

Investigations of the Material of a 50 Years old Dam in the Course of the Deepened Examination

V. Bettzieche¹ and A. Bieberstein²

¹ Ruhrverband, Kronprinzenstr. 37, D-45128 Essen, Germany

E-mail: vbe@ruhrverband.de

² Universitaet Karlsruhe, Postfach 6980, D-76128 Karlsruhe,

E-mail: andreas.bieberstein@ibf.uka.de

Abstract

A new version of DIN 19700 that postulates the technical rules for dams in Germany was published in July 2004. The DIN requires deepened examination of dams about every 10 years. A crucial part of the deepened examination is a consideration of the stability of the dam.

At the Bigge Dam, owned by the Ruhrverband it was postulated at the construction time, that one has to reckon with a certain reduction of the shear strength over time. Since that, it was decided to take samples of the material from the dam during the deepened examination and to perform sieve analyses, large triaxial tests and water permeability tests.

The results of the material tests formed the basis for determining the material parameters so that the stability of the dam could be safely proven.

Introduction

In July 2004 a new version of the german DIN 19700 [1] was published, being the central body of technical rules for dams in Germany. It postulates the regular, deepened examination of dams as a rule of technology, a procedure that has proven itself in practice [5]:

“The deepened examination should re-record all relevant safety cases for which changes have occurred in the input parameters with the latest valid characteristics and according to the technical regulations applicable in each case.” [1]

The Ruhrverband is one of Germany's biggest dam operators. Its nine reservoirs have a total retaining capacity of 474 million m³. The largest reservoir of the Ruhrverband is the Bigge Reservoir, which is the 5th of the great reservoirs in Germany with a capacity of 172 million m³. After more than 40 years of operation, the supervising authorities ordered the Ruhrverband to carry out an deepened examination of the Bigge Dam.

The Deepened Examination

Dams require not only regular or annual inspections but also special tests and examinations to reliably assess the stability of the facility, irrespective of their age. Apart from wear and ageing of the facilities and installations, updated legislation, new or altered requirements of society as regards the operation and stability of dams can also be a reason for supplemental examinations and investigations. This can consequently lead to a need for action that can take the form of individual building measures through to an extensive rehabilitation of the overall facility.

The German sets of regulations in DIN 19700 [1] and the DVWK-Bulletin 231/1995 „Handbook for Safety Reports on Dams“ [2] recommend a so-called "deepened examination" of the basic static, hydrological and hydraulic design principles of the dam at intervals of around 10 years or after extraordinary events. With reference to the procedure when building a new dam, the corresponding dam has to be investigated in accordance with the generally acknowledged rules of technology and derived requirements, where by all former experience with its operation and measurements also have to be taken into account. This examination of dams in the Federal Republic of Germany is based upon different regulations in each state, which is shown in [5].

A crucial part of the deepened examination is a consideration of the stability of the barrier. Changes in loads that can occur through new flood calculations or altered earthquake parameters often have to be taken into account. But changes may also have occurred in the dam materials or subsoil, e.g. through weathering or aging. There may be certain indications that the subsoil and dam materials have to be examined.

The Bigge Dam

The Reservoir and the Dam

The Bigge reservoir is located in the German midlands, 70 km to the east of Cologne. With a retaining capacity of 171.7hm³ it is Germany's fifth largest reservoir. Its main job is to ensure water supplies for 5 million people in the Ruhr Region and provide flood protection for the Ruhr. A power station with 5 MW output is an additional benefit.

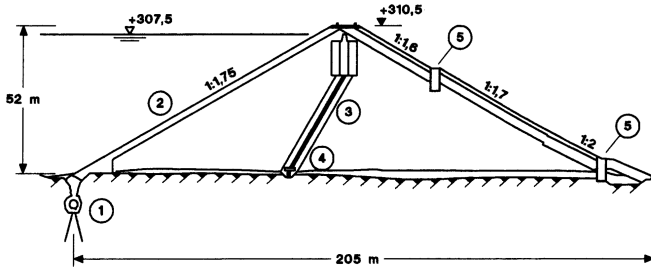


Fig. 1: Principal cross section

- 1 Inspection gallery
- 2 Upstream asphalt facing
- 3 Asphalt core
- 4 Drain pipes
- 5 Test pit

The rockfill embankment dam was built between 1957 and 1965 with an asphalt facing (Fig. 1). An asphalt core (called retarding zone) was installed in case the surface seal failed. Another particularity is that the dam stretches across two valleys, the Ihne valley and the Bigge valley, and a small ridge between the two (Fig. 2).

TABLE 1: TECHNICAL DATA FOR THE BIGGE DAM

Reservoir capacity	171.7 hm ³
Top water level	307.50 m amsl
Reservoir surface	8.76 km ²
Catchment area	287 km ²
Design flood discharge without retention	347 m ³ /s
Elevation number of the dam crest	310.5 m amsl
Crest length	636.17 m
Crest width	10 m
Crest height above foundation bottom	57 m
Greatest width at the dam toe	220 m
Slope ratio upstream side	1:1.75
Slope ratio downstream side	1:1.6 to 1:2.0

The documents available at the start of the examination contained hardly any data on the materials used in the Bigge dam and its properties. Practically no characteristics that are necessary to calculate the stability (unit weight, strength and deformation parameters, permeability, etc.) were available.

Some publications were discovered during research in the pertinent literature and numerous plans, documents and photos from the construction period following a close study of files in the archives of the Ruhrverband, though these too provided little further information that can be regarded as reliable.

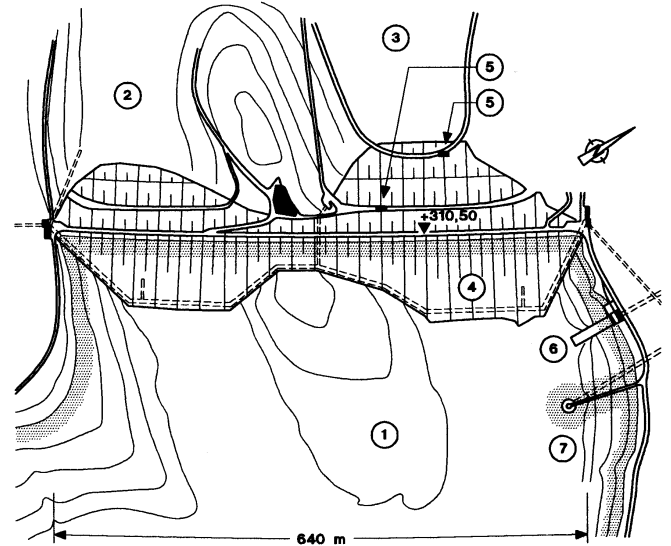


Fig. 2: Location of the dam with test pits

- 1 Reservoir
- 2 Ihne valley
- 3 Bigge valley
- 4 Upstream asphalt facing
- 5 Test pit
- 6 Bottom Outlet
- 7 Spillway tower

Subsoil, cut-off wall, inspection gallery and grouting curtain

Information on the subsoil of the Bigge dam was available from earlier evaluations of WD tests and grouting results. The subsoil consists of sandy slate and small sandstone banks as well as siltstone and sandy siltstone in the foot of the dam. The ground is heavily weathered and fissured on the surface with the sandstones displaying more joints than the slates. Weathering and jointing increases significantly the lower one goes so that the ground can be split into four zones depending on the depth with permeabilities of $k = 3 \cdot 10^{-5} / 3 \cdot 10^{-6} / 3 \cdot 10^{-7}$ and $1 \cdot 10^{-8}$ m/s.

The cut-off wall at the base of the upstream dam extends into the underground by up to 12m and connects to the inspection gallery of unreinforced concrete. Both are assumed to be watertight and are modelled with standard parameters from literature within the scope of the calculations. A grouting curtain has been built down to a depth of around 60m starting from the inspection gallery. The permeabilities of the grouting curtain were closely examined to obtain reliable characteristics:

$k = 0.8 \cdot 10^{-8}$ to $2.2 \cdot 10^{-8}$ m/s, on average $k = 1.35 \cdot 10^{-8}$ m/s.

Alluvial Clay and Stream Gravel

In 1959, the Institute of Soil Mechanics and Foundation Engineering of the Technical University Karlsruhe under the direction of Prof. Leussink drew up a “Soil mechanical expertise on the dam underground” [3]. The natural bedrock is covered to a large extent by a thin weathering layer of hillside debris or hillside clay; in the river valleys overlying layers of alluvial clay with stream gravel layers below this are predominant. The alluvial clay layer has an insufficient shearing strength and was removed at the start of the construction work.

For the stream gravel wet unit weights of 19.5 to 22.2kN/m³ (on average $\gamma_F = 21.1\text{kN/m}^3$) with water contents of between 5 to 15% were determined in tests, resulting in a mean dry unit weight of $g_d = 19.2\text{kN/m}^3$. The angle of friction was between 32° and 39° (on average: $\phi = 35^\circ$), there was no significant cohesion. The shearing strength of the stream gravel was considered to be sufficient after numerous calculations (e.g. shearing stresses, Renduliç method) so that the dam was founded on the stream gravel apart from the downstream toe and the asphalt core and transition zone (cf. Fig. 1).

Dam Fill and Filter Base

From an internal “Report on the geological investigations for the extraction of dam fill material” of the Ruhrverband dated January 19, 1961, it emerges that there were two basic requirements when choosing the extraction sites for the material: the possibility of removal from the later storage area and a maximum transportation distance of 2 - 3 km. Quote from the report: “This is why the demands on the quality of the material had to be largely reduced.”

In the end a decision was taken to use coarse rock material that was extracted from three quarries on the neighbouring hill “Gilberg” and was generally referred to as “Graywacke” in contemporary documents. The quarry material varied from firm, grey mostly coarse blocks with an edge length of up to 80cm, brown, easily broken chunks right down to heavily weathered material and very flat and schistous rock with frequent salvages and weathering zones (Devonian graywacke slate). High-grade fragmented rock material was mainly used for the drains on the downstream dam toe and the dam body, the majority of this was extracted from one of the three quarries (quarry B, cf. Fig. 3).

Reports were available on the placing and compacting tests with various built-in layer heights and compacting equipment that had been carried out at the beginning of the construction work. The optimum compaction was achieved with a heavy-duty crane vibration unit (Fig. 4) and a layer height of 1.20m; the dry unit weight in this case was around $\gamma_d = 20\text{kN/m}^3$.



Fig. 3: Quarry B at the hill Gilberg



Fig. 4: Compaction with a crane vibration unit

Surface Seal, Retarding Zone, Crest Securing Structure

The surface seal consists of a two-layer, continuous asphalt concrete layer. A drainage layer is integrated between the two layers of the surface seal that drains off any seepage water through seepage pipes in the inspection gallery. The lower end of the asphalt facing connects to the cut-off wall.

A 1m thick asphalt core in the middle of the dam forms the retarding zone. This consists of a mixture of hot bitumen and sand into which stones with an edge length of 25 to 35cm have been pressed. There are 2.5m thick supporting zones of fragmentary material on either side of this retarding zone. The retarding zone has a slope of under 60° on the upstream side and its base is embedded around 1m deep in the natural bedrock. Documents were available on the quality tests of the bitumen and on a simple in situ permeability test that resulted in a permeability of $k_F = 10^{-5}\text{m/s}$. The base of the retarding zone is interrupted by ten concrete pipes ($\varnothing 30\text{cm}$) each in the Ihne valley and Bigge valley whose job is to drain the upstream side of the dam if the water level drops suddenly so as to prevent any uplift of the surface seal. The pipes have been taken into account in the calculations as “blurred”.

The head of the retarding zone connects to a crest securing structure that reinforces the dam. This consists of 14.5m high and 24m long reinforced concrete walls with articulated joints that are anchored in the rock in the valley sides. The reinforced concrete has been assumed to be linear elastic with standard characteristics from the pertinent literature in the calculations.

Investigation of the Dam Materials

The aforementioned “Report on the geological investigations for the extraction of dam fill material” contained not only references to the properties of the investigated rocks but also a passage stating that “one has to reckon with a certain reduction of the shear strength over time.” Since no other information was available on the shear strength of the dam fill in the documents that were available, it was decided to take samples of the material from the dam during the “deepened examination” and to perform sieve analyses, large triaxial tests and water permeability tests with this material.



Fig. 5: Test pit on the downstream toe



Fig. 6: Material (depth 9.0m)

Extraction

The material was extracted in September 2003 in two test pits. Both explorations were in the area of the Bigge valley, one on the downstream dam toe, the other on the berm (Fig. 1

and 2). The test pits were excavated with hydraulic diggers and secured with the aid of a sliding rail support. The dimensions were around 4m x 6m; the excavation at the dam toe reached a depth of around 9m and at the berm around 7m. A total of around 46.6t of dam material was removed for further testing.

The main knowledge gained from the explorations can be summarised as follows [6]:

- no different material could be identified at the dam toe for the filter zone, the dam packing and the stream gravel; on the contrary, the material here is almost identical with the dam fill material at the berm.
- the natural bedrock was reached at the expected depth of around 9m.
- ground water was discovered in the area of the dam toe.
- the structure of the dam on the berm (topsoil - stony clay - coarse stone packing - dam fill) corresponds to that shown in the plans.

In situ Tests

A total of three in situ tests were carried out in the test pits using the water displacement method to determine the density (Fig. 7 and 8). The material was removed separately by hand, weighed and the water content determined.



Fig. 7: Preparing the in situ tests



Fig. 8: In situ density tests with the water displacement method

The unit weights and water contents were as follows:

Table 2: Unit Weights and Water Contents

	γ^F	w	γ^d
Dam toe 5.5m:	20.7kN/m ³	5.5%	19.6kN/m ³
Berm 5.6m:	21.2kN/m ³	4.9%	20.2kN/m ³
Berm 7.0m:	21.7kN/m ³	5.2%	20.6kN/m ³

The careful installation of the material with the density achieved in the placing and compacting tests of $\gamma_d = 20\text{kN/m}^3$ could thus be confirmed.

Large Scale Test on Rockfill Material

STRESS-STRAIN PROPERTIES AND SHEAR STRENGTH

In principle the behaviour of rockfill differs from that of granular soils and cannot be defined by empirical data. As experience shows, the mechanical behaviour of rockfill is predominantly controlled by the breakdown of the particles (rock fragments) and can show significant collapse under the first submersion with water. Thus, a term such as 'angle of internal friction' is misleading for the understanding of the resistance against shear of such a material. The hardness and packing of the particles, and the gradation and porosity of rockfill (the latter being influenced by compaction) are the most important factors governing the overall behaviour. In view of this, appropriate testing, using specimens which have been properly prepared in conditions similar to those of onsite handling is required, if realistic results and design parameters are to be obtained [9].

According to experience, the following aspects have to be taken into account in the testing procedure:

- The material must be carefully prepared according to a gradation which simulates the site conditions.
- The material is placed in a stiff and strong sample former and compacted in layers by static load to the desired dry density (according to the site conditions); special protection techniques must be used, to prevent damage to the rubber sleeve within the sample former (Fig. 9).
- Enlarged endplates with lubricated surfaces must be used, to allow for homogeneous deformation of short specimens ($H/D = 1/1$) throughout the test (Fig. 10).
- The axial and lateral strains must be carefully monitored and evaluated in the course of the test (Fig. 11), to obtain full information on the stress-strain relationship and to be able to control the testing procedure according to the reaction of the specimen.



Fig. 9: Sample in the sample former after compaction under static load, with the protective metal strips being removed one by one before mounting of the top plate

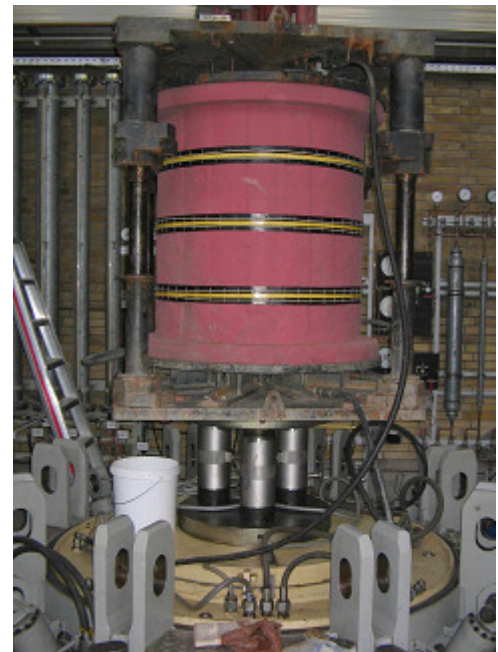


Fig. 10: Cylindrical specimen before triaxial testing in the large scale triaxial cell



Fig. 11: Large scale triaxial cell for tests on $H = D = 800$ mm specimens (cell pressure up to 20 bar)

Tests on rockfill material should not be carried out at constant strain rates. As the breakdown of the rock fragments takes time, there are certain creep effects which call for axial loading in steps and observation of the reaction of the material. Watching times for one load step may be as long as 60 minutes or even more, depending on the type of material, particularly at elevated cell pressures and higher shear stresses.

RESULTS OF LARGE SCALE TRIAXIAL TESTS

The material from the test pits was examined in the way described above at the Institute for Soil Mechanics and Rock Mechanics of the University Karlsruhe since this is the only institute in Germany that has a large triaxial unit [6]. The device had already been used for material tests when planning the Bigge dam in the 1960's and has been much improved in the meantime.

Large sieve analyses were carried out to determine the grain-size distribution which is shown for the material taken from the berm by way of example in Fig. 12. The material at the dam toe displays an almost identical grain-size distribution.

The main focus of the attention was on the performance of a total of three series of three triaxial tests on material from the berm in a dry and saturated state and from the dam toe in a saturated state. The sample dimensions were 800mm in diameter and 800mm in height. The maximum grain in the dam fill of $d > 150$ mm had to be removed for technical reasons and replaced by material with a $d = 100$ to 150mm. The specified lateral pressures in each series were $\sigma_3 = 0.1 / 0.3 / 0.6$ MPa.

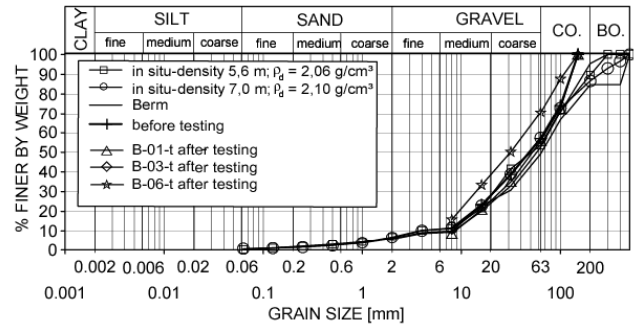


Fig.12: Grain size analysis of the material of the berm

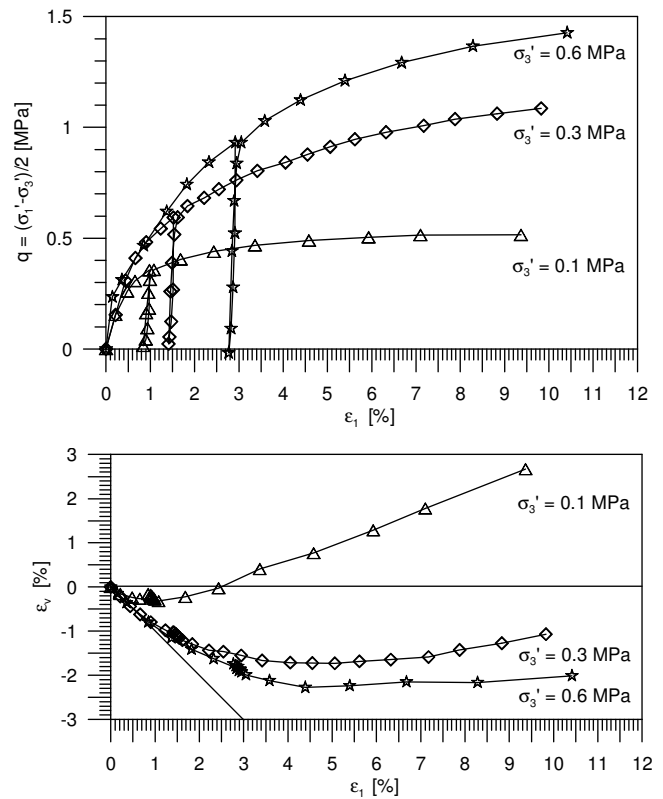


Fig. 13: Result of a series of triaxial tests on large specimens of material form the berm (tests in dry state)

Even under high dry densities of heavily precompacted rockfill (here: $\rho_d = 2.00$ t/m³) the material deforms as in one-dimensional compression ($\epsilon_v = \epsilon_1, \epsilon_3 = 0$) in the first phase of a triaxial test, and under continuous volumetric compression up to high axial strains, as is shown for the samples consisting of Bigge material from the berm (tests in dry state, cf. Fig. 13). Accordingly, no peak shear strength, not even a plateau value of strength, is observed here; instead the material consolidates under deformation and gains more and more strength (cf. Fig. 13).

Because of the special testing conditions in the triaxial cell, the specimens show homogeneous deformation. The inner structure of the specimen is shown in Fig. 15 compared to the structure of the shell material observed in the test pit (Fig. 14).

Fig. 16 shows the Mohr circles at the end of the test on the materials from the berm in a dry condition. It becomes clear that the inner angle of friction is affected by the level of stress. At a low pressure level the angle of friction is around 58° and drops to around 32° at a higher pressure level.

Large scale sieving analyses after testing allow to quantify the particle breakdown effects (Fig. 12).



Fig. 14: Structure of the rockfill inside the test pit of Bigge Dam



Fig. 15 View on inner section of a specimen after triaxial testing

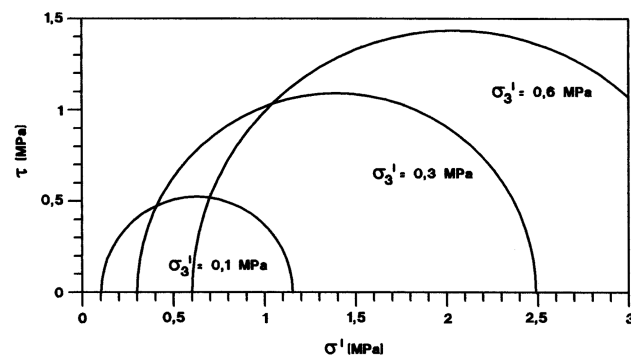


Fig. 16: Mohr circles for the material from the berm (dry state)

LARGE SCALE WATER PERMEABILITY TEST

The water permeability test was performed on a sample with 1200 mm in diameter and 1200 mm in height (Fig. 17). The maximum grain size used here was 200 mm. The rockfill material could only be compacted by means of a vibrating compactor.



Fig. 17: Water permeability test on rockfill (Diameter of specimen: $d = 1200$ mm)

For the test a flow through the specimen was realized in upward direction. The flow-rate was increased in stages and it was kept constant to reach stationary conditions in each stage. The resulting pressure profile inside the specimen was measured by means of piezometer tubes in different heights (Fig. 17).

As expected, during the tests a turbulent flow occurred at even small hydraulic gradients, so that the filter law according

to Darcy was no longer valid. The evaluation of the test was carried out according to the approach of Forchheimer, which considers the influence of non-linear effects of flow. The result is given in Fig. 18.

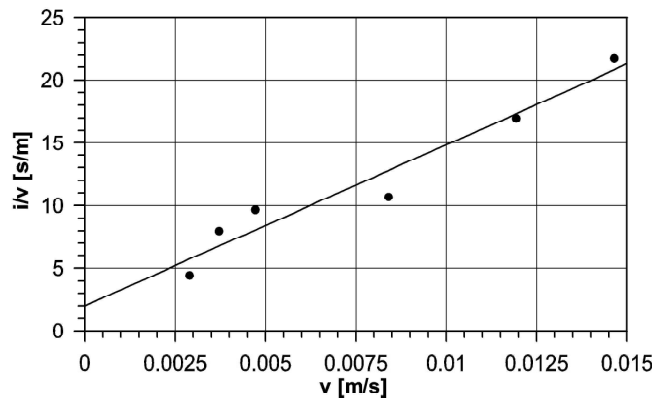


Fig. 18: Result of water permeability test (approach of Forchheimer)

A coefficient of permeability of $k = 0.5 \text{ m/s}$ was concluded for very small flow velocities. For the assessment of this value it has to be taken into account that the density achieved in the specimen might not be identical with the values on site.

Stability Analyses

Dams and their foundations must be stable and intact as a whole [1]. The interaction between the dam and foundation thus has to be taken into account. Load case catalogues divide the loads on a barrier into three groups:

- Group 1: constant of frequently recurring effects;
- Group 2: rare or temporary effects;
- Group 3: extraordinary effects.

The material properties of the dam and foundation are defined by parameters that describe the deformability, shear strength and permeability of the barrier and foundation as well as the efficiency of structural installations. These can normally only be quoted within ranges. Three bearing resistance conditions have to be taken into account depending on the extent of the ranges and the efficiency of the structural installations:

- Bearing resistance condition A (probable condition):
- for safe or generally recognised characteristics (either standardised or ascertained through test results or safely estimated from experience) and
 - fully effective structural installations;

- Bearing resistance condition B (unlikely conditions):
- for unfavourable characteristics within safe ranges or
 - with limited effect of one of the structural installations;

Bearing resistance condition C (improbable conditions):

- for unfavourable characteristics on thresholds or
- with a failure of one of the structural installations.

		Load case combinations					
		LC 1		LC 2		LC 3	
Group 1	Own weight	x	x	x	x	x	x
	Live load						
	Static water pressure						
Group 2	Static water pressure and seepage pressure at full water level (tailback)	x				x	x
	Snow pressure			x			
	Construction						
Group 3	Static water pressure and seepage pressure at highest water level (tailback) Z_{H1}						
	Extraordinary operating states					x	
	Safety earthquake						x

Fig. 11: Load case catalogues according to DIN 19700-11

The combination of the load cases and bearing resistance conditions result in three groups of dimensioning situations to be proven:

- DS I: constant dimensioning situation,
- DS II: temporary dimensioning situation,
- DS III: unusual dimensioning situation.

Table 2: Dimensioning situations

Load case	Bearing resistance conditions		
	A	B	C
1	DS I	DS II	DS III
2	DS II	DS III	-
3	DS III	-	-

The bearing safety must be proven for all decisive dimensioning situations and for all possible types of failure.

The stability analyses was carried out by finite element calculations, using the PLAXIS program system [4] for all dimensioning situations including extraordinary scenarios. Fig. 12 shows the distribution of shear stresses for the calculated case of a damaged surface seal with an assumed hole near the water level at a top water level as an example [8].

In conclusion, the stability of the Bigge Dam could be safely proven with the new examined characteristics of the material for all calculation situations.

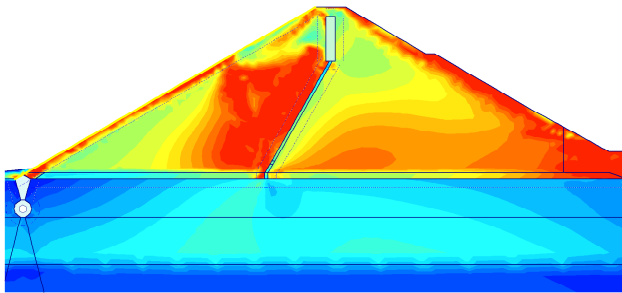


Fig. 12: Load case hole in the surface seal with top water level: shear stresses

Conclusion

After more than 40 years of operation, a deepened examination of the Bigge Dam was carried out according to the German DIN 19700 in its actual version of 2004 [1]. Due to lack of sufficient data comprehensive investigations on the material of the dam body were carried out. The required measures for sampling are described.

Subsequent large scale tests on rockfill material were performed in the laboratories of the Institute for Soil Mechanics and Rock Mechanics, University of Karlsruhe. The stability of the Bigge Dam could be safely proven with the new examined characteristics of the materials for all calculation situations.

It should be taken into account, that besides field testing of the handling and compaction of rockfill materials, which, depending on their nature, vary widely in behaviour, triaxial testing of these materials can provide important information on the parameters of both stress-strain and strength properties required in the design of embankments made of the kinds of materials tested here.

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