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Environmental loads on embankment dams in mountainous regions

Miljølaster på fyllingsdammer i høyfjellsmagasin

Fjola Gudrun Sigtryggsdottir/NTNU

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Environmental loads on embankment dams in mountainous regions : miljølaster på fyllingsdammer i høyfjellsmagasin

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Sammendrag: Målet for prosjektet er å få mer kunnskap om miljølaster på norske fyllingsdammer på høyfjellsmagasiner. Det er spesielt fokus på laster som blir påvirket av klimaendringer og har betydning for damsikkerheten, deriblant vind og bølger. Rapporten sammenlikner beregningsmetoder i Norge, Canada, USA og UK for å estimere vindgenererte bølger, bølgeoppskylling og størrelse av plastringstein i oppstrøms skråningsvern. Studiene er gjort for et konseptuelt magasin samt for 2 magasin i Norge. En konklusjon er at det finnes begrenset med vinddata fra høyfjellet. Analyse av tilgjengelige vinddata ga litt lavere vindhastigheter enn beregninger utført etter Norsk Standard.
Beregninger av signifikant bølgehøyde varierer relativt mye med valgt metode, og anbefalt metode i NVEs retningslinjer er dårlig tilpasset lange smale magasin. Beregningsmetodikken for dimensjonering av stein i oppstrøms skråningsvern inneholder usikkerheter, som kommer fra både valg av stabilitetskoeffisienter og beregning av bølgehøyde.

Resultatene fra prosjektet er brukt som underlag i en vurdering av relevante norske retningslinjer og veiledere, og det er gitt anbefalinger om videre forskning.

Emneord: Damsikkerhet, miljølaster, vind, bølger, fyllingsdammer, skråningsvern, bølgeoppskylling.

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Forord

NVE har en rekke veiledere og retningslinjer som utdyper bestemmelser i forskrift om sikkerhet ved vassdragsanlegg (damsikkerhetsforskriften). Veilederne bygger på forskning og kunnskap som er utviklet over tid og delvis i andre land. De siste tiårene har det også kommet mer kunnskap om effekter av klimaendringer, deriblant effekter på miljølaster som er viktig for damsikkerheten. Det er derfor stadig behov for å oppdatere og utvikle kunnskapsgrunnlaget for veilederne.

NTNU har gjennomført et studium av miljølaster på høyfjellsmagasin i Norge (FOU-prosjekt 80407) som gir nyttige innspill til fremtidige revisjoner av veileder for fyllingsdammer (NVE-veileder 4/2012) og NVEs retningslinjer for laster og dimensjonering (2003). Prosjektet viser at det er behov for mer data fra høyfjellsmagasin, spesielt vinddata. NVE vil derfor anbefale dameiere å etablere vindmålinger og samle inn data som underlag for å bestemme dimensjonerende vindhastighet ved dammer på høyfjellet. Prosjektet viser videre at det er stor usikkerhet i beregning av bølgehøyder og dimensjonering av steinstørrelser i oppstrøms skråning av fyllingsdammer, da beregningsmetoder og formler er basert på store, brede magasin. NVE anbefaler derfor at man allerede nå tar høyde for usikkerhetene, spesielt for lange, smale magasin, inkludert magasin med buet form. Prosjektet viser også at det generelt er behov for mer forskning på temaet miljølaster, for å sikre et best mulig underlag for arbeidet med å revidere regelverket for damsikkerhet. Samtidige vind- og bølgemålinger vil være til stor nytte i denne sammenheng.

Oslo 28.11.2022

Lars Grøttå
seksjonssjef
Seksjon for damsikkerhet

Dokumentet sendes uten underskrift. Det er godkjent i henhold til interne rutiner.

Environmental loads on embankment dams in mountainous regions

Fjóla Guðrún Sigtryggsdóttir

Abstract

The goal of the present study is to enhance knowledge relating to assessment of environmental loads on Norwegian embankment dams in mountainous regions. Focus is on loads that are affected by climate change and that are of relevance for dam design as well as under regular review of dam safety. This comprises mainly wind-generated waves and ice. The height of wind generated waves influences the freeboard assigned to a dam. Furthermore, the wave height along with ice load govern dimensioning of riprap used in upstream slope protection of embankment dams.

The report investigates and compares different methods to estimate fetches and predict the significant wave height on reservoirs. These predictions are further used when comparing different methods to predict run-up heights on dams for freeboard evaluation as well as to determine sizes of riprap stones in a protective layer on upstream dam slopes. Additionally, wind measurements and wind predictions and selected reservoirs are investigated.

The report concludes with an appraisal of the Norwegian Dam Safety guidelines on the topics studied and recommendations for further studies.

Abstrakt

Målet for prosjektet er å få mer kunnskap om miljølaster på norske fyllingsdammer ved høyfjellsmagasiner. Det er fokus på laster som påvirkes av klimaendringer og som bestemmer dimensjonering knyttet fyllingsdammer, dvs. vindgenererte bølger og is. Høyden av vindgenererte bølger er brukt til bestemmelse av fribord på dammer. Beregnet bølgehøyde påvirker også størrelse av plastringstein i oppstrøms skråningsvern sammen med islast.

I rapporten undersøkes og sammenlignes beregningsmetoder for å estimere strøk og vindgenererte bølger i Norge, Canada, USA og UK. Dette er videre brukt i formuler for å beregne og sammenligne bølgeoppskylling og størrelse plastringstein i oppstrøms skråningsvern på fyllingsdammer. Undersøkelsene gjøres blant annet med bruk av et konseptuelt magasin samt for to magasiner i Norge. I tillegg gjøres vurdering av tilgengelig vinddata og beregningsmetoder for vind etter Norsk Standard.

Undersøkelser i rapporten viser at vinddata fra høyfjellet er begrenset, og da spesielt fra magasiner. Analyse av tilgjengelig vinddata fra to magasiner gav lidt lavere verdier av vindhastighet med forskjellig returperioder sammenlignet med Norsk Standard. Rapporten viser videre at beregning av signifikant bølgehøyde varierer relativt mye etter den metode som er brukt. Samtidig viser rapporten at metoden som er brukt i gjeldene retningslinje for laster og dimensjonering for å beregne bølgehøyde tilpasses ikke godt lange smale magasiner som finnes for eksempel på høyfjellet. Beregningsmetodikk har også betydning for dimensjonering av stein i oppstrøms skråningsvern på fyllingsdammer. I den sammenheng er det videre usikkerheter knyttet eksperimental stabilitets koeffisienter som til sammen med bølgehøyde legges inn i en dimensjoneringsformel for plastringstein.

Rapporten konkluderer med en vurdering av relevante norske retningslinjer og veiledere på det som er undersøkt. Det anbefales å styrke grunnlaget med videre forskning.

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1 Introduction

1.1 Overview and contract

The goal of the present study is to enhance knowledge relating to assessment of environmental loads on Norwegian embankment dams in mountainous regions. Focus is on loads that are affected by climate change and that are of relevance for dam design as well as regular review of dam safety. This comprises mainly wind-generated waves and ice. The height of wind generated waves influences the freeboard assigned to a dam. Furthermore, the wave height along with ice load govern dimensioning of riprap used in upstream slope protection of embankment dams.

The project was initiated as a pre-study in collaboration with the Norwegian Water Resources and Energy Directorate (NVE).

This report is prepared in compliance with a contract between NVE and NTNU on project number: 80407 in NVE registry, with the Norwegian title: "*Miljølaster på fyllingsdammer ved høyfjellmagasiner*", or "*Environmental loads on embankment dams in mountainous regions*" in English. The report was finalized in 2020 and reviewed by NVE in 2021. The 2020 version of the report was additionally made available to several dam owners in Norway as well as engineering consultancies interested in the subject. The report has provided foundation for further research projects, e.g. InMoDam[1], into the Norwegian conditions that include field surveys and experimental testing as recommended herein.

1.1.1 Problems to be addressed (Contract questions)

According to NVE's description of the project, the contract is fulfilled when the following, referred to here as contract questions, has been completed (see also overview in Table 1.1):

1. The following relating to ice loads and wind generated waves on reservoirs.

    a. Compare methods used in Norway, Canada, USA and potentially other relevant countries.

    b. Appraisal of the Norwegian guidelines NVE(2003)

[1] InMoDam (Innovative monitoring and modelling of loads and damage processes on embankment dams) received funding from the Norwegian Research Council in 2022.

   c. Assessment of how the wind velocities and different methods influence the sizing of riprap.

2. The following relating to meteorological data and predictions:

   a. Investigate whether meteorological measurements are available from the highlands of Norway, particularly measurements at mountain reservoirs.

   b. Furthermore, compare the available wind data and predictions according to the Norwegian Standard if that is possible within the timeframe of the project.

3. Using the method studied in 1a) to study significant wave height predictions at least two cases of mountain reservoirs in Norway, one located in somewhat open terrain and the other framed by mountain slopes.

1.2 Report organization and links to contract questions

The report is organized in the following manner:

- In Chapter 2, environmental loads will be outlined in general with focus on mountain reservoirs and dams. In this context wind measurements in Norway are reviewed (Contract question 2a).

- In Chapter 3, wind generated waves are addressed. First the wind predictions are investigated (Contract question 2b), and then different methods of evaluating the fetch to use in the different wave height predictions, also reviewed in Chapter 3 (Contract question 1a). Cases of two reservoirs are studied for the wind predictions (Contract question 3).

- In Chapter 4, the different methods reviewed in Chapter 3 are investigated further (Contract question 1a) both through two cases of reservoirs in Norway (Contract question 3), and through a conceptual reservoir allowing a rough comparison of measurements from reservoirs available in the literature. The investigation through the conceptual reservoir is outside the direct scope of the contract but enhances appraisal of the method used in the Norwegian dam safety guidelines (Contract question 1b).

- In Chapter 5, different methods of estimating required freeboard of embankment dams through prediction of run-up heights or through criteria for allowable discharges. This is outside the scope of the contract from NVEs perspective but enhances appraisal of the method used in the Norwegian dam safety guidelines (Contract question 1 b).

- In Chapter 6, a summary on ice loads on embankment dams is provided. (Part of Contract Question 1).

- In Chapter 7, methods of riprap design are reviewed and compared both generally, and through the case of the Nesjøen Reservoir in Norway. (Contract question 1c)

- In Chapter 8, a summary is provided with the appraisal of the Norwegian dam safety guidelines on the problems investigated (Contract Question 1b), as well as with a summary relating to wind measurements and prediction (Contract Question 2).

Table 1.1 provides an overview in the context of linking contract questions, i.e. the problems to be addressed, to chapters in the report.

Table 1.1 Contract Questions/Problems to be addressed linked to chapters in the report

Nr		Contract question	Addressed in
1		The following relating to ice loads and wind generated waves on reservoirs.	Chapter 3 (wind, waves) Chapter 6 (ice action)
	1a	Compare methods used in Norway, Canada, USA and potentially other relevant countries.	Chapter 4 , 5 and 7
	1b	Appraisal of the Norwegian guidelines NVE(2003)	Chapter 8
	1c	Assessment of how the wind velocities and different methods influence the sizing of riprap.	Chapter 7
2		The following relating to meteorological data and predictions:	
	2a	Investigate whether meteorological measurements are available from the highlands of Norway, particularly measurements at mountain reservoirs.	Chapter 2.2; Chapter 8
	2b	Furthermore, compare the available wind data and predictions according to the Norwegian Standard if that is possible within the timeframe of the project.	Chapter 3.2.5-3.2.6
3		Using the method studied in 1a) to study at least two cases of mountain reservoirs in Norway, one located in somewhat open terrain and the other framed by mountain slopes.	Chapter 3.2.5-3.2.6 Chapter 4.3.5(Šuoikkátjávri) Chapter 4.3.6 (Nesjøen) Chapter 7.6.2 (Nesjøen)

2 Environmental loads

2.1 General

Environmental conditions cover natural phenomena, which may contribute to damage, operation disturbances or failures. The environmental phenomena are usually described by physical variables of statistical nature. The statistical description should reveal the extreme conditions as well as the long- and short-term variations. If a reliable simultaneous database exists, the environmental phenomena can be described by joint probabilities. Different return periods are selected for the characteristic value of the environmental load depending on the design situation considered. For example, a return period of 50 years is generally associated with conditions of normal use (the persistent design situation) for structures with design working life of 50 years. However, in some cases the character of the action and/or the selected design situation makes another fractile and/or return period more appropriate.

Environmental loads are caused by environmental phenomena. NVE (2003) defines environmental load as those that are caused by environmental conditions relating to climate, microclimate and topography. Microclimate is the climate at or near the ground surfaces, such as within the vegetation and soil layer. Microclimate exist, for example, near water bodies such as reservoirs. Furthermore, high peak and deep valleys create extremes of localized microclimate.

Environmental loads need to be considered in design of dams, furthermore, the environmental conditions arising from a certain environmental phenomenon influences dimensioning of a dam. The environmental loads to consider are e.g. those arising from environmental phenomena such as: water inflow, water level, wind, waves, ice, temperature, and earthquake. The focus here is on the climatic natural phenomena, mainly wind blowing over mountain reservoirs, wind generated waves and ice on reservoirs.

In this chapter, environmental loads in a changing climate will be discussed, with special focus on wind conditions and measurements in the Norwegian highlands. Subsequently environmental loads in the context of reservoirs and dams are introduced and each natural phenomenon linked to a further discussion in other chapters of this report

2.2 Environmental loads in a changing climate

In Norway there are large variations in climate caused by the large variations in geography, topography and latitude of the country (Asvall, 2008). Climate change may further bring potentially more frequent extremes in the weather. This may affect reservoir and dams in mountainous regions with more extreme wind and temperature.

2.2.1 Wind conditions in Norway

Wind loads on structures for engineering application are determined by the European standard. Methods provided in standards for wind predictions are discussed in Section 3.2. However, in a recent report (Førland et al., 2016) presenting supplementary studies performed at the Norwegian Meteorological Institute (MET Norway) for the rapport “Klima in Norge 2100” (I. Hanssen-Bauer, 2015) it is stated that wind conditions in Norway are difficult to analyze. It is further explained that this concerns both long-term variations as well as local effects.

In this section local wind funneling effects at mountain reservoirs is shortly reviewed along with wind measurements in Norway with focus on the highlands. Finally, the long-term variations are briefly discussed.

Mountain reservoirs and wind funneling

Reservoirs are often in mountainous regions or highland, with suitable dam sites found in narrow valleys, flanked by steeply rising mountains. Topographic conditions as these are often the case for reservoirs in Norway. Such topographic conditions can cause a funneling effect on the wind. The funneling can cause changes in the wind speed and tends to shift the wind direction so that the wind blows along the reservoir shape (see e.g. (Owen, 1987)). Methods provided in standards for wind predictions are discussed in Chapter 3.2 While these methods do not embrace effects of local conditions causing wind funneling, as may arise in valleys, factors that account for wind direction and altitude are considered. More detailed expert analyses would be required to estimate the local conditions in more detail to assess wind funneling effects.

Wind measurements in Norway with focus on the highlands

Observation practices and instrumentation for wind measurements has developed over the past fifty years, from visual (Beaufort scale) to instrumental (Førland et al., 2016). According to information downloaded from eklima.no, there are currently, in April 2020, total of 553 weather stations in Norway that monitor wind conditions. These stations are mainly operated by the Norwegian Meteorological Institute (MET Norway). Elevation of these stations versus the year from which the weather monitoring has been operated are plotted in Figure 2.1. The different observational practices must be kept in mind for the early stations.

Most of the weather stations above 400 m a.s.l were installed after the year 2000, with only about 25 installed prior to this. Furthermore, most stations above 800 m a.s.l. are installed after the year 2010 (see Figure 2.1). Thus, longer monitoring period is generally required for a good statistical evaluation of the wind conditions in the highlands. Førland et al. (2016) explain for example that modeled wind calculated from numerical atmospheric models are often used instead of direct wind measurements, at the same time they state that the local topography is not adequately described by such models. Thus, local wind direction and wind speed may differ significantly from the modeled wind. This must be considered e.g. for mountain reservoirs where for example wind funneling can occur as mentioned above.

The Figure 2.1 demonstrates that wind monitoring has been enhanced the last 20 years with installation of new stations, including at higher elevations. Table 2.1 groups the stations by elevation and gives number of station within each group. Obviously the largest number of wind station are located at lower elevation, or 387 stations, below 400 m a.s.l.. Hence, of the 553 stations 166 stations are located above elevation 400 m a.s.l. Thus, considering the extend of the Norwegian highlands, wind monitoring at high elevation can be considered rather limited.

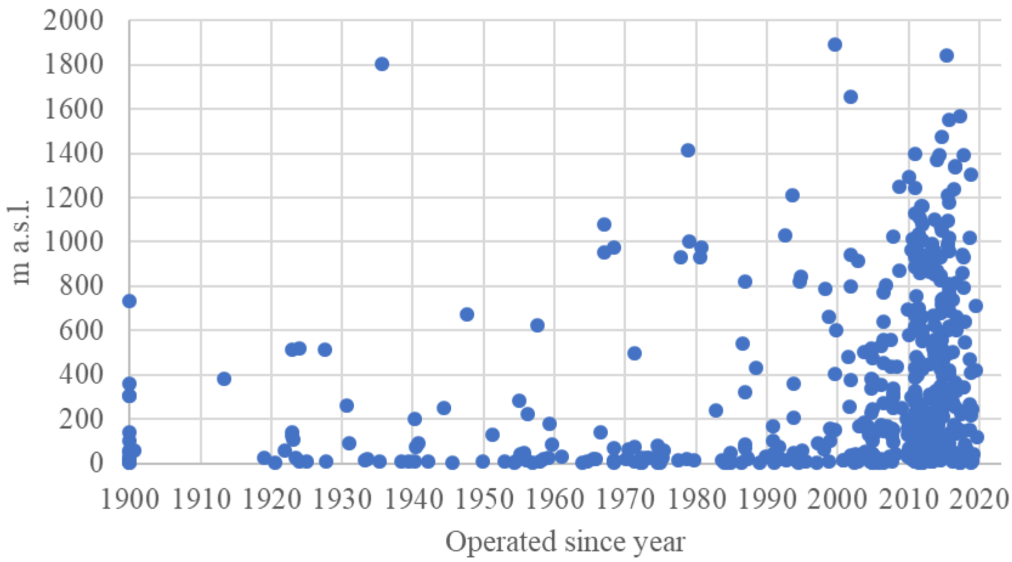


Figure 2.1 Elevation of weather stations in Norway with wind measurements versus year since start of operation. Stations operated since before 1900 are all aligned on the year 1900. (Data obtained from eklima.no)

Table 2.1 Number of stations in Norway monitoring wind grouped by station elevation.

Station elevation m a.s.l.	*0-400*	*>400-700*	*>700-900*	*>900-1000*	*>1000-1300*	*>1300-1500*	*>1500*
Number of Stations	*387*	*69*	*30*	*27*	*25*	*9*	*6*

While some of the 166 stations in the eklima.no database elevated above 400 m a.s.l. may be located at or in the vicinity of a reservoir, such as the examples the stations (Stations SN) provided in Table 2.2, most of those stations are not. Considering that there are 966 reservoirs in Norway with the highest regulated water elevation above elevation 400 m a.s.l., it can be stated that official monitoring (i.e. by MET Norway) of local wind conditions at reservoir sites are limited. It is possible that dam reservoir owners operate wind monitoring station, that are not included in the eklima.no database, but an overview of those stations is currently not available. However, considering that dam safety monitoring generally does not require monitoring of the wind conditions, such additional stations cannot be expected beforehand. Still, two such cases the Nesjøen and Aursjøen Reservoirs are considered in Chapter 3.2.5 and 3.2.6, and listed in Table 2.2.

The highest recorded wind velocities within the monitoring period at the stations listed in Table 2.2 range from 21 m/s (Aursjøen) to 31 m/s (Honnegrasnuten). Return period is not linked to these values, and statistical analysis may result in a 50-year with higher wind velocity than listed in the table. This is for example the case for the Nesjøen and Aursjøen Reservoirs as discussed in Chapter 3.

Table 2.2 Examples of highest recorded wind velocity form weather stations in the vicinity of mountain reservoirs in Norway. The information provided from stations with station number are from https://klimaservicesenter.no/.

Name of station (Kommune, Fylke)	Station number	Highest recorded wind velocity	Monitoring period		Elevation of station	Closest Reservoir
	SN	m/s	From	To	m a.s.l.	
Honnegrasnuten (Vinje, Vestfold & Telemark)	33300	31	2015	2019	1340	Between Totak and Bitdalsvatn
Finsevatn (Ulvik, Vestland)	25830	30.3	2002	2020*	1216	Finsevatn
Sisovatnet (Sørfold, Nordland)	82720	29.8	2018	2020*	671	Sisovatnet
Blåsjø (Hjelmeland,Rogaland)	40510	28.8	2011	2020*	1055	4.7 km from Blåsjø
Haukeliseter Testfelt (Vinje, Vestfold & Telemark)	33950	27.5	2015	2020*	990	Kjelavatn (Ståvatn, Bordalsvatn)
Klevavatnet (Aurland, Vestland)	53480	24.1	2014	2020*	960	Klevavatnet (and less than 10 km South from Viddalsvannet)
*April 2020						
Information extracted from monitoring data provided by the dam owner (see Chapter 3)						
Nesjøen (Tydal, Trøndelag)		21.4**	1998	2019		Nesjøen
Aursjøen (Skjåk, Innlandet)		21**	1998	2019		Aursjøen

**Measured data considered as 60 minute mean wind, these are values for 10 minute mean wind.

Wind and climate change

Førland et al., (2016) consider long-term effects, i.e. climate change in Norway. The findings of Førland et al., (2016) include that the strongest winds occur along the coast and in the mountains. Furthermore, that during the period 1961 to 2010 most of Norway has experienced a slight increase in wind velocities, however large variations are observed locally as well as from year to year. Predictions for future wind indicate small changes in the median value of the wind speed exceeded in 1% of the time. Conversely, for absolute maximum values there are increases from all seasons of up to 20% (depending on prediction model). This should be considered in the assessment of environmental loadings on for example embankment dams, in particular considering risk assessments. Measurements are required at mountain reservoirs to follow potential future changes in the wind climate, including regarding extreme weather events.

2.2.2 Ice conditions

Historically, the lowest winter temperatures in Norway are found in the interior close to the Swedish border in the southern part and in the interior of the Finnmark plateau in the northern part. Normally there are weather conditions promoting ice formation in reservoirs all over the country. There are, however, large regional variations. The duration of ice cover varies with the location and the size of reservoirs. Generally, ice formation starts by November, first in the most continental areas, and in the north. Ice release may be as late as May or June in the highest elevations in the south and in the north. (Asvall, 2008).

Recent study on climate in Norway includes investigation into historical climate trends conducted by Hanssen-Bauer and Førland (2016). They concluded from studying 12 Norwegian weather stations with records over 60 years, that there is a general tendency for higher increase in minimum temperature than in mean temperature. Furthermore, that this increase in minimum temperature has significantly contributed to the increase in daily mean temperature. Additionally, they report an increase in the frequency of days with mean temperature exceeding 20°C.

There are not many weather stations with long records, and Hanssen-Bauer and Førland (2016) investigated only 12 stations. None of those are located at a very high altitude and might not properly represent mountainous regions. Dam design in mountainous regions and/or risk assessments considering environmental loads preferably needs further studies to properly consider the climate change.

2.3 Environmental loads and dam reservoirs

2.3.1 Environmental loads in dam design

Environmental loads on embankment dams arising from natural phenomena relating to the climatic conditions comprise mainly wind-generated waves and ice. The height of wind generated waves (see Chapter 3 and 4) influences the freeboard assigned to a dam (see run up heights in Chapter 5). Furthermore, the wave height along with ice load (Chapter 6) govern dimensioning of riprap used in upstream slope protection of embankment dams (see Chapter 0).

2.3.2 Wind waves on reservoirs

The energy of the wind blowing over a water is transferred to the water surface and generates a range of wave heights and periods. The wave growth, i.e. the increase in wave height and the wave length (or period), will depend on the wind speed and the distance over which the wind can act, called the fetch. Thus, methods for predicting the wind speeds and the fetches are required. Methods for wind predictions are discussed in Section 3.2, while procedures for defining the fetches are discussed in Section 3.4.

The wind generated wave growth can be fetch-limited or duration-limited. On very long fetches in open oceans the wave growth may be limited by the duration of the wind. On reservoirs or inland lakes, i.e. on restricted fetches, the wave growth can be considered limited by the fetch length and by the maximum wind speed. (See e.g. (Ozeren and Wren, 2009)(Pullen et al., 2018)). The fetch-limited condition is of interest for the subject of this report, and particularly considering mountain resevoir that in some cases are curved or ckranked. Yarde et al., (1996) state that there is anecdotal evidence for wind funneling causing the wind to follow the axis of a curved/cranked reservoir in a valley, and that bent fetches, following the axis of the reservoir into the upper reaches should be considered. The different procedures for defining the fetches on restricted fetches such as on reservoirs, will be discussed in Section 3.4

Various techniques have been used to predict wave heights on the ocean and in coastal area. These techniques have been applied to predict wave height on inland lakes and reservoirs, however with some adjustments. The methods applied for dam engineering purposes are of interest for this study. The earliest methods were based on empirical relationships between wave height and wave length, and wind speed, wind duration, and fetch. Later, the development of the wave spectrum resulted in that new formulas were derived, however of similar format and with similar input parameters as the initial empirical relationships. The selection of formulas or approach (i.e. the initial empirical formulas, or new ones derived from a wave spectrum) is not uniform between the different countries. In Chapter 3.3, the early methods will be shortly reviewed followed by the wave spectrum approach. Then the guidelines in selected countries on predicting the wave height will be reviewed and compared in Chapter 4, e.g. by the use of a conceptual rectangular reservoir. Furthermore, the methods to predict the runup heights to determine the freeboard of a dam are discussed and compared in Chapter 5, while a wave erosion protection comprising riprap is discussed in Chapter 7.

2.3.3 Ice on reservoirs

In areas with cold climate the effects of ice in different forms has to be considered in the design of dams (see e.g. (ICOLD, 1996). Of interest her is the formation of ice cover on reservoirs that will cause loading on the upstream slope of an embankment dam. This will be discussed in Chapter 6, as well as chapter 7.5.

3 Wind generated waves

3.1 Introduction

The generation of wind waves and thus wave heights in a restricted area of water, such as a reservoir, depend upon the strength of the overwater wind (wind velocity) and the length of time (duration) that the wind acts a certain distance (fetch) over the water. These factors have been incorporated into simple prediction equations for calculating the wave heights to use in dimensioning certain features of the embankment dam, such as the freeboard and upstream wave protection.

The steps in making any simple prediction of waves on a given reservoir are to first predict extreme wind speeds and directions, second, calculate fetch lengths over which the wind can act, calculate wave heights from the selected formula for prediction a wave height usually referred to as the significant wave height. The predicted wave height is used in formulas of sizing of riprap stones as well as for freeboard evaluations through formulas for calculating wave-runup, and allowable discharge.

Different formulas for calculating the significant wave height are available in the literature, each associated with a specific way of calculating the fetch lengths. These methods will be discussed in the following, after a review on how to predict the extreme wind speeds.

3.2 Methods for predicting wind velocities

Wind actions fluctuate with time and act as pressures on surfaces. The energy transferred by the wind to the water surface generates a range of wave height and periods. A reasonable prediction of the wind speed is required for calculation of such wind generated waves. In some countries hourly wind velocities are considered, while in others 10-minute mean wind velocities are used or dependent on the fetch length and thus duration. Furthermore, different return periods of the wind velocity are associated with different reservoir water levels as later described in Section 3.2.3.

The prediction of the appropriate wind speed usually follows standard procedure given in national standards or guidelines. There are mainly two approaches which are referred to here as the standard approach and the data analysis approach.

In the standard approach a basic wind velocity is found in tables or read from charts given in national standards for different region of a country, this basic value must be adjusted to account for local conditions. In this case the general expression for wind velocity to be used in the prediction of wind generated waves on reservoirs using, is for example:

$$U = S_T\, S_r\, S_o\, S_p\, S_{alt}\, S_{dir}\, S_{season}\, U_{b,0} \tag{3.1}$$

where S_T is a duration factor, S_r is a factor that considers roughness of the surface over which the wind blows, S_o is an orthography facture, S_p is a probability factor, S_{alt} is an altitude factor, S_{dir} is a directional factor, S_{season} is a seasonal factor, and $U_{b,0}$ is the fundamental value of a basic wind speed. The orthography factor S_o is rarely of interest for reservoir sites, since it only accounts for isolated changes in the orthography, hills or escarpments.

In the data analysis approach, historical records of meteorological data containing wind velocities and directions are processed and analyzed. The meteorological data should be taken from a meteorological station of relevance for the site in question, so that most of the adjustment factors in Eq (3.1) are not required, but included in the recorded value. However, adjustment for height of the monitoring station may be needed as well as adjustment of data from inland stations to account for increase in the wind velocity when blowing over water. Furthermore, processing of the data must include statistical analysis so that wind velocities with different return periods can be extracted.

Here the procedure given by the Eurocodes for determining the factors in Eq (3.1) is described (see (NS EN 1991-1-4, 2009)). EN 1991-1-4 (2009) uses different symbols than presented above, and does not present a formula with the different factors aligned like above, but defines a basic wind velocity from the fundamental value defined as the characteristic 10 minutes mean wind velocity multiplied with the seasonal, directional and altitude factors, that is if the altitude factors are not included in the fundamental value. This will shortly be reviewed in the following. Additionally, wind velocity calculations in Canada and the UK that is of relevance for dam engineering purposes are shortly reviewed

3.2.1 Wind velocity according to the Eurocodes (Norway)

NVE (2003) (Retningslinjer for laster og dimensjonering) dam safety guidelines give recommendations on wind velocity predictions for wave prediction purposes. The regulation refer to an obsolete Norwegian standard that has been replaced by the Eurocodes (NS EN 1991-1-4, 2009). The procedure provided in NS EN 1991-1-4 (2009) will be described here combined with relevant information in NVE (2003) that is not provided in the Eurocode, such as a duration factor.

The Eurocode (NS EN 1991-1-4, 2009) considers that the wind velocity fluctuates with time and thus is composed of a mean and a fluctuating component. The mean wind velocity, v_m, is determined from the basic wind velocity, v_b, and the height variation of the wind.

Basic wind

The basic wind velocity depends on wind climate and includes influences of wind direction and season of the year on the fundamental value of the wind velocity, $v_{b,o}$. The basic values of the wind velocity are characteristic values having annual probability of exceedance of 0.02 (p = 0.02), which is equivalent to a mean return period of 50 years.

The fundamental value of the basic wind velocity, $v_{b,o}$, is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and season, at 10 m above ground level in open country terrain with low vegetation, corresponding to terrain category II as defined in Table 4.1 of EN 1991-1-4 (2010). Thus, basic wind velocity, v_b, can expressed as follows[2]:

$$v_b = c_{alt} \cdot c_{dir} \cdot c_{season} \cdot v_{b,o} \tag{3.2}$$

where c_{alt} is the altitude factor, c_{dir} is the directional factor, c_{season} is the seasonal factor.

The National Annex (NA EN1991, 2009) provides values of c_{alt}, c_{dir} and c_{season} for Norway:

- The recommended value for c_{season} in EN 1991-1-4, (2009) is 1.0.
- The value of c_{dir} can in all cases be taken as 1,0, however the possibility of a lower values for certain directions are provided in the National Annex (NA EN1991, 2009) for different regions (see Tabell NA.4 (901.4) in NA EN1991, (2009), and the value of c_{dir} can range from 0.6 to 1.0. The lowest value c_{dir}=0.6 is for example given for East and South-East direction in Møre og Romsdal (ytre), while c_{dir} is 1.0 for South-West and West directions for the same region.
- The altitude factor, c_{alt}, accounts for increase in wind velocity with altitude above sea level. The factor can be determined from Equation NA.4(901.1) in NA EN1991 (2009), and is a function of the altitude above sea level as well as the geographic region accounted for in Table NA.4(901.2) in NA EN1991 (2009) and reproduced in Table 3.1 below. Three geographic regions are identified, Region 1 (No: Område 1) embraces South of Norway however excluding North-Trøndelag, Region 2 (No: Område 2) includes North-Trøndelag, Nordland and Troms, and finally Region 3 (No: Område 3) includes Finnmark and Svalbard. The formula given for the c_{alt} is such that c_{alt} is always larger than or equal to 1.0. Furthermore, the formula results in that c_{alt} scales the fundamental basic wind velocity to a value less than or equal to 30 m/s. The multiplication of c_{alt} and the fundamental basic value of the wind velocity ($v_{b,o}$) will be highest for a certain predefined maximum elevation and have a value of 30 m/s in all three regions. This can be understood by studying Tabell NA.4(901.3) in NA EN1991 (2009).

Table 3.1 Regions and values of H and H_0 in calculation of c_{alt}

Region		H_0 (m a.s.l)	H_{topp} (m a.s.l)
Region 1	South of Norway, excluding North Trøndelag	900	1500
Region 2	North-Trøndalag, Nordland, Troms	700	1300
Region 3	Finnmark, Svalbard	400	1000

[2] This expression applies when the effect of altitude (c_{alt}) is not included in the basic velocities ($v_{b,o}$), as for example is the case for the values given in the Norwegian National Annex.

The value of $v_{b,o}$ in Eq. (3.2) is given in the relevant National Annex, such as this for Norway in NA EN1991 (2009) in a table (Tabell NA.4(901.1)) and on a figure (Figur NA.4 (901.1)). In Norway the fundamental basic value ranges from $v_{b,o}$= 22 m/s to 31 m/s. Offshore these values are in some cases higher or up to 33 m/s.

The basic characteristic wind values (such as v_b) are defined to have annual probabilities of exceedance of 0.02. Thus, to estimate the 10 minutes, mean wind velocity having the probability p for an annual exceedance is determined by multiplying the basic wind velocity by the probability factor, c_{prob}, given by the following expression:

$$c_{prob} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))} \right)^n \tag{3.3}$$

where K and n are given in NA EN1991 (2009) as 0.2 and 0.5 respectively.

- Example: for probability p=0.001 representing mean return period of 1000 years, the probability factor calculated from the above formula is: c_{prob} =1.16.

Hence, the basic wind velocity having probability p for the annual exceedance can be expressed as follows (note this is not a direct expression from the Eurocode):

$$\begin{aligned} v_{b,p} &= c_{prob} \cdot c_{alt} \cdot c_{dir} \cdot c_{season} \cdot v_{b,o} \\ &= c_{prob} \cdot v_b \end{aligned} \tag{3.4}$$

Mean wind

The characteristic basic value of the wind velocity is defined, as previously mentioned, at 10 m height above the ground and for a certain terrain roughness. The mean wind velocity, v_m, at a certain site additionally depends on the actual terrain roughness over which the wind blows, as well as the elevation to consider, i.e. height, z. The mean wind velocity is determined as follows:

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \tag{3.5}$$

where $c_r(z)$ is the roughness factor and $c_o(z)$ is the orthography factor. These factors are further discussed below.

The orthography factor, $c_o(z)$, accounts for the increase of the wind speed over isolated hills and escarpments only, i.e. not undulating and mountainous regions. The largest increase of the wind velocities occurs near the top of a slope or escarpments. Situations for which the effect of orthography is to be taken into account are given in EN 1991-1-4, (2009). Otherwise, the orthography factor is to be taken as $c_o(z)$ =1.0.

The procedure recommended by the Eurocode to determine the roughness factor at height z is given by the following expression:

$$c_r(z) = k_t \cdot \ln\left(\frac{z}{z_o}\right) \qquad \text{for } z_{\min} \le z \le z_{\max} \quad (z_{\max} = 200\text{ m}),$$
$$c_r(z) = c_r(z_{\min}) \qquad \text{for } z \le z_{\min} \tag{3.6}$$

where z_o is the roughness length provided in Table 4.1 of EN 1991-1-4 (2010), and k_t is a terrain factor that accounts for the actual terrain roughness estimate z_o with the roughness for terrain category II $z_{o,II}$ = 2 m (used when defining the fundamental basic value $v_{b,0}$).

The terrain factor, k_t, is expressed as follows:

$$k_t = 0.19 \cdot \left(\frac{z_o}{z_{o,II}}\right)^{0.07} \tag{3.7}$$

Equation summarizing the Eurocode prediction

Summarizing the above procedure for the basic and mean wind velocities, the mean velocity with p=0.02 (mean return period of 50 years) can be expressed as:

$$\begin{aligned} v_m(z) &= c_r(z) \cdot c_o(z) \cdot c_{alt} \cdot c_{dir} \cdot c_{season} \cdot v_{b,o} \\ &= c_r(z) \cdot c_o(z) \cdot v_b \end{aligned} \tag{3.8}$$

and the mean wind velocity with probability p of exceedence can be expressed as:

$$\begin{aligned} v_{m,p}(z) &= c_r(z) \cdot c_o(z) \cdot c_p \cdot c_{alt} \cdot c_{dir} \cdot c_{season} \cdot v_{b,o} \\ &= c_r(z) \cdot c_o(z) \cdot c_p \cdot v_b \\ &= c_r(z) \cdot c_o(z) \cdot v_{b,p} \end{aligned} \tag{3.9}$$

- The Eurocode uses the 10 minute mean wind velocities, which are to be used in prediction of wind generated waves on reservoirs when following the Norwegian Dam Safety Regulations (NVE, 2003). However, duration of the wind must also be considered.

Duration factor added

The general expression given by Eq. (3.1) includes all the same influences that are given in Eq. (3.9), and additionally incorporates a duration factor. The duration factor is used to convert the given wind speed duration, e.g. 10 minutes in the case of the mean wind calculated according to EN 1991-1-4 (2010), to the required duration. Formula for this purpose is provided in NVE (2003). The formula is derived from monitoring data from five meteorological stations in Norway and expressed as follows (Harstveit, 2002):

$$\frac{v_{m,T}}{v_{m,10\min}} = e^{-0.01(T-10)^{0.5}} \qquad 10\min \le T \le 360\min \tag{3.10}$$

where T is the required duration of the wind velocity.

Thus, a duration factor can be expressed as:

$$c_T(T) = e^{-0.01(T-10)^{0.5}} \qquad 10\,\text{min} \le T \le 360\,\text{min} \tag{3.11}$$

And an expression for the mean wind velocity with probability of exceedance, p, and duration T is as follows:

$$\begin{aligned} v_{m,p,T}(z) &= c_T(T)\cdot c_r(z)\cdot c_o(z)\cdot c_p\cdot c_{alt}\cdot c_{dir}\cdot c_{season}\cdot v_{b,o} \\ &= c_T(T)\cdot c_r(z)\cdot c_o(z)\cdot c_p\cdot v_b \\ &= c_T(T)\cdot c_r(z)\cdot c_o(z)\cdot v_{b,p} \\ &= c_T(T)\cdot v_{m,p}(z) \end{aligned} \tag{3.12}$$

And accounts for the same influences that are included in the general expression Eq. (3.1).

3.2.2 Wind velocity for wave prediction in UK, USA and Canada

Methodologies used in the UK, USA and Canada for calculating wind velocities to use in wave prediction is reflected upon in the following. The methodology used in the UK is comparable to the one described above from the Eurocodes, but in the USA and Canada a different approach is taken using available monitoring data.

UK, Wind velocity according to ICE (2015)

In the United Kingdom (UK) methodology for calculation of wind velocities for the purpose of predicting waves on reservoirs for consideration of wave overtopping and dam freeboard, is provided in guidelines called “Floods and Reservoir Safety” (ICE, 2015). The fourth edition of these guidelines were issued in 2015.

The fundamental basic wind speed used in the calculation is a maximum hourly wind speed with a mean return period of 50 years (1 in 50 years) reduced to sea level. This wind speed is denoted U_{50} and obtained from a chart given in ICE (2015) that refers to the British national Annex to the Eurocode (BS EN 1991-1-4). As in the general expression Eq (3.1), a number of adjustment factors are introduced to adjust these wind speed for: return period (factor f_T), altitude (factor f_A), direction (factor f_N), duration (factor f_D), and surface roughness or overwater effects (factor f_W). The wind speed required for calculation of the significant height is then given as:

$$U = U_{50}\, f_T\, f_A\, f_W\, f_D\, f_N \tag{3.13}$$

Comparing expression Eq (3.13) and Eq (3.12) the different adjustment factors can be related as follows:

- f_T corresponds to factor c_p
- f_A corresponds to factor c_{alt},

- f_W is given in Table 3.2. This factor corresponds to factor $c_r(z)$ (see footnote[3]), however relates to the fetch length (discussed in the following chapter).
- f_D corresponds to factor $c_T(T)$
- f_N corresponds to factor c_{dir}

The influence of topography ($c_0(z)$) and season (c_{season}) are not considered, and in fact these factor will generally be set to 1.0 in Eq (3.12) (following the Eurocode) when considering wave generation on reservoirs.

The wind speed adjustment overwater factor (f_W) from ICE (2015) and the values in Table 3.2, are also given in NVE (1981). Saville et al., (1962) originally presented these factors and explained that while the overwater factors represent a reasonable approximation for the land -to-water wind variation for the two reservoir from which they were obtained, a more detailed investigation of the problem is advisable, particularly where relatively small reservoirs are involved. Thus, the overwater factors in Table 3.2 should be used thoughtfully, considering that these are median values of observed data from only two reservoirs, and for which considerable scatter in the data points is reported by Saville et al., (1962).

- ICE (2015) uses the hourly wind velocities in calculation of the significant wave height, however, assigns the duration factor f_D to backcalculate to for example 10-minute mean wind.

Table 3.2 Wind speed adjustments (f_W) over water (table from ICE (2015) with reference to *Saville et al., (1962)*)

Fetch length (m)	*1000*	*2000*	*4000*	*8000*	*12000*
f_W	1.1	1.16	1.23	1.28	1.31

USA: Wind velocity according to USBR (2012)

In the USA the U.S. Department of the Interior, Bureau of Reclamation (USBR) issued in 2012, a design standard on Embankments dams that includes a chapter on freeboard calculations (USBR, 2012a). The USBR uses processed wind data in predicting wave heights on reservoirs for uses in freeboard and riprap analysis. The wind data to be used was compiled by the Battelle National Laboratories for wind energy purposes but processed in a program created by the USBR into hourly probability relationships. The program generates a site-specific curve of hourly probability of wind versus wind velocity. The wind velocities are usually measured over land. Thus, the wind velocities need to be adjusted to overwater wind velocities. A graph is provided for this purposes (Figure B-2 in

[3] With the terrain being lake, Terrain category I of NS EN 1991-1-4 (2009)

USBR (2012)) which originates from the Coastal Engineering Manual (USACE, 2008) of the US Army Corps of Engineers (USACE).

Additionally, normal freeboard requirement are to be checked considering a wave runup and setup from a wind of 100 mph (miles per hour) or 44.7 m/s.

- USBR (2012) uses the hourly wind velocities in calculation of the significant wave height.

Canada: Wind velocity according to SEBJ (1997)

In Canada, like in the USA, wind velocities are generally predicted from processed wind data, e.g. from the historical climate database located on Environment Canada's website (see e.g. Damov and Warren (2012)). The wind speed is generally observed at 10 m above the ground measured in km/h. Wind direction is also provided. These records are analyzed statistically, e.g. using Gumbel's distribution law, and wind rose sketched for wind velocities of relevant return period.

The monitoring data is usually from inland meteorological stations and the inland wind velocity values must be converted to overwater values. The following equations, e.g. presented in SEBJ (1997), are for example used for this transformation to overwater values:

$$\begin{aligned} U_e &= 1.5U_t && \text{for } U_t \le 50 \text{ km/h} \\ U_e &= 0.643U_t + 42.9 && \text{for } 50 < U_t < 120 \text{ km/h} \\ U_e &= U_t && \text{for } U_t \ge 120 \text{ km/h} \end{aligned} \tag{3.14}$$

where U_e is the overwater wind speed (in km/h) and U_t is the inland wind speed.

In formulas for calculating the significant wave height according to SEBJ (1997) (discussed in Chapter 3.3.3) a wind stress factor is used but not the wind velocity. The wind stress factor relates to the wind velocity through the following expression originating from the Shore Protection Manual issued by USACE in 1984 (SPM, 1984)

$$U_a = 0.71U_e^{1.23} \tag{3.15}$$

where U_a is the wind stress factor, and U_e is the overwater wind velocity in m/s.

- The SEBJ (1997) uses the hourly wind velocities in calculation of the significant wave height.

3.2.3 Wind velocities for wave height predictions, Norway

In wave prediction on reservoirs in Norway wind velocities are required for calculation of the significant wave height, both for freeboard consideration and for dimensioning of riprap stones in an upstream wave protection.

Wind velocity to use in relevant design situations

For freeboard considerations, wind velocity with a mean return period of 50 years is associated with the design water level (in Norwegian: *dimensjonerende flomvannstand* (DFV)) while a wind with a return period of 1000 years, is associated with the highest regulated water level (the full supply level (in Norwegian: *Høyeste regulerte vannstand* (HRV)) in freeboard calculations. NVE (2003) informs that a velocity 30 m/s for wind blowing over the reservoir can be used as an alternative to the 50 year wind used in combination with the design water level (DFV) for determining the freeboard.

Conversely, for dimensioning of riprap for erosion protection of the upstream slope, a wave generated by a wind with return period of 1000 year is to be used, alternatively waves generated by wind with velocity 30 m/s. This is somewhat in contrast with the wind velocity for freeboard consideration where 30 m/s is associated with a return period of 50 years. NVE (2003) regulation states that in cases where it is considered appropriate the wind velocity can be calculated according to the Norwegian Standard for wind loads, which today is the Eurocode (NS EN 1991-1-4, 2009) discussed above. The exact phrase in Norwegian is:

«Dersom det anses hensiktsmessig å gjennomføre fullstendige beregninger skal beregning av vindhastighet med henholdsvis 50 og 1000 års gjentaksintervall utføres i ht. NS 3491-4 "Vindlaster".» (NS 3491-4 has been replaced by NS EN 1991-1-4)

Thus the NVE (2003) regulation seemingly does not require that the wind loads are calculated by the national standards on wind loads, but rather gives this as an option to use where appropriate or suitable.

Comparison of suggested values in NVE(2003) and the Eurocode

It is of interest to compare the value provided by the NVE (2003) regulation for the 1 in 1000 year of the 10 minute mean wind velocity, to the range of values that can be obtained by NS EN 1991-1-4 (2009).

In Norway the fundamental basic value given in the National Annex to NS EN 1991-1-4 (2009), range from $v_{b,o}$= 22 m/s to 31 m/s (See NA EN1991 (2009): table (Tabell NA.4(901.1)) and on a figure (Figur NA.4 (901.1))). Figure 3.1 shows relation between the fundamental basic values ($v_{b,o}$) and the resulting 10 minute mean wind velocity (line vm,10 on the Figure 3.1) calculated according to Eq 3.8, with the following values: c_{alt} =1, c_{dir} =1, c_{season} =1, $c_o(z)$ =1 and $c_r(z)$=1.1726. The roughness factor ($c_r(z)$) is calulated according to Eq 3.6 supported by Eq 3.7, with z= 10 m, z_o=0.01 and $z_{o,II}$=0.05. The value z_o=0.01 represents terrain category I („terrengruhetkategori I") desribed as „*Lakes or flat and horizontal area with negligble vegetation and without obstaces*". (In Norwegian: „*kystnær, opprørt sjø. Åpne vidder og strandsoner uten trør eller busker*"). Figure 3.1 also plots the fundamental basic value against itself (line vb,o) and the 60 minute mean wind (line vm,60) calculated by multiplying ($v_{b,o}$) with the duration factor calculated from Eq 3.11 using T=60 minutes. The NVE (2003) recommended value of 30 m/s for the 50 year wind for freeboard considerations is also plotted in Figure 3.1.

The Figure 3.1 demonstrates that when the fundamental basic value is larger than 25.6 m/s the 10 minute mean wind velocity (one in 50 year) is larger than the 30 m/s recommended by NVE (2003) for this value. The directional factor, c_{dir}, has a value of 1 in the plots of Figure 3.1. However, when applying the NS EN 1991-1-4 (2009) to calculate the wind velocity, the directional factor should be used with the values as given in the National Annex, NA EN1991 (2009), still with due consideration of local conditions potentially arising in mountainous regions. Thus, for certain wind directions the 10-minute mean wind velocity may have lower value than that presented in the figure provided that c_{alt} =1.

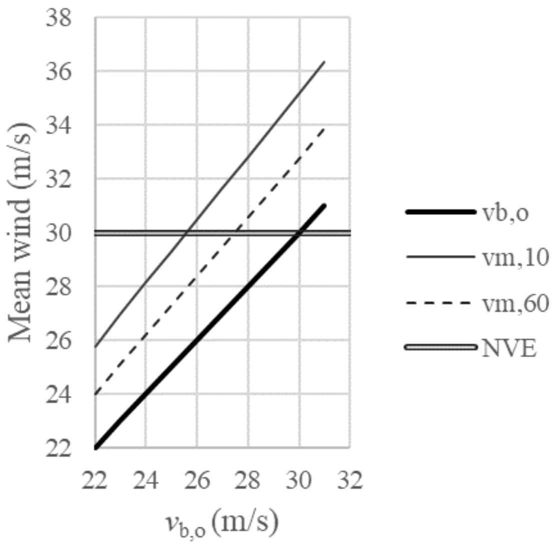


*Figure 3.1 Mean wind velocities (10 minutes (*vm,10*) and 60 minutes (*vm,60*)) with return period of 50 years according to NS EN 1991-1-4 (2009) versus fundamental basic value, $v_{b,o}$. The value of the* NVE (2003) *recommended10 minute mean wind velocity (30 m/s) by is also plotted (labelled:*NVE*), as well as the basic wind velocity (labelled:* vb,o*)*

The value of c_{alt} ranges from 1,0 for areas with elevations lower than a reference value, to maximum of $30/v_{b,o}$ for the maximum elevation to consider in a given Region. For example, for $v_{b,o}$ = 20 m/s, and the maximum elevation in any of the three regions defined, the value of c_{alt} is 1.5 (and obviously the multiplication $c_{alt}v_{b,o}$ becomes 30 m/s). Figure 3.2 demonstrates the effect of c_{alt} for the different regions in Norway. Values of c_{alt} are given in Tabel NA.4.(901.3) in the National Annex to NS EN 1991-1-4 (2009).

About 250 reservoirs in Norway have a regulated water elevation of 900 m a.s.l. or larger (see Table 3.3). These reservoirs are mainly in South-Norway. It can be expected that the value of c_{alt} *for* these reservoirs will be larger than 1.0 and thus result in a higher value of the mean wind velocity than given in Figure 3.1., however this depends also on the fundamental basic value for the particular location.

- For dams and reservoirs in Norway, the 10 minute mean wind velocity is given in the NVE (2003) regulation or assessed from the NS EN 1991-1-4 (2009).

(a) c_{alt} *(b)* $c_{alt}\, v_{b,o}$

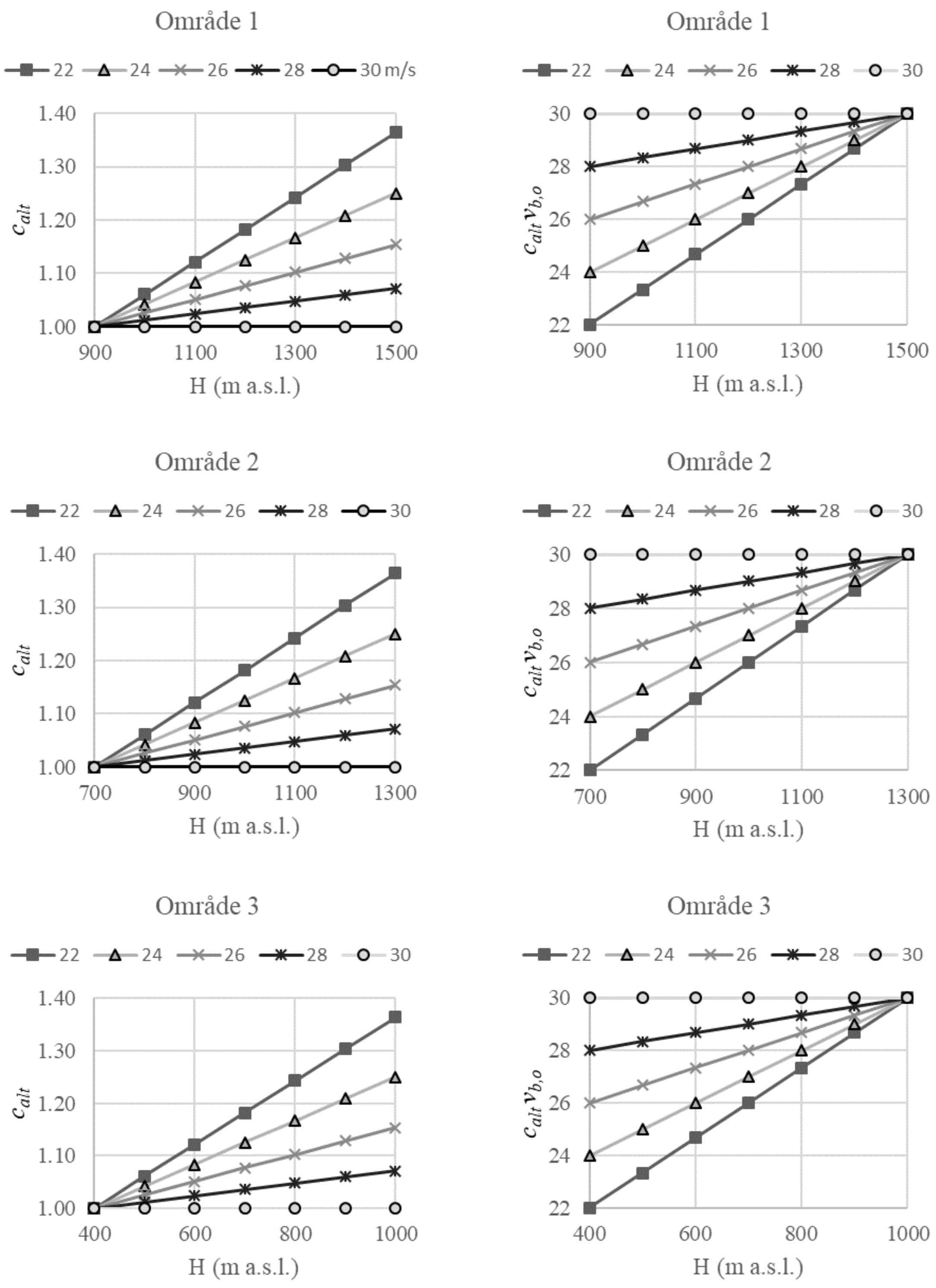


Figure 3.2 Effect of c_{alt} *for the different regions. Column (a) shows variation of the altitude coefficient with elevation above seelevel and the fundamental basic value,* $v_{b,o}$*. Column (b) shows variatin of the multiplication* $c_{alt}\, v_{b,o}$ *with with elevation above seelevel and the fundamental basic value,* $v_{b,o}$*. (See also Tabel NA.4.(901.3).*

Table 3.3 Number of reservoir in Norway with full supply level (no: HRV) within ranges defined in Table 3.1 for the calculation of c_{alt} for the different regions. (Data extracted from the file: VannkraftMagasin.shp downloaded from NVE website).

FSL (HRV) m a.s.l.	*0-400*	*>400-700*	*>700-900*	*>900-1000*	*>1000-1300*	*>1300-1500*
Number of reservoirs	*1490*	*492*	*224*	*76*	*143*	*31*

3.2.4 Mountain reservoirs in Norway – examples with Eurocode approach

Below are a few examples of predicting the design wind velocity according to the Eurocodes. The predictions are compared to the wind velocity values recommended by NVE (2003).

Kreklevotni Reservoir. The reservoir registered with the highest full supply elevation in m above sea level, is Kreklevotni Reservoir with FSL at 1477 m a.s.l. This reservoir is located in Aurland Municipally in Sogn and Fjordana County, or Region 1 (South Norway) defined in Table 3.1. According to the national annex NA EN1991, (2009), the basis wind speed is to be taken as $v_{b,o}$ =25 m/s in Aurland. Here c_{alt} has to be calculated, since the H=1477 m a.s.l. and thus larger than H_o =900 in Table 3.1. The value of c_{alt}= for H=1477 m and Region 1 is: c_{alt}=1.19; which results in v_b = 29.8 m/s (with c_{dir} =1, c_{season} =1) and $v_{m,10}$ =35 m/s (with $c_r(z)$ =1.172 and $c_o(z)$=1). (Without consideration of altitude, $v_{m,10}$ would have the value of 29.4 m/s).

Sildvikvatn with full supply level at 680 m a.s.l. The reservoir is located in Narvik Municipally Nordland County- According to the national annex NA EN1991, (2009), the basis wind speed is to be taken as $v_{b,o}$ =28 m/s in Narvik. Here c_{alt} is not calculated, since the H=680 m a.s.l. and thus less than H_o =700 Table 3.1. Thus c_{alt} =1 c_{dir} =1, c_{season} =1, $c_r(z)$ =1.172 and $c_o(z)$=1 results in $v_{m,10}$ =32.8 m/s.

Altevatnet, full supply level is at 265 m a.s.l. Alta Kommune, Finnmark, According to the national annex NA EN1991, (2009), the basis wind speed is to be taken as $v_{b,o}$ =28 m/s in Alta. Here c_{alt} is not calculated, since the H=265 m a.s.l. and thus less than H_o =400 in Table 3.1. Thus c_{alt} =1 c_{dir} =1, c_{season} =1, $c_r(z)$ =1.172 and $c_o(z)$=1 results in $v_{m,10}$ =32.8 m/s.

Suoikkatjavri, Kvænangen, Troms, with full supply level at 529 m a.s.l. According to the national annex NA EN1991, (2009), the basis wind speed is to be taken as $v_{b,o}$ =28 m/s in Kvænangen. Here c_{alt} is not calculated, since the H=529 m a.s.l. and thus less than H_o =700 in Table 3.1. Thus c_{alt} =1 c_{dir} =1, c_{season} =1, $c_r(z)$ =1.172 and $c_o(z)$=1 results in $v_{m,10}$ =32.8 m/s .

- For the examples above of mountain reservoirs in Norway, higher overwater wind velocities are predicted according to NS EN 1991-1-4 (2009) than the recommendation in NVE (2003) of 30 m/s, while for others the NS EN 1991-1-4 (2009) may result in lower overwater wind velocities.

* The overwater wind velocities are discussed further in Section 3.2.7.

It is further of interest to compare the standard values to potential available monitoring data. Monitoring data from two reservoirs were obtained and are discussed in the following two subsection.

3.2.5 The case of the Nesjøen Mountain Reservoir

The Nesjøen Reservoir is a mountain reservoir in South-Trøndelag (see Figure 3.3). The Reservoir is further used as a case in Chapter 4.3.6 and Chapter 7.6.2. One weather station is located downstream the associated dam, the Nesjøen dam (see Figure 3.3b). The purpose of the weather station is primarily to complement precipitation measurements for potential wind related corrections (Even Loe, Statkraft, email communication). Records from this weather station were provided by Statkraft, the dam owner.

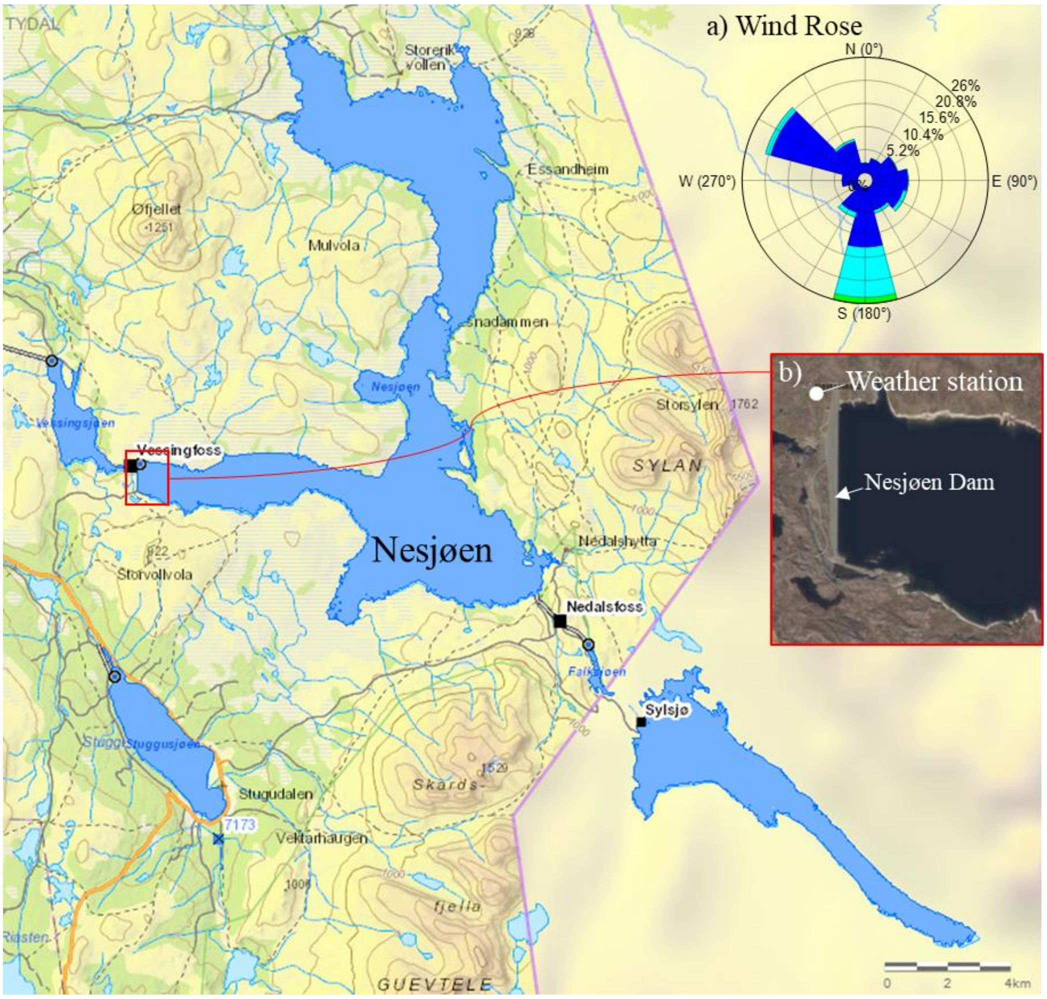


Figure 3.3 The Nesjøen reservoir, with wind rose in the upper right corner from a weather station located downstream the Nesjøen dam as shown in the (The map is obtained from NVE map services, the location of the weather stations is provided by Statkraft).

The wind velocity and wind direction records start 25th of October 1990 and 18th of November 2014, respectively. The wind velocity record provides a value for each hour and represent a value for the mean hourly wind velocity (60 minute mean wind) (Even Loe, Statkraft, email communication). The measured wind velocity is plotted in Figure 3.4, the wind velocity plotted with a red colour can be associated to a recorded wind direction. A wind rose for this period (November 2014 to April 2019) is plotted in Figure 3.5a and the measured wind velocity against wind direction is plotted in Figure 3.6.

The wind rose (Figure 3.5a) demonstrates that the wind blows most frequently from West-North-West (285° to 315°) and from the South (165° to 195°) directions. Frequency of the wind direction is generally less than 10% and generally of lower intensities (see also Figure 3.6). Statkraft has informed that, the weather station is on a mast, but probably at a lower elevation than the dam crest (Even Loe, Statkraft, email communication). Still, it is unclear whether and to which extent values recorded by the weather station are affected by the station's location downstream the dam. Although, the wind rose strongly indicates such effects. Wind directional factors for East Trøndelag from the Norwegian National Document to NS EN 1991-1-4 (2009), are shown in Figure 3.5b, this shows that wind velocities blowing from the North, North-East and East can be reduced by multiplying with c_{dir}=0.8, 0.7 and 0.8, respectively. Thus, lower wind intensities from these directions are to be expected in the area. To investigate further easterly wind direction in the area other available stations in the period 2014 to 2019 were considered. However, it must be recognized that local conditions influence the wind conditions and thus the different stations are not directly comparable.

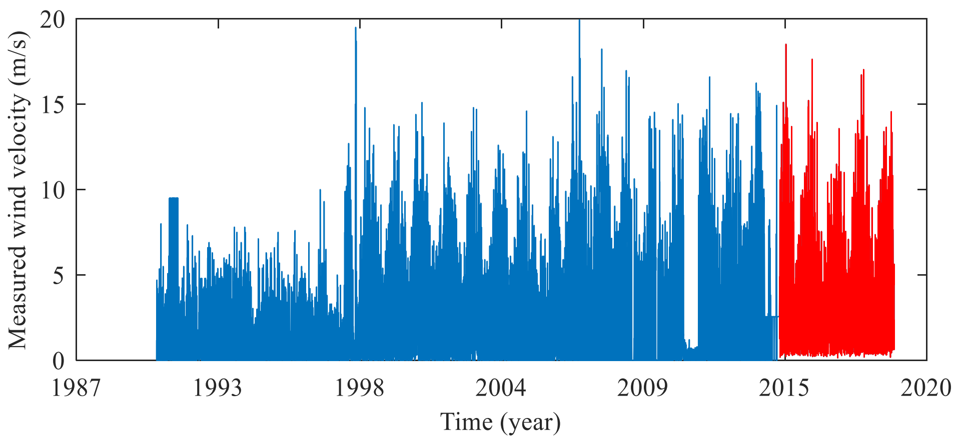


Figure 3.4 Nesjøen Reservoir. Measured wind velocity (hourly values). The wind velocity plotted in red can be associated with recorded wind direction.

On eklima.no one can request a wind rose for a certain station to be plotted from available data. The available stations on eklima.no, closest to Nesjøen dam but still 45 to 50 km away, are stations 69380 Meråker-Vardetun (ca 45 km North of Nesjøen), 68290 Selbu II (ca 45 km North-West of Nesjøen) , and 10380 Røros Lufthavn (ca. 50 km South-West of Nesjøen) (See Figure 3.7). Wind roses for the period 2014 to 2019 for these stations are plotted in Figure 3.8. The Selbu station and Nesjøen both have a frequent wind blowing from West-Northernly direction, approximately aligned with the valley Tydal between these stations. Compared to the Nesjøen stations, the stations in Figure 3.7 have a larger percentage of the wind blowing from East (75° to 105°) and/or South-East directions

(ESE 105° to 135°, and SSE 135° to 165°), however not as nearly as frequent wind blowing from the South (ca 10%) as for the Nesjøen station (ca 26%). The low intensity and infrequency of the wind blowing from the eastly directions, i.e. across the reservoir towards the dam, indicate that the Nesjøen station is in the lee of the dam for the wind blowing from these wind directions, and one cannot assume that overwater effects are included in the data.

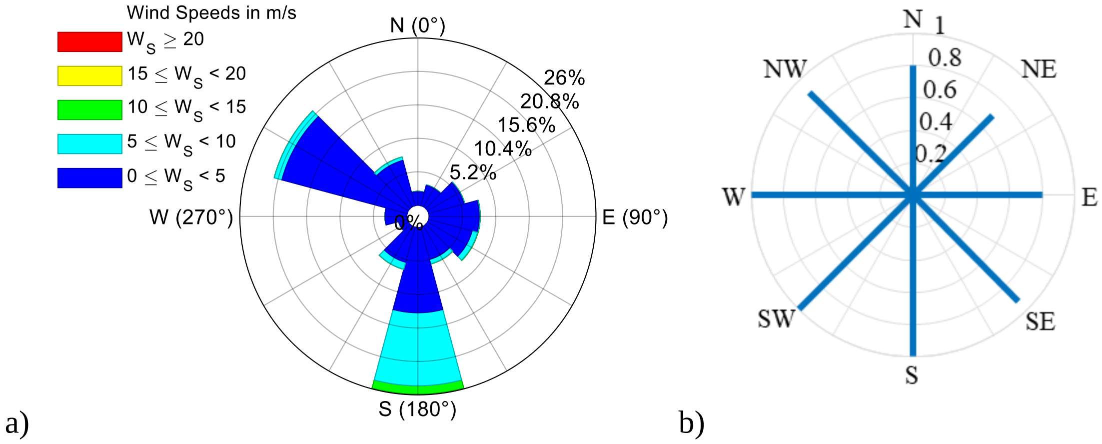


Figure 3.5 Wind Rose for Nesjøen dam and directional factors for East-Trøndelag. a)Wind Rose (12 directional subdivisions) from measurement at Nesjøen Dam for the period November 2014 to April 2019. The legend for the wind velocities is in m/s. (Plotted using a Matlab script from Daniel Pereira Valadés). b) Directional factors for East-Trøndelag from the Norwegian National Document to NS EN 1991-1-4 (2009).

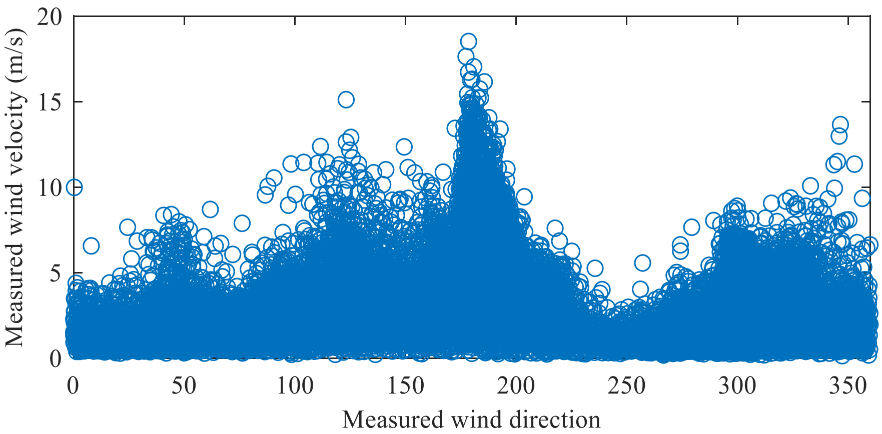


Figure 3.6 Wind velocity versus wind direction measured at Nesjøen Dam for the period November 2014 to April 2019.

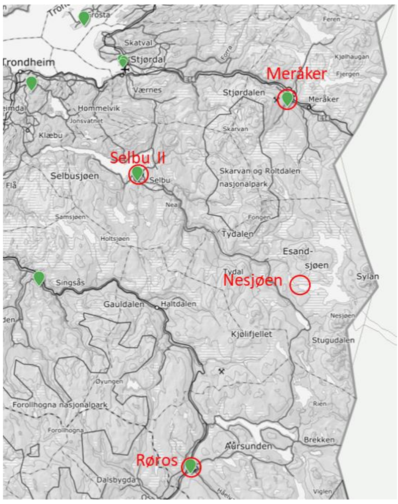


Figure 3.7 Location of the weather stations considered (map from eklima.no). For an idea of distances: the distance (linear) between the station at Nesjøen and Selbu is for example about 45 km.

The wind roses for the period 2014 to 2019 from Meråker-Vardetun and Selbu II (see Figure 3.8), give south-easterly winds (135° to 165°) as the most frequent, 20% and 25% respectively compared to 5% for the Nesjø station. Reading from the wind roses, the wind velocities (10 minute mean value) for Meråker-Vardetun and Selbu II , are generally with highest wind velocities in the range 10.3 to 15.2 m/s for the south-easterly direction, compared to 10-12 m/s for the 60 minute mean wind for Nesjø (reading from Figure 3.6, the green colour for the south-east direction in Figure 3.5a is hard to see), which scales roughly to about 11 to 13 for the 10 minute mean wind value with duration factor (see Eq. 3.11). This comparison of the velocity intensities indicates that the wind velocity recorded at Nesjøen wind station is not necessarily influenced by potential effects of the reservoir (i.e. overwater wind speed effect are not included in the measurements), and that any comparison of standard prediction to the values recorded at the Nesjø station should rather consider a roughness coefficient of z_{II} than z_I.

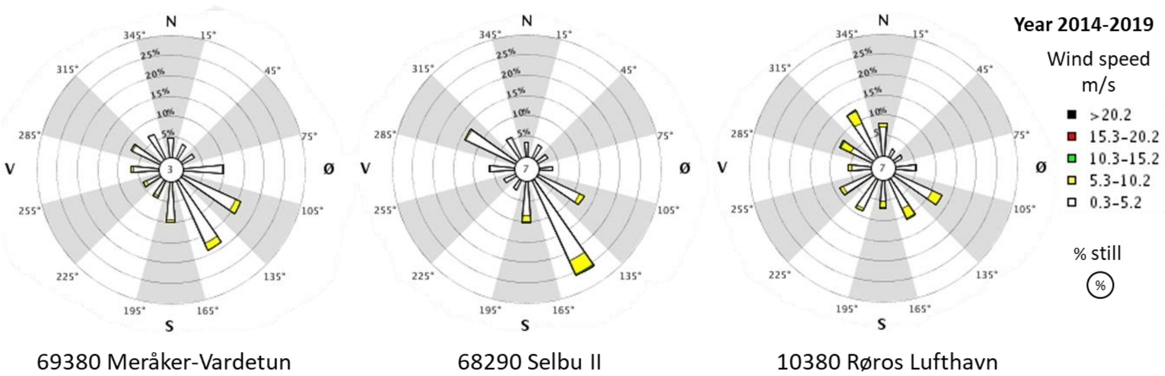


Figure 3.8 Wind Roses for the period 2014 to 2019 (12 directional subdivisions of the data) from measurement at available weather stations closest to Nesjøen. Legend for the wind velocities (in m/s) is provided. (The wind roses are created by eklima.no).

Wind velocities predicted from measured data compared to standard values

The wind time series on Figure 3.4 shows a different pattern in period 1990 to 1998, with considerably lower wind velocities. The data before 1998 is here considered spurious and not used further. The maximum recorded wind velocities (irrespective of the wind direction) for each year from 1998 to 2014 were extracted and an Extreme Value Distribution evaluated using Matlab. The distribution is provided in Figure 3.9, while the cumulated probability is plotted in Figure 3.10. Additionally, in Figure 3.10, data points are plotted representing wind velocities obtained for return periods of 50 year and 1000 year using the following predictions/recommendations: prediction from the probability density function obtained using the monitoring data (Figure 3.9); prediction using Eurocode standard approach (considering both roughness for terrain category I (z_{I}) and II (z_{II})); and recommendations by NVE (2003). The data for these points is given in Table 3.4. The predictions consider that Nesjøen is in Tydal, Sør-Trøndelag with $v_{b,0}$ = 25 m/s according to the Norwegian National Document to NS EN 1991-1-4 (2009).

As previously noted, the wind velocity record provides a value for each hour and represent a value for the mean hourly wind velocity (60 minute mean wind). Thus, for the comparison in Figure 3.10, the Eurocode and NVE (2003) predictions/recommendation of the 10 minute mean wind values are scaled with the duration factor (see Eq. 3.11). The duration factor has the value of 0.932 when scaling from 10 minute mean wind to obtain a 60 minute mean wind. Both the10 and 60 minute mean wind values are given in Table 3.4.

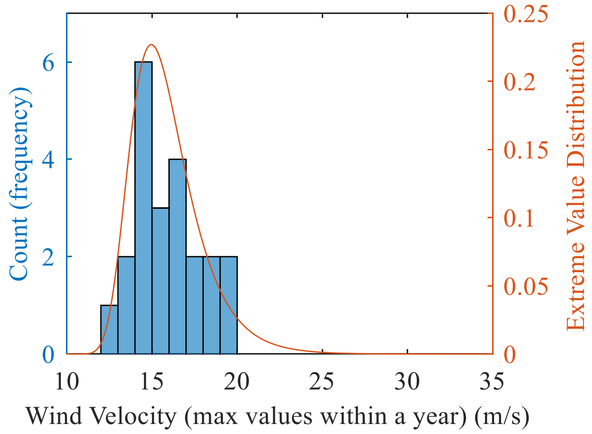


Figure 3.9 Nesjøen dam. Extreme Value Distribution created from the data plotted in the histogram showing counts of annual maximum wind velocity measured at Nesjøen Dam for the period 1998 to 2019.

The data points in Figure 3.10 and Table 3.4 show that the predictions obtained from the data based probability distribution, compares reasonably to prediction using the Eurocode approach with roughtess category z_{II}. Conversely, the wind velocities recommended by NVE (2003) for overwater wind, compares reasonably to the wind velocity predicted with the Eurocode approach for roughness category z_{I}. Potential utilization of the wind velocity measurements from the Nesjøen weather station for wave prediction would have to consider the overwater adjustements of the wind speed. This could for example be done by mulitplying with the rougness factor according to the Eurocode as for the values in Table 3.4, or by using an adjustment factor which additionally considers the fetch length (see Table 3.2).

Table 3.4 Nesjøen, wind velocity predictions

Return period Years	Wind velocity (m/s) 60 minute mean wind				
	EVD Nesjøen	Eurocode z_{II}	Eurocode z_I	NVE	EVD Nesjøen, z_I
50	21.3	23.4	27.3	28	24.9
1000	26.2	27.1	31.6	32.5	30.6
Return period Years	**Wind velocity (m/s) 10 minute mean wind**				
	EVD Nesjøen	Eurocode z_{II}	Eurocode z_I	NVE	EVD Nesjøen, z_I
50	22.9	25.2	29.3	30	26.8
1000	28.1	29.1	33.9	34.8	33

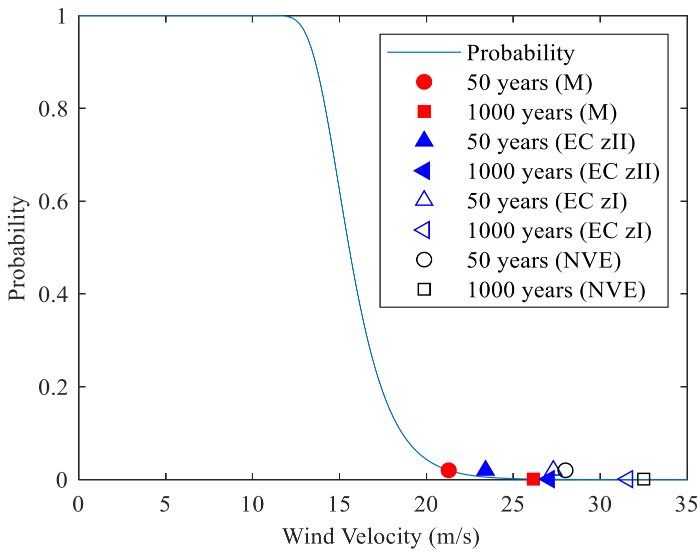


Figure 3.10 Nesjøen Dam. Cumulative Extrema Value Distribution (see distribution in Figure 3.7) and data points for 60 minute mean wind velocity of return periods 50 and 1000 year obtained with different methods.

Summary

- The Nesjøen weather station is installed to complement precipitation measurements, and thus is not conveniently located for measuring overwater wind. In fact, wind blowing from the east towards the dam are infrequent (ca. 1% in the period 2014 to 2019) and of low intensity (< 5 m/s). This may indicate that the wind measured at the Nesjøen station is in the lee of the dam for the easterly winds.

- However, in East-Trøndelag, winds blowing from North, North-East and East can respectively be reduced with a direction factor 0.8, 0.8 and 0.7 for wind predictions with the Eurocode.

- Wind velocities obtained according to NS EN 1991-1-4 (2009) for roughness category II (z_{II}) compare reasonably to predictions basing on Extreme Value Distribution of the recorded data at the Nesjøen weather station. Multiplying the Nesjøen values with a roughness factor for potential overwater effects (roughness category I (z_I)), brings these closer to NVE (2003) recommended values (see Table 3.4) for wave prediction.

- NVE recommended wind velocities in wave prediction are somewhat higher than obtained from predictions basing on the Eurocode. Furthermore, predictions basing on the Eurocode compare better to the measured values from Nesjøen.

3.2.6 The case of the Aursjøen Mountain Reservoir

The Aursjøen Reservoir is a mountain reservoir in Skjåk Innlandet (previously Oppland). One weather station is located downstream the associated dam, the Aursjøen dam (see Figure 3.11). Records from this weather station were provided by Statkraft (by email from Robert von Hirsch, Statkraft) the dam owner, and start 13th of December 1997 and extend to 14th of May 2019. The wind velocity record provides a value for each hour and is considered to represent a value for the mean hourly wind velocity (60 minute mean wind). The wind direction is also measured but the measurement is not complete and thus not used here. The measured wind velocity is plotted in Figure 3.12.

The weather station is in a small bay on the south side of the reservoir and some local effect may be included in the records.

Figure 3.11 Aursjøen. Location of the weather station.

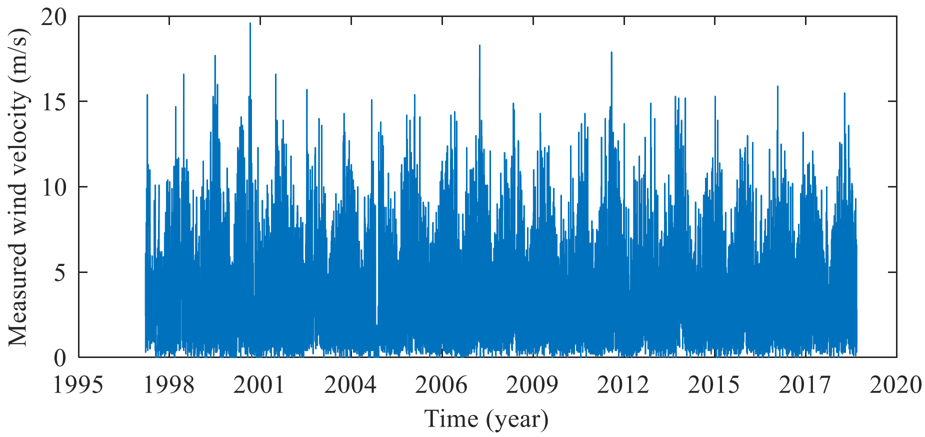


Figure 3.12 Aursjøen. Measured wind velocity. (Hourly values)

Wind velocities predicted from measured data compared to standard values

The maximum recorded wind velocities (irrespective of the wind direction) for each year were extracted from the records and an Extreme Value Distribution evaluated using Matlab. The distribution is provided in Figure 3.13, while the cumulated probability is plotted in Figure 3.14. Additionally, in Figure 3.14, data points are plotted representing wind velocities obtained for return periods of 50 year and 1000 year using the following predictions/recommendations: prediction from the probability density function obtained using the monitoring data (Figure 3.12); prediction using Eurocode standard approach (considering both roughness for terrain category I (z_I) and II (z_{II})); and recommendations by NVE (2003). The predictions consider that Aursjøen is in Skjåk, Innlandet with $v_{b,0}$ equal to 25 m/s according to the Norwegian National Document to NS EN 1991-1-4 (2009).

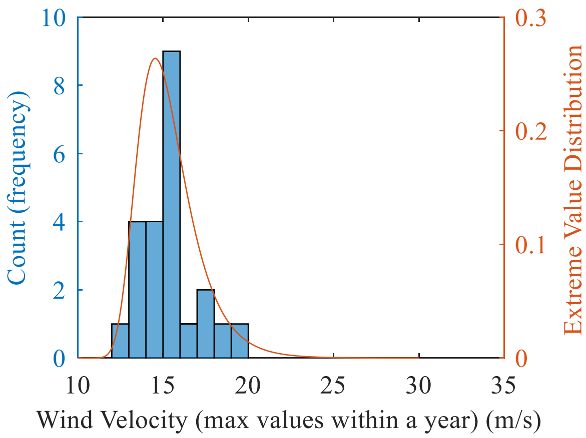


Figure 3.13 Aursjøen dam. Extreme Value Distribution created from the data plotted in the histogram showing counts of annual maximum wind velocity measured at Aursjøen Dam for the period December 1997 to May 2019.

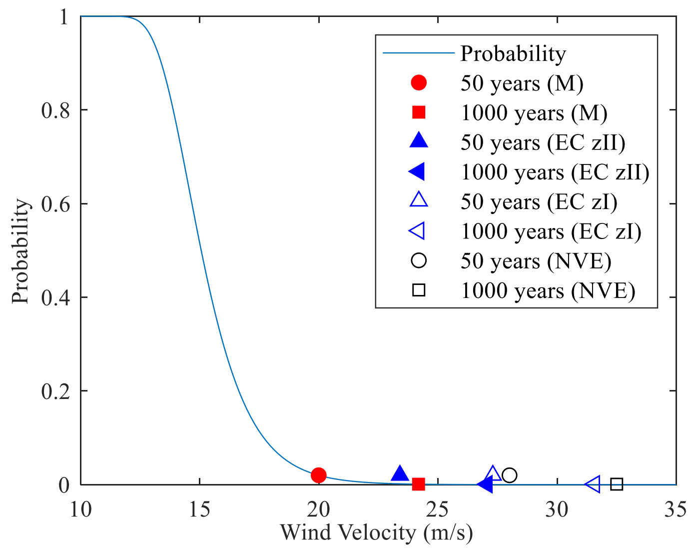


Figure 3.14 Aursjøen Dam. Cumulative Extrema Value Distribution (see distribution in Figure 3.13) and data points for 60 mintue mean wind velocity of return periods 50 and 1000 year obtained with different methods.

Summary

- Wind velocities obtained according to NS EN 1991-1-4 (2009) for roughness category II (z_{II}) result in higher values than predictions basing on Extreme Value Distribution of the recorded data at the Aursjøen weather station. The values are 12% and 17% higher for the 50 year and 1000 year event, respectively.

- Wind velocities obtained according to NS EN 1991-1-4 (2009) for roughness category I (z_I) compare reasonably to NVE recommended wind velocities for wave prediction.

3.2.7 Wind over water

For a given meteorological conditions wind velocities over water are higher than those over land. This has been accounted for in wave predictions on reservoirs by overwater wind factors presented by Saville et al. (1962) and given in Table 3.2. These adjustment factors are referred to in ICE (2015) as well as NVE (1981). SEBJ (1997) also converts inland velocities using Eq 3.14, however does not consider fetch lengths. In wind prediction by Eurocode, the roughness length (z_o) accounts for the surface over which the wind blows. The roughness factor for terrain category I, is z_o=0.01 m, i.e. for wind blowing over lakes or flat and horizontal area with negligible vegetation and without obstacles. Furthermore, for terrain category 0, the roughness length is z_o=0.003 m, i.e. for wind blowing sea or coastal area exposed to the open sea. The roughness factor (c_r) is 1.17 and 1.266 for a roughness length of 0.01 and 0.003 respectively. These roughness factors can be compared to the adjustment factors given in Table 3.2.

For consideration of wind speed over water, USBR (2012) presents a chart with a reference to USACE (2008). USACE (2008) in turn refers to Resio and Vincent (1977), who studied wind measurements over the Great Lakes in USA. The graph in USACE

(2008) (and USBR (2012)), given in Figure 3.15, shows the ratio of wind speed over water to wind speed over land as a function of wind speed over land. According to the graph, the ratio decreases with increased wind speed over land. This is because of the behavior of water roughness as a function of wind speed, i.e. the wind generated waves increase the surface roughness (roughness length z_o).

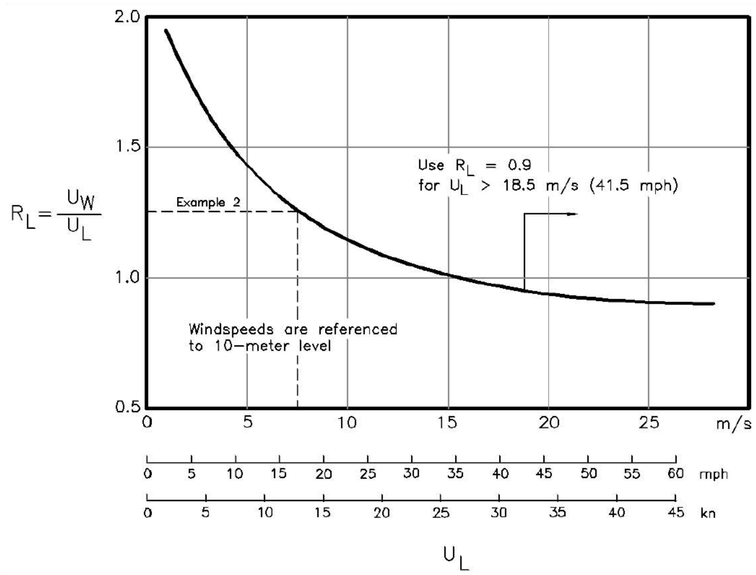


Figure 3.15 Ratio of wind speed over water (U_W*) to wind speed over land (*U_L*) as a function of wind speed over land (From USACE (2008)).*

Bishop et al., (1992) investigated the ratio R_L of overwater wind speed to overland wind speed at 10 m, measured at Lake Erie North America in 1979, as a function of overland wind speed and air/water temperature difference. The measured overland wind speed is less than 10 m/s. The study indicated that the ratio R_L is sensitive to air-water temperature differences of +/- 5°C. However, it also confirmes the trend that the ratio reduces with increased overland wind speed, at least for the reservoir considered.

The conversion of inland wind speeds to overwater wind speeds by the different approaches is compared in Figure 3.16. Both the approaches in ICE (2015) and USBR (2012) (USACE (2008)) refer to studies from 1962 and 1977, respectively, which base on measurements on certain reservoirs and lakes and can only be considered as simplified estimations. For example Saville et al. (1962) recommend a more detailed investigation into this. With regard to the approach in SEBJ (1997) (Eq 3.14,), it is unclear how this is obtained, although it can be assumed that this is a result of the wind measurements carried out on the La Grande Complex. Considering this and the plots in Figure 3.16, it seems reasonable for inland wind prediction by the Eurocode that result in inland wind speed lower than 25 m/s, to convert to overwater wind speed a roughness factor c_r = 1.17, regardless of the fetch length. However, this may be conservative (on the safe side) for wind speeds relevant for the wave prediction in Norway, which will in many cases result in inland wind speeds larger than 25 m/s.

Comparison to mainly the SEBJ (1997) approach to estimate the overwater effect from the inland wind speed, indicates that a limit of the overwater wind speed of 30 m/s (recommended in NVE (2003)) may be reasonable for reservoirs where the inland wind speed is between 25 to 30 m/s. However, too high for lower inland wind speeds compared to both SEBJ (1997) and Eurocode (with c_r = 1.17) , and possible too low for inland wind speeds larger than 30 m/s compared to the recommendation of SEBJ (1997) (and Eurocode with c_r = 1.17). Whether flattening of the curve for the higher wind velocities is appropriate for mountain reservoirs in Norway, as SEBJ (1997) considers for reservoirs in Canada, can only be confirmed with meteorological measurements.

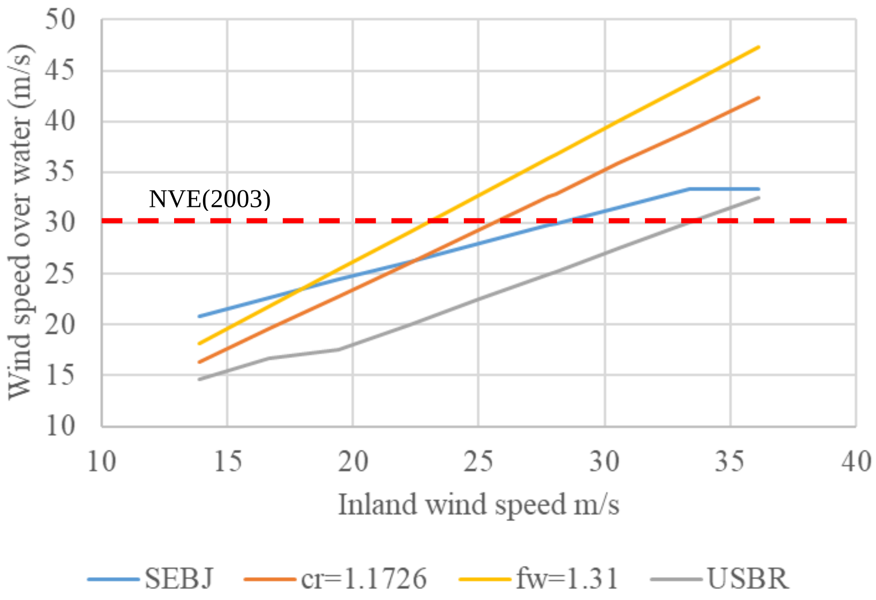


Figure 3.16 Plot of wind speed over water to wind speed over land as a function of wind speed over land. Comparison of different approaches. The NVE (2003) recommended 30 m/s as overwater wind speed is shown as red dashed line. (in the legend cr is the roughness factor for z_o=0.01 (Eurocode), and fw=1.31 is the highest adjustment factor from Table 3.2. SEBJ and USBR refer to the guidelines SEBJ (1997) and USBR(2012))

3.3 Wave forecasting

The earliest wave forecasting formulas go back more than 150 years or the work of Stevenson (1853). The early formulas were all empirical and based on visual sea wave measurements in coastal areas. The first formulas considered the wave height as a function of fetch only (e.g. Stevensson‘s formula), but later included the consideration of both fetch and wind speed (e.g. Molitor’s formula (Creager et al., 1945)). However, the first documented relationships among various wave-generation parameters and resulting wave conditions were presented by Sverdrup & Munk (1947).

In the formulation of the relationship, Sverdrup & Munk (1947) introduced the concept of a significant wave to quantitatively define what had previously been referred to as “larger waves”. The significant wave height was defined as waves having average height and period of the on-third of the highest waves. Sverdrup & Munk (1947) explain the selection, of the one-third of the highest waves as the significant wave, from their experience indicating that a careful observer who attempts to establish the character of the higher waves will tend to record the significant waves as defined by them (Sverdrup and Munk). Bretschneider, (1964) further clarifies that it was found from the analysis of wave records that the significant wave height, usually denoted H_s, is approximately the height reported from visual observation. In statistical analysis of wave records, the significant wave height, H_s, is defined as average height of the highest one-third of the highest waves of a given wave group. In the wave spectrum approach, H_{m0}, is used to denote the estimate of significant wave height from spectral analysis

The first steps in any simple prediction of waves are to predict extreme wind velocities and calculate the fetch lengths or areas over which the wind can act. Prediction of the wind velocities were discussed in the previous section. Here the wave forecasting formulas will be introduced first, before discussing calculation of the fetches. However, it should be noted that the method of calculating the fetches are an important part of the different formulas introduced in the following sections.

3.3.1 Dimensional considerations and the effective fetch (USA)

Sverdrup & Munk (1947) studied the processes by which energy is transmitted from wind to waves and derived at energy equations which relate wave height and wave speed to wind speed, duration of wind and fetch. The solutions to the equation relating waves to wind was represented as relations between nondimensional parameters and included certain constants resulting from mathematical integration that could not be determined by theory alone, thus measurements were required to arrive at empirical prediction formulas. Thus, the empirical prediction method considers that interrelationships among dimensionless wave parameters will be governed by universal law. The most fundamental of these laws is probably the fetch-growth law (USACE, 2008). The expectation is that the wave will reach a stationary fetch-limited state of development in a situation where a constant wind speed acts in the same direction over a fixed fetch. In this situation, wave heights remain constant through time but will vary along the fetch.

Sverdrup, Munk and Bretschneider (SMB) method.

Sverdrup & Munk (1947) derived at the following relations based on dimensional consideration (shown here with the presentation of Bretschneider (1964)):

$$\frac{gH}{U^2} = \Psi_1\left[\frac{gF}{U^2}, \frac{gt}{U}\right] \tag{3.16}$$

$$\frac{C}{U} = \Psi_2\left[\frac{gF}{U^2}, \frac{gt}{U}\right] \tag{3.17}$$

$$\frac{gT}{U} = \Psi_3\left[\frac{gF}{U^2}, \frac{gt}{U}\right] \tag{3.18}$$

in the above equations (Eq. 3.16-3.18) H is the wave height T the wave period, F the fetch length, U the wind speed and t the duration of that wind speed, C is the wave speed. Ψ_1, Ψ_2, and Ψ_3 are functional relations that must be determined using measured wave data. The non-dimensional parameters gH/U^2, gF/U^2, gt/U, C/U, and gT/U are defined respectively as the wave height parameter $\hat{H}$, the fetch parameter $\hat{F}$, the wind duration parameter $\hat{t}$, the wave speed parameter, and the wave period parameter $\hat{T}$.

Sverdrup & Munk (1947) produced graphs showing the relation between the nondimensional parameters, and thus relating significant wave height and wave period to the wind speed, wind duration and fetch length. When Sverdrup & Munk (1947) conducted their work available empirical data was incomplete and some years later Bretschneider (1951) revised the curves by utilizing field data from Abbotts lagoon, California (Johnson, 1950) and other sources, as well as laboratory data. Further revisions were made again by Bretschneider (1958). These forecasting relationships are generally referred to as the SMB method, a short for the Sverdrup, Munk and Bretschneider method.

In the 1977 edition of the Shore Protection Manual (SPM, 1977) the original SMB equations have been modified and are expressed as follows:

$$\frac{gH_s}{U^2} = 0.283\tanh\left[0.0125\left(\frac{gF}{U^2}\right)^{0.42}\right] \tag{3.19}$$

$$\frac{gT}{U} = 2\pi \cdot 1.2 \cdot \tanh\left[0.077\left(\frac{gF}{U^2}\right)^{0.25}\right] \tag{3.20}$$

in addition to a formula for gt/U not provided here.

The SPM (1977) explains that for inland waters, such as lakes and reservoirs, fetches are limited by the shoreline and are often long in comparison to width. In these cases, the fetch width may result in wave generation that is significantly lower than those that would develop over more open waters for the otherwise same generating conditions. For such cases, the SPM (1977) recommends to estimate the fetch *F* with the effective fetch as defined by Saville (1954) (see also Saville et al. (1962)). Saville (1954) method of calculating the effective fetch considers radials extended +/-45° from the point of interest in the upwind direction to the boundary of the water body. The methodology is discussed in Section 3.4.

The SMB-S relations were also used in the 1978 edition of the UK guidelines, Floods and Reservoir Safety (ICE, 1978), however with an older version, or from 1952 according to Owen and Steel (1988), of the equations presented above.

3.3.2 A wave spectrum approach (Europe/USA/Canada)

In the 1950s, or around the same time that the SMB method was presented, theoretical concepts developed that considered the random nature of the sea surface and a representation of this by a wave spectrum. The concept of a spectrum is based on work by Joseph Fourier (1768-1830), who showed that almost every function ***f**(t)* (or ***f**(x)* etc.) can be represented over a certain interval as the sum of an infinite series of sine and cosine functions with harmonic wave frequencies. This can be expanded to include series that represent surfaces. Thus, any surface, such as the surface of the ocean (or a water body), $\zeta(t)$, can be represented as an infinite series of sines and cosines functions oriented in all possible directions.

Some important contributions for the wave spectrum approach include this of Pierson, Neumann and James (1955), Phillips (1957), Miles (1957), Pierson and Moskowitz (1964). Furthermore, the contribution of Hasselmann et al. (1973) in the Joint North Sea Wave Project (JONSWAP), a cooperative venture by a number of scientists in Germany, Holland, England and the United States, has been of importance, and e.g. influenced wave predictions for dam engineering purposes. This new approach was for example included in the Shore Protection manual issued in 1984 (SPM, 1984), as well as in ICE (1996) and later in ICE (2015).

JONSWAP

The JONSWAP project included extensive measurements of wind and wave conditions at several locations in the southern North Seas, along with analysis of the collected data. The spectral energy density function which came out of the JONSWAP experiment, is referred to as the JONSWAP spectrum and can be written as a function of the frequency as follows:

$$E_J(f) = \alpha \mathrm{g}^2 \left(2\pi\right)^{-4} f^{-5} \exp\left(-\frac{5}{4}\left(\frac{f}{f_m}\right)^{-4}\right) \chi^{\exp\left(\frac{-(f-f_m)^2}{2\sigma^2 f_m^2}\right)} \tag{3.21}$$

where $\alpha = 0.032\left(f_m U / g\right)^{2/3}$, g is the acceleration gravity, f is frequency, f_m represents the frequency at the maximum of the spectrum, χ=3.3 and σ has the numerical valued 0.07 for $f \leq f_m$ but 0.09 for $f \geq f_m$.

The following equation relates the peak frequency, f_m, to the wind speed (U) and the fetch (F),

$$\hat{f}_m = \frac{f_m U}{g} = 3.5\left(\frac{gF}{U^2}\right)^{-0.33} = 3.5\hat{F}^{-0.33} \tag{3.22}$$

From which the dimensionless peak period, $\hat{T} = 1/\hat{f}_m$, can be obtained as:

$$\hat{T} = 0.286\hat{F}^{0.33} \tag{3.23}$$

The significant wave height can be related to the standard deviation of the water-surface displacement $\zeta(t)$ (see e.g Hoffman and Karst (1975)). The standard deviation is equal to the square root of the variance, represented here as $\left\langle \zeta^2 \right\rangle$, and the significant wave height can be expressed as:

$$H_s = 4\sqrt{\left\langle \zeta^2 \right\rangle} \tag{3.24}$$

where the variance of the water surface elevation $\zeta(t)$ can be obtained by integrating the spectral density function from Eq (3.21) over all frequencies as follows,

$$\left\langle \zeta^2 \right\rangle = \int E_J(f)df \tag{3.25}$$

Expression for the integration can be extracted from Hasselmann et al. (1973) and written as follows,

$$\int E_J(f)df = 1.6\times 10^{-7}\left(\frac{gF}{U^2}\right)\times\frac{U^4}{g^2} \tag{3.26}$$

Inserting this expression into Eq (3.24) and multiplying with g/U^2 on each side of the equation results in:

$$\frac{gH_s}{U^2} = 0.0016\left(\frac{gF}{U^2}\right)^{1/2} \tag{3.27}$$

The equation can be expressed as $\hat{H}_s = k\hat{F}^{1/2}$, where k has in Eq (3.27) the value 0.0016. However, another value of k, k=0.00178, has been used by Hydraulics Research, Wallingford UK (Owen, 1987) and is recommended also in British dam safety guidelines such as ICE (2015). Owen (1987) refers to a personal communication with Hasselmann (of the JONSWAP project) for the value k=0.00178. The prediction formula from

Hasselmann et al. (1973) bases on measurements over a wide range of fetches with the site of the wave study comprising 160 km measuring profile. Furthermore, the wind speeds range upto 15 m/s (Hasselmann et al. (1973), but bases primarily of wind speed between 5-10 m/s (according to the information provided on figures in Hasselmann et al. (1973) , see also Resio and Vincent (1979)). This should be kept in mind in the use of the formulas.

JONSWAP in SPM (1984)

In the 4th edition of the Shore Protection Manual (SPM ,1984) the wave prediction equations are expressed as follows:

$$\frac{gH_{mo}}{U_A^2} = 0.0016\left(\frac{gF}{U_A^2}\right)^{1/2} \tag{3.28}$$

$$\frac{gT_m}{U_A} = 0.2857\left(\frac{gF}{U_A^2}\right)^{1/3} \tag{3.29}$$

H_{mo} is the spectrally based significant wave height (in deep water H_{mo} is approximately equal to the significant wave height H_s which is based on counting and measuring individual waves), T_m is the peak spectral period, and U_A is the wind stress factor or adjusted wind speed. The wind speed U (in m/s) is converted to a wind stress factor by the following formula:

$$U_A = 0.71U^{1.23} \tag{3.30}$$

Bishop et al. (1992) note that SPM (1984) does not provide a comparative justification for using U_A instead of the wind velocity U used in SPM (1977) and in the JONSWAP formulation (Hasselmann et al., 1973) and thus further compared these two approaches. Bishop et al. (1992) conclude that comparison with measured wave data from various sources reveals that the use of U_A leads to overprediction of wave height and period, furthermore that the SPM (1984) approach leads to the poorest statistical results of four different methods used. Thus the SPM (1984) approach with the use of U_A can be considered conservative for the benefit of safety.

The SPM (1984) discusses narrow fetch conditions, such as on lakes and reservoirs, and explains, basing on a study by Resio and Vincent (1979), that straight-line fetch shows reasonable aggreement to measured data. Hence, SPM (1984) emphasizes *that the effective fetch should not be used with the above formulas.* Thus the effective fetch approach by Saville when dealing with narrow fetches was no longer recommended.

Owen (1987) discusses this reversal of previous recommendations in the SPM (1984) and notes that details are missing from the paper of Resio and Vincent (1979). Such details would enable a third party can check the validity of their findings on which the recommendation in the SPM (1984) bases, with regard to only using straight line fetches.

Modified JONSWAP in CEM (USACE (2008) and USBR (2012))

The 4th edition of the Shore Protection Manual (SPM,1984) has been updated and replaced by the Coastal Engineering Manual CEM, numbered EM-1110-2-1100, the current version was issued in 2008 (with changes 4 in 2015). New formulas in SI units are presented for simplified wave prediction. The new formulas seemingly base on JONSWAP spectrum with some modifications, however, this is not clear in the manual and no reference is provided for this updated formulation. USBR (2012) has incorporated these new formulas for dam engineering purposes and presents them for use with English units.

The 2008 edition of (USACE, 2008) provides the following equation that can be used to determine whether or not waves in a particular situation can be categorized as fetch limited:

$$t_{F,U} = 77.23 \frac{F^{0.67}}{U^{0.34} g^{0.33}} \tag{3.31}$$

where $t_{F,U}$ is the time required for waves crossing a fetch of length F under a wind velocity U to become fetch-limited.

USACE (2008) further provides the following simplified wave prediction formulas,

$$\frac{gH_{mo}}{U_*^2} = 4.13 \times 10^{-2} \left(\frac{gF}{U_*^2} \right)^{1/2} \tag{3.32}$$

$$\frac{gT_p}{U_*} = 0.651 \left(\frac{gF}{U_*^2} \right)^{1/3} \tag{3.33}$$

where T_p is the wave period, U_* is friction velocity $U_* = C_D^{1/2} U_{10}$, where C_D is a drag coefficient expressed as,

$$C_D = \frac{U_*^2}{U_{10}^2} = 0.001 \left(1.1 + 0.035\, U_{10} \right) \tag{3.34}$$

USACE (2008) explains that SI units should be used in the Eqs (3.31-3.34) above. Furthermore, USACE (2008) recommends that straight-line fetches be used to define the fetch length, F.

Appendix B of USBR (2012) outlines computations for embankment dam freeboard analyses for wind loadings. USBR (2012), expresses the Eqs (3.31-3.33) above. for use with English units (or United States customary units (USCU)) on the following form:

$$t_{\min} = t_{F,U} = 1.87 \frac{F_{\mathrm{mi}}^{0.67}}{U_{MPH}^{\;0.34}} \tag{3.35}$$

Where t_{min} is defined as the minimum duration required to generate the maximum wave height (provided in hours), the F_{mi} is the fetch in miles, and U_{MPH} is the wind velocity over water in miles per hour.

$$H_s = 0.0245\, F_{mi}^{1/2} U_{MPH} \left(1.1 + 0.0156 U_{MPH}\right)^{1/2} \tag{3.36}$$

$$T = 0.464 F_{mi}^{1/3} U_{MPH}^{1/3} \left(1.1 + 0.0156 U_{MPH}\right)^{1/6} \tag{3.37}$$

USBR (2012) recommends a procedure for estimating the fetch over an inland reservoir having an irregularly shaped shoreline. The procedure consists of drawing nine radials from the point of interest at 3-degree intervals and extending these radials to the shoreline. This method will be discussed with other methods for calculating the fetch in Section 3.4. *USBR (2012) emphasizes that the effective fetch must not be used with methods of the standard*.

JONSWAP in ICE (2015)

In the 2015 edition of guidelines, Flood and Reservoir Safety (ICE,2015) the wave prediction equations are expressed as follows: (ICE, 2015)

$$H_s = 0.00178 U \left(\frac{F}{g}\right)^{1/2} \tag{3.38}$$

The Eq 3.38 can be rewritten on a dimensionless form as:

$$\frac{gH_s}{U^2} = 0.00178 \left(\frac{gF}{U^2}\right)^{1/2} \tag{3.39}$$

(ICE, 2015) also provides the following formula for the peak wave period, T_p, to consider for wave action on UK reservoirs:

$$T_p = 0.0712 F^{0.3} U^{0.4} \tag{3.40}$$

ICE (2015) discusses different methods of defining the fetches depending on the shape of the water surface, including methods discussed by Owen and Steel (1988) and Herbert et al. (1995). These methods will be discussed later in Section 3.4.

The Donelan (1980) model (Canada)

Donelan (1980) developed a model for wave prediction at the National Water Research Institute, Canada. The model based on extensive measurements carried out in Lake Ontario, Canada. The data was used to obtain a full wave energy/frequency/direction spectra. Donelan (1980) based on the same reasoning as Hasselmann et al. (1973) used in the JONSWAP project, but worked with different data to obtain the spectrum.

The following equation resulted from Donelan‘s work for bodies of water with an irregular shoreline (obtained from Owen (1987)):

$$\frac{gH_s}{(U\cos\Theta)^2} = 0.00366\left(\frac{gF_\phi}{U^2\cos^2\Theta}\right)^{0.38} \tag{3.41}$$

$$\frac{gT_p}{U\cos\Theta} = 0.54\left(\frac{gF_\phi}{U^2\cos^2\Theta}\right)^{0.23} \tag{3.42}$$

in which $\Theta = \theta - \phi$ is the angle between the direction of the wind (θ) and the predominant wave direction ϕ, and F_ϕ is the straight-line fetch along the wave direction ϕ.

The Donelan (1980) wave prediction formulas are of somewhat different form than the formulas previously presented. The main difference lies in that Donelan does not assume coincident wind and wave directions. The formulas have been compared to other methods and provided good results for Lake Ontario (Bishop, 1983), but grossly overpredicted wave heigths for most wind directions when compared to measurements on the Megget Reservoir in the UK (Owen and Steel, 1988). The prediction formulas seemingly have not gained much acceptance, however the Donelan‘s idea of using the fetch along the predominant wave direction has been used in the UK as discussed in the next section (see also Section 3.4) along with the JONSWAP formulas as presented below as Donelan/JONSWAP.

The DonJon (Donelan/JONSWAP) approach (an option in ICE (2015))

Owen and Steel (1988) compared the different wave prediction formulas, and suggested to combine Donelan‘s consideration on using a fetch along the predominant wave direction and the JONSWAP formulas. They referred to this approach as Donelan Jonswap method (DonJon method). Predictions from the DonJon method was favourable in comparison to other methods, (SMB and JONSWAP (with different means of determining the fetch), when compared to measured data on the long and narrow reservoir Loch Glascarnoch. The DonJon, however overpredicted the wave height on the Megget Reservoir when compared to measured data. Owen and Steel (1988) concluded that the DonJon was probably the best of the methods examined, which included SMB-Saville, JONSWAP (with straigth fetch and fetches according to Seymour (1977)), Donelan and DonJon.

The DonJon approach was later recommended by Herbert et al. (1995) for determination of the fetch length on restricted fetches, furthermore Herbert et al. (1995) is referred to in ICE (2015) for cases where more detailed approach is required for determining the fetches, which is the DonJon approach.

The Donelan/JONSWAP formulation can be expressed as follows:

$$H_s = 0.00178U\cos\Theta\left(\frac{F_\phi}{g}\right)^{1/2} \tag{3.43}$$

$$T_p = 0.0712 F_\phi^{0.3} \left(U \cos\Theta\right)^{0.4} \tag{3.44}$$

The method of determining the fetch, F_ϕ, along the predominant wave direction is outlined in Section 3.4.

3.3.3 Dimensional analysis and analysis of monitoring data (Canada) (SEBJ, 1997)

Extensive wind and wave monitoring program was implemented in the 1990's on four large and wide reservoirs of the La Grande Complex in northern Québec, Canada (SEBJ, 1997). Measurements were made from 1992 to 1995 of both wind and waves (Dupuis et al., 1996). The monitoring program included: measurements of waves, measurements of overwater winds at unobstructed sites representative of local conditions; and measurements of inland winds at regional airports (SEBJ, 1997).

The monitoring data revealed that in the case of the large reservoirs studied, the formulas used in the Shore Protection Manual from 1984 (SPM, 1984) that based on the JONSWAP spectrum, resulted in poor hindcast, with overprediction of small wave heights and under prediction of large wave height events, when compared to the measured values from the L Grande Complex reservoirs (large and wide reservoirs). Thus, Dupuis et al. (1996) assumed a general form of the dimensional relations and used the monitoring data to yield the required coefficient and exponent values.

Dupuis et al. (1996) used the following general form of the functional relations Ψ_1, Ψ_2 and Ψ_3 (introduced in Eq. 3.16 and 3.18):

$$\frac{gH}{U_a^2} = \alpha_H \left(\frac{gF}{U_a^2}\right)^{\beta_H} \tag{3.45}$$

$$\frac{gT}{U_a} = \alpha_T \left(\frac{gF}{U_a^2}\right)^{\beta_T} \tag{3.46}$$

$$\frac{gt}{U_a} = \alpha_t \left(\frac{gF}{U_a^2}\right)^{\beta_t} \tag{3.47}$$

where the coefficients α and the exponents β were determined by a linear regression of the available measured data, and U_a is a wind stress factor U_a=0.71 $U^{1.23}$. Linear regression was applied to obtain the coefficients and exponents. The quality of the regressions improved when the wind stress factor was used in lieu of the wind speed, hence U_a in the equations above.

In addition to the measured data, the values of the coefficients α and exponents β, depend on the fetch. **Thus, the method of calculating the fetch and the values of the coefficient and exponents are interrelated and cannot be disassociated**. SEBJ (1997) highlights this fact. Dupuis et al. (1996) and SEBJ (1997) presented a modified version of the effective fetch that considers radials extended +/-90° from the point of interest in the upwind direction to the shoreline. This effective fetch is denoted here $F_{e,180°}$ and is further discussed in Section 3.4.

Extensive analysis of the data resulted in the following equations SEBJ (1997):

$$\frac{gH_{mo}}{U_A^2} = 0.00248\left(\frac{gF_{e,180°}}{U_A^2}\right)^{0.45} \qquad (3.48)$$

$$\frac{gT_m}{U_A} = 0.510\left(\frac{gF_{e,180°}}{U_A^2}\right)^{0.225} \qquad (3.49)$$

and

$$\frac{gt}{U_a} = 31.82\left(\frac{gF_{e,180°}}{U_a^2}\right)^{0.775} \qquad (3.50)$$

where H_{mo} is here the significant wave height = H_s, and T_{02} average wave period. $F_{e,SEBJ}$ is the effective fetch according to Eq 3.56 for the relevant θ.

SEBJ (1997) rewrites the above equations, isolating variables of interest and using the relation between U and U_a (U_a=0.71 $U^{1.23}$) to obtain the following:

$$H_{mo} = 0.001917\, F_{e,180°}^{0.45}\, U^{1.353} \qquad \text{[m]} \qquad (3.51)$$

$$T_{02} = 0.143\, F_{e,180°}^{0.225}\, U^{0.676} \qquad \text{[s]} \qquad (3.52)$$

$$t = 3.21\, F_{e,180°}^{0.775}\, U^{-0.676} \qquad \text{[hour]} \qquad (3.53)$$

with $F_{e,180°}$ in km, and U is an hourly wind in km/h.

The Eqs 3.51 to 3.53, and the SEBJ method of calculating the fetches were incorporated into Norwegian dam safety regulations in 2003 (NVE, 2003). The method is also referred to by the Canadian Dam Association Guidelines (CDA) (according to information in Damov and Warren (2012)), however, along with the method provided in SPM (1984)).

Generally, one can assume that prediction formulas are best suited to conditions that fall within the measured ranges on which the regression bases. The range of the measured data at the reservoirs is not given directly by neither Dupuis et al. (1996) nor SEBJ, (1997), other than with an example in Dupuis et al. (1996) from one station shown in Figure 3.21.

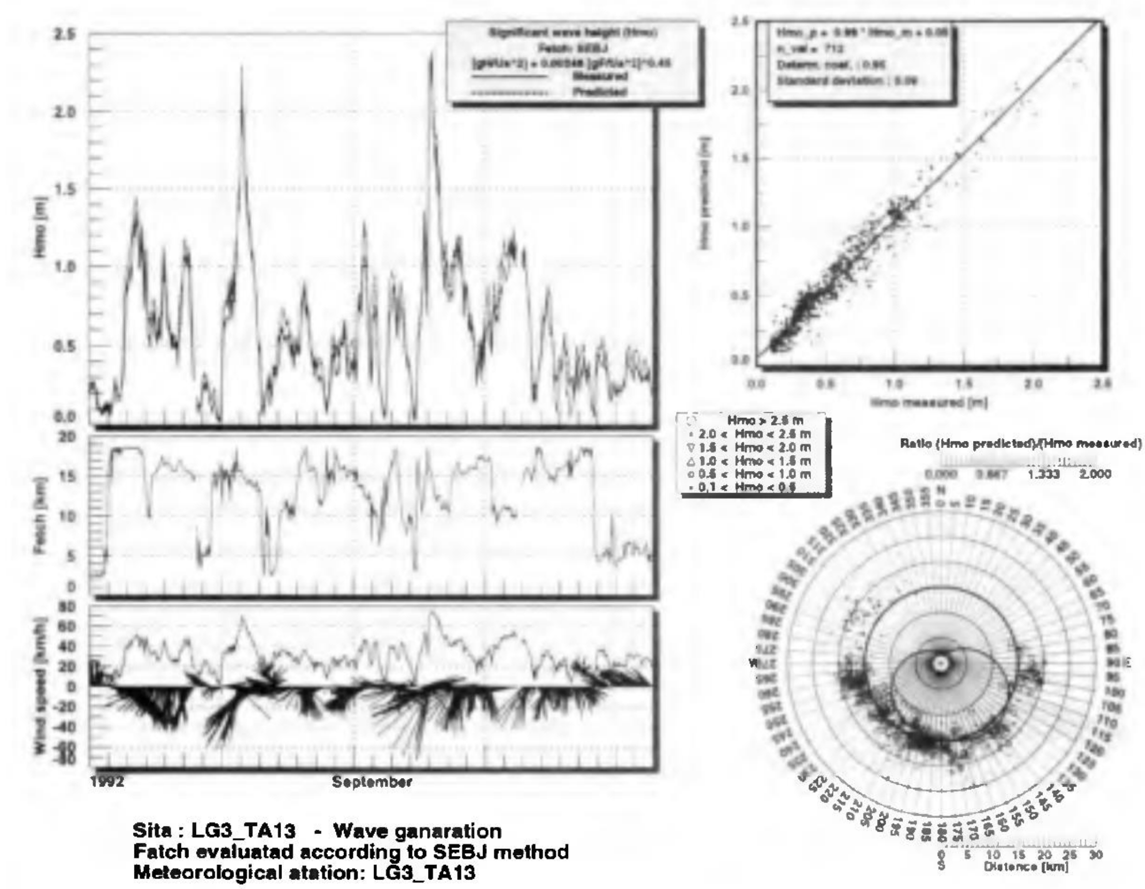


Figure 3.17 Example from Dupuis et al. (1996) of wave hindcast from overwater wind data.

Dupuis et al. (1996) state that hindcast at eight other sites gave similar results. If one assumes that approximately the same range of wind velocities, wave height and fetches were measured at all stations, then the ranges can be estimated from Figure 3.17. Fetches range from about 2 km to about 19 km, wind speed ranges from 0 to 75 km/h (or 0 to about 20.9 m/s), and the measured wave height range from 0 to about 2.4 m. Most of the time the wind is blowing over fetches that are larger than 10 km, and the fetch is over 15 km for about half of the monitoring record. The shortest fetches are about 3 to 5 km and the wind blowing across these shortest fetches is less than approximately 20 km/h (about 6 m/s).

3.3.4 Third generation spectral wave models

The wave forecasting formulas introduced above are simple means of assessing the wave conditions. The wave models have developed since the Jonswap spectra and related work, are referred to as third or fourth generation spectral wave models. Nowadays numerical models, basing on third generation spectral wave models, are used for example in cases where bathymetric configuration is irregular and/or when the wind conditions is not homogeneous over the water domain or changing with time. Such methods are outside the scope of this report but are certainly of interest to cases of large dams and reservoir with complex wind and wave conditions from e.g. topography.

3.4 Procedures for calculating fetch lengths for restricted fetches

3.4.1 General on fetches

A region in which the wind speed and direction are reasonably constant defines a fetch (SPM, 1984). The fetch may be limited by the spreading of isobars or a definite shift in the wind direction (SPM, 1984). A shoreline upwind from the point of interest also limits a fetch. In the open ocean, fetch width for wind wave generation are considered of the same order of magnitude as the fetch lengths. Thus, only the fetch length is considered important, and not the shape of the fetch. However, in the case of wind wave generation in inland waters, such as reservoirs, lakes and bays, the fetch is limited by the geometry defined by the shoreline of the water body. In these cases, the fetches can be long in comparison to the width, and are called restricted fetches

3.4.2 The fetch -overview of calculation procedures

Investigations on wind wave generation on restricted fetches have been carried out by several researchers (Kirby and Dempster, 2006; Owen and Steel, 1988; Saville, 1954; Saville et al., 1962; SEBJ, 1997; Smith, 1991). Saville (1954) introduced the concept of an effective fetch, a calculation procedure that considers radials extended +/-45° from the point of interest in the upwind direction to the shoreline (boundary of the water body). This calculation procedure is further discussed in the following sections. The effective fetch was an attempt to account for the effect of the proximity of the shoreline along the fetch length. It is worth to note that the effective fetch length as defined by Saville (1954) was derived on an empirical bases in order to achieve good agreement between, on one hand the SMB wave prediction curves, and on the other, measurements of wind and wave carried out in reservoirs in the USA between 1952 and 1964. Furthermore, the SMB curves where shown to overpredict wave heights for smaller values of the fetch, and thus the effective fetch was required to adjust the results to better fit measured data. Thus, Saville's effective fetch can be considered as an attempt to calibrate the SMB wave prediction for the cases studied and thus uncertain how this applies in general.

The effective fetch formulation by Saville (1954) was widely used for dam engineering purposes worldwide, and referred to in most standard reference books, such as the SPM(1977) until late 1970' along with the SMB prediction formulation (or curves). However, in in the 1984 edition of the SPM (SPM, 1984), new prediction formulas were introduced based on the JONSWAP spectrum. At the same time, some of the arguments on which the effective fetch was based were no longer considered valid basing on e.g. investigations carried out by Resio and Vincent (1979). Data from inland reservoirs was compared to prediction of the significant wave height H_s made with both an effective and a straight-line fetch. The comparison revealed that straight-line fetch showed reasonable agreement with the data when using the new JONSWAP based prediction formulas, whereas the effective fetch calculations resulted in wave heights that were too low.

Thus SPM (1984) recommended the use of straight line fetches with the JONSWAP based prediction formulas. For irregular shorelines, SPM (1984) further recommends a procedure to determine the fetch length, that considers radials extended +/-12° from the point of interest in the upwind direction to the shoreline.

The recommendation for straight line fetches is also provided in USACE (2008). However, USACE (2008) does not provide the same recommendation for irregular shorelines as SPM (1984), still a procedure for estimating the fetch is given and states that the fetch should be defined so that wind direction variations do not exceed 15° degrees and wind speed variations do not exceed 2.5 m/s from the mean. The consideration of wind direction variations of 15° is understood in the present study as a recommendation to consider radials e.g. +/-6° (or +/-7.5°) from the point of interest. Nevertheless, USBR (2012) that bases on USACE (2008) recommends the approach in SPM (1984) of radials extended +/-12° from the point of interest in the upwind direction to the shoreline.

The extensive monitoring program carried out in the 1990's on four large and wide reservoirs of the La Grande Complex in northern Québec, Canada (Dupuis et al. (1996) and SEBJ (1997)), resulted in a modified version of the effective fetch calculations by Saville (1954) to use with the resulting prediction formulas. The calculation procedure by SEBJ (1997) considers radials extended +/-90° from the point of interest in the upwind direction to the shoreline. This method of calculating the fetch is associated with the wave prediction formulas provided in SEBJ (1997) (see Eqs. 3.51-3.53), and is referred to in NVE (2003), and also by the Canadian Dam Association Guidelines (CDA) (according to information in Damov and Warren (2012)). However, the methodology is neither mentioned in relevant guidelines in the USA, such as USACE (2008) and USBR (2012), nor in the UK guidelines ICE (2015).

Another method for fetches was developed by Donelan (1980) at the National Water Research Institute in Canada Centre for Inland Waters. In Donelan‘s method, the fetch length is not defined by the wind direction, but is based on the wave direction. Hence, the method considers wave generation on fetch lengths in off-wind directions with a reduced wind forcing. This method is indirectly referred to in the UK guidelines ICE (2015) with a reference to Herbert, Lovenbury, Allsop, and Reader (1995). The Donelan's method as presented by Herbert et al. (1995) (and (Owen and Steel, 1988)) will be discussed below.

The UK guidelines ICE (2015) discusses water surfaces and fetches. ICE (2015) defines the fetch as the maximum straight-line overwater distance for a particular wind direction, however noting that waves can be generated at up to 45° either side of the main wind direction. ICE (2015) comment on fetches on circular reservoirs, elongated reservoirs, reservoirs with complex shorelines and reservoirs that are curved or cranked, as well as "banana-shaped" reservoirs. For reservoirs that are relatively circular, ICE (2015) notes that realistic resolution for predicted wind direction should be obtained by fetch radial drawn at 30° increments. While for elongated reservoirs, or reservoirs with irregular shoreline, the spacing of the fetch radials may be narrowed. The approach recommended in ICE (2015) for 'banana'shaped reservoirs is discussed further in the following.

It is important to note that the UK, US and Canada guidelines referred to above recognize that the wind speed is not the same for all wind directions (or use a wave height reduction due to angular spread considering the angle between the fetch and the dam axis), while for example NVE (2003) requires the most unfavorable wind direction to be used either along with the overwater wind speed of 30 m/s or as evaluated by the Eurocode. In the Eurocode approach the wind direction should however be considered.

The effective fetch ($F_{e,90°}$) (Saville)

The SMB forecasting formulas are derived from ocean studies for predicting wave conditions in open waters, and thus corrections are required to account for restricted fetch lengths, such as on reservoirs or in-land lakes. The USACE initiated a study in 1948 with wind and wave measurements in three reservoirs, two of which were deep reservoirs. The measurements carried out in these provided extensive set of wave and wind data. During preliminary analysis of the data straight-line fetches were used, i.e. the greatest straight-line distance over water in the wind direction. However, where the width of the fetch area was small compared with its length, measured waves were lower than the ones predicted using straight-line fetch. Conversely, measured waves were higher than predicted when the wind acted over short wide fetches. Thus different methods of defining the fetch lengths were considered. Saville (1954) investigated different fetch calculation methods and gave details of the one resulting in the most accurate prediction with the SMB method compared to the measured data. The fetch calculated by this method (Saville, 1954)(Saville et al., 1962) is referred to as the effective fetch and considers radials extended +/-45° from the point of interest in the upwind direction to the boundary of the water body.

The procedure for determining the effective fetch by Saville is illustrated in Figure 3.18. It consists of drawing 15 radials, originating at the point of interest for wave prediction, at intervals of for example 6° (limited by an angle of 45° on either side of the wind direction θ) and extending these radials to the shoreline. (Saville et al., 1962).

The SPM (1977) explains that for inland waters, such as lakes and reservoirs, fetches are limited by the shoreline and are often long in comparison to width. In these cases, the fetch width may result in wave generation that is significantly lower than those that would develop over more open waters for the otherwise same generating conditions. For such cases, the SPM (1977) recommends to estimate the fetch F with the effective fetch $F_{e,90°}$ as per the above calculation method by Saville et al. (1962) which can be summarized with the following fetch function:

$$F_{e,90^\circ} = \frac{\sum_{\gamma=-42^\circ}^{42^\circ} R_\gamma \cos^2\gamma}{\sum_{i=-42^\circ}^{42^\circ} \cos\gamma} \qquad \textbf{(3.54)}$$

Where the effective fetch is calculated for a certain wind direction, θ, and thus with a central radial also in the direction θ, R_γ is the radial at angle γ from the direction θ.

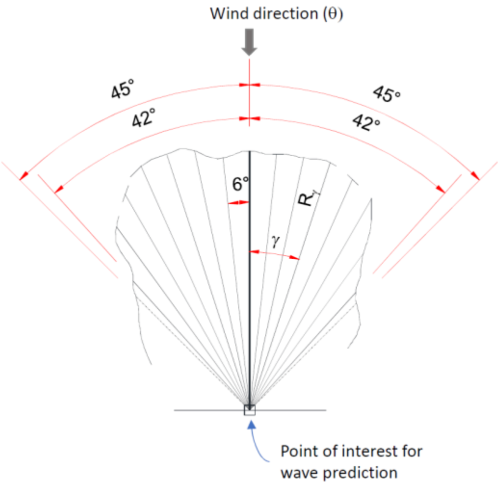


Figure 3.18 Illustration for calculating the effective fetch according to Saville et al. (1962).

The above method for calculating the effective fetch is based on the following assumptions taken from SPM (1977):

(a) "*Wind moving over water surface transfers energy to the water surface in the direction of the wind and in all directions within 45° on either side of the wind direction.*

(b) *The wind transfers a unit amount of energy to the water along the central radial in the direction of the wind and along any other radial an amount modified by the cosine of the angle between the radial and the wind direction.*

(c) *Waves are completely absorbed at shorelines*."

Saville et al. (1962) methodology for calculating the effective fetch, was for example referred to in the US Shore Protection Manual (SPM, 1973)(SPM, 1975)(SPM, 1977) and also recommended in Norwegian Regulations issued in 1981 (NVE, 1981).

The regulations NVE (1981) were complemented with guidelines on embankment dams prepared by NGI (Norges Geotekniske Institutt) in 1983 (NGI, 1983). The guidelines provide two potential methods by Saville for calculating the fetch, the one described above along with a method where a diagram is used to adjust the fetch basing on width to length ratio of the reservoir see Saville (1954). The diagram bases on an idealized rectangular fetch. A reference for the figure in NGI (1983) (NGI, 1983; Fig.42) is given as "Vassdrags og havnelaboratoriet", although it is most likely originally from Saville (1954). NGI (1983) explains that the effective fetch according to Eq. 3.54 will result in 10 to 30% lower values compared to the effective fetch obtained by using the diagram. Furthermore, that the difference is largest for low B/F values.

Procedure for calculating the fetch length, $F_{24°}$ in (SPM, 1984) and USBR (2012)

SPM (1984) recommended the use of straight-line fetches with the JONSWAP based prediction formulas. For irregular shorelines, SPM (1984) further recommended a procedure to determine the fetch length, that considers radials extended +/-12° from the point of interest in the upwind direction to the shoreline. The procedure consists of constructing nine radials from the point of interest at e.g. 3-degree intervals (other small angular spacing is also allowed) and extending these radials until they intersect the shoreline. The length of each radial is measured and arithmetically averaged as follows:

$$F_{24^\circ} = \frac{\sum_{i=1}^{9} Length\ of\ radials}{Number\ of\ radials\ (=9)} = \frac{\sum_{\gamma=-12^\circ}^{12^\circ} R_\gamma}{9} \qquad \textbf{(3.55)}$$

This procedure is used in USBR (2012) and illustrated with Figure 3.19.

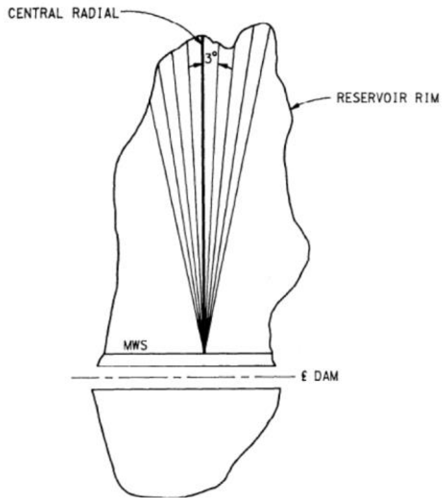


Figure 3.19 Illustration for calculating the fetch according to USBR (2012).(Figure from USBR (2012))

Straight-line fetch F_{SL} and $F_{12°}$ (USACE (2008))

USACE (2008), like SPM (1984), recommends to use straight line fetches with the simplified prediction formula provided in the respective guidelines. USACE (2008) further explains that the fetch should be defined so that wind direction variations do not exceed 15° degrees. The consideration of wind direction variations of +/-15° is understood in the present study as a recommendation to consider radials e.g. +/-6° or +/-7.5° from the point of interest.

Figure 3.20 presents a single fetch (F_{SL})., and a fetch defined for an angle of +/-6 from the point of interest ($F_{12°}$).

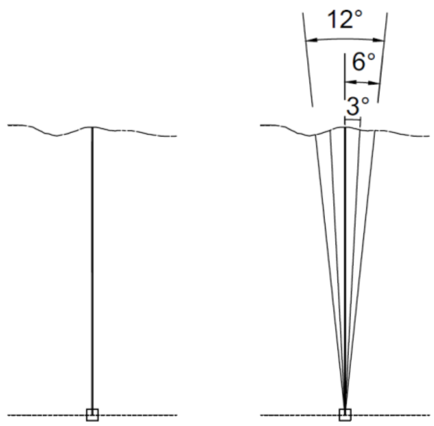


Figure 3.20 Illustration for calculating the fetch according to USACE (2008). (Straight line fetches F_{SL} *and* $F_{12°}$*)*

Effective fetch, $F_{e,180°}$, in SEBJ (1997)

Dupuis et al. (1996) and SEBJ (1997) presented a modified version of the effective fetch by Saville (1954) with a new fetch function to use with associated wave prediction formulas. The effective fetch. according to this modified fetch function, calculated for a certain wind direction, θ, and thus with a central radial also in the direction θ, can be written as follows:

$$F_{e,180°} = \frac{\sum_{\gamma=-90°}^{90°} R_\gamma \cos^2 \gamma}{\sum_{i=-90°}^{90°} \cos \gamma} \tag{3.56}$$

where $F_{e,180°}$ effective length (m) of the fetch for the direction θ ; R_γ is the radial length (m) in direction γ from the central radial, γ is the angle in degrees (°) formed by the radial R_γ and the central radial of the sector.

Figure 3.21 illustrates the calculation procedure for $F_{e,180°}$.

This calculation procedure is recommended in NVE (2003), however with the notation that special requirements may apply for certain cases such as when topography can affect the wind climate and thus the wave generation.

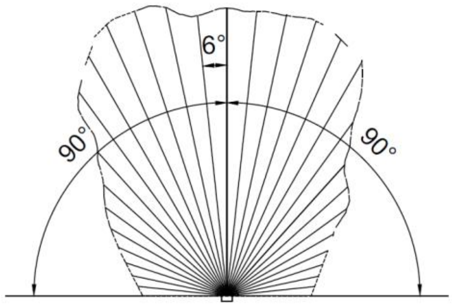


Figure 3.21 Illustration for calculating the fetch according to Dupuis et al. (1996) and SEBJ (1997)

Fetch length F_ϕ (DonJon)

Donelan (1980) argued that the fetch length should be measured along the predominant wave direction rather than the fetch along the wind direction (as in the other methods presented herein). Thus, the wind speed to be used in the wave prediction formulas should be the component along the wave direction.[4] See definition sketch on Figure 3.22.

Donelan (1980) defined the predominant wave direction as that which produces the largest peak wave period, T_p. Owen and Steel (1988), and later Herbert et al. (1995), use the approach of Donelan for determining the predominant fetch length, however, combine this with the JONSWAP Eqs. 3.39 - 3.40, i.e. those referred to in (ICE, 2015), the new formulas are provided with Eqs. 3.43 and 3.44.

If, as before, the fetch length along the predominant wave direction ϕ_p is denoted, F_ϕ , and Θ is the angle between the direction of the wind θ and the direction of the waves ϕ, then T_p from Eq (3.40), modified to consider the wave direction by using compenent of the wind speed along the wave prediction formula results in Eq (3.44) written below as follows:

$$T_p = 0.0712\left(U\cos\Theta\right)^{0.4} F_\phi^{0.3} = 0.0712U^{0.4}\cos^{0.4}\left(\theta-\phi\right)F_\phi^{0.3} \qquad (3.57)$$

From the above equation it can be observed that T_p is maximised when the product $\cos^{0.4}\left(\theta-\phi\right)F_\phi^{0.3}$ reaches its peak within the range $\left|\theta-\phi\right| \le 90°$. A trial and error method is subsequently used to determine the predominant wave direction. The steps taken in the methodology are described by Owen and Steel (1988) (and again by Herbert et al. (1995)) and outlined below.

---

[4] Owen (1987) provides the following example to demonstrate the relevance of Donelans approach: if winds are blowing along a short fetch, but there exists a relatively long fetch at a modest angle to the wind direction, then it seems reasonable to assume that the predominent wave direction will be biassed towards the longer fetch direction.

For a given wind direction, θ, the required steps are as follows:

i) Mark out fetch radials from the wave prediction point for all directions within +/-90°of the wind direction. Any angular spacing can be used, for example 6° (Herbert et al. (1995) give 5° as suitable in most reservoir applications). The angular steps do not need to be uniform, so for example smaller increments can be used when fetch lengths are changing rapidly with direction, and conversely, larger increments where the fetch length is almost constant.

ii) Measure the fetch lengths along each radial.

iii) For each fetch radial within +/-90°of the wind direction calculate the value of $\cos^{0.4}\left(\theta-\phi\right)F_{\phi}^{0.3}$. Identify the maximum v alue of this product, the direction of the fetch radial along with the fetch length at that direction. Fort the given wind direction, this fetch radial direction represents the predominant wave direction.

The calculation procedure is straightforward and easily performed for example in Excel. The calculations have to be performed only once for each reservoir.

Example calculations are provided for the Nesjøen Reservoir in a subsection of Chapter 4.3.6

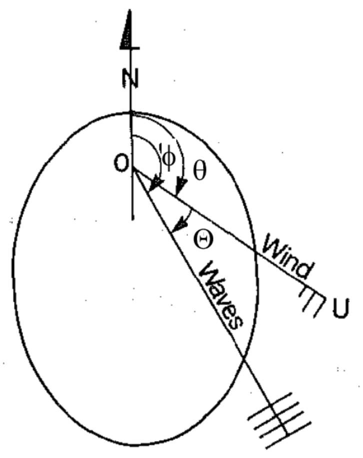


Figure 3.22 Definition sketch for calculating the fetch F_ϕ according to Donelan (1980) (here on an elleptical lake). (Figure modified from Bishop (1983)*)*

Fetch lengths in "banana" shaped reservoirs

ICE (2015) explains that in case of reservoirs shapes that are curved or cranked the wind may follow the valley/reservoir shape. For these cases the wave heights are underestimated when only straight-line fetch radials are used. Thus, for "banana-shaped" reservoirs (Figure 3.23) , ICE (2015) recommends to consider bent fetch radials that follow the axes of the reservoir into the upper reaches. This approach was introduced by Yarde et al. (1996) who stated that anecdotal evidence existed for that wave heights tend to be under-predicted for ‘banana’ shaped reservoir where the wind direction changes with the axis of the reservoir. However, Yarde et al. (1996) emphasize that this approach bases on their engineering judgement, but is not supported by scientific data.

ICE (2015) notes that single fetch distances may be used with simple wave prediction formulas, but at the same time explains that more complete methods, such as the one identified by Herbert et al. (1995) (presented also by (Owen and Steel, 1988)) with roots in Donelan’s methodology, may assess the contribution of a spread of fetches either side of the principal direction being considered. The fetch radials drawn in Figure 3.23 are not for calculation of the effective fetch but to use in e.g. Donelan’s method to find the fetch along the predominant wave direction. This may result in that a fetch length used into the prediction formulas becomes the bent reservoir length, which for very long and narrow reservoir may be conservative towards too high wave height prediction.

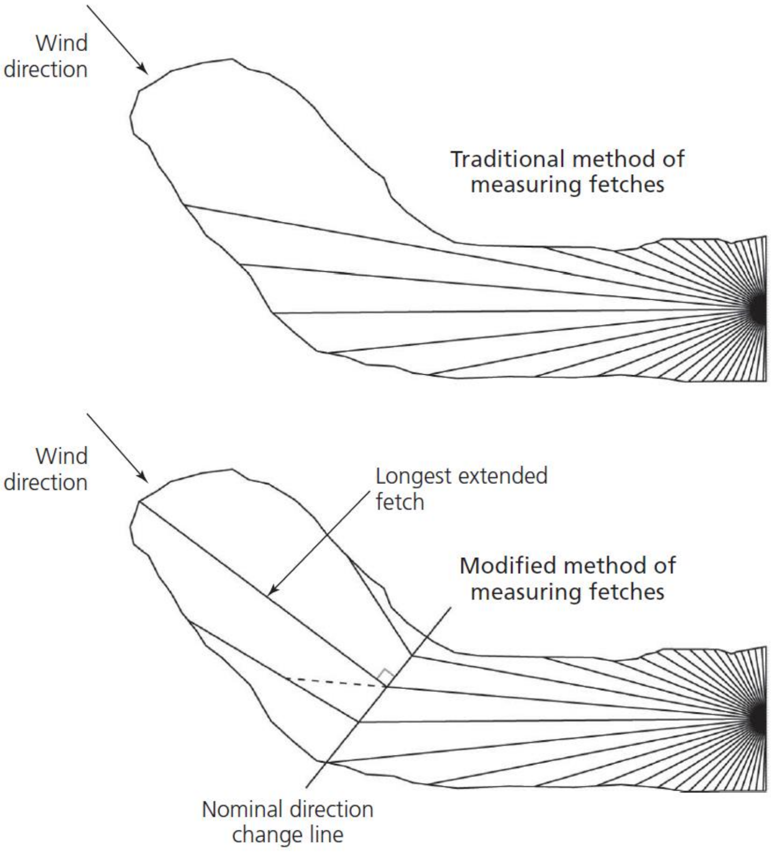


Figure 3.23 Bent or „banana“ shaped reservoir/fetches (Figure from ICE (2015)) (The fetch radials are here understood as to be used with Donelans method described above, to find the F_{ϕ})

Kirby and Dempster (2006) investigated the relevance of the simplified approach of a bent fetch shape for two long and narrow reservoirs in Scotland, the 27 km long Loch Shin and the 12 km long Loch Fannich (see Figure 3.24). They used wave analysis methods from the maritime sector to model wave generation on two reservoirs. Their results confirmed that narrow reservoir widths limit wave generation. Furthermore, that wave heigths calculated in the numerical model were slightly greater than those calculated assuming a straight line fetch length, confirming the anecdotal evidence that waves to a certain extent can be streered along the ‘banana shaped reservoir. Additionally, the wave heigths generated in the numerical model were significantly lower than those calculated assuming the total length of the reservoir for the fetch used.

Kirby and Dempster (2006) concluded that a significant engineering judgement is required to assess the influence of reservoir shape on wave generation in the determination of an appropriate fetch length to use in the relevant prediction formulas.

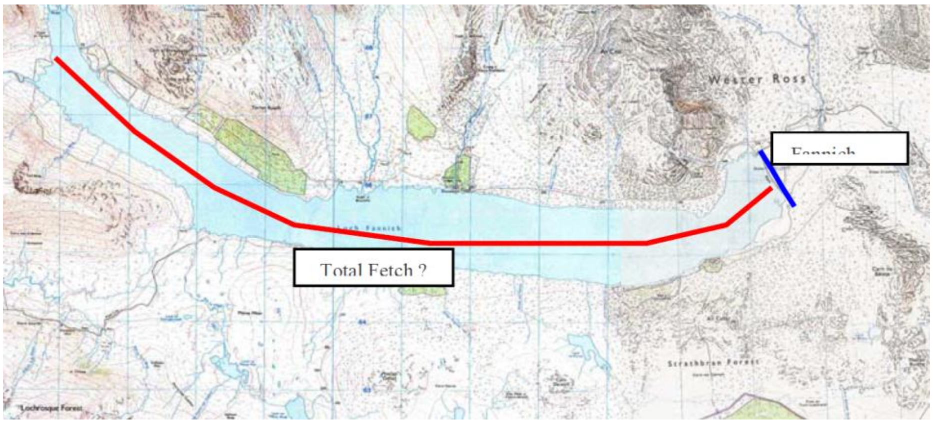


Figure 3.24 Loch Fannich. Fetch along the longitudinal axis of a „banana“ shaped reservoir? (Figure from Kirby and Dempster (2006))

3.4.3 Summary on fetches

Figure 3.25 provides overview on the different procedures to determine the fetch length. Again, it must be kept in mind that the different procedures belong to a certain formulation of the wave prediction. The method for “banana” shaped reservoirs is not shown on the Figure 3.25.

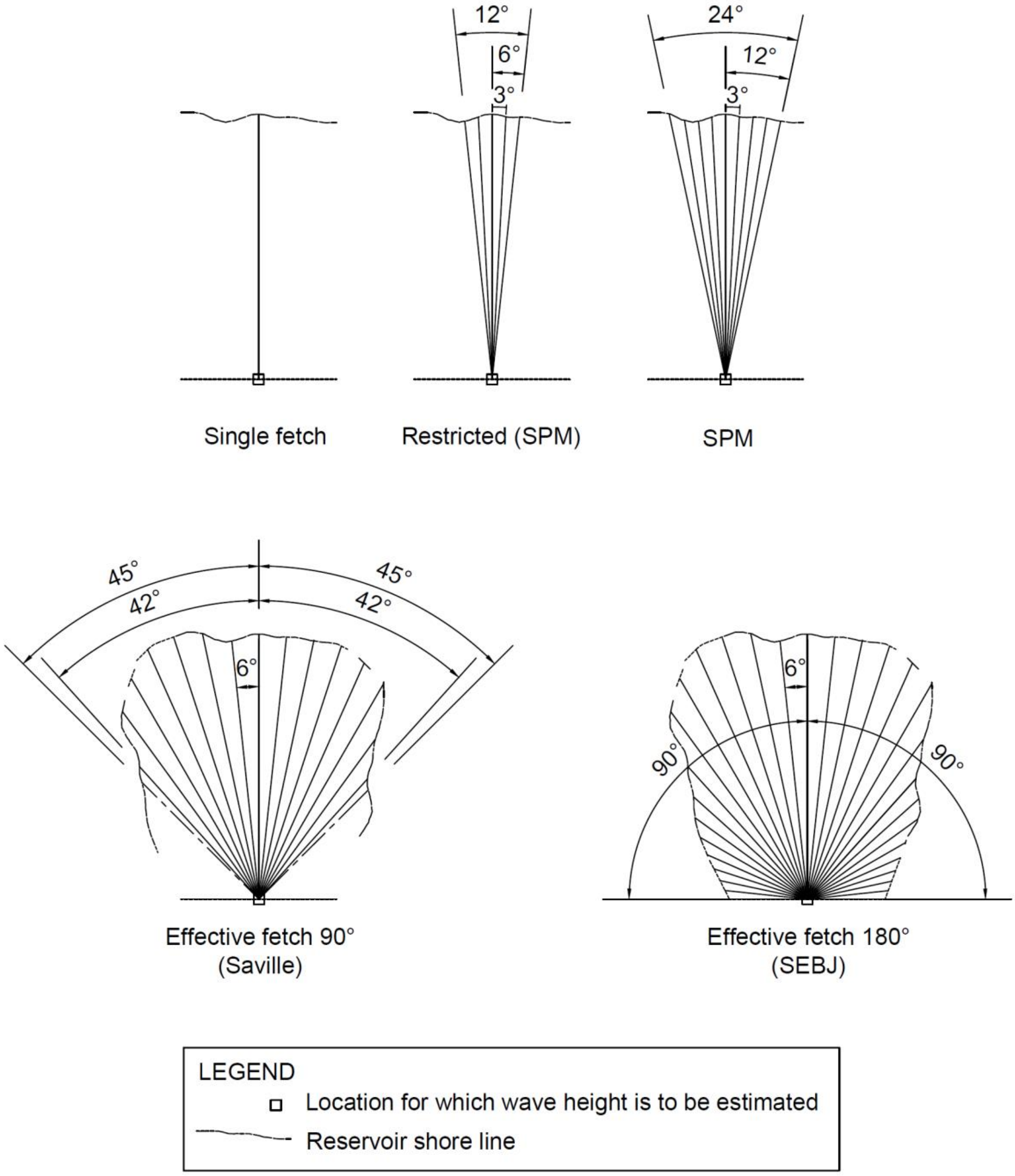


Figure 3.25 Summary of different procedures for estimating the fetch length. (Method for banana shaped reservoir not shown).

4 Guidelines on wave prediction

4.1 Introduction

Dam safety guidelines and/or engineering manuals influencing these, are frequently referred to in the previous chapter when discussing the background for predicting wind generated waves. Section 4.2 of this chapter summarizes the previous chapter with a focus on the development of the engineering manuals and dam safety guidelines, in the USA, UK, Canada and Norway. These guidelines are easily found and available on the internet, particularly those in USA, UK and Norway. In Section 4.3 the different methods are investigated through a conceptual reservoir and some cases, including two reservoirs in Norway. Finally, in Section 4.4 investigations into the different methods are summarized and discussed.

4.2 Development of guidelines in the USA, UK, Canada and Norway

In the USA a technical report Shore Protection Planning and Design, published in the year 1954 by the USACE BEB, revised in 1957, 1961 and 1966 (CERC, 1966). This technical report was superseded by the Shore Protection Manual of the U.S. Army Coastal Engineering Research Center in 1973. The technical reports and first three editions of the Shore Protection Manual (SPM, 1973)(SPM, 1975)(SPM, 1977) gave graphs and formulas using the SMB method and recommended an effective fetch computation for restricted fetches, such as on reservoirs and lakes.

The recommended effective fetch computation based on the work of Saville (1954) (see also (Saville et al., 1962)) and considers radials extended +/-45° from the point of interest in the upwind direction to the boundary of the water body. The joint utilization of the SBM and Saville's effective fetch computation, is referred to here as SBM-S. The SBM-S methodology as presented in the 1966 edition of the technical report (CERC, 1966) was for example recommended in the first official Norwegian guidelines associated with laws on dam safety (NVE, 1981). Further as an example, in the UK, the Institution of Civil Engineers guide to reservoir safety (ICE, 1978) quoted the earliest versions of the SMB graphs (see discussion in (Owen and Steel, 1988)).

In the period that the SBM-S methodology was included in the Shore Protection Manual, theoretical concepts relating to statistical representation of the sea by wave spectra concepts developed rapidly along with available data for practical application the methodology. In this context the contribution of Hasselmann et al. (1973) in the Joint North Sea Wave Project (JONSWAP) has been of importance, and e.g. influenced wave predictions for dam engineering purposes. The Jonswap spectra is derived from ocean wave measurement. Prediction formulas derived from the JONSWAP spectrum were included in the 1984 Shore Protection Manual (SPM ,1984) and UK guidelines (ICE, 1996). These prediction formulas were further modified and presented in a new form in the US Coastal Engineering Manual (USACE, 2008), but are however unchanged in the latest version of the UK guidelines (ICE, 2015).

In late 1990's a monitoring program on large lakes in Canada resulted in new prediction formulas derived from the site specific monitoring data and the guidelines SEBJ (1997). These formulas and a method of calculating the effective fetch were included in Norwegian dam safety guidelines NVE (2003). Furthermore, according to in the Canadian report by Damov and Warren(2012) the SEBJ (1997) guidelines are referenced in dam safety guidelines of the Canadian Dam Association (CDA) from 2007, however along with SPM (1984).

The different approach to determining the fetch to use in the calculations should be considered, the different guidelines recommend a fetch defined from a single fetch to a reduced value through e.g. an effective fetch. Furthermore, while in the UK and USA the fetch should be considered for different wind directions, in other countries, such as Norway the most unfavorable wind direction is to be used and a minimum value of the wind speed is specified.

4.2.1 Summary of wave forecasting and fetch calculation

Table 4.1 below provides a summary of the simple wave forecasting formulas discussed in the previous Chapter 3, along with the guidelines that refer to the respective formula.

The SMB-Saville methods are no longer in use, but were for example referred to in now obsolete regulations in Norway from 1981 (NVE ,1981), and provided as charts. A Technical Memorandum from USACE: "Waves in inland reservoirs" from 1962 is referenced to under the chart in NVE (1981). Furthermore, the technical report: "Shore protection, planning and design", from 1966 is given in the reference list of the NVE (1981) guidelines. Thus, older version of the SMB-Saville method is referred to although the SMB-Saville equations had been modified in the Shore Protection Manual from 1977 (SPM, 1977). This oldest version of the SMB-Saville method, i.e. from 1962 (or 1966) is not included in this report, but noted in Table 4.1.

The Jonswap formulation is still in use in the UK (see ICE, 2015), with and without the option to use Donelan's approach to the fetch length evaluation (the DonJon approach). Similarly, a modified form of the Jonswap prediction formulas is seemingly recommended in the USA (see USACE, 2008; USBR, 2012).

The Canadian Dam Association (CDA) on the other hand references in its guidelines from 2007 (CDA, 2007), to the original Jonswap formula as presented in SPM (1984), as well as the methodology presented in SEBJ (1997) (information in report by Damov and Warren (2012)).

Table 4.1 Summary of wave forecasting formulas and related fetch calculation

Method	Wind	Wave prediction formula	Fetch	Guidelines
SMB-Sav	NVE (1981): wind speed in m/s, at least 30 m/s)	Charts in NVE (1981). The Technical report:" Shore protection, planning and design", from 1966 is given in the reference list of the guidelines.	$F_{e,90}$	(Charts in NVE (1981))
SMB-Sav Modified	(SPM (1977): mean free wind speed and direction)	$\frac{gH_s}{U^2} = 0.283\tanh\left[0.0125\left(\frac{gF}{U^2}\right)^{0.42}\right]$ $\frac{gT}{U} = 2\pi \cdot 1.2 \cdot \tanh\left[0.077\left(\frac{gF}{U^2}\right)^{0.25}\right]$	$F_{e,90}$	(SPM, 1977)
Jonswap	$U_A = 0.71U^{1.23}$	$\frac{gH_{mo}}{U_A^2} = 0.0016\left(\frac{gF}{U_A^2}\right)^{1/2}$ $\frac{gT_m}{U_A} = 0.2857\left(\frac{gF}{U_A^2}\right)^{1/3}$	F_{SL} $F_{24°}$	SPM (1984) (Referenced in CDA (2007))
Jonswap-cem (SI units)	$U_* = C_D^{1/2}U_{10}$ 10 minute mean wind	$\frac{gH_{mo}}{U_*^2} = 4.13\times10^{-2}\left(\frac{gF}{U_*^2}\right)^{1/2}$ $\frac{gT_p}{U_*} = 0.651\left(\frac{gF}{U_*^2}\right)^{1/3}$	F_{SL} ($F_{12°}$)	USACE (2008)
Jonswap-cem (English units)	U_{MPH} wind velocity over water in miles per hour	$H_s = 0.0245\, F_{mi}^{1/2}U_{MPH}\left(1.1+0.0156U_{MPH}\right)^{1/2}$ $T = 0.464F_{mi}^{1/3}U_{MPH}^{1/3}\left(1.1+0.0156U_{MPH}\right)^{1/6}$ $t_{\min} = t_{F,U} = 1.87\frac{F_{\mathrm{mi}}^{0.67}}{U_{MPH}^{0.34}}$	$F_{24°}$	USBR (2012)
Jonswap-ice	Duration factor considered for the wind	$H_s = 0.00178U\left(\frac{F}{g}\right)^{1/2}$ $T_p = 0.0712F^{0.3}U^{0.4}$	Varies	ICE (2015)
DonJon (Jonswap/ Dunelan)	Hourly wind	$H_s = 0.00178U\cos\Theta\left(\frac{F_\phi}{g}\right)^{1/2}$ $T_p = 0.0712F_\phi^{0.3}\left(U\cos\Theta\right)^{0.4}$	F_ϕ	An option in ICE (2015) by referring to (Herbert et al., 1995)
SEBJ	Hourly wind in SEBJ (1997). 10 minute mean values in NVE (2003)	$H_{mo} = 0.001917\, F_{e,180°}^{0.45}\, U^{1.353}$ $T_{02} = 0.143\, F_{e,180°}^{0.225}\, U^{0.676}$ $t = 3.21\, F_{e,180°}^{0.775}\, U^{-0.676}$	$F_{e,180°}$.	SEBJ (1997) (CDA,2007) (NVE, 2003).

The current Norwegian dam safety regulations NVE (2003) only refer to the SEBJ (1997) calculation procedure but at the same time emphasize that special requirements may be demanded by the authorities for certain cases such as when the reservoir geometry and the surrounding topography can affect the wind climate and thus the wave generation.

4.3 Comparison of different methods

Most of the formulas in Table 4.1 are originally derived from data collected in open waters, but the effect of restricted fetches incorporated through different procedures for defining the fetch length used in the prediction. The equations presented by SEBJ (1997) are an exception, these were derived from measurements on large reservoirs of the La Grande Complex in northern Québec, Canada. The derivation of the SEBJ (1997) includes the use of the effective fetch $F_{e,180°}$ describe above. Of the guidelines discussed herein the SEBJ (1997) formulation is only included in the Norwegian regulations NVE (2003) and as an option in the Canadian CAD guidelines. The Canadian CAD guidelines additionally references SPM (1984). The SEBJ (1997) methodology is not mentioned in neither the UK guidelines ICE (2015) nor US guidelines USBR (2012) on dam safety.

Many reservoirs in Norway are relatively narrow and with a different shape from the wide reservoirs for which the SEBJ (1997) formulation is derived. Although NVE (2003) notes that special requirements may apply for certain reservoirs due to their shape or local climate conditions, these requirements are not specified further in the guidelines. However, it can be assumed that the requirements are further specified by the authorities for individual cases.

It is of interest to compare the different prediction formulas and associated fetch lengths for a simplified case and investigate the effect of the reservoir width on the wave height prediction. For this purpose, a simple idealized rectangular shaped reservoir is investigated, the length of this conceptual reservoir is L and the width B. This shape is investigated for ratios of B/L varying from 0.1 to 1.0.

The conceptual reservoir is introduced in Section 4.3.1 and investigated through different cases in Sections 4.3.2 to 4.3.5. The different recommendations in the guidelines studied, to estimate overwater wind from the overland wind, such as demonstrated in Figure 3.16 are not considered but assumed that the wind speed used in each case is the overwater wind. In other words, more direct comparison of the wave prediction equations is preferred over complicating this with different assessments of the overwater wind.

Furthermore, the potential reduction in the wind velocity when looking at the duration needed for the wind to develop the significant wave height along a fetch of certain length is not accounted for in sections 4.3.2 to 4.3.5. Conversely, this is investigated for the case of the Nesjøen Reservoir in Section 4.3.6.

Finally the influence of the velocity in the different formulas are discussed in Section 4.3.7

4.3.1 Conceptual rectangular reservoir

A definition sketch of the conceptual reservoir is provided in Figure 4.1 along with different procedures for defining the fetch length for a wind blowing along the centerline (CL) of the reservoir.

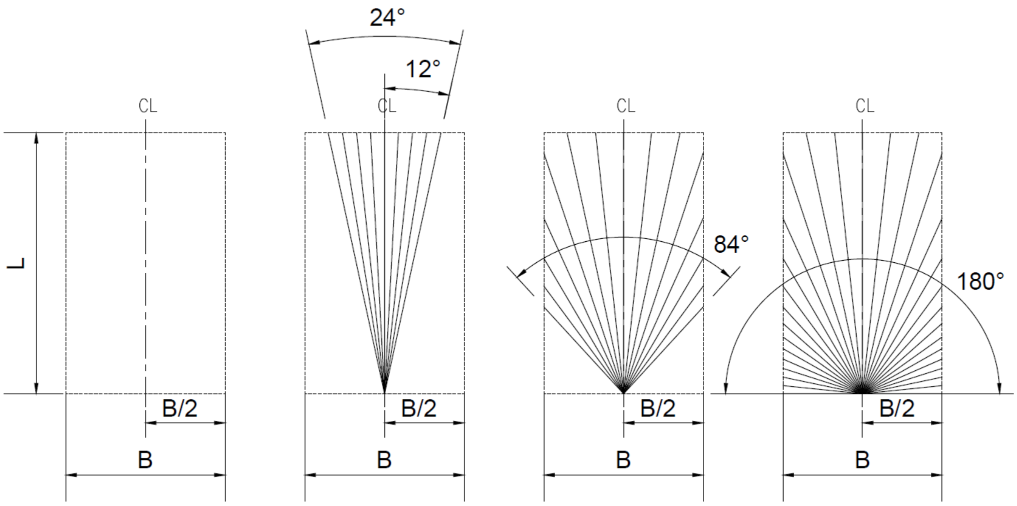


Figure 4.1 Definition sketch of a conceptual reservoir of a rectangular shape defined by a length L and width B, along with different procedures for defining the fetch length for a wind blowing along the centerline (CL) of the reservoir.

The fetch calculation procedures shown on Figure 4.1 are those used by USBR (2012) ($F_{24°}$), SPM (1977)(Saville's effective fetch) ($F_{90°}$), and SEBJ (1997)/NVE (2003) ($F_{180°}$). In addition to these fetches and associated prediction formulas, the prediction formulas in ICE(2015) are investigated for the conceptual model. For the wind blowing along the centerline of the reservoir the predominant wave direction is also along the centerline. Thus the fetch length used in with the ICE (2015) prediction formulas is the reservoir length *L*.

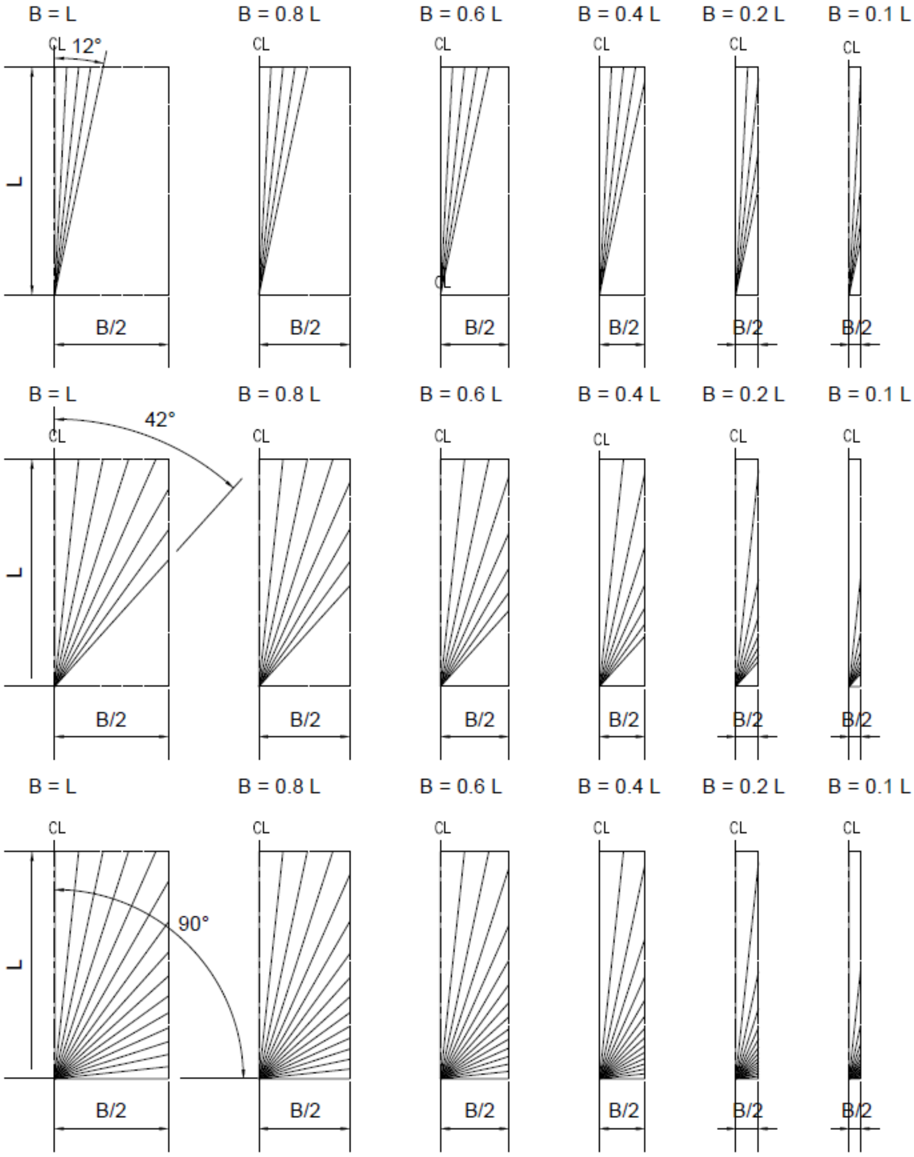


Figure 4.2 Overview of the different procdures for calculating the fetch length for a wind blowing along the centerline (CL) of the reservoir and the different shape rations B/L of the conceptual reservoir. The fetch length resulting from DonJon methodology is equal to L for the wind direction considered.

Fetches

The fetch lengths, calculated with the different procedures and presented as the ratio F/L (F is the fetch length resulting from each procedure, are shown in Figure 4.3 as a function of different shape ratios (B/L) of the conceptual rectangular reservoir (width B, length L). As the reservoir width decreases (decreasing B/L ratios) the difference in the procedures for calculating the fetches increases. For the wind blowing along the center line (CL) of the reservoir, obviously only the Donelan's approach (for DonJon) does not depend on the shape ratio for the wind direction considered. Furthermore, the shortest fetch lengths are obviously obtained with the procedures for the effective fetch calculation with 180° ($F_{180°}$)

resulting in the shortest fetches. Here it must be highlighted that the method of calculating the fetches is associated with a certain prediction formula. Furthermore, that although the effective fetch length is shorter than fetch lengths using other procedures, the associated prediction formulas for the wave heigth can result in higher wave.

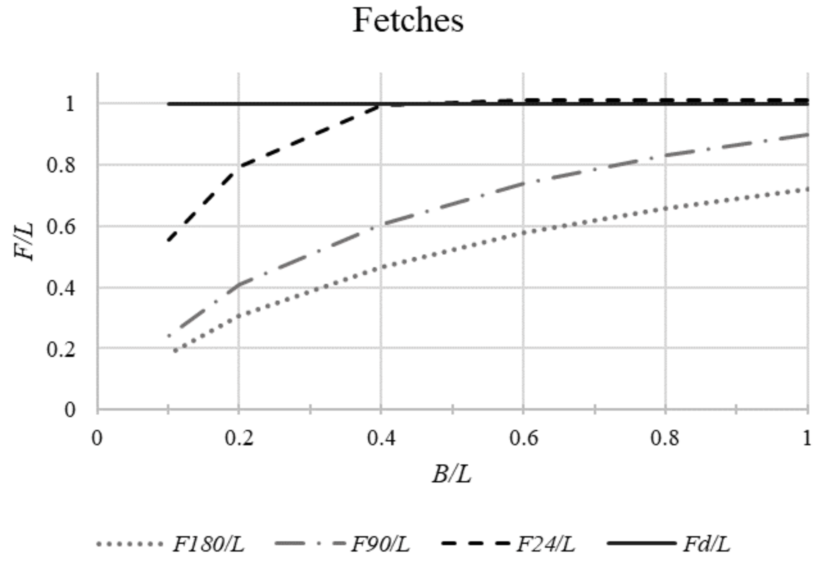


Figure 4.3 Dimensionless fetch lengths (F/L) versus the conceptual reservoir shape ratio (B/L) (Reservoir width against (B) against reservoir length (L)). Different procedures are used to calculate the fetch lengths. In the legend F180/L refers to $F_{180°}/L$ *(to use with* SEBJ (1997)*, F90/L refers to* $F_{90°}/L$ *(to use with* SPM (1977)*), F24/L refers to* $F_{24°}/L$ *(to use with* USBR (2012)*, Fd/L refers to* F_{ϕ}/L *(fetches in DonJon to use with* ICE (2015)*).*

Prediction of wave heigth for different wind speed and fetches

The significant wave height was predicted for the conceptual reservoir with the formulation given in ICE (2015). (Note that the DonJon Eq. 3.43 reduces to Eq 3.38 in ICE (2015) when $\Theta = 0°$ as for the conceptual case considered), SEBJ (1997) (and thus NVE (2003)), SPM (1977) with Saville‘s effective fetch, and USBR (2012) which uses the same prediction formulas as in USACE (2008). The prediction considers the shape ratio (*B/L*) ranging from 0.1 to 1, and wind velocities ranging from 2 m/s to 30 m/s, and length of the reservoir ranging from L=1000 to L=20 000 m.

In this subsection predictions of the wave height are extracted, for wind velocities of 15 m/s and 30 m/s, and two different lengths of the reservoir, *L*=1000 and *L*=10 000 m, and presented in Figure 4.4 to Figure 4.5. For improved comprehension of the different methods the same results are presented in Figure 4.6 and Figure 4.7, with a ratio of the wave heights ($H_s/H_{s,ICE}$) against the reservoir shape ratio (*B/L*). The ratio of the wave heights consists of the wave height each of the method used, i.e. the method referred to in the legend of the figure, against the wave height calculated with the ICE (2015) formulation. The wave height according to ICE (2015) is used as a reference value since this is constant over all reservoir shapes rations, due to the fact that the wind is blowing along the longitudinal axis of the reservoir.

Comparison of the figures below reveals that a wide range of prediction is obtained with the different formulas. Comparing the wave height ratios in Figure 4.6 and Figure 4.7, with the stable DonJon wave prediction as a reference wave height, the variation in the prediction ranges from 0.83 to 1.73 for the widest reservoir (*B/L*=1) and 0.64 to 1.08 for for the narrowest reservoir (*B/L* =0.1). For the narrowest reservoir (*B/L* =0.1) only the prediction formula SPM(1984) results in a slightly higher wave height than (at maximum 1.08 times) the DonJon approach. The same comparision considering the wave heigth in m (Figure 4.4 to Figure 4.5), wind velocity 30 m/s and reservoir length of 10 km the variation in the prediction ranges from 1 m to 1.54 for the widest reservoir (*B/L*=1) and 0.75 m to 1.04 for for the narrowest reservoir (*B/L* =0.1), all depending on the method used.

Generally, for higher shape ratios the SEBJ formulation gives the highest value of the wave height, while for the lowest shape ratios other formulas will give higher wave heights. It is worth while to consider the results for wind velocity of 30 m/s that is recommended in the Norwegian Regulations (NVE, 2003). For the shorter reservoir length, of 1000 m, the SEBJ formulation predicts the highest wave height for approximately B/L>0.35, while for B/L<0.35 the SPM(1984) predicts the highest value. For a reservoir length of 10 000 m, the SEBJ formulation predicts the highest wave height for B/L>0.6, while for B/L<0.6 the SPM(1984) predicts the highest value for all but B/L=0.1 for which the DonJon approach results in slightly higher value (or 1.04 times higher).

Which formula gives the best prediction?

It is impossible to conclude which formula gives the best prediction in the absence of measurements to compare with. Still, each formulation has, by various researcher, to some degree been tested against mesured data and deemed to provide the best prediction for the relevant reservoir in each case, these considerations are discussed in the following sections. Generally, it is reasonable to assume that the formulation used for a particular reservoir should be determined based on the actual shape of the reservoir as well as the wind conditions, including the strength of the wind.

In the following section predictions using the conceptual reservoir will be compared to measurements given in the litterature from reservoirs of different sizes and shapes.

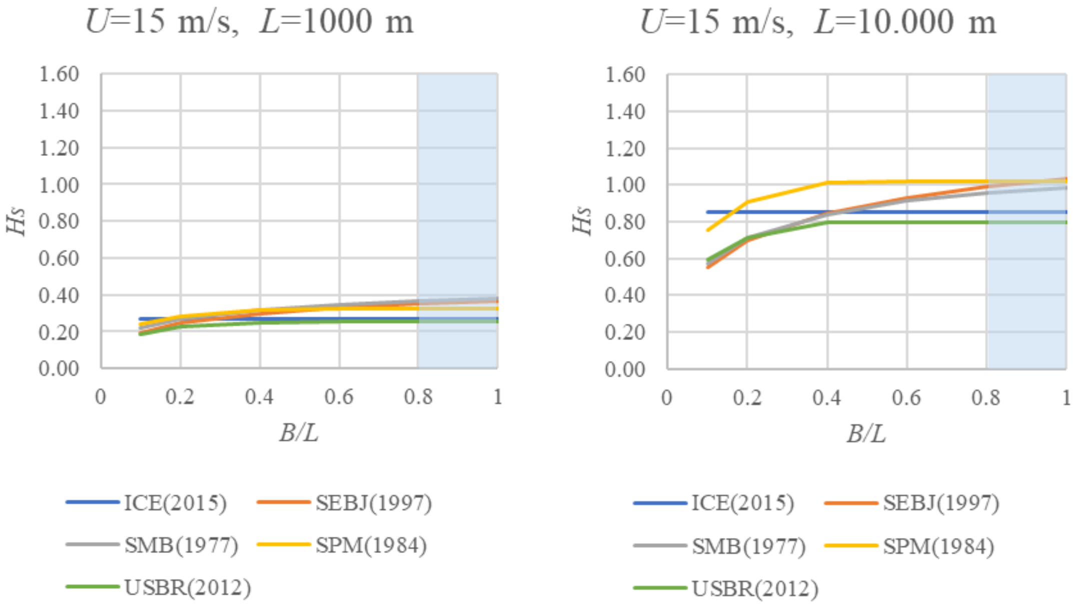


Figure 4.4 Significant wave height versus shape ratios of the conceptual reservoir using different formulas in standards and guideline. The predictions are for wind speed 15 m/s and reservoir lengths L=1000 and 10 000 m.

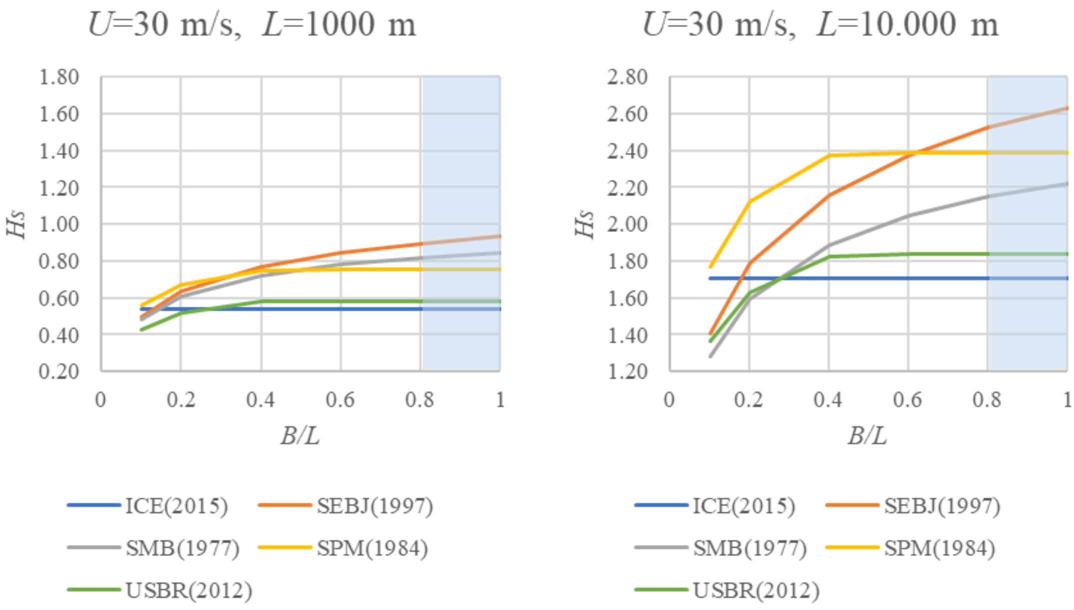


Figure 4.5 Significant wave height versus shape ratios of the conceptual reservoir using different formulas in standards and guidelines. The predictions are for wind speed 30 m/s and reservoir lengths L=1000 and 10 000 m.

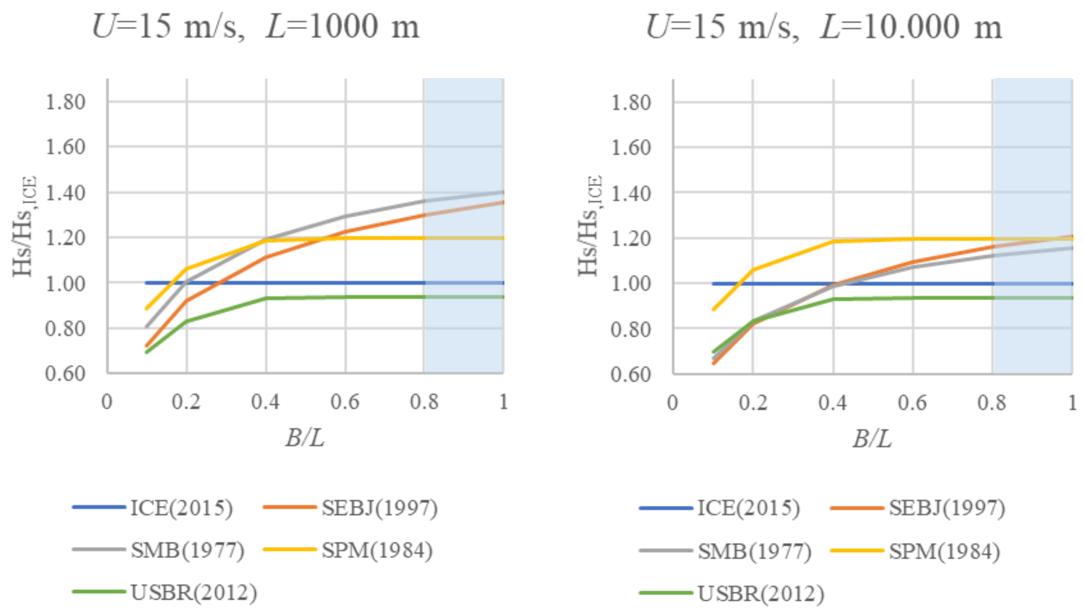


Figure 4.6 Ratio of wave heights ($H_s/H_{s,ICE}$) against the reservoir shape ratio (B/L). The ratio of the wave heights consists of the significant wave heights shown in Figure 4.4 against the wave height predicted with the ICE (2015) formulation. The predictions are for wind speed 15 m/s and reservoir lengths L=1000 and 10 000 m.

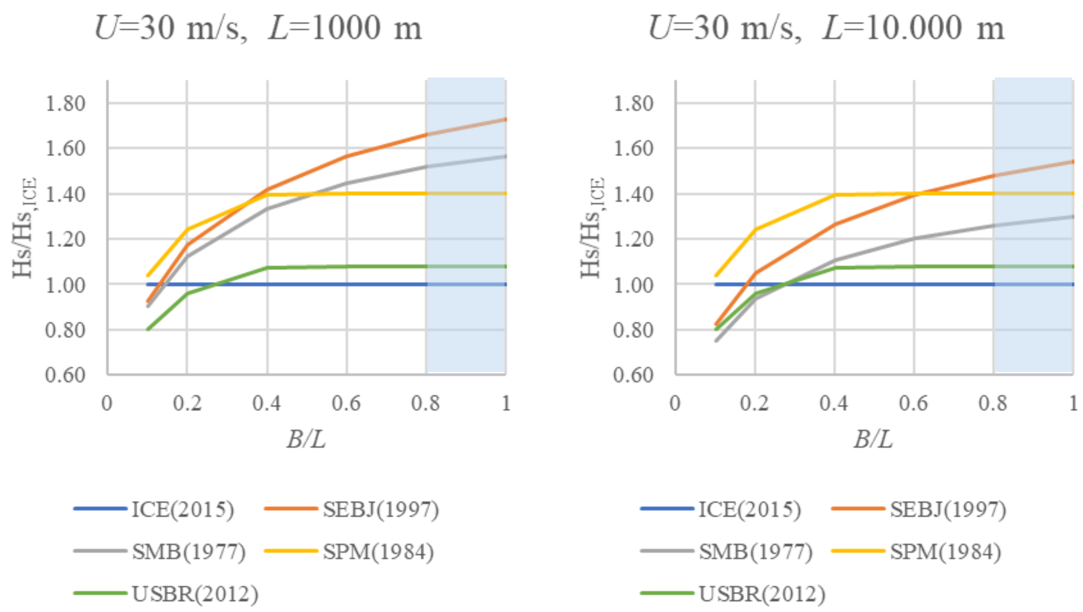


Figure 4.7 Ratio of wave heights ($H_s/H_{s,ICE}$) against the reservoir shape ratio (B/L). The ratio of the wave heights consists of the significant wave heights shown in Figure 4.5 against the wave height predicted with the ICE (2015) formulation. The predictions are for wind speed 30 m/s and reservoir lengths L=1000 and 10 000 m.

4.3.2 Comparison to data from Lake Ontario

The recommendation in SPM (1984), to use straight line fetches or F24 for irregular shorelines based on investigations carried out by Resio and Vincent (1979). Data from bay areas as well as from a large inland lake, Lake Ontario in Canada (see Figure 4.8), was compared to prediction of the significant wave height H_s made with both an effective and a straight-line fetch. Only the inland lake (reservoir) is of interest here, Lake Ontario is about 240 km long and approximately 65 km wide, resulting in approximate simplified shape ratio of B/L=0.3. Resio and Vincent (1979) reported that the observation point used on Lake Ontario has a straight-line fetch of about 240 km and an effective fetch (by Saville, i.e. F_{90}) of 100 km. Considering the conceptual reservoir of length L=240 km, shape ratio of B/L=0.2 will result in an effective fetch of about 98 km with Saville's method. The comparison revealed that straight-line fetch showed reasonable agreement with measured data when using the new JONSWAP based prediction formulas, whereas the effective fetch calculations resulted in wave heights that were too low.

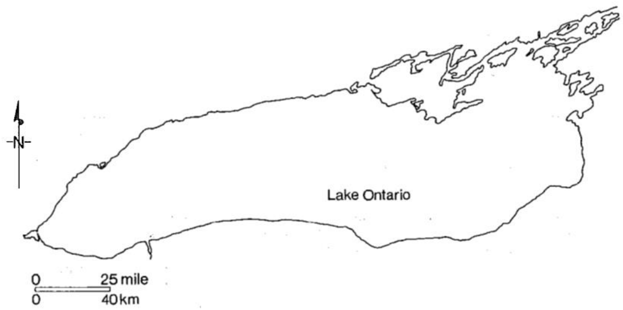


Figure 4.8 Lake Ontario, Canada. Figure modified from (Bishop, 1983).

The data measured at the above mentioned observation point on the Lake Ontario (given by Resio and Vincent (1979)) is shown in Figure 4.9 as points HA and HB along with prediction by the methods listed in Table 4.1. The prediction made by Resio and Vincent (1979) is not included in the figure since it is somewhat unclear which formula they used, although it is likely that they used formula as the one presented in Eq(3.27), i.e. not with the adjustment of the wind velocity as done in SPM (1984). Thus their prediction resulted in lower values than given for SPM(1984) in Figure 4.9.

The comparison of the predictions in Figure 4.9 and the measured data points indicates that SEBJ (1997) provides an acceptable prediction for Lake Ontario, furthermore that USBR (2012) SPM (1984) and ICE (2015) provide prediction that is on the safe side, USBR (2012) with a reasonable safety margin but substantial in the case of SPM (1984) and ICE (2015). SPM(1977) gives a good prediction for point HB but underestimates point HA.

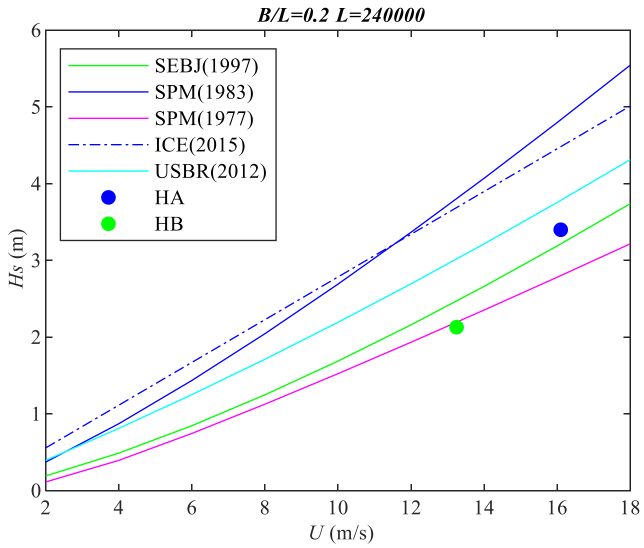


Figure 4.9 Prediction for the conceptual reservoir considering shape parameters of Lake Ontario, with B/L=0.2 and length L=240 km. HA *and* HB *are measured data point from Lake Ontario given in Resio and Vincent (1979) (Note: the wind speed given with the data points is adjusted to represent 10 m over land/water but not 19.5 m as used in Resio and Vincent (1979))*

4.3.3 Comparison of SPM(1984) and SEBJ(1997)

As previously mentioned, Dupuis et al. (1996), reported that the prediction formulas in SPM (1984) resulted in poor hindcast, with overprediction of small wave heights and under prediction of large wave height events, when compared to the measured values from the La Grande Complex reservoirs (large and wide reservoirs). Hence, Dupuis et al. (1996) analyzed the collected data and the resulting prediction formulas are those provided in SEBJ (1997). It is thus interesting to compare the predictions from SEBJ (1997) and SPM (1984) in light of this observation by Dupuis et al. (1996). However, in such comparison it must be noted that Dupuis et al. (1996) do not specify which wave conditions (reservoir shape, fetch length, wind velocity) are likely to lead to either overprediction or underprediction. Furthermore, that Dupuis et al. (1996) do not identify values of “small wave heights” and “large wave heights”.

The investigation considers the graphs in the figures previously presented and additionally a 3D mesh in Figure 4.10 of the predictions according to SEBJ (1997) and SPM (1984) over a range of wind velocities (from 2 to 20 m/s) and reservoir lengths (*L* from 1 to 20 km). The ranges selected approximately present the range of wind data and fetch lengths used in deriving the SEBJ (1997) prediction formulas (see Figure 3.17). The 3D representation provides the best overview for comparison of the two methods (SEBJ and SPM(1984)).

The observation by Dupuis et al. (1996) on that SPM(1984) did not predict the measured data sufficiently, and thus necessitating the SEBJ (1997) prediction formulas (which then should improve prediction of the measured data), can be restated as follows for the purpose of the investigation:

i. SPM (1984) will result in overprediction compared to SEBJ (1997) for “low” wave heights.

ii. SPM (1984) will result in underprediction compared to SEBJ (1997) for “high” wave heights.

For the conceptual reservoir considered:

- The statement i) above is true for predictions considering low values of the wind velocity ($U \leq 6$ m/s) and reservoir lengths of 1 to 20 km and all B/L ratios. For wind velocities in this range the prediction always results in values of the wave height below 0.5 m (that can be considered low wave height).

- The statement ii) above is always true for $B/L=1$ and wind velocities $18 \leq U \leq 20$ m/s and for reservoir lengths 1 to 20 km.

- The statement ii) above is generally not true for $B/L=0.8$, except for upper values of the velocities and lower values of the reservoir lengths.

- The statement ii) above is mostly not true for $B/L=0.6$, except for the highest values considered of the velocities and lowest values of the reservoir lengths ($L \leq 2$ km).

- The statement ii) above is never true for $0.1 \leq B/L \leq 0.4$.

The measured data from which the SEBJ (1997) prediction formulas are derived were collected on the wide and large La Grande Complex reservoirs. A shape ratio of $0.8 \leq B/L \leq 1$ (or even larger) can be assumed to represent the shapes of these reservoirs. Thus it is not surprising that the statement ii) from Dupuis et al. (1996) (rephrased here to consider comparison to the SEBJ (1997) prediction) does not necessarily apply for shape ratios that were not considered when deriving at the SEBJ (1997) prediction formulas.

Considering the reservoirs on which the SEBJ (1997) formulation bases in combination with the investigation into the statements i) and ii), it cannot be recommended to use the SEBJ (1997) prediction formulas for reservoir which shape can be represented with the ratio $B/L \leq 0.4$, unless possibly if the reservoir is very large as in the case of Lake Ontario (see Figure 4.9). Furthermore, considering the range of wind speeds on which the SEBJ (1997) formulation seem to base on, prediction with the formulas using wind speeds larger than 20 m/s must be carried out with caution and due consideration of that the prediction is most likely carried out outside the range on which the prediction formulas bases.

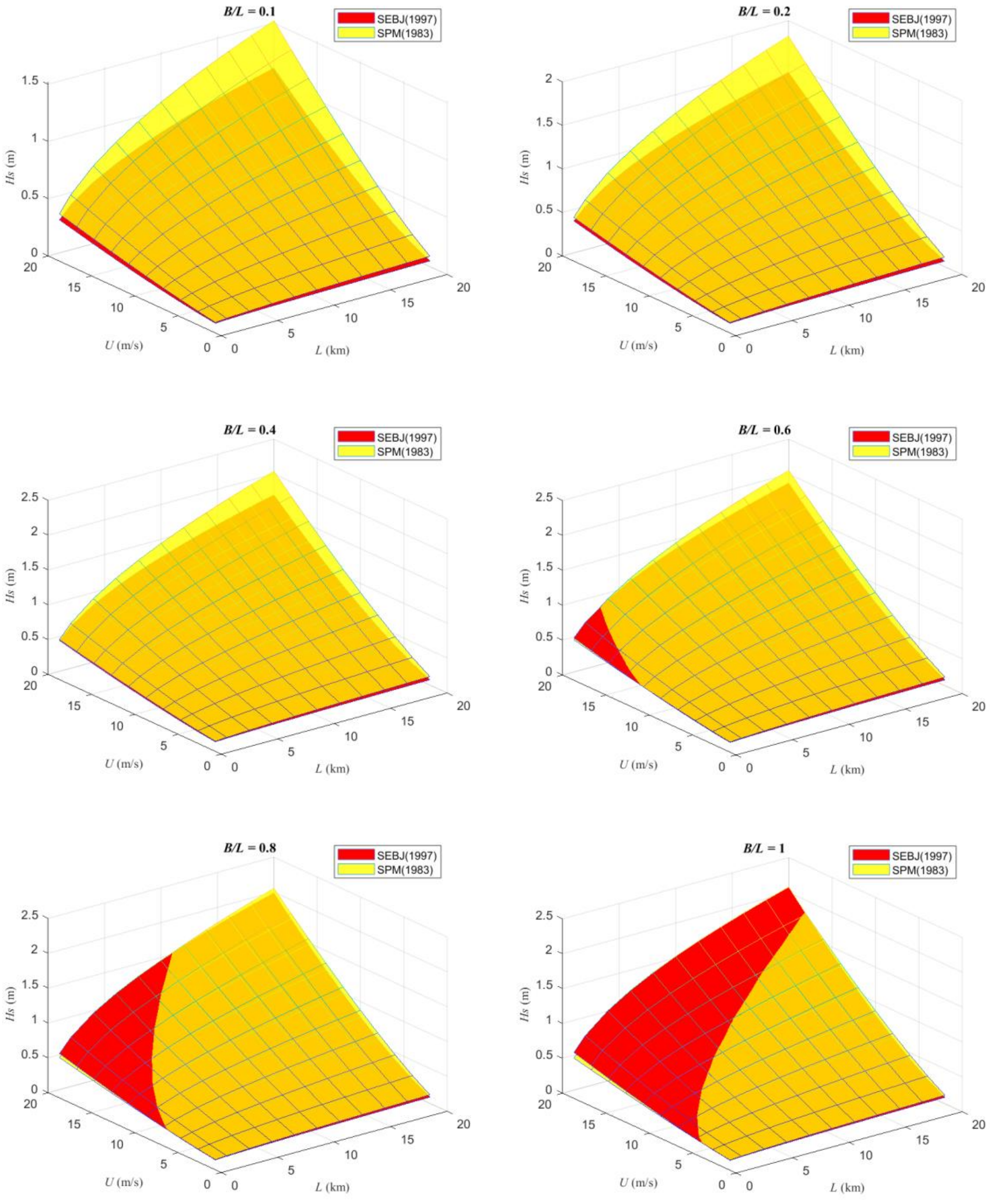


Figure 4.10 Plots in 3D of wave heights predicted with SPM (1984) and SEBJ (1997) formulation for reservoir shape ratios (B/L): 0.1, 0.2, 0.4, 0.6, 0.8 and 1.0. The predictions are for wind speed ranging from 2 m/s to 20 m/s and reservoir lengths from 1 to 20km. (Note: The orange colour on the surfaces comes from making the yellow surface partly transparent, thus orange colour means that the SEBJ prediction is under (and thus lower than) the SPM prediction).

4.3.4 Comparison to measured data in the UK

Owen and Steel (1988) compared different prediction methods and measured data on two reservoirs in UK, the Megget Reservoir and Loch Glascarnoch (see Figure 4.11). The Megget Reservoir is about 3.5 km long and 0.6 km wide, and can be represented with a shape ratio B/L=0.17 (or 0.2). Loch Glascarnoch is about 7 km long and about 0.74 km wide, and can be represented with a shape ratio B/L=0.1.

Owen and Steel (1988) compared prediction with six different methods over a range of wind velocities as well as different wind direction and associated fetch lengths. The comparison included methods relating to ICE (2015) and SPM (1977), in addition to some other methods that have not been discussed here. Owen and Steel (1988) concluded from their comprehensive study that none of the wave prediction methods that they examined gave particularly good agreement with the measured wave height.

- In the Megget Reservoir the original SMB/Saville method gave the best agreement, while this method seriously underestimated wave heights in Loch Glascarnoch.
- In Loch Glascarnoch the method referred to in this report as DonJon, the method recommended in ICE (2015), gave the best agreement according to Owen and Steel (1988), but they also reported that this method significantly overestimated the wave heights in the Megget Reservoir.

The shapes of the two reservoirs Megget and Loch Glascarnoch (Figure 4.11) can easily be represented by the conceptual rectangular reservoir, repectively with shape ratios about 0.2 and 0.1. The measurements corresponding to the wind blowing approximately along the longitudinal axis of the reservoirs can be used to compare with the different prediction methods considered in the present study (see methods in Table 4.1). (Note that here only one wind direction is considered while Owen and Steel (1988) compared measured values and prediction for a wide range of wind directions and wind velocities.)

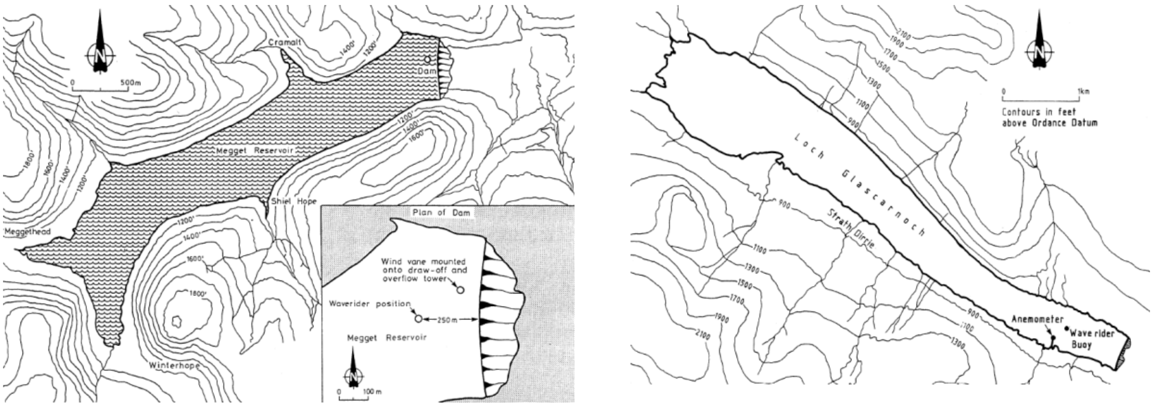


Figure 4.11 The Megget Reservoir (to the left) and the Loch Glascarnoch (to the right).

Table 4.2 summarizes measured wave heights and corresponding wind velocity for winds blowing approximately along the longitudianal axis of the reservoirs. The measured wave heigth are read from graphs and tables in Owen and Steel (1988). The value referred to in Table 4.2 as “base” is read from the graphs, while an average value of the wave height measured for the given wind speed (approximately) and wind direction (approximately along the longitudinal axis) is read from tables. Owen and Steel (1988) do not explain clearly the line denoted “base” on their graphs, while average value can be read from tables and is derived directly from the measurements. Rough investigation into the tables and graphs, indicate that the “base” from the graphs somewhat (but not fully) compare to the lowest values of the wave heights (base) from the tables. Thus, here the average value is used in comparison to the different prediction methods with the use of the conceptual rectangular reservoir although the base value is also given since this is on the comparison graphs produced by Owen and Steel (1988).

Table 4.2 Measured data on the Megget Reservoir and Loch Glascarnoch.

Reservoir	*L* km	*B* km	*B/L*	Measured data for wind blowing approximately along the longitudinal axis of the reservoir. (ca 240° to 245° for the Megget and 296-304°for Glascarnoch)		
				Wind velocity (m/s)	Wave height (m) „Base“ read from graphs in Owen and Steel (1988)	Wave height (m) Average value read from tables in Owen and Steel (1988)
Megget	3.5	0.6	0.17	14	0.35	0.46
				22	0.62	0.81
Glascarnoch	7	0.74	0.1	8	0.25	0.35
				14	0.5	0.63

Figure 4.12 and Figure 4.13 compare predictions (see methods in Table 4.1) and the measured wave height at the Megget Reservoir and Loch Glascarnoch, respectively. The average value of the measured wave height (see Table 4.2) is denoted with an x in the figures and the base value with an o.

Comparison of the predictions in Figure 4.12 and Figure 4.13 to the average value indicates that SPM (1977), SEBJ (1997), and USBR (2012) may underestimate the wave height on narrow reservoirs (B/L<0.2). Furthermore, that SPM (1984) may underestimate the wave height on the narrowest reservoirs (B/L=0.1) but may give a quite good prediction for shorter and narrow (but less narrow) reservoirs (0.1<B/L≤0.2). Additionally, the comparison to the average value and the prediction using the method of ICE (2015) indicates that this may give results with a mild safety margin for very narrow reservoirs (B/L=0.1), and same for a narrow but less narrow reservoirs (0.1<B/L≤0.2) however depending on wind speed. ICE (2015) underestimates the average value for wind speed of 22 m/s.

A completely different conclusions are drawn from the figures Figure 4.12 and Figure 4.13 when comparing with the base value read from graphs in Owen and Steel (1988) for wind blowing approximately along the longitudinal axis of the reservoirs. Comparing the base value and results for the Megget Reservoir in Figure 4.12 gives the same conclusions Owen and Steel (1988), i.e. that SPM (1977) gives reasonable prediction of the base value, while this is overestimated by ICE (2015). Furthermore, SPM (1977) underestimates the base value in the case of Loch Glascarnoch but not seriously. Conversely the base value is overestimated by ICE (2015) for this case. Thus, for comparison with the base value ICE

(2015) provides prediction on the safe side for both reservoirs. The other methods, predict the base value reasonably for the Megget Reservoir, the USBR(2012) in particular.

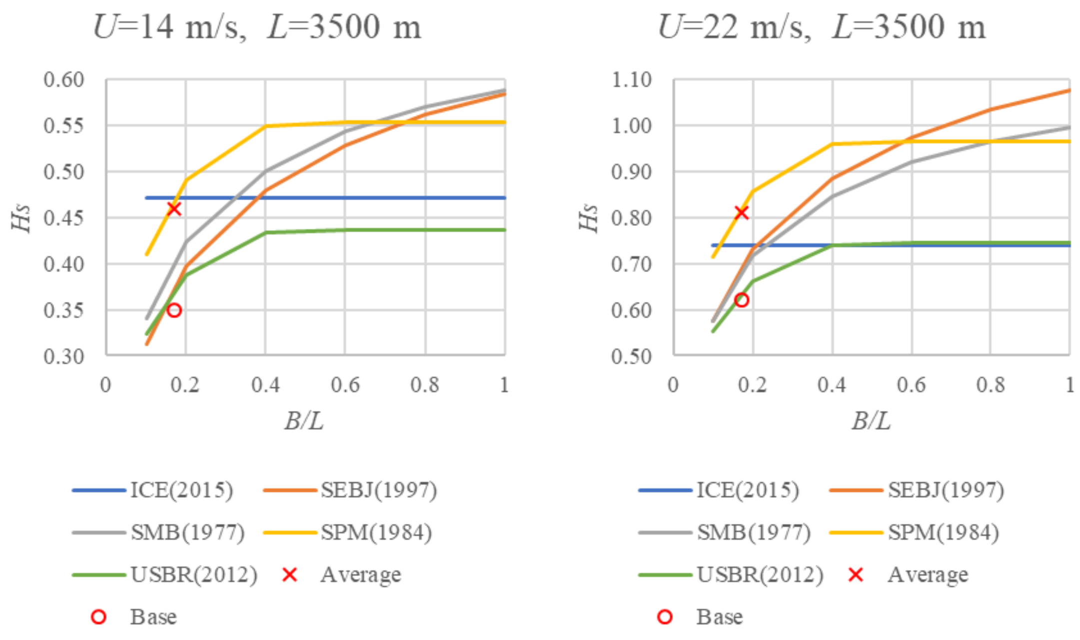


Figure 4.12 The conceptual reservoir. Comparison of different prediction formulation to wave measurements (average and base values) on the Megget Reservoir B/L~0.2.

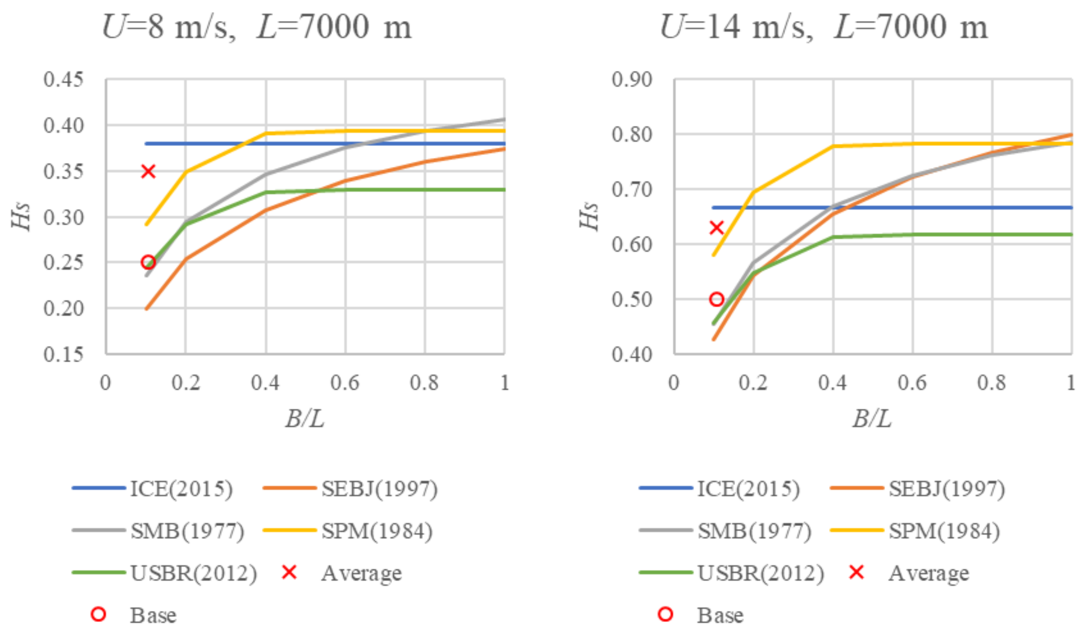


Figure 4.13 The conceptual reservoir. Comparison of different prediction formulation to wave measurements (points Gla) (average and base values) on the Loch Glascarnoch B/L~0.1.

USBR(2012) also predicts the base value for the Loch Glascarnoch reasonably for the low wind velocity but underestimates this for the larger velocity. SEBJ(1997) underestimates the base value in the case of Loch Glascarnoch.

The reason for that Owen and Steel (1988) compare to a base value rather than the average wave height is unclear. In absent of explanation to this, the author of this report finds more appropriate to compare to the average value clearly presented in tables of the report by Owen and Steel (1988). Hence the conclusion of this comparison is that ICE(2015) overall provides a reasonable prediction for these two narrow reservoirs, i.e. with B/L<0.2, indicating that this might be the most appropriate prediction formula for such cases of the methods studies.

4.3.5 Rough comparison for a reservoir in Norway

No wave measurements have been found to exist from reservoirs in Norway, at least not to the knowledge of author of this report. However, an event of run-up and minor overtopping (splashing) of waves noted at the reservoir Šuoikkátjávri in Kvænangen Nord-Troms (Figure 4.14, Figure 4.15 and Figure 4.16) may give a single rough measuring point to exercise with. The information is from Vebjørn Pedersen, NVE, with reservoir related data confirmed by the dam owner Troms Kraft Produksjon AS, via Øyvind Steffenach. **The following can only be taken as an exercise for a very rough comparison of the methods**.

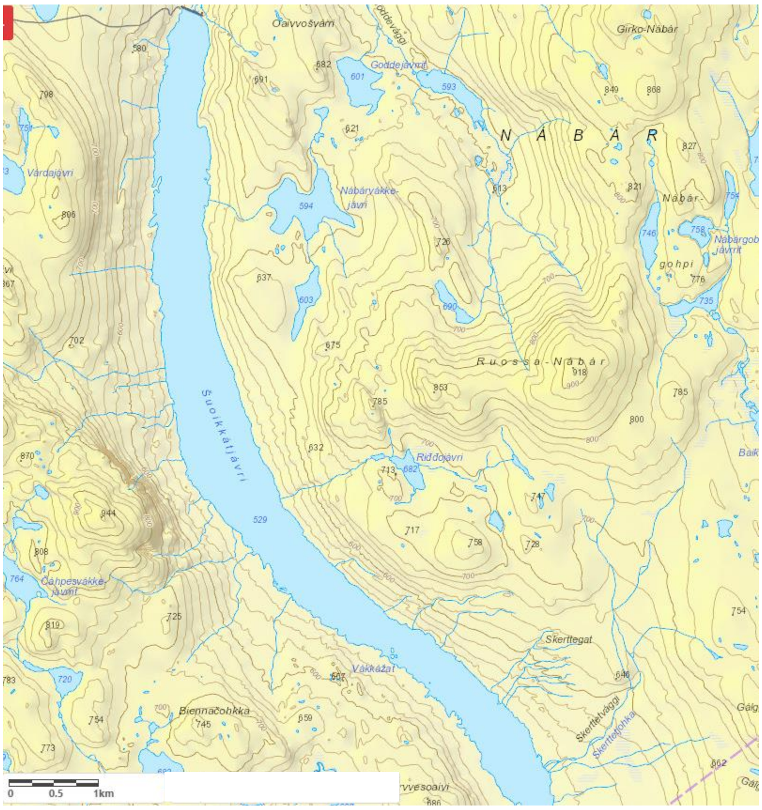


Figure 4.14 The Šuoikkátjávri reservoir. (Map from atlas.nve.no)

Figure 4.15 The Šuoikktjávri reservoir 6. september 2017. (Vebjørn Pedersen, NVE)

Figure 4.16 The Šuoikkátjávri reservoir 6. september 2017. Figures from Vebjørn Pedersen, NVE.

The Šuoikkátjávri reservoir is about 10.8 km long, measured along the full length of the reservoir, and the width is typically from about 0.5 km to about 0.7 km. The reservoir is very narrow (and with a „banana“ like shape) and the B/L ratio is less than 0.1. The dam retaining the reservoir is a concrete faced rockfill dam, i.e. the dam has a concrete slab on the upstream side, and the upstream slope‘s inclination is 1:1. (See Figure 4.16).

The elevation at the dam crest is 532 m a.s.l. and the full supply level (FSL) („høyeste regulerte vannstand, HRV“) is 529 m a.s.l. The water elevation (still water level) at the time of the event considered, the 6th of September 2017, was 527 m a.s.l., i.e. about 2 m under the FSL and 5 m below the dam crest. The waves that overtopped or nearly overtopped the dam crest (see Figure 4.16) had to run-up these 5 m (vertical distance). The wind velocity during this event was recorded by an instrument within a helicopter at the site. The wind velocity recorded was 20 m/s and the wind was observed to be blowing along the length of the reservoir (i.e. funnelling effects).

To roughly extimate the potential wave height associated with the observed runup the following equation can be used (NVE, 2003) to backcalculate the required wave heigth for full runup on the slope:

$$R_u = m\frac{2.4H}{n^{0.44}} \quad \text{[m]} \tag{4.1}$$

where R_u is the runup (vertical), n is the horizontal part of the upstream slope of the dam and should be less than 2.7, m is a correction factor accounting for the material in the upstream slope and H is the significant wave height for which the runup is to be estimated.

Backcalculation for the wave height, with R_u =5, n=1 and m=1.8 to 2 for a concrete face slab results in a value of 1.04 to 1.16 m.

- Hence, a wave height ranging from 1 to 1.2 m is considered here as an approximate boundary within which the prediction of the significant wave height should lie.

The significant wave height can be predicted with the different prediction formulas in Table 4.1, for a wind speed of 20 m/s (as measured in the helicopter) and fetch lengths associated with each methodology.

Fetches associated with different prediction formulas

The fetches were calculated for the Šuoikkátjávri reservoir using the different procedures reviewed in Section 3.4. Radials used to calculate $F_{180°}$, $F_{90°}$, $F_{24°}$ and F_{SF}, are shown in Figure 4.17. F_{SF} is for this case a bent fetch accounting for the potential funnelling of the wind as is blows along the reservoir.

The procedure for „banana“ shaped reservoirs explained in Figure 3.23 from (ICE, 2015) is to be used when considering different wind and wave directions. **Thus, it is not understood here that the radials in Figure 3.23 are to be used to calculate some sort of an effective fetch.** However, out of curiousity the different procedures for calculating $F_{180°}$, $F_{90°}$, and $F_{24°}$ where used with extended radials as shown in Figure 4.18

The calculated fetch lengths are summarized in Table 4.3

Table 4.3 Šuoikkátjávri reservoir. Calculated fetch lengths according to recommended procedures as well as for extended radials. (Calculations: Eirik Øvregård).

Method	Angle	Fetch length According to recommended procedures (see *Figure 4.17*)	Fetch length Extending (bending) lines along reservoir (see *Figure 4.18*)
SEBJ(1997)	+/-90° (180°)	$F_{180°}=0.93$ *km*	$F_{180°Ext}=1.74$ *km*
SPM(1977) (Saville)	+/-45° (90°)	$F_{90°}=1.24$ *km*	$F_{90°Ext}=1.3$ *km*
SPM(1983) and USBR(2012)	+/-12° (24°)	$F_{24°}=2.49$ *km*	$F_{24°Ext}=3.41$ *km*
ICE(2015)	Single fetch	$F_{SF}=10.38$ *km*	*(„banana“ shape)*

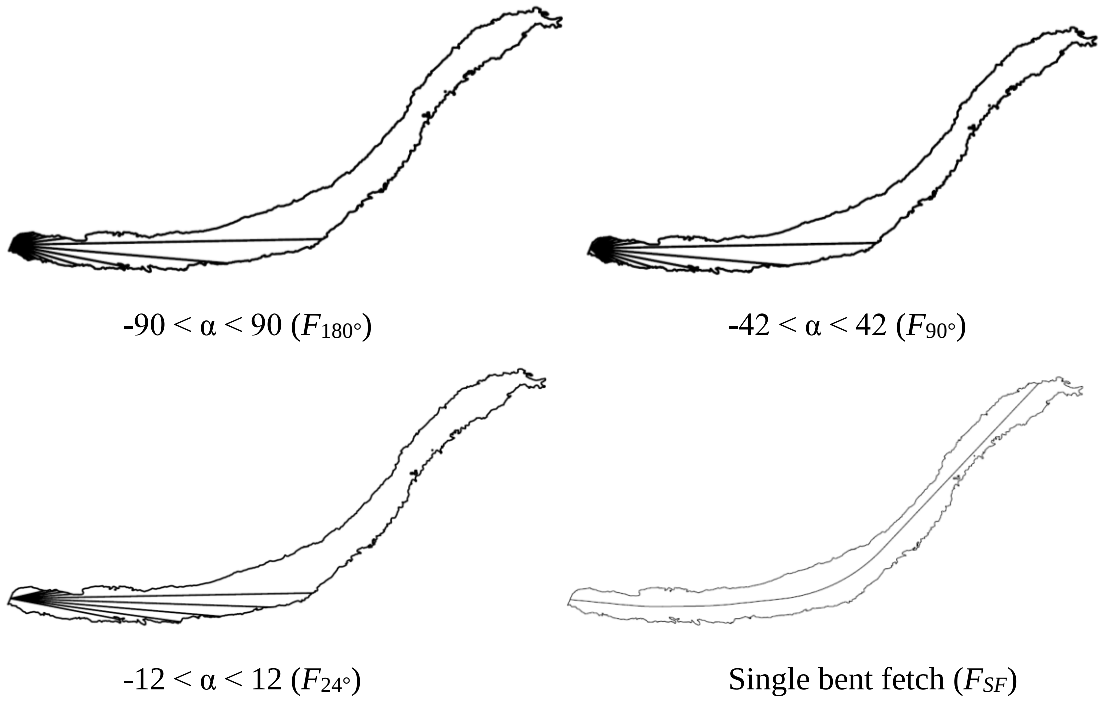
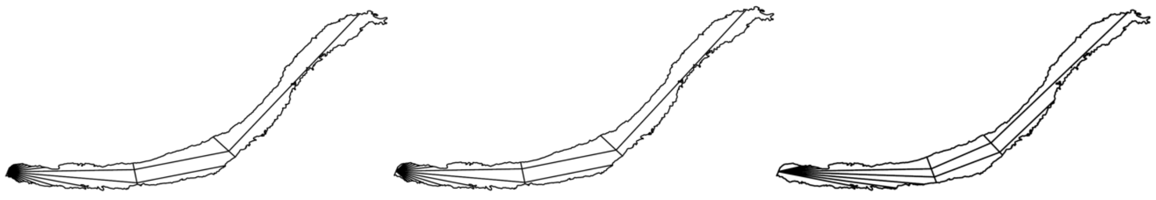


Figure 4.17 Outlines of the Šuoikkátjávri reservoir and radials used to calculate the fetch lengths ($F_{180°}$,$F_{90°}$, $F_{24°}$ and F_{SF}) to use in the different prediction formula. The fetch radials were drawn by Eirik Øvregård. (Outline of reservoir obtained from NVE map services).



$F_{180°\mathrm{Ext}}$ $-90 < \alpha < 90$ | $F_{90°\mathrm{Ext}}$ $-42 < \alpha < 42$ | $F_{24°\mathrm{Ext}}$ $-12 < \alpha < 12$

Figure 4.18 Exercise: Outlines of the Šuoikkátjávri reservoir and extended radials used to calculate the extended fetch lengths ($F_{180°\mathrm{Ext}}$, $F_{90°\mathrm{Ext}}$, $F_{24°\mathrm{Ext}}$) that further are used in the different prediction formula. The extended fetch radials were drawn by Eirik Øvregård. (Outline of reservoir obtained from NVE map services).

Predicted wave heights for the observed 2017 event

The significant wave height was predicted with the different prediction formulas in Table 4.1 and using the fetch lengths calculated above for the Šuoikkátjávri Reservoir. The results are summarized in Table 4.4, for both the fetch lengths associated with the methodology (see Figure 4.17) and the exercise with the extended fetch lenghts (see

Figure *4.18*). The information in Table 4.4 is further illustrated in Figure 4.19. The dotted lines, denoted ObsBoundary* refer to the wave height calculateded above from the runup formula for two values of the correction factor m (m=1.8 and 2).

Table 4.4 Predicted wave heights at Šuoikkátjávri for U=20 m/s and different methods.

Method	Fetch lengths according to the methodology used		Exercise of using extended radials when calculating the fetches	
	Fetch length (km)	Calculated wave height in m	Fetch length extended (km)	Calculated wave height in m
SEBJ(1997)	$F_{180°}=0.93$	0.60	$F_{180°Ext}=1.74$	0.80
SPM(1977)	$F_{90°}=1.24$	0.60	$F_{90°Ext}=1.3$	0.62
SPM(1983)	$F_{24°}=2.49$	0.72	$F_{24°Ext}=3.41$	0.84
USBR(2012)	$F_{24°}=2.49$	0.56	$F_{24°Ext}=3.41$	0.65
ICE(2015)	$F_{SF}=10.38$	1.16	$F_{SF}=10.38$	1.16
Rough estimate of wave height from observed runup		1-1.2 m		1-1.2 m

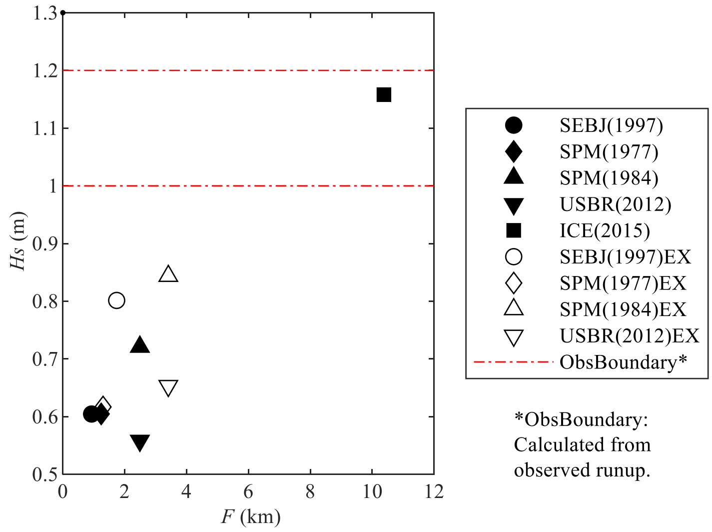


Figure 4.19 Šuoikkátjávri reservoir. Prediction of wave heights versus the fetch length used in the prediction. The legend refers to the methodology used, with the those ending with EX (for example SEBJ(1997)EX) refering to the exercise of using the extended fetches into the relevant prediction formula.

Figure 4.19 clearly demonstrates that all prediction methods, except the one following ICE (2015), underestimate the wave height backcalculated from the observed runup. This applies also for the exercise where the predictions are carried out with the extended fetches. However, when experimenting with the extended fetches the prediction formula in SPM (1984) reaches towards the lower boundary of the wave height associated with the observed runup. Similar, applies to SEBJ (1997) when using the extended fetches. Still, these two methods also underestimate the estimated observed value.

Exercise through the conceptual reservoir (not included here), with the shape ratio of the Šuoikkátjávri Reservoir set to B/L=0.06, also indicate that ICE (2015) is likely give the best prediction for a narrow reservoir.

Attention is drawn to the fact that the method recommended by ICE (2015) bases on the investigation of (Owen and Steel, 1988) into different methods and how these apply for reservoirs that are long and narrow, as in the case of the Śuoikkátjávri Reservoir.

Exercise relating to using SEBJ(1997) and different fethces

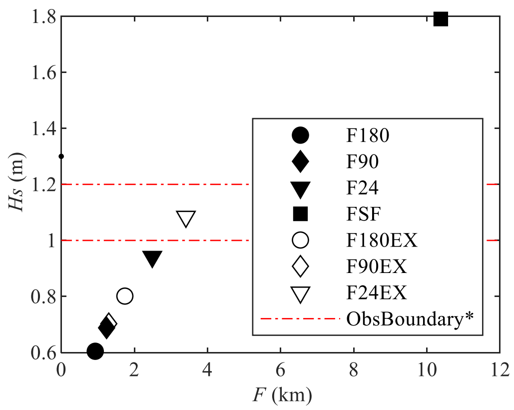
The Norwegian Regulations (NVE, 2003) recommend the methodology presented by SEBJ (1997). Thus, it is of interest to investigate the predictions from this methodology for all the fetch lengths presented above in Table 4.3. **However, this can only be considered as an exercise, bearing in mind that SEBJ (1997) emphasizes that the method of calculation and the values given to the coefficients and exponents of their prediction equations are inter-related and cannot be disassociated**. Thus it is not correct to extend the fetches as done here.

The resulting prediction is summarized in Table 4.5 and further illustrated in Figure 4.20. The Figure 4.20 demonstrates that prediction using SEBJ (1997) and the procedure recommended by (USBR, 2012a) to calculate the fetch length (**$F_{24°}$**) will for the current case (the Śuoikkátjávri Reservoir) lie within the boundary provided by the observed runup on September 6th 2017. Furthermore, that the bent single fetch (**F_{SF}**) will result in unacceptable overestimation. All other methods of calculating the fetches result in underestimation. The largest underestimation is obviously when using the effective fetch (**$F_{180°}$**) (the shortest fetch length) associated with the SEBJ (1997) prediction formulas.

Table 4.5 Predicted wave heights at Šuoikkátjávri for U=20 m/s the predictions formula from SEBJ(1997) used with different methods of defining the fetches. (Note: SEBJ(1997) warns that the predictions formulas are associated with the method of calculating the effective fetch $F_{180°}$, thus this is not a recommended procedure)

Method	Fetch lengths from Figure 4.17		Extended fetches from Figure 4.18	
	Fetch length used in calculation (km)	Calculated wave height in m	Extended Fetch length used in calculations (km)	Calculated wave height in m
SEBJ(1997)	$F_{180°}=0.93$	0.60	$F_{180°Ext}=1.74$	0.80
*SEBJ(1997)**	$F_{90°}=1.24$	0.69	$F_{90°Ext}=1.3$	0.70
*SEBJ(1997)**	$F_{24°}=2.49$	0.94	$F_{24°Ext}=3.41$	1.08
*SEBJ(1997)**	$F_{SF}=10.38$	1.8	$F_{SF}=10.38$	1.8

* Not with the recommended process of calculating the fetch to use with SEBJ(1997) prediction formulas.



*ObsBoundary refers to the wave height calculated from observed runup

Figure 4.20 Šuoikkátjávri reservoir. Wave heights predicted with SEBJ(1997) versus the fetch length used in the prediction. The legend refers to the methodology used to calculate the fetches, with the those ending with EX (for example F180EX) refering to the exercise of calculating the extended fetches.

4.3.6 Comparison of different methods for Nesjøen Reservoir

Nesjøen Reservoir is a mountain reservoir in Trøndelag (see Figure 3.3). In 1976 the rockfill dam retaining the reservoir was exposed to considerable wave action during a easterly winter storm, resulting in damages to the upstream slope (NGI (Norges Geotekniske Institutt), 1976). The riprap on the upstream slope was later rehabilitated and upgraded. However, considering that wave generation on the Nesjøen Reservoir has led to erosion damages, it is of interest to study this reservoir further and use this as a case study. The different methods to calculate the significant wave height will be employed as well as the two options in NVE(2003) for the wind velocity (30 m/s for the overwater wind or prediction by Eurocode). The Donelan Jonswap approach referred to in ICE(2015) will also be investigated. Additionally, the time (duration) required for the wind to generate the wave will be considered.

Nesjøen Reservoir-Fetches

The Nesjøen Reservoir is in relatively open terrain; however, the mountain sides seemingly allow for potential wind funnelling. The different fetches were drawn and calculated for the Nesjøen Reservoir (see Figure 4.21 and Figure 4.22). The resulting fetch lengths are summarized in Table 4.6

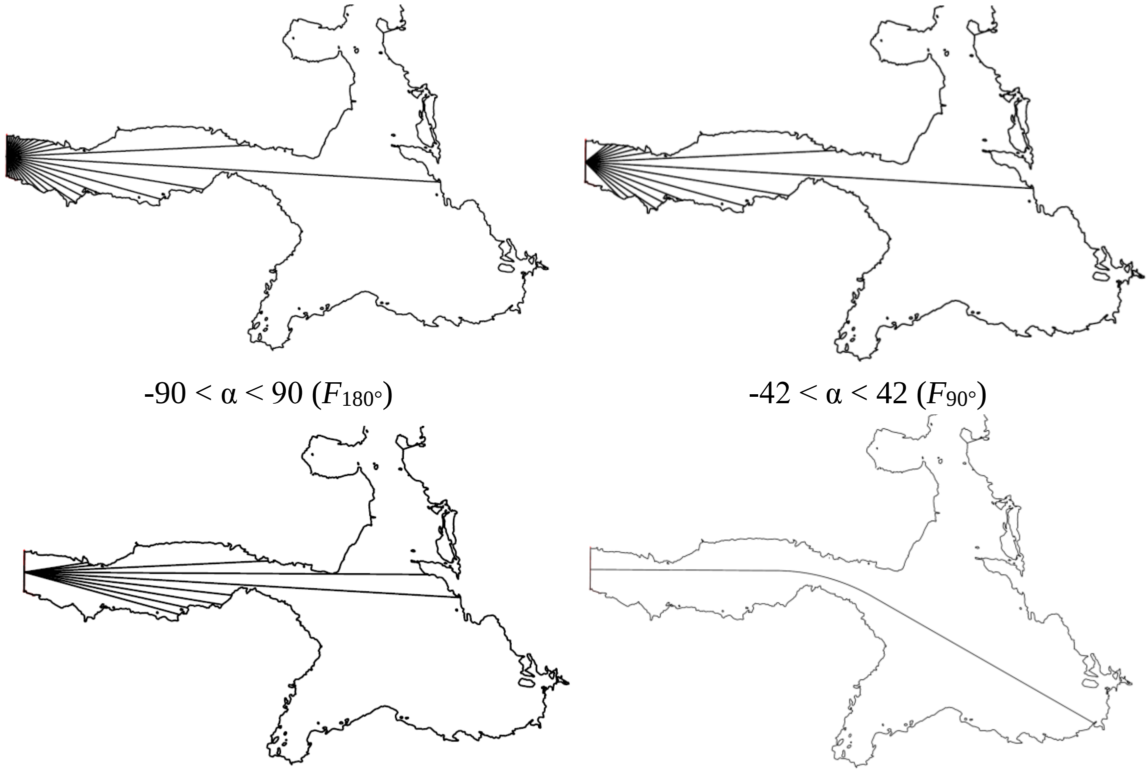
Table 4.6 Nesjøen reservoir. Calculated fetch lengths according to recommended procedures as well as for extended radials.

Method	Angle	Fetch length According to recommended procedures (see *Figure 4.21*)	Fetch length Extending (bending) lines along reservoir (see *Figure 4.22*)
SEBJ(1997)	+/-90° (180°)	$F_{180°}=2.07\ km$	$F_{180°Ext}=2.45\ km$
SPM(1977) (Saville)	+/-45° (90°)	$F_{90°}=2.8\ km$	$F_{90°Ext}=3.33\ km$
SPM(1983) and USBR(2012)	+/-12° (24°)	$F_{24°}=5.35\ km$	$F_{24°Ext}=6.46\ km$
ICE(2015)	Single fetch	$F_{SF}=13.3\ km$	*(„banana“ shape)*

Wind velocity 30 m/s regardless of fetch length

The significant wave heights, for the 10 minute mean wind velocity recommended by NVE (2003) (30 m/s), calculated employing the different formulas are summarized in Table 4.7 and compared in Figure 4.23a.

The significant wave height read from graphs in NVE (1981) that bases on the SMB original formulation is also given in Table 4.7. Furthermore, the significant wave height calculated in NGI (1976) is included in the table. NGI (1976) uses method described in (NGI, 1983) where a straight line fetch is modified by multiplying this with a shape coefficient to account for the reservoir shape when determining the effective fetch. The shape coefficient is a function of the reservoir shape (width against length) and read from a graph provided in NGI (1976).



-90 < α < 90 ($F_{180°}$) -42 < α < 42 ($F_{90°}$)

-12 < α < 12 ($F_{24°}$) Single bent fetch (F_{SF})

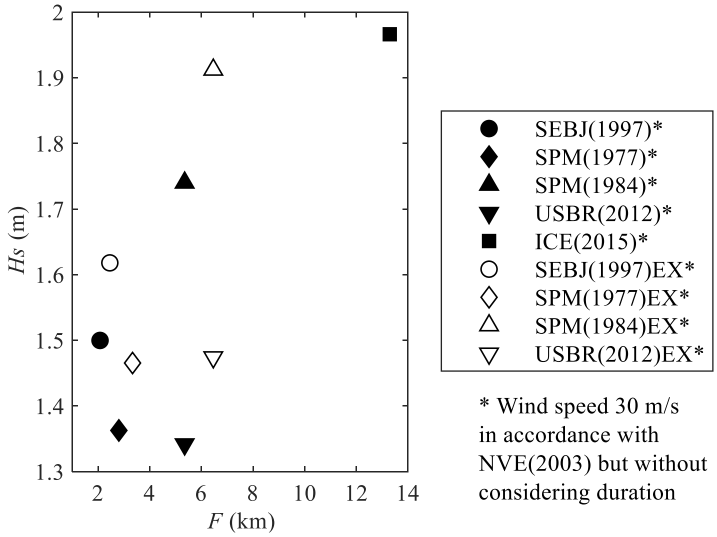
Figure 4.21 Outlines of the Nesjøen reservoir and radials used to calculate the fetch lengths ($F_{180°}$,$F_{90°}$, $F_{24°}$ and F_{SF}) to use in the different prediction formula. The fetch radials were drawn by Eirik Øvregård. (Outline of reservoir obtained from NVE map services).

$F_{180°EXT}$ $F_{90°EXT}$ $F_{24°EXT}$

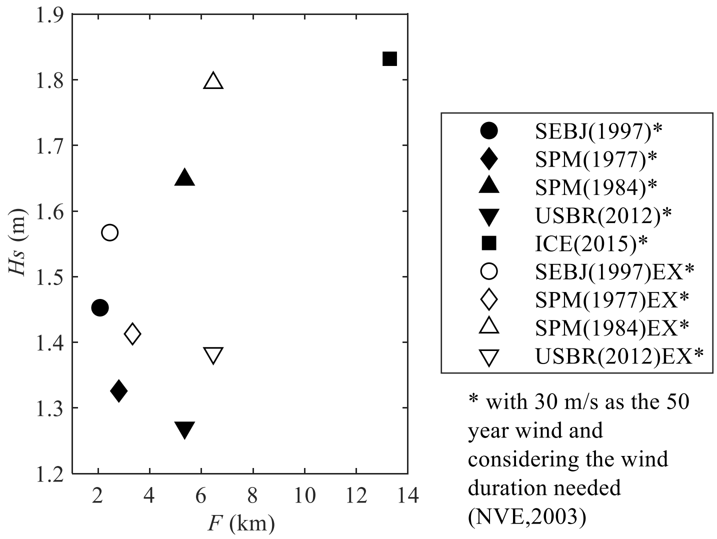
-90 < α < 90 -42 < α < 42 -12 < α < 12

Figure 4.22 Outlines of the Nesjøen reservoir and extended radials used to calculate the extended fetch lengths ($F_{180°Ext}$, $F_{90°Ext}$, $F_{24°Ext}$) that further are used in the different prediction formula. (Outline of reservoir obtained from NVE map services).

In both Table 4.7 and Figure 4.23a, neither the duration of the wind required to obtain the calculated significant wave height nor the wind direction is accounted for (as hitherto in this report), but this will also be studied in the following for the Nesjøen Reservoir. Accounting for the duration can result in that lower wind velocities values are applicable, particularly for the longer fetches, see for example Table 4.8 and compared in Figure 4.23b, that are further discussed in the following .



a) The wind velocity (and the wind direction) used in the calculations is the one recommended in NVE(2003) for the 50 year wind, however without correction for duration (i.e. the value 30 m/s is used in all cases).



b) The wind velocities consider required duration of the wind, using 30 m/s as the 10-minute mean wind).

Figure 4.23 Nesjøen Reservoir. Prediction of wave heights versus the fetch length used in the prediction. The legend refers to the methodology used, with the those ending with EX (for example SEBJ(1997)EX) referring to the exercise of using the extended fetches into the relevant prediction formula. a) The wind velocity (and the wind direction) used in the calculations is the one recommended in NVE(2003) for the 50 year wind, however without correction for duration (i.e. the value 30 m/s is used in all cases). b) Same wind velocities as in a) but here required duration of the wind along the fetches is considered (using 30 m/s as the 10 minute mean wind).

Duration needed to generate the highest waves

The duration needed to generate the highest waves for a given fetch is plotted in Figure 4.24. Two graphs (the blue lines) are plotted for the duration, considering a wind velocity of 30 m/s, one using the formula given by SEBJ(1997) (Eq. 3.53) and the other using the formula given by USBR(2012). The duration by SEBJ(1997) is comparable to the duration read from diagrams in NVE (1981), thus this duration will be considered for adjusting the wind velocities used. This method also gives shorter duration and thus higher velocities.

Datapoints for the Nesjøen Reservoir are also plotted in Figure 4.24, and represent the duration required for a wind speed of 30 m/s to generate the highest wave along the different fetches calculated (see fetches in Table 4.6). Additionally, a graph of duration adjusted wind velocities is plotted, basing on 30 m/s as the 10-minute mean wind. Thus from Figure 4.24 one can estimate the wind velocity to be used with the different fetch lengths, given that the 10 minute mean wind is 30 m/s.

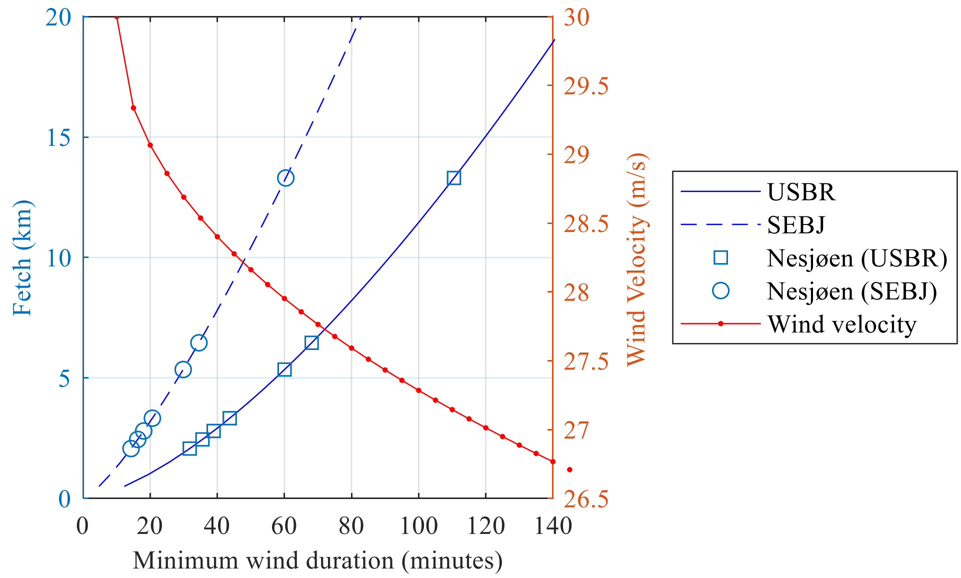


Figure 4.24 Nesjøen Reservoir. The duration needed according to SEBJ(1997) and USBR(2012) to generate the highest waves for a given fetch and a 10 minute mean overwater wind velocity of 30 m/s (the NVE(2003) value). The duration for the different fetches calculated for Nesjøen are identified. The mean wind velocity for different duration is also shown.

Duration adjusted wind velocity with 30 m/s as the 10 min. mean wind

The significant wave heights, calculated considering approximately the required duration of the wind and consequent reduction in the wind velocity with a base value of 30 m/s for the 10 minutes mean wind, are given in Table 4.8 and compared in Figure 4.23b. The column with the wind velocity and duration over which this is averaged, gives the mean wind velocity over the duration specified. For example, the value 29.3 (15), in the first row of that column, gives the 15 minutes mean wind velocity as 29.3 m/s. Similarly, the value 27.9 (60) gives the 60 minutes mean wind velocity as 27.9 m/s.

Table 4.9 gives the same information for the 1000 year wind basing on 30 m/s as the 50 year wind. The same duration is used as for the 50 year wind, although the duration is slightly lower with the increase in velocity. The resulting significant wave height will be plotted later in a figure comparing all the methods and the different wind velocities used.

Table 4.7 Predicted wave heights at Nesjøen *for U=30 m/s and different methods.*

Method	Fetch lengths according to the methodology used		Exercise of using extended radials when calculating the fetches	
	Fetch length (km)	Calculated wave height in m	Fetch length extended (km)	Calculated wave height in m
SEBJ(1997)	$F_{180°}=2.07$	1.50	$F_{180°Ext}=2.39$	1.62
SPM(1977)	$F_{90°}=2.8$	1.36	$F_{90°Ext}=3.25$	1.47
SPM(1983)	$F_{24°}=5.35$	1.74	$F_{24°Ext}=6.5$	1.91
USBR(2012)	$F_{24°}=5.35$	1.34	$F_{24°Ext}=6.5$	1.47
ICE(2015)	$F_{SF}=13.3$	1.97	$F_{SF}=13.3$	1.97
Read from charts in NVE(1981)				
NVE(1981)	$F_{90°}=2.8$	~1.2	$F_{90°Ext}=3.25$	~1.4
Estimated by NGI in the year 1976 (NGI, 1976). The fetch is estimated considering the reservoir shape.				
	$F_{NGI}=3.4$	1.5		

Table 4.8 Predicted wave heights at Nesjøen *considering different methods and wind duration (basing on a wind velocity of 30 m/s for the 10 minute mean wind).*

Method	Fetch lengths according to the methodology used			Exercise of using extended radials when calculating the fetches		
	Fetch length (km)	Wind in m/s (duration in minutes)	H_s m	Fetch length extended (km)	Wind in m/s (duration in minutes)	H_s M
SEBJ(1997)	$F_{180°}=2.07$	29.3 (15)	1.45	$F_{180°Ext}=2.39$	29.3 (15)	1.57
SPM(1977)	$F_{90°}=2.8$	29.3 (15)	1.32	$F_{90°Ext}=3.25$	29.0 (20)	1.40
SPM(1983)	$F_{24°}=5.35$	28.7(30)	1.65	$F_{24°Ext}=6.5$	28.5 (35)	1.80
USBR(2012)	$F_{24°}=5.35$	28.7(30)	1.27	$F_{24°Ext}=6.5$	28.5 (35)	1.38
ICE(2015)	$F_{SF}=13.3$	27.9 (60)	1.83	$F_{SF}=13.3$	27.9 (60)	1.83

Table 4.9 Predicted wave heights at Nesjøen *considering different methods, wind duration and the 1000 year NVE(2003) wind (basing on a wind velocity of 30 m/s as the 50 year 10 minute mean wind).*

Method	Fetch lengths according to the methodology used			Exercise of using extended radials when calculating the fetches		
	Fetch length (km)	Wind in m/s (duration in minutes)*	H_s m	Fetch length extended (km)	Wind in m/s (duration in minutes)*	H_s m
SEBJ(1997)	$F_{180°}=2.07$	33.9 (15)	1.77	$F_{180°Ext}=2.39$	33.9 (15)	1.90
SPM(1977)	$F_{90°}=2.8$	33.9 (15)	1.57	$F_{90°Ext}=3.25$	33.5 (20)	1.67
SPM(1983)	$F_{24°}=5.35$	33.2 (30)	1.97	$F_{24°Ext}=6.5$	32.9 (35)	2.15
USBR(2012)	$F_{24°}=5.35$	33.2 (30)	1.52	$F_{24°Ext}=6.5$	32.9(35)	1.65
ICE(2015)	$F_{SF}=13.3$	32.3 (60)	2.11	$F_{SF}=13.3$	32.3 (60)	2.11

*same approximate duration is used as for the 50 year wind, although this will be slightly lower with the increase in velocity.

Consideration of the direction of the wind and/or the predominant wave

The Eurocode approach to predicting the wind velocity includes the effect of wind direction. The directional factors to consider at Nesjøen Reservoir according to the Eurocode are given in Section 3.2.1. However, before calculating the significant wave heights from the different formulas in Table 4.1 using the Eurocode approach to predicting the wind velocities, it is interesting to study how the wind direction and the predominant wave height are considered with the Donelan-Jonswap method, referred to in ICE(2015).

ICE (2015) bases on the Eurocode approach to predict the wind velocities, however instead of the roughness factor (c_r) a factor for wind speed adjustment over water are provided (f_W). ICE (2015) explains, that simple methods of wave estimation may use a single fetch distances as demonstrated in the guidelines. At the same time, ICE(2015) refers to Herbert et al. (1995) for a more complete methods that may assess the contributions of a spread of fetches either side of the principal direction being considered.

The method referred to in ICE(2015), the Donelan-Jonswap (DonJon) method, was identified by Owen and Steel (1988) as “probably the best” for narrow long reservoirs, when they compared several methods to measured data at the Megget Reservoir and Loch Glascarnogh. Thus, it is interesting to apply this to the Nesjøen Reservoir as well as the more simple single fetch approach also given in ICE(2015) with an extended fetch.

The predominant wave direction was found for wind directions of 90° (East), 135° (South East), 165° (South South East) and 180° (South). Example of the calculations is provided in Table 4.10 for wind directions: 135°, 165° and 180°. The significant wave height is calculated employing Eq. 3.43, considering the prodominant wave direction and the different parameters for the wind velocity. Calculations of the wind velocities and significant wave heights are carried out in Table 4.11, Table 4.12 and Table 4.13.

In Table 4.11 the notation and formulation of the Eurocode is applied for the different wind factors, except that the roughness factor (c_r) is replaced with the wind speed adjustment overwater factor (f_W) from ICE (2015) (also given in NVE, 1981) but originally by Saville et al., (1962). As previously mentioned, these factors were median values of considerably scattered data points from only two reservoirs.

In Table 4.12 the 50 year wind is predicted fully according to the Eurocode, but is otherwise the same as Table 4.11. The seasonal factor (c_{season}) and the orthography factor ($c_o(z)$) are both set equal to 1.0. Furthermore, the 1000 year wind according to the Eurocode is used in the prediction in Table 4.13. The 1000 year wind is to be used in the design of riprap on the upstream slope of embankment dams in Norway according to the regulation NVE (2003) or a wind velocity over water of 30 m/s.

The case of the single bent fetch (see Figure 4.21) in Table 4.11, Table 4.12 and Table 4.13, considers that wind blowing approximately from South-East is funnelled along the fetch.

Table 4.10 Example of finding the predominant wave direction for a given wind direction.

Wind direction			θ = 135°		θ = 165°		θ = 180°	
	Angle of fetch ϕ (°)	Fetch length (F) (m)	cos(θ-ϕ)	$\cos^4(\theta-\phi) F^{0.3}$	cos(θ-ϕ)	$\cos^4(\theta-\phi) F^{0.3}$	cos(θ-ϕ)	$\cos^4(\theta-\phi) F^{0.3}$
N	3	503	-0.669		-0.951		-0.999	
	9	507	-0.588		-0.914		-0.988	
	15	523	-0.500		-0.866		-0.966	
	21	509	-0.407		-0.809		-0.934	
	27	520	-0.309		-0.743		-0.891	
	33	554	-0.208		-0.669		-0.839	
	39	552	-0.105		-0.588		-0.777	
	45	595	0.000		-0.500		-0.707	
N-E	51	681	0.105	2.87	-0.407		-0.629	
	57	790	0.208	3.95	-0.309		-0.545	
	63	898	0.309	4.81	-0.208		-0.454	
	69	1096	0.407	5.70	-0.105		-0.358	
	75	1327	0.500	6.55	0.000	0.00	-0.259	
	81	1592	0.588	7.38	0.105	3.70	-0.156	
	87	6160	0.669	11.67	0.208	7.31	-0.052	
	90	9807	0.707	13.72	0.259	9.18	0.000	0.00
E	93	10735	0.743	*14.38	0.309	*10.12	0.052	4.97
	96	4935	0.777	11.59	0.358	8.51	0.105	5.20
	102	3961	0.839	11.19	0.454	8.75	0.208	6.40
	108	2503	0.891	9.99	0.545	8.20	0.309	6.54
	114	2107	0.934	9.66	0.629	8.25	0.407	6.93
	120	2062	0.966	9.73	0.707	8.59	0.500	*7.48
	126	1319	0.988	8.59	0.777	7.80	0.588	6.98
	132	1172	0.999	8.33	0.839	7.76	0.669	7.09
S-E	138	859	0.999	7.59	0.891	7.25	0.743	6.74
	144	794	0.988	7.38	0.934	7.21	0.809	6.81
	150	626	0.966	6.81	0.966	6.81	0.866	6.52
	156	613	0.934	6.67	0.988	6.82	0.914	6.62
	162	546	0.891	6.33	0.999	6.62	0.951	6.49
	168	518	0.839	6.08	0.999	6.52	0.978	6.46
	174	467	0.777	5.71	0.988	6.29	0.995	6.31
S	180							
*Maximum value-predominant wave direction			ϕ=93°, for fetch length F=10 735 m		ϕ=93°, for fetch length F=10 735 m		ϕ=120°, for fetch length F=2 062 m	

Table 4.11 Nesjøen, example of significant wave height calculations basing on ICE(2015) and the DonJon (Donelan/Jonswap) option, for the 50 year wind.

	Straight line fetches				Bent fetch
	E	SE	SSE	S	SE
Wind direction, θ	90°	135°	165°	180°	135
Predominant wave direction, ϕ	93°	93°	93°	120°	not used
$\cos(\theta - \phi)$	0.999	0.743	0.309	0.500	not used
Fetch (m) along ϕ	10 735	10 735	10 735	2 062	13 300
$v_{b,o}$ (m/s)	25	25	25	25	25
Duration needed (minutes)	50	50	50	15	60
c_{prob} (f_T)	1	1	1	1	1
c_{alt} (f_A)	1	1	1	1	1
c_T (f_D)	0.939	0.939	0.939	0.978	0.932
f_W	1.3	1.3	1.3	1.16	1.31
c_{dir} (f_N)	0.8	0.9	1	1	0.9
$v_{m,p,T}$ (m/s) (50 year wind)	24.4	27.5	30.5	28.4	27.5
H_s (m)	1.44*	1.20	0.56	0.37	1.8**

*Governing if only straight-line fetches are considered. ** Upper boundary

Table 4.12 Nesjøen, example of significant wave height calculations basing on ICE(2015) and the DonJon option. Wind prediction (return period of 50 year) after Eurocode.

	Straight line fetches				Bent fetch
	E	SE	SSE	S	SE
Wind direction, θ	90°	135°	165°	180°	135
Predominant wave direction, ϕ	93°	93°	93°	120°	not used
$\cos(\theta - \phi)$	0.999	0.743	0.309	0.500	not used
Fetch (m) along ϕ	10 735	10 735	10 735	2 062	13 300
$v_{b,o}$ (m/s)	25	25	25	25	25
Duration needed (minutes)	50	50	50	15	60
c_{prob} (f_T)	1	1	1	1	1
c_{alt} (f_A)	1	1	1	1	1
c_T (f_D)	0.939	0.939	0.939	0.978	0.932
c_r(z)	1.17	1.17	1.17	1.17	1.17
c_{dir} (f_N)	0.8	0.9	1	1	0.9
$v_{m,p,T}$ (m/s) (50 year wind)	22.0	24.7	27.5	28.6	24.5
H_s (m)	1.3*	1.1	0.5	0.4	1.6**

*Governing if only straight-line fetches are considered. ** Use?

Table 4.13 Nesjøen, example of significant wave height calculations basing on ICE(2015) and the DonJon option. Wind prediction (return period of 1000 year) after Eurocode.

	Straight line fetches				Bent fetch
	E	SE	SSE	S	SE
Wind direction, θ	90°	135°	165°	180°	135
Predominant wave direction, ϕ	93°	93°	93°	120°	not used
$\cos(\theta - \phi)$	0.999	0.743	0.309	0.500	not used
Fetch (m) along ϕ	10 735	10 735	10 735	2 062	13 300
$v_{b,o}$ (m/s)	25	25	25	25	25
Duration needed (minutes)	50	50	50	15	60
c_{prob} (f_T)	1.16	1.16	1.16	1.16	1.16
c_{alt} (f_A)	1	1	1	1	1
c_T (f_D)	0.939	0.939	0.939	0.978	0.932
$c_r(z)$	1.17	1.17	1.17	1.17	1.17
c_{dir} (f_N)	0.8	0.9	1	1	0.9
$v_{m,p,T}$ (m/s) (1000 year wind)	25.5	28.7	31.9	33.2	28.5
H_s (m)	1.5*	1.3	0.6	0.4	1.9**

*Governing if only straight-line fetches are considered. ** Use?

Wind velocity predicted according to the Eurocode used to calculate the significant wave height.

The significant wave heights, calculated according to the formulas listed in Table 4.1 and using wind velocities according to the Eurocode, are summarized in Table 4.14 and Table 4.15 for the 50 and 1000 year wind respectively. The time (duration) required for the wind to generate the wave height along the given fetch is considered in calculating the values given in the tables, as well as the wind direction. In the exercise of using the extended radials when calculating the fetches, the wind direction is determined as in the case of the single bent fetch, i.e. considers that wind blowing approximately from the South-East is funnelled along the extended fetch.

Table 4.14 Predicted wave heights at Nesjøen considering different methods, wind duration and the 50 year wind according to Eurocode.

Method	Fetch lengths according to the methodology used			Exercise of using extended radials when calculating the fetches		
	Fetch length (km)	Wind in m/s (duration in minutes) Dir	H_s m	Fetch length extended (km)	Wind in m/s (duration in minutes) Dir	H_s m
SEBJ(1997)	$F_{180°}=2.07$	22.9 (15)E	1.04	$F_{180°Ext}=2.39$	25.8 (15)SE	1.32
SPM(1977)	$F_{90°}=2.8$	22.7 (20)E	0.99	$F_{90°Ext}=3.25$	25.5 (20)SE	1.22
SPM(1983)	$F_{24°}=5.35$	22.3 (35)E	1.21	$F_{24°Ext}=6.5$	25.1 (35)SE	1.53
USBR(2012)	$F_{24°}=5.35$	22.3 (35)E	0.93	$F_{24°Ext}=6.5$	25.1 (35)SE	1.18
ICE(2015)	$F_{SF}=13.3$	24.5 (60)SE	1.61	$F_{SF}=13.3$	24.5 (60)SE	1.61

Table 4.15 Predicted wave heights at Nesjøen considering different methods, wind duration and the 1000 year wind according to Eurocode.

Method	Fetch lengths according to the methodology used			Exercise of using extended radials when calculating the fetches		
	Fetch length (km)	Wind in m/s (duration in minutes)	H_s m	Fetch length extended (km)	Wind in m/s (duration in minutes)	H_s m
SEBJ(1997)	$F_{180°}=2.07$	26.5 (15)E	1.27	$F_{180°Ext}=2.39$	29.8 (15)SE	1.60
SPM(1977)	$F_{90°}=2.8$	26.3 (20)E	1.17	$F_{90°Ext}=3.25$	29.5 (20)SE	1.44
SPM(1983)	$F_{24°}=5.35$	25.9 (30)E	1.45	$F_{24°Ext}=6.5$	29.0 (35)SE	1.83
USBR(2012)	$F_{24°}=5.35$	25.9 (30)E	1.12	$F_{24°Ext}=6.5$	29.0 (35)SE	1.41
ICE(2015)	$F_{SF}=13.3$	28.4 (60)SE	1.86	$F_{SF}=13.3$	28.4 (60)SE	1.86

Comparison of the different methods when using wind velocity 30 m/s

Comparing the values in Figure 4.23 a and b, as well as the significant wave heights in Table 4.7 and Table 4.8, the following is noted:

1. There is a wide difference in the significant wave height values depending on the method employed, the increase from the lowest to the highest ranges from ca 30% for the extended fetches and duration adjusted wind velocities to ca 60% for the basic fetches according to methodology and a fixed wind speed of 30 m/s. Excluding the NVE (1981) chart approach the increase from the lowest to the highest for the basic fetches is 45%.

2. The wind velocity should in theory be duration adjusted, particularly for long fetches, as done in calculation of the significant wave height in Table 4.8. However, this has only a minor effect on the significant wave height. The reduction in the significant wave height is obviously largest for the longer fetches or up to 7% for the ICE(2015) calculated wave height. The reduction is for example only 3% for the SEBJ(1997) method.

3. The NVE (1981) approach basing on the original SMB formulation and Saville's effective fetch, results in the lowest significant wave height estimates for the Nesjøen Reservoir. NGI (1976) evaluation on the Nesjøen Reservoir bases on the same methodology, except that the fetch is estimated by taking the longest fetch and apply a reduction factor considering the reservoir's shape (see (NGI, 1983)).

Comparison relating to the ICE(2015) related DonJon approach

The ICE (2015) approach considers the wind direction and duration over straight or bent fetches, but also refers to a more detailed approach with the consideration of the predominant wave direction (the DonJon method). Investigation into this method demonstrates that for the Nesjøen Reservoir the longest fetch, which is the bent fetch, will always give the largest significant wave height (compare Table 4.11, Table 4.12, and Table 4.13). However, if one only considers straight line fetches, the longest fetch occurs for the wind blowing from the East (with directional factor 0.8) and this will give the highest significant wave height.

The bent fetch approach assumes that the wind will be funneled along the reservoir longitudinal axis. The predominant wave direction is not used in the case of the bent fetch, (or equivalently the value of $\cos(\theta - \phi)$ is equal to 1.0, i.e. the wind direction (varying along the bent fetch) always follows the predominant wave direction (varying along the bent fetch)). The significant wave height predicted along the bent fetch can be considered an upper boundary to the predictions. It has previously been mentioned, that numerical modelling of two reservoirs in the UK and comparison to the straight line and bent fetches, revealed that the straight line fetches gave slightly lower values compared to results from the numerical model, while the bent fetches gave considerably higher significant wave heights.

It is interesting to compare the significant wave height predictions from the NGI memorandum from 1976 (NGI, 1976) following the event leading to damages to the riprap on the Nesjøen dam. NGI considers a wind blowing from the East and uses an effective fetch of 3.4 km and wind velocities of 25 m/s and 30 m/s, respectively resulting in a significant wave height of 1.25 m and 1.5. m. The 1.25 m NGI value for wind velocity of 25 m/s can be compared to the value of H_s of 1.44 m and 1.3 m in the column for the wind blowing from the East in Table 4.11 and Table 4.12, respectively. Similarly, the 1.5 m NGI value (a check on this value by reading from the graphs in NVE (1981) gives a value between 1.35 to 1.4 m) for wind velocity of 30 m/s can be compared to the value of H_s of 1.5 m in the column for the wind blowing from the East in Table 4.13. Furthermore, SEBJ, (1997) used in NVE (2003), also results in a value of 1.5 m for 30 m/s, but 1.62 m for the extended fetches (which is not a recommended approach).

Table 4.11, Table 4.12 and Table 4.13, use different wind velocity. Table 4.11 and Table 4.12, both consider wind with a return period of 50 years. However, in Table 4.11 the ICE (2015) approach is used, which is essentially prediction after the Eurocode, except that the rouhness factor is not used but the wind speed adjustment overwater factor (f_W) (also given in NVE (1981), but originally by Saville et al., (1962)). As previously mentioned, these factors were median values of considerably scattered data points from only two reservoirs. Thus, it is questionable how well these represent in general the increase in the wind as it blows over water. Hence, in Table 4.12 the Eurocode prediction is employed to the fullest, i.e. with the roughness factor ($c_r(z)$), with a fixed value of $c_r(z)$=1.173 for z_o=0.01. Table 4.13, summarizes the same for the 1000 year wind.

It should be investigated to what extent the roughness factor z_o varies with the length of the fetch, for example the value given in the Eurocode for sea or coastal area exposed to the open sea is z_o=0.003, resulting in $c_r(z)$=1.266. Thus, it is reasonable to assume an upper value of the roughness factor to be $c_r(z)$=1.266, when predicting wind overwater velocities for reservoirs[5]. However, the relation between the overwater and inland wind

[5] The significant wave height developing along the bent fetch and the 50 year wind, is 1.8 m when using the overwater wind factors (f_W), and 1.6 m when using the roughness factor, $c_r(z)$=1.173. A roughness factor of $c_r(z)$=1.266 will result in a wave height of 1.74 m. The significant wave height developing along the bent fetch and the 1000 year wind is 1.9 m (or 1.86m) when using the roughness factor, $c_r(z)$=1.173, in comparison a roughness factor of $c_r(z)$=1.266 will result in a wave height of 2.0 m.

velocities given by both (SEBJ, 1997) and (USBR, 2012a) and how this changes with increase in inland wind velocities should also be considered (see Figure 3.16).

Comparison of methods and the two approaches in NVE(2003) for determining the wind velocity

The significant wave height calculated employing the different methods and the two approaches in NVE(2003) for determining the wind velocity are compared in Figure 4.25. The two approaches in NVE(2003) is to either use an overwater wind speed of 30 m/s, see Table 4.7 to Table 4.9) alternatively predict the wind velocity from the Norwegian Standard (Norsk Standard) that today are the Eurocodes, see Table 4.14 and Table 4.15.

The significant wave heights values in Table 4.7 to Table 4.9, as well as Table 4.14 and Table 4.15 are plotted in Figure 4.25. The different symbols in the figure refer to the methodology used to predict the significant wave height, while the different colours refer to the different approaches in estimating the wind velocities. As previously mentioned, wind direction South-East is associated with the extended fetches in the Eurocode approach.

Reddish colours in Figure 4.25 represent the wind velocity of 30 m/s as the 10 minute mean wind. The three different shade of red from darkest, medium and the lightest shade, respectively represent: a constant value of 30 m/s, duration adjusted value with 30 m/s as the 50 year 10 minute mean wind, and duration adjusted value of the 1000 year wind, basing on the 30 m/s as the 50 year 10 minute mean wind.

The bluish colours in Figure 4.25 represent Eurocode prediction of the 50 year and 1000 year wind velocities adjusted for duration, where the darker shade of blue is the 50 year wind.

The two lines in Figure 4.25, one dashed and the other dash-dot, represent values from NGI (1976). The dash-dotted line is drawn between two values of the significant wave height, H_s, predicted in NGI (1976) for wind velocity of 25 m/s and 30 m/s, while the dashed line is the design wave height, H_d, , given as 1.3* H_s and used for dimensioning of riprap stones.

There is a considerable spread in the prediction of the significant wave heights, depending on the method chosen, and without measurements/observations one cannot say with certainty which one provides the best estimate. Still, basing on previous investigations into the methods using the conceptual reservoir (e.g. comparison of the average wave heigths values in case of the Megget Reservoir and Loch Glascarnoch see Chapter 4.3.4), the prediction according to ICE(2015) or SPM (1984), both basing on the JONSWAP spectrum, would be considered as potentially best suited for this case, i.e. a narrow reservoir with low *B/L* ratio.

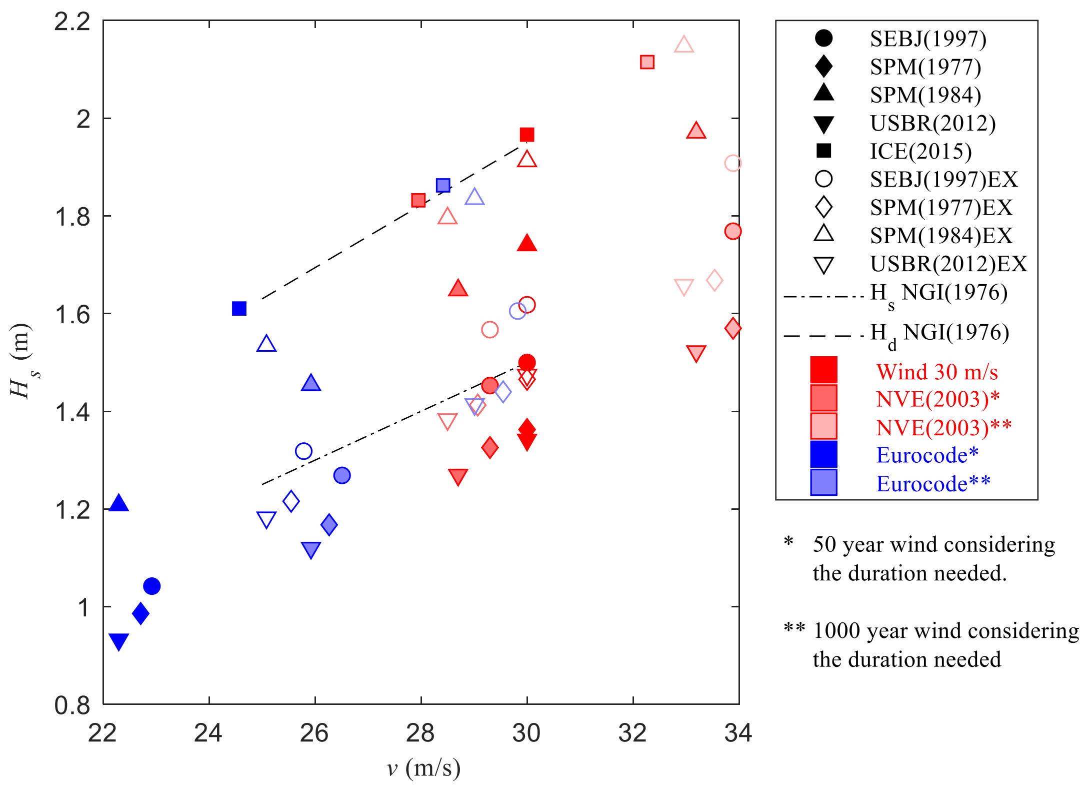


Figure 4.25 Nesjøen Reservoir. Prediction of wave heights versus wind velocity used in the prediction. The different symbols in the legend refers to the method used to predict the significant wave height, with the those ending with EX (for example SEBJ(1997)EX) referring to the exercise of using the extended fetches into the relevant prediction formula. The different colors of the symbols represent the method of predicting the wind velocity.

4.3.7 Effect of wind velocity in the prediction equations

The effect of wind velocity in the prediction equations in Table 4.1, other than SPM (1977) is brought forth in Figure 4.26 a) and b), as well as in Figure 4.27. The wind factor in each of the equations (e.g. U in the prediction formula in ICE (2015) and $U^{1.353}$ in SEBJ (1997), etc), is calculated and then scaled to the wind factor corresponding to the lowest velocity in Figure 4.26 but the highest in Figure 4.27. In Figure 4.26 a) and b), the lowest wind velocity on the horizontal scale are a) and b), respectively 5 m/s and 15 m/s. The highest wind velocity on the horizontal scale of Figure 4.27 is 30 m/s.

The effect of the wind factor is greatest in SEBJ (1997) prediction formula, while the increase in ICE (2015) with the linear relation to U obviously gives a lower boundary (see Figure 4.26).

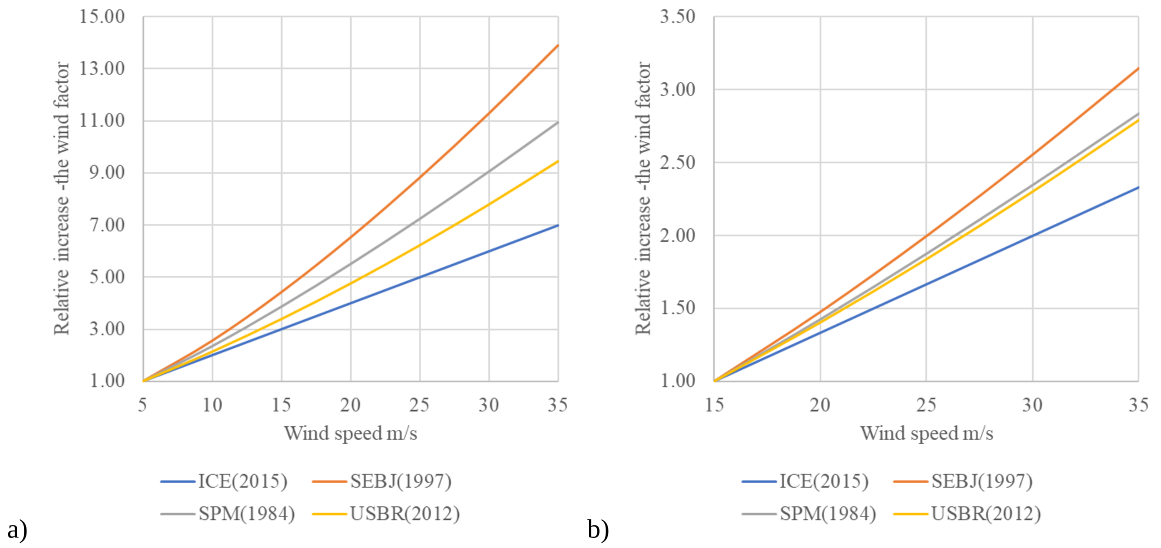


Figure 4.26 Relative increase in the wind factor in different wave heigth prediction formulas. a) Scaled against the windfactor for each method calculated at 5 m/s. b) Scaled against a windfactor for each method calculated at 15 m/s

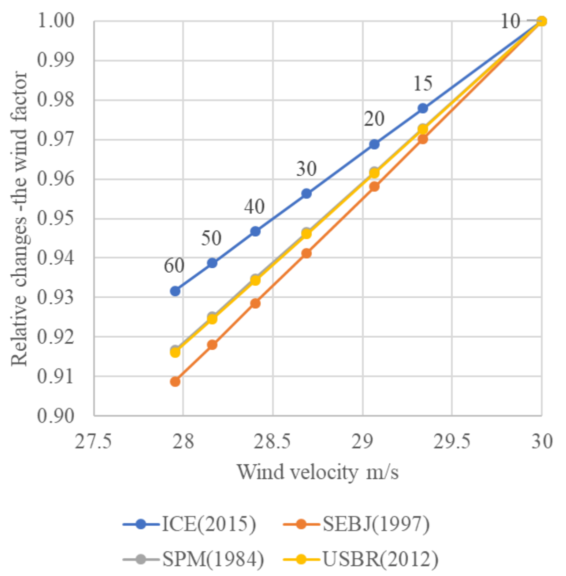


Figure 4.27 Wind duration and relative changes in the wind factor in different wave heigth prediction formulas, scaled against the windfactor for each method calculated for the 30 m/s. Given that 30 m/s is the 10 minutes mean wind, then the values 10 upto 60 above the data points represent the time period in minutes over which the wind velocity is averaged (e.g. 60 refers to the 60 minute mean wind).

The wind velocities used to create horizontal value of the data points in Figure 4.27 are calculated using the duration factor $C_t(T)$, with T the period of wind duration giving the T minutes mean wind velocity. Figure 4.27 assumes that the 10 minutes mean wind velocity is 30 m/s, and the duration T is listed above the corresponding data points given by the wind velocity $v_{m,\mathrm{p},T}$ = $v_{m,0.02,T}$ on the horizontal scale and a ratio of the wind factor (for $v_{m,0.02,T}$ versus $v_{m,0.02,10\ minutes}$ = 30 m/s) in the different formulation on the vertical axis. The

Figure 4.27 demonstrates that if the duration needed for fully developing the wave height along a certain fetch is 60 minutes or more, the significant wave height can be reduced by a factor of at least 0.91 (SEBJ, 1997) to 0.93 (ICE, 2015) or by at least 7% to 10%.

A holistic overview on the relationship between the duration required, fetch and wind velocity is given with the 3D graph in Figure 4.28.

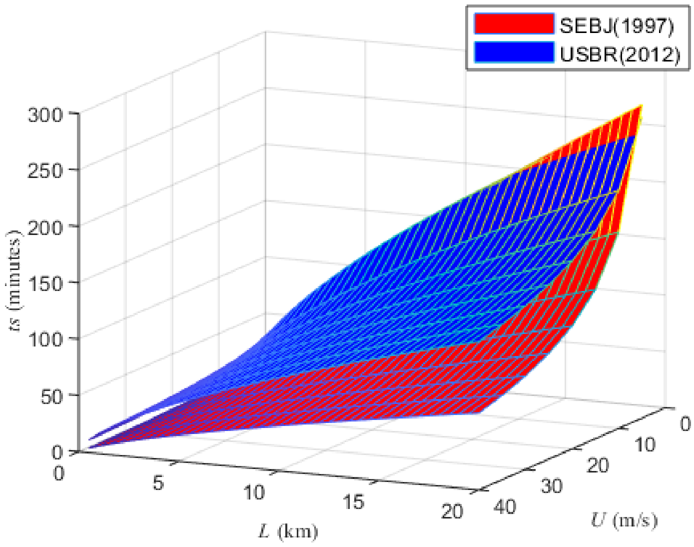


Figure 4.28 Relationship between the duration needed (t_S), velocity (U) and fetch (L) using the formulation for the duration given in SEBJ (1997) and USBR (2012).

Range of the wind velocities used in deriving at the formulas

The SEBJ (1997) prediction formula were obtained from wind velocities ranging up to 20 m/s, while ICE (2015) bases on the Jonswap formulation (Hasselmann et al., 1973) derived from measurement with wind velocities up to 15 m/s (but mostly wind velocity between 5 and 10 m/s). SPM (1984) also bases on the Jonswap formulation, although use a wind stress factor without further explanation. The author of this report has not found the ranges of data used to derive at the formulation in USBR (2012).

Hence, prediction for wind velocities larger than 20 m/s can generally be expected to be outside the range of measured data from which the prediction formulas were derived. , Measurements of the wind and waves would be valuable to investigate how well the prediction formulas predict wave heights when the wind velocity is larger than 20 m/s.

4.4 Summary on wave height predictions

Wind generated wave prediction formulas given in different dam safety guidelines or manuals in USA, UK, Canada and Norway have been compared.

- The prediction formulas considered from US guidelines include those presented in two different versions of the Shore Protection Manual (from 1977 and 1984), and the dam engineering version of USBR (2012) of the Coastal Engineering Manual (USACE, 2008).

- The prediction formulas considered from Canada are those given in SEBJ(1997), which is also the prediction formulas referred to in Norwegian guidelines NVE(2003). However, in Canada the use of SPM(1984) may also be an option.

- The prediction formulas considered from the UK are presented in ICE(2015). These base on the JONSWAP formula, as do the formulas in SPM(1984). However, the method of estimating the fetches is different in the two guidelines. Furthermore, for the same input values of the fetch and wind velocity into the formulas, the UK guidelines will always result in 0.026/0.016 times higher significant wave height values than the values obtained from SPM(1984).

The formulas all relate the significant wave height to wind velocity and the fetch along which the wind blows. However, with slightly different constant and coefficients. Furthermore, the fetch is defined in different ways and the wind velocity over water is estimated differently. Additionally, the different guidelines may use different return periods assigned to the wind velocities used for dam engineering purposes.

In the comparison carried out in this Chapter in all cases except for the Nesjøen Reservoir, the same basic value of the wind velocity was inserted into the different prediction formulas, while the values of the fetches inserted are different and according to each methodology. Thus, the duration required for the wind to develop the wave height along the fetch, which will yield slightly different values of the wind velocity, is not considered, except for the Nesjøen Reservoir. This is to make direct comparison of the formulas used easier and more transparent.

Regarding ways to determine the fetch used into the formulas, the Canadian guidelines SEBJ(1997), clearly state that the method presented to calculate the fetch is inherent into the methodology. Thus, the exercise of extending the fetches as done here in this Chapter cannot be recommended when using prediction formulas in SEBJ(1997). Conversely, the prediction methods given in SPM(1984) and USBR(2012), both relate to the JONSWAP spectrum, as does the prediction formula given in ICE(2015). Thus, extended fetches, or the single fetch approach should also be applicable with SPM(1984) and USBR(2012) as with ICE(2015).

The different prediction formulas are compared through a conceptual reservoir of different shape ratios. However, the predictions cannot be properly validated without comparison to measurements on actual reservoirs. Thus, to evaluate the predictions to some extent, available measurements found in the literature from actual reservoirs were compared to predictions made for a conceptual reservoir of similar shape ratio as the actual reservoir. Additionally, statements relating to comparing SEBJ(1997) and SPM(1984) were investigated through the conceptual reservoir. The following findings can be extracted from this comparison:

- Measurements from Lake Ontario, a large lake of length L=240 km and with a shape ratio of B/L=0.2, compare reasonably to prediction by SEBJ(1997) calculated through the conceptual reservoir of length L=240 km and with B/L=0.2.

- The measurements on the large La Grande Complex Reservoirs did not compare well to predictions by SPM(1984). Dupuis et al.(1996) explain that SPM(1984) overpredicted small wave heights and under predicted large wave heights measured on the La Grande Complex Reservoirs. Thus, Dupuis et al.(1996) analyzed the collected data on the La Grande Complex and suggested the prediction formulas presented in SEBJ(1997). Hence, the prediction formulas and associated methodology in SEBJ(1997) apply particularly well to the La Grande Complex reservoirs and may at least reasonably apply to reservoirs that are large and wide as the La Grande Complex Reservoirs. However, it is uncertain how well the methodology applies to narrow reservoirs. In fact, testing through the conceptual reservoirs the observation and resulting statement by Dupuis et al.(1996) regarding SPM(1984) overpredicting small wave heights and underpredicting large wave heights, reveals that this only applies to reservoirs with shape ratios 0.8 to 1.0. Thus, the methodology presented in SEBJ(1997) may not apply well to narrow reservoirs, unless potentially if these are extremely large and wide as in the case of the Lake Ontario.

- Measurements (average values) from the Megget Reservoir, of length L=3.5 km and with a shape ratio of B/L=0.17 ~0.2, compare reasonably to prediction by SPM(1984) calculated through the conceptual reservoir of length L=3.5 km and with B/L=0.2. Prediction by ICE(2015) can also be considered to compare reasonable to the measurements, although not as well as the predictions by SPM(1984). Other prediction methods, used through the conceptual reservoir, underestimate the measurements (average values).

- Measurements (average values) from Loch Glascarnoch, of length L=7 km and with a shape ratio of B/L~0.1, compare reasonably to predictions by ICE(2015) and SPM(1984) calculated through the conceptual reservoir of length 7 km and with B/L=0.2. The measurements (average values) fall between the prediction by these two methods. Other prediction methods, used through the conceptual reservoir, underestimate the measurements (average values).

- Rough exercise for a reservoir in Norway, involves back calculation that bases on a single observation at the Suoikktjávri reservoir (L=10.4 km and B/L=0.06). The exercise indicates that predictions by ICE(2015) reasonably agrees with the observations, whereas other prediction methods result in a lower wave height compared to the back calculated value. Underprediction of the other methods (not including the extended fetches), range from approximately 50% to 70% of the back calculated value. For example, prediction by SEBJ(1997) is only about 50% to 60% of the back calculated wave height value.

- The significant wave height predictions carried out for the Nesjøen Reservoir using the different methodologies, demonstrate that there is a wide range in the predicted values depending on methodology. The quality of the method used for predictions cannot be assessed in the absence of measurements or observations to compare with.

The effect of the wind velocity in the predictions of the significant wave height was compared for the methods studied. The comparison revealed that the effect of the wind velocity is most profound with the SEBJ(1997) prediction, whereas this is least profound with the ICE(2015). The other methods fall between these two. A direct numerical example is provided with the case of the Nesjøen Reservoir. For example, using the SEBJ(1997) a 30% increase in wind velocity from 22.9 m/s to 30 m/s for the 50 year wind results in increase of the significant wave height of about 44%. Conversely, using ICE(2015) which has a linear relationship between the wind velocity and the significant wave height, a 22% increase in the wind velocity from 24.5 to 30 m/s for the 50 year wind results in about 22% increase in the significant wave height. These increases can also be taken as the increase in applying wind predictions from the Eurocode for the case considered to the fixed wind velocity of 30 m/s provided by NVE.

It is impossible to conclude which formula gives the best prediction in the absent of measurements. Measurements of wind and waves on Norwegian mountain reservoirs is needed for verification and improved prediction.

5 Freeboard, wave runup and overtopping

5.1 Introduction

Waves that are generated on a reservoir by wind blowing in any direction towards a dam will run up the dam upstream face and may overtop the dam if the freeboard is insufficient. The freeboard (*f*) is the vertical distance from the still-water level (*h*) to the dam crest (see Figure 5.1). Embankment dams are vulnerable to overtopping, hence are built with freeboard to protect them from overtopping as a result of wind-generate wave runup and wind setup. Generally, such as mentioned in USBR (2012), some splashover or spray by occasional waves under extreme conditions may be allowed if such occurrences are of a magnitudes and durations that do not threaten the safety of the dam. Furthermore, ICE (2015) accounts for that a portion of the waves may overtop the dam and provides limits to mean overtopping discharges in line with prediction methods of the EurOtop Manual (Pullen et al., 2007) (Note latest version of the EurOtop Manual is from 2018).

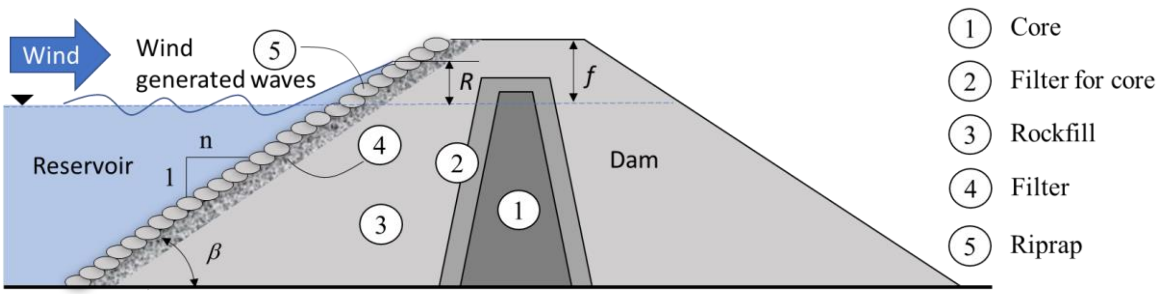


Figure 5.1 Schematic figure of a wave runup on a dam (riprap protection on the dam crest and downstream slope not shown). In the figure: f is the freeboard and R is the runup height.

5.1.1 Requirements in dam safety guidelines

The freeboard of an embankment dam is generally evaluated for normal and extreme conditions and the crest level set so that the dam is protected against the most critical outcome of design situations such as the following, however, usually considering also specified minimum freeboard requirements:

- The reservoir elevation at the full supply level (or the normal water level), wind setup and waves generated by a wind with either a large return period, often the

1 in 1000 years wind, or with a predefined value (e.g. 30 m/s in the Norwegian regulations (NVE, 2003), and 45 m/s (100 mile-per-hour) in USBR (2012)).

- The reservoir elevation at the design flood level, wind setup and waves generated by a wind with considerably lower return period than in the previous case. The Norwegian regulations (NVE, 2003) specify a 10 minute mean wind with a return period of 50 years, (2% change of being exceeded in any one year) or a value of 30 m/s, while for example USBR (2012) specifies hourly winds with return period of 10 years (10% change of being exceeded in any one year).

In Norway the same criteria applies to new and existing dams, while for example USBR (2012) provides different criteria for existing dams.

Some regulations, such as the Norwegian regulations (Damsikkerhetforskriften) (OED, 2010), require that no overtopping should occur for the defined design situations, which in practice becomes no overtopping of 99% of the waves (NVE, 2003). Conversely, dam safety guidelines by the Canadian Dam Association (CDA, 2007) specify "no overtopping by 95% of the waves" for the relevant design situations defined in the guidelines. Similarly, in USA (USBR, 2012) wave height statistics are to be used to compute wave runup. USBR (2012) gives examples: a) For cases where e.g. the crest and downstream slope are adequately protected against erosion, or when public traffic will not be interrupted, a wave height equal to the average height of the highest 10 percent of the waves (1.27 times the height of the significant wave) should be used to compute the runup. b) For cases where only overtopping of infrequent waves is permissible, the highest 1 percent of the waves (1.67 times the height of the significant wave) should be considered.

Hence, wind generated wave runup and setup are the two primary factors in freeboard analysis, while discharge may also be considered. In the following the freeboard evaluation in four guidelines will be reviewed and the associated formula and criteria for either wave runup or overtopping dischargers compared. Formulas for the wind setup are not investigated here. First the run-up approach is discussed, followed by the discharge approach and finally a comparison of methods considering the conceptual reservoir.

5.2 Wave runup approach

5.2.1 General

The wave run-up height, R_u, is measured vertically from the still water line to the wave run-up level. $R_{u2\%}$ denotes the runup height which is exceeded by 2% of the number of incident waves, this is generally used for coastal structures (see e.g. (van der Meer et al., 2018)). The number of waves exceeding this level is generally related to the number of incoming waves. The wave runup has mainly been investigated for coastal structures and results from this has been incorporated into the freeboard design of dams.

Run-up is relevant for embankment slopes. Wave run-up does not have an equivalent parameter for vertical structures. The percentage or number of overtopping waves and consideration of discharges, however, is relevant for each type of structure.

The wave runup approaches considered here are those given in guidelines and regulations in Norway, Canada and the USA. (In the UK overtopping discharges are considered as discussed later). The wave runup equation from these countries are provided below. However, first some general expression for the runup are reviewed.

5.2.2 Expressions for the runup

A general formula for the wave runup has previously been expressed as (see e.g. (van der Meer et al., 1993) and (SEBJ, 1997)):

$$R_u = a\xi_m H_s \qquad \text{for } \xi_m \leq 1.5 \qquad \textbf{(5.1)}$$

$$R_u = b\xi_m^c H_s \qquad \text{for } \xi_m \geq 1.5 \qquad \textbf{(5.2)}$$

where R_u is the runup (vertical), *a, b* and *c* are constant, H_s is the significant wave at the toe of the structure, ξ_m is the surf similarity parameter or breaker parameter based on the mean wave period T_m and expressed as follows:

$$\xi_m = \frac{\tan\alpha_u}{\sqrt{s_m}} = \frac{1}{n\sqrt{s_m}} \qquad \textbf{(5.3)}$$

where α_u is the upstream slope angle, *n* is the horizontal component of the slope (i.e. tan α_u =1/*n*) and s_m is the wave steepness expressed as:

$$s_m = \sqrt{\frac{2\pi H_s}{gT_m^2}} = \sqrt{\frac{H_s}{L_m}} \qquad \textbf{(5.4)}$$

where L_m is the deep wave length $L_m = gT^2/2\pi$.

Values of the required constants *a, b* and *c* in the equations above have been assessed through numerous experimental tests (see e.g van der Meer et al., (1993)) as well as wave measurements on reservoirs (SEBJ, 1997). The constants will depend on the steepness of the slope as well as the risk of exceedance and can be found for example in a table in SEBJ (1997), reproduced in Table 5.1.

Summarizing the above, and considering a roughness factor m_r to account for the roughness of the upstream slope, the equation for the run-up height can be expressed with equations of the following form:

$$R_u = m_r \frac{a}{\sqrt{s_m}} \frac{H_s}{n} = m_r \frac{C_a H_s}{n} \qquad \text{for } \xi_m \leq 1.5 \qquad \textbf{(5.5)}$$

$$R_u = m_r \frac{b}{s_m^{c/2}} \frac{H_s}{n^c} = m_r \frac{C_b H_s}{n^c} \qquad \text{for } \xi_m \geq 1.5 \qquad \textbf{(5.6)}$$

Table 5.1 Risk of exceedance and corresponding coefficients

	Coefficients		
Risk of exceedance %	a	b	c
0.10 %	1.12	1.34	0.55
1 %	1.01	1.24	0.48
2 %	0.96	1.17	0.46
5 %	0.86	1.05	0.44
10 %	0.77	0.94	0.42
33 %	0.72	0.88	0.41
50 %	0.47	0.6	0.34

5.2.3 R_u in the SEBJ (1997) guidelines (Canada)

In SEBJ (1997) a 5% risk of exceedance is deemed acceptable for the design conditions. Thus the coefficients to be used in Eq 5.5 and 5.6 are 0.86, 1.05 and 0.44. for respectively *a*, *b* and *c*. Furthermore, SEBJ (1997) uses s_m =0.06 based on their measurements at La Grande Complex in Canada. This is also the risk considered in the Canadian dam safety guidelines (CDA, 2007).

Two equations are provided for the design runup, depending on the inclination of the upstream slope:

1. The design runup for an upstream slope of riprap with inclination steeper than 1 horizontal to 2.7 horizontal, can be expressed as follows:

$$R_u = \frac{1.95 H_s}{n^{0.44}} \qquad [\text{m}] \qquad \textbf{(5.7)}$$

   where R_u is the runup (vertical), n is the horizontal component of the upstream slope of the dam and should be less than 2.7, and H_s is the significant wave height from the SEBJ (1997) formulation.

2. The design runup for an upstream slope with inclination less steep than 1 horizontal to 2.7 horizontal, can be expressed as follows:

$$R_u = \frac{3.5 H_s}{n} \qquad [\text{m}] \qquad \textbf{(5.8)}$$

   where n is the horizontal part of the upstream slope of the dam and n>2.7.

5.2.4 Norway (NVE (2003))

In NVE (2003) the design runup is said to correspond to design event with 1% likelihood of being exceeded, thus the coefficients to be used in Eq 5.5 and 5.6 are 1.01, 1.24 and 0.48 for respectively *a*, *b* and *c*. (***However, NVE (2003) uses 0.44 for c in the power of the slope n without explaining that further***). Furthermore, investigation into the resulting formulas in NVE (2003) suggests that s_m =0.06 is used.

Thus, two equations are provided for the design runup, depending on the inclination of the upstream slope.

1. The design runup for an upstream slope with inclination steeper than 1 horizontal to 2.7 horizontal, can be expressed as follows:

$$R_u = m_r \frac{2.4 H_s}{n^{0.44}} \quad \text{[m]} \qquad \textbf{(5.9)}$$

   where R_u is the runup (vertical), *n* is the horizontal component of the upstream slope of the dam and should be less than 2.7, m_r is the roughness factor for the upstream slope and H_s is the significant wave height for which the runup is to be estimated using the SEBJ (1997) formulation. For placed riprap m_r =1.0, values of m_r for other materials is provided in a table reproduced in Table 5.2. NVE (2003) refers to the Rock Manual from 1995 for these values in Table 5.2., that can be used in the absence of other documented values.

2. The design runup for an upstream slope with inclination less steep than 1 horizontal to 2.7 horizontal, can be expressed as follows:

$$R_u = m_r \frac{4.1 H_s}{n} \quad \text{[m]} \qquad \textbf{(5.10)}$$

   where *n* is the horizontal part of the upstream slope of the dam and *n*>2.7.

Runups of waves that are not normal to the dam axis should be reduced according to the following equation, where β_{damaxis} is the angle between the wind- (and here also the wave-) direction and the dam axis:

$$R_{u,reduced} = R_u \sin \beta_{\text{damaxis}} \qquad \textbf{(5.11)}$$

Table 5.2 Roughness factors m_r for the upstream slope in (NVE, 2003) runup equation.

Upstream slope, surface	**Roughness factor, m_r**
Smooth (concrete, asphalt, etc)	1.8-2.0
Rough (rough/uneven concrete, bricks, etc)	1.6-1.8
One layer of stones with an impervious layer underneath.	1-1.2
Placed riprap, rounded stones	1.3-1.4
Placed riprap, blasted stones	1.2

5.2.5 USA (USBR (2012))

The US guidelines, USBR(2012), present the following general expressions for the wave runup resulting in a runup height in feets (when using the coefficient of USBR(2012)).

$$R = \gamma_r \gamma_b \gamma_h \gamma_\beta \left(A\xi_p + C \right) H_s \qquad \textbf{(5.12)}$$

where ξ_p is the surf similarity defined for the peak wave heights, *A* and *C* are coefficients depending on ξ_p and the probability of the runup (% risk of exceeding the calculated value). Furthermore, $\gamma_r \gamma_b \gamma_h \gamma_\beta$ are reductions factor as follows: γ_r is the surface roughness reduction factor, γ_b is a reduction factor for the influence of a berm, γ_h is a reduction factor for the influence of shallow-water conditions, γ_β accounts for reduction in runup due to the direction of the fetch relative to the dam axis and is read from a figure (see Figure 5.2).

The values suggested for surface roughness reduction factor, γ_r, are for example for riprap: 0.55 -0.6 for one layer of rock and 0.5 to 0.55 for two or more layers of rock (USBR, 2012a).

In the guidelines USBR (2012a) 2% risk of exceedance is deemed acceptable for the design conditions. For this case the coefficients A and C are 1.6 and 0 respectively. Furthermore, for runup calculations on most embankment dam freeboard analysis, γ_b and γ_h are set equal to 1.0. The guidelines refer to USACE EM 1110-2-1100 (Part VI) in case there is a berm on the upstream slope for the relevant reduction factor. Most embankment dams will not be subject to shallow water condition.

Hence, considering this, the expression for the runup (with 2% risk of exceedance) can be expressed as follows for most embankment dam slopes:

$$R = \gamma_r \gamma_\beta \left(1.6\xi_p \right) H_s \qquad \textbf{(5.13)}$$

Thus in addition to the reduction factors, an expression for ξ_p is required:

$$\xi_p = \frac{2.26T \tan\alpha}{\sqrt{H_s}} = \frac{2.26\,T}{n\sqrt{H_s}} \qquad \textbf{(5.14)}$$

Where *T* and H_s can be obtained from the formulas given from USB (2012a) in Table 4.1, and *n* is as before the horizontal component of the upstream slope inclination. The equation is only valid for dam slopes of 1 vertical to 5 horizontal (1:5) or steeper.

- It should be noted that USBR, (2012a) has the same formula for R for mild slopes (1(V):5(H)) to steep slopes , such as 1(V): 1.5(H). Other guidelines, such as SEBJ (1997), provide formulas that account for that the runup is less on very steep slope.

Table 5.3 Roughness reduction factors γ_r (Valid for $1 < \xi_p < 3\text{-}4$)(from USBR, (2012a)).

Type of slope surface	γ_r
Smooth, concrete, asphalt	1.0
Smooth block revetment	1.0
Grass (3 centimeters in length)	0.90 – 1.0
One layer of rock, diameter *D*, (H_s/D = 1.5 – 3.0)	0.55 – 0.6
Two or more layers of rock, (H_s/D = 1.5 – 6.0)	0.50 – 0.55

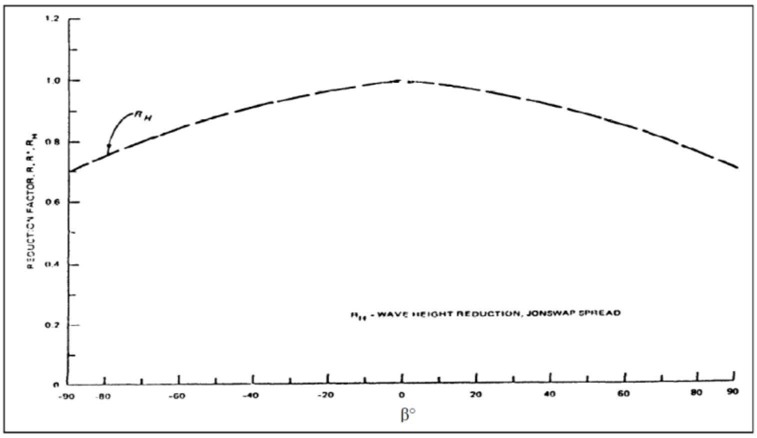


Figure 5.2 Wave height reduction, γ_β due to angular spread. Figure from USBR, (2012a).

5.2.6 The EurOtop Manual

Probably the most recent relevant publication on waves and overtopping is the EurOtop Manual (van der Meer et al., 2018) for coastal structures. The following equation is provided for $R_{u2\%}$ considering steep slopes (n<2) :

$$R_{u2\%} = (0.8n + 1.6)\,H_{m0} \qquad 1.8\,H_{m0} \leq R_{u2\%} \leq 3\,H_{m0} \qquad \textbf{(5.15)}$$

where H_{m0} is the estimate of significant wave height from spectral analysis, other symbols the same. ($R_{u2\%}$ denotes the runup height which is exceeded by 2% of the number of incident waves, this is generally used for coastal structures (see e.g. (van der Meer et al., 2018)).) No influence factors, e.g. for roughness, are in the above equation, while EurOtop includes influence factors in a similar equation provided for milder slopes (e.g. influence factors for roughness, oblique waves, currents, composite slopes and berms, and wave or storm walls).

5.3 Overtopping discharge approach

ICE (2015) considers that a portion of the waves running up the dam slope will overtop the structure. Furthermore, that while the overtopping discharge over any chosen length of the dam face will be variable in time, the mean overtopping discharge (for example over 1000 waves) can be reliably predicted for the general structure configurations using overtopping prediction methods of the EurOtop manual (Pullen et al., 2007). ICE (2015) explains that the mean overtopping discharge so calculated will be substantially lower than any peak discharge and emphasizes that this must never be taken as an estimate of an equivalent steady state discharge. ICE (2015) suggests limits to the mean overtopping discharge that already include the natural difference between mean and peak discharges.

5.3.1 Equations for mean overtopping discharges (ICE, 2015)

Wave overtopping discharges depend on the dam structure configuration, the upstream face roughness, the freeboard and the wave condition. The freeboard in ICE (2015) formulation is denoted, R_c, and is as before the vertical height difference between the structure crest level and the water level. ICE (2015) provides prediction equations for vertical and steep walls, as well as embankment slopes (including small wave wall at top of slope).

Vertical and steep walls

ICE (2015) explains that for dams where the main control on overtopping is given by a vertical or very steep face, the mean overtopping discharge may be predicted by the following:

$$\frac{q}{\sqrt{gH_s^3}} = 0.04\exp\left(-2.6\frac{R_c}{H_s}\right), \quad 0.1 < \frac{R_c}{H_s} < 3.5 \qquad \textbf{(5.16)}$$

where q is the mean overtopping discharge in m^3/s per m along the dam crest.

Embankment slopes

ICE (2015) also provides a similar equation for embankment slopes, including or excluding small wave wall at the top of the slope, expressed as follows:

$$\frac{q}{\sqrt{gH_s^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1.0}\exp\left(-4.75\frac{R_c}{\xi_{m-1.0}H_s\gamma_b\gamma_f\gamma_\beta\gamma_v}\right) \qquad \textbf{(5.17)}$$

with a maximum of

$$\frac{q}{\sqrt{gH_s^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_s\gamma_f\gamma_\beta}\right) \qquad \textbf{(5.18)}$$

The prediction of the mean overtopping discharges on embankment slopes from the above equations requires the use of the breaker parameter, expressed as follows in ICE (2015) (the expression in the parenthesis is added here):

$$\xi_{m-1.0} = \frac{\tan\alpha_u}{\sqrt{\frac{H_s}{L_{m-1.0}}}} \qquad \left(= \frac{1}{n\sqrt{\frac{H_s}{L_{m-1.0}}}} \right) \tag{5.19}$$

where $L_{m-1.0}$ is the deep wave length and $L_m = gT_{m-1}{}^2/2\pi$, where T_{m-1} is given as T_{m-1} /1.1, and T_p is originating from ICE (2015) is given in Table 4.1.

Furthermore, values of the influence parameters γ_b, γ_f, γ_β and γ_v are required, where γ_b is the influence factor for a berm, taken as 1.0 on a simple slope; γ_f is the roughness factor, γ_β is the influence factor for wave obliquity, taken as 1.0 for normal wave attach; and γ_v is the influence factor for a small wall on the slope values of which are provided by a formula in ICE (2015), but for slopes with no wave wall γ_v is taken as 1.0.

Given the acceptable mean overtopping discharge value, the required freeboard can be isolated from the equations above for the mean overtopping discharges. The required freeboard can be considered equivalent to the wave runup height.

5.3.2 Acceptable mean overtopping discharges

ICE (2015) provides guidance on acceptable mean wave overtopping discharges considering the protection provided by the surface material on the crest and downstream slope. The suggested limits are provided in a table that is partly reproduced in Table 5.4. However, information is missing on earth-rockfill dams, and dams with riprap protection downstream.

The EurOtop (van der Meer et al., 2018) suggest limits for rubble mound breakwaters (which includes riprap) and other materials as given in Table 5.5. For rubber mound breakwaters, a limit of 1 l/s per m is given if no damage is to be expected, while a limit of 5 to 10 l/s is given if the breakwater is protected on the downstream side.

The EurOtop (van der Meer et al., 2018) further classifies the mean overtopping discharge as follows:

- $q < 0.1$ l/s per m: Insignificant with respect to strength of crest and rear of a structure.
- $q = 1$ l/s per m: On crest and landward slopes bad grass covers or clay may start to erode. It will not give erosion to rubble mound structures.
- $q = 10$ l/s per m: Significant overtopping for dikes, embankments. For large wave heights it may lead to severe erosion on the harbour side of rubble mound breakwaters.
- $q = 100$ l/s per m: Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated and the armour should cover crest and landward slope.

Table 5.4 Suggested limits for allowable mean wave overtopping discharge on embankment dams (from ICE (2015))

Protection level provided	Allowable mean overtopping discharge q: l/s/m
Dam crest and downstream face of good grass-covered clay fill	1
Dam crest and downstream face are of bare clay fill or grass covered erodible fill, or poor grass cover	0.1

Table 5.5 EurOtop suggested limits for allowable mean wave overtopping discharge for structural design of breakwaters, seawalls, dikes and dams. Considering also hazard type (size of wave) (from EurOtop (van der Meer et al., 2018))

Hazard type and reason	Allowable mean overtopping discharge q: l/s/m
Rubble mound breakwaters; H_{m0}>5m; No damage	1
Rubble mound breakwaters; H_{m0}>5m; rear side designed for wave overtopping	5-10
Grass covered crest and landward slope; maintained and closed grass cover, H_{m0}=1-3 m;	5
Grass covered crest and landward slope; not maintained cover, open spots, moss, bare patches, H_{m0}=0.5-3 m;	0.1
Grass covered crest and landward slope; H_{m0}< 1 m;	5-10
Grass covered crest and landward slope; H_{m0}< 0.3 m;	No limit

The EurOtop (van der Meer et al., 2018) further points out that it is not only the average overtopping discharge that classifies the severity of overtopping, but also the height of the wave that causes the overtopping. A large wave height gives more severe overtopping than a low wave height, for the same overtopping discharge. The wave height is considered in Table 5.5.

The limits for the acceptable mean overtopping discharges in Table 5.4. and Table 5.5 are based on assessments on how much each type of surface layer/protection will tolerate. However, both ICE (2015) and EurOtop (van der Meer et al., 2018) also give guidance on limits based on hazard potential considering e.g. traffic on the structure. For example, ICE (2015) explains that the safety of people accessing the dam crest during periods of wave overtopping should be considered, and refers to 2007 version of the EurOtop (Pullen et al., 2007), which was replaced with the 2018 issue (van der Meer et al., 2018). A similar guidance from the new EurOtop manual is in Table 5.6.

Table 5.6 EurOtop suggested limits for overtopping for people and vehicles (copied from EurOtop (van der Meer et al., 2018))

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea.		
H_{m0} = 3 m	0.3	600
H_{m0} = 2 m	1	600
H_{m0} = 1 m	10-20	600
H_{m0} < 0.5 m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest		
H_{m0} = 3 m	<5	2000
H_{m0} = 2 m	10-20	2000
H_{m0} = 1 m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

5.3.3 Acceptable discharges related to acceptable runup heights

The limits for the acceptable mean overtopping discharges in Table 5.4. and Table 5.5 are based on assessments on how much each type of surface layer/protection will tolerate, while when using runup heights one considers that only a certain percentage of the design waves is allowed to splash over the dam (i.e. no consideration of what the dam may tolerate). Dam safety guidelines, such as USBR, (2012a) specify a 2% risk of exceedance as acceptable for the design conditions, and thus uses $R_{u2\%}$ when defining the freeboard.

The EurOtop (van der Meer et al., 2018) (page 111) assumes that an overtopping discharge of 1 l/s per m would be quite close to a run-up level of 2%, and thus EurOtop takes the required crest level for an overtopping discharge of 1 l/s per m as $R_{u2\%}$.

- For comparison of runup height approaches and overtopping discharge approaches it is reasonable to assume as EurOtop, and take the required crest level for an overtopping discharge of 1 l/s per m as $R_{u2\%}$.
- Obviously as the allowable mean discharge increases the required freeboard will be lower than obtained for 1 l/s per m.

5.3.4 Norway. Design discharges for the downstream slope

The downstream slope of embankment dams in Norway is to be designed to withstand 300 to 500 l/s/m or more, depending on the consequence class. These overtopping discharges can arise from other actions than overtopping of waves, e.g. from extreme flood events or accidents. Still, in theory the freeboard design from wave overtopping could take this into consideration. For example, the EurOtop manual allows overtopping discharges of 5-10 l/s/m of large waves (H_{m0}>5 m) for rubble mound breakwaters with downstream side (rear side) designed for wave overtopping discharge (Table 5.5), however with stricter requirement depending on the hazard posed to others (Table 5.6). But then again, the strict requirement in the Norwegian dam safety regulations of no-overtopping, makes the above mentioned theoretical possibility irrelevant for practical considerations of the run-up heights and freeboard.

5.4 Comparison using the conceptual reservoir

The wave run up height from the different methods, is plotted on the following figures (Figure 5.3 to Figure 5.6) for the conceptual reservoir, again the method that results in the highest wave runup will overlay the other surfaces. In calculating the wave runup heights according to a specific guideline, the significant wave height calculated according to the same guideline is used, embracing also the associated procedure for determining the fetch in each case. For all the cases calculated an upstream slope of 1:1.5 is used, i.e. n=1.5.

Figure 5.3 compares the wave runup heights using the formulation recommended in ICE (2015), USBR (2012), NVE (2003) and SEBJ (1997). In calculating the runup heights according to ICE (2015) and USBR (2012), roughness reduction factor of 0.55 for riprap is used, while a roughness correction factor of 1.3 is used when calculating the runup according to NVE (2003) and SEBJ (1997). ICE (2015) uses the allowable discharge approach, and in the comparison the runup heights (the freeboard) are calculated considering q= 1 l/s per m, which corresponds to 2% risk of exceedance according to EurOtop (van der Meer et al., 2018). Conversely, NVE (2003) consider 1% risk of exceedance and the formulation bases on SEBJ (1997). (Note: the coefficient for the power of n, c, is not the one derived for 1% risk of exceedance and NVE (2003) does not explain why a different c is selected). Both SEBJ (1997) and USBR (2012) recommend the runup height that has 2% risk of exceedance.

Figure 5.3 demonstrates that NVE (2003) (with the strictest requirement 1% risk of exceedance) generally results in the highest runup heights, and always for the wind velocities that would be used for the design situations that need to be inspected. Excluding NVE (2003), the runup heights according to USBR (2012) would govern in all cases compared to ICE (2015) and compared to SEBJ (1997), when B/L<0.6. (For B/L≥0.6 USBR (2012) governs SEBJ (1997) for most cases except the highest velocities and the shortest reservoirs.)

NVE (2003) bases on the formulation provided in SEBJ (1997). However, SEBJ (1997) considers runup heights with 2% risk of exceedance while NVE (2003) is stricter with requirements for 1% of exceedance. Figure 5.4 compares runup heights according to SEBJ (1997), considering 2% risk of exceedance and roughness correction factor of 1.3, to runup heights according to ICE (2015) (the same runup heights as plotted in). Figure 5.4, with similar measure of the risk of exceedance for both methods shown, presents a similar picture of the runup heights as previously presented for the significant wave height. In other words, the runup heights according to ICE (2015) govern for the narrowest reservoirs (B/L=0.1), and also for B/L=0.2 in case of reservoirs longer than 5 km, and partly for reservoirs with shape ratio B/L=0.4. Since ICE (2015) governs in the case of the narrowest reservoirs (B/L=0.1) it will also govern if a lower roughness correction factor (<1.3) was used with the SEBJ (1997) 2% risk of exceedance formula. As the reservoirs become wider (B/L>0.4) a SEBJ (1997) governs over larger parts of the ranges of velocities and reservoir lengths considered.

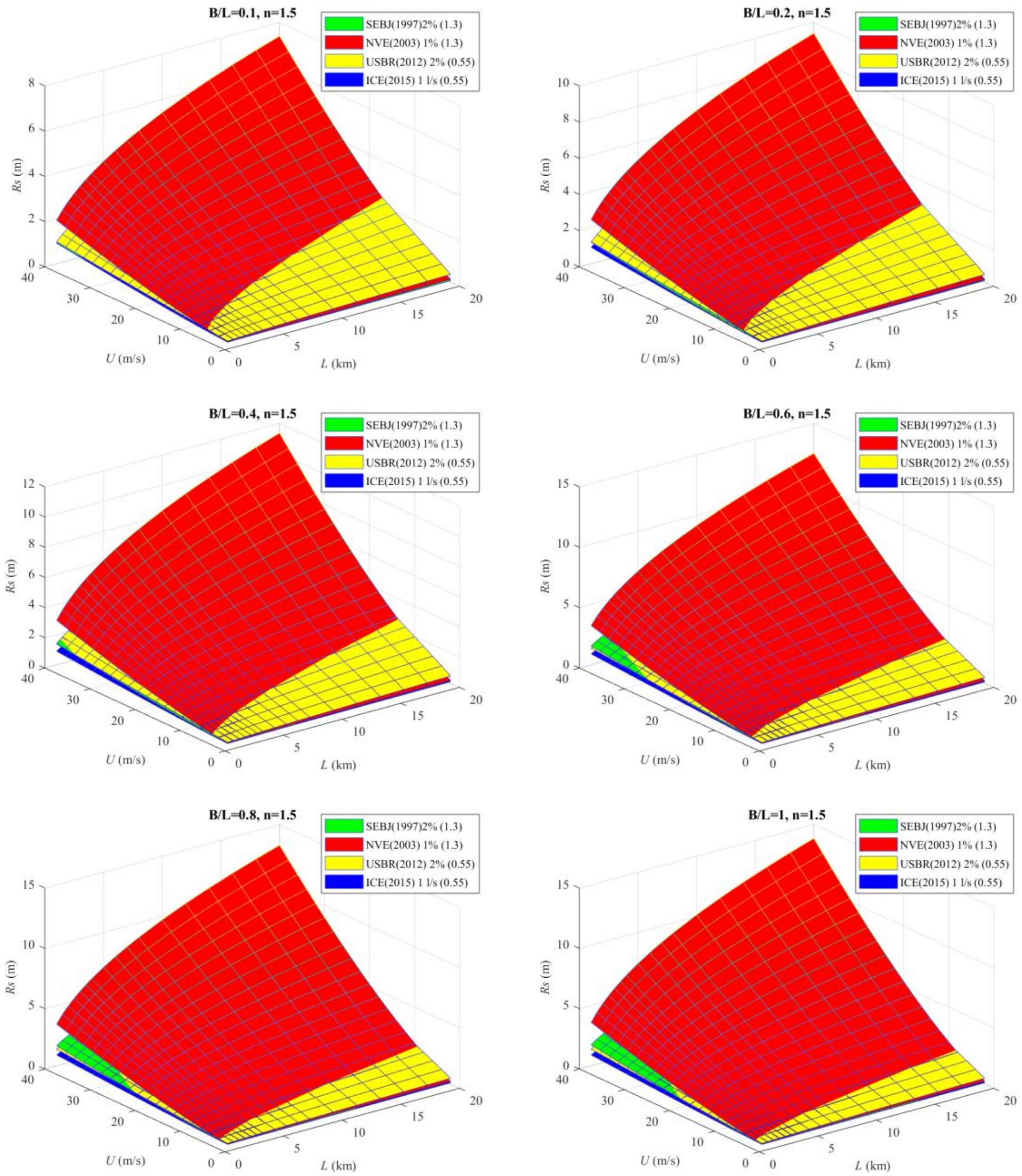


Figure 5.3 Comparison of wave run up heights for the conceptual reservoir using SEBJ(1997) formulas (with roughness factor 1.3) that consider 2% of excedence, NVE(2003) formulas (with roughness factor 1.3) that consider 1% risk of exceedance, formulation from USBR(2012) (with influence factor for riprap of 0.55) which considers 2% risk of exceedance and formulation from ICE(2015) considering q=1 l/s per m (and with influence factor for riprap of 0.55). Significant wave heights associated with each methodology is used.*

It is further interesting to compare the runup height according to NVE (2003) to ICE (2015), as presented in Figure 5.5. Here a roughness factor of 1.0 is used in calculating the runup heights according to NVE (2003).

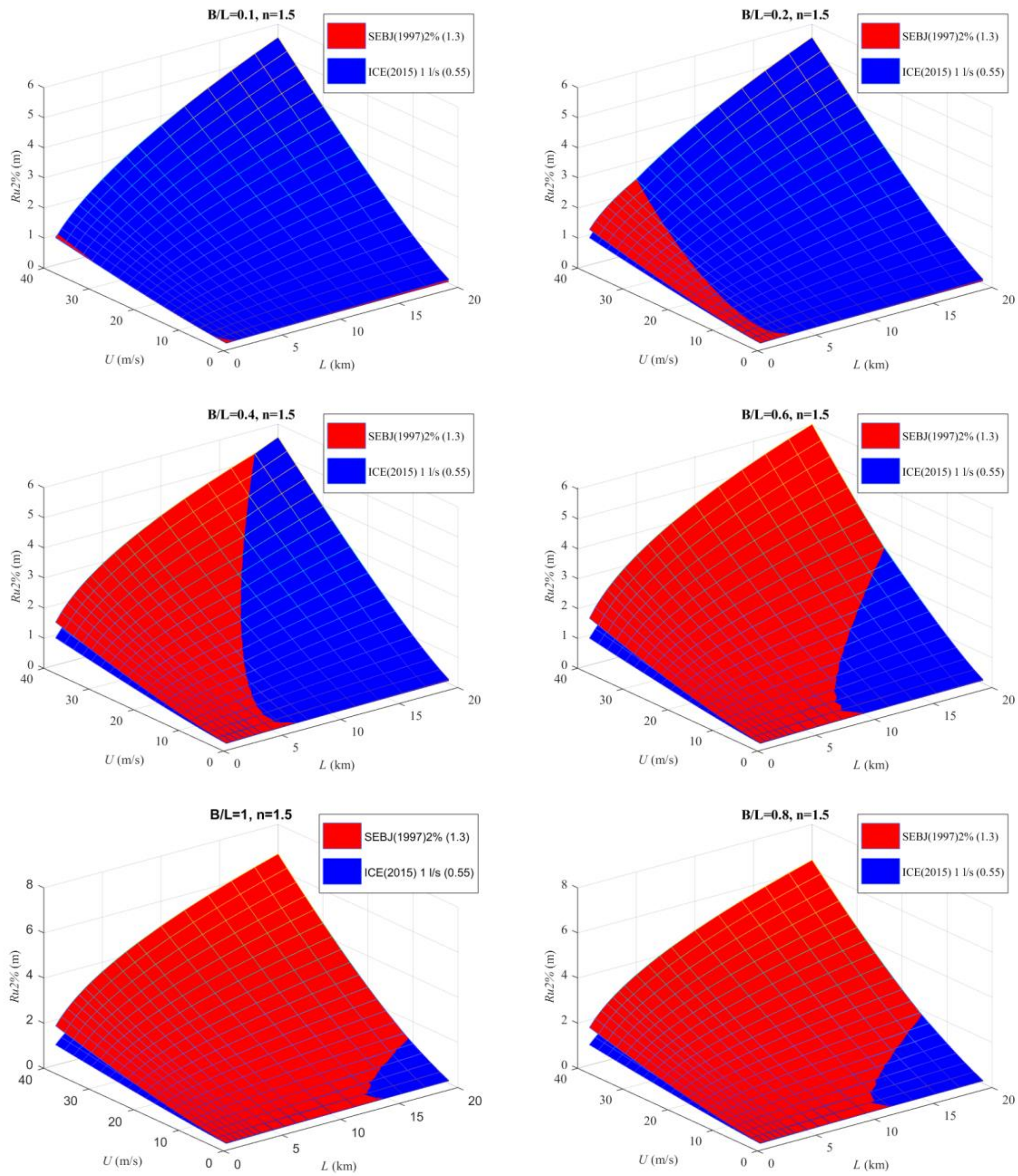


Figure 5.4 Comparison of wave run up heights for the conceptual reservoir using SEBJ(1997) formulas considering 2% risk of exceedance (and asssuming a roughness factor of 1.3 for riprap)and formulation from ICE(2015) considering q=1 l/s per m (and 0.55 influence factor for riprap). Significant wave heights associated with the methodologies is used.

In Figure 5.5, the NVE (2003) runup height governs for all the cases considered and all the reservoir shapes, except in the case of the narrowest reservoirs, B/L=0.1. For the reservoirs with B/L =0.1 the runup heights according to NVE (2003) govern for the highest wind velocities considered over the range of reservoir lengths considered, as well as for the shortest reservoirs and the lowest wind velocities. Figure 5.6, further investigates the runup heights for reservoirs with shape factor B/L=0.1, and other roughness correction factors for NVE (2003) runup heights, i.e. 1.2 and 1.3. In these cases the NVE (2003) runup heights are higher than the once calculated according to ICE (2015).

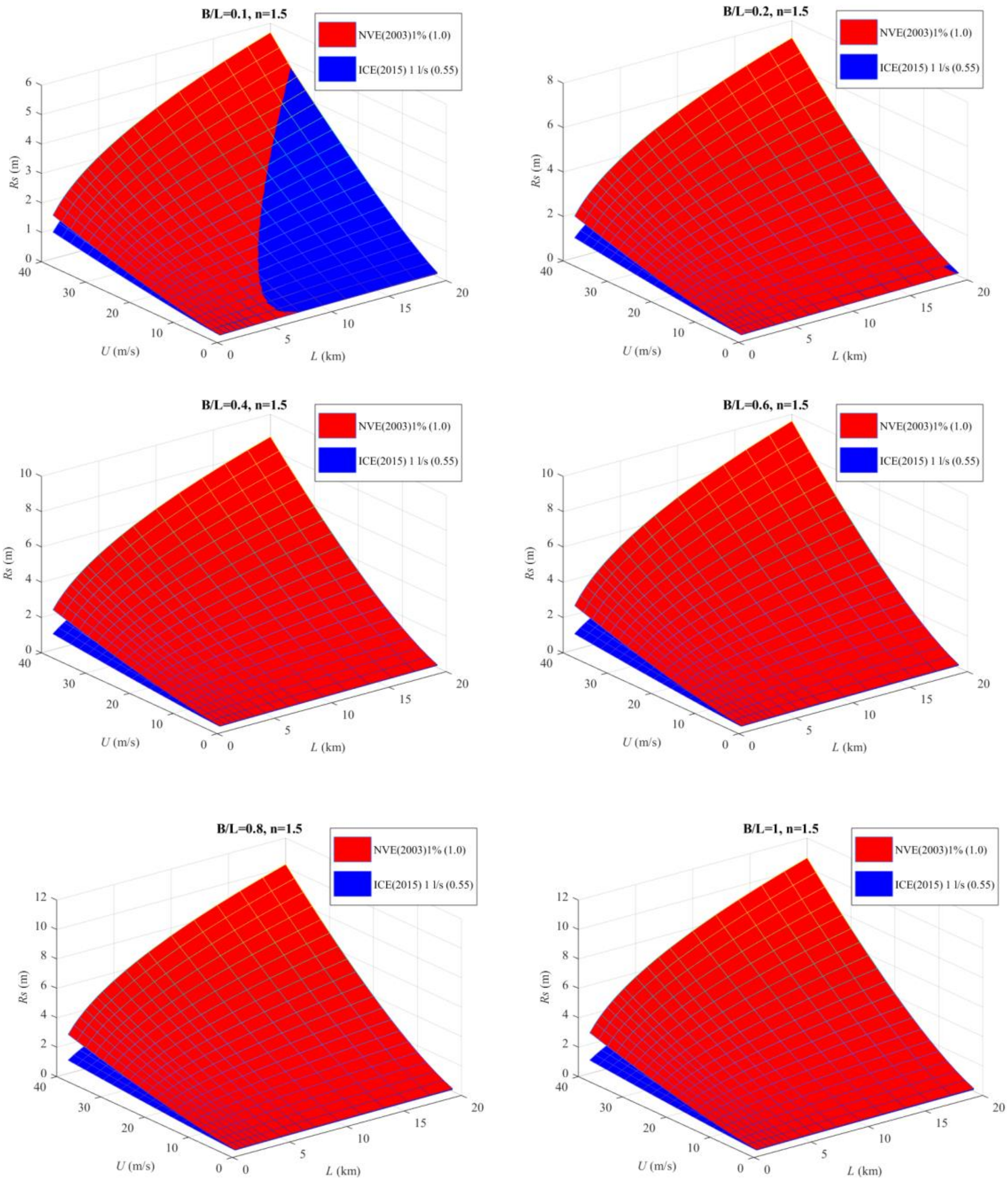
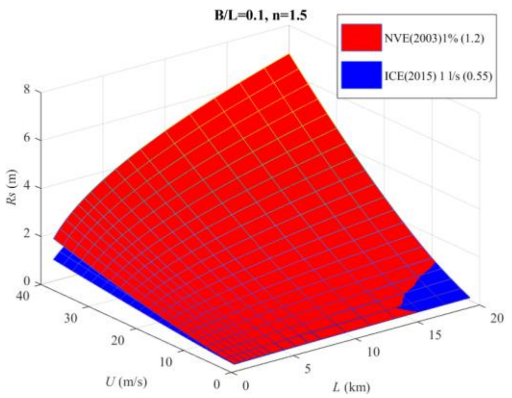
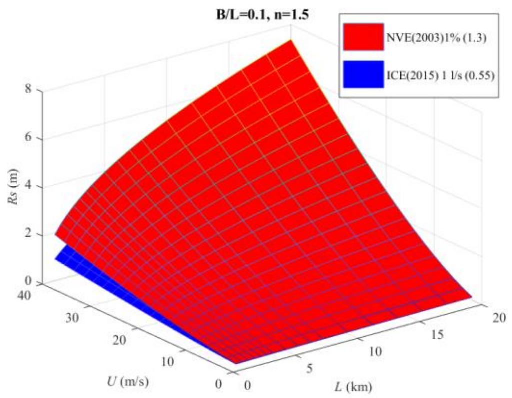


Figure 5.5 Comparison of wave run up heights for the conceptual reservoir using NVE(2003) formulas that consider 1% risk of exceedance (and asssuming a roughness factor of 1.0) and formulation from ICE(2015) considering q=1 l/s per m (and 0.55 influence factor for riprap). Significant wave heights associated with the methodologies is used. (*the NVE formulation does not use the power c on n for 1% risk of exceedance, resulting in that the wave runup is slightly higher than obtained selecting the appropriate coefficients for1% risk of exceedance).*



a



b

Figure 5.6 Comparison of wave run up heights for the conceptual reservoir with B/L=0.1; using NVE(2003) formulas that consider 1% risk of exceedance and formulation from ICE(2015) considering q=1 l/s per m (and 0.55 influence factor for riprap). Significant wave heights associated with the methodologies is used. a) A roughness factor of 1.2 is used in the NVE formulation. b) A roughness factor of 1.3 is used in the NVE formulation.*
*(*the NVE formulation does not use the power c on n for 1% risk of exceedance, resulting in that the wave runup is slightly higher than obtained selecting the appropriate coefficients for1% risk of exceedance).*

5.5 Summary on wave run up heights

In this chapter the freeboard evaluation in four dam engineering guidelines were reviewed and the associated formula and criteria for either wave runup or overtopping discharges compared. The four dam engineering guidelines are: the Norwegian guidelines NVE (2003) , Canadian guidelines SEBJ (1997) (combined with CDA (2007)), the US guidelines USBR(2012), the UK guidelines, ICE (2015). The Norwegian guidelines base on the Canadian guidelines SEBJ (1997), however with stricter requirements on the risk of exceedance. Additionally, the European guidelines EurOtop Manual (van der Meer et al., 2018) for coastal structure were considered in this chapter.

The wave runup approach is considered in the Norwegian, Canadian and US guidelines, while overtopping discharge is considered in the UK guidelines as well as in the EurOtop Manual. The following points summarize main requirements of each method:

- The Norwegian regulations (Damsikkerhetforskriften) (OED, 2010), require that no overtopping occurs for the defined design situations, which in practice becomes no overtopping of 99% of the waves with the given formulation in NVE (2003). (Equivalently, run up heights with 1% risk of exceedance).
- The Canadian Dam Association (CDA, 2007) specify “no overtopping by 95% of the waves” for the relevant design situations defined in the guidelines. This is in line with SEBJ (1997) recommendation of considering the run-up heights with 5% risk of exceedance.

- In USA, USBR (2012), recommend the use of wave height statistics to compute wave runup. USBR (2012) and the requirements are different depending on quality of the downstream erosion protection and/or if traffic is allowed over the dam.
- The European manual EurOtop (van der Meer et al., 2018) for coastal structures considers run-up heights with 2% risk of exceedance. Furthermore, the EurOtop manual assumes that an overtopping discharge of 1 l/s per m would be quite close to a run-up level with 2% risk of exceedance.
- The UK guidelines ICE (2015) considers that a portion of the waves running up the dam slope will overtop the structure, and uses overtopping prediction methods of the EurOtop manual (Pullen et al., 2007). Acceptable mean overtopping discharges consider the protection provided by the surface material on the crest and the downstream slope.

Run-up height calculated according to: NVE(2003), SEBJ(1997), USBR(2012) and ICE(2015); have been compared. Conclusions drawn from the comparison are limited by the following:

- The comparison in this report bases on applying the full methodology of each guideline (fetch estimate, significant wave height and run-up or discharge formula) excluding only modification of the wind velocity. In other words, the comparison is not clear on the direct formulas for calculating only the run-up heights and discharges. The method of estimating the fetch and calculating the significant wave height are a large part of the process and influences the results.
- The different methods recommend somewhat different factors to represent the roughness of the upstream slope surface.

The comparison of the run-up heights calculated according to: NVE(2003), SEBJ(1997), USBR(2012) and ICE(2015); leads to the following conclusions (limited by the points above):

- NVE(2003) has the strictest requirement (1% risk of exceedance). NVE(2003) generally results in the highest value for the run-up height compared to the other methods studied, and always for the ranges of relevant wind velocities. The method of calculating the significant wave height in each case influences the results.
- The NVE formulation does not use the power c for 1% risk of exceedance, resulting in that the wave runup is slightly higher than obtained selecting the appropriate coefficients for 1% risk of exceedance.

For comparison of the run-up height methods and discharge methods, SEBJ(1997) run-up height formulation with 2% risk of exceedance, was compared to the ICE(2015) discharge formulation considering allowable discharge that corresponds to the 2% risk of exceedance criteria, i.e. 1 l/s/m (according to EurOtop). SEBJ(1997) results in higher run-up heights for reservoir shapes in the range $0.6 \leq B/L \leq 1$ for relevant wind velocities. Conversely, ICE(2015) results in higher run-up heights as the reservoir become narrower. The method of calculating the significant wave heights in each case influences the results.

6 Ice load on embankment dams

6.1 Introduction

Ice load on rigid structures has been the subject of many publications, while the effect of ice on the upstream slope of embankment dams, and thus the riprap is considerably less extensive (ICOLD, 1996). Still, research into ice loads on coastal structures have in the last two decades or so also included focus on riprap, such as this of Daly et al., (2008), as well as Sodhi et al., (1997, 1996) and Sodhi and Donnelly (1999). Their work will be discussed in the next chapter, Chapter 7, on riprap design. This chapter only provides a short summary on ice action on embankment slope.

6.2 Ice conditions

Knowledge on the ice conditions as well as the meteorological conditions is required to solve potential problems relating to the interaction of ice with dams and other structures in cold regions (CIRIA et al., 2007). The ice conditions are described e.g. by the ice growth, coverage, movement, features, formations, geometry, properties and strength. Ice growth can be estimated from temperatures with the freezing degree day method, that requires local calibration. The freezing degree day method can be used to generate ice thickness statistics from historical temperature data. Thus, the ice thickness can be derived indirectly from historic temperature data. Similarly, ice drift can be indirectly derived from historic wind data.

There are challenges associated with characterizing the ice conditions since these vary, e.g. with the season and periods of freeze-up and break-up. A good summary on the ice conditions, considering ice growth, ice formation and typical winter ice action is found e.g. in the Rock Manual (CIRIA et al., 2007).

6.3 Ice action on an embankment slope

There are three major types of ice action that can cause degradation and affect the integrity of a riprap on an embankment slope, these have been described as follows (see Matheson (1988) and McDonald (1988))

1. Movement of large area. The displacement of large areas of rock by wind or current driven ice floes. The thrust from the ice sheet against the slope can cause a slider or shear along a horizontal surface through the embankment.

2. Gouging or shoving of individual stones: Large ice floes moving along the shoreline gouges riprap stones from their normal location as they strike the slope, or the ice rides up on the slope and displaces the riprap stones.

3. Plucking: Plucking of individual rocks from the side slope by rising ice levels when the rocks are imbedded in the ice sheet. The ice sheet grips individual stones or groups of riprap stones and, with rising water (and ice) levels, lifts the stones from their bedding. Alternatively, with a falling reservoir level a slab of ice can act as a cantilever and pull the stone out of place.

Additionally, the fourth action that causes degradation can be defined from frost/thaw cycles:

4. Cracking of stones by frost/thaw cycles reducing the stone size.

The calculation of the ice forces depends e.g. on ice thickness, flexural strength, and modulus of elasticity. Detailed assessment of the ice load should be made based on the ice characteristics and wind conditions (wind speed, direction and fetch). The resistance of the structure to the ice action is dependent on the riprap stone sizes, thickness of the riprap layer, and inclination of the embankment slope. Structural response related to ice is for example discussed in CIRIA et al., (2007).

7 Riprap design for waves and ice

7.1 Introduction

The upstream slope of embankment dam is typically protected with a layer of quarry stones, referred to as riprap. The purpose of the riprap is to prevent erosion, scour or sloughing of the embankment. Thus, the riprap must counteract environmental loads, i.e. actions arising from wind generated waves and ice. The resistance of the riprap to these actions relates to the individual stone size, weight and durability. Additionally, gradation and thickness of the riprap layer and the underlying layer,as well as the interlocking between the riprap stones, are important factors.

In dam engineering riprap refers to a protective layer of rock fragments, that is usually well graded within wide size limits (see e.g. USBR (2012b)), but can also be uniformly graded (see e.g. SEBJ (1997)). Furthermore, the word riprap refers also to the stone that is used. Conversely, in the European manual, The Rock Manual- The use of rock in hydraulic engineering, (CIRIA et al., 2007), individual stones in the protective layer are referred to as armour stone or unit , and defined as "*a relatively large quarry stone or concrete block that is selected to fit specified requirements of mass and shape; it is placed in a cover layer*". The cover layer is referred to as the armour layer, defined as the "*outer layer of larger and/or more durable material used as protection against waves and/or currents and/or ice loads*". Furthermore, in the Rock Manual riprap is defined as a "*wide-graded quarry stone normally bulk-placed as a protective layer to prevent erosion of the sea bed and/or river bed, riverbanks or other slopes (possibly including the adjoining crest) by current and/or wave action*".

Here the dam engineering use of the word riprap (see e.g. USBR (2012a) and SEBJ (1997)), is generally applied. However, when the word armourstone is used the definition of the Rock Manual applies (CIRIA et al., 2007).

The sizing of the riprap to resist wave action is well established with simple empirical formulas. In dam engineering manuals and guidelines, such as NVE (2012) (Norway), USBR (2012a) (USA) and SEBJ (1997) (Canada), these empirical formulas are basically the Hudson formula discussed in section 7.4. Conversely, sizing of riprap in cold regions to resist ice action is less understood (CIRIA et al., 2007; ICOLD, 1996) and guidance on the design limited as further discussed in Section 7.5.

CIRIA et al., (2007) present the Hudson formula along with other most common formulas for riprap sizing due to wave action and state that the formulae presented in the manual should be used for the conceptual design of the relevant structures. Furthermore, that the conceptual design should be confirmed and optimised with physical model tests. The model testing is rarely carried out for individual dams, however the recommendations in the Canadian guidelines SEBJ (1997), base on such testing as well as field surveys. Similarly, the recommendations in the US guidlines USBR (2012a) are supported by field surveys carried out on riprap performance, as well as tests carried out in the nineteen-seventies.

The discussion on riprap design for waves and ice in this chapter is organized in the following manner: First some definition of the dimensions of a riprap stone along with gradation are introduced (Section 7.2), followed by damage criterion and damage level (Section 7.3). Then the empirical stone sizing formulas to resist wave action are summarized (Section 7.4), considering the following European, Canadian, Norwegian, Canadian and USA guidelines CIRIA et al., (2007) (Europe), SEBJ (1997) (Canada), NVE (2012) (Norway), and USBR (2012a) (USA). The different definition of wave heights used in the different formulas are discussed as well as different empirical coefficeints and damage criterions, as well as design events. Then riprap sizing for ice action is briefly mentioned (Section 7.5) before comparing the different methods for riprap sizing with numerical examples (Section 7.6).

7.2 Riprap stone dimensions and riprap gradation

The mass of a riprap stone can be accurately determined by weighing. However, often it is convenient to describe the size by the dimensions and back calculate this from the mass and relate to for example square opening of a sieve. This is important, e.g. to be able to draw a gradation curve, as well as determine maximum and minimum dimensions in a riprap layer and relations between this. The riprap layer can be well graded or uniformly (narrowly) graded.

In designing protection comprising of well graded riprap, the median weight of the stones (W_{50}) or median diameter (D_{50}) are generally considered. However, in the design of a riprap protection comprising uniformly graded riprap stones, the minimum weight (W_{min}) of an individual stone is usually defined (see SEBJ (1997) and NVE (2012)), although the median weight (W_{50}) is also used for armourstones (see e.g. (CIRIA et al., 2007)).

Riprap stone dimensions

From the mass of a riprap stone the dimension of an equivalent cube, D_n, known as the nominal diameter, can be used. According to CIRIA et al.(2007) the nominal diameter is used in design, rather than diameter of an equivalent sphere. The relationship between gradings of riprap stones, i.e. the square opening sieve size D, and the nominal diameter has been experimentally determined as (see CIRIA et al.,(2007)):

$$D_n = 0.84\, D \tag{7.1}$$

Using this, the median sieve size D_{50}, the median nominal diameter D_{n50} and the median mass M_{50} and consequently also the median weight W_{50}, are related as follows (see CIRIA et al.,(2007)):

$$M_{50} = \rho_r D_{n50}^3 = 0.6 \rho_r D_{50}^3 \tag{7.2}$$

and thus

$$W_{50} = gM_{50} = g\rho_r D_{n50}^3 = g0.6\rho_r D_{50}^3 = 0.6\gamma_r D_{50}^3 \tag{7.3}$$

where ρ_r is the unit mass of the rock (e.g. kg/m^3), γ_r is the unit weight of the rock (e.g. kN/m^3).

Hence, given the required mass or weight, the median sieve size D_{50} can be obtained from the above expressions. However, first the mass or weight required to resist wave or ice action, etc has to be determined, and in doing so allowable damage to the riprap layer needs to be defined for the given design scenario.

In SEBJ (1997) as well as the current Norwegian guidelines (NVE, 2012) for embankment dams the relationship above is used to obtain the stone diameter from the calculated required weight (see Section 7.3).

$$D = \sqrt[3]{\frac{W}{C_f \gamma_r}} \tag{7.4}$$

Where W is the required weight of the stone in kN, D is the opening of the sieve, γ_r is the unit weight of the rock and C_f is a coefficient of form. C_f can vary between 0.4 to 0.8, with increasing value as the stone becomes more cubical. SEBJ (1997) recommends to use 0.6 (same constant as recommended by CIRIA et al.,(2007)), unless more specific values are available from the quarry.

Uniformly (narrowly) graded riprap

The Canadian riprap sizing guidelines, SEBJ(1997), base on the results of extensive field surveys on the behaviour retaining structures as well as results of scale model tests. Findings from this show that a uniform riprap with a narrow gradation behaves better than a well-graded riprap. The recommended grading in SEBJ (1997), bases on practical observation of what is possible in practice, can be expressed as follows:

$$W_{\max} = 5\, W_{\min} \tag{7.5}$$

The above relation is also recommended in the Norwegian guidelines NVE (2012).

In CIRIA et al.(2007) narrow or single sized gradation is defined for the mass ratio M_{85}/M_{15} (the weight ratio could be equivalently used) to be in the range 1.7 to 2.7, while wide gradation is defined to be in the range 2.7 to 16. Thus the grading recommended in the Canada and Norwegian guidelines fall into the narrowest range of the wide gradation definition of CIRIA et al.(2007).

Well graded riprap

The riprap design according to the US dam guidelines, USBR (2012b), considers riprap gradation basing on the $W_{50,}$ value from the rock sizing formula given in the guidelines. The resulting gradation should result in a well-graded material from the maximum to the minimum size. In other words, the riprap is not uniformly graded as generally on, for example the Norwegian dams. The maximum and minimum weight of the riprap are determined as: $W_{max} = 4\ W_{50,}$ and $W_{min} = W_{50}/8$, where 100% of the rock in the riprap gradation is smaller than W_{max} and approximately 5 to 20% is smaller than W_{min}. To comprehend the range of the stone sizes used in well graded riprap, the USBR (2012a) criteria can be rewritten to compare with Eq (7.5), as follows: $W_{max,USBR} = 32\ W_{min,USBR,}$.

The riprap design according to the US dam guidelines, USBR (2012a), seemingly fall into the CIRIA et al.(2007) armourstone grading of very wide or quarry run gradation, defined with the mass ratio (or equivalently the weight ratio) M_{85}/M_{15} to be in the range 16-125.

7.3 Damage criterion and damage level

The allowable damage level of a riprap layer can be described by allowable differences noted before and after storms or severe ice conditions, e.g. by the number of displaced units or by the development of damage such as differences in the cross section. (see e.g. CIRIA et al.,(2007)).

The damage to the riprap layer can be given as a percentage of displaced stones related to a certain area. The damage percentage D(%) was defined in the Shore Protection Manual from 1984 (SPM, 1984) as: „*The normalized eroded volume in the active zone, from the middle of the crest down to 1Hs below still water level (SWL)*."

CIRIA et al., (2007) explain that acceptable displacement for design purposes, or the „no-damage" condition, corresponds to displacement of 0-5% of the armour stones from the region between the crest and a level of one wave height below still water.

The damage has also been described more clearly by the erosion area around the still water level (see CIRIA et al.,(2007)). This erosion area has been related to the stones size, resulting in the definition of a non-dimensional damage level parameter, S_d

$$S_d = A_e / D_{n50}^2 \tag{7.6}$$

where A_e is the eroded (damaged) area around the still water level (m^2).

SEBJ (1997) defines a damage index (S here replace by S_d) in the same manner as presented above, while the damage area A_e is not limited to around the still water level, but defined as damaged area at section of the protective layer. Furthermore, the nominal size of the stones is not specifically related to the median size.

A physical description of the damage level parameter, S_d, is the number of squares with a side of D_{n50} that fit into the eroded area. S_d is further described in SEBJ (1997) as the theoretical number of stones of cubic form, with a dimension D_n (or D_{n50}), which is eroded from the riprap face, with a strip width equal to D_n.

For example, using the average mass, a damage level parameter S_d = 2 corresponds to a condition of 0-5% damage (van der Meer, 1988), this corresponds to the notion of no damage in the Shore Protection Manual. Furthermore. Furthermore, for S_d = 8, the cushion is exposed in two layer riprap, and S_d = 12, is described as extreme damage. (SEBJ, 1997). For other values see Table 7.1.

The damage levels in Table 7.1 apply for randomly placed armourstone. Damage levels and criterion need to be defined for placed armourstones/riprap.

Table 7.1 Damage D with corresponding damage level S_d *. The damage D is on randomly placed armourstone in two layers and non-breaking waves on the foreshore. (Information in the table is from CIRIA et al.,(2007) and SEBJ (1997))*

Damage D (%)	0-5	5-10	10-15	15-20	20-30	30-40	40-50
$S_{d,\mathrm{CIRIA}}$	2	6	10	14	20	28	36
Damage description	Zero damage (ZD)	Tolerable damage (TD)					
$S_{d,\mathrm{SEBJ}}$	2.5	5					

7.4 Sizing of riprap to resist wave action

Various researchers have developed relationships to express the wave action and the resisting forces offered by the riprap. These relationships aim at prediction the size of riprap (and armourstones). Empirical relationships proposed in the last 80 years have resulted in widely used design methods. Here formulas that base on Hudson (Hudson, 1959, 1953) are reviewed as these are extensively used in dam engineering applications. However, for example the more complex Van der Meer formulae (Van der Meer, 1988) for deep water conditions (see e.g. CIRIA et al., (2007)) is not discussed. The relationships basing on the Hudson formula, that are presented in this chapter, will be studied further with numerical examples in the following section.

The relationship between the response of a sloping structure under wave attack and the wave condition can be described by the stability number, N_s, as follows (CIRIA et al., 2007):

$$N_s = \frac{H}{\Delta D} \le K_1^a K_2^b K_3^c ... \tag{7.7}$$

where, H is the wave height, Δ is the relative buoyant density of the stone $\Delta=(\rho_r / \rho_w - 1)$ with ρ_w being the mass density of water and ρ_r the apparent rock density (kg/m3), and D is the characteristic size or diameter, the median nominal diameter D_{n50} in case of armour riprap stone. The factors on the right-hand side, $K_1^a K_2^b K_3^c ...$ etc, are stability factors that depend on all other parameters influencing the stability.

Riprap structures for which no or minor damage to the riprap layer is allowed under the design conditions, are defined as statically stable. Statically stable structures have stability numbers N_s in the range of 1 to 4 (CIRIA et al., 2007). Conversily, dynamically stable (reshaping) structures are structures that are allowed to be reshaped by wave attack, resulting in a development of their profile. Dynamically stable structures have stability numbers N_s greater than 6. (CIRIA et al., 2007)

The right hand side of Eq. (7.7) has been widely explored and is often rewritten to express the mass or weight of the required armour stone. Formulas, used to determine necessary median weight of a stone (W_{50}) to resist wave action on dams, base e.g. on relationship originally developed by Hudson (Hudson, 1959, 1953). The relationship based on model tests with regular waves on non-overtopped rock structures with a permeable core.

The Hudson formula is presented in SI units according to CIRIA et al., (2007) as follows:

$$W_{50} = \frac{\rho_r g H^3}{K_D \Delta^3 \cot\alpha} \tag{7.8}$$

where W_{50} is the median weight of the armour stone in N , H is the wave height in m at the toe of the structure, K_D is stability coefficient relating e,g. to the damage condition, ρ_r is the apparent rock density (kg/m^3), α is the embankment slope angle, and Δ is the relative buoyant density of the stone $\Delta=(\rho_r / \rho_w - 1)$ with ρ_w being the mass density of water. CIRIA et al., (2007) explain that K_D values suggested for design often correspond to the no damage condition.

In CIRIA et al., (2007) Eq (7.8) is rewritten to represent the stability number (see Eq. (7.7)), with the wave height H replaced by $H = 1.27\ H_s$, as follows:

$$\frac{H_s}{\Delta D_{n50}} = 0.7\left(K_D \cot\alpha\right)^{1/3} S_d^{0.15} \tag{7.9}$$

In dam engineering guidelines, such as NVE (2012), USBR (2012a) and SEBJ (1997), the Hudson formula is however presented in the form similar to this of Eq. (7.8). Furthermore, definition of the wave height used varies, as well as the stability coefficients. These differences, including the different stability coefficients, are discussed in the next section, while the formulas are introduced below.

The Hudson formula in SI units is presented as follows in SEBJ (1997)

$$M_{\min} = \frac{\rho_r H_s^{\ 3}}{K\left(S_r - 1\right)^3 \cot \alpha} \tag{7.10}$$

where $M_{\min}$ is the minimum mass of the stone in kg, H_s is the significant wave height (assumingly as calculated according to SEBJ (1997)), K is experimental stability coefficient the different values are discussed below, ρ_r is the apparent rock density (kg/m^3), α is the slope angle, and S_r is the specific gravity of rock $S_r = \rho_r / \rho_w$.

In the Norwegian Guidelines for embankment dams (NVE, 2012) the Hudson formula in SI units is for presented as follows:

$$W_{\min} = \frac{\gamma_r H_s^{\ 3}}{K\left(\dfrac{\gamma_r}{\gamma_w} - 1\right)^3 n} \tag{7.11}$$

where $W_{\min}$ is the minimum weight of the armour stone in kN, H_s is the significant wave height in m, K is defined as a constant that is dependent on the shape of the armour stone and how it is placed, γ_r is the unit weight of the rock (kN/m^3), γ_w is the unit weight of the water (kN/m^3) set equal to 10 kN/m^3, n is the horizontal component of the inclination of the upstream dam slope, ($\cot\alpha = n$).

In the obsolete Norwegian regulations from 1981 (NVE, 1981), the Hudson formula is presented in similar manner as Eq (7.11), except that the left hand side is the median weight (W_{50}) and instead of the wave height H_s a design wave height H_d is introduced. This is further discussed in the next section. However, although the left-hand side is introduced as the median weight it is explained that this will give the minimum weight on an embankment slope that is protected with uniformly graded riprap.

Finally, the Hudson formula in the US dam engineering guidelines USBR (2012a) is presented as follows, the notation and coefficients given in English Units and thus presented in the same manner here:

$$W_{50} = \frac{\gamma_r H^a}{K\left(G_s - 1\right)^3 (\cot \alpha)^b} \tag{7.12}$$

where W_{50} is the median weight of riprap in pounds, H is the design wave height in feets, γ_r is the unit weight of the rock (pounds per cubic foot), G_s is the specific gravity of rock, α is the slope angle measured from horizontal, K are experimentally determined coefficients and same for the exponents a and b.

The Eqs (7.10) to (7.12), as well as Eq (7.9), will be investigated in Section 7.6. But first the different experimentally determined coefficients will be discussed (see subsection 7.4.2) as well as the different wave heights H used in the formulas (see the next subsection

7.4.1.). Finally, within this section, the design events and damage levels are jointly discussed in subsection 7.4.3.

7.4.1 Different wave heights used in the Hudson formula

Definition of the wave height used into the Hudson formula Eqs (7.10) to (7.12), is not the same in all guidelines discussed here. In some guidelines the wave height is defined as the significant wave height and in others a design wave height is defined. The significant wave height is defined as the highest one third of (~33%) all waves, while the design wave may consider average of higher waves, such as H_{10}, the average of highest 10% of all waves (H_{10}= 1.27 H_s) or H_5, average of highest 5% of all waves (H_5= 1.37 H_s).

CIRIA et al., (2007) discusses the different selections of the wave heights into the Hudson formula (developed for regular waves), and explains for example that in the Shore Protection Manual from 1977 (SPM, 1977) the significant wave height is recommended, while the version from 1984 (SPM, 1984) advises to use H_{10} (= 1.27 H_s) as the design wave height. More accurately the SPM (1984) explains that the design wave height usually ranges from H_5 (= 1.37 H_s) to the significant wave height (H_s) for flexible structures such as rubble-mound or riprap structures. Furthermore, SPM (1984) states that a design wave height H_{10} (= 1.27 H_s) was at the time favoured for most coastal breakwaters or jetties.

Similar can be observed in the development of the Norwegian dam safety guidelines as well as those of the US Army Corps of Engineers (USACE), which has generally based on the Shore Protection Manual.

In the current Norwegian guidelines (NVE, 2012) for embankment dams, the wave height into the Hudson formula (Eq 7.11) is defined as the significant wave height, H_s, calculated according to the NVE (2003) (that bases on SEBJ (1997)), for a wind velocity with a return period for 1000 year or a wind with a velocity of 30 m/s. The SEBJ (1997) guidelines (Practicial Guide-RipRap sizing) also uses the significant wave height according to the formula of the guidelines and recommends to use the 1000 year wind controlled with the 100 year wind, each associated with a different damage level as later discussed.

However, in the obsolete Norwegian Regulations from 1981 (NVE, 1981), the wave height used in the Hudson formula as presented there is defined as this for the design wave, H_d , (essentially H_{10}) defined as follows:

$$H_{d,NVE1981} = 1.3\ H_{s,NVE1981} \tag{7.13}$$

where the significant wave height, $H_{s,NVE1981}$, was determined based on early versions of the SMB-Saville wave predictions read from graphs.

Similarly, the current US dam engineering guidelines USBR (2012a) use the design wave into its version of the Hudson formula. The design wave in USBR (2012a) bases on the significant wave height prediction according to (USBR, 2012a) i.e as presented in Eq.

(3.36), which gives smaller values in comparison with other prediction formulas as studied in Chapter 4. USBR (2012a) defines the design wave as the average height of the largest 10 percent of waves within a wave series, or as:

$$H_{10,USBR} = 1.27\ H_{s,USBR} \tag{7.14}$$

Prior to the guidelines issued in 2012, the USBR used a different formula to calculate the significant wave height (see Eq (3.28)), and used this directly into the Hudson formula for the riprap sizing. However, the new (USBR, 2012a) formulation of the significant wave height results in about 25% smaller heights than the previous formulation. This would in turn result in smaller riprap would the new significant wave height be used in the Hudson formula. Thus the history of riprap sizing on US Reclamation dams was studied along with performance (see Appendix D in USBR (2012a)). The study showed that using the $H_{10,USBR}$ resulted in rirpap sizes expected to perform well and without unnecessary conservatism and undue cost (USBR, 2012b).

As previously noted, CIRIA et al., (2007) has incorporated the design wave height as $H_{10} = 1.27\ H_s$, into Eq (7.9).

7.4.2 Stability coefficients and damage levels

In addition to the different definition of the wave heights into the Hudson related stone sizing formulas, different criteria are given for the stability coefficients and damage levels.

Canada, SEBJ (1997)

The SEBJ (1997) base on laboratory testing and extensive survey of embankment dams (see Mansard et al., (1996) and Tournier et al., (1996)). SEBJ(1997) highlight that a linear variation of the stability coefficient K, as a function of S_d, at least up to S_d =5, exists according to the laboratory tests conducted with irregular waves and using the nominal diameter of the minimum stone weight. Furthermore, that this corresponded to an acceptable degree of damage. Additionally, beginning of damage was fixed at S_d =2.5, which is equivalent to the zero-damage condition. This recommendation assumes the nominal diameter of the minimum stone weight.

SEBJ (1997) give the following values of the stability coefficient for two layered riprap dumped in bulk on a slope equal to or flatter than 1.8 (horizontal component):

K = 1.75 for no damage (Damage level $S_d \leq 2.5$)

K = 3.5 for an acceptable level of damage (Damage level $S_d = 5$)

SEBJ (1997) recommends the use of a design wave height with a return period of 1000 years and a stability coefficient of K = 3.5 to determine the minimum weight of the riprap. Furthermore, introduces a control measure, that entails assuring that for the selected weights meet the critiera for no damage for the 100 year event.

Here it must be noted that SEBJ (1997) recommendation for the stability coefficient applies for an embankment slope with a horizontal component of 1.8 or flatter. Furthermore, that SEBJ (1997) recommends to limit the steepness of the slope of the riprap layer to 1.8:1 (Horizontal:Vertical). This recommendation bases on that damage was observed in the field on very steep slopes. Furthermore, SEBJ (1997) states that during scale-model tests, sudden failures by sliding were observed on 1.5:1 (H:V) slopes but not on slopes of 1.8:1 (H:V). However, no reference is provided on these scale-model tests, and without this it is impossible to evaluate the conditions that lead to this statement. In the reference list of SEBJ (1997) one can find Mansard et al., (1996) and Tournier et al., (1996), but they do not report on any model tests on slopes 1.5:1.

Norway

In the current Norwegian guidelines (NVE, 2012) for embankment dams, the stability coefficent, K, into the Hudson formula (Eq 7.11) is presented as a constant that is dependent on the shape of the stone and how it is placed on the slope. Thus an upperboundary of 2.5 is defined for Norwegian conditions, i.e. $K \leq 2.5$. The guidelines do not discuss the damage level associated with this selection of K, but the sizing is to be carried out for the 1000 year wind.

In the obsolete Norwegian Regulations from 1981 (NVE, 1981), different values are provided for K depending on the stone shape. For stones that are cubical and placed onto the slope in an interlocking manner, K values are said to vary between 5.7 to 13.6 For stones that are elonget and placed in an interlocking manner with the longitudinal axis normal to the slope, the K values are said to be larger than for the cubical stones.

USA

The design standard USBR (2012a), provides two sets for the experimentally determined coefficients and exponents into Eq (7.12), one for the zero damage level design scenario, and another for the tolerable damage.

USBR (2012a) lists in an appendix various coefficients used in the empirical riprap equation, but recommends the following values: For the zero damage design scenario, the values are: K=3.62, a=2 and b=0.67; while for the tolerable damage design scenario, the values are: K=4.37, a=3, and b=1. These recommended values are based on USACE (1975).

USBR (2012a) emphasizes that the riprap sizing equations presented in the standard are applicable for embankment slopes from about 2:1 to 5:1 (H:V), since these were the slopes used in deriving the relationships.

The Rock Manual, CIRIA et al.(2007)

From the CIRIA et al.(2007) guidelines the modified version of the Hudson formula Eq (7.9) is considered here. The Eq (7.9) includes within the formula itself both the coefficient K_D, as well as the damage level S_d. Thus the K_D value recommended by CIRIA et al., (2007) cannot directly be compared to the K values recommended by other guidelines. The damage level S_d. must be determined for example from considering Table 7.1., whereas two different K_D value are recommended: K_D = 1 for structures with an impermeable core (essentially meaning the riprap layer is on an impermeable embankment), and K_D =4 for structures with a permeable core (essentially meaning that the riprap layer is on a permeable embankment).

CIRIA et al., (2007) further gives examples from the Shore Protection Manual from 1977 (SPM,1977) and 1984 (SPM, 1984) for values of K_D into the Hudson equation presented with Eq (7.9). The values given from the SPM(1977) for rough, angular, randomly placed armourstone in two layers on a breakwater trunk where K_D =3.5 for breaking waves on the foreshore, and K_D =4 for non-breaking waves on the foreshore. CIRIA et al., (2007) explains that the design wave height should be used with these values. The values given from the SPM(1984) are K_D =2 for breaking waves on the foreshore, and K_D=4 for non-breaking waves. SPM(1984) further recommends the design wave H_{10} to be used with these values into the formulae.

Stability of interlocked and dense riprap layer (armour layer)

The Rock Manual explains that the most design methods base on experimental tests on randomly placed armour stones (ripraps) that are different in nature than armour layers (riprap layers) of packed and well interlocked stones. This applies for example for the formulas and stability coefficients discussed in this report. Stewart et al. (2003) investigated the effects of stone packing on the properties of carefully placed stones in armour layers, and found that this exceeded the stability of randomly placed layers (dumped riprap).

7.4.3 Design event and damage levels

It is natural in civil engineering pratice to associate acceptable damage levels to the return period of the action considered. In terms of riprap design, the choice of the damage level should idealy be related to the return period of the maximum wave action, which is directly related to the return period of the wind velocity used in the wave prediction. The damage level/design event relations should preferably be clearly stated in relevant guidelines so the basis of the design is understood. Here the different approaches in the different guidelines are summarized.

Canada, SEBJ (1997)

SEBJ(1997) recommends to adopt a return period of 1000 year for the wind velocity and thus the wave action for the design of riprap on embankment dams. Furthermore, that a certain level of damage (S_d=5), not affecting the integrity of the structure, should be tolerated. SEBJ(1997) further explains that this corresponds to a 5% risk of exceedance that an event will occur during an economic live span of 50 years. SEBJ(1997) further recommends to verify that the zero-damage condition is met for a return period of around 100 years.

Norway

In the current Norwegian guidelines (NVE, 2012) the 1000 year wind velocity is to be used to determine the wave height used into the riprap sizing formula. Alternatively, a wind velocity of 30 m/s can be used. Here it must be noted that in associated guidelines NVE (2003) the wind velocity of 30 m/s is associated with a return period of 50 years. The design scenarios are not related to damage levels.

USA

USBR (2012a) explains that a wind velocity with 100-year return period is assumed in the riprap design of most embankment dam. Furthermore, that this may be loosely associated with the remaining life of the structure. However, that economic factors can influence the decision to use a different return period, for example if it is difficult to obtain riprap.

Europe (CIRIA et al.(2007))

The CIRIA et al.(2007) guidelines do not focus on dams and thus will not be discussed in the context of selecting design events and suitable damage levels. However, it is interesting to note that while CIRIA et al.(2007) present the most common formulas for riprap sizing due to wave action, it is stated in the guidelines that the formulae presented should be used for the conceptual design of the relevant structures. Furthermore, that the conceptual design should be confirmed and optimised with physical model tests.

7.5 Riprap sizing for ice action

As previously mentioned, sizing of riprap in cold regions to resist ice action is less understood compared to sizing to resist ice action (CIRIA et al., 2007; ICOLD, 1996) and guidance on the design limited. Still, investigation into the problem have been carried out e.g. by Daly et al., (2008), as well as Sodhi et al., (1997, 1996) and Sodhi and Donnelly (1999). In practice, the emphasis is generally on designing riprap to resist the design wave action and assume that riprap capable of absorbing wave energy is often also capable of absorbing ice forces (CIRIA et al., 2007; ICOLD, 1996). However, some minimum weights of the riprap stone is for example specified in the Norwegian guidelines (NVE, 2012). Similarly, although not related to ice action, minimum weights are specified in USBR (2012b), and minimum design wave heights are recommended in SEBJ(1997).

7.5.1 Laboratory tests

Randomly placed riprap

Laboratory tests for the study of ice action on riprap of randomly placed stones (dumped stones) conducted by Sodhi et al., (1996) and Sodhi and Donnelly (1999) , suggested that the diameter of an average size stone must be 2 and 3 time the ice thickness, respectively for slopes of 3:1 (Horizontal:Vertical) and 1.5:1 (Horizontal: Vertical). These findings may however be difficult to incorporate for severe artic conditions resulting in ice thickness exceeding 1 m and consequently extremely large stone sizes, Sodhi et al., (1996) state for example that it may be more cost effective to carry out repairs than to design to these requirements.

In the design of concrete dams in Norway, ice thickness of 0.5 m is to be considered. Transferring this design criteria to embankment dams with the consideration of the above-mentioned laboratory findings, would result in riprap diameter of 1 to 1.5 m. However, it must be considered that the riprap on dams in Norway is not randomly placed and thus it is not reasonable to apply the findings by Sodhi et al. (1996).

Sodhi et al. (1996) found that smooth riprap surfaces resist the ice forces better in the case of the randomly placed riprap. McDonald (1988) also stated that smooth slopes of graded riprap resisted ice load better than rough uniformly graded riprap. McDonald (1988) emphasized the importance of well keyed stones.

Selectively placed riprap

Daly et al., (2008) conducted laboratory tests on riprap layers, using selective placement of armor stones. The selective placement was described as carefully placing individual stone to maximize interlock and support between the stones. This was found to have a significant and positive impact on the stability. For example, a selectively placed 3600 kg (35 kN) armor stone (calculated to ca 1.3 m in diameter with γ_r=27 kN/m^3 but Daly et al (2008) refer to larger diameter of 2 m (unit weight not given)) resisted ice shoves (of about 1.5 m thick ice sheet) very well on a 1.5:1 (Horizontal: Vertical) slope. It is important that the riprap is selectively placed, and due care is taken in the design and placement of the toe stones.

The results from Daly et al., (2008) can be simplified to indicate roughly that the diameter of the armour stone that is selectively placed into a riprap layer, should be 0.86 to 1.35 times the ice thickness. Considering an ice thickness of 0.5 m, this leads to a stone diameter of approximately 0.45 to 0.65 m. (0.45 m for γ_r=27 kN/m^3).

7.5.2 Minimum weight requirements in guidelines

In the Norwegian guidelines (NVE, 2012) the following minimum weight of the riprap stones are recommended: 2.5 kN for slopes of 1.5:1 (H:V), while for flatter slope such as 2:1and 3:1 (H:V), minimum weights of 1.6 kN should be used. For a stone γ_r=27 kN/m^3, [6] this gives a stone diameter of 0.55 for a slope of 1.5:1 (H:V).

In the Canadian guidelines, SEBJ(1997), the use of a minimum design wave of 1.0 m is recommended for the riprap sizing, unless there are other special reasons to do otherwise. Furthermore, SEBJ(1997) considers “that the influence of ice on the riprap is at the most marginal and that the minimum stone size resulting from the design wave calculations is adequate to resist ice forces“. However, it is interesting to calculate minimum weights

[6] γ_r=27 kN/m^3 bases on NGI(1976) (see Section 7.6.2), however NVE(2012) recommends max. 26 kN/m^3

from the minimum design wave recommended. Using the minimum design wave value, H_s=1 m, into the Hudson formula as presented in SEBJ(1997) (same as the Norwegian guidelines (NVE, 2012)) with the SEBJ(1997) recommendation of K=1.75, and additionally selecting the steepest slope recommended, 1.8:1 (H:V), and γ_r=27 kN/m^3, results in W_{min}= 1.7 kN. Similarly, using this value, H_s=1 m, into the Hudson formula as presented in the Norwegian guidelines (NVE, 2012), with K=2.5, and the common upstream slope in Norway with n=1.5 and γ_r=27 kN/m^3, results in W_{min}= 1.5 kN.

In the US dam guidelines, USBR (2012a), design situations including items such as ice or extreme freeze-thaw conditions are among other items mentioned as an examples of situation that may influence the design of riprap. However, no guidance is provided on how to design for these conditions other than these should guide the designer‘s judgment when selecting design parameters. Although, USBR (2012a), gives values for both minimum and maximum median weights of well graded riprap, these do not consider ice action. The minimum value, $W_{50,\ min}$, is set to 160 Pounds (or 0.7 kN) and the maximum value, $W_{50,\ max}$, is set to 2000 Pounds (or 8.9 kN). The maximum value is based on a research presented in an appendix to the guidelines indicating that 2000 pounds would have been adequate to avoid riprap failure by erosion for all the historical cases studied.

The Rock Manual (CIRIA et al., 2007) discusses structural response related to ice, but does not specify additional sizing requirements due to ice action.

7.6 Comparison of methods

The methods discussed in the previous section on riprap sizing due to wave action are compared in this section with numerical examples. The simplest comparison is to assume the same wind wave prediction and use this in the formulas, this is carried out in subsection 7.6.1. Thus, the method of calculating the significant wave height will further influence comparison between methods. Conversely, the full methodology in each guideline is compared to some extend in the subsection 7.6.2, for the case of the Nesjøen Reservoir.

7.6.1 Simple comparison

The comparison of the riprap sizing formulas assumes the same wind wave prediction and use this in the formulas. Thus, the method of calculating the significant wave height would further influence comparison between methods. However, the full methodology is not compared in this section, but the effect of the different approaches in determining the significant wave heights can further be realized by studying significant wave heights results from the conceptual reservoir in previous chapters. The methods are compared in Figure 7.1 and Figure 7.2.

The comparison in Figure 7.1 and Figure 7.2 assumes constant unit weight of the rock. The upstream slopes are 2:1 (Horizontal:Vertical) and 1.5:1 (Horizonal: Vertical) in Figure 7.1 a) and b) respectively. The comparison for slope of 2:1 (Horizontal: Vertical), is selected since all the guidelines, except NVE (2012), emphasise that the formulas and coefficients were not derived for steeper slopes than 2:1 (USBR, 2012a) or 1.8:1 (Horizontal: Vertical) (SEBJ,1997).

Regarding Figure 7.1, it is important to note that the significant wave height on the horizontal axis is multiplied with 1.27 into the USBR (2012a) formulation, and that this effect is included in Eq. (7.9) from CIRIA et al.(2007). Furthermore, it must be remembered the USBR, (2012b) method of predicting the significant wave height results in lower values than for most of the other methods studied in this report. To comprehend the difference, the same curves are plotted in Figure 7.2, however with the significant wave height into all the formulas (i.e. the value H on the horizonal axis). This entails for example, that the Eq. 7.9 from CIRIA et al.(2007), is divided by 1.27 to obtain the value for the significant wave height (the Eq. 7.9 as presented includes the design wave as 1.27 H_s). The CIRIA et al (2007) curves in Figure 7.2 are in the following referred to as modified CIRIA et al (2007) curves.

Two values of the damage level S_d, 2.5 and 5, are considered in the relevant formulas compared in Figure 7.1 and Figure 7.2, i.e. the S_d values proposed by SEBJ(1997). Similarly, for the USBR (2012b) version of the Hudson formula, both the coefficients representing Zero Damage (ZD) and those representing Tolerable Damage (TD) are plotted in the figures.

It is important to note that the 1000 year wave heights (calculated using the 1000 year wind) are to be used with the SEBJ(1997) S_d = 5 curves, representing tolerable damage; while the 100 year wave heights (calculated using the 1000 year wind) are to be used with the S_d = 2.5 curves, representing Zero Damage. Similarly can be considered for the CIRIA et al.(2007) curves (also the modified curves).

It should be noted that the impervious case from CIRIA et al (2007) does not represent typical riprap on rockfill dams, since the riprap protection layer are not placed on impermeable (IMP) shoulders. Thus, beforehand one would assume that the pervious (PER) case was more appropriate for the comparison to rockfill embankment dams. These two cases present upper and lower boundaries for the stone sizing formulas considered. The CIRIA et al (2007) IMP curves for zero damage (S_d = 2.5) represents the upper boundary, while the curve for the CIRIA et al (2007) PER curves for tolerable damage (S_d = 5) represents the lower boundary.

The NVE (2012) curve of the stone size in Figure 7.1 and Figure 7.2, is to be calculated for the 1000 year wind, and thus it should be reasonable to assume that tolerable damages are allowed. In Figure 7.1 the NVE (2012) curve for the minimum stone weight, $W_{\min}$, lies between the SEBJ(1997) and USBR(2012) curves for tolerable damage (or S_d = 5). While in Figure 7.2 the NVE (2012) curve for the minimum stone weight, $W_{\min}$, compares reasonable to the modified CIRIA et al.(2007) IMP (Impervious structure) curve for W_{50}, with S_d = 5, i.e. tolerable damage (Eq. 7.9 here divided by 1.27). Effectively, the NVE (2012) curve represents larger stones, since NVE (2012) gives the minimum value and CIRIA et al.(2007) the median value.

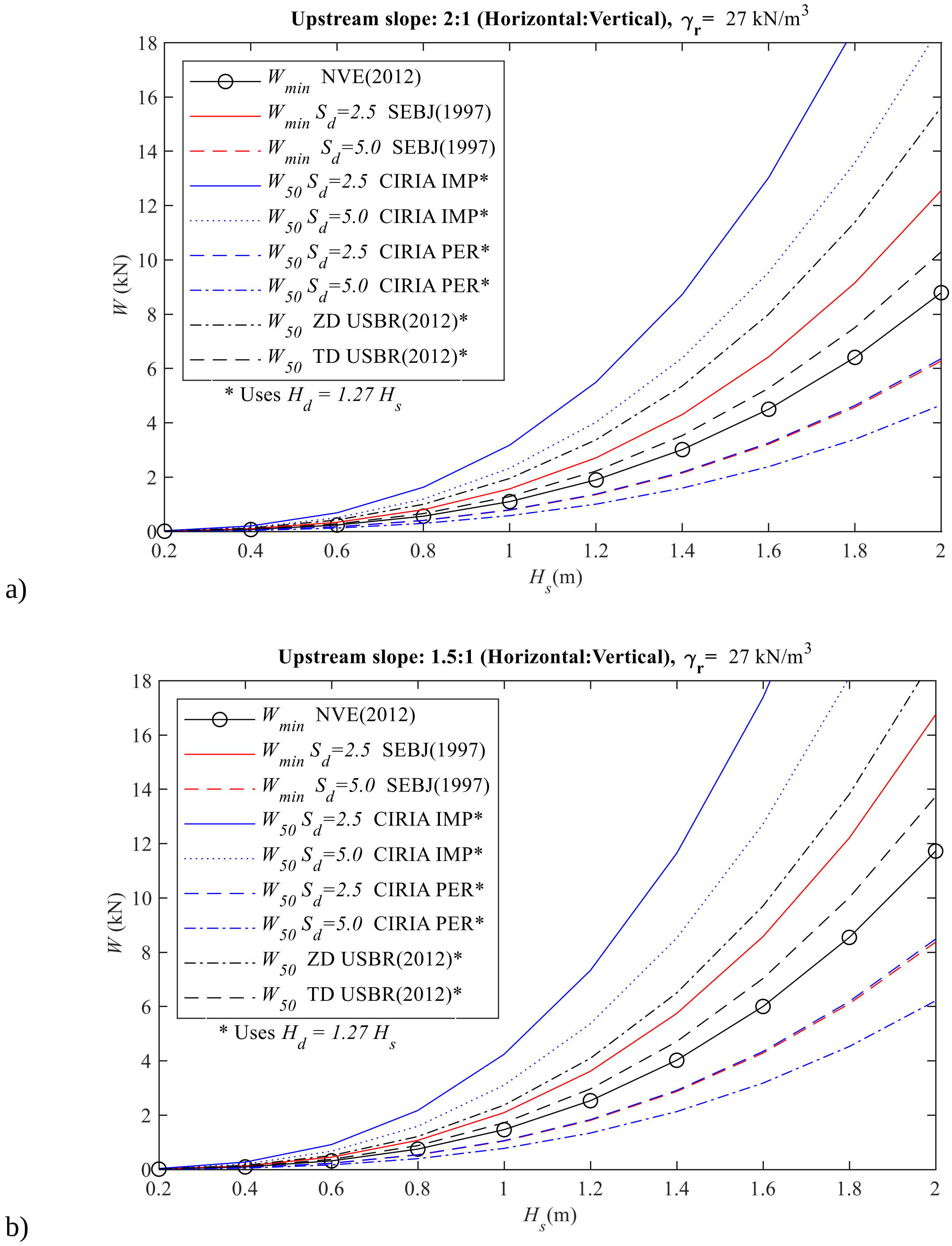
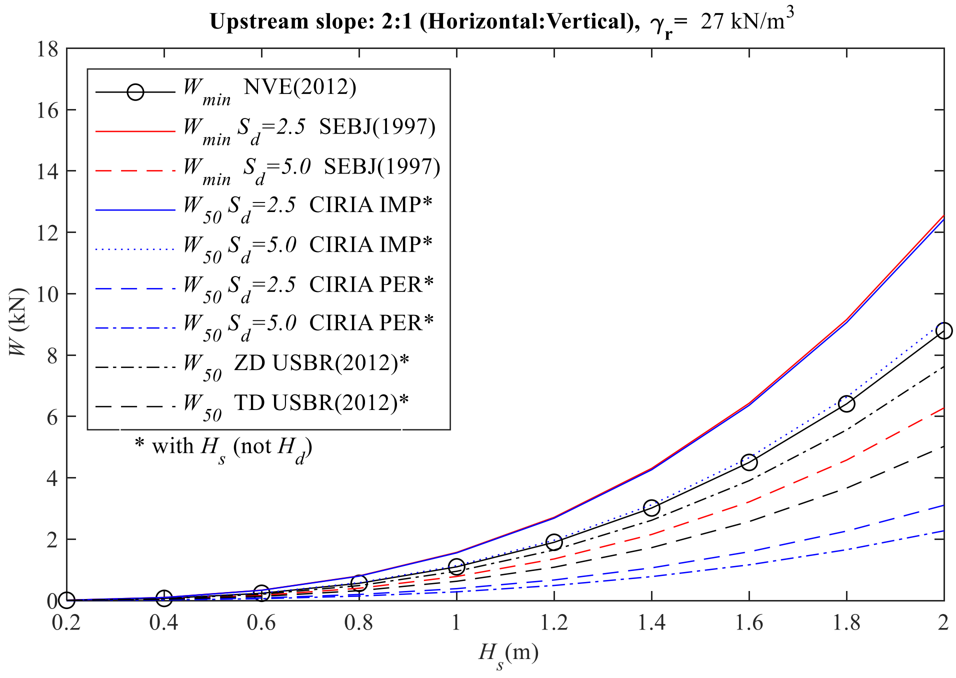
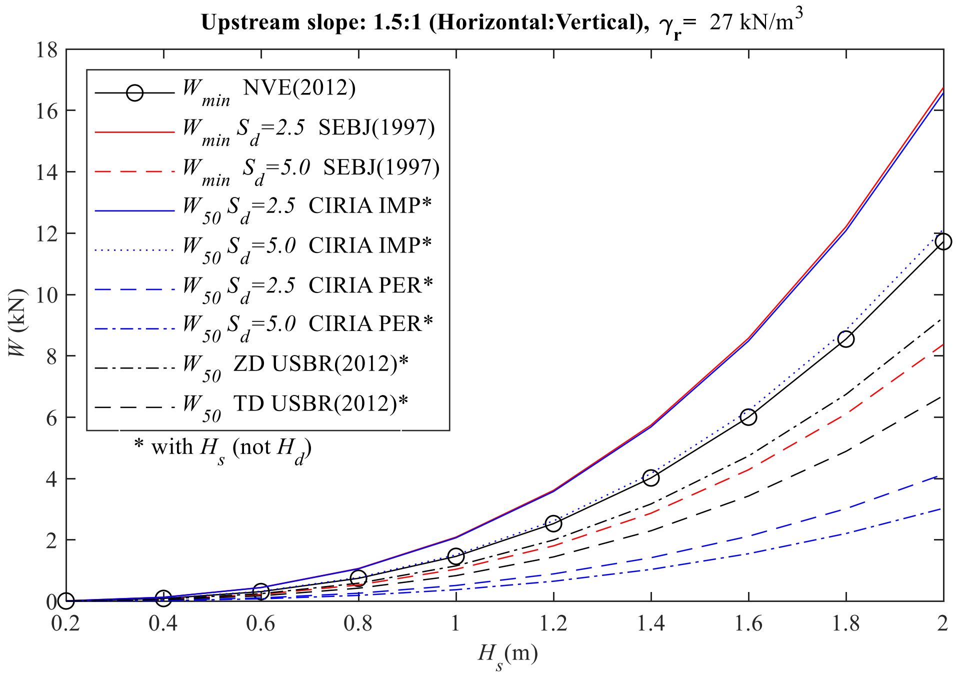


Figure 7.1 Riprap sizing – comparison of formulas- H_s and H_d. Unit weight of riprap 27 kN/m^3. a) Slope of 2:1 (Horizontal to vertical). b) Slope of 1.5:1 (H:V). Legend: [ZD and TD (as well as S_d =2.5 and 5) *in the legend refer to zero damage and tolerable damage repsectively], [IMP and PER: Coefficients for an impermeable (IMP) or permeable (PER) core is inserted into CIRIA et al.(2007), Eq. 7.9] [CIRIA and USBR use H_d=1.27H_s]*



a)



b)

Figure 7.2 Riprap sizing – comparison of formulas – H used in all formulas. Unit weight of riprap 27 kN/m^3*. a) Slope of 2:1 (Horizontal to vertical). b) Slope of 1.5:1 (Horizontal to vertical). Legend: [IMP and PER: Coefficients for an impermeable (IMP) or permeable (PER) core is inserted into CIRIA et al.(2007), Eq. 7.9] [CIRIA et al.(2007), Eq. 7.9 is divided by 1.27 (here: Modified CIRIA et al. (2007))].[The wave height on the horizontal axis is used directly into the USBR(2012) formulas (i.e. not multiplied with 1.27)] .*

In Figure 7.2, the SEBJ(1997) S_d = 2.5 (No Damage) curve for the minimum stone weight, $W_{\min}$, is almost identical to the modified CIRIA IMP curve for W_{50}, with S_d = 2.5, i.e. No Damage. However, the modified CIRIA et al.(2007) formulation results in larger riprap sizes for the Tolerable Damage condition with S_d = 5, than is obtained with the SEBJ(1997) formulation. (In Figure 7.2, Eq. 7.9 is divided by 1.27).

The curves plotted after USBR (2012a) represent the W_{50}, of widely but well graded rock material. The other curves can be considered to represent narrowly graded material (either the minimum value or the median value). In Figure 7.1 the SEBJ(1997) S_d = 2.5 curve, i.e. for no damage, lies between the USBR(2012) curves of zero and tolerable damage. While in Figure 7.2, when the same wave height is inserted into the formulas, the SEBJ(1997) S_d = 5 curve, i.e. for Tolerable Damage, lies between the two USBR(2012) curves.

7.6.2 Comparison of methods for the Nesjøen Reservoir

The volume of individual riprap stones on the dam retaining the Nesjøen Reservoir, were provided by the Dam Owner (the information was given to Tiril Berg Bjørsom, MSc Student at NTNU in connection with her MSc thesis work). The information was presented as the minimum volume of installed stones, with different values depending on location on the dam. The minimum volume of individual stones installed at stations 0 to 600 is 0.25 m^3, while on stations 600 to 1050, the minimum volume installed is 0.5 m^3. In the NGI report from 1976 (NGI, 1976), the significant wave height and consequently the required stone volume were calculated for two values of the wind velocity 25 m/s and 30 m/s, respectively resulting in values of 0.185 m^3 and 0.315 m^3.

Thus, here the volume of the stones will be compared rather than the weights, using that the volume is the stone weight divided by the unit weight of rock given in the NGI report (NGI, 1976). The comparison is given in Figure 7.3 to Figure 7.5. The minimum volumes of individual stones placed on the dam are included in the figures. Additionally, a line is drawn between the values calculated by NGI that base on the design wave calculated as $H_d = 1.3\,H_s$.

The calculated volumes in Figure 7.3 are obtained using the NVE(2012) Eq 7.11, and significant wave heights previously calculated for the Nesjøen Reservoir (see Table 4.7 to Table 4.9, Table 4.14 and Table 4.15). Conversely, the calculated volumes in Figure 7.4 and Figure 7.5 base on the stone sizing formulas relating to the method used to calculate the significant wave heights (see Table 4.7 to Table 4.9, Table 4.14 and Table 4.15). In Figure 7.4 the original stone sizing formulas from CIRIA et al (2007) and USBR(2012) are used (with H_d), while the modified versions are used in Figure 7.5 (using H_s).

In Figure 7.3 the symbols refer to the different methods of calculating the significant wave height, while the colors signify the method (NVE or Eurocode) of predicting or selecting the wind velocities of either 50 or 1000 year return period. Obviously the different volumes calculated vary as do the significant wave heights in Figure 4.25, since the same formula (Eq 7.11) is used for calculating the riprap sizes in all cases. The values marked with the symbol for H_s SEBJ(1997), are the only values that are fully according to the Norwegian guidelines (NVE, 2012, 2003). All the other values use significant wave height by other methods than the one recommended in the Norwegian guidelines (NVE, 2003).

It is interesting to note that the values according to the Norwegian guidelines (NVE, 2012, 2003) are all below the minimum volume installed on the dam, also the values calculated for the fixed wind velocity of 30 m/s given as an option in (NVE, 2012). Only the values calculated for 1000 year wind basing on the 30 m/s as the 50 year wind (NVE, 2003), result in a larger volume than the minimum volume installed on the dam at St. 0 to 600. However, according to NVE(2012) it is sufficient to use the fixed velocity of 30 m/s. Furthermore, the method used in NGI (1976) (before the issue of official regulations) resulted in larger riprap stones than the current version (NVE, 2012, 2003).

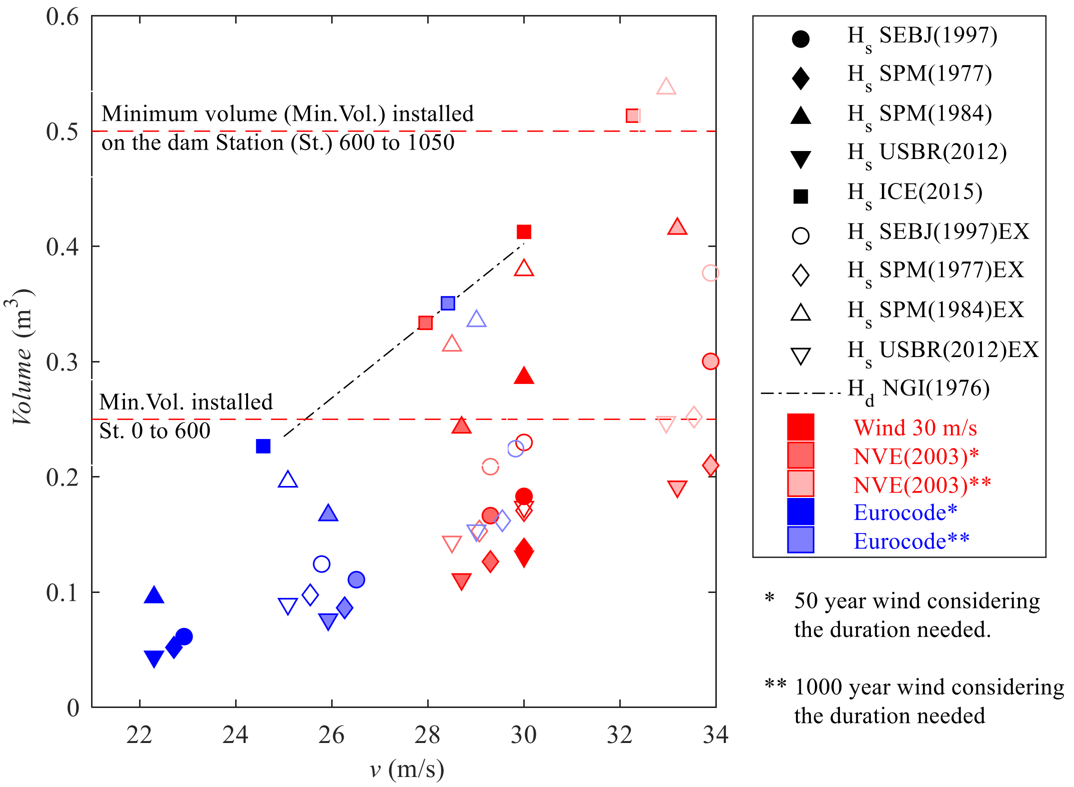


Figure 7.3 Riprap sizing with NVE(2012) formula and different predictions of the significant wave heights. Unit weight of riprap 27 kN/m^3. (The values marked with the symbol for H_s SEBJ(1997), are the only values that are fully according to the Norwegian guidelines (NVE, 2012, 2003)). In the legend: Symbols refer to method to calculate the significant wave heigth. Color refer to the method of predicting the wind. [ZD and TD in the legend refer to zero damage and tolerable damage repsectively].[CIRIA et al.(2007), Eq. 7.9 is divided by 1.27, and uses the coefficients for impermeable core].

It is further interesting to note that the significant wave heights predictions according to ICE(2015) results in volumes that line up along the line drawn between the two values calculated in NGI (1976), but only with the NVE(2012) Eq 7.11. The same trend is not observed in Figure 7.4 and Figure 7.5, where the CIRIA et al.(2007) Eq. 7.9 (original in Figure 7.4, modified in Figure 7.5) is used along with the ICE(2015) prediction of the significant wave height.

In Figure 7.5 the symbols refer to the different methods of predicting the significant wave heights, as well as the formula (guidelines) used for the riprap sizing. As before the colors signify the method (NVE or Eurocode) of predicting or selecting the wind velocities of either 50 or 1000 year return period, but additionally the colors also refer to the Zero Damage level or Tolerable Damage level for the riprap sizing where that applies, that is for riprap sizing according to SEBJ(1997), USBR(2012) and CIRIA(2007).

For example, the legend "H_s SEBJ(1997)EX, W_{min} NVE(2012)", refers to the significant wave height H_s calculated according to SEBJ(1997) using the extended fetches (EX), while the riprap sizing (the weight W_{min} from which the volume is calculated) is according to the Eq 7.11 from NVE(2012). The extended fetch was selected here due to a clause in NVE (2003) stating that narrow reservoir shapes may need to be considered. However, again, it must be emphasized that extending the fetches is not according to the methodology presented in SEBJ(1997) and should not be used with the prediction formulas provided in the Canadian guidelines, that in turn are referred to in the Norwegian guidelines NVE (2003).

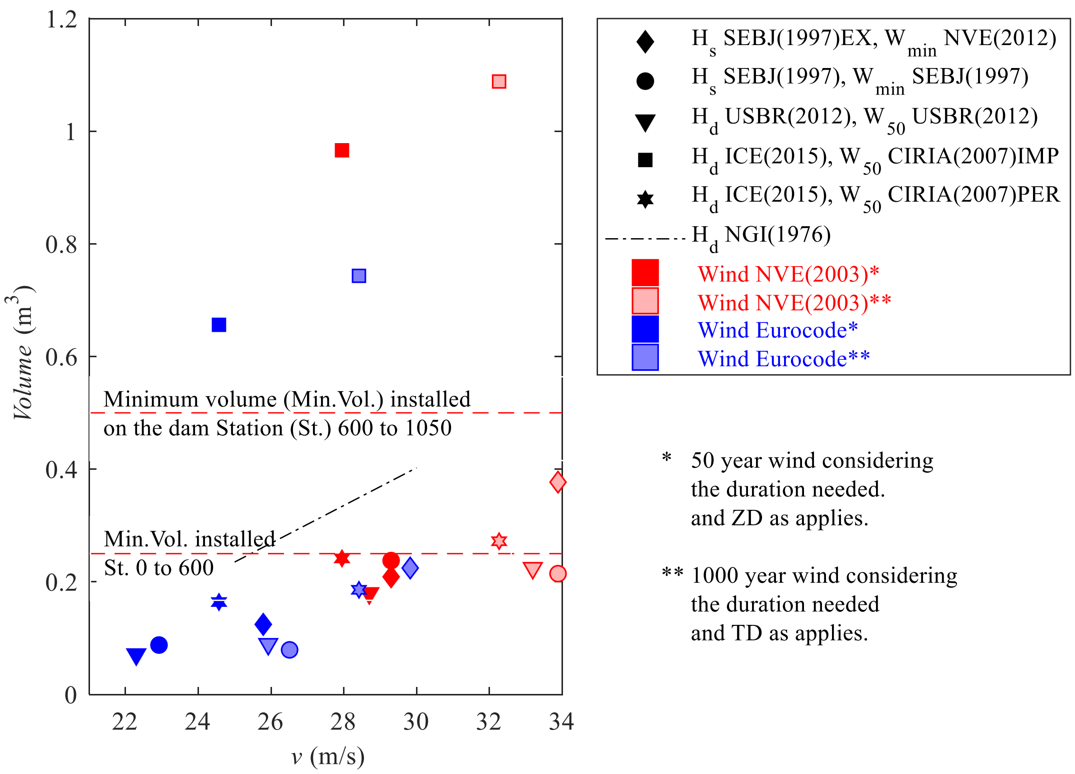


Figure 7.4 Nesjøen. Riprap sizing – comparison of formulas. Unit weight of riprap 27 kN/m³. [ZD and TD *in the legend refer to zero damage and tolerable damage repsectively].*

In Figure 7.3, where the design situation is not considered with different damage level, and the same riprap sizing formula is used in all cases, the riprap size increases with the wind velocity in all cases. The effect of including the damage level into the prediction can be seen from Figure 7.4 and Figure 7.5. For example, following the blue circles in Figure 7.5, which denote the SEBJ(1997) methodology, the 1000 year wind velocity is associated with tolerable damage level and results in slightly smaller riprap size than the 50 year wind velocity (should be 100 year according to SEBJ(1997), but 50 years is used here as this has been calculated before in this report) that is here associated with the zero damage level. Similarly, is observed for the red circles, as well as all the symbols, except the with rock sizing according to NVE (2012), which does not consider the damage level.

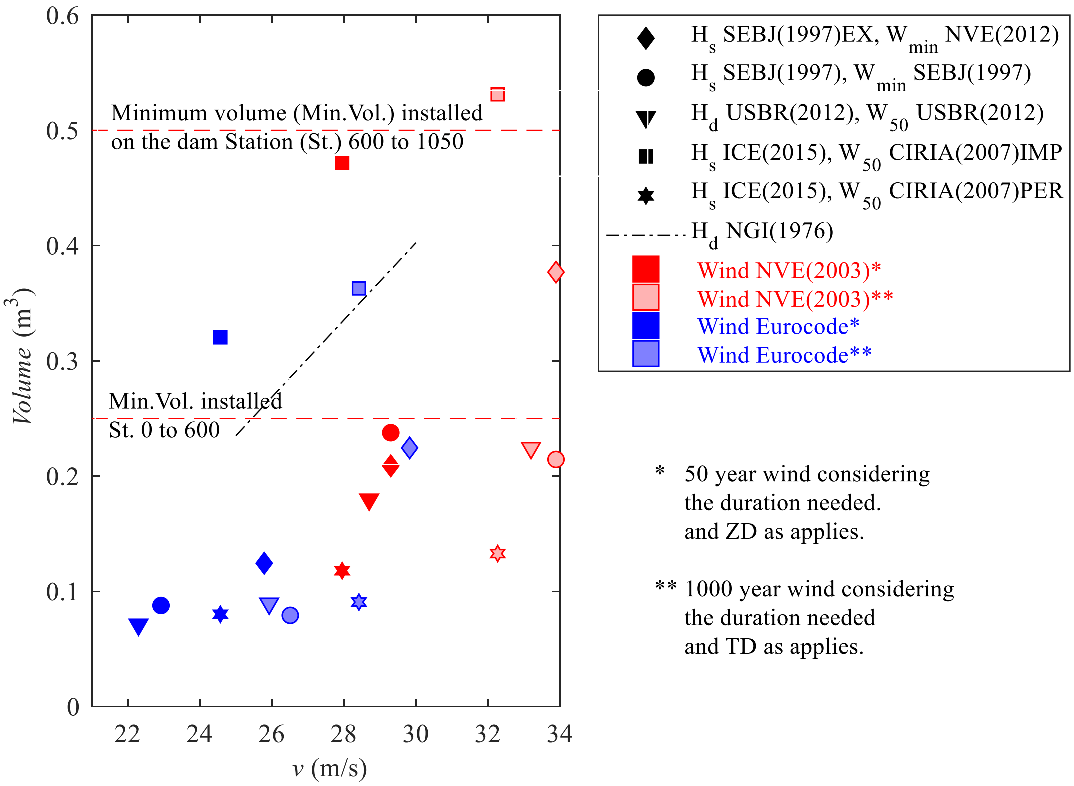


Figure 7.5 Nesjøen. Riprap sizing – comparison of formulas. Unit weight of riprap 27 kN/m^3. [ZD and TD in the legend refer to zero damage and tolerable damage repsectively].(CIRIA et al.(2007), Eq. 7.9 is divided by 1.27)

7.7 Summary on sizing of riprap (armourstone)

Stone sizing formulas given in different dam safety guidelines and a rock manual have been compared. All the formulas base on the empirical Hudson formulae, which relates wave action and properties relevant for stability of the upstream riprap protection. The stability depends e.g. on the stone size (weight), upstream slope inclination, placement of the stones and more. The relation is established through experimentally determined stability coefficients.

The guidelines considered here are issued in USA (USBR, 2012), Canada (SEBJ, 1997), Norway (NVE, 2012) and the manual is the European Rock Manual (CIRIA et al, 2007).

- All the guidelines/manual, except the Norwegian regulations consider different damage levels and associate this to different return period of the design action, where the design action is the wind generated wave.

- Furthermore, all the guidelines, except the Norwegian, refer to experimental tests on which the recommended stability coefficient bases.

- Only the Rock Manual, that emphasizes coastal structures, considers through the stability coefficients whether the armourstone/riprap protection layer is on impermeable or permeable structures (core of a rock structure in the Rock Manual, i.e. not the same as the core of a rockfill dam).

- The US guidelines (USBR, 2012b) and the Rock Manual (CIRIA et al., 2007) consider the design wave height into their stone sizing formulas. While the Norwegian (NVE, 2012) and Canadian guidelines (SEBJ, 1997) consider the significant wave height. USBR(2012) use of the design wave height considers that the associated method of predicting the significant wave height results in lower values than the method previously used. Both the USBR(2012) and the SEBJ (1997) approaches additionally base on field surveys of dams or historic performance data.

A comparison of the methods was conducted, first, inserting the same wave height into the stone sizing formulas with and without considering design wave (relevant only when following USBR(2012) and CIRIA et al (2007)), and secondly through the case of the Nesjøen Reservoir using the full methodology of each guidelines.

In the first comparison that assumes the same significant wave height into all the formulas, the CIRIA et al. (2007) (that converts to the design wave) results in the largest stone sizes in case of impermeable core regardless of damage level, but the lowest in case of the permeable core (effectively a permeable shoulder for dam considerations).

The case of the Nesjøen Reservoir and dam was used to compare not only the stone sizing formulas, but also how the different methods and formulas for calculating the significant wave height influences the stone sizing. This was investigated in two ways: Firstly, by using the stone sizing formula in NVE(2012), and the significant wave height according to different guidelines (Figure 7.3). Secondly, by applying the complete method of each guidelines, i.e. calculate the significant wave height and use into associated stone sizing formula of the same or associated guidelines (Figure 7.4 and Figure 7.5). However, the wind velocity is in all cases either according to NVE(2003) or the Eurocode. The actual stone sizes used on the Nesjøen dam were included in the comparison.

The comparison that used the Norwegian stone sizing formula along with the different significant wave heights (Figure 7.3), demonstrates that in the case of the Nesjøen Reservoir, the current Norwegian regulations with a fixed wind velocity of 30 m/s and not considering duration, result in stone sizes that are 37% and 73% of the installed minimum volumes, 0.5 m^3 and 0.25 m^3, respectively. Employing the 1000 year wind according to Eurocode results in even lower values. Similar is also the case even if extended fetches are used, which is strictly not a correct approach (see SEBJ (1997)), however in that case the volume is about 46% and 91%, respectively of the installed minimum volumes, 0.5 m^3 and 0.25 m^3.

The significant wave height calculated according to ICE(2015) and the NVE(2012) stone sizing formula (Figure 7.3), result in stone sizes that correspond well to the stone sizes calculated by NGI(1976). Furthermore, ignoring duration and assuming wind velocity of 30 m/s when calculating the significant wave height results in stone volumes that are respectively, 86% and 165% of the installed minimum volumes, 0.5 m^3 and 0.25 m^3.

Figure 7.3, demonstrates how the method of calculating the significant wave ultimately influences the stone sizing, in this case according to the formula in NVE(2012). Thus, uncertainties in the significant wave height prediction, further results in uncertainties in the sizing of stones for erosion protection of the upstream slope. Similar is observed from Figure 7.4 and Figure 7.5.

Figure 7.3 gives the stone sizes for different return periods of the action; although, the Norwegian regulations only considers the 1000-year event, or alternatively a wind velocity of 30 m/s. Thus, the stone sizing in Figure 7.3 does not consider damage level. Consequently, increased return period of the wind and the resulting increase in the significant wave height, simultaneously results in larger stone sizes. Conversely, Figure 7.4 and Figure 7.5, use the full methodology of the guidelines SEBJ(1997) and UBSR(2012); which both provide stone sizing formulas that are associated with prediction equations for the significant wave height. Both guidelines consider damage levels. The balance in determining the stone sizes when damage levels are associated with return periods of the action can be realized from the Figure 7.4 and Figure 7.5, for the these two methods, and similarly for the combination of ICE(2015) and CIRIA et al. (2007).

The US guidelines USBR(2012) on embankment dams are the most recent dam engineering guidelines of those considered herein. In an appendix to the guidelines the recommendations are compared to historic performance of US dams to strengthen the basis of the formulation. Through this investigation of the historic performance, the use of the design wave height into the stone sizing formula, is interconnected with the USBR(2012) prediction of the significant wave height. Furthermore, the stability coefficients recommended by the guidelines consider dumped well graded riprap on slopes that are not steeper than 2:1 (Horizontal:Vertical). The stone sizing by USBR(2012) for the case of the Nesjøen dam compare reasonably to the Canadian guidelines, SEBJ(1997) (full method), as seen in Figure 7.4 and Figure 7.5. The stone sizing formula in SEBJ(1997) also considers dumped riprap, but of uniform gradation on slopes that are not steeper than 1.8:1 (Horizontal:Vertical). These two North-American dam engineering guidelines result in slightly smaller riprap stones, than the European approach in Figure 7.4 for the permeable (PER) case for the stone sizing with the coastal engineering riprap guide CIRIA et al. (2007) and the ICE(2015) dam engineering prediction of the significant wave height. The impervious case in Figure 7.4 gives unreasonable stone sizes and does not apply directly to typical embankment dams. However, the modified version of the CIRIA et al (2007) stone sizing for the impervious case in Figure 7.5 results in stone sizes for the NVE(2012) recommended wind velocities (red colors) that are comparable to the stone volumes installed on the dam.

Further evaluation of the different results presented in Figure 7.3 to Figure 7.5 for the Nesjøen dam with the objective of finding the best approach, requires: First, experimental tests to confirm stability of the riprap stones for the Norwegian condition as it appears on the Nesjøen dam. Second, historic data on the environmental loads that the riprap protection on the upstream slope of the Nesjøen dam has been exposed to as well as how the installed riprap protection has performed. This conclusion can further be extended to other dams on mountain reservoirs, with placed riprap on the upstream slope.

The Norwegian guidelines can be enhanced when it comes to sizing of stones to use in erosion protection of the upstream slope of embankment dams. However, this entails research into the Norwegian conditions that must include field surveys and experimental testing. The field surveys should provide basis for evaluating the performance of different riprap design as well as the stone sizes. The experimental tests are necessary to determine the stability coefficient that represents the Norwegian condition. The requirement of interlocked and dense riprap stone layer is quite different in nature to experimental testing with randomly placed ripraps on which the design methods discussed are based. Investigations into the effects of stone packing on the properties of amour layers, indicate that the stability generally exceeds this of randomly placed layers (see Stewart et al., (2003)). The stability coefficient K (or constant as this is called in the guidelines) is seemingly not supported by a research into the Norwegian conditions referred to.

In the absence of defined damage levels, the riprap performance is difficult to measure against the design criteria. Additionally, the stone sizing must base on reasonable estimate of the wave action, e.g. through the significant wave height. However, different prediction formulas and fetch estimates give quite different results. Thus, there are considerable uncertainties in the prediction of the significant wave height. For example, the prediction formulas recommended by NVE(2003) cannot be recommended for reservoirs other than those that are large and wide, i.e. with high B/L ratios or similar to those reservoirs from which monitoring data was collected and regressed to provide the predictions.

The minimum stone weights recommended in NVE (2012) to resist ice action seem, according to a simplified and rough estimate, to compare reasonable with available (but limited) findings from the literature.

8 Summary and appraisal of the Norwegian dam safety guidelines

8.1 Overview

Previous chapters 2 to 7 of this report, have addressed various problems relating to environmental actions from ice and wind generated waves on embankment dam. For a summary on the topics of chapters 4 to 7, the summary in the last section of each of these chapters is referred to.

This Chapter 8 provides:

1. A summary relating to contract question 1 (See Table 1.1) with an appraisal of the Norwegian dam safety guidelines.
2. A summary relating to contract question 2 (See Table 1.1) on meteorological data and predictions from the highlands of Norway and at mountain reservoir. (This is investigated in Chapter 2 and 3).

8.2 Contract question 1: Appraisal of the Norwegian dam safety guidelines

8.2.1 Significant wave height (Chapter 4)

The method currently referred to in the Norwegian guidelines NVE(2003) for the calculation of the significant wave height, is the methodology proposed in SEBJ(1997) that bases on measurements on the La Grande Reservoirs in Canada. The SEBJ(1997) guidelines are not referred to in any of the engineering guidelines and manuals studied herein, except in Canada (according to Damov and Warren, (2012)) in which however the SPM(1984) is also referenced (according to Damov and Warren (2012)).

The investigations carried out in Chapter 4 strongly indicate that the SEBJ(1997) methodology mainly applies for large reservoirs, but is likely to underestimate wave heights generated on narrow reservoirs, such as mountain reservoirs. Thus the method referred to in NVE(2003) cannot be recommended for narrow mountain reservoirs widely found in Norway.

The method of determining the fetches is inherent in the SEBJ(1997) prediction formulas as clearly stated in SEBJ(1997). Thus, the exercise of extending the fetches as done in this report cannot be recommended as a general procedure to account for the narrow shape of a reservoir compared to the wide and large La Grande Reservoir in Canada from which the method originates.

Of the methods available, the one proposed by ICE(2015) (or SPM(1984)) basing on the JONSWAP spectrum and/or the Donelan-JONSWAP approach might be better suited for long narrow mountain reservoirs. However, a monitoring and research program is needed to further investigate the quality of the different methodologies and applicability to reservoirs in Norway, as well as in other mountainous countries.

Any potential modification to predicting the wave action must also consider the effect on the design criteria for the wave runup, as well as design of the upstream erosion protection, i.e. the riprap layer.

In various studies from the literature, the importance of engineering judgement is emphasized in the steps taken to predict the significant wave height. The direct and simple approach in the NVE guidelines, does not necessarily encourage engineering judgment. Given the input parameters, the calculation steps require limited knowledge and understanding. The outcome is uncertain as evident from the spread in the predictions for the reservoirs studied in this report.

8.2.2 Wave run-up (Chapter 5)

The Norwegian Regulations are rather strict when it comes to freeboard on embankment dams, with the requirement of no overtopping as well as predefined freeboard requirements and downstream erosion protection. The predefined freeboard requirements for dams in high and very high consequence classes may for example exceed the freeboard needed according to wave runup predictions.

The run-up height is dependent on the action, i.e. the wave height, as well as features of the slope. Uncertainties are associated with prediction of the significant wave height depending on which method is selected for the prediction. Additionally, there are always uncertainties relating to empirical equations and experimentally defined factors, such as the roughness factors used into the run-up equations. Thus, it is difficult to assess the quality of the prediction without comparison to measurements or records of performance. Still, judging from the comparison to the other methods considered here, the Norwegian guidelines (NVE, 2003) seem to be on the safe side with the current wave run-up heights predictions with 1% risk of exceedance. (The formulation given in the guidelines result in prediction that is slightly stricter than the 1% risk of exceedance (with the selection of the power *c* on *n*)).

The requirements regarding either definition of “no overtopping” or allowable discharge is different between the guidelines considered in this report. The guidelines (e.g. ICE(2015) and USBR(2012)) that consider different requirement depending on the conditions at the site in addition to different design situations, encourages engineering evaluation of the dam structures, with important risk and safety considerations. Conversely, the Norwegian guidelines, that mainly consider the extreme conditions, do not automatically encourage further risk and safety consideration.

8.2.3 Riprap sizing (Chapter 7)

The Norwegian guidelines (NVE, 2012) refer to SEBJ(1997) for the given stone sizing formula. Still, the guidelines do not make use of the full SEBJ(1997) approach and excludes important aspects of the riprap design, mainly associating stability coefficients to damage levels and this to different return period of the wave action. Additionally, the value of the constant *K* into the NVE(2012) stone sizing seems to has a weak basis.

According to NVE(2012) the value of *K* should be less than or equal to 2.5 for Norwegian conditions. However, no reference is provided to support the given *K* value, such as results from: experiments considering the Norwegian condition (and requirement of interlocking riprap), field surveys or evaluations of historical performance.

Experimental tests on which other guidelines, such as SEBJ(1997) and USBR(2012), base the stability coefficient into their version of the Hudson formulae, have been on slopes that are less steep than the upstream slope on most Norwegian dams. Furthermore, the experiments have been conducted on dumped riprap (randomly placed stones), which is not according to the current requirements in Norway. To the knowledge of the author of this report, no experiments have been carried out to support the upper value (2.5) of the constant *K* given in NVE(2012) for Norwegian conditions. The constant *K* in NVE(2012) formulation is the stability coefficient of the Hudson formula.

The dam safety guidelines from USA (USBR, 2012) and Canada (SEBJ, 2007), as well as the European Rock Manual (CIRIA et al, 2007) associate the stability coefficient to damage levels. The damage levels are in turn associated to return period of the wave action. Such approach is in line with acknowledged civil engineering practice. However, damage levels are not considered in the Norwegian guidelines NVE(2012) in the sizing of riprap stones. Still, such consideration embraces in a more holistic manner the design of the riprap as a protective layer rather than just sizing of individual stones. Furthermore, it provides a basis for a common understanding of acceptable damage as well as means to measure the performance of the riprap layer, i.e. the upstream erosion protection.

Consideration of damage levels provides an opportunity to monitor the performance of the upstream protection layer against the defined criteria. Damage criteria and levels need to be defined for placed riprap. Historic performance, measured in a predefined manner and compared against the specified damage levels, can further be used to enhance the criteria given for the stone sizing and the design of the protective layer. Holistic records of the performance should also include the environmental actions that the upstream slope has been exposed to.

The Norwegian guidelines can be enhanced when it comes to sizing of stones to use in erosion protection of the upstream slope of embankment dams. However, this entails research into the Norwegian conditions that must include field surveys and experimental testing. The aim should be to strengthen the prediction of the significant wave height as well as the stone sizing to resist this wave action. Ice actions are also of interest in this context, although, the minimum stone weights recommended in NVE (2012) to resist ice action seem, with simplified and rough estimate, to compare reasonable with available, but limited, findings from the literature.

8.3 Contract question 2: Summary on meteorological data and predictions.

Meteorological measurements in the Norwegian highlands that include monitoring of wind are currently conducted at 553 weather stations operated by the Norwegian Meteorological Institute (MET Norway). Of those 166 stations are located above elevation 400 m a.s.l. In view of the extend of the Norwegian highlands, wind monitoring at high elevation can be considered limited. Most of the weather stations above 400 m.a.s.l were installed after the year 2000, and most stations above 800 m a.s.l. are installed after the year 2010. Thus, longer monitoring period is generally required for a good statistical evaluation of the wind conditions in the highlands where the stations are located.

While some of the 166 stations in the eklima.no database elevated above 400 m a.s.l. may be located at or in the vicinity of a reservoir, most of those stations are not. Considering that there are 966 reservoirs in Norway with the highest regulated water elevation above elevation 400 m a.s.l., it can be stated that official monitoring (i.e. by MET Norway) of local wind conditions at reservoir sites are limited. It is possible that dam reservoir owners operate wind monitoring station, that are not included in the eklima.no database, but an overview of those stations is currently not available. Still, two such cases are investigated in the report, for the Nesjøen Reservoir and Aursjøen Reservoir, and the monitoring compared to prediction by the Eurocode (NS EN 1991-1-4, 2009) and NVE recommended values.

The Nesjøen weather station is installed to complement precipitation measurements, and thus is not conveniently located for measuring overwater wind. NVE recommended wind velocities in wave prediction are somewhat higher than wind velocities obtained for the Nesjøen area from predictions basing on the Eurocode. Furthermore, predictions basing on the Eurocode (NS EN 1991-1-4 (2009)) compare better to predictions based on statistical analysis of the measured values from Nesjøen than the NVE recommended wind velocities. Conversely, for the Aursjøen Reservoir, predictions based on statistical analysis of the measured data are 12% to 17% lower than predictions according to the Eurocode. However, for the Aursjøen Reservoir the NVE recommended wind velocities compare reasonably to these according to the Eurocode. Hence, for the two cases studied the Eurocode predictions can be considered reasonable. Further analysis of wind data from the highlands is required to establish a general conclusion on this considering wind prediction for mountain reservoirs.

Dam owners may operate weather stations at their dam sites and reservoirs or at other hydropower structures. However, these stations are not included in the official database operated by e.g. eklima.no. It is of interest to create a list of these stations, and include in the eklima.no database, alternatively create a new database with these stations.

9 Conclusions

The Norwegian guidelines can be enhanced when it comes to predicting wave height on reservoirs and sizing of stones to use in erosion protection of the upstream slope of embankment dams. The wave prediction is further required for calculating run-up heights or overtopping discharges.

The Norwegian Regulations are rather strict, compared to relevant other dam engineering guidelines, when it comes to freeboard on embankment dams. This includes the requirement of no overtopping as well as predefined freeboard requirements and downstream erosion protection. Strict requirements do not automatically encourage further engineering evaluations with risk and safety considerations.

Dam engineering application of simplified methods, to predict wind generated wave height and related design, requires engineering judgement to evaluate inherent uncertainties and site conditions. The uncertainty involved in the design process is evident from wide spread in the predictions depending on methodology. There is also uncertainty involved in the wind predictions for mountain reservoirs, including to what extend potential wind funneling should be accounted for, e.g. with the fetch used in the prediction. The uncertainty in the wind predictions further relates to the fact that there seem to be limited meteorological measurements in mountainous regions by relevant governmental institutes, particularly at reservoir sites.

The stone sizing formula in NVE (2012) disregards important aspects of the riprap design, mainly associating stability coefficients to damage levels and this to different return period of the wave action. Additionally, the value of the constant K into the NVE(2012) stone sizing formula seems to have a weak basis that should be strengthen with combined field surveys and experimental testing considering the Norwegian conditions with riprap placed in an interlocking manner or other relevant existing design. Associated study on the wind wave generation would enhance the overall outcome.

Augmenting the current design procedure entails further research into the Norwegian conditions and past performances, that should include field surveys and experimental testing. Combined evaluation of these factors is required to compensate the uncertainties relating to the stone sizing and placement, arising from both the action and the reaction.

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