis and 73 kg/cm^2 in the model test

(3) As described above, the maximum stress in the model test is a little larger than that of theoretical analysis. But, as a whole, the results of all three methods coincide relatively well and none of the methods shows a specific tendency to offer greater value than another. This can be seen in Fig.8 which shows the maximum principal stresses at crown and abutment.

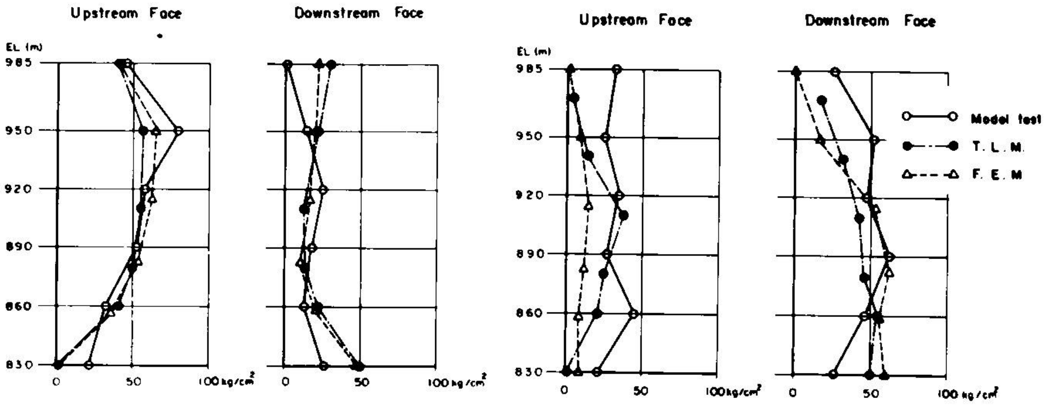
Tab.2 indicates the comparison of tri-axial stresses at the crown by these three methods, and Fig.6 indicates tri-axial stress by F.E.M..

(kg/cm²)

		trial load method		** finite element method		model test	
		up-stream	down-stream	up-stream	down-stream	up-stream	down-stream
EL.985	X	41	30	34	20	46	1
	Y	0	0	2	0	0	0
	Z	0	0	3	-1	0	0
EL.950	X	56	23	53	20	80	14
	Y	3	0	5	0	3	0
	Z	1	15	9	-4	10	1
EL.914	X	55	12	54	17	57	25
	Y	6	0	7	1	6	0
	Z	25	2	10	-5	4	6
EL.882	X	50	14	45	11	52	-1
	Y	10	0	9	1	10	0
	Z	23	12	9	-8	28	17
EL.858	X	41	11	30	6	32	0
	Y	13	0	9	-	13	0
	Z	19	22	0	3	12	13
EL.830	X	0	0			0	0
	Y	15	0			15	0
	Z	1	50			21	26

Tab.2 Comparison of tri-axial stresses at the crown by three methods

Note:
X,Y,Z: Stress components (See Fig.4)
** : approximation



(a) Maximum Principal Stress at Crown

(b) Maximum Principal Stress at Abutment (Left Bank)

Fig.8 Comparison of Maximum Principal Stresses

3. Measurement of the Dam Behavior

3-1 Instrumentation

At Nagawado Dam, several measuring facilities were installed for the control of the dam construction work, for the maintenance of the dam after completion and also for analysis of the behavior of the dam and foundation.

The kinds of instruments installed are as follows:

(1) Carlson type instruments (thermometer, strain-meter, etc.)
(2) Water pressure gauges
(3) Plumb lines
(4) Inclinometers
(5) Drainage-discharge-meters
(6) Others

The number of installed instruments totals about 1400. Most of these instruments can be read and recorded remotely and automatically from the control center.

3-2 The Result of Observations

3-2-1 General Descriptions

Filling of Nagawado Dam was started in March, 1969, and the water level reached the designed high water level in August of the same year. After that, as this plant is a pumped storage plant to meet daily peak load, the reservoir was kept around high water level, except during the summer season when the stored water is used for irrigation purposes. In 1973, the water level of the reservoir notably decreased, because of an unusual water shortage in the downstream area. (see Fig.10)

Measurements of the dam after completion have proved that the behavior of the dam and foundation is quite normal and no trouble has been noticed.

It can be said generally that the behavior of an arch dam is indicated typically by the stress and displacement. At Nagawado Dam, therefore, 243 elements of strain-meters and nine sets of plumb line were installed to measure strain and displacement. Stress analysis of the dam by strain measurement is now under investigation, so that the structural behavior of the dam based on the displacement is described in this paper. The arrangement of plumb lines at the crown is shown in Fig 9.

Observed value of the radial and tangential displacements of the dam is shown in Fig.11. The maximum radial displacement, to date, is 61 mm toward downstream.

As the water level has been kept nearly constant, after having reached its maximum, variation of the displacement would be caused mainly by annual temperature variation.

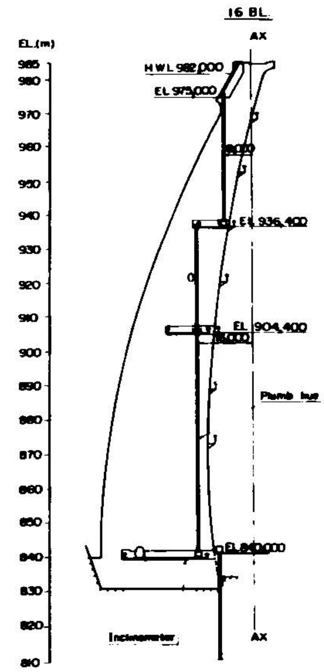


Fig.9 Arrangement of Plumb Lines at Crown

3-2-2 Temperature

Air temperature and water temperature of the reservoir are shown in Fig 10,

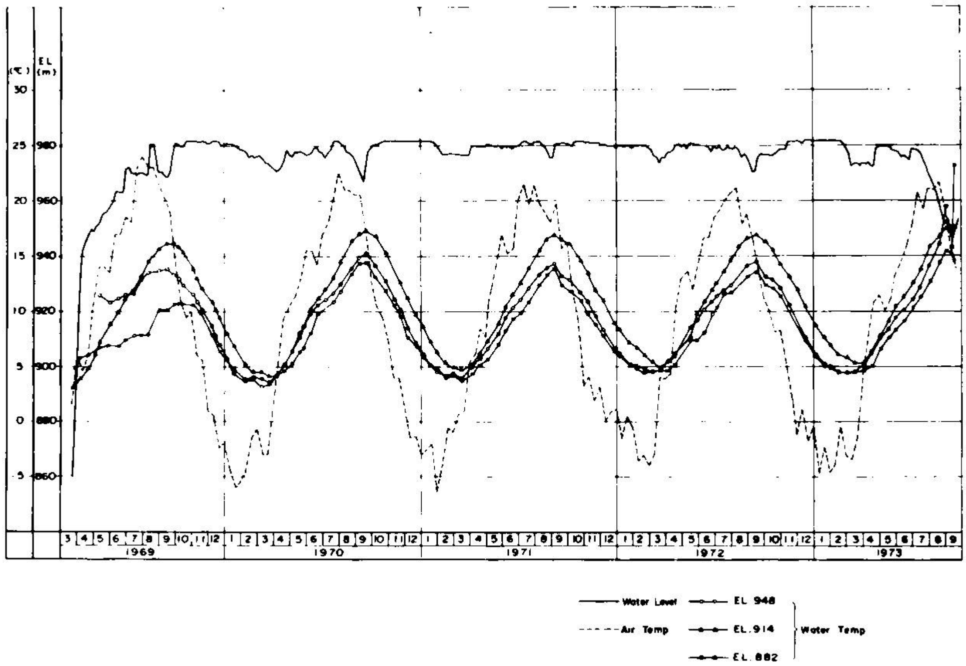


Fig.10 Water Level, Air Temperature and Water Temperature

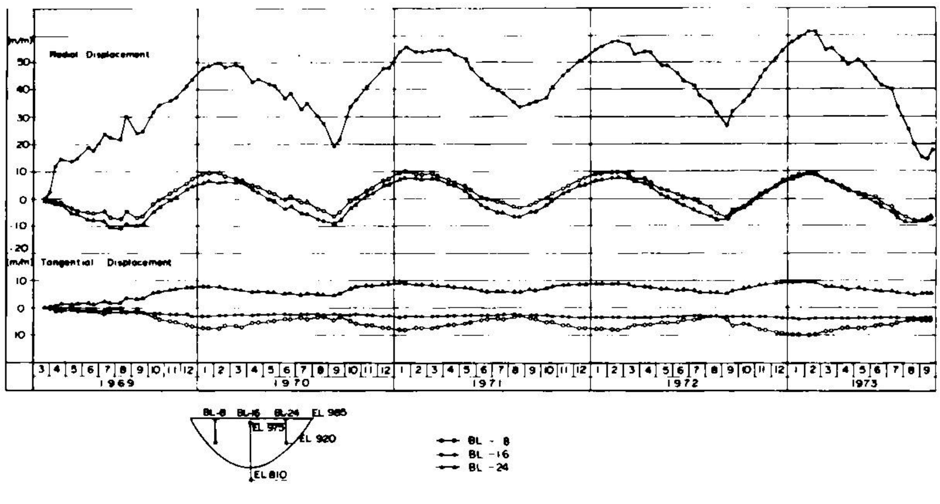


Fig.11 Observed Displacements of the Dam

and can be approximately expressed as follows:

$$T = T_a \sin(\frac{\pi}{6}(\tau - \alpha)) + T_m$$

where T_a: amplitude of annual temperature variation (°C)
T_m: annual mean temperature (°C)
τ : time in days after January 1st
α : phase difference

T_a, T_m, and α of air and water temperature are shown in Tab.3.

	EL(m)	T_a(°C)	T_m(°C)	α (month)
air temperature	---	12.1	8.1	4.3
water temperature	978	9.9	10.6	5.0
	946	5.8	10.6	5.9
	914	5.2	9.0	5.5
	882	4.7	8.4	5.7

Tab.3 Values of T_a, T_m and α

The authors previously investigated the temperature distribution in Sudagai Dam which is a concrete gravity dam with a height of 73 m.

The amplitude of annual temperature variation in a dam decreases as the distance from the concrete face becomes greater. This relation was approximately expressed at Sudagai Dam as:

$$T_a = 124/(x+3)^2$$

where x is the distance from the concrete face (m)
This relation can also be applied to Nagawado Dam. (see Fig.12)

3-2-3 Displacement

As it is assumed that the displacement of the dam is brought about by three factors, i.e. temperature effect, water pressure effect and time effect, the authors attempted factor analysis with respect to the radial displacement at the crown of the crest arch, that is in general one of the most important measurements, using the following expression:

δ : radial displacement $= f_1(t, \theta) + f_2(h) + f_3(\tau) + K$

$f_1(t, \theta)$: temperature effect $= a_1 t_{978} + a_2 t_{946} + a_3 t_{914} + a_4 t_{882} + b_1 \theta_{967} + b_2 \theta_{946} + b_3 \theta_{914} + b_4 \theta_{882}$

$f_2(h)$: water pressure effect $= c_1 h^2 + c_2 h + c_3$

$f_3(\tau)$: time effect $= d_1(1-\exp(-0.1\tau)) + d_2(1-\exp(-0.01\tau)) + d_3(1-\exp(-0.001\tau))$

where

$a_1, a_2, a_3, a_4, b_1, b_2, b_3, b_4, c_1, c_2, c_3, d_1, d_2, d_3, K$

= constants

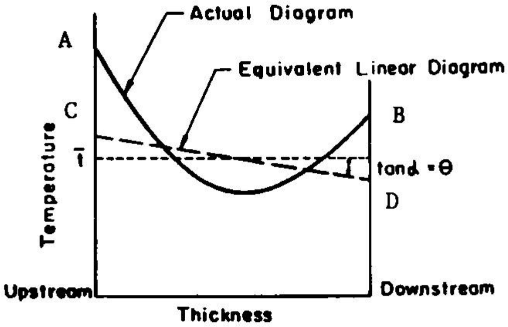
t_i = mean temperature at crown of EL. i m

θ_i = temperature gradient at crown of EL. i m

h = water level of reservoir

τ = time in days after the start of filling reservoir

Mean temperature and temperature gradient at cross section are determined following the conventional procedure shown below.



AB : actual diagram

CD : equivalent linear diagram

AB and CD are equal in area and in geometrical moment of area.

$\bar{t}$: mean temperature

θ : temperature gradient

As a result of calculation by least square method, the following values are obtained as coefficients:

a_1 = -0.79454 a_2 = 1.47466 a_3 = -0.57538

a_4 = -1.17459 b_1 = -0.95341 b_2 = -2.38942

b_3 = 50.77273 b_4 = -11.28073 c_1 = 0.00400

c_2 = -7.09773 c_3 = 3149.8873 d_1 = 3.70546

d_2 = 1 87414 d_3 = 15.54632 K = 3.89437

The calculated values of $f_1(t,\theta)$, $f_2(h)$, and $f_3(\tau)$ using the above coefficients, are plotted in Fig.13.

It can be seen that the variation of displacement due to annual temperature variation shows approximately a sinusoidal curve with an amplitude of 11.5 mm and a period of 1 year. In this case, the phase of the annual displacement variation lags 2 months behind the annual temperature variation.

As for $f_2(h)$, the relation between water level and displacement are shown in Fig.14. According to this figure, the displacement at high water level is 43 mm.

$f_3(\tau)$ shows that the displacement due to time effect gradually approaches the maximum of 21 mm which is expected from the asymptote, in Fig.13. At the present time, four and a half years after starting to fill the reservoir, $f_3(\tau)$ is 18 mm. Therefore, displacement has reached about 90% of the expected terminal value.

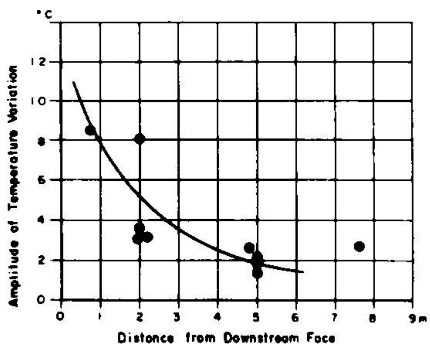


Fig.12 Amplitude of Temperature Variation in the Concrete

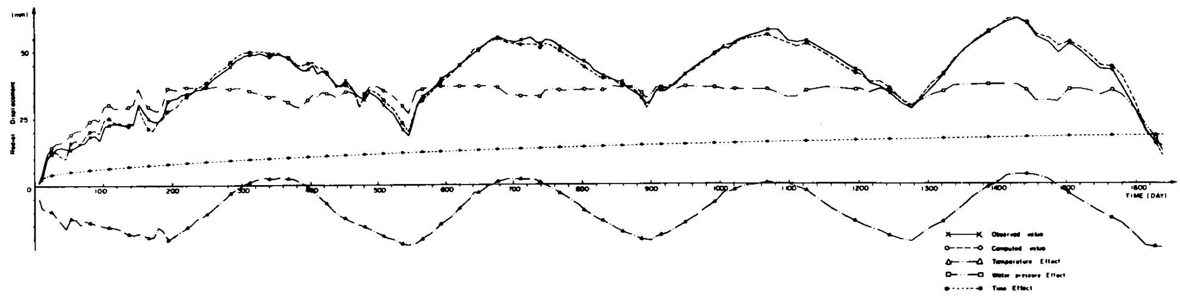


Fig.13 Factor Analysis of the Displacement at Crown

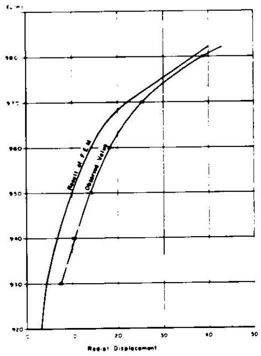


Fig.14 Displacement at Crown due to Water Pressure

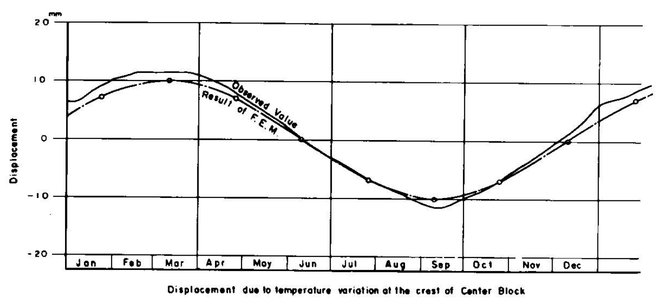


Fig.15 Displacement at Crown due to Temperature Variation

4. Comparison between Observation and Analysis

The radial displacement of the crest crown, due to water pressure (including silt pressure), at high water level, is as follows:

Trial load method	34 mm
Finite element method	44 mm
Model test	45 mm
Observation	42 mm

which well coincide with each other.

As for the displacement due to water pressure variation, the comparison between observed value $f_2(h)$ and computed value by finite element method, is shown in Fig.15. Both likewise coincide well.

As for the displacement due to temperature variation, an amplitude of the annual variation shows 11.5 mm in observation and 10 mm in computation by finite element method.

As mentioned above, the structural behavior of the dam as indicated by $f_1(t,\theta)$ as well as by $f_2(h)$ is quite similar to what was expected from the analysis.

5. Conclusion

During the design stage of Nagawado Dam, the theoretical analyses making use of trial load and finite element methods, and also model tests were carried out for the estimation of the dam stress. The results of these different procedures of analysis have been proved to be well coincident each other.

Moreover, the investigation of the observation of the actual dam has shown that the behavior of the dam is quite similar to expectations at the design stage.

6. Acknowledgements

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7. Reference

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(2) Tokyo Electric Power Co. Inc., Model Study of Arch Dam, 1966.
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Summary

This paper briefly describes the design and the structural behavior of Nagawado Dam, an arch dam 155 meters high, being in service since 1969. The design of the dam was carried out on the basis of theoretical calculation and experimental study including both static and dynamic model tests. Numerous measurements of the dam make it clear that the structural behavior of the dam, after the filling of the reservoir has been similar to expectations at the design stage.

Résumé

Ce rapport donne une explication brève sur le calcul et le comportement structural de Barrage de Nagawado, barrage-voûte de hauteur de 155 m, mis au service de l'année 1969. Le calcul de barrage a été exécuté par le calcul théorique et l'etude expérimental suivant les essais statiques et dynamiques sur modèle réduit. De nombreuses measures sur le barrage ont montré que le comportement structural du barrage après le remplissage du réservoir est approximativement égal à celui qu'on a estimé au stade de calcul.

Zusammenfassung

Dieser Aufsatz beschreibt einen Entwurf und Gefügeverhalten von Nagawado Talsperre, die als 155 meter hoher Bogendam seit 1969 in Betrieb ist. Der Entwurf dieser Talsperre wurde gemacht, auf der Basis der theoretischen Berechnung, und der statischen und auch dynamischen Modellversuchen. Durch die Ergebnisse von vielen Messungen wurde klar gemacht, daß das Gefügeverhalten der Talsperre nach dem Einlaufen des Reservoirs aus dem Entwurf erwarteten Verhalten etwa entspricht.