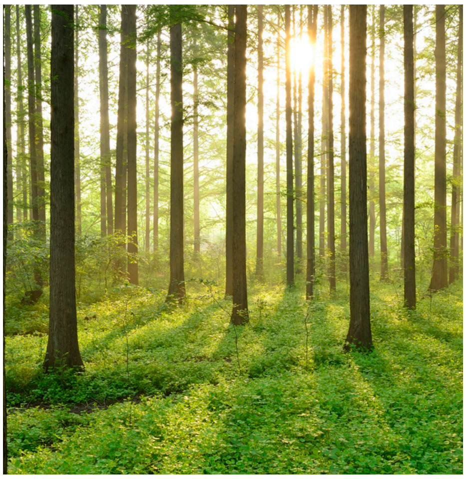
PROBABILISTIC MODEL CODE FOR CONCRETE DAMS

REPORT 2016:292



DAMMSÄKERHET

Energiforsk

Probabilistic model code for concrete dams

Part I, Part II, Part III and example

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Preface

In previous project within Energiforsk, needs for development related to stability analysis of concrete dams has been identified. This need is driven both by the level of knowledge today and by uncertainties in the analysis.

This report presents a methodology for probability-based assessments of concrete dams. It is based on the present knowledge and intended for reliability analysis of concrete dams.

As part of the background work calculations were carried out for a number of dams, with the two-fold purpose of i) testing the methodology and ii) defining an acceptable level of safety. The work behind the methodology and analysis of the performed calculations is described in report no 2016:291 (Energiforsk).

The work has been carried out by Marie Westberg Wilde, KTH/ÅF, and Fredrik Johansson, KTH/SWECO, with support from students and from a reference group with representatives from the industry and a group of experts.

The project has been a part of Energiforsks dam safety R&D program with participation from hydro power companies and Svenska kraftnät.

Stockholm September 2016

Sara Sandberg

Authors Preface

This document is intended for use to assess the safety of concrete dams. It applies to design of new concrete dams as well as to the assessment of the structural integrity of existing dams. It is limited to use for buttress and gravity type structures. The use is primarily intended for dam consequence class A and B facilities and larger/more important facilities in class C, but may be used for all facilities. The purpose is to give a basis that may be used for probability-based assessment of existing dams. Another purpose is to give a background that may be used for future development towards a partial-factor format.

This document describes a probability-based approach and follows the structure in the Probabilistic model code, issued by Joint Committee of Structural Safety in 2001 (JCSS, 2001). The Probabilistic model code by JCSS "is a first attempt to put together in a consistent way some of the rules, regulations, and explanations that are necessary for the design of new structures, or the assessment of existing ones from a probabilistic point of view". JCSS (2001) describes basis of design, loads and resistances for structural engineering.

In the same way, the probabilistic model code for concrete dams presented here is a first attempt to put together in a consistent way rules, regulations and explanations necessary for design and assessment of concrete dams from a probabilistic point of view. The Probabilistic Model Code for Concrete dams is divided into three parts: Part I: *Basis of design*, Part II: *Load models* and Part III: *Resistance models*. The first part of *Basis of design* is based on JCSS (2001), Eurocode 1990 (2001) and ISO 2394 (second edition from 1998 and draft for new version from 2013). The formulations below are mainly taken from Eurocode 1990, but similar formulations are also found in JCSS and ISO 2394. Therefore, relevant parts that are not included in Eurocode 1990 have been taken from these other documents. Where necessary, changes has been made to the formulations from the above documents, in order to be applicable for concrete dams and dam safety. Relevant information from JCSS (2001) and Eurocode 1990-1997 (2001) are included, but since neither of them includes dams, there is a significant lack of information and other references as well as our own work has therefore been used for input. Despite this, there are some areas where relevant information is still limited. The intention for this first version of the Probabilistic Model Code for Concrete Dams is to be updated when enabled by development and improvements in relevant areas.

To harmonize the design of concrete dam structures with design of other structures, the present document also describes where the information may be found in the Eurocodes. Note that only relevant sections from Eurocode are presented. Eurocode describes a partial factor format, but also allows for probability-based approach. The approach described in this document is probability-based and no partial factors are given.

Funding for the present work has been raised by dam owners through Elforsk/Energiforsk.

Marie Westberg Wilde & Fredrik Johansson

Stockholm 2016

Sammanfattning

Probabilistic model code for concrete dams som visas i detta dokument är ett första försök att på ett konsistent sätt sammanställa regler, riktlinjer och förklaringar för dimensionering och utvärdering av betongdammar utgående från ett sannolikhetsbaserat synsätt. Avsikten är att detta dokument skall vara till hjälp att på ett systematiskt vis genomföra sannolikhetsbaserade utvärderingar av betongdammar.

Probabilistic model code for concrete dams är indelad I tre delar: Del I: *Dimensioneringsförutsättningar*, Del II: *Lastmodeller* och Del III: *Bärförmågemodeller*. Dokumentet innehåller också ett exempel på en sannolikhetsbaserad utvärdering.

Dimensioneringsförutsättningar baseras på JCSS (2001), SS-EN 1990 och ISO 2394, med visa ändringar för att göra den bättre anpassad till betongdammar och dammsäkerhet. Denna del innehåller generella principer, information om hur sannolikhetsbaserad verifiering utförs; gränstillstånd, dimensioneringssituationer och tillförlitlighetsnivåer som är relevanta för betongdammar. Det innehåller även en del om uppdatering av parameterskattningar. Tillförlitlighetsnivåerna är likartade som de i SS-EN 1990, men de är baserade på kalibrering mot existerande praxis. Detta finns beskrivet i detalj i Westberg Wilde & Johansson (2016).

Del II Lastmodeller innehåller allmän beskrivning av laster och lastmodeller, baserat på JCSS (2001). Sedan görs en genomgång av laster som är relevanta för betongdammar och statistiska fördelningar baserade på ”bästa skattningar” presenteras. Del II inkluderar islast, hydrostatiskt tryck, upptryck och jordtryck.

Del III Bärförmågemodeller innehåller allmän beskrivning av bärförmåga och bärförmågemodeller, baserat på JCSS (2001). Sedan görs en genomgång av bärförmågeparametrar som är relevanta för betongdammar och statistiska fördelningar baserade på ”bästa skattningar” presenteras. Del III innehåller egentyngd, friktion i kontakten mellan berg och betong samt i berg, materialparametrar (betong, berg, stål), bergbultar och bergförankringar.

Slutligen ges ett exempel. I exemplet utförs en sannolikhetsbaserad värdering av en gravitationsdamm, baserat på Probabilistic model code for concrete dams.

Processen för framtagandet av detta dokument och kalibreringsprocessen finns beskriven i Westberg Wilde & Johansson (2016).

Summary

The *Probabilistic model code for concrete dams* presented in this document is a first attempt to put together in a consistent way rules, regulations and explanations necessary for design and assessment of concrete dams from a probabilistic point of view. It is believed that this document will be helpful to perform probabilistic assessments of concrete dams in a systematic way.

The Probabilistic Model Code for Concrete dams is divided into three parts: Part I: *Basis of design*, Part II: *Load models* and Part III: *Resistance models*. The document also contains one example of a probabilistic assessment.

Basis of design is based on JCSS (2001), Eurocode 1990 (2001) and ISO 2394 , with some changes has been in order to be applicable for concrete dams and dam safety. Basis of design contains general principles, information of how a probabilistic verification is performed; limit states and design situations, limit state functions and target reliabilities relevant for concrete dams. It also contains a part on updating of prior estimates. The target reliabilities applied are similar to those described in e.g. SS-EN 1990, but are based on calibration of the existing practice. This is further described in Westberg Wilde & Johansson (2016).

In *Part II Load models* general descriptions on loads and load modelling are given, based on JCSS (2001). Next relevant loads for concrete dams are discussed and "best estimates" on statistical descriptions are presented. Part II includes ice loads, hydrostatic pressure, uplift and earth pressure.

In *Part III Resistance models* general descriptions on resistance and resistance modelling is given, based on JCSS (2001). Next relevant resistance parameters for concrete dams are discussed and "best estimates" on statistical descriptions are presented. The resistance parameters included are self weight, friction properties of concrete/rock contact and in rock, material properties (concrete, rock, steel), rock bolts and rock anchors.

Finally an example is given. In the example a probabilistic analysis of a concrete gravity dam is performed, based on the Probabilistic model code.

The process of bringing forth this document and the calibration procedure is further described in Westberg Wilde & Johansson (2016).

List of content

Symbols

<u>Roman letters</u>

A	Area
A_c	Area bonded concrete-rock interface
a	Horizontal length of crushed zone for adjusted overturning / . parameter for trapezoidal distribution
a_1	Constant in beta distribution describing uplift
a_2	Constant in beta distribution describing uplift
Beta	Beta distribution
b	Parameter for trapezoidal distribution
C	Random variable for uplift force
C_m	Random variable for uplift moment
c	Parameter for trapezoidal distribution
c'	Internal cohesion for soils under drained . conditions
c_c	Cohesion, concrete rock-interface, bonded contact
c_u	Undrained shear strength for soils
d_e	Random variable of water depth exceeding rwl
$E(*)$	Expected value of *
E_d	Drain efficiency
$F*$	Action of *
F_0*	Basic action variable of *
$F(*)$	Cumulative distribution function of *
f_{cc}	Uniaxial compressive strength of concrete
f_{ck}	Characteristic value of the uniaxial compressive strength of . concrete
f_{cm}	Mean value of the uniaxial compressive strength of concrete
$f_{cm,is}$	Mean value of the uniaxial compressive strength of concrete . in-situ after 28 days
$f_{c,rock\ mass}$	Uniaxial compressive strength of the rock mass
f_{ctm}	Mean tensile strength in bending for concrete
f_y	Yield strength of steel
f_{yk}	Characteristic yield strength of steel
G_x	Limit state of failure mode x
G_w	Self weight
H	Sum of forces parallel the sliding plane or reservoir water . level
h	tailwater level
h_{de}	Water depth above retention water level
h_{drains}	Uplift pressure at location of drains
h_I	Thickness of ice
h_w	Water depth
h_{rwl}	Water depth at retention water level
I	Ice load
I_m	Maximum ice load
i_c	Contribution from roughness, concrete-rock interface
i_F	Contribution from roughness, rock fracture
K_A	Coefficient for active earth pressure

K_0	Coefficient for earth pressure at rest
K_d	Constant for drain efficiency
LN	Log-normal distribution
L	Length of dam in flow direction
M_R	Resisting moments
M_S	Driving moments
M_{tp}	Moment around center of gravity
N	Normal distribution
N'	Effective normal load
n	Number of test/number of realizations
n_n	Return period of flow n
n_{rwl}	Return period for retention water level
P	Probability
P_f	Probability of failure
P^t_f	Target probability of failure
P(x,t)	Pre-stressing force in rock anchors
ΔP(x,t)	Losses of pre-stressing force in rock anchors
P_0	Jacking force in rock anchors
Q_n	Flow for return period n
q	Distributed load
S_x	Resulting force from earth pressure of state x (At rest, active . or passive)
s	Coefficient in ageing model for concrete
$T_{R.x}$	Shear resistance of sliding plane with contact of type x (rock . fracture, interface bonded or unbonded)
t	time in days
U	Uplift force
U_c	Force from full uplift pressure under area with tensile stresses
U_d	Resultant uplift force
U_{cm}	Moment from full uplift pressure under area with tensile . stresses
U_{dm}	Resultant uplift moment
V	Volume
V_x	Coefficient of variation of x
$Var(x)$	Variance of x
W	Modulus of bending
x_L	Location along dam in flow direction

<u>Greek letters</u>

Γ_x	Variance reduction of x
β	Safety index
β_{cc}	Factor for the ageing model of concrete
β_{crack}	Target safety index with respect to tensile stresses at dam heel
β_n	Safety index for a reference period of n years
β_1	Safety index for a reference period of 1 years
β_T	Target safety index
γ	Unit weight
γ'	Saturated unit weight
γ_w	Unit weight water

μ	Mean value
μ'	á priori mean value
μ''	á posteriori mean value
ϕ'	Internal friction angle for soils under drained . conditions
$\phi_{b,c}$	Basic friction angle concrete rock-interface
$\phi_{b,F}$	Basic friction angle rock fracture
$\phi_{i,c}$	Internal friction angle concrete-rock interface, bonded contact
$\phi_{tot,c}$	Total friction angle concrete rock interface
$\phi_{tot,F}$	Total friction angle rock fracture
Φ	Standard normal cumulative distribution function
ρ	Correlation coefficient
ρ_x	Density of material x
σ_x	Standard deviation of parameter x
φ	Action function
φ'	Internal friction angle for friction soil under drained conditions

1 Introduction

The Probabilistic model code for concrete dams is divided into three parts:

Part I : Basis of design

Part II: Load models

Part III: Resistance models

This model code treats the principles for probabilistic design and assessment of concrete dams. As a model code it is a background document for writing design recommendations. Updates of the model code should be done continuously. The authors take no responsibility to the use and interpretation of this document.

Principles and basis of probability based design is described in EN 1990 (2001), JCSS (2001) and ISO 2395 (2015). In Part I relevant information is taken from these documents and reference is given to the right of the text. Principles relevant for concrete dams are referred to as PMCD (Probabilistic model code of concrete dams). Methods for probability based design are described in the above documents. However, only a short section concerning the theory behind probabilistic design is included in section 5 and 7 of Part I of this model code. For a more comprehensive description, the reader is referred to textbooks such as Ang & Tang (1975), Melchers (1999), Thoft-Christiensen & Baker (1982) among others.

The work behind this model code as well as the calibration process behind the target values included is described in Westberg Wilde & Johansson (2016).

2 Limitations

The methodology described here is intended for reliability analysis of concrete dams. It does not describe a full risk analysis, although parts of it may be used for a full risk analysis.

The documents of this probabilistic model code for concrete dams are written based on the present knowledge. Changes will be inevitable as more knowledge or additional information is gained.

Target values are calibrated based on assumptions in this document and changes in these assumptions may affect the target value.

3 Definitions

Basic variables
Variables representing physical quantities which characterize actions and environmental influences, material and soil properties and geometrical quantities.

FORM/SORM (First/Second Order Reliability Methods)
The numerical methods used for determination of the reliability index β.

Limit state
A state beyond which a structure no longer satisfies the specified design criteria.

Limit state function
A function $g\left(X_1, X_2, ..., X_2\right)$ of the basic variables, which characterizes a limit state $g\left(X_1, X_2, ..., X_2\right) = 0$.

Partial factor see semi probabilistic methods

Probabilistic methods
Verification methods in which the relevant basic variables are treated as random variables, random processes and random fields, discrete or continuous.

Reliability
The ability of a structure or structural element to fulfil specified requirements including the working life for which it has been designed. Reliability is often expressed in terms of probability.

Reference period

The period of time used as a basis for assessing the design value of variable and/or accidental actions.

Reliability index *β*
A substitute for the failure probability, $\beta = -\Phi^{-1}(P_f)$ where Φ^{-1} is the inverse standardized normal distribution.

Risk
An undesired event that represents a danger for humans, environment or properties. In general, it is a function of the probability and consequences.

Robustness
Robustness is an inherent property of systems that enables them to survive unforeseen or unusual events without excessive damage or loss of function (Eurocode 2001). It has also been defined as "ability of the structure to withstand local damage without disproportionate collapse" (Val & Val 2006).

Semi probabilistic or partial factor methods
Verification method in which allowance is made for the uncertainties and variability assigned to the basic variables by means of representative values, partial factors and, if relevant, additive quantities.

Serviceability
The ability of a structure or structural element to perform adequately for a normal use under all expected actions.

Serviceability limit state
A limit state concerning the criteria governing the function related to normal use.

Structural safety
Ability (of a structure or structural element) to avoid exceedance of ultimate limit states including the effects of specified accidental phenomena with a specified level of reliability, during its construction and anticipated use.

Ultimate limit state
Limit states associated with collapse or with other similar forms of structural failure

Part I: Basis of Design

I:1 Reliability

Different levels of reliability may be adopted: EN 1990 2.2

- for structural resistance ;
- for serviceability.

The choice of the levels of reliability for a particular structure should take account of the relevant factors, including :

- the possible cause and /or mode of attaining a limit state ;
- the possible consequences of failure in terms of risk to life, injury, and potential economic losses ;
- public aversion to failure ;
- the expense and procedures necessary to reduce the risk of failure.

The levels of reliability that apply to a particular structure may be specified according to:

- the classification of the structure as a whole or;
- the classification of its components.

The levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of :

a) preventative and protective measures (e.g. implementation of safety barriers, active and passive protective measures against fire, protection against risks of corrosion such as painting or cathodic protection) ;
b) measures relating to design calculations :
c) measures relating to quality management ;
d) measures aimed to reduce errors in design and execution of the structure, and gross human errors ;
e) other measures relating to the following other design matters :

- the basic requirements ;
- the degree of robustness (structural integrity) ;
- durability, including the choice of the design working life ;
- the extent and quality of preliminary investigations of soils and possible environmental influences ;
- the accuracy of the mechanical models used ;
- the detailing ;

f) efficient execution, *e.g.* in accordance with execution standards referred to in EN 1991 to EN 1999.

g) adequate inspection and maintenance according to procedures specified in the project documentation.

The measures to prevent potential causes of failure and/or reduce their consequences may, in appropriate circumstances, be interchanged to a limited extent provided that the required reliability levels are maintained.

I:1.1 DESIGN WORKING LIFE

The design working life should be specified. EN 1990 2.3

Table PI - 1-1. Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common
5	100	Monumental building structures, bridges, and

PMCD Note: Dams are usually related to category 5. PMCD

I:1.2 DURABILITY

The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance. EN 1990 2.4

In order to achieve an adequately durable structure several factors should be taken into account:

- the intended or foreseeable use of the structure ;
- the required design criteria ;
- the expected environmental conditions and other loads reducing durability ;
- the composition, properties and performance of the materials and products ;
- the properties of the soil ;
- the choice of the structural system ;
- the shape of members and the structural detailing ;
- the quality of workmanship, and the level of control ;
- the particular protective measures ;
- the intended maintenance during the design working life.

I:1.3 APPROACHES TO DESIGN AND ASSESSMENT

In order to verify whether the structure is in compliance with the objectives for all design/assessment situations, one of the following levels shall be chosen: ISO 2394, relevant parts

1. Risk based: it shall be proven that the sum of all costs (building costs, maintenance etc.) and risks (with respect to failure or malfunctioning) is at a minimum; additional constraints with respect to human safety shall be considered in consistency public law and codes (more information in clause 4 of ISO 2394).
2. Reliability based: the structure shall fulfil a set of reliability requirements formulated as maximum admissible failure probabilities or minimum values for the reliability levels.
3. Semi probabilistic: the structure shall fulfil a set of inequalities using certain design values of the basic variables.

Lower levels of verification shall be calibrated to the higher levels using code calibration principles (Note: Usually this calibration is performed by code committees allowing the designer to use semi-probabilistic verification methods; only for special structures a reliability or risk based verification will be performed.)

Design and assessment decisions shall take basis in information concerning their implied risks. When the consequences of failure and damage are well understood reliability based assessments can be applied instead of full risk assessments. Semi-probabilistic approaches as a further simplification are only appropriate when in addition to the consequences also the failure modes and the uncertainty representation may be categorized and standardized.

In risk informed design and assessment the decisions shall be optimized with due consideration of the total risks […]. Assessment of the total risk shall take basis in a scenario representation and by probabilistic models of the exposures, the constituent damage and failure events as well as the direct and indirect consequences, see section I:1.3.1.

For structures where failure and damage may imply very serious consequences a risk based robustness assessment is recommended to be undertaken as part of the design and/ or assessment verification.

I:1.3.1 Risk based robustness

According to ISO 2394 design of structures shall be supported by risk based robustness assessments and/or by consideration of robustness provisions in dependence of the exposures acting on the structure, the structural system and the consequences of system failure. Annex F of ISO 2394 gives a framework for such assessment. The following is a summary of Annex F. ISO 2394, Annex F

Structures are classified according to their consequences into 5 classes, where expected consequences for class 1 are "*insignificant material damages*", for class 3 are "*material losses and functionality losses of societal significance, causing regional disruptions and delays in important societal services over several weeks… number of fatalities less than 50*" and for class 5 are "*catastrophic events causing losses of societal services and disruptions and delays beyond national scale […] significant damages to the environment […], number of fatalities larger than 500*".

For low dam consequences (class 1-2) (corresponds roughly to dam consequence U and C) no specific consideration regarding robustness has to be done, but depending on specific circumstances may be performed. PMCD

For high dam consequence classes (3-5) (corresponds roughly to dam consequence class B and A) robustness assessment should be done. For class 3 this involves a systematic identification of scenarios leading to structural collapse, where prescriptive design and detailing rules may be utilized and reliability and risk analyses addressing direct and indirect consequences should be used as basis for simplifications and idealizations.

For class 4 an extensive study and analysis of scenarios leading to structural collapse should be done, utilizing risk screening meetings and involving experts on all relevant subject matters. Detailed assessments must be undertaken using dynamic and non-linear structural analyses and risk analyses rigorously addressing direct and indirect consequences. ISO 2394, Annex F

For class 5 the procedure for class 4 should be followed and an external expert/review panel should be involved for quality control.

In formal risk assessment carried out for the purpose of decision-making, a scenario approach can be used as defined by the three steps given below

Step 1: the modelling of the hazards (exposure)

Step 2: the assessment of the direct damage (often local)

Step 3: the assessment of follow-up structural behaviour and corresponding total consequences.

For a given system exposed to a hazard, the elements of the system can be considered as its first defence against a hazard. The damage to the system caused by failures of the components is considered as "direct consequences" (may be e.g. monetary losses, loss of lives, and damage to the environment….). Depending on the combination of events of element failure and the corresponding consequences, follow-up ("indirect") consequences may occur. If the structure is robust, these follow-up consequences, or the probability thereof, will be small. The opposite is true when a structure is not robust.

I:1.4 DOCUMENTATION

Decisions related to the design of structures, as well as their verification with respect to acceptance criteria, shall be documented in a manner that is tractable and transparent for all involved stakeholders. This concerns design of individual structures as well as development and calibration of design codes. ISO 2394

The documentation shall include all relevant information utilized for the design of the structures, including site specific data, test results, models of the performance indicators, inspection results, information regarding damages as well as maintenance and repairs, acceptance criteria and their verifications, quality control schemes and results etc.

In addition, all relevant assumptions shall be identified, discussed with respect to their significance for the reliability of the structure, and documented. This also includes assumptions concerning the use of the structures, envisaged maintenance as well as possible requirements for performance specified by the owner of the structure.

For structures for which the consequences of failure and damage are high, i.e. Consequence Class 3-5 defined in section I:1.3.1, a Structural Certificate must be issued. It is the responsibility of the owner that this certificate is established, safely kept and regularly updated.

- Owner specified requirements to the geometry, materials, use and performance of the structure
- References to the documentation for the design and construction of the structure, whether based on risk, reliability or semi-probabilistic approaches.
- Documentation of assumptions with respect to strategies and procedures for condition control, inspection, maintenance and repair.
- Documentation of the quality control undertaken concerning materials production, design and construction.
- Documentation of the structure “as-built” – commissioning - together with an assessment of possible nonconformities and how these have been treated.
- Documentation on performed condition control, inspections and maintenance as well as repairs and other modifications.
- A documentation of emergency action plans and other loss reduction activities for relevant types of accidents and incidents.

RIDAS recommendations comply with the above recommendations concerning Structural Certificate through the DTU-manual. PMCD

I:2 Principles of limit state design

I:2.1 PERFORMANCE AND LIMIT STATE CONCEPT

In order to assess the performance of a structure, the response space shall be divided into two domains consisting of desirable and undesirable states. The boundary between these domains is called the limit state and entering the undesirable domain is defined as failure. The limit state function is denoted G(**x**) where x are all basic variables (e.g. x_1 may be the load and x_2 the resistance). The limit state occur when ISO 2394

$$G(\mathbf{x}) = 0 \qquad \text{(PI. 2-1)}$$

The below figure show a schematic picture of the limit state PMCD

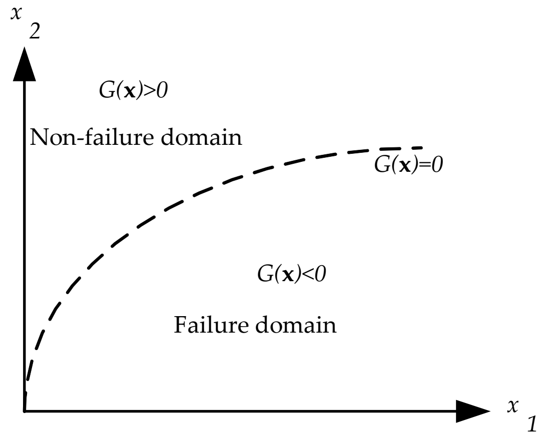

Figure PI- 2-1. Limit state, non-failure domain and failure domain.

I:2.2 GENERAL

A distinction shall be made between ultimate limit states and serviceability limit states. EN 1990 3.1

Verification of one of the two categories of limit states may be omitted provided that sufficient information is available to prove that it is satisfied by the other.

Limit states shall be related to design situations, see I:2.3

PMCD Notes: Design situation for concrete dams, see I:7.2.4. PMCD

This document deals primarily with ultimate limit state for stability analysis of concrete dams.. Later versions may include also serviceability limit states.

In this code, design is based directly on reliability-based methods. More detailed information of probabilistic methods may be found in section I:5.

I:2.3 DESIGN SITUATIONS

The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfil its function. EN 1990 3.2

Design situations shall be classified as follows :

- persistent design situations, which refer to the conditions of normal use ;
- transient design situations, which refer to temporary conditions applicable to the structure, e.g. during execution or repair ;
- accidental design situations, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact or the consequences of localized failure, surcharge in flood situations (the latter not included in EN) ;
- seismic design situations, which refer to conditions applicable to the structure when subjected to seismic events.

The selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure.

I:2.4 ULTIMATE LIMIT STATES

The limit states that concern : EN 1990 3.3

- the safety of people, and/or
- the collapse of the structure

shall be classified as ultimate limit states.

States prior to structural collapse, which, for simplicity, are considered in place of the collapse itself, may be treated as ultimate limit states.

I:2.5 SERVICEABILITY LIMIT STATES

The limit states that concern : EN 1990 3.4

- the functioning of the structure or structural members under normal use ;
- the comfort of people ;
- the appearance of the construction works,
- excessive maintenance

shall be classified as serviceability limit states.

A distinction shall be made between reversible and irreversible serviceability limit states.

I:2.6 LIMIT STATE DESIGN

Design for limit states shall be based on the use of structural and load models for relevant limit states. EN 1990 3.5

For each specific limit state the relevant basic variables should be identified, i.e. the variables which characterize: JCSS 3.2

- actions and environmental influences
- properties of materials and soils
- geometrical parameters

Such variables may be time dependent. Models, which describe the behaviour of a structure, should be established for each limit state. These models include mechanical models, which describe the structural behaviour, as well as other physical or chemical models, which describe the effects of environmental influences on the material properties. The parameters of such models should in principle be treated in the same way as basic variables.

In a component analysis where there is one dominating failure mode the limit state condition can normally be described by one equation according to eq. (2-1). In a system analysis, where more than one failure mode may be determining, there are several such equations.

I:3 Basis of uncertainty modelling

I:3.1 BASIC VARIABLES

The calculation model for each limit state considered should contain a specified set of basic variables, i.e. physical quantities which characterize actions and environmental influences, material and soil properties and geometrical quantities. The model should also contain model parameters which characterize the model itself and which are treated as basic variables. Finally there are also parameters which describe the requirements or functional limits (e.g. serviceability constraints) and which may be treated as basic variables. The basic variables (in the wide sense given above) are assumed to carry the entire input information to the calculation model. JCSS 4.1

The basic variables may be random variables (including the special case deterministic variables) or stochastic processes or random fields. Each basic variable is defined by a number of parameters such as mean, standard deviation, parameters determining the correlation structure etc.

I:3.2 TYPES OF UNCERTAINTY

Basic variables are usually subject to uncertainty. These uncertain basic variables represent physical uncertainties, statistical uncertainties, measurement uncertainties, uncertainties such as related to the precision of new information and model uncertainties. All main sources of uncertainty shall be identified. ISO 2394 6.1

The physical uncertainties are typically uncertainties associated with the loading environment, the geometry of the structure and the material properties and are often referred to as aleatory uncertainties.

Uncertainties arising from insufficient information e.g. due to a small number of materials tests or idealized models are often referred to as epistemic.

A random variable may represent both aleatory and epistemic uncertainties. Furthermore, uncertainty may change nature in different phases of the life-time of a structure. For example, material strength is considered as aleatory uncertainty before the construction of a structure; once the structure is constructed, it may be considered as epistemic uncertainty.

PMCD Note: In this document both types of uncertainty is treated by the Bayesian approach. Further discussion of this may be found in section I:8. PMCD

I:4 Models for physical behaviour

Summary of JCSS 5.

Models should generally be regarded as simplifications which take account of decisive factors and neglect the less important ones. It is often possible and convenient to distinguish between

- action models
- structural models which give action effects (internal forces, moments etc.)
- resistance models which give resistances corresponding to the action effects, and are based on
- material models and geometry models .

A complete *action model* should describe several properties of the action such as its magnitude, position, direction, duration etc. In some cases there is an interaction between the different properties and also between these properties and the response of the structure. Such interactions should be taken into account.

The geometrical quantities which are included in the model generally refer to nominal values, i.e. the values given in drawings, descriptions, etc. Normally the geometrical quantities of a real structure differ from their nominal value due to geometrical imperfections. These shall be included in the model. Effects of deformations that cause significant deviations from nominal values and are of importance for the structural behaviour should be accounted for.

Material models consider relations between forces or stresses and deformations, i.e. constitutive relationships. The parameters of such relations are generally considered as random variables, sometimes time and/or space dependent and often correlated (e.g. modulus of elasticity and ultimate strength of concrete).

The following mechanical models may be classified

- models describing static response
- models describing dynamic response
- models for fatigue;

they may also affect each other (e.g. fatigue affecting static response).

I:5 Reliability

A reliability based decision implies that the probability of failure P_f does not exceed a specified target P_f^t for a given reference period:

ISO 2394,

EN 1990 C6

$$P_f \leq P_f^t \quad \text{(PI. 5-1)}$$

Failure is associated with a limit state. The undesired limit state is defined by

$$g(\mathbf{x}) \leq 0 \quad \text{(PI. 5-2)}$$

where **x** is a vector containing the realizations of the basic random variables **X** which are relevant to the problem. When $g(\mathbf{x}) > 0$ the structure is considered to survive.

For most ultimate limit states, and for some serviceability limit states, the probability of failure can be written

$$P_f = P\left[g(\mathbf{x}) \leq 0\right] \quad \text{(PI. 5-3)}$$

Due to the dependence upon time, P_f shall be referred to a certain a priori specified period of time, the reference period.

Probability procedures for reliability calculations are divided into two levels

- full probabilistic methods (Level III), and
- first order reliability methods (FORM) (Level II).

Full probabilistic methods (Level III) give, in principle, correct answers to the reliability problem as stated. Level III methods are seldom used in the calibration of design codes and practical design because of the frequent lack of statistical data.

The level II methods make use of certain well defined approximations and lead to results which for most structural applications can be considered sufficiently accurate. In the Level II procedures, an alternative measure of reliability is conventionally defined by the reliability index β which is related to P_f by :

$$P_f = \Phi(-\beta) \quad \text{(PI. 5-4)}$$

Φ is the standard normal cumulative distribution function. The relation between β and P_f is given in Table PI-5-1.

Table PI - 5-1. Relation between β and P_f

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,28	2,32	3,09	3,72	4,27	4,75	5,20

If *R* is the resistance and *E* the effect of actions, the limit state function *g* is :

$$g = R - E \qquad \text{(PI. 5-5)}$$

with *R* and *E* as random variables.

If *R* and *E* are normally distributed, β is taken as :

$$\beta = \frac{\mu_g}{\sigma_g} \qquad \text{(PI. 5-6)}$$

where μ_g and σ_g are the mean value and the standard deviation of *g*, respectively.

PMCD Note: Reliability analysis principles including time-dependent reliability problems are described in JCSS (2001) Annex C, PROVERBS (1999), Melchers (1999), Thoft-Christenssen & Baker (1982). PMCD

I:5.1 COMPONENT RELIABILITY AND SYSTEM RELIABILITY

Component reliability is the reliability of one single structural component which has one dominating failure mode. For concrete dam stability, component reliability refers to the reliability of one monolith for one failure mode. Each monolith may have several different failure modes. JCSS 6.2

System reliability is the reliability of a structural system composed of a number of components or the reliability of a single component which has several failure modes of nearly equal importance. The following type of systems can be classified:

- *redundant systems* where the components are "fail safe", i.e. local failure of one component does not directly result in failure of the structure. Redundant systems may be modelled as parallel systems where all or at least many components have to fail, in order for a system failure to occur;
- *non-redundant* systems where local failure of one component leads rapidly to failure of the structure. Non-redundant systems may be modelled by series systems, where failure of one component leads to complete failure.

There are also combined systems where some load re-distribution is possible, e.g. brittle parallel systems.

ISO 2394

System behaviour is of concern because a system failure is usually the most serious consequence associated with failure of a structure. It is therefore of interest to assess the probability of system failure following an initial element failure. Element reliability requirements should depend upon the systems characteristics. See also clause I:1.3.1 concerning robustness assessment.

Concrete dams typically consist of several monoliths, each with several failure modes. Concrete dam stability may in general be regarded as failure of a non-redundant system and hence failure modes and monoliths has to be analysed as a series system. PMCD

I:5.2 METHODS FOR RELIABILITY ANALYSIS AND CALCULATION

JCSS 6.3

I:5.2.1 General

The numerical value of the reliability measure is obtained by a reliability analysis and calculation method (see previously recommended literature).

The reliability method used should be capable of producing a sensitivity analysis including importance factors for uncertain parameters. The choice of the method should be justified in general. The justification can for example be based on another relevant computation method or by reference to appropriate literature.

Two fundamental accuracy requirements are:

- Overestimation of the reliability due to use of an approximate calculation method shall be within limits generally accepted for the specific type of structure.
- The overestimation of the reliability index should not exceed 5 % with respect to the target level.

The accuracy of the reliability calculation method is linked to the sensitivity with respect to structural dimensions and material properties in the resulting design.

I:5.2.2 Methods for use

Verification by FORM (First Order Reliability Method) should be used where possible. For β close to zero, FORM-calculation should be validated by SORM (Second Order Reliability Method) or Monte Carlo simulations. For discontinuous limit state functions (as present in most systems) Monte Carlo simulations or other simulation methodology may be used. PMCD

I:6 Target reliability

The requirement to safety of the structure is the accepted minimum reliability index (or accepted maximum failure probability). The accepted minimum reliability index is denoted the target reliability or target safety index. PMCD

Note that there are also possibilities to apply target values from Eurocodes or JCSS. Since target values are nominal and dependant on assumptions made in the analysis it is recommended to use the below target values. Changes to the probabilistic model code for concrete dams may necessitate re-calibration and a new target safety index.

According to JCSS it is also possible to incorporate cost of safety measures, degree of uncertainty, quality assurance and inspections and existing structures when a target value is defined. This is theoretically preferable, but not necessarily practical and politically preferable. It has not been applied in the below recommendations, but the possibility should be noted.

I:6.1 COMPONENT AND SYSTEM RELIABILITY

Target reliabilities given in this section refers to component reliability. A component may refer to a physical member as well as to a single failure mode. PMCD

For large systems (e.g. when a large number of monoliths are present), a higher safety class may have to be chosen. This has to be defined case specific, preferably after risk analysis.

According to the limit states defined in I:7.2.3, each failure mode for a dam monolith is analysed separately. For high consequence dams, and for dams consisting of a large number of monoliths, this may not be sufficient. In that case the probability-based assessment may be used as input to a quantitative risk analysis, where also correlation between different failure modes and load cases are included. No further discussion concerning this is given, but systems reliability is treated in e.g. Westberg & Johansson (2013). JCSS 7.2.1

I:6.2 CONSEQUENCES CLASSES

Reliability differentiation may be established by considering dam consequences classes according to Miljöbalken (1998:808, SFS 2014:114), see Table PI-6-1.

Miljöbalken, 1998:808 and SFS 2014:114

Table PI - 6-1. Dam consequence classes according to Miljöbalken (free translation).

Dam consequence class	Consequences	
A	May cause loss of many human lives	Failure may lead to a crisis affecting many people and large parts of the society and threaten fundamental values and functions.
B	May cause loss of human lives	Failure may lead to large regional and local consequences and disturbances
C	Negligible risk of loss of human lives	
U	Without dam consequence class.	

I:6.3 DIFFERENTIATION BY TARGET SAFETY INDEX

The target safety index presented here has been defined from calibration to existing dams that fulfil the deterministic requirements and may be applied for new structures and existing structures. A summary of the calibration is available in Westberg Wilde & Johansson (2016) and complete background documents can be retrieved from Marie Westberg Wilde or Fredrik Johansson. Minimum values for β (target safety index) in the ultimate limit state for each dam consequence class is given in Table PI-6-2. The target safety index refers to a reference period of 1 year and apply to the physical and statistical models described in this document.

PMCD

Table PI - 6-2. Minimum values for β in ultimate limit states. Reference period 1 year.

Dam consequence class	Minimum β minimum
A	5,2
B	4,8
C	4,2
U	3,8

Note 1: All ultimate limit states should fulfil the above recommendations. For most dam structures overturning will result in considerably higher β-values than sliding, but it must be checked that the value exceeds the target β.

Note 2: Calibration was done for dam consequence class B only. Target values for dam consequence class A, C and U are derived on similar basis as Eurocodes. The failure probability of B is 10 times higher than that of A and C is 100 times higher.

Note 3: JCSS 7.2.1 recommends target safety levels related to dam consequence and relative cost of safety measure. For relations between (construction cost + failure cost)/(construction cost) of > 10 a full cost-benefit analysis is recommended. This falls outside of the scope here, but recommendations are to consider higher reliability for extreme consequences. For extreme consequences (upper dam consequence class A) higher reliability should be considered.

Note 6: For new structures it may the choice of one level higher reliability class should considered during design and is recommended unless it can be properly demonstrated that it is unnecessary. When a structure is designed and built the extra expenses are small in relation to the building costs and in relation to additional strengthening if dam consequence class is increased for some reason.

Note 5: For existing structures that does not fulfil target values a cost-benefit analysis should be performed to analyse the need of further risk-reduction measures.

Note 4: Swedish national appendix SS-EN 1990 does not allow for use of appendix B and reliability index 4,8; 4,2; 3,8 for reliability classes. Instead safety class 3 (highest) is related to reliability index 4,8 and the safety class 2 and 1 to lower reliability.

For the substitute ultimate limit state of tensile stress in the dam heel (discussed in I:7.2) the target safety index is

β_{crack} = 1,3.

Failure consequences also depend on the type of failure, which can be classified according to: JCSS 7.2.1

- ductile failure
- brittle failure

Consequently a structural element which would be likely to collapse suddenly without warning should be designed for a higher level of reliability than one for which a collapse is preceded by some kind of warning which enables measures to be taken to avoid severe consequences.

Concrete dams may conservatively be expected to experience a brittle failure with limited signs of warning. PMCD

I:6.4 TARGET VALUE FOR SERVICEABILITY LIMIT STATE

No target values for serviceability limit states has been calibrated for concrete dams. General recommendations in Eurocode 1990 should be applied.

I:6.5 REFERENCE PERIOD

All basic variables should be related to a reference period of one year. Target reliability values above are thus also related to a one year reference period. If other reference periods are analysed, the values of β for a different reference period can be calculated using the following expression: EN 1990-C6

$$\Phi(\beta_n) = \left[\Phi(\beta_1)\right]^n \qquad \text{(PI. 6-1)}$$

where β_n is the reliability index for a reference period of n years, and β_1 is the reliability index for one year.

I:7 Verification by probabilistic analysis

I:7.1 GENERAL

A reliability based decision implies that the probability of failure, P_f, does not exceed a specified target, P_f^t, for a given reference period:

Adapted from ISO 2394

$$P_f \leq P_f^t \tag{PI. 7-1}$$

For all relevant limit states $g(\mathbf{x})$ ($\mathbf{x}$ are realizations of the basic random variables $\mathbf{X}$) the probability of the undesired event, defined by

$$P_f = P\left[g(\mathbf{x}) \leq 0\right] \tag{PI. 7-2}$$

is calculated.

The limit states relevant for concrete dams are defined in section I:7.2.2.

I:7.2 ULTIMATE LIMIT STATE

I:7.2.1 General

Ultimate limit states pertain to the following undesirable states (non-exhaustive):

ISO 2394

- Loss of equilibrium of the structures or part of it considered as a rigid body.
- Instantaneous attainment of the maximum capacity of members or connections by yielding, rupture or excessive deformations.
- Failure of members or connections caused by fracture, fatigue or other time-dependent accumulation effects.
- Instability of the structure or part of it.
- Sudden change of the assumed structural system to a new system (e.g. snap through, large crack formation).
- Foundation failure.

I:7.2.2 Failure modes for concrete dams

Failure of a concrete dam monolith will occur if the monolith fails somewhere along the contact surface, in the foundation rock beneath the structure or within the structure itself. PMCD

Failure in contact

Failure in the contact may occur as *sliding* or as *adjusted overturning*.

For *sliding* the shear resistance of the bonded contact (where cohesion exist) is first considered to be mobilized. Only if the bonded contact fails the shear resistance of the unbounded contact is mobilized.

On the basis of the Mohr-Coulomb failure criterion, the shear resistance for the bonded contact of the concrete-rock interface, $T_{R,Bc}$, can be expressed as

$$T_{R,Bc} = c_c \cdot A_c + N' \cdot \tan\phi_i \qquad \text{(PI. 7-3)}$$

where c_c is the cohesion of the bonded concrete-rock contact, A_c is the total bonded concrete-rock contact area, as indicated in Fig. 1. N' is effective normal load acting on the bonded concrete-rock contact, and ϕ_i is the internal friction angle for the bonded contact.

The contact may also be partially bonded.

On the basis of the failure criterion suggested by Patton (1966), the shear resistance of the un-bonded contact, $T_{R,Uc}$, can under low normal stresses be expressed as

$$T_{R,Uc} = N' \cdot tan(\emptyset_{b,c} + i_c) \qquad \text{(PI. 7-4)}$$

where $\phi_{b,c}$ is the basic friction angle for the macroscopic smooth but microscopic rough concrete-rock contact and i_c is the contribution from macroscopic roughness of the contact.

When cohesion is not considered (PI. 7-4) should be used.

Overturning occurs when the turning moment exceeds the stabilizing moments. However, the overturning mode implies unrealistically high stresses in the downstream concrete and rock if the dam is modelled as a rigid body with the overturning point around the dam toe. Therefore, a combined failure mode similar to the one proposed by Fishman (2009), denoted the adjusted overturning mode, is used. It accounts for the possible crushing of concrete or rock mass at the downstream toe before overturning occurs. This is done by successively adjusting the point of rotation in the upstream direction as the crushing proceeds. In the calculations, the length of the crushed zone, a, is first estimated from vertical force equilibrium as the effective normal force divided by the weakest uniaxial compressive strength of the concrete, f_{cc}, or the rock mass, σ_{cm}

$$a = \max\left(\frac{N'}{f_{cc}}, \frac{N'}{\sigma_{cm}}\right) \quad \text{(PI. 7-5)}$$

a is shown in the below figure.

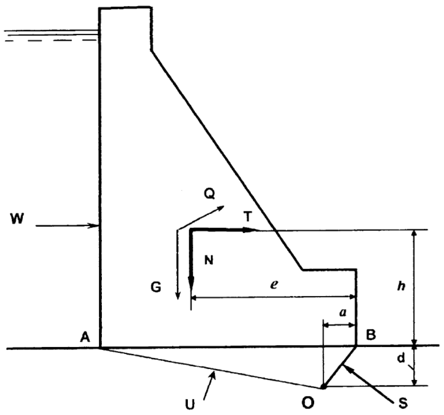

Figure PI- 7-1. Definition of limit overturning from Fishman (2007).

With this methodology, both overturning and crushing of the rock mass or the concrete are accounted for.

Tensile stresses in the dam heel

Tensile stresses in the dam heel is not a failure mode per se, as it does not imply directly failure. It is however a situation that should preferably not occur as it may give increased uplift and thus result in sliding or overturning failure. Calibrations have indicaded very high reliability related to overturning, and tensile stresses are thus analysed as substitute ultimate limit state.Stresses in the heel may be estimated based on Navier´s equation, assuming that the dam behaves as a rigid body:

$$\sigma = \frac{N'}{A} - \frac{M_{tp}}{W} \quad \text{(PI. 7-6)}$$

Failure in the rock foundation

Failure in the rock foundation can basically occur in two different failure modes. The first is sliding along persistent fractures in the foundation. The other is failure in the rock mass. For partly persistent joints, combined failure modes are possible.

The shear resistance for the rock fracture, $T_{R,F}$; can be expressed as

$$T_{R,F} = N' \cdot tan(\emptyset_{b,F} + i_F) \quad \text{(PI. 7-7)}$$

where $\phi_{b,F}$ is the basic friction angle for the macroscopic smooth but microscopic rough rock fracture and i_F is the contribution from macroscopic roughness of the fracture.

No recommendations concerning failure in the rock mass are given. In general, at least for hard crystalline rock, failure in the rock mass is not an issue for dams with moderate heights due to its strong shear resistance. However, under certain conditions such as low quality rock mass in combination with high loads on the foundation, this failure mode may have to be considered.

Failure in concrete

Failure in the concrete may occur due to degradation or due to insufficient capacity.

Degradation of the concrete may be caused by e.g. ASR-reaction or frost damage. These types of damages may be expected to progress for a long time before failure occurs. No limit states related to this failure mode is included.

Insufficient capacity may be dealt with according to Eurocodes, but it is recommended that partial factors are calibrated for dam specific loadings.

Additional failure modes

In special cases additional failure modes may be possible. These should be analysed where relevant and appropriate limit states should be formulated. Examples of additional failure modes that may occur are

- Combined failure in the ground and in the structural element
- Failure by hydraulic heave and piping
- Risk of jacking during remedial grouting
- Movement which may cause collapse or affect the appearance or efficient use of the structure or nearby structures
- Unacceptable leakage through or under the dam
- Lifting of light structures (e.g. spillways).

I:7.2.3 Limit state functions

Failure in contact

When cohesion exists the limit state for sliding of the bonded contact (Bc) is described as

$$G_{Bc} = \left(c_c \cdot A_c + N' \cdot \tan\phi_i\right) - H \qquad \text{(PI. 7-8)}$$

The limit state function for sliding of the un-bonded contact (Uc) is described as

$$G_{Uc} = N' \cdot \tan\left(\phi_{b,c} + i_c\right) - H \qquad \text{(PI. 7-9)}$$

As sliding occur only if both G_{Bc} and G_{Uc} occur

$$P_f = P\left(\min\{G_{Bc}, G_{Uc}\} \leq 0\right) \qquad \text{(PI. 7-10)}$$

If c = 0,

$$P_f = P\left(G_{Uc} \leq 0\right) \qquad \text{(PI. 7-11)}$$

The limit state function for adjusted overturning is

$$G_O = M_R - M_S \qquad \text{(PI. 7-12)}$$

Where M_R are the resisting moments and M_S the driving moments, taken around the point *a* described in I:7.2.2.

The limit state function of tensile stresses in the dam heel is given by Navier's equation as:

$$G_\sigma = \frac{N'}{A} - \frac{M_{tp}}{W} \qquad \text{(PI. 7-13)}$$

The limit state function related to tensile stresses in the dam heel is a state prior to collapse and it is used as a substitute ultimate limit state to capture the possibility of missing out overturning failures or combinations of overturning and sliding failures for complex structures.

Failure in rock joints

Sliding failure along a rock fracture may be written as

$$G_{R_j} = N' \cdot \tan\left(\phi_{b,F} + i_F\right) - H \qquad \text{(PI. 7-14)}$$

Where $\phi_{b,F}$ is the basic friction angle for a macroscopic smooth but microscopic rough surface and i_F is the contribution from the macroscopic surface roughness of the joint.

Failure due to sliding along persistent fractures in the foundation is difficult to analyse if proper investigations have not been performed. The main reason is that the safety against sliding to a large extent is affected by the depth of the fracture together with its strike and dip. In addition, it is often highly uncertain if any persistent fractures actually exist. If it is assumed that they exist, it has a significant impact on the calculated safety of the structure. Therefore, it is important that the assessment is performed based on data where the uncertainties regarding the persistent fractures have been minimized. As a consequence, a methodology is recommended where it is first calculated at which depth, z, a persistent horizontal fracture need to be located for the safety of the structure to be acceptable. After z has been determined, geological investigations of the rock volume under the monolith, from the rock surface down to the depth z, is performed in order to locate potential persistent fractures. If a persistent fracture is located, calculations are performed for this specific plane.

It should be noted that the above recommendation is not valid when the rock surface has a slope in the downstream direction. Under such conditions the depth necessary to investigate may be larger, since joints sloping downstream could constitute potential sliding planes.

For buttress dams there may also be restrictions in the minimum depth of a possible fracture, as shallow fractures may lead to lifting f the rock due to uplift.

Failure in concrete

No limit state functions of failure in the concrete structure (dam body) are included.

I:7.2.4 Design situations

Table PI-7-1 defines the design situations to be analysed. Only variable loads have been indicated in the table, while permanent loads are not included. Design situations are based on "Lastkombinationer" (load combinations) from RIDAS and relevant load combinations in RIDAS have been indicated.

The below sections give some important input in defining design situations. For further discussion on loads see part II.

Uplift

Uplift reduction due to the effect of drains and grouting may only be accounted for in calculations if

- Continuous monitoring of uplift pressures is performed
- Maintenance programme for the drainage system is available
- An action plan on how to quickly take care of upcoming problems is present

The reason is that in time, clogging will reduce the drain efficiency and leaching will reduce the effect of the grout curtain. In addition, even though measured pore pressure could indicate low values, a future event with clogged drains could govern the probability of failure (see e.g. Spross et al. 2014). Without monitoring and analysis of monitoring results there is no possibility to identify trends and thus it is not possible to know when these processes start and how they develop.

Hydrostatic pressure

Modelling of the hydrostatic pressure is described as

$$h_w = h_{rwl} + d_e \qquad \text{(PI. 7-15)}$$

where h_w is the water depth, h_{rwl} is the water depth at rwl (retention water level) and d_e is the water exceeding the h_{rwl}. h_{rwl} is assumed to be constant, while d_e in many cases may be described by a trapezoidal distribution with a certain probability to occur. This is further discussed in part II.

To cover all possibilities of high water levels caused by e.g. failure of gates, one design situation with water level at crest has been identified. This may be omitted if a reliability analysis of gates is performed and $h_w > h_{rwl}$ is modelled according to the description in Part II.

Classification of design situation is indicated by

- P – persistent
- T- Transient
- A – accidental

No seismic design situations have been included.

Ice loads

Shall not be combined with high water levels.

Table PI - 7-1. Design situations

DS	Type of Design Sit.	Upstream water level (hw)	Ice	Uplift	Downstream water level	Spillways	β-target	RIDAS LC
1	P	rwl	Y	a) monitoring : functioning drains b) no monitoring: malfunction drains	normal [2]	closed	β_T	1
2	T	rwl	N (if ice reduction is available)	a) monitoring : functioning drains b) no monitoring: malfunction drains	normal [2]	temporary closure	β_T	2
3	T	rwl	N (if ice reduction is available)	a) monitoring : functioning drains b) no monitoring: malfunction drains	normal [2]	one closed, one with temporary closure (stop log)	β_T	3
4.1	P	rwl	N	a) monitoring : functioning drains b) no monitoring: malfunction drains	depending on flow [3]	most unfavourable combination	β_T[1]	4-7
4.2	T	> rwl (de >0)	N	a) monitoring : functioning drains b) no monitoring: malfunction drains	depending on flow [3]	most unfavourable combination		
5	T	Dam crest***	N	a) monitoring : functioning drains b) no monitoring: malfunction drains	normal [2]	closed	β_T[4]	6
6	A	rwl	Y	full uplift (accidental when monitoring is present)	normal [2]	closed	β_T[1]	8
7	P	rwl	Y, asymmetrical	a) monitoring : functioning drains b) no monitoring: malfunction drains	normal [2]	closed	β_T	9
8	T	loads in building phase, defined from above when relevant			depending on event		β_x	β_T[1]
9	A	accidental water levels caused by malfunction of gates			depending on event		β_x	β_T[1]
10	A	accidental water levels due to slope failure in small reservoirs			depending on event		β_x	β_T[1]
11	A	sabotage, explosion or other accidents giving extreme loads			depending on event		β_x	β_T[1]

β_T = target value defined in section I:6.

[1] conditional probability of event must be considered. $\beta = \Phi^{-1}(\Phi(-\beta_T)*P(\text{event}))$.

As an example in LC 4 $P_f = \Phi^{-1}(-\beta_{4.1})\cdot P(LC\ 4.1) + \Phi^{-1}(-\beta_{4.2})\cdot P(LC\ 4.2) \approx \Phi^{-1}(-\beta_{4.1})\cdot P(LC\ 4.1) + \Phi^{-1}(-\beta_{4.2})\cdot P(LC\ 4.2)$ and so $\Phi(P_f) > \beta_T$

[2] water levels normal for the dam in case of closed gates

[3] need to be defined for each dam. If upstream and downstream water levels are correlated this has to be taken to account.

[4] if the dam fulfils β_T in this load case more analysis is not necessary. If not further analysis of the spillway system and reliability of gates is necessary.

I:7.3 SERVICEABILITY LIMIT STATES

I:7.3.1 General

In general the verification of serviceability limit states should be based on criteria concerning the following aspects :

- deformations that affect appearance, the comfort of users, or, the functioning of the structure;
- vibrations
- damage that is likely to adversely affect the appearance, durability, or the functioning of the structure.

I:7.3.2 Failure modes for concrete dams

In this version of Probabilistic model code only ultimate limit states are considered

I:7.3.3 Design situations

The above serviceability limit state should be considered for the same design situations as the ultimate limit states.

I:8 Updating of a prior estimation

Sometimes it is advantageous to update a first crude estimation of a parameter with observations or measurements. If the inherent variability of a normally distributed parameter is known (e.g. from an accepted coefficient of variation), but there is an uncertainty regarding the mean value, a Bayesian updating procedure can be applicable. The methodology is extensively described in Ang & Tang (2007), and applied in geotechnical engineering by e.g. Zhang et al. (2004) and Krounis et al. (2015). The general procedure implies that measurements of the parameter are used to reduce the uncertainty related to the magnitude of the mean value. PMCD

The total uncertainty of a parameter can be approximated as the product of independent stochastic variables (Goodman 1960). For a design value $\bar{Y}$ determined from measurements of X, $V^2_{\bar{Y}|X}$, is calculated as:

$$V^2_{\bar{Y}|X} \approx V^2_{\text{inh},\bar{X}} + V^2_{\text{stat},\bar{X}} + V^2_{\text{m.e.},\bar{X}} + V^2_{\text{trans},X} \qquad \text{(PI. 8-1)}$$

where $V_{\text{inh},\bar{X}}$ is the inherent variability of X averaged over the failure domain, $V_{\text{stat},\bar{X}}$ is the statistical uncertainty, $V_{\text{m.e.},\bar{X}}$ is the coefficient of variationassociated with measurement error determined based on the total measurement error and the number of uncorrelated tests with respect to $\bar{X}$, and $V_{\text{trans},X}$ is the transformation error associated with the estimation of Y from X.

In addition, it has to be noted that the coefficient of variation from measurements is only a measure of the uncertainty if another sample was taken from the population. In the stability analyses of concrete dams, we are interested in the average value and its variation over the failure plane. To obtain this, the estimated coefficient of variation from tests has to be adjusted with respect to the spatial correlation of the parameter. This can be done with the variance reduction factor proposed by Vanmarcke (2010), which results in:

$$V^2_{\text{inh},\bar{X}} = \left(V^2_X - V^2_{\text{m.e.},X}\right) \cdot \Gamma^2_X \qquad \text{(PI. 8-2)}$$

where V_X is the V assessed from measurements of X, and Γ^2_X is the variance function dependent on the size of the average domain and the scale of fluctuation of X. If the variance reduction is not considered, which is usually the case since knowledge about the spatial correlation rarely exists, $\Gamma_X = 1$.

The simplest case of Bayesian updating is inference about the mean, μ, when the standard deviation, σ, is known for a normally distributed random variable, $X \sim N(\mu, \sigma)$, using conjugate prior. Since the average value μ is unknown, it is treated as a random variable that, for mathematical convenience and simplicity, also follows a normal prior distribution with expected value $E(\mu')$ and variance $Var(\mu')$. Based on the definition of conjugate priors, it follows that the posterior distribution of the parameter's average is also normal with expected value $E(\mu'')$ and variance $Var(\mu'')$, where $E(\mu'')$ is an average of $E(\mu')$ and the mean of tests, m, weighted inversely by the respective variances:

$$E(\mu'') = \frac{m \cdot Var(\mu') + E(\mu') \cdot \frac{\sigma^2}{n}}{Var(\mu') + \frac{\sigma^2}{n}} \qquad \text{(PI. 8-3)}$$

where n is the number of observations or tests. The posterior variance of the average is given by:

$$Var(\mu'') = \frac{Var(\mu') \cdot \frac{\sigma^2}{n}}{Var(\mu') + \frac{\sigma^2}{n}} \qquad \text{(PI. 8-4)}$$

A problem with the above methodology is that there are few cases in which the variance is known in practice. However, the approach is still applicable since the results are not much different compared to when the uncertainty in the variance is included, especially when data from many tests is available (Lee 2004). If the data is limited, it is possible to perform a sensitivity analyses on the assumed standard deviation. If it is judged unsuitable to use the assumption of a known standard deviation, Markov Monte Carlo simulations can instead be used to numerically estimate the posterior distribution.

Example: Basic friction angle

The following example is from Krounis et al. (2015) and describes how the prior knowledge of the basic friction angle of the concrete-rock contact can be updated with results from shear testing. In this example the measurements error is ignored, which may be reasonable since the test results probably can be considered accurate. In addition, no transformation error exists, since the parameter is measured directly in the tests. Since no information of the correlation length exists, $\Gamma_{\phi_b} = 1$. This gives:

$$V^2_{\text{inh},\bar{\phi}_b} \approx V^2_{\phi_b} \qquad \text{(PI. 8-5)}$$

This means that total uncertainty associated with $\bar{\phi}_b$can be estimated as:

$$V^2_{\bar{\phi}_b} \approx V^2_{\text{inh},\bar{\phi}_b} + V^2_{\text{stat},\bar{\phi}_b} \approx V^2_{\phi_b} + V^2_{\text{stat},\bar{\phi}_b} \qquad \text{(PI. 8-6)}$$

Since only three tests were performed, both V_{ϕ_b} and $V_{\text{stat},\bar{\phi}_b}$, are determined by combining the data with prior knowledge consisting of information regarding basic friction angles of concrete-rock interfaces. According to Lo and Hethy (1998) is the mean basic friction angle not very sensitive to rock type and varies from 30° to 39° (note that it represents the distribution of the average value). Based on this value is the prior mean, μ'_{ϕ_b}, assumed normally distributed with expected value,

$E\left(\mu'_{\phi_b}\right)$ = 35° and variance $Var\left(\mu'_{\phi_b}\right) = 2.8^2 = 7.84$. In previous studies of rock fractures (Johansson et al. 2010, Park et al. 2005, Pathak and Nilsen 2004, Duzgun et al. 2003 and Park and West 2001), which behave in a manner similar to concrete-rock interfaces, a $V_{\phi_b} \approx 0.10$ has been suggested. The standard deviation of the underlying population, σ_{ϕ_b}, is therefore assumed equal to 3.5°.

The likelihood is determined based on the results from the direct shear tests. The results from the tests were 40.1°, 40.5°, 42,1°, with a sample mean of m_{ϕ_b} = 40.9°. If the prior distribution is combined with the test results using Eqs. (PI. 9-3) and (PI. 9-4), the updated (posterior) distribution of the mean, $E\left(\mu''_{\phi_b}\right)$ = 38.9° and $\mathrm{Var}\left(\mu''_{\phi_b}\right)$ = 2.69°. This gives $V_{\phi_b} = \sigma_{\phi_b}/E\left(\mu''_{\phi_b}\right)$ and $V_{\mathrm{stat},\bar{\phi}_b} = \mathrm{Var}\left(\mu''_{\phi_b}\right)^{0.5}/E\left(\mu''_{\phi_b}\right)$, which can then be calculated and inserted in Eq. (PI. 9-6), which results in $V_{\bar{\phi}_b}$ = 0.10.

I:9 References

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Part II: Load models

II:1 Load models

II:1.1 GENERAL PRINCIPLES

The environment in which a structure function gives rise to internal forces, deformations, material deterioration and other short-term or long-term effects in the system. The causes of these effects are termed actions. The environment from which the actions originate can be of a natural character, for example, flood, ice, wind and earthquake. It can also be associated with human activities. JCSS Part II, 2.0.1

The following concepts are used:

- An action (load) is an assembly of concentrated or distributed forces acting on the structure.
- Indirect actions are caused by imposed displacements, thermal effects or environmentally induced effects (e.g. moisture, shrinkage) in the structure.
- Actions causing changes with time in the internal material properties or in the dimensions of a structure.

Action descriptions are in most cases based on suitably simple mathematical models, describing the temporal, spatial and directional properties of the action across the structure. The choice of the level of richness of details is guided by a balance between the quality of the available or obtainable information, and a reasonably accurate modelling of the action effect.

II:1.2 CLASSIFICATION

Loads should be classified according to their variation in time as; EN 1990-4.1.1

- Permanent actions, *G*, e.g. self weight, earth pressure as well as indirect action (caused by e.g. shrinkage, creep or settlements).
- Variable actions, *Q*, e.g. actions caused by the use of the structure and external loads.
- Accidental actions, *A*, e.g. explosions and impact loads.

For variable and accidental actions probability distributions should relate to annual maximum values.

Some actions, e.g. earthquake actions, may be either accidental loads or variable loads.

Actions caused by water may be permanent or variable depending on the variation with time.

Actions are also classified

- Related to origin as *direct* or *indirect*
- Related to spatial fluctuation as *fixed* or *free*
- Related to their nature or the reaction of the structure as *static* or *dynamic*.

An action should be described by a model, its magnitude being represented in the most common cases by one scalar which may have several representative values.

II:1.3 LOAD MODELLING

II:1.3.1 Modelling of actions

There are two main aspects of the description of an action: the physical and the statistical aspect. In most cases these aspects can be clearly separated and the physical description gives the types of physical data which characterise the action model; for example, vertical forces distributed over a given area. The statistical description gives the statistical properties of the variables; for example a probability distribution function. In some cases the physical and statistical aspects are so integrated that they cannot be considered separately. JCSS 2.0.3-2.0.7

A complete action model consists in general, of several constituents, which describe the magnitude, the position, the direction, the duration etc. of the action. Sometimes there is an interaction between the components. There may in certain cases also be an interaction between the action and the response of the structure.

An action F may in general be described by the variables F_o and W as

$$F = \varphi(F_o, W) \qquad \text{(PII. 1-1)}$$

where F_o is a basic action variable, which is directly associated with the event causing the action. It should be defined so that it is, as far as possible, independent of the structure. For example, for snow load F_o is the snow load on the ground on a flat horizontal surface. The time variability is normally included in F_o. In probabilistic modelling all action variables are in principle assumed to be random variables or processes

W is a kind of conversion factor or model parameter appearing in the transformation from the basic action to the action F which affects the particular structure. W may depend on the form and size of the structure etc. For the snow load example, W is the factor which transforms the snow load on ground to the snow load on roof, and which depends on the roof slope, the type of roof surface etc. Spatial variability of an action is in most cases included in W. Sometimes model parameters may themselves be random variables, for example when the model allows for statistical uncertainty due to small sample sizes.

φ is a suitable function, often a simple product.

An action model may consist of one or several actions F, each modelled by F_{oi} and W_i. Each model may be described by:

- stochastic processes or random fields
- sequences of random variables
- individual random variables
- deterministic values or functions

II:1.3.2 Models for fluctuations in time

To describe time dependent loads, one needs the probability distribution for the "arbitrary point in time values" and a description of the variations in time. Typical models are:

- Continuous and differentiable process
- Random sequence
- Point pulse process with random intervals
- Rectangular wave process with random intervals
- Rectangular wave process with equidistant intervals Δ

No further discussion regarding time-dependant loads is given here. Reference period for load parameters is one year (that is statistical parameters of loads should be based on annual maximum values).

II:1.3.3 Interactions and correlation between actions

For describing dependencies between various actions it is useful to distinguish between:

- Actions of the same nature (e.g. hydrostatic load and uplift). They may often be considered as components of one action. The various components are normally described by similar probabilistic models
- Actions of different nature (e.g. hydrostatic load and ice load). Actions of different nature may show quite complex physical interactions. One example is ice load and hydrostatic pressure. Large variations of regulation amplitude may break the ice and reduce the ice load substantially, while low regulation

amplitude may increase the ice load. The highest ice loads may be expected in late winter-spring when water levels may be low. So one need to build a more advanced physical model on the one hand and conditional probability models of one load given the (extreme) condition of the other. In most cases it may be convenient to define one of the processes as the “leading one” and describe arrival times and amplitudes of the second process conditional upon the occurrence and amplitude of the first one.

More information of combination of actions and mathematical techniques for calculation of load combination may be found in annex 3 of JCSS part II.

II:1.4 LIMITATIONS

In this part the following limitations have been made:

- No correlation is assumed between uplift and hydrostatic pressure. As described in the section on uplift pressure, uplift may exhibit a non-linear relation to the hydrostatic pressure in case of increased head water level. Further information on this is not available and thus this interaction is not included in the load model.
- Seismic actions are not included. In the present recommendations for analysis of dams in Sweden, seismic actions are not included and further discussion on the topic is relevant.

II:2 Ice loads

II:2.1 INTRODUCTION

Ice load affects dam structures. The following section is based on a literature study (Johansson et al. 2013). The most important factors affecting the extent of the ice load are 1) loads imposed by temperature variations, 2) loads imposed by water level fluctuation, 3) loads imposed by wind and flowing water.

II:2.1.1 Thermal ice loads

When temperature increases the ice expands (heat expansion coefficient 5 times that of steel), but only in the upper part of the ice layer. This expansion results in compressive stresses varying over the ice thickness. The lower side of the ice has a constant temperature of 0 °C that prevents the ice from expanding, leading to bending moment and cracking.

Although theoretical calculations indicate that thermal ice pressure can become very large, several factors reduce this pressure, or decrease the frequency of the high pressure. A small cover of snow reduces the thermal ice loads substantially. The possibility of combinations of thick ice without snow cover and extreme temperature variations is the largest in early winter and in late winter. In late winter the upper part of the ice is often a combination of snow and ice, resulting in lower ice loads. In early winter the ice thickness is generally smaller.

Another factor affecting the thermal loads is restraint from the shores. A shallow slope will have less restraint than a steep or vertical slope and thus a shallow slope gives less thermal ice loads.

II:2.1.2 Water level fluctuation

Monitoring of ice loads (Comfort et al, 2000) show that the water level fluctuations may affect the ice loads. A continuous decrease creates no ice loads and a single large fluctuation in water level results in a significantly smaller ice load for the rest of the year. Small variations (0.1-0.15 m) with slow variations (0-0.5 times/day) give insignificant contribution to the load, whereas medium fluctuations (0.1-0.2 m) occurring often (1-2 times/day) increase the ice load. This is shown in Figure PII-2-1.

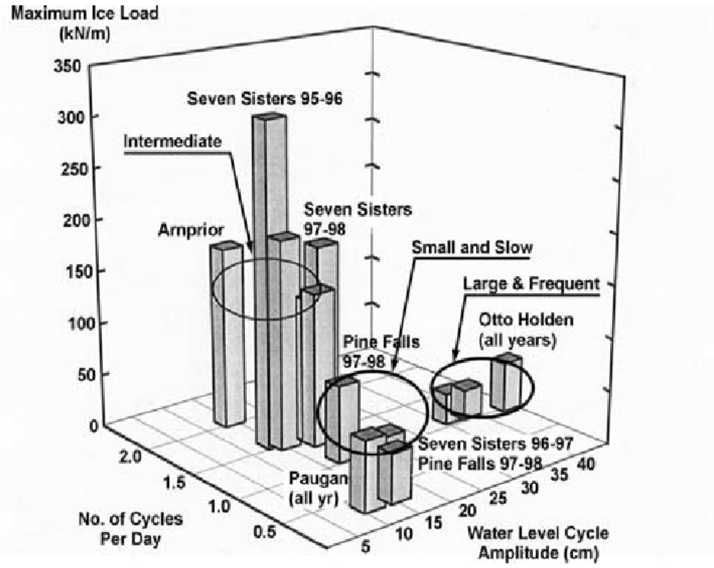

Figure PII- 2-1. Observed ice loads as a function of number of cycles per day and water level cycle amplitude (Comfort 2000).

Stander (2006) proposed that the following conditions in water level fluctuation must exist in order to influence the ice loads:

- Fluctuations must be large enough to produce an active crack along the sides, but not large enough to disconnect the ice cover from the dam body.
- Fluctuations must be around a mean value. In case of successive decrease no connection to the dam body will be possible.
- The ice cover must have restricted possibility for movement, otherwise it will move towards the free water surface, resulting in negligible compressive stresses.
- The temperature must be low enough to enable icing in the crack. Snow or water on the ice reduces this possibility.
- Close to the shore, stresses increase if the ice cover is resting on the bottom of the reservoir.

Stander further proposes that the ice load increase due to water level fluctuations may be reduced by lowering and rising of the reservoir approximately 0,3-0,4 m during the first weeks after the ice cover is formed. The explanation is that the reservoir lowering and rising caused a "hinge" that prevented larger ice loads and the proposal was based on observations.

II:2.1.3 Loads imposed by wind and water

Flowing water and strong wind gives forces on the ice (see USACE, 2002). The total area affected by strong winds and/or currents used to calculate the total ice load on a dam structure is difficult to assign. For large ice surfaces the total force may be larger than the forces resulting in failure of the ice. In those cases the ice load is limited by failure loads due to crushing, bending, buckling and splitter of the ice.

Carter et al (1998) support the theory of limiting factors regarding ice loads on dams. They observed cracks parallel to the dam on a regular distance of 6.5 m, well in agreement with the theoretical distance for maximal bending moment for a thin floating ice cover.

II:2.1.4 Deterministic design according to RIDAS

According to RIDAS, horizontal ice load has intensity 50-200 kN/m, depending on geographic location, altitude above sea level and local conditions at the dam. As a guidance, for dams located at low altitude in the southern part of Sweden (Skåne, Blekinge, Halland, Bohuslän och Västergötland) 50 kN/m may be assumed. Further north, up to a line between Stockholm and Karlstad 100 kN/m may be assumed. For the rest of the country 200 kN/m may be assumed.

According to previous investigations (Elforsk 02:03) larger ice loads than 200 kN/m may appear for thin structures.

According to the Norwegian guideline loads of 100-150 kN/m should be assumed.

II:2.1.5 Summary of measurements of ice loads

A summary of performed measurements on year-maximum values of ice loads was performed by Adolfi & Eriksson (2013). They collected all measurements available from literature (in total 27 samples). The summary shows that ice loads may be assumed to have a log-normal distribution with mean value μ = 81 kN/m and standard deviation σ = 86 kN/m, resulting in a coefficient of variation of, V_I=1,07.

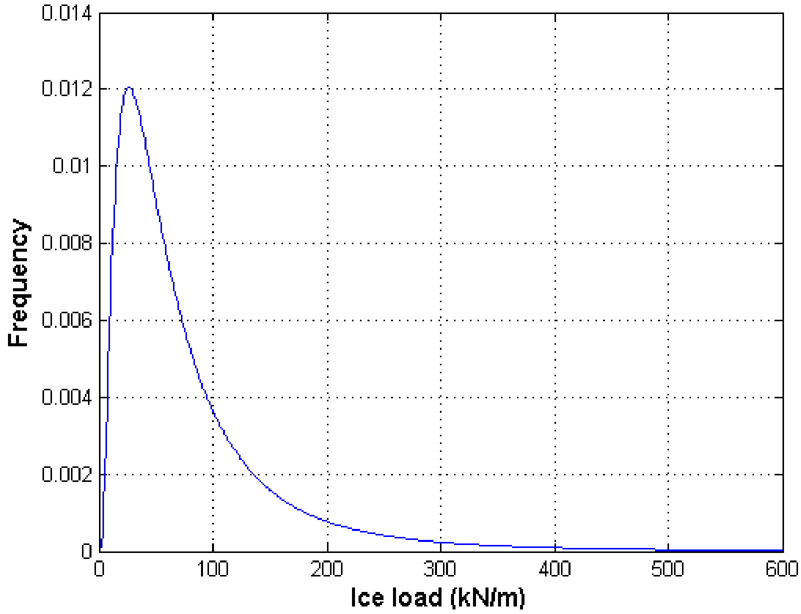

Figure PII- 2-2.. Ice load based on measured data (Adolfi and Eriksson 2013).

II:2.1.6 Basic model

Apart from the summary by Adolfi & Eriksson (2013) there is no new information regarding possible mean values and variance of ice loads. Theoretical modelling by Bergdahl & Wernersson (1978) and Fredriksson & Persson (2005), summarized in Westberg (2007), show lower coefficient of variation of about 40%.

At the present time, it is not possible to understand the basis of the ice loads presented in RIDAS, but they are most probably based on the work by Starostolsky (1979) as described in ICOLD bulletin 105 (1996).

As an engineering assumption the mean value and standard deviation from Adolfi & Eriksson is used for calculation. This is a rough assumption, but as more detailed information is not available, it is believed to be the best possible assumption given the available information. It is believed, however, that the ice measurement summarized in Adolfi and Eriksson (2013) has mainly taken place where the ice thickness is large; hence it is assumed that the ice forces according to Adolfi & Eriksson (2013) applies to regions with thick ice covers and is therefore relevant for the northern part of Sweden.

The above assumption, however, give a large probability of extremely high loads, that have not been measured in practice. A reasonable approach is thus to truncate the ice load, to limit the high ice loads. Carter et al (1998) proposed that the maximum ice load on a concrete dam is limited by the capacity of the ice related to buckling, since a crack parallel to the structure is formed approximately 6-10 m from the structure. They recommended the following equation:

$$I_m = 253h^{1,5} \qquad \text{(PII. 2-1)}$$

where I_m is the horizontal ice load on the dam, in kN/m, and h_I is the ice thickness in meters. The maximum ice thickness in different parts of Sweden is summarized in Eklund (1998). Figures from Eklund (1998) of maximum ice thickness in different parts of Sweden are shown below. The results in Eklund (1998) are based on measurements in 30 lakes for more than 40 years, and somewhat shorter series (16-39 years) in 5 other lakes. It is concluded that there is a variation between nearby lakes, but the numbers are considered representative.

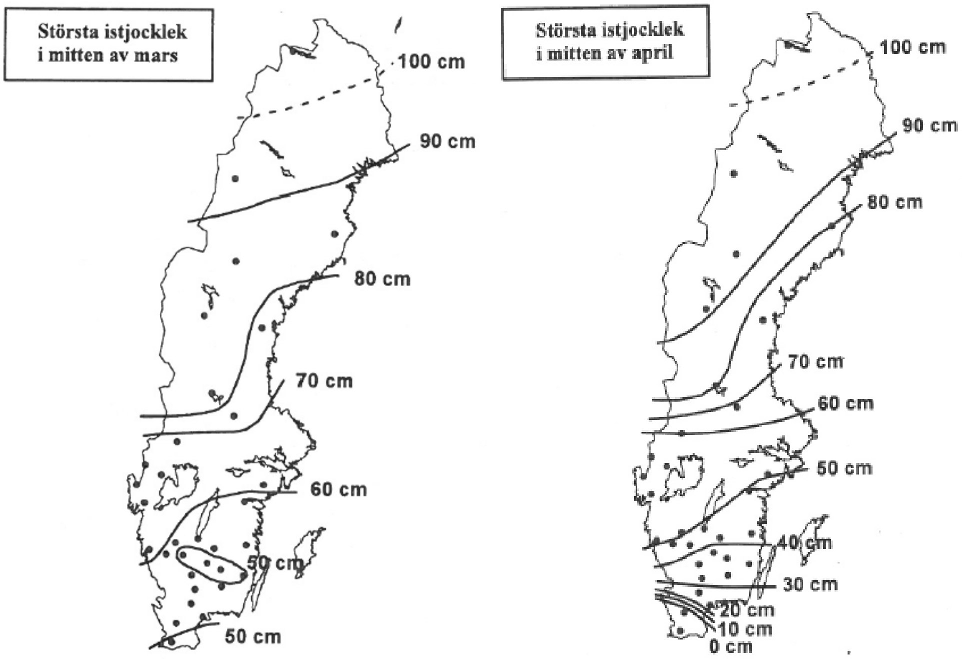

Figure PII- 2-3. Maximum ice thickness in a) mid March b) mid April. From Eklund (1998).

II:2.2 PROBABILITY DENSITY FUNCTIONS

Ice load is modelled by a lognormal distribution that describes annual maximum loads.

There is large uncertainty related to ice load, both to the distribution but also to the maximum value.

The assumption applied in this work is thus based on the work of Adolfi & Eriksson (2013) with truncation according to Carter et al (1998) and ice thickness according to Eklund (1998). Hence,

$I \sim \mathrm{LN}(\mu_I, \sigma_I)$, $I \leq I_m$, $I_m \sim \mathrm{N}(\mu_{Im}, \sigma_{Im})$

For ice thickness 1 m the mean value is assumed to be 80 kN/m and the coefficient of variation is assumed to be 1, giving standard deviation of 80 kN/m. For thinner ice thickness it is assumed that the ice load is proportional to the ice thickness, hence the mean value and standard deviation is estimated as

$$\mu_I(h) = \mu_I(1) \cdot h_I \qquad \text{(PII. 2-1)}$$

and

$$\sigma_I(h) = \mu_I(1) \cdot V_I \qquad \text{(PII. 2-2)}$$

where $\mu_I(h)$ and $\sigma_I(h)$ is the mean value and standard deviation, respectively, of ice for thickness h in m, $\mu_I(1)$ is the mean value for 1 m thickness.

The maximum ice load I_m (truncation value) will, in some cases, have a large impact on the final result. For this reason a model uncertainty with mean value 1 and coefficient of variation of 0,1 is introduced for this parameter.

Ice thickness, mean value and standard deviation of ice load and maximum ice pressure is summarized in Table PII-2-1..

Table PII- 2-1. Properties of ice in different parts of Sweden.

		Ice load, I		Maximum ice load (truncation), I_m	
Position	**Max ice thickness [m]**	μ_I [kN/m]	σ_I [kN/m]	μ_{Im} (kN/m)	σ_{Im} (kN/m)
Götaland	0,60	48	48	120	12
Svealand	0,80	64	64	180	18
Norrland	1,00	80	80	250	25

The position of the resultant ice load may be considered to act at one third of the ice thickness below the retention level h_{rwl}.

II:2.3 REFERENCES

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II:3 Hydrostatic pressure/water level

II:3.1 INTRODUCTION

Hydrostatic loading is a function of both natural variation due to rainfall, snowmelt etc., and to operation of the hydropower station. Dams have restrictions regarding the allowed maximum water levels (rwl) and lower limit (SG). Only in cases of very large floods or failure of discharge facilities, will water rise above the retention water level.

The water depth (or level) may be described as

$$h_w = h_{rwl} + d_e \qquad \text{(PII. 3-1)}$$

where h_w is the water depth, h_{rwl} is the water depth at rwl and d_e is the water exceeding rwl. h_{rwl} may be assumed constant when year-maximum is considered, while d_e in many cases can be described by an statistical distribution with a certain probability to occur. That is, two load situations are identified:

1. $h_w = h_{rwl}$. Since a water level at, or very close to h_{rwl} may be expected nearly every year the probability of this to occur is close to 1.
2. $h_w > h_{rwl}$. Occurs with a small probability that is dependent on natural variation, operation, availability of gates and power station. The statistical distribution of d_e is found by approximation as shown in II:3.2.1 or by a simulation methodology as described in II:3.2.2.

The two load combinations above are mutually exclusive (can not happen at the same time). The probability of failure of a dam may, according to the law of total probability, be written as

$$\begin{aligned} P(failure) &= P(failure \mid h_w \le h_{rwl}) \cdot P(h_w \le h_{rwl}) + \\ &+ P(failure \mid d_e > 0) \cdot P(h_w > h_{rwl}) \end{aligned} \qquad \text{(PII. 3-2)}$$

$P(failure \mid h_w \le h_{rwl}) = P(failure \mid d_e \le 0)$ is given by the limit state functions and design situations defined in part I for $d_e = 0$, hence $h_w = h_{rwl}$. Similarly, $P(failure \mid d_e > 0)$ is given by the limit state functions and design situations defined in part I for $d_e > 0$, hence $h_w = h_{rwl} + d_e$.

II:3.1.1 Large regulation amplitude

For regulation dams with large reservoirs and large regulation amplitude the above description may be somewhat different, and if there are proof that the maximum water level each year is in general lower than retention water level, the above description may be adjusted.

II:3.2 PROBABILITY DENSITY FUNCTION

h_{rwl} is a deterministic variable

d_e is a random variable, given that $h_w > h_{rwl}$, and may be assumed to have a trapezoidal distribution.

Derivation of the properties of d_e may be performed by two different procedures. In the first procedure (simplified) only floods are assumed to give high water levels. In the second procedure, assumptions are made regarding the probability of power station shut down and gate availability. Using the second approach, gate reliability and its effect on the safety of the concrete structures may be investigated.

The second approach is only possible to use for dams where the downstream water level does not directly affect the dam. For dams where the downstream level affect the dam there is a relationship between upstream water level, discharge and downstream water levels. The downstream water level is high in case of floods and gates are open. In the case when malfunction of gates is included, the relationship between upstream water level and discharge is not the same. The reason is that surcharge may occur both due to large floods (high downstream levels) and due to smaller floods and gate mal-function (not affecting the downstream level).

All calibrations of target reliability levels in this model code is performed using the first, simplified approach. For future calculations, or where LC5 indicates that failure of the structure is sensitive to gate failure, the second approach may be used to investigate the effect of gate reliability.

II:3.2.1 Simplified procedure

Probability of levels above rwl

First estimate

$$P\left(h_w > h_{rwl}\right) = P\left(d_e > 0\right) \tag{PII. 3-3}$$

For consequence class 1 and 2 according to RIDAS (approximately similar to dam consequence class B and C according to Miljöbalken) the 100-years flood shall be possible to discharge with headwater at retention water level. In many cases the floods corresponding to the 1000-year flood, or even the design flood, is possible to discharge at retention water level for consequence class 1 facilities. The design flood Q_{dim} may, according to Flödeskommittén (1999, 2007), be assumed to have a return period of approximately 5000 to 20 000 years. Generally, 10 000 years is assumed.

For dams in the beginning of a river system, equation PII. 3-4 has been shown to give a reasonable approximation of the flow. This equation relates inflow and return period according to:

$$\frac{\log(100)}{Q_{100}} = \frac{\log(n_n)}{Q_n} \qquad \text{(PII. 3-4)}$$

where n_n is the return period of flow Q_n, and Q_{100} is the flow with 100 year return period. From known discharge at rwl, Q_{rwl}, and 100-year flood, Q_{100}, the return period of a Q_{rwl} may be approximated, using eqn PII.3-4. From this

$$P(d_e > 0) \approx \frac{1}{n_{rwl}} \qquad \text{(PII. 3-5)}$$

It is necessary to verify the assumption of equation PII.3-4. This may be done by calculating the return period of Q_{dim} by eqn PII.3-4 and verify that this return period is between 5 000 to 20 000 years.

For dams in lower parts of a river system, or when inflow is to a large extent affected by larger reservoirs, eqn PII.3-4 often overestimates the flow. Simple analyses show that this overestimation may be more than 20 %. For the purpose of this guideline, an overestimation is on the conservative side and may be used as a first assumption. For dams in the southern part of Sweden, where runoff is more dependent on rain than on snowmelt, the formula may underestimate the flow. In that case, or if the final probabilistic calculations reveal that the sensitivity to d_e is large, a complete hydrological study is recommended.

The annual probability of $h_w \leq h_{rwl}$ is given by

$$P(h_w \leq h_{rwl}) = 1 - P(d_e > 0) \qquad \text{(PII. 3-6)}$$

Statistical distribution of d_e (annual maximum)

When equation eqn PII.3-4 has been verified it is possible to estimate return periods for flow with return periods larger than Q_{rwl}.

For a specific dam the outflow in a certain situation is dependent on discharge through gates, and for high water levels also outflow over gates and over the dam. For a certain water level s, $h_{rwl}<s<\infty$, it is possible to estimate the discharge by combining flow through gates with discharge of water above gates, that will occur for high water levels. For even higher water levels the dam crest will overflow. This is illustrated by the below figure where the black line indicate rwl and outflow through gates. The grey long dashed line represents a water level that will result in outflow above the surface spillway gate; outflow over the concrete dam is indicated by the short-dashed line and outflow over the rock fill dam by the small-dashed line. Outflow above gates and dams may be approximated by simple hydraulic equations.

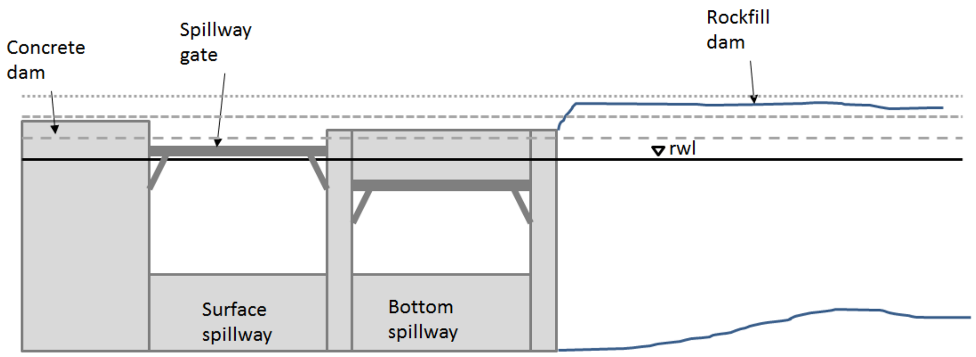

Figure PII- 3-1. Outflow at different water levels.

For different water levels the discharge may be found and related to return periods by eqn PII.3-4. From this relation a statistical distribution of d_e may be defined, e.g. by maximum likelihood estimation. Note that it is generally necessary to divide (PII.3-2) into several sub-cases since the parameters of the statistical distribution are different depending on where outflow occurs.

d_e has been found to be best described by trapezoidal distribution with parameters a, b and c as indicated in the below figure. The cumulative distribution may be written as

$$F(x)=\begin{cases}0 & for\; x<a\\ \dfrac{(x-a)^2}{(b-a)(c-a)} & for\; a\le x\le c\\ 1-\dfrac{(b-x)^2}{(b-a)(b-c)} & for\; c<x\le b\\ 1 & for\; b<x\end{cases} \qquad \text{(PII. 3-7)}$$

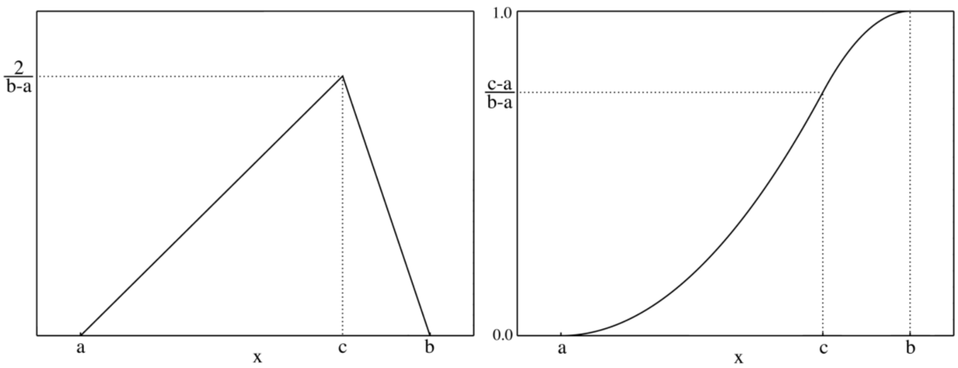

Figure PII- 3-2. PDF and CDF of trapezoidal distribution.

An exponential distribution may also be used if it is found to be more suitable.

An example where d_e is divided into two sub-cases is shown below. The blue line corresponds to results from calculation of water levels and return periods and the red and green line corresponds to the trapezoidal distribution of subcases 1 and 2 (having a,b,c [0;0;2.13] and [0.12;0.12;1.084]).

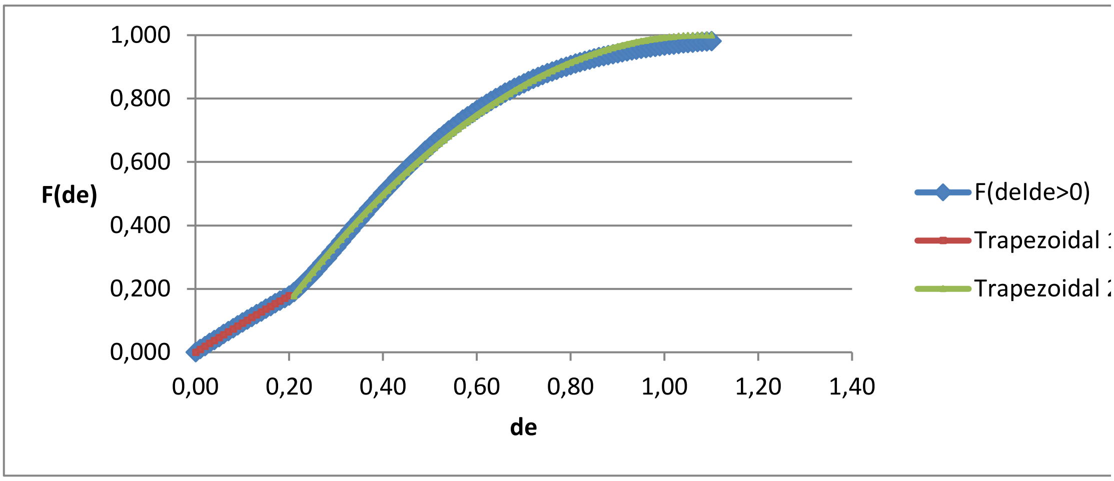

Figure PII- 3-3.Example of CDF of de divided into two subcases.

The probability of failure is given by

$$P(failure) = P(failure|h_w \le h_{rwl}) \cdot P(h_w \le h_{rwl}) + P(failure|0 < d_e < x_1) \cdot P(0 < d_e < x_1) + P(failure|x_1 < d_e < x_2) \cdot P(x_1 < d_e < x_2) + \ldots P(failure|x_n \le d_e) \cdot P(x_n \le d_e) \qquad \text{(PII. 3-8)}$$

Where x_1, x_2, x_n are water levels above rwl. For practical analysis it is necessary to perform several calculations and in each calculation it is necessary to restrict the values of d_e by truncating at maximum and minimum values.

II:3.2.2 Simulation procedure

A simulation procedure may be used if the availability of gates is considered important or need to be investigated for other reasons. Note that the below described simulation procedure may be considered as a risk analysis related to discharge facilities.

1. Define data for the specific facility: number of gates, width of gates, levels for bottom and top of gate, information of μ (describing discharge volume), length and crest height of dam, power station discharge, etc.
2. Historical data of the flow during operation, which may be used for a frequency analysis of floods up to approximately 50-100 years.
3. Estimate return period of floods between the 100-year flood and the design flood by (PII 3-4). Verify (PII 3-4) to $Q_{10\,000}$.
4. Assume availability for each gate and assume probability of tripping of the power station (power station shut down causing immediate necessity to open gates).
5. For floods of return period 1-10 000 years (or higher), estimate the water level d_e and the probability of $d_e>0$ given the flood and availability of gates and power station.
6. Define probability distribution of d_e.

For large regulation reservoirs, the "previous pool level", i.e. the water depth prior to the start of an event is of outmost importance and should be included.

II:3.3 REFERENCES

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II:4 Hydrostatic pressure downstream

The water level downstream of a dam, in case it exists, varies depending on the discharge.

In some cases physical model tests exist and discharge and related water levels downstream are known with good precision. In other cases only experience of “floods” from personnel on site is available. In the first case the relationship between discharge, probability of a certain discharge and water levels may be used to estimate the probability density function of the downstream water level. This may be done similar to the simplified procedure introduced in II:3.2.1.

If only experience from water levels of “floods” exist, a qualitative assumption of downstream water levels must be done. This should be on the “safe side”. Preferably, different assumptions are analysed to estimate the sensitivity of downstream water levels on the stability.

II:5 Uplift pressure

II:5.1 INTRODUCTION

Uplift pressure is produced in a dam rock foundation by water in fractures or pores in the foundation rock. At the upstream side the pressure equals the reservoir head and at the downstream side it equals the tail water head. Between these points the uplift pressure varies depending on the loads acting on the dam, the temperature in the surroundings, the geology of the foundation rock, type and extent of foundation treatment and efficiency of foundation drainage system.

II:5.1.1 Basic design assumption

In deterministic design the common assumption, based on Darcy's law, is linear reduction of pressure from upstream to downstream. In case of an inspection tunnel at the rock surface, and drilled drainage holes, the uplift downstream of the tunnel may be assumed to be 30% of the headwater minus the tail water. In case of an inspection tunnel in the dam body (i.e. not directly on the rock surface), a 50% reduction in the upstream line of the inspection tunnel may be assumed. A grout curtain may, according to RIDAS, be assumed to reduce the uplift by approximately 50%, but it may only be assumed as an extra margin of safety in case uplift pressure monitoring and re-grouting with predefined time intervals is not performed. Figures PII-5-1 to PII-5-5 show design assumptions according to RIDAS. USACE (2003) and USBR (1987) both include the drain effectiveness in the calculation of uplift and uplift at the line of drains is given by equations such as:

$$H_d = K_d\left((H-h)\left(\frac{L-x_d}{L}\right)\right)+h \qquad \text{(PII. 5-1)}$$

Where K_d = 1-E_d and x_d is the loacation of drains. According to USBR 0 < E_d < 0,66 and consequently 0,33 < K_d < 1. According to USACE the drain efficiency E_d is 0 < E_d < 1, but is "assumed to be 25-50%. If foundation testing provide supporting justification, 67% drain effectiveness may be assumed."

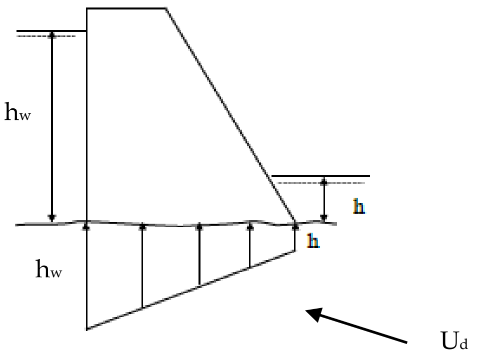


Figure PII- 5-1.Design assumption (RIDAS, 2012).

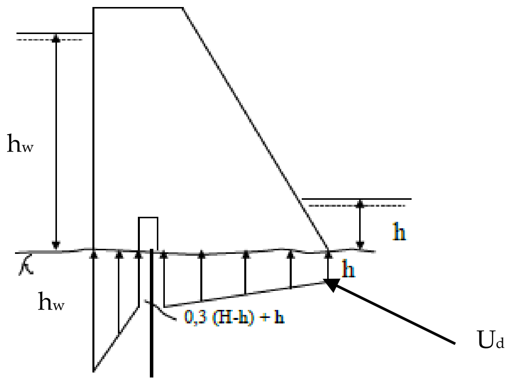


Figure PII- 5-2.Design assumption when inspection gallery and drainage exist. (RIDAS, 2012)

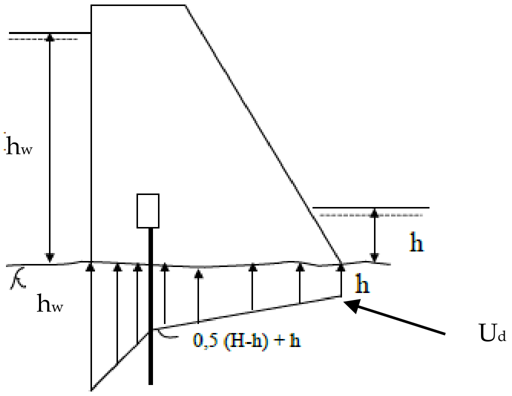


Figure PII- 5-3. Design assumption when inspection gallery and drainage are located in the dam body. (RIDAS, 2012)

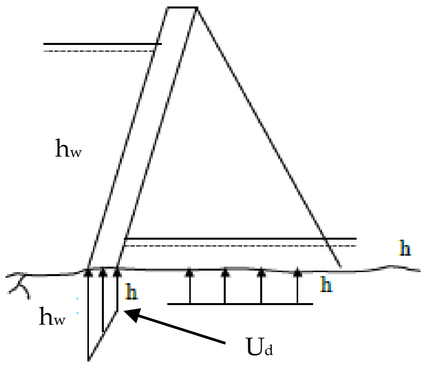


Figure PII- 5-4. Design assumption for buttress dam. (RIDAS, 2012)

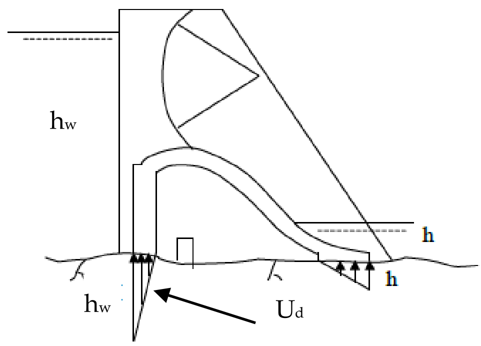


Figure PII- 5-5. Design assumption for spillway section. (RIDAS, 2012)

II:5.1.2 Influence of drains and grouting

Ruggeri et al (2001) performed a large investigation on uplift and concluded that drains is the single most effective means of reducing uplift pressure.

Ruggeri et al (2001 concluded that “while it is agreed that a well-constructed grout curtain can reduce the amount of seepage through a dam foundation the influence of the curtain on uplift pressures is still a topic of debate”. Analysis of different studies of uplift showed that there were both excellent examples of grout curtain effective in reducing uplift, to situations where the grout curtain was not effective at all in reducing uplift.

II:5.1.3 Influence of stress distribution/high water level

Grenoble et al (1995) compare the pressure distribution of tapering joints to the pressure distribution in a foundation. A schematic picture of tapering joints is shown below.

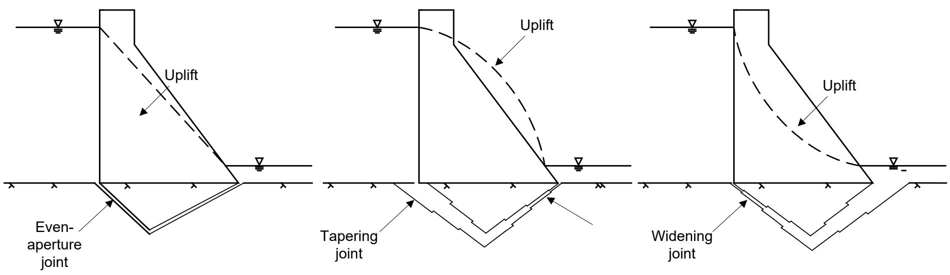


Figure PII- 5-6. Schematic picture of tapering joints.

Depending on the aperture of the joints, they are more or less affected by increased stress. For small joint apertures, a stress increase will have considerable effect on the hydraulic aperture thus affecting the hydraulic conductivity. This may be compared to the pressure distribution of a tapering joint.

For a rise of the reservoir head, the stress distribution reduces compressive stresses at the heel and increase compressive stresses at the toe. Thus horizontal joints below the base of the dam may open near the heel and close near the toe. Investigations presented in Ruggeri et al (2001) showed that there are examples of linear behaviour of uplift increase for increasing head water, as well as non-linear behaviour. Non-linear behaviour would be expected for foundations with tight, un-grouted joints and large variations in reservoir level. Due to lack of information non-linear effects are not included, see section II:1.4 Limitations.

II:5.1.4 Influence of temperature changes

Uplift pressures vary throughout the year. According to Guidicini & Andrade (1988) the reasons are volumetric variations in the concrete structures, volumetric variations of the discontinuous rock medium, influence of the water flow through the rock mass and variations in kinematic viscosity of the water.

II:5.1.5 Monitoring of uplift

It is often stated that direct measurements of uplift preferably should be used instead of the common empirical assumption for safety reassessments of concrete dams founded on rock (Stone and Webster Engineering Corporation 1992; Ruggeri 2004a, 2004b; U.S. Army Corps of Engineers 2005). The argument is that by measuring the uplift pressure, the rather conservative empirical assumption is not needed. As a consequence, over-conservative stability-enhancing improvements could be avoided. However, this requires that the drainage system and the grout curtain remain fully functional for the foreseeable future. Spross et al. (2014) has shown how the probability of sliding failure of a dam is highly dependent on the functionality of the drainage system and the grout curtain. Thus, if the safety reassessment is to be based on measurements of the uplift pressure, it must be proven that the drainage system and grout curtain are maintained in such a manner that the probability of future uplift increase is sufficiently small.

Monitoring of uplift pressure does however serve other purposes. Even if it is hard to prove the future drain and grout curtain efficiency so that measurement data could be used directly in dam safety reassessments, reliable uplift monitoring will indicate needs for re-grouting or drainage maintenance to avoid clogging, which otherwise could lead to uplift increase.

To sum up, it is not recommended to use uplift monitoring results for analysis of stability, but uplift monitoring is necessary to verify that drainage holes and grout curtain function as intended.

II:5.1.6 Uplift for buttress dams

For buttress dams uplift is present beneath the front plate and beneath thick columns. According to RIDAS, uplift may be left out for columns with thickness b less than 2 m. For larger thickness linear uplift reduction may be assumed over a distance of $b/2$.

II:5.1.7 Uplift in rock joints

If joints exist in the rock mass, with connection to the upstream side, uplift in those joints is assumed in accordance to that of a gravity structure. If drains penetrate the joint and have sufficient capacity, uplift reduction may be assumed.

II:5.1.8 Cracking of the dam heel

If non-compressive vertical stress appear at the heel of the dam (upstream side), cracks are considered to appear. The reason for this assumption is that concrete and rock are brittle materials and that cracks are often present in the rock mass and they will open up easily under tensile stresses.

A common assumption is that full uplift pressure may be assumed along the whole crack length. Design according to USACE (2003) should be based on full uplift pressure for the part of the base that is not in compression. If that part stretches beyond the drains, they should be considered to malfunction.

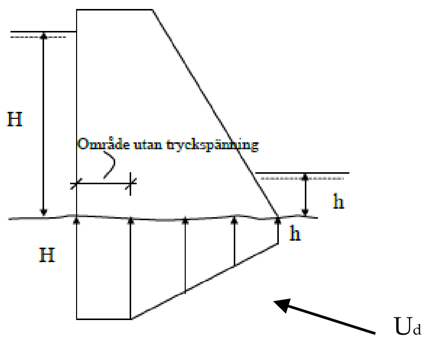


Figure PII- 5-7. Design assumption if non-compressive stresses exist in the heel. (RIDAS, 2012)

II:5.2 BASIC MODEL

The resultant force of uplift pressure is denoted U and resulting moment of uplift pressure is denoted U_m. They are described as

$$U = U_d \cdot C \qquad \text{(PII. 5-2)}$$
$$U_m = U_{dm} \cdot C_m$$

Where U_d and U_{dm} are the resultant force and resulting moment of the linearly decreasing uplift usually assumed in design and C and C_m are random variables. U_d for different design assumptions are shown in Figures PII-5-1 to PII-5-5 and in Figure PII-5-7 C_m includes variability in both force and resultant location.

II:5.2.1 Variability of uplift

The variability of uplift pressure (in terms of year-maximum values) is not known. Literature on the subject is very limited. Westberg (2007) performed simulation of uplift based on the hydraulic conductivity of the rock mass, and from this the variability of uplift was obtained.

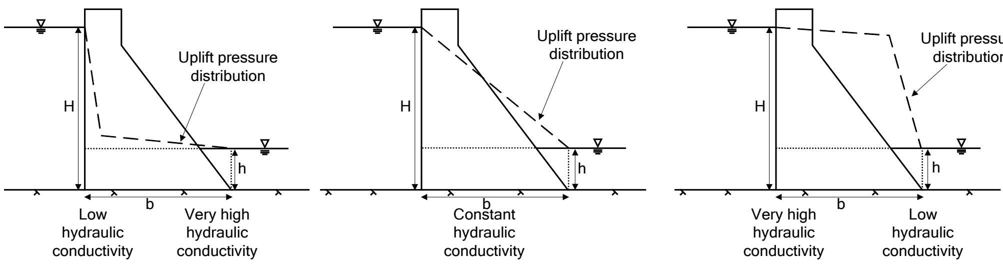
In Westberg (2007) different assumptions on variance and correlation with distance of the hydraulic conductivity K was investigated. For large variance, or large range (or both), uplift may become very high or very low (illustrated also by PII-5-4). These extremes are considered possible for cases of bad rock where long horizontal joints lead water in beneath the dam. During the period when most Swedish dams were constructed, knowledge on foundation geology and foundation treatment was good. It may thus be considered that for bad geological conditions a gravity structure without drainage tunnel and drainage holes would not have been built. For structures without drains it may be expected that the variability of uplift around U_d is relatively small. A mean value of 1 and a coefficient of variation of 5% of C and C_m may be assumed.

If geological conditions suggest that the rock mass consists of heavily fractured rock, or other conditions suggest that other assumptions of variance are more appropriate, they should be used.

When drains are present the variation can be expected to be higher. Based on simulations by Westberg (2007) a variance and range of the hydraulic conductivity of (ln(K))= 16 and Range = 12 m (correlation distance) is assumed (see further information in Westberg 2007). The result is then that C is described by a Beta distribution with parameters (1.96; 1.95) and C_m by a Beta distribution with parameters (2.22; 1.33).

For a gravity type structure without drains C may, due to physical limitations, never become less than 0 and never larger than 2. The physical limitation is set up by the fact that the lowest resultant force of uplift pressure is 0 and the maximum resultant force is when there is full pressure beneath the whole surface, resulting in the force to be twice that of the commonly assumed linear case, see Figures PII-5-8 and PII-5-9.

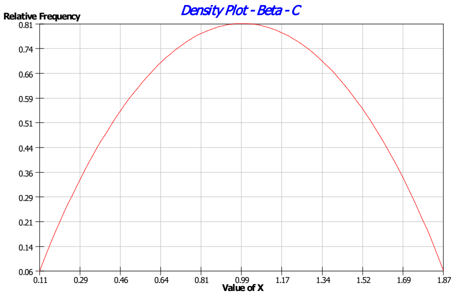
The limits for C_m for a gravity type structure are 0 and 1.5. The distribution for C_m thus includes both the variation in total force and the resultant location. For practical reasons it may be assumed that limits are for C [0.08-1.9] and for C_m [0.11-1.4]. For gravity type structures with drains the limits are somewhat different, but use of the same C and C_m is conservative and may be used. For buttress type structures C is the same, while limits for C_m depends on the width and length of the front plate, and width and length of the column according to Figure PII-5-9. It must be noted, however, that C and C_m were based mainly on simulations for a case with no drains, hence more careful calculations for a specific structure may give better estimates.



$U_{\min} = U_{tail} + U_L = U_{tail} + 0 \cdot u$ $\quad$ $U = U_{tail} + U_L = U_{tail} + u$ $\quad$ $U = U_{tail} + U_L = U_{tail} + u \cdot 2$

$M_{\min} = M_{tail} + M_L = M_{tail} + 0 \cdot m$ $\quad$ $M = M_{tail} + M_L = M_{tail} + m$ $\quad$ $M = M_{tail} + M_L = M_{tail} + m \cdot 1{,}5$

Figure PII- 5-8. Uplift distribution resulting in minimum uplift force and moment, the linear case and uplift distribution resulting in maximum uplift force and moment.



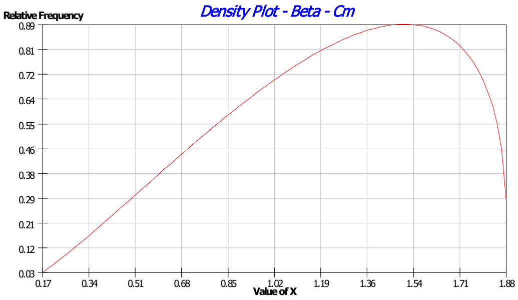


Figure PII- 5-9. Density plot (statistical distribution) of force of uplift and moment of uplift.

The complete description of C and C_m thus becomes

For structures without drains

C ~N (1.0; 0,05)

C_m ~N(1;0.05)

For structures with drains

C ~Beta (1.96; 1.95; 0.08; 1.9)

C_m ~Beta (2.22; 1.33; a_1; a_2)

Where a_1; a_2 = 0.11; 1.49 for gravity type structures and from Figure PII-5-9 for buttress type structures

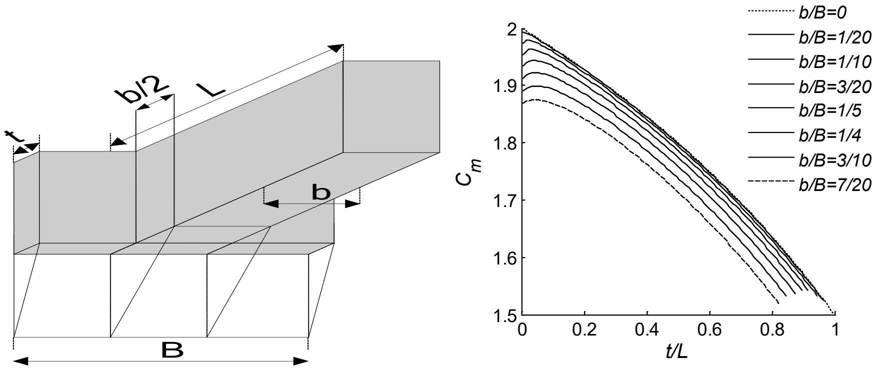


Figure PII- 5-10 a) uplift distribution assumed for buttress dam b) limits of Cm for varying t and b

II:5.3 ITERATION OF UPLIFT PRESSURE

Stresses in the base of the dam should be calculated. If non-compressive stresses appear, the part in non-compression should be considered to have full uplift pressure. In that case, the following equations apply:

$$U = U_c + U_d \cdot C \qquad \text{(PII. 5-3)}$$
$$U_m = U_{cm} + U_{dm} \cdot C_m$$

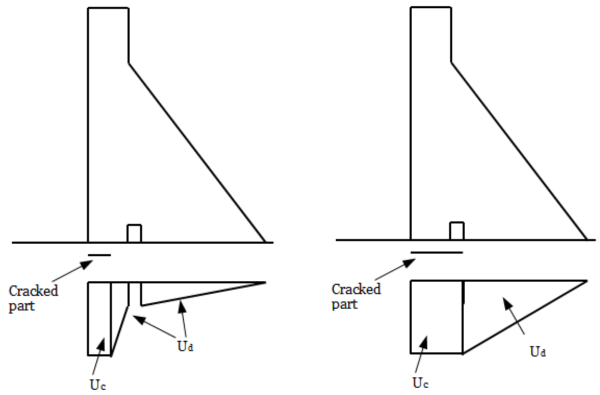


Figure PII- 5-11. Definition of Uc and Ud depending on length of cracked part.

II:5.4 REFERENCES

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II:6 Earth pressure and soil properties

II:6.1 INTRODUCTION

When a concrete dam has backfill consisting of earth or rock fill, earth pressure at rest shall be assumed. When movement occur towards the fill, the pressure will increase from earth pressure at rest until it reaches the maximum passive earth pressure. When movement occur away from the fill, the earth pressure will decrease until it reaches the active pressure. Before movement occurs the pressure is, on the other hand, equal to the earth fill pressure at rest.

It should be noted that earth pressure is an internal pressure that depends on parameters such as density and the shear strength of the soil. It arises when gravity mobilizes the weight of the soil and its shear strength. An analytical model is often used to describe this relation, which also give rise to a model uncertainty. If the earth pressure is integrated over the surface, the total internal force acting on the dam is obtained. If the components in the system: dam, water and soil etc. are separated and only consider the dam as a rigid body, the earth pressure can be considered as an action (load) on the dam.

As mentioned above, the degree of mobilized earth pressure depends on the degree of mobilized shear strength in the soil and varies with the deformations; as a consequence, it can be both an action (active pressure) and a resistance (passive pressure). However, when the stability of the dam is evaluated, the earth pressure at rest is used since it is uncertain how the other loads and resistances in the system vary with the deformation. This is a conservative assumption and is one of the reasons why no model uncertainty has been given to the earth pressure in the present version of this code.

Density and coefficient of earth pressure may be taken from tests or from the information below.

Table PII- 6-1. Prior indications of Soil properties of non cohesive soils. From JCSS (2001).

Non cohesive soil type	Density	Dry Unit Weight, γ [kN/m^3]	Saturated Unit Weight, γ' [kN/m^3]	Internal Friction angle, ϕ' [°]
Coarse gravel, Boulders	loose medium dense	15-17 17-18 18-20	19-20 20-21 21-23	33-36 35-40 38-42
Sand, gravel Uniform grain Size	loose medium dense	15-16 17-18 18-19	19-20 20-21 21-22	30-33 33-36 35-40
Sand, gravel Non-uniform Grain size	loose medium dense	17-19 18-20 20-21	20-22 21-23 22-24	30-35 32-37 35-40
Sand Slightly Silty Silty		18-20 18-20	20-21.5 19.5-20.5	27-33 24-31

Table PII- 6-2. Prior indications of Soil properties of cohesive soils From JCSS (2001)

RIDAS 2012, Bygg 173:5 (1961)

Cohesive Soil type	Consist-ency	Saturated Unit Weight, γ' [kN/m^3]	Internal Friction angle, ϕ' (drained) [°]	Cohesion. c' (drained) [kN/m^2]	Undrained Shear Strength, c_u [kN/m^2]
Inorganic cohesive soils, Plastic	soft	16-18	15-20	0-5	10-20
	stiff	17-19	15-20	5-15	20-50
	very stiff	20-22	15-20	15-30	50-100
Inorganic cohesive soils, Medium plastic	soft	17-19	19-23	0-5	0-10
	stiff	18-20	19-23	5-10	15-30
	very stiff	19-21	19-23	10-20	40-100
Inorganic cohesive soils, Weakly plastic		18-20	22-31	0-5	0-10
Boulder clay		20-24	27-33	20-30	-
Organic cohesive soils, silt	soft	13-18	13-16	0-5	5-20
	stiff	14-19	13-16	5-10	15-30

II:6.2 BASIC MODEL

RIDAS 2012, (Handboken Bygg Geoteknik, 1984)

The total earth pressure at rest is in general given by

$$S_0 = K_0 \cdot \sigma_v' + u_w$$

Where S_0 is the pressure, K_0 is the earth pressure coefficient at rest, σ_v' is the effective pressure and u_w is the water pressure. (Handboken Bygg, 1984).

For horizontal ground K_0 is given by

$$K_0 = 1 - \sin\varphi \qquad \text{(PII. 6-1)}$$

For inclined ground $K_{0inclined}$ is given by

$$K_0 = \frac{K_{Ainclined}}{K_{Ahorizontal}} \qquad \text{(PII. 6-2)}$$

Calculation of $K_{Ainclined}$ and $K_{Ahorizontal}$ is e.g. described G05:3 in Handboken bygg (1984). For inclined wall calculation method is also described in Handboken bygg (1984)

The picture below show an example of earth pressure on vertical wall. In this example $q+\gamma h_1+\gamma h'_2$ is the effective pressure σ_v'.

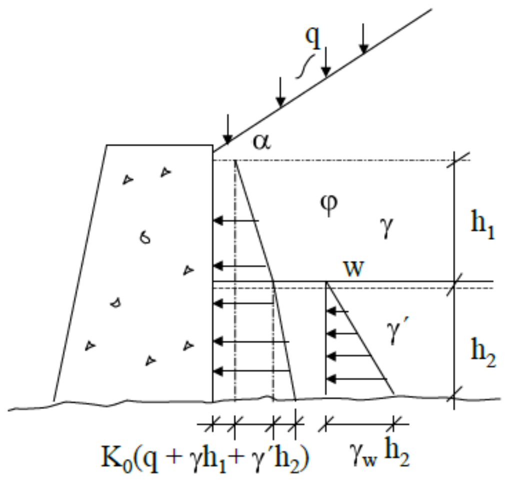

Figure PII- 6-1. Actions from water and earth fill on a vertical structure. Inclined ground. (from RIDAS 2012).

II:6.3 PROBABILITY DENSITY FUNCTIONS

If tests are not available the density may be assumed to have a Normal distribution with mean value E from Table PII-6-4 and coefficient of variation

Energiforsk

V_ρ from Table PII-6-4 (standard deviation $\sigma = V_\rho \cdot E$), based on the information in Table PII-6-2 to Table PII-6-3.

The internal friction angle tan ϕ may be assumed to have a normal distribution with mean value E from Table PII-6-4 and coefficient of variation V_ϕ from Table PII-6-5 (standard deviation $\sigma = V_\phi \cdot E$).

Table PII- 6-3.Mean values of density and friction angle to be used, based on JCSS (2001).

Non cohesive soil type	*Density*	*Dry Unit Weight [kN/m3]*	*Saturated Unit Weight [kN/m3]*	*Internal Friction tan(ϕ')*	*Internal friction angle [°]*
Coarse gravel, Boulders	loose medium dense	16 17.5 19	19.5 20.5 22	0.69 0.76 0.84	35 37 40
Sand, gravel Uniform grain Size	loose medium dense	15.5 17.5 18.5	19.5 20.5 21.5	0.61 0.69 0.76	31 35 37
Sand, gravel Non-uniform Grain size	loose medium dense	18 19 20.5	21 22 23	0.63 0.68 0.77	32 34 38
Sand Slightly Silty Silty		 19 19	 20.5 20	 0.57 0.52	 30 28
Moraine (till)	dense	21	23	0.68	34*

*From RIDAS (2012)

Table PII- 6-4.Standard deviations of soil properties to be used, based on JCSS (2001)

Soil property	V_X
Unit weight [kN/m3]	0,10
Internal Friction tan(ϕ') (drained)	0,15
Earth pressure coefficient at rest K_0 = 1-sin(ϕ)	0,15

Table PII- 6-5. Indicative standard deviations of soil properties.

Soil property	Standard deviation [% of expected mean value]
Unit weight [kN/m3]	5 – 10 %
Internal Friction tan(ϕ′) (drained)	10 – 20 %
Drained Cohesion [kN/m2]	10 – 50 %
Undrained Shear Strength [kN/m2]	10 – 40 %

II:6.4 REFERENCES

Joint Committee on Structural Safety (JCSS). (2001). Probabilistic model code.

RIDAS 2012, Kraftindustrins Riktlinjer för Dammsäkerhet. Svensk Energi.

Handboken Bygg Geoteknik (1984)

Part III: Resistance models

III:1 Resistance parameters

III:.1 GENERAL PRINCIPLES

The structural resistance is dependent on material properties.

III:.2 INTRODUCTION

JCSS 2001, part 3
3. 0.1

The description of each material property consists of a mathematical model (e.g. elastic-plastic model, creep model, etc.) and random variables or random fields (e.g. modulus of elasticity, creep coefficient). Functional relationships between the various variables may be part of the material model (e.g. the relation between tensile stress and compressive stress for concrete).

In general, it is the response to static and time dependent mechanical loading that matters for structural design. However, also the response to physical, mechanical, chemical and biological actions is important as it may affect the mechanical properties and behaviour.

It is understood that modelling is an art of reasonable simplification of reality such that the outcome is sufficiently explanatory and predictive in an engineering sense. An important aspect of an engineering model also is its operation ability, i.e. the ease in handling it in applications.

Models and values should follow from (standardised) tests, representing the actual environmental and loading conditions as good as possible. The set of tested specimen should be representative for the production of the relevant fabrication sites, cover a sufficient long period of time and may include the effect of standard quality control measures. Allowance should be made for possible differences between test circumstances and structural environment (conversion).

III:.3 MATERIAL PROPERTIES

Partly JCSS 2001, part 3 3.0.1

Material properties are defined as the properties of material specimens of defined size and conditioning, sampled according to given rules. Specimens are then and subjected to an agreed testing procedure, the results of which are evaluated according to specified procedures.

Several different properties may have to be described, e.g. modulus of elasticity, strength of the material, strain at rupture (local phenomenon that may heavily depend on the shape and dimensions of specimen), temperature effects, humidity effects, etc.

In general, the various properties of one material may be correlated.

III:.4 LIMITATIONS

Spatial correlation in interface are not included (they are considered to affect the result, but the information at this point is not sufficient)

Cohesion is not included for in evaluation of shear resistance. The reason is difficulties in the description of cohesion and large uncertainties.

III:2 Self-weight

III:.1 SELF-WEIGHT OF BUILT STRUCTURES

This section relates to the self-weight of concrete structural components. The information given below and assumptions regarding distribution type, mean value and variability may be used as prior assumptions in most cases. When investigations indicate possibility of other material properties, e.g. when the concrete is severely leached, testing may be necessary. Inclusion of test results is described in section 11 of Part I.

Self-weight may be regarded as a load, or as a resistance parameter, depending on the problem at hand. For information of different material, JCSS (2001) give detailed information.

III:.1.1 Introduction

The main characteristics of the self-weight are that variability with time is normally negligible (permanent action).

Uncertainties of the magnitude are normally small in comparison with other kinds of loads. Concerning the uncertainties one can distinguish between

- Variability within a structural member
- Variability between different structural parts of the same structure

The variability within a structural part is normally small and can then be neglected. However, for some types of problem (e.g. static equilibrium) it may be important.

III:.1.2 Basic model

The self-weight G of a structural part is determined by the relation

$$G = \int_{Vol} \gamma \, dV \qquad \text{(PIII. 2-1)}$$

where V is the volume described by the boundary of the structural part. The volume of V is Vol. and γ is the weight density of the material.

III:.1.3 Probability density distribution functions

The weight density and the dimensions of a structural part are assumed to have Gaussian (normal) distributions. To simplify the calculations the self-weight, G, may as an approximation be assumed to have a Gaussian (normal) distribution.

III:.1.4 Weight density

The mean values and coefficient of variation for weight density of different materials can be found in JCSS (2001). Recommendations for basic assumption for weight density of concrete dams are given below (from CIB W81).

Table PIII 2-1. Basic assumptions on weight density, JCSS (2001).

Material	Mean value (kN/m³)	Coefficient of variation
Ordinary concrete (without reinforcement), f_{cc} = 20 MPa	23,5	0,04
Ordinary concrete (without reinforcement), f_{cc} = 40 MPa	24,5	0,04

For large structures the variability of the global weight density may be taken as $V_G \cdot \rho_o$ and for structures consisting of many members it may be taken as $V_G \cdot \rho_m$. ρ_o is 0,85 and ρ_m 0,70.

For dams the coefficient of variation may be taken as $V_\gamma = 0{,}04 \cdot 0{,}85 = 0{,}034$.

For large structures such as dams it is possible that the variance reduction may give even smaller variability, and the above recommendation is a conservative assumption.

III:.1.5 Volume

The mean value of the volume is calculated directly from the mean values of the dimensions.

The table below show mean values and standard deviations for deviations of cross section dimensions from their nominal values (JCSS, 2001).

Table PIII 2-2.Cross sectional dimensions, JCSS (2001).

Structure or structural member	Mean value	Standard deviation
Concrete members		
a_{nom}<100 mm	0,03 a_{nom}	4+0,06a_{nom}
a_{nom}>1000 mm	3 mm	10mm

Information for other materials may be found in JCSS, 2001).

III:.2 SELF-WEIGHT OF SOIL

Self-weight of soils are treated in section 7 (earth pressure) of Part II.

III:.3 SELF-WEIGHT OF ROCK

Density of rock γ_m is dependent on mineral composition and porosity.

The mean value and standard deviation for the density of rock types at the Forsmark area are presented in Table 1 (SKB 2005). These are based on intact rock. The density for some other types of rock is presented in Table 2 (Parasnis 1951).

Table PIII 2-3. Mean value and standard deviation of the density γ_m for rock types at the Forsmark area (SKB 2005).

Rock Type	Mean value (t/m³)	Standard deviation (t/m³)
Amphibolite	2,988	0,060
Diorite, quartz diorite and gabbro, metamorphic	2,934	0,10
Granodiorite, metamorphic	2,689	0,018
Granite (to granodiorite), metamorphic, mediumgrained	2,657	0,015
Pegmatitic granite, pegmatite	2,627	0,006
Granite, fine to mediumgrained	2,638	0,009

Table PIII 2-4. Mean value of the density γ_m for different types of rock (Parasnis 1951).

Rock Type	Mean value (t/m³)
Silurian Limestones and shales	2,70
Old red sandstone	2,52
Carboniferous limestone	2,60
Middle Coal Measures Shales and Sandstones	2,48
Keuper Sandstone	2,27
Keuper Marl	2,42
Upper Lias	2,34
Chalk	1,94
Malvernian Gneiss	2,69

Based on this, but accounting for the increased variation due to different mineral composition and porosity, variation between sites etc., the rock mass density for hard crystalline rock such as Granite and Gneiss, can be assumed to be normal distributed having $N\epsilon(2{,}65;0{,}054)$ t/m³.

Due to possible variations in mineral composition and porosity, it is recommended that the density of the rock mass is determined on a site specific basis for other types of rocks than Granite and Gneiss.

III:.4 REFERENCES

CIB W81, Actions on structures, Self weight, Report no. 115, Rotterdam

JCSS (2001) Probabilistic model code.

Parasnis 1951 A study of rock densities in the English midlands. Geophysical Supplements to MNRAS, Vol. 6, Issue 5, pp. 252-271.

SKB (2005) Preliminary Site Description Forsmark Area – Version 1.2. Report R-05-18.

III:3 Friction angle of concrete-rock interface

III:.1 INTRODUCTION

The following section is from Gustafsson et al. (2008). The shear strength for the concrete-rock interface has been studied by Lo and Gras (1994), EPRI (1992), Lo et al. (1991a and 1991b) Lo et al. (1990) among others.

The general opinion among these researchers is that the concrete-rock interface can exhibit a relatively high cohesion and tensile strength if the bond is intact. For an interface with cohesion the failure occurs as a brittle failure, without any sliding (relative movement). At the point of failure, the shear strength can be described with the Mohr-Coulomb failure criterion with cohesion, c_c, and an internal friction angle, $\phi_{i,c}$. On the other hand, if no bond exists or the intact bond has been broken, no cohesion exists and the total friction angle can be expressed as the sum of two components, a basic friction angle from a macroscopic smooth but microscopic rough surface, $\phi_{b,c}$, and a dilation angle, i_c, that originates from the inclination of larger asperities in the concrete-rock interface. This means that failure can occur at different degrees of deformation, without any relative deformations (failure of the intact bond) and with a relative deformation in range of a few millimeters to a few centimeters if no intact bond exist.

The shear strength can therefore be divided into two separate cases; when an intact bond exists and cohesion is accounted for and when the bond is broken and no cohesion is included. Even though cohesion may exist in the interface, the uncertainties associated with this parameter are large. Furthermore, the Swedish power company's guidelines for dam safety, RIDAS (ELFORSK 2011), does not recommend that cohesion should be included in the shear strength. In this model code, a fully broken bond is assumed and no cohesion is included in the shear strength, as is also described in the limit state equations in Part I.

There is most likely a spatial variability in the statistical distribution of the shear strength. In short spatial variability is the "similarity" in properties between points located close to each other, whereas points located further away may be expected to have "less in common". Spatial variability has large effect in many cases, e.g. in soil parameters. The extent of spatial variability in frictional properties of the interface is not known. If the correlation length is small the effect is a variance reduction. In this case there is not enough information of the correlation length to assess the possible variance reduction, and for that case spatial variability is not considered. This is a conservative assumption and if testing reveal spatial variability this may be included in the analysis.

III:.2 BASIC MODEL

For a broken concrete rock interface the total friction angle, ϕ_{tot}, can be described with the following equation.

$$\emptyset_{tot,c} = \emptyset_{\mathrm{b,c}} + i_{\mathrm{c}}$$

Where $\phi_{b,c}$ is the basic friction angle for a macroscopic smooth but microscopic rough surface and i_c is the contribution from large scale asperities in the interface.

Lo and Hefny (1998) summarized the results of measurements of ϕ_b for different types of 324 contact and stated that the basic friction angle of concrete-rock contacts is not very sensitive 325 to rock type and varies from 30° to 39°.

The dilation angle that originates from large scale asperities requires knowledge about the surface roughness. According to Lo et al. (1991b), the inclination angle from large scale asperities can be based on measurements of the rock surface at the time of construction. In order to be able to include the contribution from a single asperity, it has to be sufficiently large to prevent shearing through the intact rock asperity or through the concrete. This implies that the size of the asperities that could be assumed to contribute to the shear strength is connected to the height of the dam. For normal Swedish buttress dams with a height of 10-30 m it is considered that the rock asperity should have a length of at least 5% of the height of the dam. For gravity dams, the required length of the asperity could be smaller if it has a larger extension perpendicular to the stream direction.

III:.3 PROBABILITY DENSITY FUNCTIONS

$\phi_{b,c}$ can be assumed to have a mean value of 35° and a standard deviation of 1,75)° based on the results of Lo & Hefny (1998) previously described and a coefficient of variation of 5%. Shear tests could be performed to obtain additional information about the residual friction angle for the concrete-rock contact. In a study by Krounis (2016) $\phi_{b,c}$ was shown to be the most influential parameters of the system considered. Calibrations in Westberg Wilde & Johansson (2016) also indicate this and hence shear tests are recommended. The methodology for doing this is described in Lo et al. (1991b). If such tests are performed, the distribution could be updated using the Bayesian statistics as described in Section 8 in Part I.

If no tests are performed the tangent friction angle may be assumed to follow a normal distribution with $\tan(\phi_{b,c}) \epsilon N(0{,}7;0{,}031)$.

The dilation angle, i_c, for a blasted rough surface can be assumed to have a mean value of 15° and a standard deviation of 3°. This value has to be supported by measurements of the inclination of larger asperities at the rock surface. For existing dams, this could be achieved from measurements available on construction drawings done at the time of construction. If outcrops of rock surfaces exist in spaces under or adjacent to the dam, complementary measurements could also be done there.

If the dam was founded on smooth surfaces, or if it can´t be verified that the dam is founded on a blasted rough surface, the dilation angle i_c may be assumed to have mean value 5° and standard deviation 1°.

Without further tests the following assumptions are recommended:

For a blasted rough surface: $\tan(i_c) \in LN(0{,}268;0{,}0524)$.

For a smooth surface: $\tan(i_c) \in LN(0{,}087;0{,}0175)$.

For practical purpose, addition of angles may be done according to

$$\tan(\phi_{b,c} + i_c) = \frac{\tan(\phi_{b,c}) + \tan(i_c)}{1 - \tan(\phi_{b,c}) \cdot \tan(i_c)}$$ (trigonometry identity).

III:.4 REFERENCES

Electrical Power and Research Institute, EPRI, (1992). Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams, Volume 1, Rapport nr: EPRI TR-100345.

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Lo K.Y., Grass J.D., (1994), Recent Experience with Safety Assessment of Concrete Dams on Rock Foundations, Dam Safety PP. 231-250.

Westberg Wilde, M. & Johansson, F. (2016) *Sannolikhetsbaserad evaluering av betongdammars stabilitet*. Energiforskrapport. In press.

III:4 Friction angle of rock fractures

III:.1 INTRODUCTION

The total friction angle of rock joints is complex parameter to describe. The joints can exist as unfilled but they can also have infilling material such as sand and clay. For and unfilled joint, the friction angle depends on the joint surface roughness, the joint wall compressive strength and the applied normal stress. In addition to this, the friction angle is also dependent on the scale (sample size) together with the matedness between the upper and lower part of the joint. Due to the influence from these parameters on the total friction angle for rock joints, it is often associated with large uncertainties.

III:.2 BASIC MODEL

For an unfilled persistent rock fractures under the monolith the total friction angle, $\phi_{\text{tot,F}}$, can be expressed as

$$\emptyset_{\text{tot,F}} = \emptyset_{b,F} + i_F$$

Where $\phi_{b,F}$ is the residual friction angle for a macroscopic smooth but microscopic rough surface and i_F is the contribution from the surface roughness of the joint.

For filled rock fractures the friction angle is determined by the infilling material if the thickness of the infilling material is larger than 1,0-1,4 times the amplitude of the asperities of surface roughness (CEATI 1998).

There exist different methodologies to estimate the friction angle of a rock fractures. The most reliable way is to perform in-situ shear tests (ISRM 1981). However, this if often difficult and also expensive and time-consuming. More common is to perform laboratory shear tests and correct the results with respect to scale. Scale corrections could be made according to Barton and Bandis (1982) and Johansson & Stille (2014). Another method, which is the most common one, is to use Barton and Choubey (1977) empirical failure criterion. If their criterion is used, it is recommended that the contribution from roughness is estimated based on tilt tests and not through the predefined roughness profiles given by Barton & Choubey.

Due to the uncertainties associated with the estimation of the total friction angle, it is recommended that it is estimated based on in-situ or laboratory shear tests. No general value for the total friction angle of rock fractures can be recommended. Instead, this has to be determined from site-specific conditions by experts in the field of engineering geology and/or rock mechanics.

III:.3 PROBABILITY DENSITY FUNCTIONS

$\phi_{b,F}$ could be assumed to be normal distributed and i_F could be assumed to be log-normal distributed. Mean value and variance is recommended to be determined from shear tests. Updating is described in chapter 10 of Part I.

The natural inherent variability of both $\phi_{b,F}$ and i_F is relatively sparsely investigated. Johansson et al. (2010) performed fourteen shear tests, seven with a scale of 120 by 120 mm and seven with a scale of 240 by 240 mm, on the same joint under the same normal stress. These results exhibited a mean value of 35,3° with a standard deviation of 2,5°, which gave a coefficient of variation $V_{\phi b,F}$ of 0,07. The mean value for the dilation angle was 6,6° with a standard deviation of 2,9°, which gave a $V_{i,F}$ of 0,43. Based on these results, a reasonable estimate for the inherent variability of the residual friction angle and the joint roughness angle appears to be a $V_{\phi b,F}$ 0,07 and a $V_{i,F}$ =0,40.

III:.4 REFERENCES

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III:5 Compressive and tensile strength of concrete

III:.1 INTRODUCTION

For concrete the compressive resistance may be estimated according to Sustainable Bridges (2007). Strength increase due to ageing may also be included. The characteristic value of compressive strength, f_{ck}, is usually known and the relation between mean value, f_{cm}, and characteristic value is

$$f_{cm} = f_{ck} \cdot \exp(1.64 \cdot V_{fc}) \quad \text{(PIII. 5-1)}$$

where V_{fc} is the coefficient of variation. If V_{fc} is not known σ_{fc} = 5 MPa may be representative. The in-situ 28 day strength may be calculated as

$$f_{cm,is} = \kappa \cdot f_{cm} \quad \text{(PIII. 5-2)}$$

where $\mu(\kappa) = 0.85$ and $V_\kappa = 0.06$.

The coefficient of variation $V_{fc,is}$ can be determined from

$$V_{fc,is}^2 = V_{fc}^2 + V_\kappa^2 \quad \text{(PIII. 5-3)}$$

The increase in compressive strength due to ageing (up to a certain age) is dependent on cement type, temperature and curing conditions. CEB-FIP Model Code (1990) recommends the following formula for compressive strength at age t (days):

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad \text{(PIII. 5-4)}$$

where

$$\beta_{cc}(t) = \exp\left[s\left(1 - \left(\frac{28}{t}\right)^{1/2}\right)\right] \quad \text{(PIII. 5-5)}$$

s is a coefficient depending on cement type (0.2 for rapid hardening high strength, 0.25 for normal and rapid hardening and 0.38 for slow hardening). According to Sustainable Bridges (2007) predictions based on eqn PIII 5-4 are very conservative for very old concretes.

The uncertainty in the aging model may be taken as

$$\sigma_{\beta cc} = 0.3 \cdot (\beta_{cc}(t) - 1) \quad \text{(PIII. 5-6)}$$

The coefficient of variation $V_{fc,t}$ at age t is then

1. $$V_{fc,t} = \sqrt{V_{fc,28}^2 + V_{\beta cc}^2} \quad \text{(PIII. 5-7)}$$

The concrete compressive strength is assumed to have a lognormal distribution.

The mean tensile strength in bending is, according to Carlsson et al. (2007) given as

$$f_{ctm} = 0.3 \cdot f_{ck}^{2/3} \text{ [MPa]} \quad \text{(PIII. 5-8)}$$

III:.2 BASIC MODEL

In situ compressive strength with ageing effects included should be utilized.

Caution must be taken when the structure has signs of deterioration or for very old structures where the concrete was stamped during construction, or where very low concrete quality was used.

III:.3 PROBABILITY DENSITY FUNCTIONS

A lognormal distribution may be assumed for f_c and f_{ct}.

Coefficient of variation of 0.3 may be assumed for tensile strength. For compressive strength it may be calculated according to PIII. 5-7.

III:.4 REFERENCES

Sustainable Bridges (2007a): "Guideline for Load and Resistance Assessment of Existing European Railway Bridges: Advices on the use of advanced methods". Prepared by Sustainable Bridges - a project within EU FP6. Available from: www.sustainablebridges.net

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III:6 Uniaxial compressive strength of rock mass

According to section 7.2.2 in Part I no recommendations for the limit state concerning failure in the rock mass are given. Probability density distribution functions for the parameters that describe the shear resistance for the rock mass are not given here.

The only parameter used in this model code related to the rock mass is the uniaxial compressive strength, σ_{cm}, used to determine the length of the crushed zone for adjusted overturning.

The uniaxial compressive strength of rock masses vary depending on rock type and quality; their strength is therefore associated with uncertainties. The shear strength of rock masses depends on the intact rock material and the characteristics of the joints, such as orientation, length, joint roughness and infilling material among other parameters. A description of the rock mass strength is beyond the scope of this model code. A methodology for the estimation of rock mass strength can be found in Trafikverkets Projekteringshandbok (Lindfors et al, 2015).

III:.1 REFERENCES

Lindfors, U., Swindell, R., Rosengren, L., Holmberg, M. & Sjöberg, J. 2015. Projektering av bergkonstruktioner. Stockholm: Trafikverket.

III:7 Rock bolts

III:.1 INTRODUCTION

Rock bolts (anchor rods) are normally present in the concrete/rock interface. It is common that a significant number of rock bolts were placed in this zone before concrete was cast.

Ekström et al (2013) describe the different failure modes:

1. Failure in the rock
2. Failure in the contact between rock and grout
3. Failure in the contact between grout and steel
4. Failure in the steel
5. Failure in the contact between steel and concrete

Rock bolts give significant contribution to the overturning stability as well as the sliding stability, but they are inaccessible and it is not possible to investigate their status.

For rock bolts that are partly accessible it may be possible to measure the steel to determine if the steel is intact. It may also possible to detect larger defects such as holes, corrosion, bad bond between anchor and grout if the process has developed.

According to RIDAS (2012) rock bolts may be included in stability calculations

- For low consequence dams.
- When the failure criteria for shear is not reached ($\tan \phi < 1$) (when bolts are not included).
- When the resultant of forces is within the base area (when bolts are not included).

For high consequence dams rock bolts are not included in calculation due to the uncertainties in actual capacity since their status cannot be investigated and verified.

In this guideline the recommendations according to RIDAS should be followed, but the below assumptions should be applied in calculations.

III:.2 BASIC MODEL

Equations describing the failure modes may be found in Ekström et al (2013). The failure mode giving the lowest capacity should be used. When several failure modes give similar results, Monte Carlo simulation may be necessary for calculation of β. The validity of FORM-calculation should be determined by comparative MC-simulations.

III:.3 PROBABILITY DENSITY FUNCTIONS

Mean value of the volume of affected rock may be estimated according to figure 2-4 in Ekström et al (2013) and coefficient of variation may be assumed to be 0,1. Normal distribution may be assumed.

Ekström et al (2013)

JCSS (2001), PIII, 3

Self-weight of rock, see section III:.3.

Cohesion in rock as well as adhesion between rock and grout as well as between grout and steel has to be determined case-specifically as both depend on properties of grout, rock and steel. Coefficient of variation may be assumed to be 0,2 as better information is not available. Lognormal distribution may be assumed. Description related to PDF need to be further investigated.

Strength of steel is given according to JCSS (2001) and the mean value is given by

$$f_y = f_{yk} \cdot e^{-u \cdot v} - 20\ MPa \qquad \text{(PIII. 7-1)}$$

Where f_{yk} is the characteristic yield strength, u is -1,64 for f_{yk} from BBK 04 and Eurocode (lowest 5% fractile). v is the coefficient of variation and is set to 0,07. A lognormal distribution may be assumed.

The length of the steel in concrete and rock respectively, may be taken as the nominal length (from drawings) assuming a coefficient of variation of 0.1 and a normal distribution. Due to the uncertainty in the number of intact bolts, it is strongly recommended that a parametric study should be performed, where the significance of the ratio of intact bolts to total number bolts is investigated. If the ratio is high, meaning that most bolts need to be intact, further analysis and discussion on the problem is necessary.

Concrete tensile strength may be determined according to section III:.10.

III:.4 REFERENCES

Sustainable Bridges (2007a): "Guideline for Load and Resistance Assessment of Existing European Railway Bridges: Advices on the use of advanced methods". Prepared by Sustainable Bridges - a project within EU FP6. Available from: www.sustainablebridges.net

Ekström, T., Hassanzadeh, M., Janz, M., Sederholm, B., Stojanovic, B. & Ulriksen, P. (2013). *Tillståndsbedömning av förankringsstag i dammar.* Elforskrapport 13-70.

III:8 Rock anchors

III:.1 INTRODUCTION

Rock anchor are permanent or temporary anchors where the load capacity is tested. An anchor consists of an anchor head, a free anchor length and a fixed anchor length which is bonded to the ground by grout.

SSEN 1537

JCSS (2001)

Three types of steel are used as pre-stressing tendons in concrete structures:

- Cold drawn wires
- Strands
- High-strength bars.

III:.2 BASIC MODEL

The pre-stressing force applied to the structure may be expressed as

SSEN 1537

$$P(x,t) = P_0 - \Delta P(x,t) \quad \text{(PIII. 8-1)}$$

JCSS (2001)

Where P_0 is the jacking force and $\Delta P(x,t)$ losses of pre-stress. According to JCSS (2001) losses depend on the type of pre-stress (internal/external, pre/post-tensioning), properties of steel and concrete, environment etc.

The long-term pre-stressing force applied to the concrete at the time $t = \infty$ is expressed as

$$P(x,t) = P_0 - \Delta P(x,t_0) - \Delta P(x,\infty) \quad \text{(PIII. 8-2)}$$

Where $\Delta P(x,t_0)$ are the immediate loss (including losses due to elastic shortening of concrete, friction, short-term relaxation of pre-stressing steel and anchorage slip and $\Delta P(x,\infty)$ is the time-dependent loss (includes creep and shrinkage of concrete and long-term relaxation of the steel.

For a pre-stressed dam structure the load in the anchors is constant (aside losses) until loading equals resisting forces. After this point the resisting force in the anchor increase (as does deformation) up until the anchor reach its full capacity (failure in steel, grout, rock…) and the structure itself fails. At the point of failure, deformation in the dam body may be large and the loading situation may therefore be different from the initial configuration of loads (e.g. cracking in the heel may significantly increase uplift pressure, while movements may decrease/prevent ice loads). Due to the uncertainties in loading situation in the event of the actual failure, and after discussion in the reference group for the present project, the following is recommended:

The jacking force P_0 applied in analysis should be the largest of the testing force and the jacking force. It is common to apply a jacking force of approximately $0{,}6 \cdot P_{fail}$, but the steels may be tested to loads of up to $0{,}8\text{-}0{,}9 \cdot P_{fail}$ upon pre-stressing.

III:.3 PROBABILITY DENSITY FUNCTIONS

According to JCSS (2001) the variability of the jacking force is small as jacks used for pre-stressing are regularly calibrated. Variability may be estimated using data from the equipment producer and equipment calibration.

JCSS (2001)

EN 1992-1-1, 3.3.6

Statistical data to quantify uncertainties of the pre-stress losses are not available. JCSS (2001) present coefficients of variation for pre-stress loss for internally bonded tendons. This information may be used if other information is not available. In such case the mean value of pre-stress loss can be estimated in accordance with

SIS-EN 1537

Eurocode 1992-1-1:2005 (5.10.4-6) and the pre-stress loss may be treated as a normal random variable with COV as in Table PIII-8-1.

For typical magnitudes of pre-stress losses the corresponding coefficient of variations of pre-stressing forces has been estimated as in Table PIII-8-1.

Table PIII 8-1. Coefficients of variation of pre-stress losses and pre-stressing force. From JCSS (2001).

Parameter	Immediate, $t = t_0$	Long-term, $t = \infty$
Pre-stress losses, ΔP(x,t)	0.3	0.3
Pre-stressing force, P(x,t)	0.04-0.06	0.06-0.09

More information of pre-stressing and ground anchors may be found in SSEN 1537, EN 1992-1-1.

III:.4 REFERENCES

European Committee for Standardization (CEN). 2001. *Eurocode 2: Design of concrete structures – Part 1: Genaral rules for buildings. EN 1992*, Brussels, Belgium.

Joint Committee on Structural Safety (JCSS). (2001). Probabilistic model code.
SIS- EN 1537:2013 Execution of special geotechnical works - Ground anchors (2013)

IV Example of safety evaluation based on Probabilistic model code of concrete dams

The following example is a fictive dam with some relevant characteristics of an original dam. The purpose of the example is to show the reader how a probability-based assessment may be performed. The example gives reference to where information may be found in the Probabilistic model code (PMCD), but it is assumed that the reader is familiar with PMCD as well as the basis of probabilistic design.

The dam analyzed is a 18 m high concrete gravity dam. It has a front plate supported by concrete beams. It is founded on a blasted rock surface and has an inspection gallery founded directly on rock, but no drainage holes has been drilled into the rock.

The present structure also has pre-stressed anchors.

All the below calculations are shown for a section of width 1 m.

A drawing of the structure is shown below.

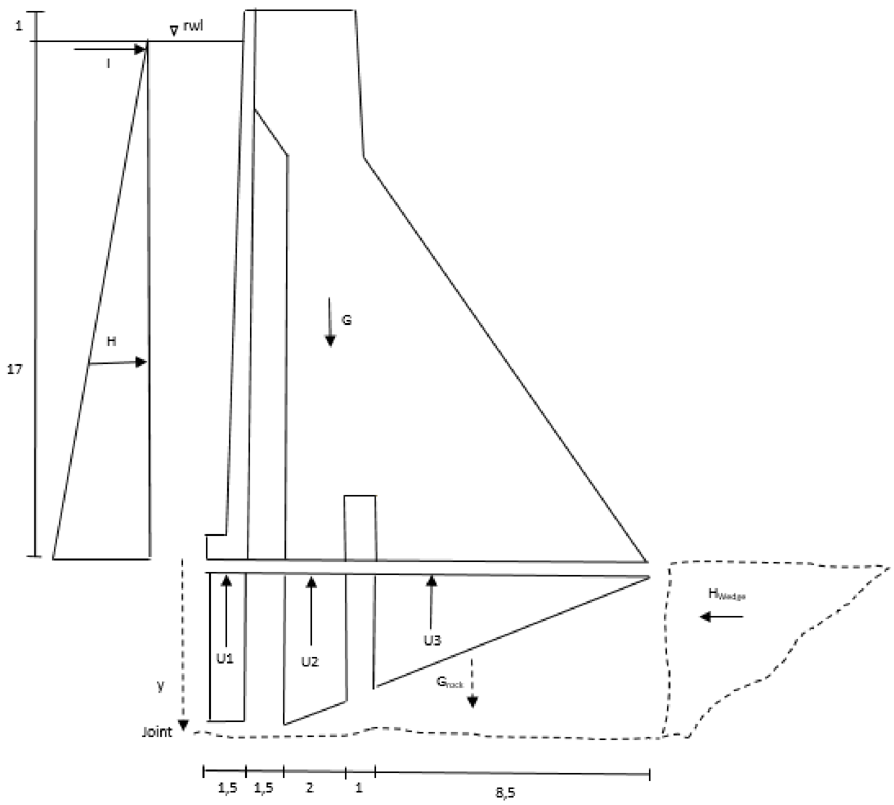

Figure E- 1. Cross Section in example 1.

IV:.1 LIMIT STATES AND DESIGN SITUATIONS

IV:.1.1 Limit states

The first step in a probabilistic design is to identify the limit states to be analyzed. In this case, as described in part I of PMCD the limit states are two ultimate limit states and one serviceability limit state.

The limit state functions are defined according to I:7.2.3 in PMCD Part I.

The ultimate limit states analyzed are:

Sliding

$$G_{Uc} = N' \cdot \tan\left(\phi_{b,c} + i_c\right) - H \qquad \text{Eqn. 1}$$

Adjusted overturning

$$G_O = M_R - M_S \qquad \text{Eqn. 2}$$

Failure in rock joint

$$G_{R_j} = N' \cdot \tan\left(\phi_{b,F} + i_F\right) - H \qquad \text{Eqn. 3}$$

The serviceability limit state analyzed is the appearance of tensile stress in the upstream face of the dam. The stress in the upstream face is estimated with Navier’s equation and the limit state function is written as

$$G_\sigma = \frac{N'}{A} - \frac{M_{tp}}{W} \qquad \text{Eqn. 4}$$

Design situations are defined according to 7.2.4 in PMCD Part I.

In this case only design situations 1 and 4 are analyzed. Design situation 1 is a permanent situation with water at retention water level (rwl) and ice load. The present structure does not have drains, hence full uplift is considered. If drains were present uplift reduction would have been assumed for dams where uplift monitoring is present. Without pressure monitoring it is not possible to know the uplift pressure and hence not to assume that pressures are reduced. Design situation 4 is water above rwl.

IV:.2 INPUT VARIABLES

Basic variables are defined in PMCD part II (load parameters) and part III (resistance parameters).

A brief summary is given here:

Unit weight of concrete and rock mass is defined in Part III, section III:2. For ordinary concrete with compressive strength (28 days) the mean value may be estimated as 23,5 kN/m^3. The coefficient of variation is 0,04, but may be reduced to 0,034 when the global density of a large structure is considered. Unit weight is modelled by a normal distribution. The rock mass consist of granite and a small variability is assumed. The mean value is 26,5 kN/m^3 and the coefficient of variation is 0,02 following a normal distribution.

According to Part III, section III:3 the basic friction angle may, without tests, be assumed to have mean value of 35° and a standard deviation of 1,75°. The friction angle is assumed according to tan($\phi_{b,c}$)ϵN(0,7;0,031).

According to Part III section III:3, the dilation angle may be assumed to have a mean value of 15° and a coefficient of variability of 3°. The dilation angle is assumed according to tan(i_c)ϵLN(0,268;0,0524).

The pre-stressing force applied to the structure may, according to Part III section III:8, be expressed as $(x, t) = P_0 - \Delta P(x, t_0) - \Delta P(x, \infty)$

Where the jacking force P_0 applied in the analysis should be the largest of the testing force and the jacking force. It is common to apply a jacking force of approximately 0,6·P_{fail}, but the steels may be tested to loads of up to 0,8-0,9·P_{fail} upon pre-stressing. Coefficient of variation of the pre-stressing force is 0,06-0,09 and here 0,075 has been used. Long and short-term pre-stress losses ΔP(x,t) are expected to be 10 % of the pre-stressing forcewith a coefficient of variation equal to 0,3.

The ice load is described in part II section II:3.2.1 as a log-normal distribution with a mean value of 80 kN/m (northern part of Sweden), and a standard deviation of 80 kN/m. There maximum possible ice load is defined as a normal distributed parameter with a mean value of 250 kN/m with a standard deviation of 25 kN/m.

The uplift coefficient *C* (*Cm* for moment) is described in part II section II:5.2 and is defined as a normal distribution with mean value 1 and coefficient of variation 0,05.

Hydraulic pressure is described in Part II, section II:3 and water levels above rwl are described below.

IV:.2.1 Crack length

To determine the uplift pressure distribution it is necessary to define a crack length, if cracking is found to occur in the dam heel. If a Monte Carlo simulation is used for the probability calculation, an iterative process may be adopted in the calculation. For a FORM-calculation it is necessary to first define a crack-length based on a deterministic calculation based on mean values, and use this in the probabilistic calculation.

In the present case the crack length is 3 m, and hence it affects the uplift only beneath the front plate.

IV:.2.2 Probability distribution function for water levels above maximum retention level

For design situation 4 there is a certain probability of getting water levels above retention water level (rwl). Depending on the characteristics of the facility, discharge possibilities and return period of floods a methodology to determine the cumulative distribution function of the water exceeding rwl is described in part II, section II:3.2.2. The water level above rwl is denoted d_e in the following.

Input data

The discharge capacity at retention level at the dam is 1920 m^3/s.

The 100-year flood is 1400 m^3/s and the design flood is 2700 m^3/s.

The probability of water levels rising above retention level is approximated as described in part II

$$\frac{\log(100)}{Q_{100}} = \frac{\log(n_n)}{Q_n} \qquad \text{Eqn. 5}$$

As an example the return period of floods larger than the discharge capacity is estimated by:

$$n_n = 10^{\left(Q_n * \frac{\log(100)}{Q_{100}}\right)} = 10^{\left(1920 * \frac{\log(100)}{1400}\right)} = 558\ years$$

The return period of the design flood is approximately 7200 years, which is within the span of 5 000-10 000 years and Eqn. 5 is expected to be a sufficiently good approximation at the facility.

For the dam specified the spillway width is a total of 48 m and the distance from spillway threshold to rwl is 8 m. The distance between rwl and concrete dam crest is 1 m. The embankment dam crest is 1 m higher. The crest length of the concrete section is 100 m and the embankment dam is 500 m. The layout is schematically shown below.

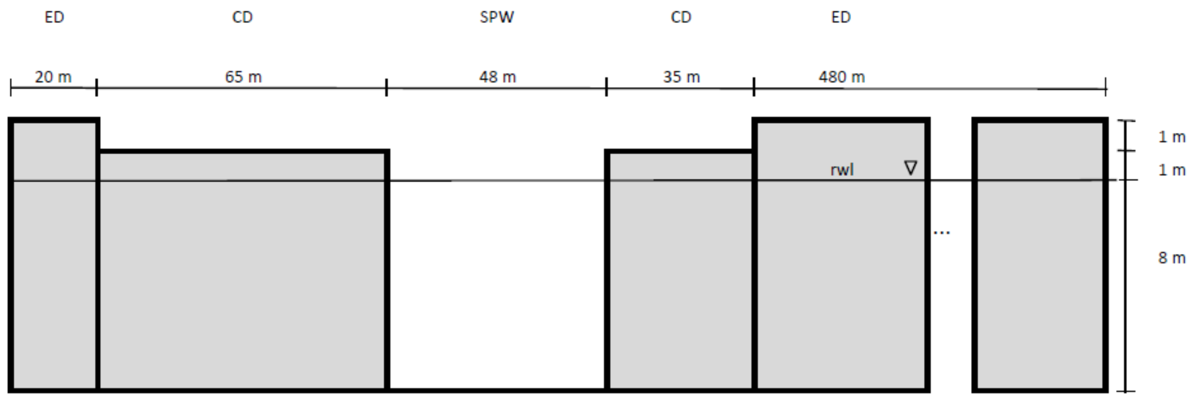

Figure E- 2. Layout of dam facility.

Estimation of return periods for different water levels

Table E-1 shows a calculation of the total discharge at different water levels, first through the spillway, then over the concrete crest and next over the embankment dam.

From Eqn. 5 the return period for each flow is estimated. When $d_e = 0$ water is at rwl. When water reach over rwl $d_e > 0$. The probability of the water level to reach a water level x, given that water has already reached above rwl, is calculated as

$$P(x|d_e > 0) = \frac{n_{de}}{n_x} \qquad \text{Eqn. 6}$$

where n_{de} is the return period of water levels above rwl and n_x is the return period of water level x (this comes from the law of conditional probability. Since x is dependent on d_e>0 we have $P(x \cap d_e > 0) = P(x)$). The cumulative distribution function of F(x|d_e>0) is now given by

$$F(x|d_e > 0) = 1 - P(x|d_e > 0) \qquad \text{Eqn. 7}$$

The result is shown in Table E-1. In the table, h = water depth above spillway threshold, Q1 discharge through gate (calculated with spillway width 48 m, μ = 0,6), Q2 discharge over concrete dam crest (length 100 m, μ = 0,55), Q3 discharge over embankment dam crest (length 500 m, μ = 0,5), Qtot is the total discharge, P_return is the return period of a flood of equal size as Qtot, calculated by Eqn. 5, where
P(x,|d_e>0) is calculated according to Eqn 6 and F(x,|d_e>0) according to Eqn 7.

Table E- 1. Probabilities of certain water levels.

h	x	Q1	Q2	Q3	Qtot	P_return	P(x|de>0)	F(x|de>0)
8,00	0,00	1922			1922	558	1,00	0,00
8,13	0,13	1968			1968	647	0,86	0,14
8,25	0,25	2013			2013	752	0,74	0,26
8,38	0,38	2059			2059	874	0,64	0,36
8,50	0,50	2105			2105	1018	0,55	0,45
8,63	0,63	2152			2152	1187	0,47	0,53
8,75	0,75	2199			2199	1385	0,40	0,60
8,88	0,88	2246			2246	1618	0,34	0,66
9,00	1,00	2294	0		2294	1892	0,29	0,71
9,13	1,13	2342	7		2349	2269	0,25	0,75
9,25	1,25	2390	20		2410	2776	0,20	0,80
9,38	1,38	2439	37		2476	3445	0,16	0,84
9,50	1,50	2488	57		2545	4323	0,13	0,87
9,63	1,63	2537	80		2617	5480	0,10	0,90
9,75	1,75	2587	105		2692	7009	0,08	0,92
9,88	1,88	2636	133		2769	9038	0,06	0,94
10,00	2,00	2687	162	0	2849	11746	0,05	0,9525
10,13	2,13	2737	194	33	2963	17117	0,03	0,9674
10,25	2,25	2788	227	92	3107	27451	0,02	0,9797
10,38	2,38	2839	262	169	3270	46953	0,01	0,9881
10,50	2,50	2891	298	261	3449	84691	0,01	0,9934
10,63	2,63	2942	336	364	3643	160031	0,00	0,9965
10,75	2,75	2995	376	479	3849	315349	0,00	0,9982
10,88	2,88	3047	417	604	4067	645877	0,00	0,9991
11,00	3,00	3100	459	738	4296	1371320	0,00	0,9996

Next the parameters of the cumulative distribution function are defined. In this example three sets of parameters are needed; one set describes the CDF in the span of water level 0-1 m, the second in the span of 1-2 m and the last from 2 m and up. The reason is of course that overtopping/discharge over the dam crest changes the discharge behavior.

Trapezoidal distributions are described by the following equation in the interval c<x≤b:

$$F(x) = 1 - \frac{(b-x)^2}{(b-a)(b-c)}$$

The following parameters are found to minimize the error in the present case:

Table E- 2. Parameters of trapezoidal distributions.

	p1	p2	p3
a	-0,1	-0,3	-4
b	2,2	2,6	3
c	0	-0,4	0
Interval (de) [m]	0-1	1-2	>2

The original CDF and the three parameter descriptions are shown in Figure E-3.

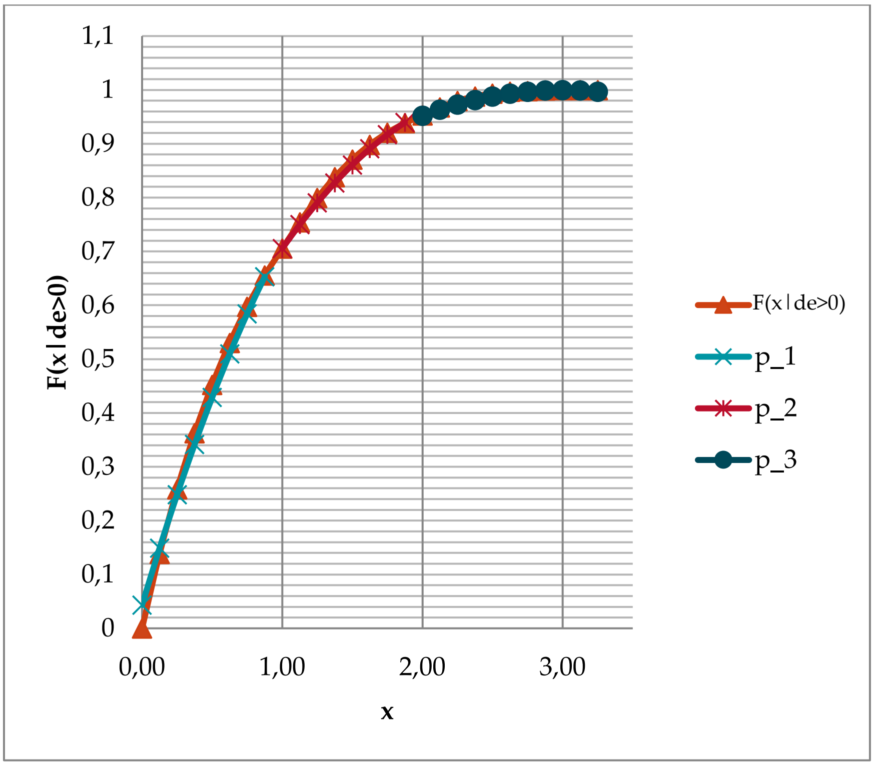

Figure E- 3. CDF of de and the trapezoidal descriptions.

The probability of reaching a water level above rwl is estimated as

P(de>0) = 1/P_return = 1/558 = $1{,}79\cdot10^{-3}$
P(de>1) = 1/P_return (de 1m) = 1/1892 = $5{,}28\cdot10^{-4}$
P(de>2) = 1/P_return (de 2m) = 1/2849 = $3{,}5\cdot10^{-4}$

P(0<de<1) = P(de>0)-P(de>1)=$1{,}26\cdot10^{-3}$
P(1<de<2) = P(de>1)-P(de>2)=$1{,}78\cdot10^{-4}$
P(de>2) = $3{,}5\cdot10^{-4}$

IV:.2.3 Summary of parameters

The following table includes all parameters included in the analysis, and short explanations are given.

Table E- 3. Parameters in the analysis.

Random Variables	Unit	Notation	Distribution	Mean Value	Standard Dev.	V_x	Comments
Unit weight concrete	kN/m^3	γ_c	Normal	23,5	0,8	0,034	
Unit weight rock mass	kN/m^3	γ_m	Normal	27	0,54	0,02	
Basic friction angle, concrete-rock	°	$\tan \emptyset_{b,c}$	Normal	35	1,75	0,05	
Dilation angle, concrete-rock	°	$\tan i_c$	Lognormal	15	3	0,2	Blasted rock surface
Basic friction angle, fracture rock mass	°	$\phi_{b,F}$	Normal	30	1,5	0,05	
Jacking Force Pre-stressed Anchors	kN/m	P_0	Normal	540	40,5	0,075	
Losses of Pre-stressed Force	kN/m	$\Delta P(x,t)$	Normal	54	16,2	0,30	
Ice Load	kN/m	I	Lognormal	80	80	1,00	Truncation at maximum
Maximum Ice Load	kN/m	I_m	Normal	250	25	0,10	
Compressive Strength of the Rock Mass	MPa	$f_{c,rock\ mass}$	Constant	20	-	-	
d_e part 1			Trapezoidal	Parameters:	-0.1; 2.2;0		Range [0-1]
d_e part 2			Trapezoidal	Parameters:	-0.3; 2.6;-0.4	-	Range [1-2]
d_e part 2			Trapezoidal	Parameters:	-4; 3; 0	-	Range [2-]

IV:.3 PERFORM RELIABILITY CALCULATION

When all limit state functions and input variables have been defined the probabilistic calculation may be performed. Different software exist. It is also possible to write a program in e.g. Matlab, especially for Normal and Lognormal distributions.

For the sake of this example software COMREL (Strurel, 2008) has been used.

IV:.3.1 Input into COMREL

The following is the input file in COMREL:

```
//******                 Example                    ******
//-----------------------------------------------------
//LIMIT STATES
//-----------------------------------------------------
FLIM(1){Sliding/Drains
Working}=(FUNC(20)*FUNC(25))+FUNC(22)
FLIM(2){OT/Drains Working}= FUNC(23)+FUNC(24)
FLIM(3){stress}=FUNC(204)-0

//-----------------------------------------------------
//DEFINITION OF RESISTANCE & LOADS
//-----------------------------------------------------
//Self-Weight

DEFFUNC(1)(){Self Weight Pilar Part 1}=Vt*dc
DEFFUNC(2)(){Moment Pilar Part 1}=FUNC(1)*(XG-FUNC(28))
//-----------------------------------------------------
//Water Pressure
//For design situation 1: FUNC(501)*0. For design
situation 4 apply function 501 for de1, de2 and de3, no
ice load.
DEFFUNC(501)(){de}=itruncp(13,D,2,10,-
4,0,0,3)//de1=(13,D,0,1,-0.1,0,0,2.2)//de2=(13,D,1,2,-
0.4,-0.3,-0.3,2.6)//de3=(13,D,2,10,-4,0,0,3)
DEFFUNC(500)(){Water level}=DG-BL+FUNC(501)*1

DEFFUNC(3)(){Horizontal Water Pressure}=-
0.5*dw*(FUNC(500))^2*bp
DEFFUNC(4)(){Moment Horizontal Water
Pressure}=FUNC(3)*((FUNC(500))/3)
DEFFUNC(5)(){Vertical Water Pressure
1}=dw*(FUNC(500))*lgfp*bp
DEFFUNC(6)(){Moment Vertical Water Pressure
1}=FUNC(5)*(l-(lgfp/2)-FUNC(28))
DEFFUNC(29)(){Vertical Water Pressure
2}=0.5*dw*(FUNC(500))*pllfp*bp
DEFFUNC(30)(){Moment Vertical Water Pressure
2}=FUNC(29)*(l-lgfp-(pllfp/3)-FUNC(28))
//-----------------------------------------------------
//Uplift
DEFFUNC(300)(){spricklängd}=spr

DEFFUNC(100)(){p inspgng}=(FUNC(500))*(l-4.2)/(l-
FUNC(300))
DEFFUNC(101)(){p inspng2}=(FUNC(500))*8.3/(l-FUNC(300))

DEFFUNC(7)(){U1}=-dw*(FUNC(500))*1.5*bp
DEFFUNC(8)(){Moment U1}=FUNC(7)*(l-0.7-FUNC(28))
DEFFUNC(9)(){U2}=-dw*FUNC(100)*2*bp
```

```
DEFFUNC(10)(){Moment U2}=FUNC(9)*(10.2-FUNC(28))
DEFFUNC(11)(){U3}=-0.5*dw*FUNC(101)*8.5
DEFFUNC(12)(){Moment U3}=FUNC(11)*(5.5-FUNC(28))
DEFFUNC(13)(){Ud}=FUNC(7)+FUNC(9)*C2+FUNC(11)*C2
DEFFUNC(14)(){Udm}=FUNC(8)+FUNC(10)*Cm2+FUNC(12)*Cm2
DEFFUNC(15)(){U=Ud*C}=FUNC(13)
DEFFUNC(16)(){Um=Udm*Cm}=FUNC(14)
//-----------------------------------------------------
//Ice Load
DEFFUNC(17)(){Truncation of the Ice
Load}=itruncm(3,X,0,Y,80,80,0,0)*0
DEFFUNC(18)(){H Ice Load}=-FUNC(17)*bp
DEFFUNC(19)(){Moment Ice Load}=FUNC(18)*((DG-BL)-
(IceTh/3))
//-----------------------------------------------------
//Sum Total Forces and Moments
DEFFUNC(20)(){N'=Vertical Forces-
Uplift}=FUNC(21)+FUNC(15)
DEFFUNC(21)(){Vertical
Forces}=FUNC(1)+FUNC(5)+FUNC(29)+FUNC(31)
DEFFUNC(22)(){H=Sum Horizontal Forces}=FUNC(3)+FUNC(18)
DEFFUNC(23)(){Positive
Moments}=FUNC(2)+FUNC(6)+FUNC(30)+FUNC(32)
DEFFUNC(24)(){Negative
Moments}=FUNC(4)+FUNC(16)+FUNC(19)
//-----------------------------------------------------
//Friction Angle
DEFFUNC(25)(){tan(phi+ic)}=(tanphi+tanic)/(1-
tanphi*tanic)
//-----------------------------------------------------
//Adjusted overturning
DEFFUNC(28)(){No drains-a=max(N'/fcc ;
N'/sigma)}=FUNC(20)/(sigmar*bp)//Assume failure in the
rock mass, sigma rock = 20 MPa
//-----------------------------------------------------
//Rock anchors
DEFFUNC(31)(){Vertical Force Anchors}=(P0-Pxt)
DEFFUNC(32)(){Moment Anchors}=FUNC(31)*d
//-----------------------------------------------------
//Stress calculation (Navier's formula)
DEFFUNC(200)(){a}=(FUNC(23)+FUNC(24))/FUNC(20)
DEFFUNC(201)(){tp}=6.32
DEFFUNC(202)(){I}=185.38
DEFFUNC(203)(){Mtp}=FUNC(20)*(FUNC(201)-FUNC(200))
DEFFUNC(204)(){sp}=FUNC(20)/12.2-FUNC(203)*(1-
FUNC(201))/FUNC(202)
//-----------------------------------------------------
```

Observe that the ice load is lognormal (distribution type no 3 in the truncation formula), but has a standard normal variable in the truncation formula.

The following is the associated parameters input into COMREL:

Symbolic Expressions | Stochastic Model | Correlations | Multiple Runs | Results | Plots

	Identifier	Comment	Distribution	Value	Value
R	dc	Concrete Density	Normal (Gauss)	23.5	0.8
R	X	Truncation Ice Load	Normal (Gauss)	0	1
R	Y	Maximum Ice Load	Normal (Gauss)	250	25
R	tanphi	tan(phi)	Normal (Gauss)	0.7	0.035
R	tanic	tan(ic)	Lognormal	0.268	0.0269538
R	P0	Jacking Force	Normal (Gauss)	540	40.5
R	Pxt	Losses of Prestress	Normal (Gauss)	54	16.2
R	C2	Uplift pressure coefficient	Normal (Gauss)	1	0.05
R	Cm2	Uplift moment coefficient	Normal (Gauss)	1	0.05
R	D	Truncation variable for de	Normal (Gauss)	0	1
R	drock	Rock density	Normal (Gauss)	27	1.35
R	C	Uplift pressure coefficinet on rock joint	Normal (Gauss)	1	0.05
P	bp	Width Pillar 1	Constant	1	
P	DG	Retention Water Level	Constant	90	
P	dw	Water Density	Constant	10	
P	BL	Bottom Level	Constant	73	
P	IceTh	Ice Thickness	Constant	1	
P	pllfp	Proyection of the Frontplate	Constant	1.2	
P	lgfp	Length of the Frontplate	Constant	0.7	
P	l	Total Length of the Dam	Constant	14.7	
P	Vt	Total Volume	Constant	108.1	
P	XG	Center of Gravity	Constant	7.97	
P	sigmar	Compressive Strength of the Rock Mass	Constant	20000	
P	d	Lever Arm Anchors	Constant	11.5	
P	spr	Lenght of crack due to tensile stress	Constant	3	
P	joint	Depth of through joint	Constant	1	
P	tanomega	Friction angle on passive wedge	Constant	0.57735	

In the present case the estimated crack length extends 3 m and the uplift pressure is assumed equal to the head water level for this part. Beyond the crack zone, linear uplift reduction is assumed. The parameters C and C_m are only applied to the uplift with linear reduction, hence full uplift is implied on the cracked zone.

IV:.4 RESULTS FROM COMREL

COMREL gives calculated β and the corresponding probability of failure as output.

For design situation 4, the probability of failure must be combined with the probability of that specific water level to occur. As an example the probability of sliding in design situation 4 is given by probability of failure for water levels above rwl but below concrete dam crest (situation 4.1), water levels above the the concrete dam crest but below the embankment dam crest (situation 4.2) and finally water levels above the embankment dam crest (situation 4.3).

$$Pf_4 = Pf_{4.1} \cdot P(rwl < wl < rwl + 1m) + Pf_{4.2} \cdot P(rwl + 1m < wl < rwl + 2m) + Pf_{4.3} \cdot P(wl > rwl + 2m) = \Phi(-\beta_{4.1}) \cdot P(rwl < wl < rwl + 1m) + \Phi(-\beta_{4.2}) \cdot P(rwl + 1m < wl < rwl + 2m) + \Phi(-\beta_{4.3}) \cdot P(wl > rwl + 2m)$$

Table E- 4. Results of probabilistic analysis.

	Design sit. 1	Design sit. 4.1	Design sit. 4.2	Design sit. 4.3	Design sit. 4, all parts	Design sit. 4, all parts
	β	β	β	β	P_f	β
Sliding	**4,71**	4,59	3,32	2,31		
Overturning	**7,82**	10,57	8,28	6,11		
Stress	**0,37**	0,67	-1,47	-4,37		
	P_f	P_f	P_f	P_f		
P(occurrence) of design sit.)	1	1,26E-03	1,78E-04	3,50E-04	**1,79E-03**	
P_f (sliding)	1,21E-06	2,83E-09	8,16E-08	3,66E-06	**3,74E-06**	**-4,48**
P_f(overturning)	2,58-15	2,56E-29	1,06E-20	1,74E-13	**1,74E-13**	**-7,27**
P_f (stress)	0,36	3,16E-04	1,65E-04	3,50E-04	**8,31E-04**	**-3,14**

IV:.4.1 Analysis of sensitivities

Output from a FORM-calculation is also sensitivity values that show the importance of different parameters on the final result.

A large sensitivity value indicates large importance of a certain parameter. A negative value means that the parameter acts as a load and a positive value that the parameter acts as a resistance.

For sliding and overturning in Design situation 1 and 4.1 the result is the following sensitivity values.

Table E- 5. Sensitivity values obtained from probabilistic analysis.

	Design situation 1		Design situation 4.1	
Parameter	Sliding	Overturning	Sliding	Overturning
ρ_c	0,42	0,65	0,45	0,74
Ice	-0,44	-0,38		
Max ice	-0,08	-0,36		
Tan(ϕ_{bc})	0,48	0	0,51	
Tan(i_c)	0,57	0	0,59	
P_0	0,19	0,44	0,21	0,51
P_{xt}	-0,08	-0,18	-0,09	
C	-0,19		-0,22	-0,2
C_m		-0,28		-0,34
D	0	0	-0,29	-0,18

Sensitivity values are a good way to check the appropriateness of the analysis. For a structure where the β value is lower than target β value the sensitivities also provides guidance on where to put more effort. In the present example it would be a good idea to investigate dilation angle i_c more closely. Reduced uncertainty regarding this would increase the sliding stability. A reduction in variability from 20% to 10% for design situation 1 means an increase in β from 4,7 to 5,6.

IV:.5 CALCULATION WITH PERSISTENT ROCK JOINT

For an estimation of the critical depth for a persistent rock joint to exist the following assumptions are made:

- Joint depth y m
- Horizontal joint through the rock mass
- Linearly decreasing uplift on joint, parameter *C* applies
- A passive wedge is formed on the downstream side

The results in the present example show that a joint located at depth smaller than 2,5-3 m may be dangerous as β-values are lower than the target β value. Due to this is that it is advisable to

a) drill drainage holes into the rock to make sure that high uplift pressures cannot build up in case a joint exist in the upper 3 m of the rock mass or
b) make a detailed investigation of the geological information from the construction of the dam to rule out the possibility of persistent joints, and/or
c) investigate the presence of joints in the rock mass down to a depth of approximately 3 m with additional core drilling combined with BIPS (Borehole Image Processing System)-logging.

Table E- 6. β-value for different joint locations.

Joint depth y [m]	β
0,5	2,24
1	2,80
1,5	3,37
2	3,97
2,5	4,54
3	5,10
3,5	5,64
4	6,16
4,5	6,63
5	7,07
5,5	7,48
6	7,85
6,5	8,19
7	8,50
7,5	8,78
8	9,04
8,5	9,26
9	9,47
9,5	9,66
10	9,83

PROBABILISTIC MODEL CODE FOR CONCRETE DAMS

This is a first attempt to put together rules, regulations and explanations necessary for design and assessment of concrete dams from a probabilistic point of view. The intent is for probabilistic assessments of concrete dams to be performed in a systematic way.

The first part contains general principles, information of how a probabilistic verification is performed; limit states and design situations, limit state functions and target reliabilities relevant for concrete dams and Bayesian updating

The second and third parts contains general descriptions on loads and resistances modelling. Next relevant loads and resistances for concrete dams are discussed and "best estimates" on statistical descriptions are presented. Loads included are ice loads, hydrostatic pressure, uplift and earth pressure. Resistance parameters included are self-weight, friction properties of concrete/rock contact and in rock, material properties (concrete, rock, steel), rock bolts and rock anchors.

The report also contains one example of a probabilistic assessment based on Probabilistic model code for concrete dams.