2019. 15(6). 458–469

СТРОИТЕЛЬНАЯ МЕХАНИКА ИНЖЕНЕРНЫХ КОНСТРУКЦИЙ И СООРУЖЕНИЙ

STRUCTURAL MECHANICS OF ENGINEERING CONSTRUCTIONS AND BUILDINGS

HTTP://JOURNALS.RUDN.RU/STRUCTURAL-MECHANICS

ЧИСЛЕННЫЕ МЕТОДЫ РАСЧЕТА КОНСТРУКЦИЙ
NUMERICAL METHODS OF ANALYSIS OF STRUCTURES

DOI 10.22363/1815-5235-2019-15-6-458-469
UDC 626.624.04

RESEARCH PAPER

Reliability and applicability of modern numerical analyses of dams

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Article history:
Received: August 12, 2019
Revised: October 17, 2019
Accepted: November 24, 2019

For citation
Lyapichev Yu.P., Mazza G., Mateu E., Zenz G., Carrère A.J. (2019). Reliability and applicability of modern numerical analyses of dams. *Structural Mechanics of Engineering Constructions and Buildings, 15*(6), 458–469. http://dx.doi.org/10.22363/1815-5235-2019-15-6-458-469

Abstract

Relevance. At present the application of numerical analyses to real problems of dam engineering has suffered at times from the gaps between the specialists of mathematical modeling and dam engineers and managers. The first group usually includes information system specialists because they are able to develop the computer models to their full potential. The professionals belonging to the second group often prefer to revert to traditional methods of calculation and empirical methods based on their proven experience. ***The aim of the work*** – based on recommendations of International workshops seminars, organized by the ICOLD Committee on Computational Aspects of Dam Analysis and Design, help dam engineers to interact with mathematical modeling specialists and to work with them without language barriers or gaps in knowledge. In this relation the assessment of reliability and applicability of numerical analyses of dams allows engineers to develop the optimal dam design. ***Methods.*** Assessment of the reliability of numerical methods of analyses of dam behavior was based on data of 10 International benchmark-workshop seminars, organized by the Committee in Italy (1991 and 1992), France (1994 and 2009), Spain (1996), USA (1999), Austria (2001), Romania (2003), China (2005), Russia (2007), in which specialists of these countries also took part.

Keywords: reliability index; numerical models and analyses; static, seismic and hydraulic behavior of the dam

Introduction

ICOLD Committee on Computational Aspects of Dam Analysis and Design published in 2013 Bulletin 155 “Guidelines for use of numerical models in dam engineering” [1]. This Bulletin is a continuation of Bulletin 122 “Reliability and applicability of computational procedures for dam engineering” [2], published in 2001.

Assessment of the reliability of numerical methods of analyses of dam behavior was based on data of 10 International benchmark-workshop seminars, organized by this Committee in Italy (1991, 1992), France (1994, 2009), Spain (1996), USA (1999), Austria (2001), Romania (2003), China (2005), Russia (2007) [3], in which 5 authors and some other Committee members took part [4–12].

Each seminar bore working character: specialists from many countries could check and compare their software programs of numerical analyses (mainly by finite element method) and models of materials in four

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different themes of important aspects of static and seismic behavior of dams (by two themes for embankment and concrete dams). Results of many analyses were compared with the monitoring data of behavior of real dams. The same numerical model, used for the simulation of given aspects of dam behavior, can be relatively reliable for different periods of its service: construction, initial filling of reservoir and operation and for different types of dams.

Reliability and applicability of modern numerical analyses of dams

Numerical modeling of main aspects of behavior of concrete and embankment dams under action of static, seismic and hydrodynamic loads is analyzed. On the basis of 10 International seminars on numerical analysis for dams the assessment of computer programs of numerical analysis (by the finite elements method) concrete and embankment dams and recommendations for choice of mathematic models for their materials are given. The classification and recommendations are given for the choice of mathematic models of soils and concrete for the numerical analyses of the stress-strain state of embankment and concrete dams and their foundations. These recommendations are included in Bulletin 122 of the International Commission on Large Dams (ICOLD) “Reliability and applicability of computational procedures for dam engineering “, prepared for dam professionals throughout the world.

In the following paragraph with reference to dam typology and to different periods of dam life, the capability of numerical modelling to analyze phenomena related to dam safety is shown. In fact, the same model used to analyze a definite phenomenon can be reliable during the various periods of life of a dam (construction, first fillings, operation) or for the various dam types. Four numerical indices (1, 2, 3, 4), defined Reliability Indices (RI), are adopted whose meaning is as follows:

RI = 1 – the phenomena related to dam safety can be confidently analyzed by numerical models;

RI = 2 – the phenomena related to dam safety can be analyzed by means of numerical models but:

– with some limitations and/or difficulties (simplifications in the computational hypotheses;

– lack or difficulty to get fully reliable experimental data, relevant cost of the analyses, etc.);

RI = 3 – the phenomena related to dam safety can be analyzed by means of numerical models, whose results can give only qualitative or comparative indications, e.g. because of the strong simplifications needed, etc.;

RI = 4 – the phenomena related to dam safety cannot be analyzed by means of numerical models.

In general, only one index is used for each item *phenomenon-dam life period*. In some cases, more indices are used if the reliability of numerical modelling is considered different for various dam types.

In the following paragraph capital letters have been used in this way: C – Concrete; E – Embankment; S – Static; D – Dynamic; H – Hydraulic.

For various dam types and their behavior aspects (CS, CD, CH, ES, ED) matrices are given $[i, j]$, where i – number of the given aspect of dam behavior (in the order of its significance for dam safety) in the given its life period: $j = 1$ (construction), $j = 2$ (first filling of reservoir), $j = 3$ (operation).

1. Concrete dams – static behavior

1.1. Stress-strain state

CS [1,1], RI = 2. *Construction phase*

Numerical methods and procedures are available for simulation of the construction phases for all dam typologies. Difficulties arise from:

a) complexity of the phenomena occurring during these phases, such as concrete setting, hardening and shrinkage (consequent stresses produce cracks and affect the stress-strain-state);

b) deformability of rock foundation, which is influenced by non-homogeneities of massive rock and by presence of potential sliding faults whose parameters of mathematical model is a very difficult task;

c) the prediction of temperature changes in early age concrete due to the heat of hydration. The influence of many variables, i.e. amount and type of cement used for the concrete mix, thickness of lifts, time interval between lifts, different height of blocks, external temperature condition, should be more deeply known. Therefore, the early age volumetric changes and crack-producing tendencies cannot be analyzed in a completely reliable way with available numerical models;

d) lack of information about construction joints grouting. Contraction joints can be grouted for arch dams to form monolithic structure. In this case grouting causes pre-loading condition, that is unknown.

Reliable numerical analyses (i.e. software codes and adequate experimental-numerical validation researches) of stress-strain state in the construction phases have been set up recently.

In most cases the evaluation of the aspects mentioned under points *a* and *c* is still based on practical experience which however has shown to be not sufficient to avoid the cracking problems. The monitoring practice of placing many thermometers and thermo-extensometers in the concrete mass during casting has been in use for many decades; unfortunately, in most cases, the relevant measurement data (besides being too local in the case of thermo-extensometers) are not complemented with the detailed additional information about construction history.

CS [1,2], RI = 2. *First filling of reservoir*

The approximation level of numerical simulation for the first filling is due to lack of information on permeability of rock foundation and concrete, which could affect foundation deformability parameters.

Difficulties also arise from non-linear phenomena induced in the dam-foundation system by the progressive increase of the hydraulic loading (settlements, creep, joints displacements). In particular:

a) consolidation, whose parameters are in general not completely known, should be taken into account to predict settlements during construction and causing stresses in dam and foundation;

b) under applied loadings the rock continues to deform (creep). The deformation characteristics of foundations are significantly influenced by the density, orientation, width and contact condition of joints and cracks of loaded rock surfaces. All this quantity is generally not known in complete way;

c) permeability of the foundation rock can affect significantly the stress-strain state, particularly in gravity dams. The permeability of the foundation including joints, fault zones and cavities is needed to determine pore pressures for analyses of stresses and stability. If foundation grouting, drainage or other treatments are involved, their effect on pore pressure distribution should be included.

The above described features affecting the approximation level of the computations are taken into account in the design phase, on the basis of the experience and of qualitative evaluations, mainly in the design of the foundation treatments, in the definition of the first filling program and in the design of the monitoring network. Usually an extensive and careful monitoring is also carried out to check the dam and foundation behavior in this delicate phase; many monitoring instruments are installed for this purpose only, and are abandoned after the successful completion of the first filling phase.

CS [1,3], RI = 1. *Operation phase*

Numerical models can be considered completely reliable because the data obtained by means of the monitoring of the dam can allow to establish good identification of dam-foundation-reservoir system.

1.2. Local and global stability related to cracking state

CS [2,1] = CS [2,2], RI = 3. *Construction and first fillings phases*

The approximation level is due to the combination of what described in notes 1.1 together with the difficulties for a fair characterization of the crack formation and propagation process.

CS [2,3], RI = 2. *Operation phase*

The numerical simulation of the dam behavior during operation could be facilitated by the knowledge of the observed performances during the previous phases of its life, allowing to remove some of the uncertainties affecting the construction and the first filling phases. The main difficulties arise from inadequacy of numerical models and lack of experimental evidences. In particular:

a) in analyzing concrete structures with realistic constitutive models, difficulties lie in the fact that uniqueness and stability of the solutions are guaranteed only below certain load levels. This means that the numerical algorithms should be very accurate otherwise numerical errors may easily trigger potential instabilities leading to an underestimation of the failure loads;

b) further difficulties are connected with fracture testing. The most important problem is the uncertainty of the reproducibility for the crack propagation phenomenon varying the shape and/or the size of the cracked structure. It is still not completely clear how far it is possible to extrapolate the laboratory results to the structure of large sizes and complex shapes. Consequently, the available mathematical models have not been sufficiently validated.

Significant cracks are usually checked in the surveillance. In addition to periodic visual inspections, instruments are installed to monitor the crack opening displacement of the main cracks. The direct monitoring of crack is necessary, because often crack propagation has little influence on measured integral quantities (crest displacements), unless cracking has not reached very significant extension.

1.3. Sliding and overturning

CS [3,1] = CS [3,2] = CS [3,3], RI = 1. *Construction, first fillings and operation phases*

In spite of the simplicity of the methods usually adopted (limit equilibrium analyses with reference to rigid body formulation) they have been proved generally reliable. Nowadays numerical methods can also be adopted; such methods are able to overcome the limitations of a rigid body formulation that cannot give any information about strains and displacements at a state close to failure.

1.4. Instability of slopes or blocks

CS [4,1] = CS [4,2] = CS [4,3], RI = 2. *Construction, first fillings and operation phases*

In this case material properties and geotechnical data can be of difficult (or expensive) evaluation.

1.5. Seepage

CS [5,2] = CS [5,3], RI = 2. *First fillings and operation phases*

The approximation level is mainly due to the non-homogeneity of the rock mass and to the difficulties in defining a computational scheme for a reliable evaluation of the phenomenon. The difficulty to model the foundation treatments (grouting, drains, etc.) and the strong coupling between permeability and stress state in jointed rock mass can reduce the reliability of numerical results.

Anyway, even for dams on sound rock foundations, grout curtains and drainage systems are built as preventive measures apt to limit the seepage and relevant pore water pressures. The check of the proper functioning of drainage and water-tightness systems is usually included in the surveillance activities. The complexity of the analyses still remains mainly because of the difficulty to define material properties (permeability as a function of the stress-strain state).

1.6. Ageing, alkali-aggregates reactions and similar phenomena

CS [6,3], RI = 3. *Operation phase*

For such phenomena, numerical analyses can be used only for qualitative evaluations, or to estimate the order of magnitude of some effects. For dams under design, the current knowledge allows to prevent the onset of such problems by means of proper choice and control of aggregates and cement type.

For existing dams alkali-aggregate reactions and ageing of materials can be detected through in situ and laboratory tests. No direct numerical modelling of the phenomena are available. However the effects due to alkali-aggregates reactions can be, for instance, evaluated by means of equivalent thermal analyses whereas the ageing can be simulated assuming a suitable progressive variation of physical-mechanical parameters (stiffness, permeability, etc.); also the effects of remedial measures, such as diamond wire cutting, can be evaluated by means of suitable numerical models.

2. Concrete dams – dynamic behavior

2.1. Stress-strain state

CD [1,1] = CD [1,2] = CD [1,3], RI = 2. *Construction, first fillings and operation phases*

The modelling of the dynamic loads introduces additional approximations compared to static analysis. In fact, for dynamic analysis some aspects need to be better highlighted. In particular:

a) the effect on strength and elastic properties of materials under dynamic loads, which alternate between tension and compression, have not been fully investigated. The elastic modulus of concrete, as well as the yield limit, varies with the rate of application of loads (e.g., the instantaneous modulus of elasticity and the strength of concrete can be more than 1.5 times the static modulus);

b) in concrete, stress-strain relationship exhibits a softening behavior when the tensile strength is exceeded. The capability of concrete to dissipate additional energy before complete failure may play an important role with regard to seismic action where the dam structure has to absorb energy arising from ground motion. Appropriate models of these aspects have not yet completely been attained;

c) In the mathematical reproduction of dynamic behavior of dams a difficulty is originated by the need of a suitable modelling of the energy dissipation mechanism through dam-rock-reservoir system;

d) the incidence of floating debris on structural stability during floods and the ice-structure interaction (pseudo-static thrust and dynamic impact) have to be also taken into account in particular situations. Consequently, the approximation index 2 has been assigned to the dynamic analyses independently from the phase of life of the dam. However, for dams in operation, when data related to the seismic behavior are available (e.g. when a seismic monitoring system is installed on the structure or when forced vibration tests have been carried out), a Reliability Index RI = 1 could be assumed.

2.2. Local and global stability related to cracking state

CD[2,1] = CD[2,2] = CD [2,3], RI = 3. *Construction, first fillings and operation phases*

Experiments under simulated seismic action should be performed to investigate fracture properties under dynamic loadings. The literature indicates that both tensile and compressive strength of concrete increases with increased strain rate; but tensile strength is more rate sensitive. The safety assessment of concrete dams when subjected to earthquakes strongly depends on the tensile and cracking behavior of the material. The assumptions regarding the incidence of water-crack interaction on the transient evolution of uplift pressures acting in cracks during earthquakes, that vary widely among dam safety guidelines (from zero, to the full reservoir crack mouth pressure), still remain an open problem even for static behavior. See, also, for applicable part, note relevant to 2.1.

2.3. Sliding and overturning

CD [3,1] = CD [3,2] = CD [3,3], RI = 2. *Construction, first fillings and operation phases*

See also for applicable part notes relevant to 1.3 and 2.1.

2.4. Instability of slopes or blocks

CD [4,1] = CD [4,2] = CD [4,3], RI = 3. *Construction, first fillings and operation phases*

See for applicable part notes relevant to 1.4 and 2.1.

3. Concrete dams – hydraulic behavior

3.1. Cavitation

CH [1,1] = CH [1,2] = CH [1,3], RI = 2. *Construction, first fillings and operation phases*

Numerical models for simulation of free surface flow over spillways are currently available. They can give sufficient information about pressures distribution on dam downstream face. Some limitations occur in presence of pressure pulsations due to development of turbulent boundary layer. In this case more advanced numerical models should be used. Small physical models are used for the problem.

3.2. Erosion

CH [2,1] = CH [2,2] = CH [2,3], RI = 3. *Construction, first fillings and operation phases*

With reference to analysis of flow velocities on dam face and to solid material content (incidence of floating debris on hydraulic response of dams and spillways) numerical models can supply useful information. For assessment of erosion it is necessary to resort to correlations with experimental data.

3.3. Dislocation of paving slabs of spillways

CH [3,2] = CH [3,3], RI = 4. *First fillings and operation phases*

Presently such phenomena, due to pressures pulsation when the spillway is in operation, can not be modeled mathematically, but reduced scale physical models can be used.

3.4. Extreme flood

CH [4,1] = CH [4,2] = CH [4,3], RI = 1. *Construction, first fillings and operation phases*

The evaluation of extreme floods is a classical problem in hydrology and several methods are currently applied which can vary from country to country because of the different traditions and type of data available. It can be said that the methods are generally simple to apply and the accuracy of results mainly depends on the availability and reliability of long sets of historical data on past extreme events. At any rate this is a topic of interest of the Committee on Dams and Floods. Once the extreme flood has been evaluated, the numerical simulation of discharge is considered fully reliable (RI = 1).

3.5. Downstream heel erosion

CH [5,2] = CH [5,3], RI = 4. *First fillings and operation phases*

In general this kind of problem, which can be included in local scour phenomena, is dealt with by means of reduced scale physical models.

3.6. Siltation

CH [6,3] = 2, RI = 2 (for coarse sediments), RI = 3 (for fine sediments). *Operation phase*

The sediments are subdivided in different classes and appropriate equations are applied to each class. A distinction is made between very fine sediments, such as silt and clay which constitute and coarser sediments, such as sand and gravel which form the bed load and suspended load. Mathematical models of coarse sediments have been extensively applied and the experience developed in their use have reached sufficient degree of reliability for practical purposes. It is strongly advisable to collect as many field data on the past evolution as possible in order to best calibrate the model parameters on the particular case under study. Therefore RI given to the first set of models of coarse sediments evolution is 2. Referring to dynamics of very fine sediments the different processes that can occur are more complicated and not fully understood. The available mathematical models have not been extensively applied so that there is little experience in their use and little knowledge on their degree of reliability.

4. Embankment dams – static behavior

4.1. Stress-strain state

Currently the most frequently used mathematical models for the analysis of embankment dams are essentially numerical ones of finite element method type. Usually the applied models depend on the type of embankment dam. An analysis in total stresses is satisfactory for rockfill dams with cement or bituminous concrete face. For clay core rockfill dams or permeable earth dams the analysis in effective stresses is strictly recommended. In the last years advanced computer codes have become available, which allow to perform coupled solid-fluid analyses. In the latter case the analysis is always performed in effective stresses and excess pore pressure is one of the field variables (this being a transient type analysis based on integration in the time domain short term response becomes a result of computation).

The constitutive models that are usually adopted to model the behavior of both the solid skeleton and the interstitial pore fluid differ depending on whether an uncoupled or a coupled analysis is performed. In the following a list of the more frequently used models is reported.

The detailed classification and selection of constitutive models of soils for embankment dam analysis was given in Appendix 2 [4] in ICOLD Bulletin 122. This classification is meant to help the practising engineer to identify those models which are of relevance in a particular situation. The brief description of principal models is given below.

Hyperbolic Duncan – Chang model [13]. This model provides quite often acceptable results as it allows to follow in satisfactory way the non-linear behavior of the material. The main characteristic of this model is that the deformation modulus is a function of both the isotropic and deviatoric stresses during monotonic loadings, like those occurring during the construction phases of an earth dam.

Elastic perfectly-plastic models (Von Mises, Mohr – Coulomb, Drucker – Prager, etc.). Usually it is necessary to use these models with a non-associated flow rule, since they would otherwise produce quite unrealistic dilatant effects.

Hardening elasto-plastic models. Isotropic, kinematic, isotropic-kinematic hardening rules with hardening due to plastic volumetric and shear strains give rise to these models of varying complexity. The Critical State (CS) model [14], models of Lade [15], Prevost [16] and Zaretsky [17] are a few examples in this category. The well-established CS model (modified Cam Clay model) is the basis of 30 models proposed for monotonic and cyclic loads. The use of such models is the conceptually correct.

Elasto-viscoplastic models. Post-constructional settlements of earth and, especially, rockfill dams are controlled by creep or time-dependent plastic strains of soils and rockfill. Due to the wet clay cores delayed consolidation is observed and the post-constructional performance of these dams is controlled more by creep than by primary consolidation. Neither the assessment of cracking in the clay cores nor the safe design of the concrete upstream facing can be carried out without taking creep into account.

ES [1,1], RI = 2. *Dam body – construction phase*

In most of the cases the excess pore pressure could be neglected in this phase; according to this, analyses of uncoupled type give acceptable results. However, it should be considered the possibility of saturation of the core during the construction phase for zoned embankment dams. In these cases it is necessary to analyze the consolidation phenomenon through a coupled analysis.

ES [1,2] = 1, RI = 2, RI = 1. *Dam body – first filling of reservoir*

The partial reliability (RI = 2) of this analysis arises from difficulty of predicting propagation of the saturation line and wetting collapse phenomena which take place during first saturation of materials; the difficulty of determination of reliable parameters makes the analysis not completely satisfactory. Only for embankment dams with the upstream facing these aspects could be not so important (RI = 1).

ES [1,3], RI = 1. *Dam body – operation phase*

The analysis of the operation phase conditions is more complex in the case of types *a, b, d*, embankment dams. This is due to the importance of transient hydraulic effects during rapid drawdown of the reservoir which are difficult to be modelled and also to the difficulty to define representative parameters to be included in uncoupled analyses. It is then necessary to perform coupled type analyses for a proper modelling of the operation phase. For any type of embankment dam a better reliability in the predictions could be obtained through back-analysis of the dam behavior in the preceding phases.

ES [2,1] = ES [2,2] = ES [2,3], RI = 2. *Foundation – construction, first filling and operation phases*

See for applicable parts ES [1,1], ES [1,2] and ES [1,3].

4.2. Stability

ES [3,1] = ES [3,2] = ES [3,3], RI = 1. *Dam body – construction, first filling and operation phases*

Global stability analyses are usually performed using both the limit analysis and limit equilibrium methods with the soil modelled as rigid, ideally plastic medium. Such methods provide measure of the distance among the stress states present in the operating life and those which correspond to different hypothetical failure mechanisms. In this case the observation of the dam behavior does not allow the validation of the results of analyses, but the professional practice has brought to adoption of safety coefficients for various failure mechanisms which are considered suitable to guarantee the safety.

For application to dams, a two-dimensional model is often sufficient to give acceptable results. In some cases, it is necessary to perform three-dimensional analyses; also in this case specific computer codes are available. A correct analysis should take into consideration, especially for the case of rockfill materials, the non-linearity of the shear strength curve as a function of the normal stress.

In some situations, the analysis of embankment dams could be integrated with stress-strain analyses (elastic-plastic) of assumed failure: such analyses could highlight phenomena of stress redistribution among the different zones of embankment as well as mechanisms of progressive failure. The analysis of embankment dams stability developed with the above mentioned techniques requires the evaluation of the pore pressure distribution through the fill; in the conditions of rapid draw-down the evaluation of pore pressures could be less reliable with respect to those evaluated in the steady state conditions.

ES [4,1] = ES [4,2] = ES [4,3], RI = 2. *Foundation, abutments, slopes – construction, first filling of reservoir and operation phases*

The analyses concerning the dam foundation or the slopes in the reservoir zone present greater uncertainties (with respect to analyses of dam body) because of the difficulty to ascertain in complete way the mechanical properties of the soils and hydraulic conditions in the subsoil.

As far as the hydraulic conditions are concerned, the models for the forecasting of the piezometric level changes in the shells or banks, in consequence of the precipitations and of the oscillations of the level of the reservoir, are still not completely reliable (particularly in conditions of rapid draw-down).

As for the dam body the global stability analysis involves in general the use of the limit equilibrium methods. In many cases the problem must be treated as three-dimensional ones. It could be sometimes necessary to extend the limit equilibrium analysis with more complete stress-strain analysis: for example when the conditions of flow through the rock/soil mass have been altered by the stress state induced by the dam or if the possibility of phenomena of progressive failure exists.

4.3. Seepage

Seepage analyses are usually of uncoupled type that is they do not take into consideration the stress-strain aspects (consolidation, compaction of materials, modification of permeability characteristics). Depending on the problem to be solved, both steady state or transient type analyses can be performed.

ES [5,2] = 2, ES [5,3], RI = 1. *Dam body – first filling of reservoir and operation phases*

The seepage analysis should not be all-important for embankment dams with facing. As already said analysis of situation during the first impounding presents greater complexity due to phenomena of saturation of materials. In operating life, the greater difficulty concerns conditions of rapid draw-down.

ES [6,1] = ES [6,2] = ES [6,3], RI = 2. Foundation – *construction, first fillings and operation phases*

The lesser reliability of results of the analyses is due to the difficulty in determining representative values and distribution of permeability by means of in situ tests; a particular case is represented by the Karst-formations. In some particular situations (permeability related to presence of joint systems) the influence on permeability of the stress state changes induced by construction of the dam is remarkable, but very difficult to be assessed quantitatively. Further uncertainties are introduced when a diaphragm wall or a grout cut off (with their possible defects) and the presence of the drains is to be modelled.

4.4. Internal erosion

ES [7,2] = ES [7,3]= ES [8,1] = ES [8,2] = ES [8,3], RI = 4. *Dam body and foundation – construction, first filling of reservoir and operation phases*

The problem is very important for zoned embankment dams, but it could be of some relevance also for the homogeneous earth dams; it seems that it can be excluded for the dams with facing, unless the consequences of a break of the facing are to be analyzed. At present, during the design phase of an embankment dam, the available procedures to prevent internal erosion are based on filter criteria. In recent years many different criteria have been established for filter design: ICOLD Bulletin 95 [9].

During construction, segregation of graded materials should be avoided, since it may itself create conditions susceptible to internal erosion in the core. Particular care is needed to guarantee continuity of junctions and homogeneity within the embankment and as much as possible in the foundation soils.

Since the leakage development and erosion cannot be prevented consequences of leakage has to be handled in order to maintain sufficient safety. In order to fulfil this requirement, the strict monitoring and investigation systems for localizing deteriorated or potentially weak zones is recommended. Quantities as water losses, turbidity of water losses, pore pressures, deformations, etc. should be monitored.

4.5. Hydraulic fracturing

ES [9,2] = ES [9,3] = ES [10,1] = ES [10,2] = ES [10,3], RI = 3. *Dam body, foundation – construction, first filling of reservoir and operation phases*

The phenomenon has been conventionally related to the event that in one or more zones of dam the minor principal stress becomes lower than pore pressure. Back-analyses carried out on embankment dams that experienced big problems of concentrated seepage have pointed out cause of such seepage in the hydraulic fracturing developed in the upper portions of the fill. In fact, due to the arching effect, in the upper part of the dam total vertical stresses can become lower than the pore pressure induced by the reservoir. The phenomenon of hydraulic fracturing cannot be *a priori* excluded for homogeneous earth dams, but probability of occurrence is higher for zoned embankment dams or in dams which present strong differences of deformability or permeability characteristics in different parts of dam.

5. Embankment dams – dynamic behavior

The level of difficulty of dynamic analyses for the forecasting of the embankment dams behavior under seismic conditions is higher with respect to the correspondent static analyses.

The reduced reliability can be attributed to greater difficulty in defining load conditions in analysis and to major uncertainties about a series of factors among which the most important are the following:

a) the difficulty to define, isolate and also to model the reflecting/absorbing boundaries of analysis domain (this is also valid for concrete dams);

b) the interaction of dam with foundation soil; in case the relatively soft subsoil it modifies greatly the seismic dam response with respect to the case of relatively rigid subsoil. It is of the importance to detect within saturated alluvium foundation the event of low density and low mechanical characteristic layers, but it remains really difficult to achieve the determination of those parameters. The incidental uncertainties on definition of foundation soils properties affect predictions of dynamic dam behavior;

c) the dependence of mechanical properties that determine the seismic response from static stress-strain state existing at the time of seismic action. These conditions are defined through preceding static analysis with consequence that uncertainties are reflected directly on reliability of dynamic analysis.

For the evaluation of the response of embankment dam to earthquake excitation, during and after the seismic action, the analysis of the stress-strain state constitutes the basis of any prediction. The global stability, liquefaction potential, the behavior after seismic action, the phenomena of concentrated seepage, the hydraulic fracturing, etc. can in fact be faced with pseudo-dynamic approaches, taking adequately into account the results of the seismic analysis of the embankment dam and its foundation.

On the other hand, it should be emphasized that, even if widely used conventional pseudo-static methods are not capable to take into consideration some important phenomena such as amplification effects, residual excess pore water pressures after seismic actions, liquefaction, stress redistribution. However, the interaction of dam with reservoir does not significantly modify frequency response of embankment. The following considerations are restricted to evaluation of stress-strain state.

5.1. Stress-strain state

ED [1,1] = ED [1,2] = ED [1,3], RI = 3. *Dam body and foundation – construction, first filling of reservoir and operation phases*

Modelling typologies. It is well known that analyses carried out by means of a linear constitutive model are incomplete, unless low intensity seismic events are considered. It is thus taken for granted that uncoupled, or better, coupled time-history analyses must be used, but in any case with non-linear constitutive models using step integration or, at least, the equivalent linear approach; the latter in fact shows itself to be sufficiently adequate for maximum acceleration, stresses and strains evaluation, but evidently inadequate to estimate permanent stress-strain phenomena. In dynamic analyses, both uncoupled and coupled with non-linear constitutive models, the main difficulty lies in introducing constitutive models which are capable to reproduce satisfactorily, under dynamic cyclic loads, the following aspects:

a) the stress-strain response corresponding to load pattern inversions, many models tend to amplify the hysteresis, compared to what is found experimentally;

b) the dissipative behavior of dam-foundation system arising from plastic deformations and viscous phenomena (this has been confirmed from experiments and workshop seminars of our Committee);

c) phenomena of cyclic degradation (variability of stress-strain law with growth of cycles number);

d) phase coupling effects (a correct prediction of excess pore pressures build up is fundamental not only for maximum acceleration forecast, but also to evaluate post seismic effects).

For all the above mentioned reasons, it is not to be taken for granted that the coupled analyses, with the use of elasto-plastic models, are more reliable than the uncoupled ones in terms of total stresses, given that the latter are provided with criteria for excess induced pore pressures predictions.

The border-line of present modelling is represented by three-phase medium to simulate behavior of partially saturated soils. It has been recently demonstrated that effect of variability of core material saturation degree shows great influence on stress-strain response under cyclic loads. However, it is right not to cross border-line of two-phase medium, provided that constitutive parameters are correctly evaluated. Difficulties in modelling mechanical behavior under seismic loads are greater for homogeneous and zoned dam but they are also highly sly for rockfill dam, the more for loose material is loose.

Parameters definition and assumption. If the dam is in highly seismic area, material properties are determined by experiments, which are: dynamic in situ tests, laboratory tests (on core materials and if it is important, on foundation materials).

These tests supply deformability parameters with the accuracy requested by the methods of analysis. Less reliable is forecast of deformability parameters to be introduced in static analyses. These are in fact hardly deducible from in situ geotechnical tests, and show themselves to be of poor reliability when obtained from conventional laboratory tests (triaxial ones), not suited to the characterization of behavior at different deformation levels relevant to various life conditions of the dam. As concerns the damping ratio to be introduced both in linear equivalent and non-linear analyses, it is now widely demonstrated that gross estimations of this parameter induce high result dispersions. This is further complicated by the remarkable variability of damping capabilities of cores in connection with excitation frequencies and confining stresses.

Heterogeneities, boundary conditions, geometrical factors. These factors appear relatively less important in amplifying intensity of seismic event. For strong-motion events physical-mechanical factors connected to non-linearity of the materials constitutive law are of great importance. On the contrary, when the analysis can be carried out with linear models (weak-motion events), factors such as geometrical variability of properties, subsoil stiffness, the valley morphology, the height/length ratio of the embankment, gain a decisive role in response evaluation.

5.2. Post seismic internal erosion

ED [8,2] = ED [8,3] = ED [9,1] = ED [9,2] = ED [9,3], RI = 4. *Dam body and foundation – first filling of reservoir and operation phases*

Since the post seismic internal erosion cannot be prevented or anticipated, in order to maintain a sufficient level of safety the installation of an appropriate monitoring system is strongly recommended. Quantities such as deformations, pore pressures, water losses and its turbidity should be measured.

6. Embankment dams – hydraulic behavior

Hydraulic phenomena considered in embankment dams are practically the same as those in concrete dams (also the relevant indices are the same for the same phenomena). See notes of paragraph 4.3.

Conclusions

It appears useful here to briefly summarize problems related to safety that do not seem at present approachable in a reliable manner by means of mathematical models (RI 3 and 4).

Concrete dams:

– local and global stability related to cracking state in construction, first filling (static and dynamic behavior, RI = 3) and in operation (dynamic behavior, RI = 3);

– damage due to ageing, alkali-aggregate reactions (only static behavior, RI = 3) and dissolution due to seepage (only static behavior, RI = 4);

– instability of slopes or blocks in construction, first filling, operation (dynamic behavior, RI = 3);

– erosion of outlets during construction, first filling and operation (hydraulic phenomena, RI = 3);

– dislocation of spillway paving slabs in first filling and operation (hydraulic phenomena, RI = 4);

– downstream heel erosion of dam in first filling and operation (hydraulic phenomena, RI = 4).

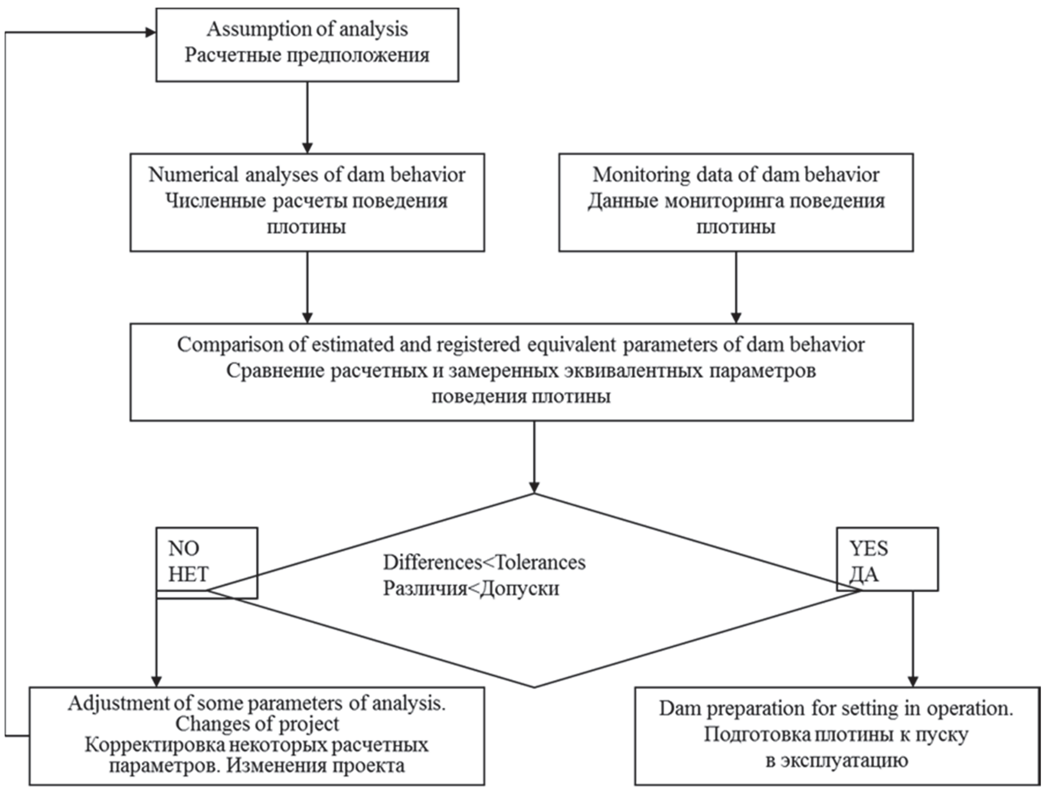

Figure 1. Interaction between monitoring data and numerical modelling of dam behavior before the first reservoir filling

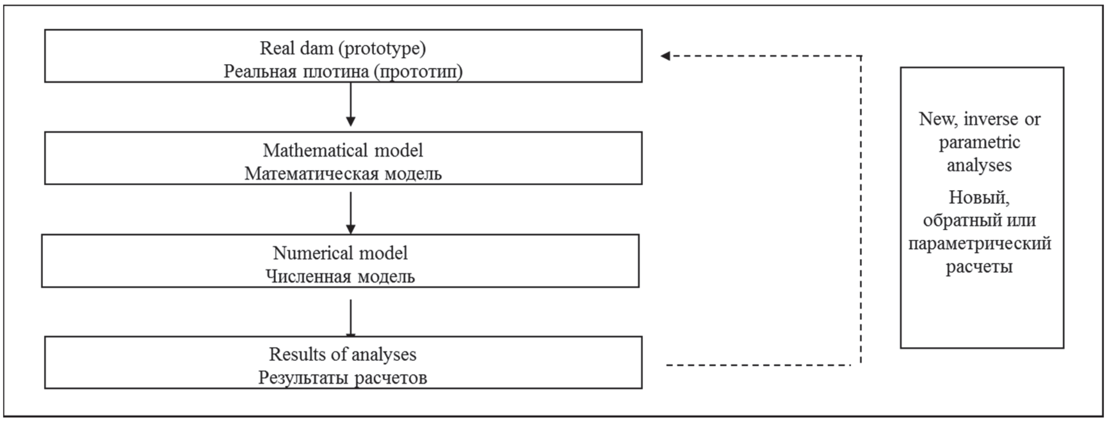

Figure 2. Four stages of numerical modelling of dam behavior during its construction

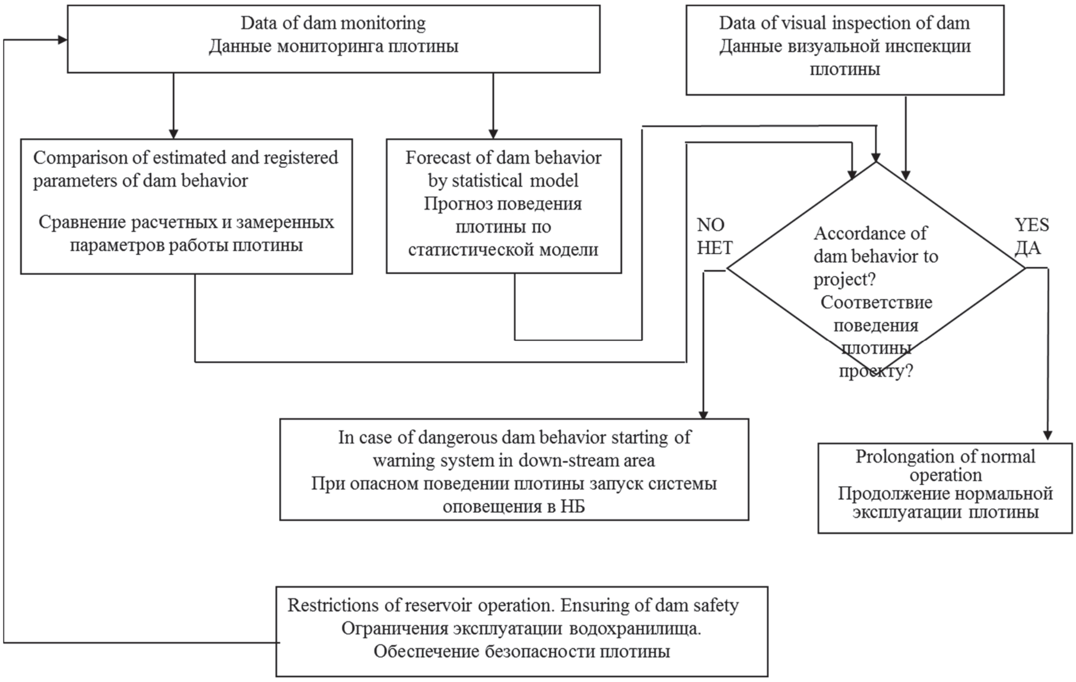

Figure 3. Interaction between monitoring data and numerical modelling of dam behavior in its operation

Embankment dams:

– internal erosion in construction, first filling and operation (static, post-seismic behavior, RI = 4);

– hydraulic fracturing in construction, first filling, operation (static, post-seismic behavior, RI = 3);

– physical-chemical deterioration phenomena (including dissolution and dispersive clays) during construction, first filling and operation (only static behavior, RI = 4);

– evaluation of stress-strain state in construction, first filling, operation (dynamic behavior, RI = 3);

– evaluation of the global stability of abutments and slopes during construction, first filling and operation (dynamic behavior, RI = 3);

– liquefaction of foundation in construction, first filling, operation (dynamic behavior, RI = 3);

– post-seismic seepage during construction, first filling and operation (dynamic behavior, RI = 3);

– erosion of outlets during construction, first filling and operation (hydraulic phenomena, RI = 3);

– downstream heel and facing erosion of dam during construction, first filling and operation (hydraulic phenomena, RI = 4).

The safety related problems listed above are, hence, those that deserve the efforts of researchers to promote the implementation of adequate tools for the advancements of the state-of-the-art in the field of dam engineering. These tools in numerical modelling of dam behavior are shown in figures 1–3.

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НАУЧНАЯ СТАТЬЯ

Достоверность и применимость современных численных расчетов плотин

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История статьи:
Поступила в редакцию: 12 август 2019 г.
Доработана: 17 октябрь 2019 г.
Принята к публикации: 24 ноябрь 2019 г.

Аннотация

Актуальность. В настоящее время применение численных расчетов к реальным проблемам плотиностроения часто страдает от расхождений между специалистами по математическому моделированию и инженерами и менеджерами по плотинам. Первая группа обычно включает в себя специалистов по информационным технологиям, так как они способны разработать

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эффективные компьютерные модели для численных расчетов плотин. Специалисты второй группы часто предпочитают обращаться к обычным методам расчета и эмпирическим методам, основанным на их проверенном опыте. ***Цель*** – на основе рекомендаций Международных рабочих семинаров, организованных Комитетом СИГБ по компьютерным аспектам расчета и проектирования плотин, помочь инженерам по плотинам взаимодействовать со специалистами по математическому моделированию и работать с ними без языковых барьеров или расхождений в знаниях. В этой связи оценка достоверности и применимости численных расчетов плотин позволяет инженерам разработать оптимальный проект плотины. ***Методы.*** Оценка достоверности численных методов расчетов поведения плотин основана на данных десяти Международных рабочих семинаров, организованных Комитетом СИГБ в Италии (1991 и 1992), Франции (1994 и 2009), Испании (1996), США (1999), Австрии (2001), Румынии (2003), Китае (2005), России (2007), в которых специалисты этих стран также принимали участие.

Ключевые слова: индекс достоверности; численные модели и расчеты; статическое, сейсмическое и гидродинамическое поведение плотин

Для цитирования
Lyapichev Yu.P., Mazza G., Mateu E., Zenz G., Carrère A.J. Reliability and applicability of modern numerical analyses of dams (Достоверность и применимость современных численных расчетов плотин) // Строительная механика инженерных конструкций и сооружений. 2019. Т. 15. № 6. С. 458–469. http://dx.doi.org/10.22363/1815-5235-2019-15-6-458-469

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