RESEARCH ON EARTHQUAKE RESISTANCE OF DAMS IN USSR

by

Sh. G. Napetvaridze *

Apart from defining the two main stages in the development of dam earthquake resistance research in the USSR, the present paper throws light on how the chief problems in this field are being solved. These problems are as follows: determination of dam design seismicity; definition of seismic load intensity; experimental research on dam earthquake resistance (field tests of large dam models, three metres in height; tests of comparatively small models, within one metre in height, on seismic platforms; nature measurements of forced vibrations for the dams already erected); translation of basic directions on design and calculation of dams for seismic areas into construction standards.

TWO MAIN STAGES IN DEVELOPMENT OF DAM SEISMIC RESISTANCE

Due to the promosing prospects opened before the hydrotechnical construction in seismic areas of the USSR, the problems of securing earthquake resistance of expensive high dams were given worthwhile concern as far back as forties.

At that time the science of earthquake resistance of structures was of empiric character, based mainly on the experience obtained mainly in the past destructive earthquakes. As to the earthquake resistance theory, it was not developed enough to rely on numerical results of calculations. This, however, did not hinder considerably the earthquake resistant construction of common apartment and industrial buildings, since at that time they would be constructed of monolithic bodies, with limited number of storeys, a lot of information available on this very type of buildings.

Under the above-mentioned conditions, the builders would content themselves with elaboration of constructive decisions reflected in construction standards, while the design for earthquake resistance was considered auxiliary.

---

* Doctor technical sciences, Department chief of Institute of structural mechanics and earthquake resistance under Academy of Sciences, Georgian SSR, Tbilisi

Up till now we don't have enough information available on the behaviour of hydro-technical structures during destructive earthquakes. Hence, calculation was the only way to solve the problem of earthquake resistance of dams. This, however, could not be started, until the fifties, when spectra of vibration of linear oscilator were obtained based on the utilization of accelerograms of destructive earthquakes.

Still in the preceding period the following important questions were solved.

Peculiarities of the dam behaviour during earthquakes were clarified; modes and frequencies of free vibrations in gravity, arch-shaped, counter-force and earth dams were theoretically determined; methods were worked out for designing strength and stability of dams as a whole, as well as of their separate parts (earth and rock-filled dams), proceeding from the assumption that either seismic vibrations of foundation soil are sine or there is a case of hit or push; the problem was investigated of relative earthquake resistance degree for different modes of dams; the initial steps were taken on elaborating the designing standards of dams and other hydro-technical structures for seismic areas.

To-day, with well developed science of earthquake resistance of structures, it became possible to determine the seismic load with due account of actual soil motion character under destructive earthquakes. As to the achievements of computing technique, they helped to obtain the actual, picture of dam stressed condition, in case it has been made of monolithic material (concrete and reinforced concrete), and maximum balance of the structure as a whole or its separate parts (earth or rock-filled dams). All this was possible, on condition that the dynamic designing structure scheme was selected correctly. It is, however, not often that this condition can be easily fulfilled, since any structure, dams in particular, behave in accordance with evident space scheme.

DESIGN SEISMICITY OF DAMS

Under methods adopted in the USSR, prior to designing dams for earthquake resistance, a certain procedure should be conducted of defining the design seismicity of structures expressed in marks in accordance with the 12-mark seismic scale. The thing is that this value is considered dependent on both the seismic construction site and the mode and responsibility of the structure itself.

Seismicity is approximately defined on the seismic-region map of the USSR with further specification by way of dividing the USSR territory into seismic micro-regions.

To obtain the design seismicity, the specified seismicity

is either increased (as a rule in the case of high dams) or decreased (in the case of inconsiderably responsible structures). It can also be left as it is (in the case of common buildings and structures).

Division into seismic micro-region is executed by analysing already available design-prospecting information on mechanical sterngth of structure foundation soil, on nature of soil geological laying and on depth of ground waters.

This approach to definition of the design seismicity is considered quite sufficient while we deal with common buildings and structures. But as soon as we begin to design highly responsible structures, like high dams, a number of additional investigations should be done to define their design seismicity.

Thus, seismic division of the dam construction site into micro-regions requires considerable information on geology and geotectonics, on imminent seismic centres (seismometric data of constant stations and ehgineer-seismological data of visual examination of destructive earthquake results).

In addition to that, the dam construction site within the radius of about 50 km. is completed with temporary seismic stations equipped with high-sensitive devices for recording displacement and acceleration of soils in microseims and slight earthquakes. These records coupled with the already available seismometric data obtained at permament stations are studied with the aim of specifying the seismic condition and the forecast of reveal force as well, as spectral composition in future earthquake for whose load the projected dam is designed.

When forecasting the earthquake strength, the dam term of service is to be taken into account.

This consideration together with the high responsibility of the dam result in diminished seismicity of high dams as compared to that of common buildings erected in the same area, though in worse soil conditions.

INTENSITY OF SEISMIC LOAD

The final aim in securing earthquake resistance of dams is to obtain proper strength conditions (dams of concrete or reinforced concrete) and stability conditions (each dams a whole or separate parts of earth and rock-filled dams).

To design the stability of a dam as a whole for displacement and collapse, it is sometimes sufficient to know maximum lateral force and bending (overturning) moment for the given earthquake, which act in a complete section of the structure. To evaluate durability of material or stability of the dam body separate parts, the seismic load intensity itself should be

given (problem of elasticity theory or maximum balance).

The second mode of design is the most significant for dams. That is why definition of the seismic load intensity was given proper concern due to the following simplifying assumptions.

Seismic vibrations of dams are considered to be elastic. In other words, any presence of considerable residual deformations, particularly cracks, in hydro-technical constructions is not tolerated. But even if cracks appear, as was shown during dynamic tests of models made of reinforced concrete or clay, vibrations are as elastic-nonlinear with slight inflence of non-linearity factor.

Two main types of seismic load are meant here: momentum load corresponding to the dam body weight with the equipment installed on it; hydro-dynamic pressure on the pressure side of the dam from seismic vibrations of dam-reservoir system.

This paper deals with the intensity of the first mode of seismic load which for simplicity will be illustrated as one-dimension problem, with coordinate x and time t .

Let $p(x,t)$ be the intensity of momentum load, while is the same for reactive (elastic) load. In accordance with the dynamic balance condition we get

$$p(x,t)=-S(x,t) \qquad \ldots (1)$$

Hence

$$p(x,t)=m(x)\frac{\partial^2}{\partial t^2}\left[U(x,t)+U_0(t)\right]$$

where: $m(x)$ – distributed mass;
$U(x,t)$ – elastic displacement;
$U_0(t)$ – seismic displacement of structure foundation.

Mode of function depends on that of elastic deformation of the system (bending, displacement or both together).

Thus, for example, when taking into account only bending deformations for a permanent section rod

$$S(x,t)=EJ\frac{\partial^4}{\partial x^4}\left[U(x,t)+\frac{\delta}{\pi\omega}\frac{\partial U(x,t)}{\partial t}\right]\approx EJ\frac{\partial^4}{\partial x^4}U(x,t)$$

considering here and below that value $\frac{\delta}{\pi}$ is small as compared to unit (δ – logarithmic decrement of free vibrations),

E – normal elasticity modulus of material;
J – moment of section inertia;
ω – circular frequency of free vibrations.

Taking into account limited number of n modes of free vibrations (as done in practice) without any intermediate calculations

$$S(x,t)=m(x)\sum_{i=1}^{i=n}\beta_i^*(t)\eta_i(x) \qquad \ldots (2)$$

$$p(x,t)=m(x)\left[\ddot{U}_1(t)-\ddot{U}_0(t)\sum_{i=1}^{i=n}\eta_i(x)+\sum_{i=1}^{i=n}\beta_i^*(t)\eta_i(x)\right. \qquad \ldots (3)$$

Here i - number of free vibrations mode;

$$\eta_i(x)=f_i(x)\frac{\int_0^l m(x)f_i(x)dx}{\int_0^l m(x)f_i^2(x)dx}$$

$$\beta_i^*(t)=-\omega_i\int_0^t e^{-\frac{\delta}{2\pi}\omega_i(t-\xi)}\ddot{U}_0(\xi)\sin\omega_i(t-\xi)d\xi$$

presents acceleration of linear oscillator with circular frequency ω_i and logarithmic decrement δ of free vibrations;

$f_i(x)$ - function of form of the free vibrations;
l - length (height) of structures.

With $n=\infty$ due to conditions of free vibrations orthogonal mode

$$\sum_{i=1}^{i=\infty}\eta_i(x)=1$$

and after taking this into account in (3) we shall obtain equality (1).

Hence, with $n\neq\infty$ or $n\neq N$ in case of system with N of concentrated masses, to define the intensity of seismic load, formula (2) or (3) should not be used as the same. It can be easily senn that, with limited number n of considered vibration modes, formula (3) can be best used to secure the most proper degree of approximation to the actual load intensity in the sense of its value and conformity of distribution over the structure.

In this case, however, some difficulties arise in computation, since the design intensity of seismic load should be defined for the time moments t for vibrations acceleration function $\beta^*(t)$ of both linear accelerator and function $\ddot{U}_0(t)$ presenting the earthquake accelerogram. Hence, the difficulties are connected with the task of elimination of parameter t in formula (3).

The most convenient and well grounded in this respect would be to treat the earthquake as a fortuitous-stochastic process.

In this case

$$S(x)=K_c q(x)\sqrt{\sum_{i=1}^{i=n}\bar{\beta}_i^2\eta_i^2(x)} \qquad \ldots (4)$$

and

$$p(x)=K_c q(x)\sqrt{1-\sum_{i=1}^{i=n}\eta_i^2(x)+\sum_{i=1}^{i=n}\bar{\beta}_i^2\eta_i^2(x)} \qquad \ldots (5)$$

Here:

$q(x)=m(x)g$ - presents distributed proper load,

g - acceleration of gravity force,

$$\bar{\beta}_i^2=\frac{1}{\pi}\int_0^\infty \Phi(\theta)\psi(\theta,\omega_i)\,d\theta$$

is the average square of dynamity coefficient.

On the other hand: $\Phi(\theta)$ - fixed spectral acceleration density, while

$$\psi(\theta,\omega_i)=\frac{\omega_i^2}{\sqrt{(\omega_i^2-\theta^2)^2+\omega_i^2\theta^2\frac{\delta^2}{\pi^2}}}$$

formulas (4) and (5) are generalized for both two and three-dimension problems.

As in the case with curve $\beta(\delta,T)$ constructed for one of the earthquakes in the Caucasus (rock foundation), with different logarithmic decrements, (Fig.1) $\beta(\delta,T)\to 1$ corresponds to a rather rigid structure and reaches its maximum under the free vibration period equal to T = 0.3-0.4 sec. Further $\beta(\delta,T)$ is drawing to a zero with $T\to\infty$. Special attention is centered on the above-mentioned outline nature $\beta(\delta,T)$ when problems of dam earthquake resistance are dealt with.

EXPERIMENTAL RESEARCH ON EARQUAKE RESISTANCE OF DAMS

As it was mentioned above, to calculate the intensity of a seismic load, it is necessary to know frequencies (periods), modes and logarithmic decrement of free vibrations of dams. For all the basic modes of dams (concrete gravity dams, counter-force dams, arch-shaped and earth dams) the first 3-5 types and frequencies of free vibrations have already been defined.

The problem of defining the design value of logarithmic decrement for various types of dams was rather difficult. It

is meanwhile, this very problem that the intensity of the seismic load itself considerably depends on (Fig.1).

The thing is that the value of logarithmic decrement is stipulated by the type of construction material, nature of structure deformation (bending, displacement or both at the same time) as well as by the degree of its stressed condition.

Since the structure in earthquake conditions has to act in the zone of high stresses, the measurements of free, forced or even resonance vibrations always limited in amplitude, due to the danger of dam damage, cannot settle the problem of defining maximum and at the same time stable (calculating) value of logarithmic decrement.

The only chance left is to investigate the strong forced vibrations of the dam model. The model should thereby be large enough, as a rule not less than 3-5- metres in height (depending on how coarse the kernels are of the dam body material) to use the same material for the construction of a real dam. Otherwise, it would not prove easy to find some other material, though different in elastic qualities, still identical in the character of free vibration damping.

A large model like this will most favourably be tested in field conditions, the model of the river canyon being used as well. The forced vibrations of the model are immitated by the system of mechanical vibrators located in certain sections of the dam body. All this is meant to allow alteration of vibration frequency and kinetic moment of eccentrics and thus to secure resonance curves corresponding at least to the first three types of free vibrations, with different values of pulsating force.

With the aid of proper experimental data the following significant problems can be solved: specification of definite theoretical frequencies and forms of the dam free vibrations with due consideration of its spatial action; definition of the design value of logarithmic decrement; evaluation of non-linearity of vibrations degree depending on the degree of the dam stressed condition; when the deformations acquire destructive character, the process is traced of appearance of residual deformations in the dam, and thus the maximum tolerated intensity of dynamic influence is evaluated.

On the below-given example of the one-degree loose system, which in actual construction will correspond to the case of resonance for any form of free vibrations, the method will be shown for the definition of the design logarithmic decrement and evaluation of vibrations non-linearity degree.

Here is the equation of motion:

$$m\ddot{U}_j(t)+R(\ddot{U}_j,t)+C_j\frac{\delta_j}{\pi}\omega_j\dot{U}_j(t)=\mathcal{P}_j(t)$$

The new symbols are:

m - point mass;
$C_j = m\omega_j^2$ - stage number of pulsating force amplitude
$R(U_j t)$ - elastic force restoration coefficient;
- reaction force consisting of linear and non-linear parts;
$\mathcal{P}_j(t)$ - outer pulsating force.

When attaining resonance, i.e. stationary periodic vibrations, we shall have

$$m\ddot{U}_j(t) + R(U_j t) = 0$$

and

$$C_j \frac{\delta_j}{\pi} \omega_j \dot{U}_j(t) = \mathcal{P}_j(t)$$

From the first equation we define $R(U_j t)$ and hence, degree of non-linearity of dam deformations under vibrations which is of interest for us.

With the aid of the second equation we calculate the value of logarithmic decrement δ_j for different stages of dynamic load up to destruction stage and hence, for the stressed condition of the dam. This done, we define the stable design value δ .

Sometimes large dam models cannot be built and tested in field conditions because of the time shortage.

In this case in the USSR comparatively small dam models would be tested on strong seismic platforms installed into field structure. The model of the river canyon would thereby also be used.

It was like this that the models were tested of 300 metres concrete arch dam, 240 metres massive counter-force dam, 300 metres and less earth and rock-filled dams.

To detect the direction of horizontal seismic force action most dangerous for the dam, the models would be tested under different orientation towards the direction of seismic platform vibrations.

Prior to investigating the models, the following tasks were set. To define the regularity of distribution over the dam height of the intensive seismic load, with account of its spatial action. To do this, it is sufficient to measure acceleration of vibrations in different points of the model with the aid of small and light acceleration indicators. This done, dynamic loads should be measured in the models of concrete and reiforced concrete dams with the aid of deformation gau-

ges. Dimensions and nature of residual seismic deformations in the dam body should also be studied, such as cracks in all types of dams; sinkings and distortions in the initial outline of the slopes and tops of the earth or rock-filled dams. Practical meaning of so estimation these residual deformation values is they can be foreseen beforehand when designing the dimensions of the dam section.

In addition to that, the question of sudden slipping of slopes or its separate parts is of great interest for the earth and rock-filled dams, since this can gravely endanger the dam safety during earthquakes.

And finally, as was proved theoretically and by tests on models, the value and nature of residual deformations in the earth and particularly rock-filled dams are influenced by interaction under the vibration of such heterogeneous in their elastic qualities dam parts as are outer prisms and the counter-filter core. In particular, this interaction serves to promote the development of above-mentioned residual deformations in rock-filled dams more than in earth dams. It was also found that in this respect the slanting counter-filter core or the plastic screen have some advantage over the central counter-filter core.

The main fault of model tests on seismic platforms lies in that practically it is impossible to model the gravity force. This concerns models of earth and rock-filled dams, since the effect of clamping the counter-filter core and outer prisms in the surface zone strongly diminishes residual deformations. Taking this into account, the designers would preliminarily pack the models of earth and rock-filled dams by vibration in temporary shields. This done, the shields would be removed and the model subjected to vibrations in accordance with the given condition.

Vibration condition consists in the following. Acceleration, frequency and duration of vibrations are modelled under the similarity theory. To obtain thereby moderate experimental accelerations and values of model deformations convenient for measurement, the material would be used different from that of the prototype. Thus, concrete would usually be replaced with concrete on rubber filler with various mixtures. Coarse gravel and rock-filling are modelled accordingly by fine sand and road metal. Material of the counter-filter core is replaced by replaced by low-modulus elastic material. The materials used for models were considered selected correctly, in case condition of immutability was observed in comparison to the prototype of transverse deformation coefficient for concrete or reinforced concrete and inner friction angle for dry substances.

In spite of certain drawbacks, the model investigations allowed to make some useful corrections into theoretical formulas and answer a number of practical questions connected

with designing of dams.

The nature tests of dam vibrations are to solve the following problems. To define by expedition investigation the modes of proper vibrations of the already built dams so that the theoretical methods be checked. Stimulation of resonance vibrations is executed with the aid of one mechanical vibrator. This done, we can with the aid of automatically engaged acceleration and tension counters installed in the dam body and on canyon sides obtain records of strong earthquakes which will give us a full idea of how intensive the seismic load is and to what degree the dam body is stressed. This will help to check the entire theory of calculation and designing of dams for seismic areas.

SECTION OF "HYDRO-TECHNICAL STRUCTURES" IN DESIGNING STANDARDS OF EARTHQUAKE-RESISTANT CONSTRUCTION IN USSR

The new designing standards for earthquake resistant construction in the USSR officially approved in 1962 have been considerably improved as compared to the sdandards that acted in 1957 preceded by those of other periods. The acting standards embrace construction of a wide variety of buildings and contain section "Hydro-technical construction". This "Section" carries directions on definition of seismic loads (hydro-dynamic pressure of water and inertia loads) for designing firmness and stability of proper types of hydro-technical structures, such as dams, canals, tunnels, pipe-lines and other structures of supporting walls type. The "Section" also underlines that any type of dam can be made earthquake resistant on condition of proper design and construction with account of seismic loads.

The dynamic effect of seismic coefficient is expressed by one coefficient as compared to usual buildings, whose standards require different dynamic coefficients depending on proper vibration periods (spectral approach). This is explained by the fact that the dam free vibration spectrum (excluding high earth dams) is limited by narrow area and in addition to that due to the absence of sufficient information available on the design of value of free vibrations logarithmic decrement, it was difficult to design a spectral curve of dynamic coefficient fro dams.

The "Section" also carries a number of directions on securing earthquake resistance of dams.

To-day preparatory work is being carried on for a new edition of the standards section which carries the design of dams for earthquake resistance in accordance with a spectral curve of dynamic coefficient designed especially for dams.

BIBLIOGRAPHY *

1. Napetvaridze, Sh.G. Earthquake Resistant of Hydro-Technical Structures. Gosstroiizdat, Moscow, 1959.

2. Kulmach, P.P. Hydrodinamics of Hydro-Technival Structures. Academizdat, Moscow, 1963.

3. Building Standards and Regulations, Vol.II, Section A, chapter 12. "Construction in the seismic areas" (designing standards). Gosstroiizdat, Moscow, 1962.

4. Earthquake Resistance of Structures. Col. papers, vol.I (XII), Institute of structural mechanics and earthquake resistance of the Academy of Sciences of the Georgian SSR. Gruzacademizdat, Tbilisi, 1964.

---

* Detailed bibliography is included in the monographs (1),(2) on the earthquake resistant of hydro-technical structures problems and so in the monograph of (4).

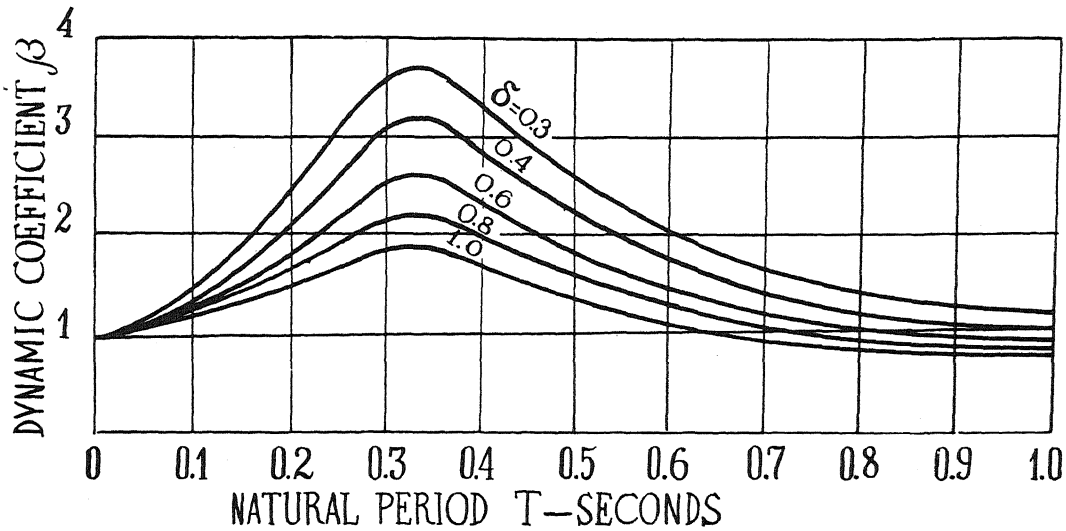

FIG. 1 SPECTRAL CURVE OF DYNAMIC COEFFICIENT β

E R R A T A

RESEARCH ON EARTHQUAKE RESISTANCE OF DAMS IN USSR

BY Sh. G. NAPETVARIDZE

PAGE 433: Equation 5; delete and replace by:

$$p(x) = K_c q(x) \sqrt{\left[1 - \sum_{i=1}^{i=n} \eta_i(x)\right]^2 + \sum_{i=1}^{i=n} \bar{\beta}_i^2 \eta_i^2(x)}$$