KYSTVERKET
NORWEGIAN COASTAL ADMINISTRATION

# NORWEGIAN BREAKWATER CONSTRUCTION
## RECOMMENDED PRACTICES

— Preliminary edition —

Berlevåg Harbour. Photo: Tore Larsen

# PREFACE

The Norwegian Coastal Administration's vision is to develop the Norwegian coast and port areas into the world's safest and cleanest.

Our primary goals are to

- improve accessibility for persons and goods across the entire country
- reduce the transport mishaps in accordance with the zero vision
- reduce the greenhouse gas emissions in accordance with a realignment to a low-emission society and reduce other negative environmental consequences
- prevent and limit environmental damage in the event of emergencies involving contamination or the danger of contamination.

The Norwegian Coastal Administration has a zero vision for nautical mishaps, which entails preventing mishaps that lead to the loss of life, serious personal injuries or contamination.

These breakwater guidelines will contribute to fulfilling these objectives and provide guideline norms for which standards will apply in the planning and construction of breakwaters intended to have a wave-attenuating function for harbours or other functions of benefit to society.

These guidelines will comprise a useful tool and source of information for both governmental bodies as well as private companies – either in exercising governmental authority, as a breakwater owner or in the project planning and execution.

Ålesund, 1 December 2018

Per Jan Osdal
NCA Director General

# SUMMARY


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|---|---|
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**Summary:**

These breakwater guidelines describe recommendations for planning, design, construction and management of breakwaters along the coast of Norway.

The introduction provides a brief history of the development of port and breakwater construction in Norway. An overview is then given of which laws and regulations apply to the construction of breakwaters, which plans for land use must be provided and which permits must be obtained before such measures are commenced.

There are many different types of breakwaters, however only a few variants are common in Norway for breakwaters intended to protect ports and other developed areas against heavy wave action. These guidelines primarily address rock breakwater construction projects with armour units that are placed in a controlled manner. This is sometimes referred to as the - in this context - somewhat misleading term "rubble mound" structures. Other types are mentioned, but are not addressed in detail.

These breakwater guidelines show how it is recommended that a breakwater project be executed, from the needs analysis to its completed construction. In the planning of a breakwater implementation, a needs analysis should first be undertaken. A chapter has been included describing the intent and methodology for such an analysis.

Furthermore, there are explanations of which preliminary investigations should be performed as well as a number of topics that must be addressed during the initial planning phase. These include waves, currents, ice, geology, geotechnology, environmental studies of sediments, archaeology, cartographic basis, property ownership relationships, natural diversity and aquatic environment, etc. Wave action is especially important here, and an overview is given of how design parameters may be acquired, either by numerical calculations or by laboratory modelling. A more detailed discussion of wave theory has been included in an appendix.  It is a prerequisite that the project planning responsible has access to specialised expertise within this technical area.

A chapter has also been included on port planning, where the placement of the breakwater in the port area is addressed.

The detailed project planning of breakwaters is treated more extensively in chapter 7. This chapter is also intended to serve as recommendations for the execution of such detailed project planning through contracts that the project owner enters into with a project planning company. In the detailed project planning process, a number of detailed requirements emerge for execution, and some such general requirements are addressed in this chapter. This chapter may thus also be used as a foundation for contracts concerning execution.

Special requirements for execution are included in chapter 8. It can be beneficial to include these in contracts with contractors.

Chapters 9–12 provide recommendations concerning monitoring, HSE (Health, Safety and the external Environment), addressing the external environment by a plan for this, as well as recommendations for inspections during the operating phase, maintenance and repairs. Special considerations are also discussed here that must be made when repairing construction projects worthy of protection.

References and an overview of some specialised literature are included in chapter13.

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APPENDIX I: GENERAL WAVE THEORY

Moskenes Harbour

# 1. GLOSSARY, ABBREVIATIONS AND SYMBOLS

## 1.1 GLOSSARY

| | |
|---|---|
| Foothold trenches | Establishment of troughs/trenches in flat rock to create footholds for the breakwater toe |
| Overtopping | The water discharge rate over the top of the breakwater; in m3 per second per m breakwater |
| Construction client | Client, project owner |

## 1.2 ABBREVIATIONS

| | |
|---|---|
| PBL | Norwegian Act relating to Planning and the Processing of Building Applications, the Planning and Building Act |
| NML | Norwegian Act relating to the Management of Biological, Geological and Landscape Diversity, the Nature Diversity Act |
| AML | Norwegian Act relating to Working Environment, Working Hours and Employment Protection, etc., the Working Environment Act |
| RAV Assessment | Risk and vulnerability assessment |
| ULS | Ultimate Limit State |
| HSWE | Health, safety and the working environment at the construction site |
| HSE | Health, safety and external environment at the construction site |

## 1.3 SYMBOLS

| SYMBOL | UNIT | COMMENTS |
|---|---|---|
| $\alpha$ | ° | Slope angle of the breakwater |
| $A_e$ | $m^2$ | Eroded area of the breakwater around still water level |
| $B_s$ | m | Berm width |
| $\Delta$ | dimensionless | $\Delta = \frac{\rho_s}{\rho_w} - 1$ ; relative buoyant density |
| D | | Water depth at toe of breakwater |
| $d_b$ | m | Height of berm above design storm surge |
| $D_{n50}$, $D_{50}$ | m | Characteristic rock size - median nominal diameter defined as the median equivalent cube size. |
| H | | Height of the breakwater above the original sea bottom |
| $H_k$ | m | Breakwater height above design high tide |
| $H_{max}$ | m | Maximum wave height |
| $H_{m0}$ | m | Spectral significant wave height, $H_{m0} = 4\sqrt{m0}$ (m0 is the a zero'th order moment), explained in more detail in item 5.4.1 |
| $H_s$ | m | Significant wave height from wave record, i.e. the average value of the highest one-third of all waves in a measurement series |
| $H_{s,dim}$ | m | Design significant wave height |
| $\Theta_m$ | dimensionless | Mean wave direction |
| $L_0$ | m | Deep water wave length |
| N | dimensionless | Number of incident waves at the toe |

| SYMBOL | UNIT | COMMENTS |
|---|---|---|
| $\xi_z$ | dimensionless | Surf similarity parameter (Iribarren number) |
| $\xi^*$ | dimensionless | Critical value of the surf similarity parameter |
| P | dimensionless | Notional permeability of the breakwater |
| $\rho_s$ | $kg/m^3$ | Density, rock |
| $\rho_w$ | $kg/m^3$ | Density, (sea) water |
| S | dimensionless | $S = A_e / (D_{n50})^2$ ; dimensionless damage parameter |
| $T_m$ | | Mean wave period |
| $T_p$ | s | Peak spectral period in wave spectrum |
| $T_z$ | s | Mean period, zero-cross period, may be estimated by $Tz \approx Tp/1.35$; |
| $W_{50}$ | kN | Block weight corresponding to $D_{50}$; $W_{50} = \rho_s D_{n50}{}^3$ |

Breakwaters outside Myre Harbour. Photo: Per Aarsleff AS

# 2. INTRODUCTION

## 2.1 ON THE USE OF THE BREAKWATER GUIDELINES

The target group for these breakwater guidelines is employees of the Norwegian Coastal Administration, counties, municipalities, port enterprises and other project owners, as well as private sector consulting companies and contractors who work with planning, project planning and development of ports and fairways, land-use zoning processes, applications concerning permits for measures pursuant to the Norwegian Harbour and Fairways Act and navigational guidance. The book provides guidelines and recommendations for project implementation, planning and execution as well as some specific requirements for the different phases of a construction project.

It is a prerequisite that the readers possess fundamental competence and have their own detailed instructions, guidelines, etc., for the implementation of specific construction initiatives.

These guidelines contain recommendations on best practices, however the Norwegian Coastal Administration does not take responsibility for the use of the book or the consequences of the book's content, in full or in part, being used as a basis for specific measures, and it cannot be deemed to comprise any official standard or similar fundamental guidelines.

## 2.2 HISTORY OF BREAKWATER CONSTRUCTION

### 2.2.1 "A harbour where there previously has not been a harbour"

The history of breakwaters stretches back 4500 years in time, and they are associated with the first civilisations that began to use the maritime routes for transporting goods and commodities. The need to protect harbours by building structures to protect against waves arose in connection with the founding of ancient towns, where the need for safe harbours was crucial. The oldest dated breakwater structure was discovered in Egypt in connection with archaeological excavations of the old harbour town Wadi al-Jarf on the Red Sea, and has been dated to 4500 B.C. The breakwater is 150 metres long and is visible today at low tide.

Along the Norwegian coast, the first harbours were naturally protected from the weather and wind, and the ships that were used possessed the advantage that they could be pulled onto the land and required little depth. The first historical source we have of breakwater construction along our coast is from the 1100s. King Øystein's Harbour at Agdenes at the entrance to Trondheim Fjord had a military purpose. A breakwater was built of a cobwork pier, which was filled with stones. The breakwater had a length of approximately 30 metres and a width of 4.5 metres. The site is mentioned in the saga literature and in the "Sons of Magnus Saga" it is stated that King Øystein built "a harbour where it previously had been harbourless". The Viking harbour at Fånestangen in the Municipality of Frosta is somewhat older, from around the year 1000. This archaeological find consists of 12 to 14 tree trunks and was possibly a box structure filled with stones, but whether this was a pier or a breakwater is not possible to ascertain. The oldest traces of maritime infrastructure associated with harbours are stored today in the Museum of Science in Trondheim, whereas the remnants of the breakwater at Agdenes are still visible in the form of timber and stone structures that can be seen at low tide as well as large stone blocks that the sea has spread across the tidal basin and which markedly distinguish themselves from the natural stones in the area. Norway's oldest breakwater is thus possible to visit, and the area is open to the public.

*Figure 2-1 Breakwater construction at Andenes in 1902.*

### 2.2.2 The seeds for a state harbours agency

The first breakwater in Norway was in fact built by the ruling king, but the building of harbours and arrangements for harbour-related enterprises were from the Middle Ages and up to the 1700s something that private sector merchants were responsible for. The Crown stimulated the establishment of mooring rings, by granting privileges and toll rights to parties who built them. It was only first after the establishment of the State Pilot Service that the State intervened in order to improve conditions at the harbours. In a simple ordinance that was adopted on 16 September 1735, the seeds of a state harbours agency were sown. The ordinance established that every harbour must have a Harbour Commission, which must report to the Vice-Roy (under the then ruling King in Copenhagen). In the decades that followed, harbour initiatives were implemented with financing through dredging fees, and in 1756 two such breakwaters were built in Larvik. The cost of these was 3000 "riksdaler", an amount so high that Larvik had to borrow half of the funds from the neighbouring harbour treasuries of Kragerø, Skien and Drammen. The "Skottebrygga" breakwaters were built of wood, but were covered by stones on its outer side and up to the "waterline". They are partly visible today. In 1730 and later in 1785, revetments were built for the entrance to Trondheim Harbour.

### 2.2.3 The Norwegian Directorate of Harbours is founded

The Directorate of Harbours was established in 1841 under the Ministry of Naval Affairs. Initially, the Directorate's responsibilities were control and the exercise of authority on an overall level, as well as representation in the municipal harbour administrations. The Directorate was also active as a harbour developer and one of the first state-sponsored breakwaters built by the Directorate of Harbours was a breakwater in Ålesund. This was a masonry breakwater, known by its popular name "Molja" It was built in 1854–55.

Only when Harbour Director Oluf Roll assumed that position in 1861 (remaining in the position until 1897) was a special contracting department established for building harbours for the fisheries. Whereas during the period of 1842–72, 30 port facilities were established, during the subsequent 40 years a staggering 300 new ones were built. The annual budgets increased nearly tenfold during this period. Norway was in the middle of nation-building and the economic boom laid the foundation for substantial investments in public infrastructure.

A crucial factor in this was the creation of the "Harbour Fund" in 1873. This was built up through a legally mandated export fee on fish products. Use of money from the fund was earmarked for fisheries-oriented development (fishery

Silda Breakwater, Sogn og Fjordane

harbours). Up to 1908 the fee was collected from Rogaland on the south-west coast and northward, and therefore projects with support from the fund could only be realized in the same area The State contributed money, for a long time 1/3 of what was taken from the fund.

In total, nearly 800 State fishing harbours have been built.

The State fishing harbours today are spread out along the entire Norwegian coast, and vary considerably in size, location and intensity of use. Many different actors have interests in the fishing harbours. The Norwegian Coastal Administration is naturally one of the central actors, both as the executor of the State's property interests, as well as being the governmental administrative authority pursuant to the Harbour and Fairways Act.

Today, the Norwegian Coastal Administration Museum is responsible for documenting and making public Norwegian harbour history.

### 2.3 STATUTES AND REGULATIONS

The following statutes with their associated regulations contain provisions that must be addressed and/or evaluated when a breakwater is to be planned or established

- Act relating to Planning and the Processing of Building Applications (LOV-2008-06-27-71 the Planning and Building Act – PBL)
- Act concerning protection against pollution and concerning waste (LOV-1981-03-13-6 the Pollution Control Act)
- Act concerning the cultural heritage (LOV-1978-06-09-50 the Cultural Heritage Act)
- Act relating to the acquisition and extraction of mineral resources (LOV-2009-06-19-101 the Minerals Act)
- Act relating to harbours and fairways, etc. (LOV-2009-04-17-19 the Harbour and Fairways Act)
- Act relating to the Management of Biological, Geological and Landscape Diversity (LOV-2009-06-19-100 the Nature Diversity Act)
- Act relating to Working Environment, Working Hours and Employment Protection, etc., (LOV-2005-06-17-62 the Working Environment Act)

This is not any exhaustive list, and in the project planning and execution of breakwater work an assessment must always be made of whether other laws in addition to the above-mentioned should be addressed.

Mehamn Breakwater. Photo GeoNord

## 2.4 TOP-LEVEL PLANS AND GOVERNMENTAL PERMITS

### 2.4.1 National and regional plans

If the harbour concerned is included in top-level plans, then fundamental plan clarification is performed on this level. If this does not exist, then a clarification must be done with national and regional authorities concerning what is needed in terms of decisions, plans, etc. This will depend upon whether it concerns the establishment of a new harbour, expansion of an existing harbour or purely the shielding of existing harbour conditions.

### 2.4.2 Municipality land-use plans

Fundamental area clarifications need to be done at the municipality land-use plan level. Before a breakwater structure can be implemented, the municipality's land-use plans must contain areas set aside for the harbour, fairways, etc. If this has not been incorporated into the municipality's land-use plan's area division before the planning for the breakwater commences, then this ought to be included in the next roll-over of the plan. As a part of the breakwater planning, the requisite plan clarifications must be undertaken, including involving the relevant sector authorities.

### 2.4.3 Development plans

Detailed plan relationships must be clarified. If no development plan already exists for the initiative, then a plan description must be made. An impact assessment (Norwegian: Konsekvensutredning - KU) or a reduced scope KU may be required. Furthermore, a risk and vulnerability assessment must be performed, cf. section 4-3 of the Norwegian Planning and Building Act. It is recommended that the planning area be somewhat larger than the area that will be covered by the project. This is particularly useful in the early stages of the plan.

For the scope of plan explanations and documents, refer to PBL, the planning part, as well as the Norwegian Regulations for Impact Assessments.

Construction projects that usually require a detailed development plan are a) the establishment of breakwaters, b) quarries and c) areas for temporary storage and sorting of different stone fractions.

### 2.4.4 Governmental permits

The developer or entity tasked with carrying out the initiative (project owner or project manager) must apply for permits for the initiative to the following authorities:

- The municipality where the site is located – permit pursuant to the Planning and Building Act.
- Norwegian Coastal Administration – permit pursuant to the Harbour and Fairways Act (possibly the regional/local port authority if the initiative is within their area of responsibility).
- County Governor (possibly the Norwegian Environment Agency) – permit pursuant to the Pollution Control Act.

Gryllefjord. Photo: Istak

If the sea bottom is classified as contaminated, then possible capping of the contaminated sea bottom or the dredging and deposition of contaminated sediments must be included if this is relevant.

For dredging of contaminated sediments, the permit application will normally contain a plan for deposition and a deposition location, plus the requisite investigations in connection with this must be expected (see the Handling Guidelines, ref. X).

Fill permits are granted with a legal basis in the Pollution Control Act, section 11, whereas the Pollution Control Regulations, chapter 22, govern dredging and dumping.

The Norwegian Environment Agency has issued a factsheet, M-1085 | 2018 "Problems with plastic in fills of quarried rock in the sea", which names 11 requirements that quarried rock producers and fill projects ought to satisfy in order to reduce contamination with plastic in the sea. In applications for permits for quarried rock fills in the sea, a plan ought to be given for how they are intended to be carried out. It is also recommended here that an explanation be included concerning the reduction of other contamination with plastic in the sea, cf. ref. X.

Moreover, procuring permits from the following authorities may be relevant:

- For initiatives in national parks: The Board/Central Office of the relevant national park
- Protected areas: The Norwegian Environment Agency or regional environmental authority (county)

Statements from the following authorities must be procured:

- Cultural heritage authority – regional cultural heritage authority (county or regional museum as per duly delegated authority for marine cultural monuments)
- Sami Parliament of Norway (for initiatives within its jurisdiction)

Opening and operating a quarry requires a permit from the Norwegian Directorate of Mining. The operating plan and decommissioning plan must be included with the application.

The Nature Diversity Act must be evaluated in the case processing for the permits mentioned above, especially by the relevant municipality and the relevant county governor. Hence compliance with requirements pursuant to this Act must be described in applications to said authorities.

This may also be necessary for other permits. The precise scope of the applications to be made must be clarified in each individual instance.

Eggum Harbour, Lofoten

# 3. CLASSIFICATION OF BREAKWATERS

### 3.1 INTRODUCTION

A breakwater is in principle an embankment using a heavy material that is laid out in the sea. This concept also includes *revetments*, i.e. an embankment from land and out into the sea such as a roadway embankment in the sea or an industrial area. Breakwaters can in general be classified as shown in figure 3-1:

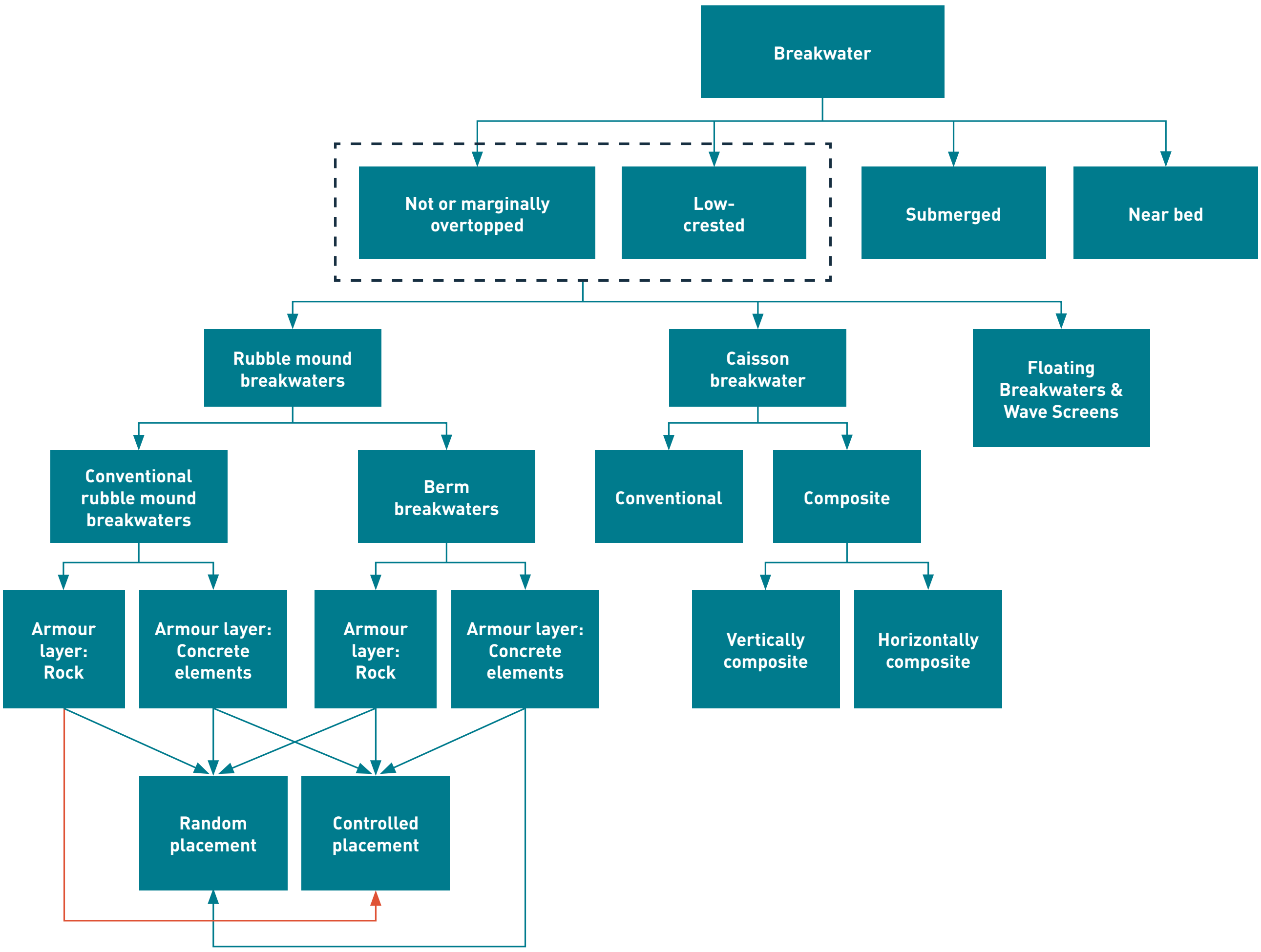



*Figure 3-1 Overview of different breakwater types.*

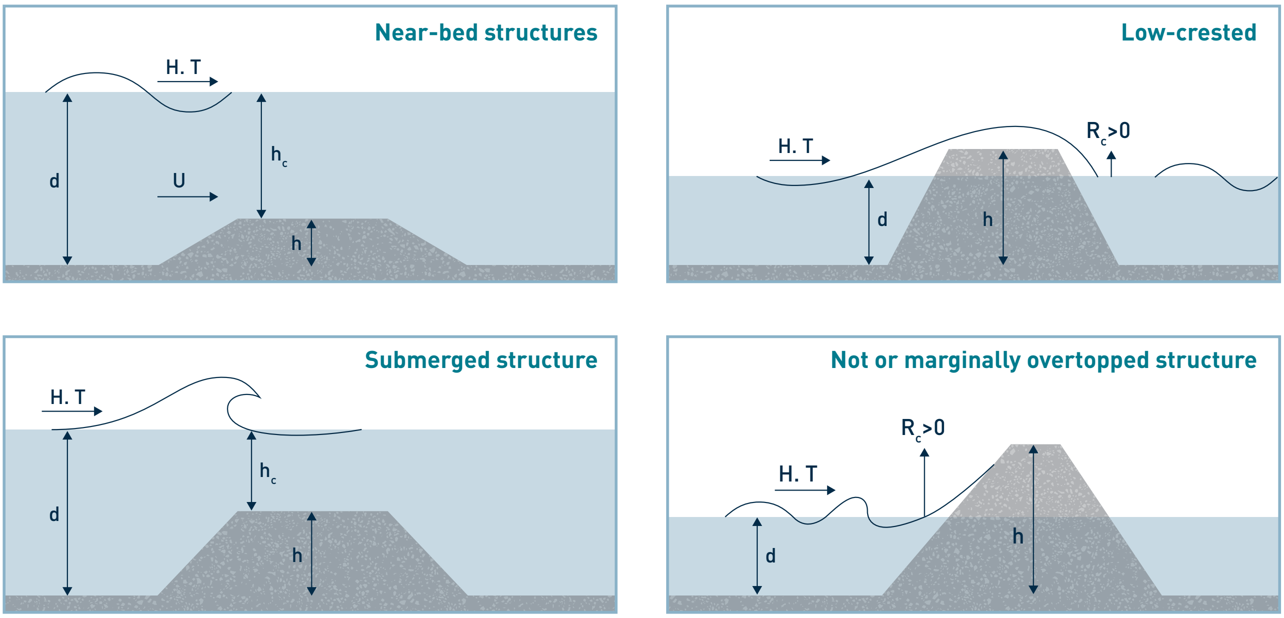



*Figure 3-2 Classification of breakwaters based on crest height.*

Breakwaters can also be classified based on their height as shown in figure 3-2:

- Near-bed
- Submerged
- Low-crested
- Not or marginally overtopped structure

These guidelines primarily address breakwaters with no or marginal overtopping.

Breakwaters are also classified with respect to the following main categories (see figure 3-3):

1. Rubble mound breakwater, with the subcategories conventional rubble mound breakwater and berm breakwater.
2. Caisson breakwater with the subcategory composite breakwater
3. Floating breakwaters
4. Wave screens

Floating breakwaters and wave screens are only used in wave-protected areas with wave periods of less than approximately 3–5 seconds.

A conventional rubble mound breakwater is either covered with large rocks blasted out at a quarry or with concrete elements. Where there is access to good-quality rock, it is common to use rock elements as the armour layer, whereas concrete elements have been used in Norway where good rock is not available or other limitations exist (such as bridge and road capacity).

Different designs have been used for concrete elements. In the selection of such elements, it is important to place an emphasis on simple casting methods and improved stability by the elements interlocking into each other with the greatest possible effectiveness.

In historical terms, the conventional rubble mound breakwater is the type that has been the most common in Norway. In recent years, berm breakwaters have become more common, especially in countries such as Iceland and Norway. The berm breakwater has the advantage that the armour layer elements are lighter than what is necessary for a conventional rubble mound breakwater at the same site. In addition, different categories of stones may be used for the armour layer, allowing better utilisation of more stone quarries. Hence the extraction of rock from a quarry becomes more economical. The rocks become easier to handle and the number of suitable rocks increases, and simpler and more basic equipment can be used.

A berm breakwater is also a more "robust/tough" structure in comparison with a conventional rubble mound breakwater, which is deemed to be "more brittle/fragile". This can be explained using figure 3-4. This recommended practice addresses rubble mound breakwaters with rock armour elements that are placed in a controlled manner. Both conventional and berm breakwaters are considered. The figure shows that a berm breakwater, to begin with, can be reshaped, but subsequently it survives even beyond the design sea state without considerable damage. In contrast, conventional rubble mound breakwaters, especially with armour units of concrete elements, exhibit quite robust behaviour up to the design sea state, at which point significant damage will quickly arise that in turn may lead to a collapse of the breakwater.

In the same way as for conventional rubble mound breakwaters, concrete elements can also be used in a berm breakwater. The placement of the rock or concrete elements can be arbitrary or done in a controlled manner. Notice that modern concrete elements usually come with a detailed placement procedure specified by the supplier and patent holder. Controlled placement will usually involve better interlocking of the elements with each other, and thus reduce the risk of damage. On the other hand, this increases the complexity of the setting out of the elements and thus affects the construction time.

These guidelines address breakwater structures that are built with quarried rock in accordance with the "controlled placement" or interlocking principle, used on conventional breakwaters and berm breakwaters. Other types of breakwaters (such as concrete elements, caisson breakwaters and floating breakwaters) are mentioned, but not addressed in detail.

Table 3-1 shows an overview of construction methods for breakwaters and embankments. These guidelines only address breakwaters that are built in accordance with Method III controlled placement. Note that most international publications concern "loosely laid, rubble mound structures", i.e. more akin to Method II Random Placement.

Norwegian breakwaters are for the most part built, and should be built, according to the description for Method III Interlocking surfaces. Applying standard formulae developed for "loosely laid, rubble mound structures" on breakwaters with interlocking quarried rock, will provide extra safety against failure. This increased safety will compensate for the fact that Norwegian breakwaters are traditionally built at much steeper slopes than is common internationally (1:1.3 vs 1:1.5).

| DESIGNATION | DESCRIPTION | REQUIREMENTS SPECIFICATIONS | AREA OF APPLICATION |
|---|---|---|---|
| **I. Random placement** | The rock material is discharged from a dumper/tipper truck. Subsequently reworked *only if necessary to do so* in order to ensure the correct slope and take down steep sections and overhangs. | • Block size<br>• Layer thickness<br>• Slope | Shielded fjord areas without impacts of swell, harbours, channels<br>$H_s$: 0–0.5 m<br>$T_p$: 0–4.0 s |
| **II. Random placement with surface treatment** | The rock material is discharged from a dumper/tipper truck. Subsequently reworked in order to ensure the correct slope and take down steep sections and overhangs.<br><br>The surface is arranged in order to ensure an even surface without visible holes or protruding elements. | • Block size<br>• Layer thickness<br>• Dead load<br>• Slope<br>• Smoothness of surface<br>• Aesthetics | Shielded fjord areas with only weak swell, harbours<br>$H_s$: 0.5–1.5 m<br>$T_p$: 0–6.0 s |
| **III. Controlled placement** | Selected rock elements must be individually placed in a locking pattern on a prepared underlayer of dimensioned filter stones. The blocks must be lifted from the top, and placed from the bottom going up. Strict requirements for block weights, block shape and dead load. | • Block size (also the underlayer)<br>• Layer thickness (also the underlayer)<br>• Dead load<br>• Slope<br>• Smoothness of surface<br>• Aesthetics<br>• Block shape<br>• Orientation of the blocks | Open fjord areas and places exposed to swell or sea waves<br>$H_s$: > 1.5 m<br>$T_p$: 0–20 s<br><br>Most used for breakwaters and embankments in exposed areas; several different construction methods |
| **IV. Masonry** | Custom-made and fitted blocks are stacked in bonded sections with a near-vertical slope (5–10:1). Only minimal openings between the blocks are allowed. Strict requirements for block size and block shape. | • Block size (also the underlayer)<br>• Layer thickness (also the underlayer)<br>• Dead load<br>• Slope<br>• Smoothness of surface<br>• Aesthetics<br>• Block shape<br>• Orientation of the blocks | Previously much used for unsheltered areas, now mostly used for aesthetic reasons in sheltered areas |

*Table 3-1 Overview of methods for building breakwaters, embankments and mounds in the sea.*

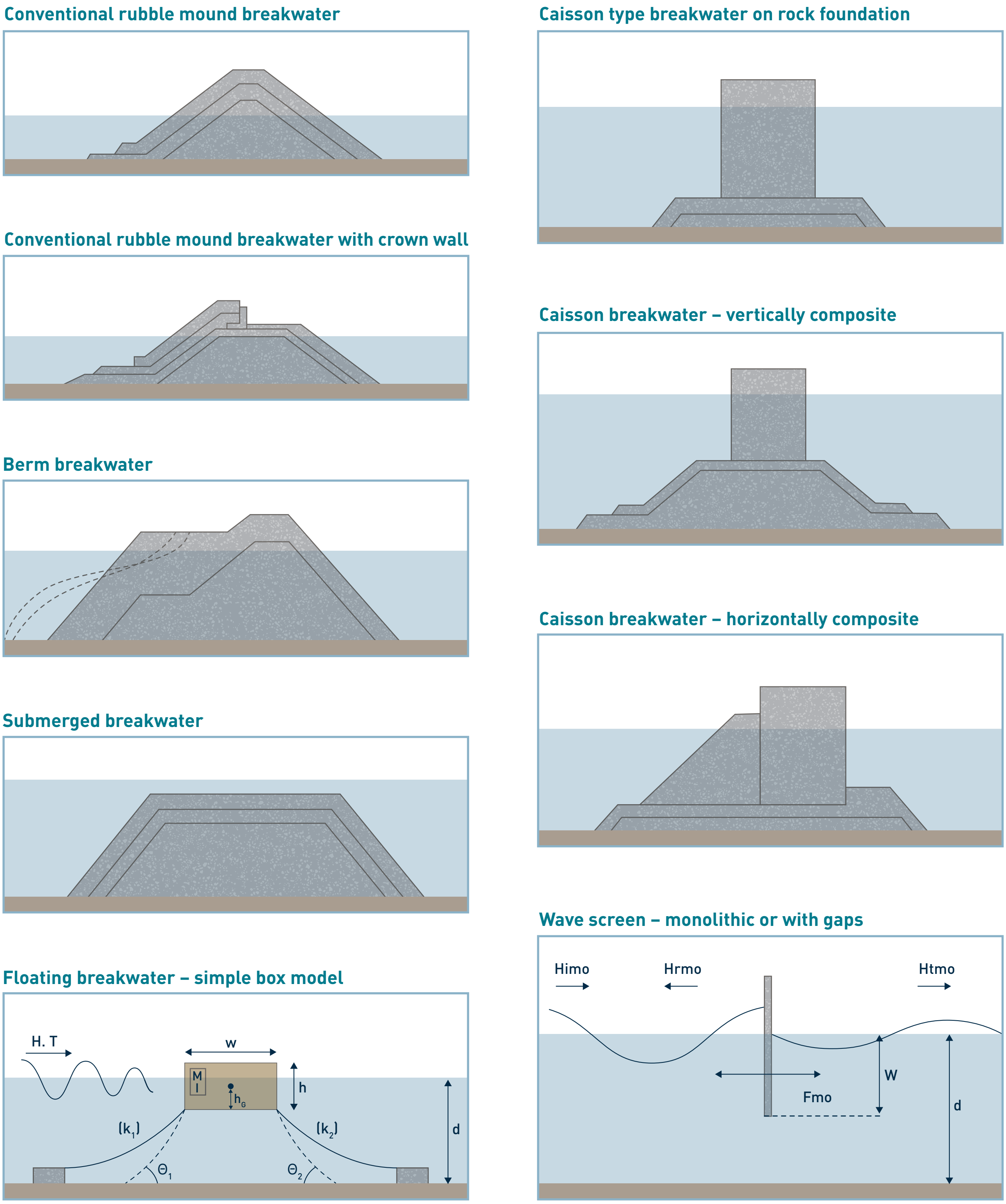



*Figure 3-3 Examples of cross-sections of different types of breakwaters, (Ciria, 2007, translated).*

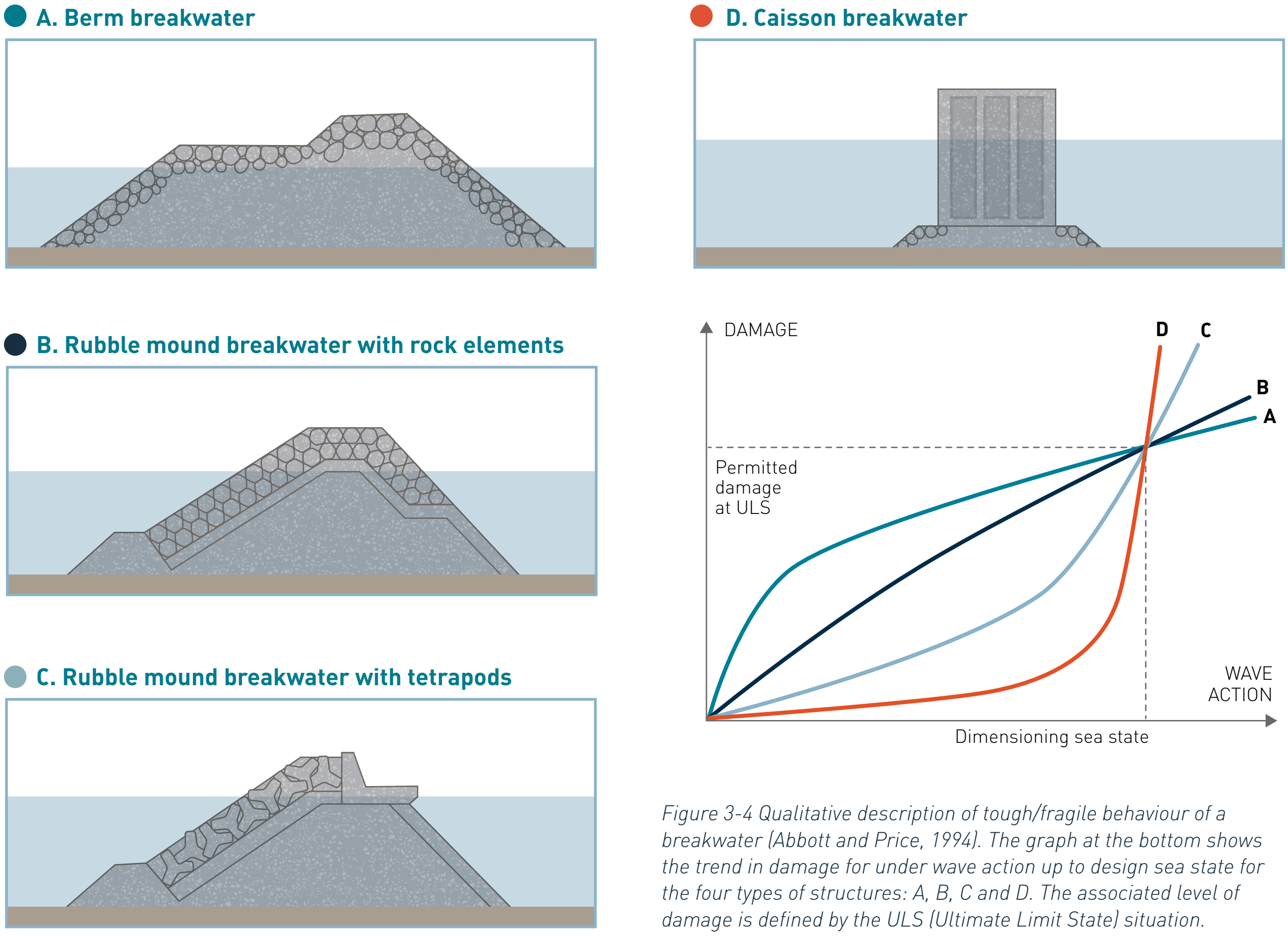



*Figure 3-4 Qualitative description of tough/fragile behaviour of a breakwater (Abbott and Price, 1994). The graph at the bottom shows the trend in damage for under wave action up to design sea state for the four types of structures: A, B, C and D. The associated level of damage is defined by the ULS (Ultimate Limit State) situation.*

Breakwaters can also be categorised with respect to their hydraulic behaviour.

Waves are a form of mechanical energy that propagates in the sea in an efficient manner.

When waves hit against a breakwater, the wave energy will be changed due to:

1. Dissipation (turbulent flow in the armour layer and filter layer as well as breaking)
2. Transmission (including overtopping)
3. Reflection

The total energy is conserved (dissipation is a transition to heat). Breakwaters can be classified based on how these three energy components are distributed.

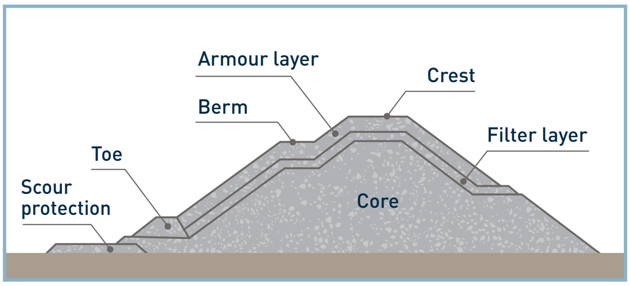
A berm breakwater will attenuate wave energy via the turbulence that is created between the rocks and waves that are breaking. The reflected energy from berm breakwaters thus is relatively low. A near vertical and completely impermeable wall will not dissipate any wave energy, and almost 100 % of the energy is reflected. In principle, a conventional rubble mound breakwater behaves in the same way as a berm breakwater. Because this breakwater type is less permeable and has steeper slopes, its capacity to dissipate wave energy is less, and the reflection of wave energy is greater than for berm breakwaters. Other parameters for the armour layer such as thickness, rock size and the front slope of the incline are also crucial in defining this property. In addition to the armour layer, both the filter layer and the core will also play an important role in the hydraulic behaviour of the breakwater because they contribute to attenuating waves by creating turbulence and increasing the pore pressure inside the core materials. An impermeable core will provide less transmission, but more reflection and more overtopping.

### 3.2 RUBBLE MOUND BREAKWATERS

#### 3.2.1 General

A rubble mound breakwater is a type of breakwater where the armour layer consists of individual units distributed on a slope facing the incident waves. These units resist the wave forces using their weight and by being interlocked into each other, where the weight of each individual element is the most important design parameter. These elements are presumed to each be statically stable on their own, which in principle means that the elements will not move under design conditions. A somewhat limited level of damage will usually be permitted, this level of damage should be defined before the design phase.

Figure 3-5 shows the most important components in a general design for a rubble mound breakwater, and table 3-2 describes the function of each component.



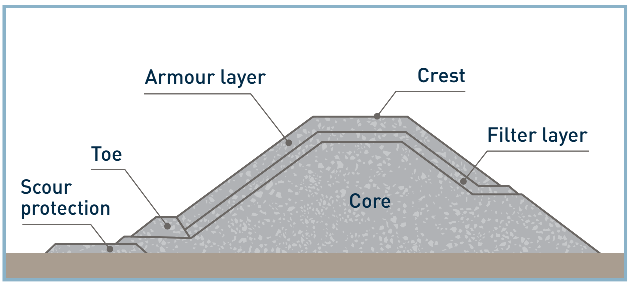




*Figure 3-5 Schematic outline of a rubble mound breakwater with and without berm*

| COMPONENT | FUNCTION |
|---|---|
| **Scour protection** | • Prevents erosion and undermining of the breakwater toe |
| **Core** | • Reduces wave transmission<br>• Foundation for armour layer and filter layer<br>• Contributes to geotechnical stability |
| **Berm** | • Reduces wave action, surges and overtopping<br>• Contributes to increased geotechnical stability |
| **Toe** | • Provides stable support against slipping of armour layer |
| **Filter layer** | • Acts as a filter<br>• Protects the core from erosion<br>• Contributes to improved drainage<br>• Forms a good foundation for placement of the armour layer elements<br>• Separates the armour layer from the finer core material and reduces the hydraulic pressure gradient in the core |
| **Armour layer** | • Prevents wave-induced erosion of the underlying materials<br>• Dissipates wave energy<br>• Stable layer that also reflects some of the wave energy |
| **Crest** | • Reduces wave overtopping<br>• Provides access for maintenance |
| **Crown wall** (not shown in the figure) | • Reduces overtopping<br>• Provides safer access for maintenance<br>• Provides a base for facilities such as cabling and pipelines |
| **Roundhead** (not shown in the figure) | • Terminates the breakwater as a stable structure<br>• Contributes to dispersing of the waves (diffraction) |

*Table 3-2 Function of the components in a rubble mound breakwater.*

### 3.2.2 Conventional rubble mound breakwaters

Conventional breakwaters are as mentioned above, the most common type of breakwater internationally. They are simple to build if there is a quarry nearby offering the potential to extract sufficient rock of the required quality. This type of breakwater continues to be most relevant to places where the wave action is low or moderate.

There are many variants of the cross-section of a conventional breakwater. Most have multiple layers (armour layer, filter layer and core). The objective is to obtain an impermeable structure in order to minimise the transfer of energy and migration of fine-grained substances out of the core, but at the same time to attenuate waves by creating turbulence and increasing pore pressure inside the core.

The breakwater trunk is comprised of the core, filter and armour layer(s). The armour units constitute a relatively thin shield that protects the interior of the breakwater against the forces of the waves. The task of the filter is to separate the armour units from the core, so that the core is not washed out between the armour units.

Conventional breakwaters can easily be repaired, improved and modified if modifications or repairs are necessary. The breakwater behaviour is simple and there is ample experience with this type of breakwater.

### 3.2.3 Berm breakwater

A berm breakwater is a type of breakwater where the front facing the sea is shaped as an absorbing embankment in front of the actual breakwater. Whereas a conventional breakwater will have a relatively thin armour layer that functions as a shield against the waves, a berm breakwater will absorb the wave energy gradually in the hollow areas found inside the berm (shoulder). An example of a berm breakwater is shown in figure 3-5. An important point about berm breakwaters is that the berm itself must consist solely of large rock elements, and that no material that is less than $W_{Min}$ is permitted in the berm.

For the exterior surface on the berm, the same requirements as for conventional breakwaters apply (block shape and interlocking). The blocks must fulfil requirements to a specified weight range, density and shape.

In the interior of the berm, blocks with a shape that otherwise would make them unsuitable for interlocking surfaces may be permitted, as long as the requirements for weight and dead weight are maintained. The armour unit blocks within the berm must not be dumped and slid down, but should be placed individually. A certain degree of dumping from the surface is allowed.

The first berm breakwaters that were built at the end of the 1980s had a design with a broad and low berm. The intent was that the sea would shape the berm so that it would gradually acquire an S-profile. Such a structure was termed a reshaping breakwater that is dynamically stable. A reshaping berm breakwater could be built with much smaller rock elements than a conventional breakwater, but the drawback was that the blocks were subject to abrasion when they moved, and that the overtopping rates increased as the S-profile developed.

Today, a statically stable design is used, where the blocks are not permitted to move. Current berm breakwaters are characterised by a tall and narrow berm, where the block size is somewhat lower than in a conventional breakwater.

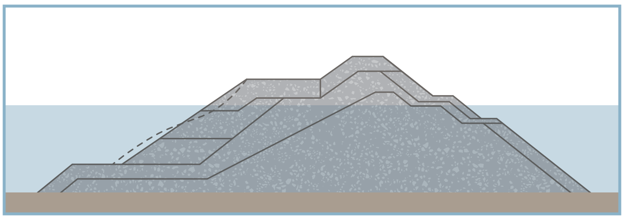
The advantage of the berm breakwater is that any damage will appear as a gradual reshaping, which allows its development to be followed and an optimum time to be selected for making repairs. If significant damage occurs to a conventional breakwater, it must as a main rule be repaired without delay to avoid accelerating damage and ultimate collapse.

Previously, berm breakwaters were classified into 3 categories; statically stable, reshaping into statically stable and dynamically stable. More recently, a new classification has been proposed (van der Meer and Sigurdarson, see ref. XVII) which is presented below in table 3-3:

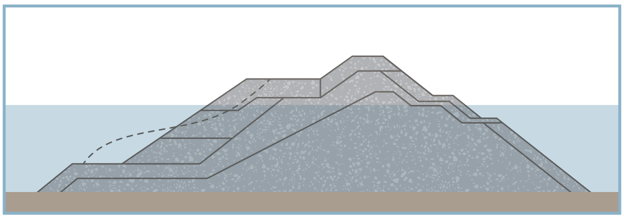
| CLASS | DESIGNATION USED IN NORWAY | PROPOSED INTERNATIONAL DESIGNATION |
|---|---|---|
| 1 | Ikke omformbar; Islandsk type | Hardly reshaping Icelandic-Type Berm Breakwaters: (HR-IC) |
| 2 | Delvis omformbar; Islandsk type | Partly reshaping Icelandic-type berm breakwaters (PR-IC) |
| 3 | Delvis omformbar; velgradert dekklag | Partly reshaping mass-armoured berm breakwaters (PR-MA) |
| 4 | Fullstendig omformbar; velgradert dekklag | Fully reshaping mass-armoured berm breakwaters (FR-MA) |

*Table 3-3 Classification of berm breakwaters.*

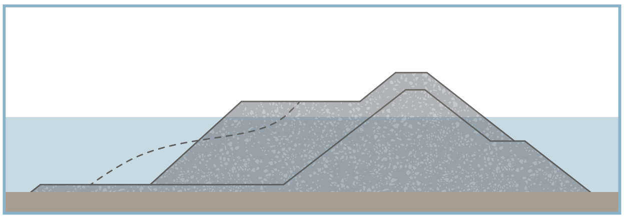
**Hardly reshaping; Icelandic type** **(Type 1)**



**Partly reshaping; Icelandic type** **(Type 2)**



**Partly reshaping; mass-armoured** **(Type 3)**



**Fully reshaping; mass-armoured** **(Type 4)**

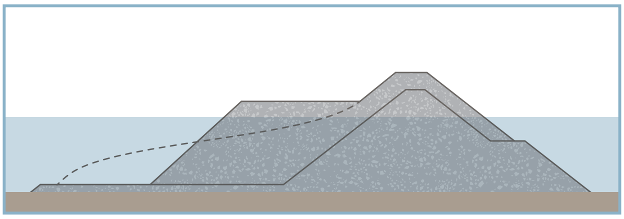


*Figure 3-6 The four types of berm breakwaters (van der Meer and Sigurdarson, 2017).*

Figure 3-6 shows cross-sections of these 4 types. This figure also shows the reshaping profile as a dashed line. For type 1, which is the type that will normally be used in Norway, we see that the reshaping should be minimal, and limited to a certain shake-down and natural repositioning of the blocks. For the other types (2–4), we see that more substantial reshaping is expected, which is generally not desirable for breakwaters in Norway.

Hardly reshaping Icelandic type berm breakwaters have been mostly used in Norway in recent years (statically stable design, where the blocks expected to move only slightly during extreme storms). Current berm breakwaters are characterised by a tall and narrow berm, where the block size continues to be somewhat smaller than in a conventional breakwater.

Berm breakwaters are used most in Norway and Iceland. The selection of the type of breakwater is a result of an economic analysis that will vary with the degree of wave exposure, access to rock and local parameters such as water depth, use, etc. In general, a berm breakwater should be considered where the design significant wave height $H_{s,dim}$ >3.0–3.5 m.

The Sirevåg berm breakwater is a good example of good design for a berm breakwater. Construction of the breakwaters was completed in 2001. It has a total length of 385 m. The breakwater is dimensioned for $H_s$=7.0 m, has a depth of 17 m and is implemented with armour units with weight $W_{50}$=20–30 t.

The breakwater in Sirevåg is dimensioned as a statically stable breakwater, however it has undergone more reshaping than was expected. The reason for the unexpected reshaping is unknown, but it may be due to uncertainties in the data material used for establishing the design wave height.

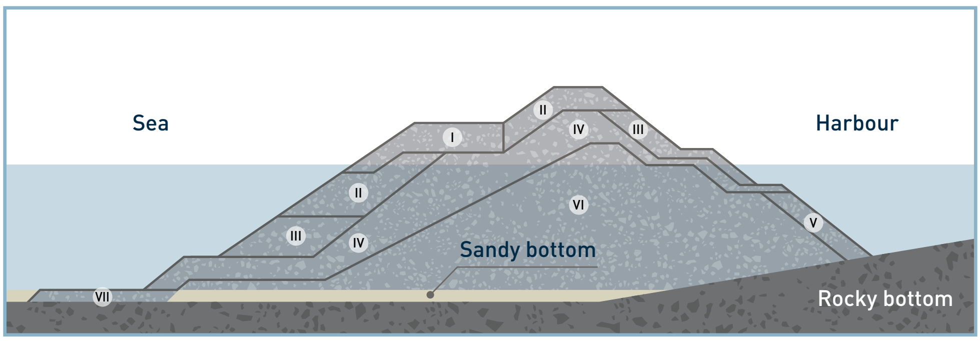
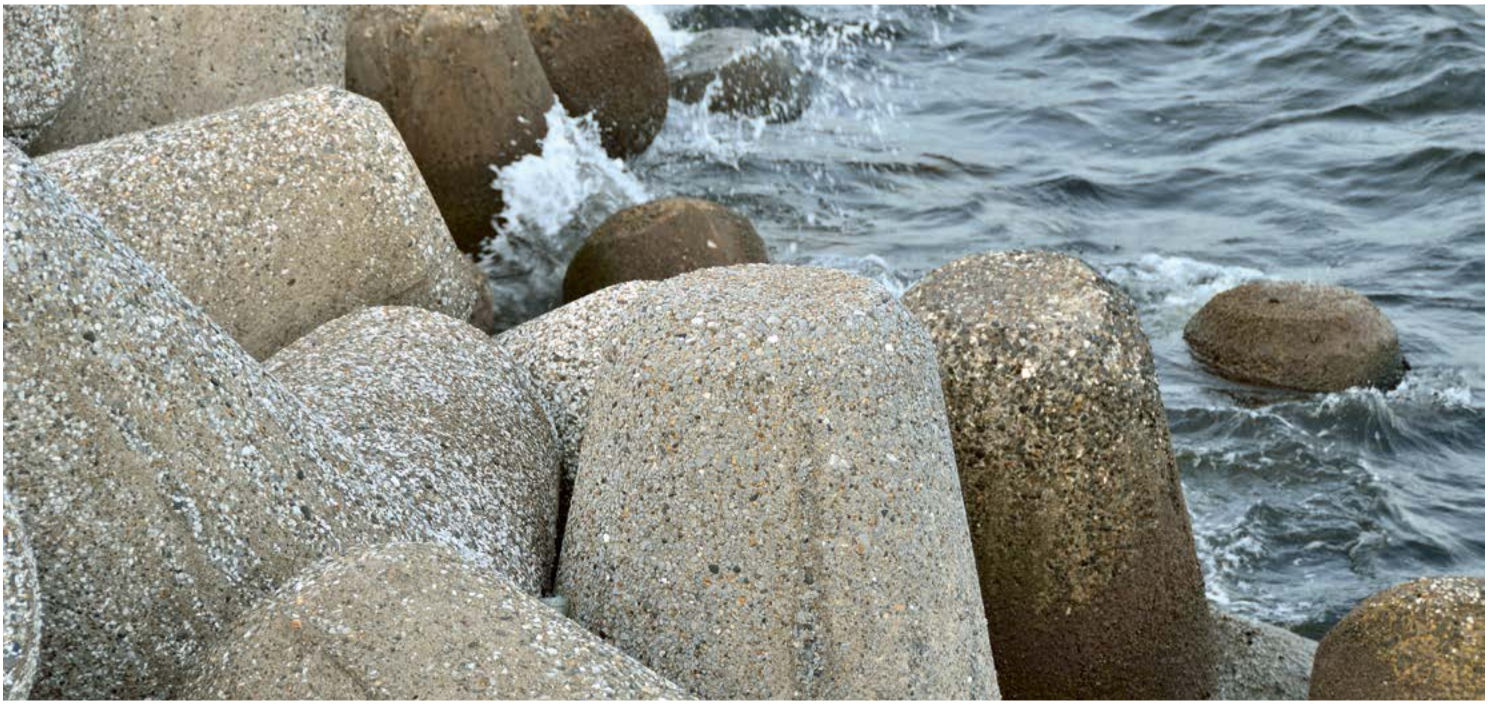



*Figure 3-7*
*Sirevåg berm breakwater*
*(I=main armour layer (20–30 t),*
*II= secondary armour layer*
*(10–20t)), (Ciria, 2007).*



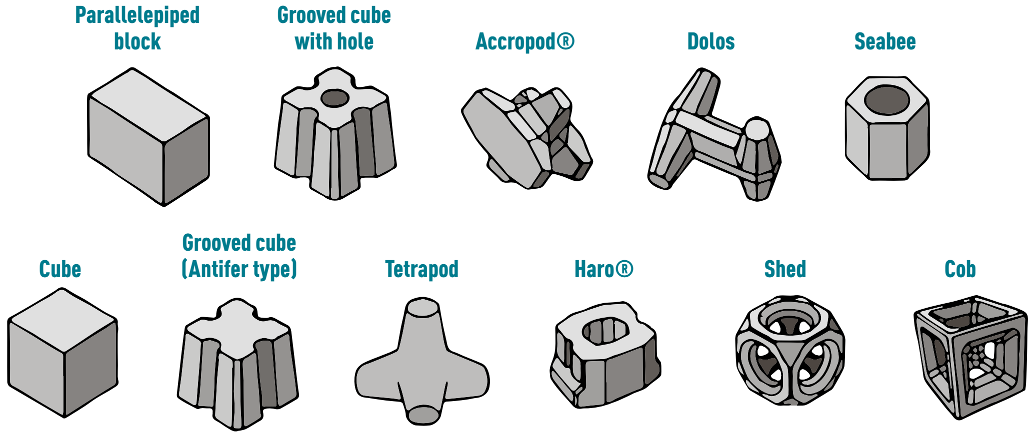



*Figure 3-8*
*Examples of concrete elements (Abbott and Price, 1994).*

### 3.2.4 Breakwaters of concrete elements

Breakwaters of concrete blocks are common abroad, but are rarely used in Norway. In Norway, these have only been used on the breakwaters in Ferkingstad in Rogaland (Karmøy) and in Berlevåg in Finnmark. With the good access to natural stone that we have in Norway, it is rarely economical to use concrete blocks. The exceptions may be found in special areas where the possibilities for transport of heavy elements is limited.

By choosing an appropriate shape for the concrete elements, the interlocking effect can be achieved. The resistance to wave action is then enhanced with more units locking in to each other.  A thoroughly thought-out shape and placement can thus enable the use of lighter elements. For some types of elements, the need for two layers in the armour layer can also be avoided (Accropode™, Core-loc™ and Xbloc®).

In the two concrete block breakwaters in Norway mentioned, a block type called Tetrapod has been used.

All the block types have or have had a patented design, and the patent owner has documented the performance and specified the requisite dimensions. For those block types that are relevant today, a fee must be paid to the patent owner when using them. This does not, however, apply for Tetrapod, because its patent has expired.

Concrete blocks have an ability to lock with each other so that their resistance to attacking waves is not dependent upon the weights of the blocks alone. The blocks thus may be smaller than blocks of natural stone. The cost will, regardless, normally be higher than for locally produced rock in Norway.

There are many types of concrete elements, where figure 3-8 shows some examples.

Breakwaters of concrete blocks are so rarely relevant in Norway that they will not be addressed in further detail here. It must be noted, however, that the two Norwegian concrete block breakwaters are performing well.

### 3.3 CAISSON BREAKWATERS

A conventional caisson breakwater consists of a vertical wall that resists wave forces. This type of breakwater is usually built of masonry with natural stone or concrete caissons.

Caisson breakwaters are typically used in deeper waters, where the use of a rubble mound breakwater would involve a large volume of rocks. Such breakwaters are also relevant on poor substrates or in instances where the lee side of the breakwater will be utilised as a pier.

The primary drawback associated with caisson breakwaters is their fragile behaviour and their requirements for design with a lower damage level and stricter safety coefficients. What is meant by "fragile" here is that any damage will arise instantaneously and lead to a more substantial or total loss of the structure, in contrast to a "tough" rubble mound breakwater, which only gradually incurs damage that can subsequently be repaired. Caisson breakwaters are vulnerable to erosion at the toe. Their erosion protection at the toe must be dimensioned in more detail than for other types of breakwaters. Caisson breakwaters reflect nearly 100 % of the wave energy, something that can be critical if there are vulnerable elements in front of the breakwater such as beaches, wharves, other breakwaters or the like.

It is possible to build a combination of a conventional breakwater and caisson breakwater, and this can reduce the costs as well as increase the resistance against local erosion at the toe of the breakwater.

### 3.4 FLOATING BREAKWATERS

A distinction should be made between floating piers, which are used as mooring facilities for smaller vessels, and floating breakwaters, which are built to attenuate waves.

- Floating piers are light structures that do not affect the waves to any substantial extent. Typical dimensions are width W ≈ 3.0 m, draught D ≈ 0.75 m. Such piers can attenuate ripples on the surface, and may to some degree serve to eliminate foam crests and breakers from incoming waves, but in general will not attenuate the waves to any significant degree. These are not addressed here in any further detail.
- Floating breakwaters are larger and heavier structures that have the purpose of attenuating waves, primarily through reflection. Typical dimensions are width W ≈ 4–6 m and draught D ≈ 2–3 m. They are delivered in elements that are 25–50 m long, and thus are so heavy and rigid that they hardly move in medium length waves (up to an approximately 4.0 s period, wave length approximately 25 m).

An illustration of the difference between the two types of floating structures is shown in figure 3-10, which is taken from a presentation by a supplier of floating breakwaters.

A floating breakwater is a floating structure that is moored to the bottom using piles, chains or other mooring systems. Waves that come in against a floating breakwater are partially reflected and partially transmitted under the breakwater. Part of the wave energy goes to setting the breakwater in motion. It is common to use submerged elements attached to the mooring lines (clump weights, drogues) that help increase the viscous damping.

Floating breakwaters have a transmission coefficient (ratio of transmitted wave height to incoming wave height) that is

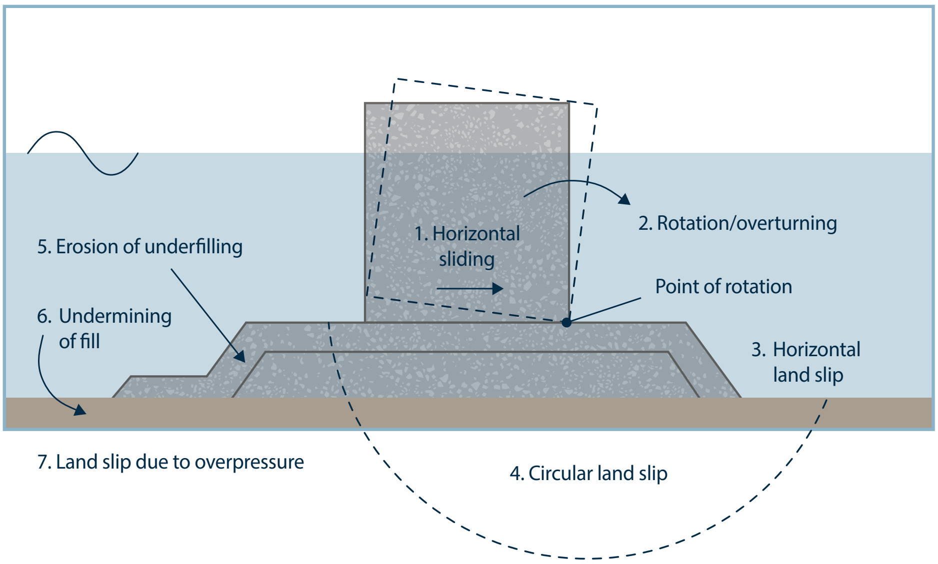



*Figure 3-9*
*Failure model for a caisson breakwater, (Ciria, 2007).*

primarily dependent upon the width, wave period and depth. Use of this type of breakwater is thus most justifiable in locations with short and low wind waves, for example inside a harbour. A floating breakwater will not have any special effect on swell with long periods. Such breakwaters are first and foremost used to shield small craft harbours and aquaculture facilities in areas with limited fetch lengths. Examples of such breakwaters are found in Molde, Bodø and Harstad.

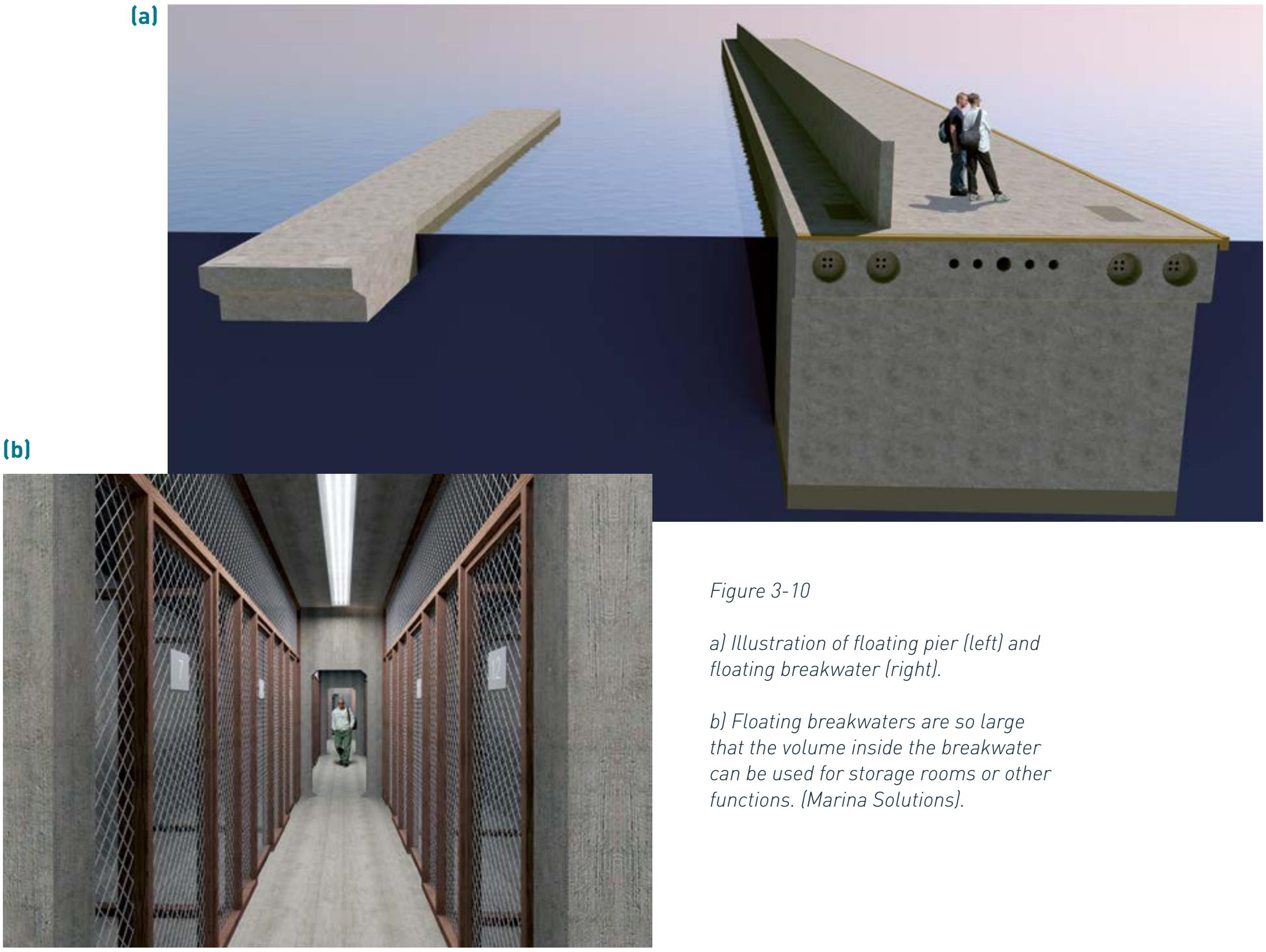



*Figure 3-10*

*a) Illustration of floating pier (left) and floating breakwater (right).*

*b) Floating breakwaters are so large that the volume inside the breakwater can be used for storage rooms or other functions. (Marina Solutions).*

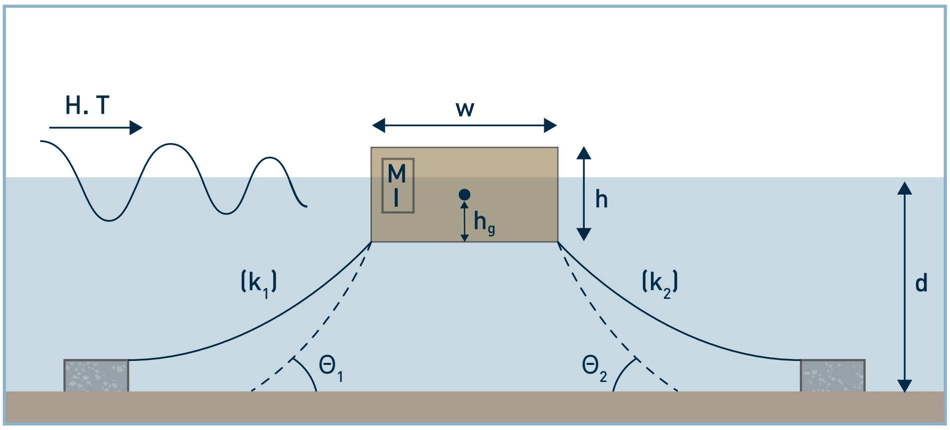



*Figure 3-11*
*Conceptual floating breakwater with main design parameters (wave height and period= H, T, width=W, height=h, depth=d, centre of gravity=hg and mooring stiffness (k,Θ)), (PIANC, 1994).*

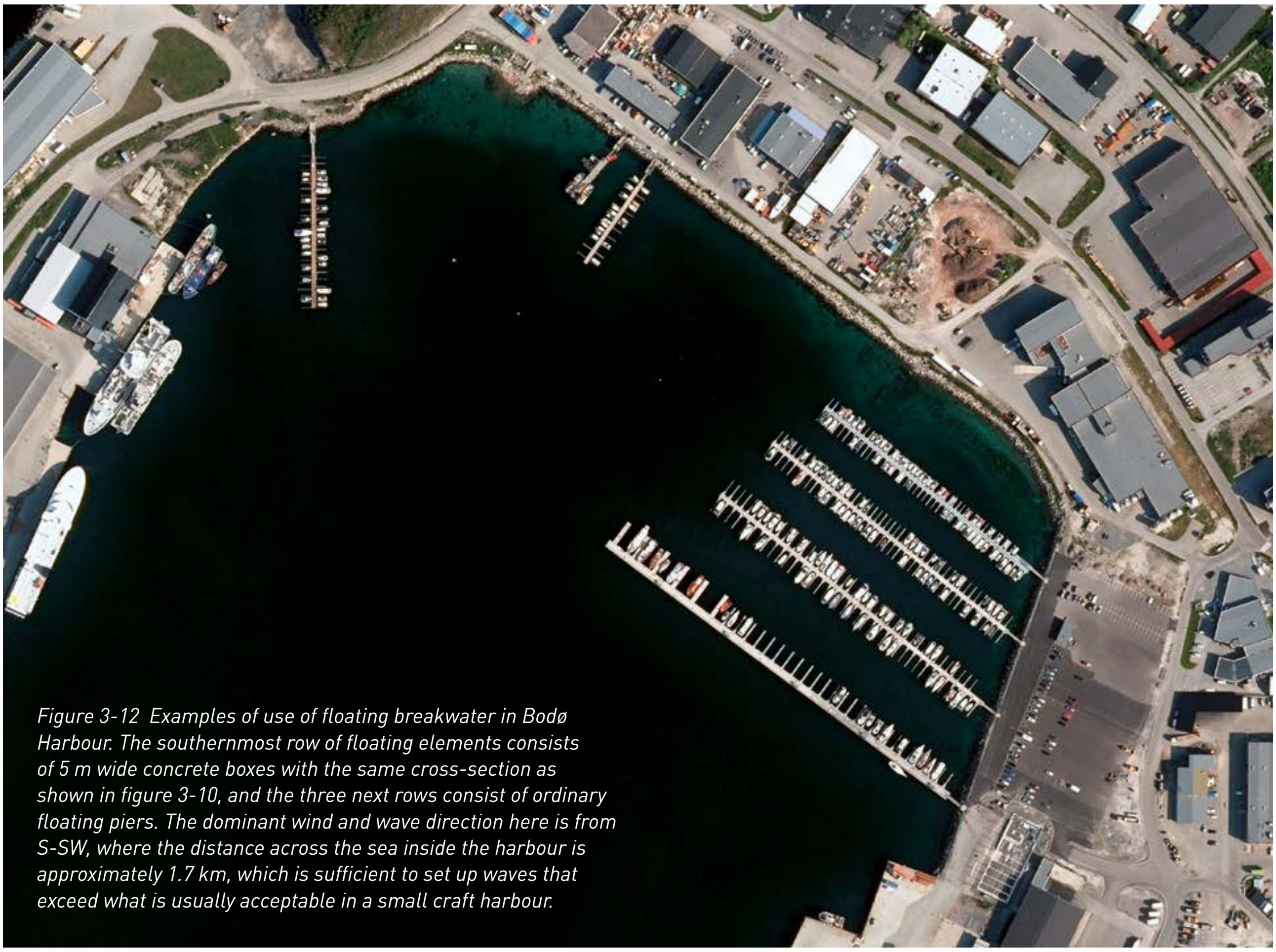


*Figure 3-12 Examples of use of floating breakwater in Bodø Harbour. The southernmost row of floating elements consists of 5 m wide concrete boxes with the same cross-section as shown in figure 3-10, and the three next rows consist of ordinary floating piers. The dominant wind and wave direction here is from S-SW, where the distance across the sea inside the harbour is approximately 1.7 km, which is sufficient to set up waves that exceed what is usually acceptable in a small craft harbour.*

The design of a floating breakwater must take into consideration movements in the floating structure and the resultant forces in the mooring systems. This is done most rationally using numerical models for wave-induced responses of floating facilities.

Use of floating breakwaters should be limited to areas where the waves are short wind waves up to $T_p \approx 4.0$ s, i.e. in closed fjords and harbour basins.

See more concerning the design of floating breakwaters in ref. XVIII.

*Figure 3-13 Example of simple floating pier (Norconsult). These are intended for mooring of smaller boats in sheltered areas, and ought not to be treated as or discussed as wave screens.*

*Figure 3-14 Example of floating wharf of unknown origin (Kirkenes, Norconsult). The structure is large and heavy enough that it will attenuate waves of up to approximately 4–5 s periods.*

## 3.5 OTHER TYPES OF BREAKWATERS

Other variants of breakwaters exist that are rarely used, where their primary objective is often to attenuate short wind waves behind the breakwater. Most of these types are experimental with little available experience that can be used in a reliable design. These types can consist of piles and fenders, submerged platforms, bubble curtains or automobile tires. Wave attenuation is achieved by creating turbulence in different ways.

## 3.6 PLAN LAYOUT

Breakwaters function best with their longitudinal direction parallel to the incident wave front. It is, however, common for the start of the breakwaters to be positioned perpendicular to the coastline in order to attain a certain depth. Some important aspects are discussed in the following and are summarised in figure 3-15.

The breakwater head is a possible weak point because the blocks here receive less locking and support from the neighbouring blocks. The head should be reinforced to prevent damage here. In some cases, the head (and only the head) will be designed as a berm breakwater. This gives the head increased strength, and contributes to reducing the wave energy that comes into the harbour.

Protruding (convex) corners are subject to the same weakening as the breakwater head. In many cases, such corners are also due to the centreline of the breakwater deliberately being placed across a skerry or a shoal in order to minimise the fill volume. It can lead to extra intense breaking at this point, which in turn may require reinforcement.

Recessed (concave) corners may be subject to concentration of wave energy and more substantial surges, which can also require reinforcement.

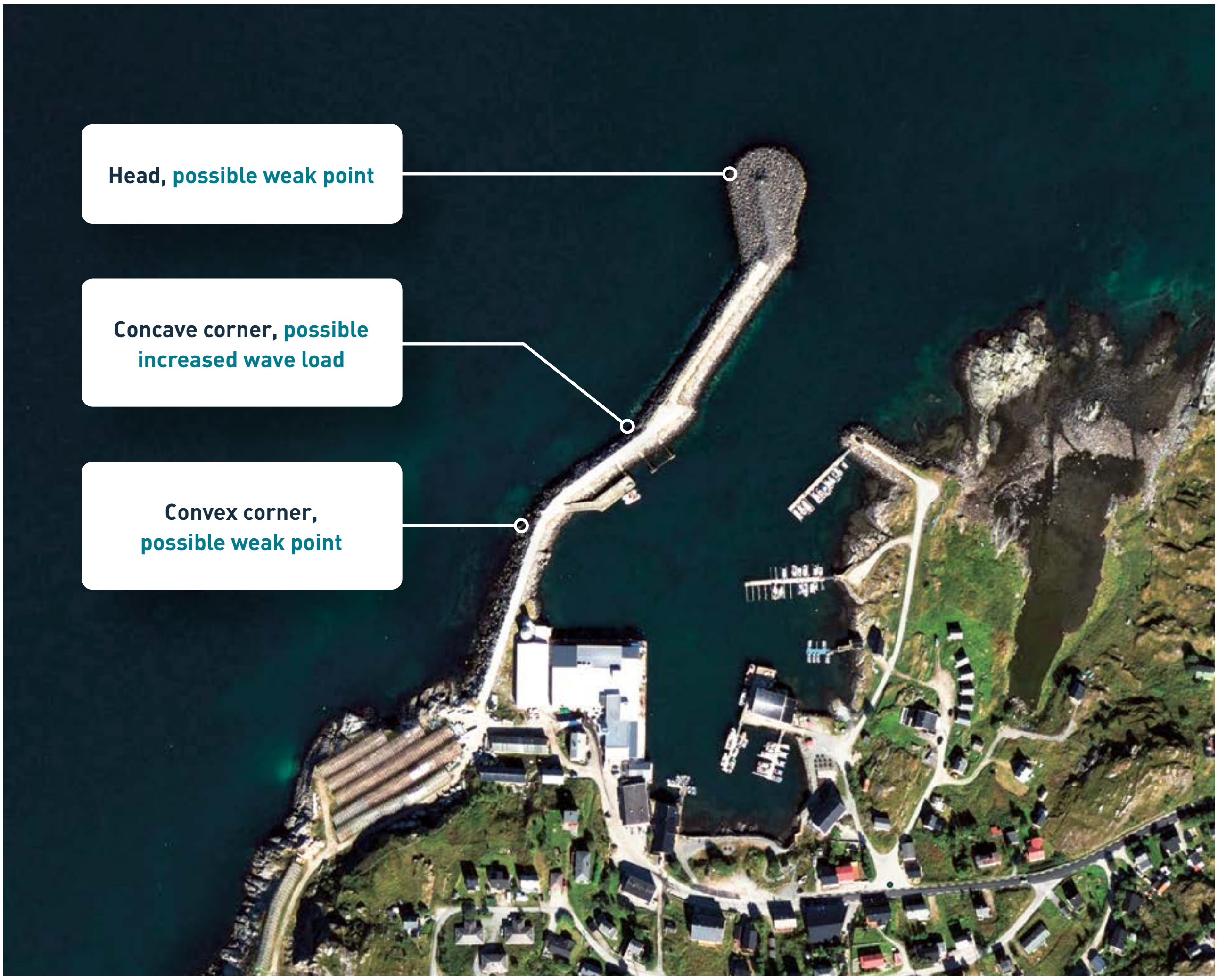



*Figure 3-15 Breakwater at Breivikbotn, constructed in three stages and terminated with a berm breakwater head. The irregular centreline is due to a desire to reduce fill volumes by utilising skerries and shoals.*

# 4. NEEDS ANALYSIS

The needs analysis is a necessary part of the benefit/cost analysis.

The objective of a needs analysis is to gain
- insight into the needs of the affected actors
- insight into potential conflicts of interest
- an introduction to what the actors and stakeholders envision they will achieve with the measure

The needs analysis must be performed during the early phase of the course of the project, before a decision is made on the selection of a solution. There may also be a need for more detailed needs analyses in later stages of the project planning, but these cannot replace those needs analyses that ought to be conducted during the early phase of the project.

The needs analysis is an important tool for finding the proper measure. It must also document why it is necessary to undertake the measure. In addition, the needs analysis is important for being able to say something about the effects of the measure. Needs analyses can be included in the basis for the decision to proceed with the project.

The needs analysis must capture all relevant societal requirements, not just the needs that are expressed through the willingness of individuals to pay. In particular, it will be important to take into consideration needs that are evidenced in overall, politically adopted national goals. Needs analyses that are conducted by the Norwegian Coastal Administration involve taking into consideration the goals in the National Transport Plan (NTP). A needs analysis must survey stakeholders and actors who are affected by the measure and what needs they have for the measure. The needs that are determined must be evaluated against the needs of the society. The needs analysis must on the overall provide answers to:

1. What is the problem?
2. How big is the problem?
3. Who owns the problem?
4. What is needed to solve the problem?
5. Positive and negative effects of the measure

To aid in finding the answers to the above questions, the methodology below can be utilised.

***What are the challenges of the current situation?***

Describe on the top level what would solve the problem without sketching out a specific solution
- Is the problem acute?
- Would it be more expensive to wait?
- Will opportunities be missed out on by waiting?

***Will the need persist?***
- How is the traffic expected to develop?
- What is the future composition of the traffic (ship types, sizes, etc.)
- What commercial development is expected?
- What is the expected trend in the population?
- What technological developments are expected?

***What will solve the problem for the actor?***
- Is a breakwater/deepening the only thing that can solve the problem?
- What placement/dimensions for a breakwater will solve the problem?
- What depth/manoeuvring area in the harbour solves the problem?
- Is any measure more important than others?

***Positive and negative effects – Suggestions for identification of effects***

What effects would the measure be able to have? Describe the positive and negative effects. Examples of effects:
- Cost reductions (waiting time, travel costs, altered wave and current-related conditions)
- Risk changes (groundings, collisions)
- Changes for the business community (waiting time, travel costs, productivity increases, cost changes, capacity increases)
- Increased traffic, and where would possible traffic increases come from? Transfers from other transport units
- Contaminated sediments/other materials
- Changes in:
  - fishing and aquaculture
  - outdoor life and recreation
  - natural environment
  - cultural monuments
- Establishment of new commercial areas

***Positive and negative effects of measures – Overall justification for implementing the measure:***

Why is there a need to commence implementation of a measure?
- Does it contribute to attaining a political objective?
- Does the measure solve a specific problem?
- Does the measure enable realisation of new possibilities?

Skarsvåg Breakwater, Finnmark

# 5. PLANNING AND PRELIMINARY INVESTIGATIONS

## 5.1 CLIMATE CHANGES

The concept of "weather" encompasses observations of how the state of the temperature, precipitation, wind, air pressure, etc. is at the moment or within a short period of some hours or of up to a day. Weather will vary throughout a year such that, for example, it is expected to be colder during the winter.

The concept of "climate" involves observations of the weather over a longer period within comparable time windows in an annual cycle, for example such as the average temperature in January for Ålesund in 2000–2010. If it is possible to see a trend in such observations across many years, it can indicate a climate change. Variations over some few years can be random and provide no reliable indications of climate changes.

Most researchers and the UN's Intergovernmental Panel On Climate Change, the IPCC, with backing from Norway, are working on the basis of a hypothesis that the planet is experiencing a climate change phase that involves an increased average temperature on the planet (but not necessarily all locations on the planet), and that the climate changes are to a large extent or completely due to human-created emissions of so-called greenhouse gases, where the primary focus is on $CO_2$ and methane.

It follows from laws and regulations that all permanent structures must take the effects of climate changes into consideration when design loads are calculated. The parameters that  are or may be relevant to breakwater construction are shown in table 5-1 together with the consequence for breakwater construction and expected trend up to 2100.

| PARAMETER | CONSEQUENCE FOR BREAKWATER CONSTRUCTION AND EXPECTED CHANGE |
|---|---|
| Wind | **Concerns wave-generating wind near the coast and inside fjords:** Probably little or insignificant change. |
| Waves | **Concerns waves generated in the open sea:** Probably little or insignificant change. |
| Storm surges and water levels | **Affects the point of attack of the waves at breakwaters and overtopping:** May in some locations involve substantial changes within the next 100 years with clear effects for breakwaters. |
| Precipitation | **The most probable change is increased intensity over short periods:** Probably little effect for breakwaters. |
| Temperature | **Concerns the danger of frost erosion of breakwater rocks:** The changes will lie within the current range of variations and will be of no consequence. |

*Table 5-1  Environmental parameters that are relevant for breakwater construction and possible changes resulting from climate change.*

**Wave heights.** Estimates of trends in wave heights in the open sea for the North Atlantic are provided by met.no (see ref. II). The report shows that it is possible that the wave heights (average value of $H_s$ and annual maxima) in the open sea will be lower towards 2100, but the trend is weak and we choose to neglect it.

**Local waves** near the coast and inside fjords are generated by local wind. No obvious trend has been observed or estimated in wind observations that indicate a significant change taking place in the wind speeds in Norway.

**Storm surges.** Storm surge are extreme, high water levels that are due to a combined effect of tides and other effects that govern the water level, such as air pressure, wind, inflows of fresh water, etc. It has been observed that the global average water level in the world oceans is rising, However the effect along the Norwegian coast varies due to varying land uplift (isostatic rebound), and because the earth's gravitational field will be affected if large quantities of permanent polar ice should melt and be distributed across the oceans. Storm surge is the parameter (of those that are mentioned in table 5-1) that is affected the most by climate changes.

**Precipitation and temperature.** Breakwaters are in general hardly affected by precipitation and temperature. Precipitation in combination with a subsequent strong frost may lead to frost erosion of rock elements, but it is not expected that any possible changes in temperature will affect breakwaters negatively. Climate changes will consequentially not lead to new challenges caused by precipitation and temperature.

The data in table 5-1 is specified here without further justification. Methods for determining water levels and storm surges are described in further detail in chapter 7.4.5.

## 5.2 CURRENTS

Currents are not a design factor for breakwaters under normal Norwegian conditions. The exception is breakwaters in rivers. A breakwater may, however, affect a current so that it changes its direction and strength, something that can lead to changes in the transport of sediments.

The construction of a breakwater can lead to a fairway being narrowed. Such a narrowing may lead to current velocities increasing locally in the fairway. Normally, this will not cause navigational problems because the current velocity will normally be in the same direction as the ship's course. In special cases, problems may arise, however, due to currents, where the currents result in vortices at the entrance to the port. Currents gradients inside a harbour can also cause problems for smaller vessels.

When breakwater construction leads to a narrowing of larger basins (>0.5 km$^2$) in areas with large differences in tides, strong currents can arise at the opening. Current velocities of up to 2.0 m/s (4 knots) are manageable for most vessels. Problems may arise, however, if the current in the opening meets a transverse current immediately outside the opening. Svartnes at Vardø is an example of such a harbour where crossing currents during shorter periods of time can make it difficult to steer a steady course in and out of the harbour.

Currents in the ocean are usually a combination of the following different types of currents:

- **Tidal currents** that alternate with the tides and arise due to gradients in water level. Such currents are strongly affected by local bathymetry and the coastline. Out on the open sea, such currents can be of an order of magnitude of some few cm/s, but in inlets the currents can reach multiple m/s. Tidal currents can be strong outside


Berlevåg, Finnmark

breakwaters due to the breakwater jutting out into the natural path of the flow of the current.

- **Wind-driven currents** that are caused by the friction between  air in movement (wind) and the surface of the sea. In areas near the coast this type of current is also affected by the local bathymetry and the coastline.
- **Density-driven surface currents** which are currents caused by horizontal variations in water density produced by changes in the salt content and/or temperature, which in turn are caused by an influx of freshwater draining from the land through river mouths, heat flows from coastal power generation plants as well as other causes.
- **Wave-induced currents.** These are currents near the coast that are caused by the horizontal variation of radiation stresses from waves.
- **Turbidity currents**. These are muddy currents that are due to the sliding of material on an inclined bottom. Such currents have the character of landslides and normally only arise once, but can be triggered by for example breakwater construction or dredging work.

Currents are important to breakwater design only to the degree they affect the waves in the vicinity of the breakwater and not as a current load. In areas with erodible sea bed, currents can be an important by removing sand that has been stirred up by the waves and deposit it far away from the breakwater structure. In the long run, this can lead to undermining of the breakwater structure (scouring) Possible scouring depths must to be evaluated and if necessary protection against erosion should be established. Direct current measurements may be needed before and after construction of the breakwater. This may be needed to assess the potential for dispersion of turbid water and suspended material.

Currents are movements of bodies of water and are usually specified as a vector magnitude with a speed and a direction that coincide with the direction in which the body of water is travelling. In contrast to wind and waves, the direction of the current is by tradition specified as the direction the water is flowing towards. Currents can be measured with a current meter that is fixed at a stationary position and records the speed of the water particles as they are flowing past it (Eulerian measurement), or it can be found by observing how a water particle moves over time in the vicinity of the current meter (Lagrangian measurement). We do not expect these different measurements to give the same value due to turbulence and due to the current not being the same over the entire area. The current also varies across the water column.

It is important to clarify how the current flow is affected by the presence of the breakwater.

### 5.3 ICE

Ice will only in very rare cases represent a direct load on a breakwater. The load may in such cases consist of:

- Ice floes that drift with the current and impact the breakwater at a high velocity, especially relevant for breakwaters in rivers.
- Ice that freezes onto the breakwater blocks and which break the blocks loose when the water level changes with the tide.

Armour units will not be able to freeze into ice and float away with the ice.

A breakwater will, however, be able to cause increased formation of ice in a harbour. By intent, the function of the breakwater is to protect the port from waves.  A breakwater typically brings about reduced circulation inside the harbour basin. This produces less dilution of fresh water in the water column. The surface water then has a lower salt content, leading to increased risk of ice formation.

Measures for reducing ice formation in harbours can consist of:

1. Cutting off of freshwater inflows: Freshwater sources (streams, industrial discharges, sewers, drainage from roads and streets, etc.) are identified and conducted out to an area outside the harbour where the water can be mixed with seawater or taken away with the current.

2. Submerging of freshwater inflows such that the inflows are conducted down to deep water. Note that this can often be a complicated solution for natural seepage (streams, etc.) because a superelevation is required at the entrance to the conduit in order to overcome friction and pressure differences (fresh water is lighter than salt water). It is also a precondition that the fresh water be conducted down to deep water (more than 10–15 m) and that there be circulation in the water at the bottom such that the injected fresh water is brought out of the harbour.

3. Increase the circulation: Opening of passages that provide flow-through in the harbour using the tide. Note that such passages must be of substantial size in order to be effective. The width of the flow-through opening ought to be of an order of magnitude of at least 10–20 % of the width of the basin that must be kept free of ice, and more if the basin is long and narrow or the distance to an open area is large. Culverts and pipes in the breakwater will not normally have any measurable effect on the circulation.

4. Shielding of ebb tide areas: Ice formation may occur in areas with gently sloping bottoms that become dry during ebb tides. With ebb tides it can become so cold that the seawater freezes or seepage and precipitation freezes during the ebb tide. At high tide, the ice floes float out and form the cores for continued ice formation. By filling up such areas or cutting them off with fills, the ice floes will be prevented from forming or drifting.

5. Ice formation in harbours can in some cases also be prevented by use of a bubble system. A bubble system can function in one of two ways:
   a). If the harbour is deep and contains "warm" bottom water, then the bubble system can be laid out inside the harbour. The flow of air bubbles going towards the surface will then draw the warm bottom water with it towards the surface, simultaneously with new bottom water being drawn in towards the bubble jet. The warm bottom water will keep the harbour free of ice.
   b). A perforated hose with emitting air bubbles can also be laid out transversely across a harbour entrance. This can be implemented in cases where it can be demonstrated that a freshwater layer at the surface can come into the harbour, which otherwise would be ice free with a mixture of warm and salt water. The flow of bottom water and air bubbles create local rise in the water surface that gives an outward-directed flow from the centre of the local rise, and will thus prevent a surface flow from coming into the harbour.

Information concerning ice formation can best be obtained from local experts, since there can be substantial local variation within small areas.

## 5.4 WAVES

### 5.4.1 General

The design loads on breakwaters are usually governed by waves alone (in combination with storm surge). Ocean waves are irregular and stochastic in shape, height, length, direction and speed of propagation. The term "sea state" is used for a wave situation where stochastic wave parameters such as significant wave height $H_s$, mean wave period $T_z$, spectral peak period $T_p$, mean wave direction θm and the like are assumed to be constant within a period. A sea state is described most efficiently using a wave spectrum. The duration of a sea state can vary. It is common to assume 3 hours as a representative duration of a sea state.

An example of a wave spectrum is shown in figure 5-1. A large number of waves with different periods, directions and heights are found in a sea state. We can show this by plotting the wave energy (in practice $H^2$) on the vertical axis and the frequency of the waves (f=1/T) on the horizontal axis. We then obtain a picture of how the wave energy is distributed across the different periods. In figure 5-1 we see that the spectrum has two peaks; one peak at 16.7 s (f= 0.06 Hz) and one peak at 7.1 s (f= 0.14 Hz). The lowest top at the lowest frequency are sea waves or swell with significant wave height $H_{m0,\,swell}$ = 1.33 m, and the highest peak is "younger" wind waves with $H_{m0,\,wind}$ = 2.22 m.

The period for the highest peak on the spectrum is called the spectral peak period $T_p$.

The area under the spectrum is called the zero order moment or $m_0$, and can be used to give an estimate of significant wave height with the expression $H_s \approx H_{m0} = 4\sqrt{m_0}$. If we had treated the entire spectrum as a sea state without regard to swell or wind-generated waves, we would have obtained $H_{m0,\,total}$= 2.59 m.

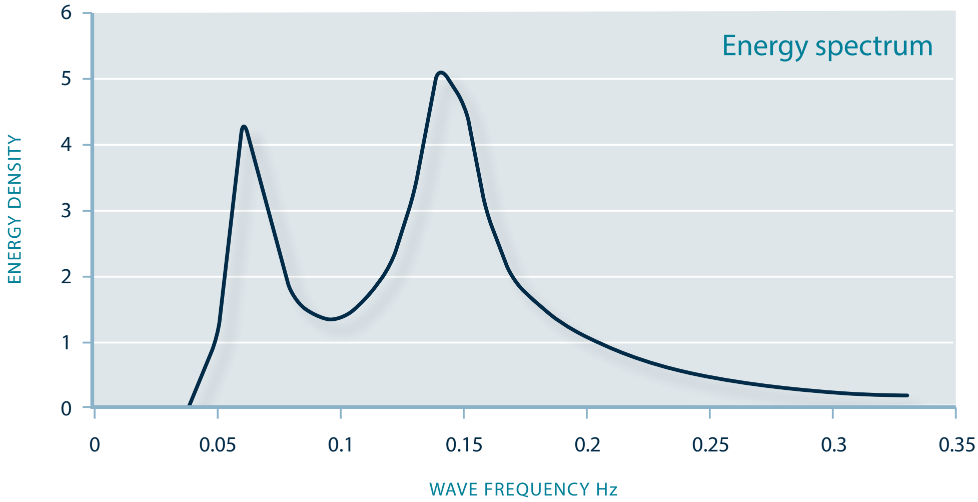



*Figure 5-1*
*Example of wave spectrum.*

Within the given 3 hours, the description of the wave action can be simplified by specifying a wave spectrum with a given analytical form of the spectrum (for example JONSWAP) defined by three parameters: $H_s$, $T_p$ and peak enhancement factor γ. $H_s$ and $T_p$ are not associated with an individual wave. They are parameters that describe the wave spectrum, and are constant for a given sea state, but vary over time.

$H_s$, and $T_p$ are wave parameters that will be used for the design of a breakwater. It thus is important to set the values for these two, especially the value of $H_s$.

Wave data from measurements provides the best basis. However, the data series is often not long enough to define extreme wave statistics with sufficient precision. The series must contain data from at least several decades in order to be able to describe a 100-year recurrence interval. Such wave measurements are not available in Norway. Hindcast data is a good alternative when wave measurements are lacking, because it estimates wave heights indirectly from meteorological conditions (primarily air pressure) that are available from historical measurements. Hindcast data, on the other hand, is only available for deep water, and usually numerical models must be used to transfer this data to the location of the breakwater. An overview of the different numerical wave models is given in chapter 5.4.2.

There are a number of databases with wave data for the open sea. NORA10 (Norwegian ReAnalysis 10 Km) is a data series with wave and wind data extracted every third hour through a hindcast reanalysis technique for a grid with a resolution of 10X10 km in the Norwegian Sea (Breivik, Reistad, Haakenstad, see ref. III), see figure 5-2. The database is administrated by the Norwegian Meteorological Institute and covers the period from 1958 up to the present.

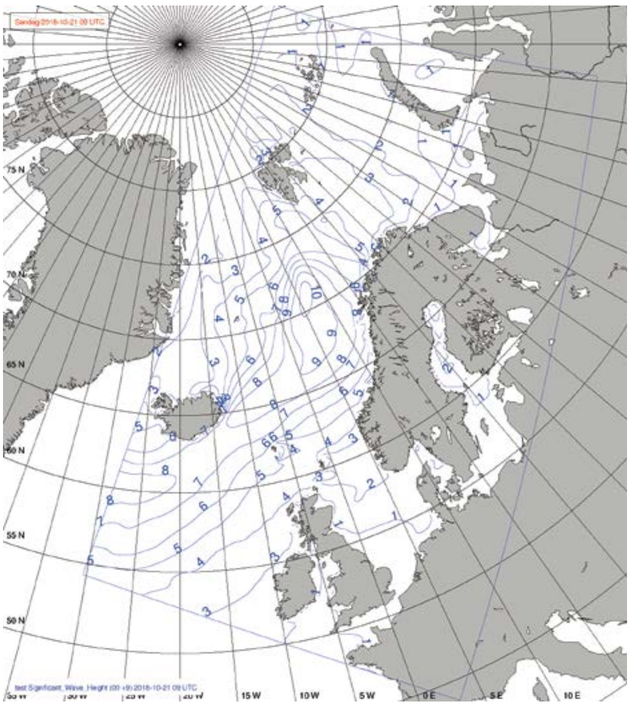


*Figure 5-2 Area covered by NORA10.*

It is common to get the wave data in 3-hour intervals. In order to establish the highest wave height associated with an exceedance probability (recurrence interval) it is common to fit a statistical distribution to the extreme instances from the data series. Two common ways of doing this are either to choose the annual maxima or to use maxima that lie above a certain threshold value (Peak Over Threshold – POT).

The POT method allows the selection of those extreme instances that are representative of extreme conditions over the course of a long data series. This technique gives us a better description of the wave conditions associated with extreme instances than what solely the annual maxima provide, because we get more points that are not necessarily associated with an annual frequency, i.e. a certain year may have several relevant storms that are not taken into account by using the annual maxima.

For further information on theoretical wave analysis, refer to Appendix I.

### 5.4.2 Numerical modelling of waves

#### 5.4.2.1 General

Numerical modelling is used for transforming waves from deep water offshore to the location where the breakwater will be built. With the use of a numerical model, the offshore wave climate is transferred to the area nearby the breakwater. In this process, the waves will undergo a number of transformation processes such as breaking, shoaling, and diffraction; and they will be subjected to any possible local currents (tidal currents, river mouths, etc.). Numerical models can also be used to analyse wave conditions behind a planned breakwater and inside the harbour (i.e. analysing harbour tranquillity).

With all types of numerical wave modelling it is important to specify the water level the waves are to be calculated for, and likewise if there are local current-related conditions that can affect the waves in the area. A detailed description of the local bathymetry is also essential in this context.

In order to be able to model the wave conditions based on hindcasting and local wind data, wave models are needed. Many different models exist (from simple and not particularly resource-demanding to complicated and resource-demanding), and it is important to use a model that fits the specific needs in the project that are to be modelled.

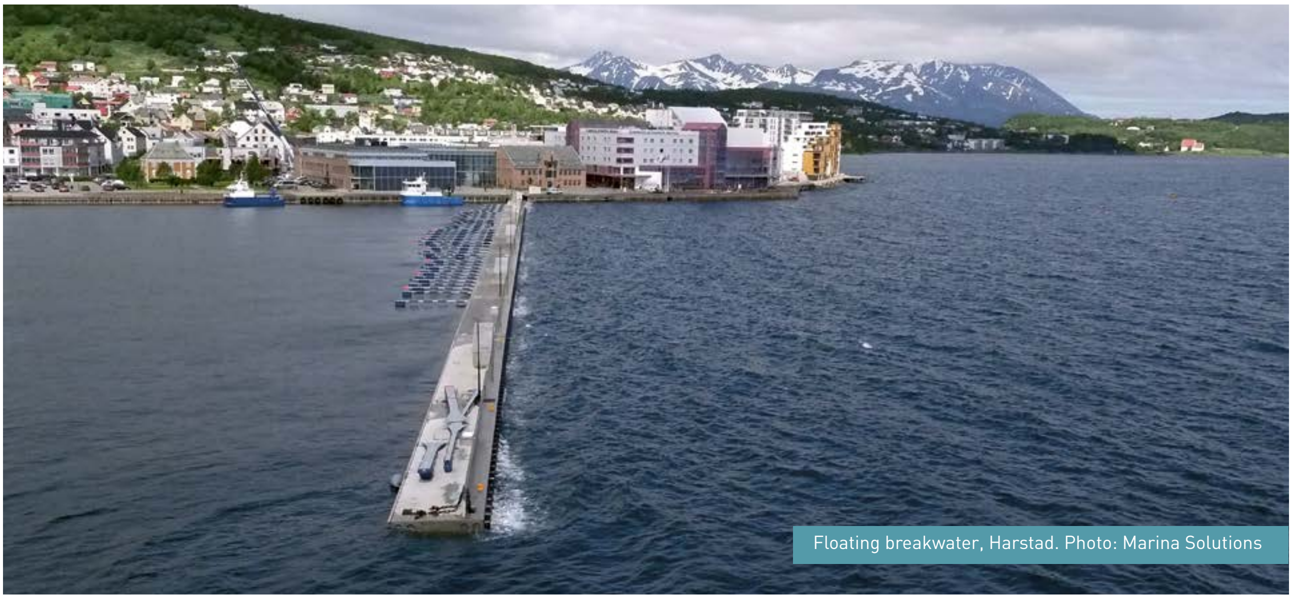
#### 5.4.2.2 Selection of model

Numerical modelling of waves can be performed for a number of purposes, of which the following two are discussed in this part:

1. for transformation of waves from deep water offshore to the location where the breakwater will be built.
2. or analysing wave conditions behind a breakwater and inside the harbour (i.e. analysing harbour tranquillity).

*5.4.2.2.1 Numerical modelling of wave conditions up to breakwater site*

In this category, there is a large number of calculation models to choose between. In Table 5-2 the methods are ranked from relatively simple and not particularly resource-demanding to the more complicated and demanding.
A description of each individual type of model is given afterwards.



Floating breakwater, Harstad. Photo: Marina Solutions

| | | |
|---|---|---|
| **Phase-averaging models (large-scale models)** | **Fetch-based analysis** | Empirical formulae – several available |
| | **Sheltering analyses** | Based on the concept of visible open sectors |
| | **Spectral wind wave models** | Based on conservation equation for wave energy or wave action<br>• Oceanic scale: – O (100 Km~5000 Km): WAM, WAVEWATCH III<br>• Coastal and somewhat larger scales: – O (10 Km~100 Km): SWAN, STWAVE, MIKE 21 SW, etc. |
| **Phase-resolving models (local-scale models)** | **Mild-slope models** | Based on linear wave theory where wave height, wave length and wave direction vary horizontally, irregular sea states can be simulated by the superpositioning of monochromatic wave simulations.<br>• Parabolic mild-slope model (REFDIF) (unable to model reflected waves going against the wave direction)<br>• Elliptic mild-slope model (CGWAVE) |
| | **Boussinesq wave models** | Depth-integrated horizontal speed, i.e. assumed constant in the vertical; the vertical speed varies nearly linearly from the bottom to the surface: (BOUSS-2D, MIKE21 BW, FUNWAVE, etc.) |
| | **Computational Fluid Dynamics (CFD)** | • Neglects all turbulence: 3D non-linear potential flow models (REEF3D)<br>• Neglects vertical speed: 3D non-linear shallow water equations (i.e. SWASH, MIKE 3 FM Wave)<br>• Navier-Stokes equations<br>– Reynolds averaged (RANS) + a turbulence model (for example k- ω, mixing length, etc.) (REEF3D, OpenFOAM, etc.) |
| | **Combination of different models in order to increase the effectiveness** | |

*Table 5-2 Relevant models for calculation of sea state.*

*5.4.2.2.2 Phase-averaging models (large-scale models)*

Fetch-based analysis is a relatively simple method based on empirical formulae for wave growth as a function of wind speed, fetch length and duration of the wind. Local wind statistics must be available. There are a number of formulae to choose between and not all give the same results. With fetch-based analysis it is clear that such processes such as refraction, wave-wave interaction, etc. are not included, nor are ocean waves (swell). With the use of fetch-based analysis it is important to be conservative in the choices and make use of experience to assess whether it is possible for waves to reach the location through refraction. Special attention must be paid to the possibility of swell or sea waves being able to come in. These types of waves will not be encompassed by fetch-based analyses.

When applying these methods in Norwegian fjords, it must be noted that the winds and waves have a tendency to follow fjords and valleys. This means that the real fetch into a point may be longer than the straight lines out from the point dictate.

With the choice of a calculation method, it is also important to keep to one and the same method throughout the entire process. It cannot for example be assumed that a fetch calculated with one method can be used as a fetch in another method.

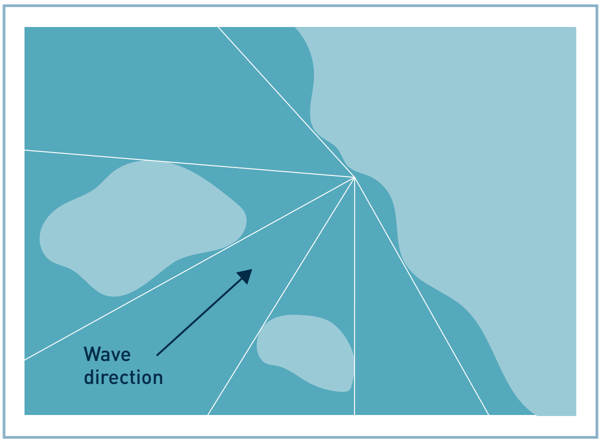



*Figure 5-3 Illustration of sheltering analysis.*

See the Rock Manual (see ref. IV) for further discussion concerning fetch-based analysis.

**Sheltering analyses** can be used where we are only interested in how sea waves propagate in towards the breakwater location and where individual sectors are sheltered due to islands and headlands. There must, however, be deep water conditions in the open sectors all the way up to the location of the breakwater. The analysis is based on an assumed directional distribution D(θ) on the open sea as well as the significant wave height in the open sea. The same directional distribution is assumed for all frequencies in the spectrum and that the wave energy in the directions for the closed sectors does not reach the location of the breakwater. See figure 5-3. This method is predicated on no shoaling effects or refraction of the waves occurring.

**Spectral wind wave models** are phase-averaging wave models. They assume that wave characteristics and depth conditions vary little across a distance corresponding to the wave length. The sea area is divided up into a grid, and a balance equation for the wave spectrum in the form of a partial differential equation is solved for this grid. The division can be extremely coarse with up to several wave lengths between each point. In this case it is possible to work with variables that are average values across a wave period (WAM), for example wave energy, or the variance spectrum. The dominant physical processes that are included are: Transfer of wind energy to wave energy, energy loss due to deep water breaking and bottom friction, refraction (due to bathymetry and local currents) and shoaling. These models can also handle depth-induced breaking. These models include non-linear transferral of energy between different wave components. Such models may, however, not describe diffraction or reflection in a correct manner. If diffraction and reflection are presumed to be negligible effects, such models can be used the entire way from the open sea to the planned placement of the breakwater. If that is not the case, the spectral model is used up to the area where diffraction and breaking are expected. Modelling further in towards the breakwater is then done by phase-resolving models.

*5.4.2.2.3 Phase-resolving models (local-scale models)*

These are models that solve the conservation equations for mass and momentum locally with a resolution of 10–50 points per wave length and wave period. They are designed to model the development of each individual wave in a wave train. It is recommended that they be used when the waves are subject to abrupt changes, i.e. in the event of significant changes to, among other things, water depth and possible currents over distances of less than the wave length. Diffraction and reflection are usually handled well in these models, in addition to breaking and shoaling, bottom friction and depth-induced breaking. The calculations based on these methods are computationally demanding and their use is thus limited to small scales in comparison with the methods described in the preceding section.

An example of the use of a phase-averaging simulation is shown in figure 5-4.a. An example of the use of a phase-resolved simulation is shown in figure 5-4.b.

**Mild-slope models** are phase-resolving calculation models based on linear wave theory. Irregular sea states are modelled by superpositioning multiple monochromatic waves in different directions. There are parabolic and elliptic mild-slope models. A parabolic model is dependent on the waves by and large propagating within a relatively narrow sector and ought not to be used if the waves are to pass around islands and headlands. These were developed mostly for use on coasts with relatively straight coastlines. Elliptic models do not have this limitation. Wave phenomena that can be simulated with such models are depth-induced breaking, diffraction due for example to breakwater and bathymetry, reflection (from structures and natural boundaries such as coastlines and abrupt changes in bathymetry), friction and depth-controlled breaking. The models normally use a triangular finite element formulation with element sizes that vary across the entire domain based on the local wave length. The model makes it possible to specify the desired reflection characteristics along the coast and other inner coastlines. The models are thus especially well-suited to simulate waves in harbours. Non-linear effects are not modelled.

**Boussinesq wave models** are phase-resolving wave models based on time step formulations of Boussinesq-type equations. These are non-linear wave equations originally developed for waves in shallow waters where it can be assumed that the horizontal particle speed in the water is constant in the vertical, while the vertical speed varies linearly from zero at the sea bottom and up to the vertical speed of the wave profile at the surface. In recent years, these types of models have continually been developed to handle greater and greater water depths. Today, there are Boussinesq models that can be used with from deep to shallow waters and that are able to simulate most of the wave phenomena of interest near the coast and in harbour basins, including shoaling/breaking over variable bathymetry, reflection/diffraction at structures, energy dissipation due to wave breaking and bottom friction, cross-spectral energy transfer due to non-linear wave-wave interactions, breaking-induced currents, wave-flow interactions and wave interaction with porous structures.

**(a)**

**(b)**

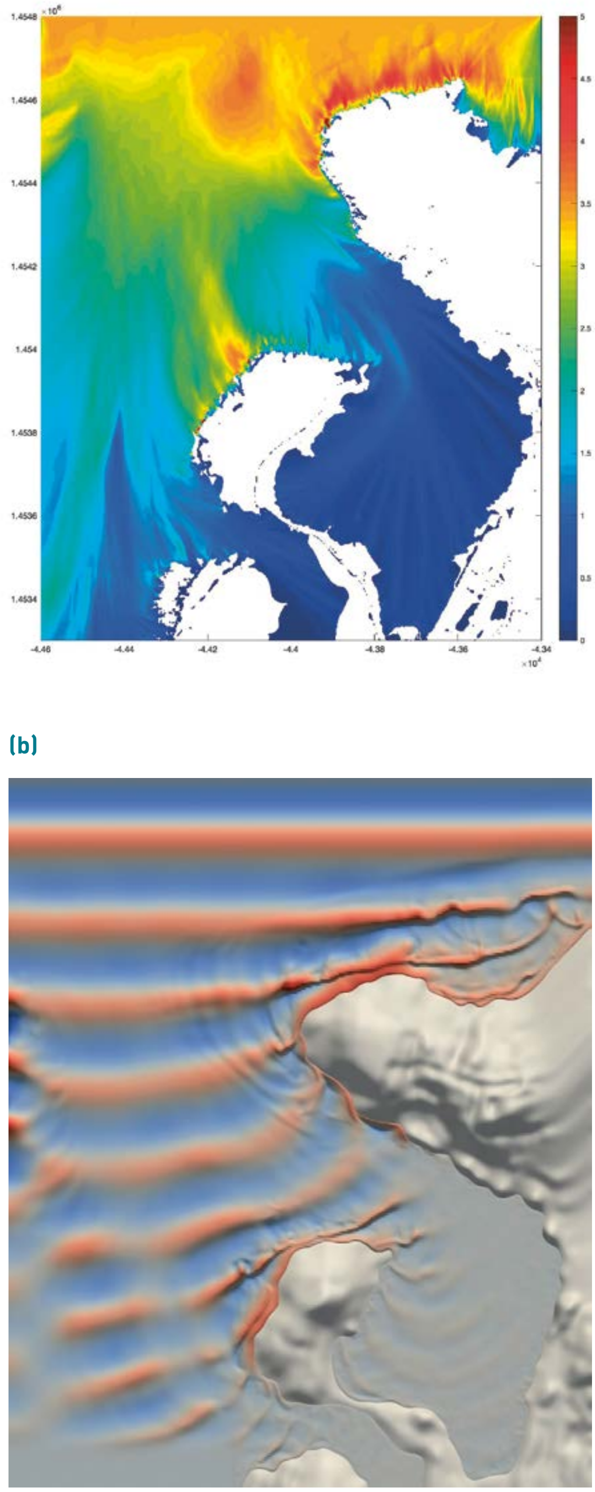


*Figure 5-4 Wave simulations Mehamn (Illustration, Norwegian University of Science and Technology).*
*a) phase-averaging simulation with the use of SWAN*
*b) phase-resolved simulation with use of REEF3D (H=9m T=9.5s).*

**Computational Fluid Dynamics (CFD)** is a branch of hydrodynamics where the fundamental Navier-Stokes equations are solved numerically, usually with certain simplifications. Unless DNS (direct numerical simulation) with a grid resolution on a millimetre scale is involved, the results depend upon the turbulence model that is being used. CFD is computationally time-consuming and this type of modelling is still not generally available for practical use for simulating waves in the coastal zone. Such modelling has been successful in simulations by numerical wave tanks with breaking waves in interaction with structural elements such as vertical cylinders and horizontal tubes, where sediment transport from the sea bottom of loose material is also included. It continues to be the case that modelling of waves in the coast zone with such models will only be possible with the use of powerful computers.

*5.4.2.2.4 Summary*

In summary, it can be said that the uses of these more advanced models are as follows:

In order to transfer offshore waves to the area in the vicinity of the breakwater, both spectral models and phase-resolving models can be used. The choice depends upon the size of the area to be calculated and the dominant processes. If diffraction and reflection effects are significant, then phase-resolving models ought to be used. In this case, phase-resolving models are often only used for a small area in front of the breakwater.

When choosing the most suitable wave model for a specific problem, the following alternatives exist:

1. Frequency distribution of wave energy or monochromatic wave. Some models are regular wave models (one single frequency and one single wave direction, a so-called monochromatic wave). However, most models that are used now are wave models for irregular waves with continuous distribution of the wave energy across a number of frequencies. This latter type of approach is preferred, as it gives a better representation of the sea state's irregular nature. In isolated instances, with for example long-period fluctuations in a harbour basin, a regular wave model will often be preferred.

2. Direction-determined spreading of wave energy. Models may possibly only handle one incident wave direction (unidirectional or long-crested waves) or they may handle directional spreading (incident waves from different directions that may also be dependent upon the wave frequency), i.e. the wave energy is spread out across a number of directions.

3. Evaluation of physical processes: Depending upon the specific case, this includes energy sources (wind energy) and/or energy drains (for example bottom friction, overtop), refraction, shoaling, diffraction, reflection etc.

4. Linear or non-linear modelling. The models can either be linear for modelling of propagation or non-linear, including non-linear effects that are due to wave-wave interactions. Usually, non-linear effects are relatively important for shallow water conditions and ought to be evaluated. For this reason, non-linear models are preferred. But since these models are quite difficult and cumbersome to use, simpler models based on linear theory for wave propagation are still used. It should also be noted that correct water level data must be known in order to ensure correct results regardless of the wave model.

### 5.4.2.3 Numerical modelling of harbour tranquillity

Here, it is only phase-resolving models described in item 5.4.2.2.3 that are relevant.


### 5.4.3 Physical Modelling – lab modelling

#### 5.4.3.1 General

The purpose of physical modelling (lab modelling) in connection with the construction of breakwaters is to verify and optimise a design that is based on pre-determined design criteria (see item 7.5). Verifying a selected design also reduces the risk of undesirable damages to or overtopping of the breakwater, which may cause damage to the infrastructure behind. Optimisation of the breakwater's design will also be able to reduce the costs of the facility.

This section briefly presents the circumstances for which it may be especially relevant or necessary to perform physical modelling. The section must also give the reader information on what to be aware of during the execution of the experiment. These are not detailed guidelines on the execution itself of lab modelling since this is normally performed by hydraulic laboratories with personnel who possess experience in experimental modelling.

In connection with harbour construction and breakwater lab modelling, there are also other types of experimental modelling. Other experiments for example such as optimisation of harbour design with respect to waves, ships that are moored to piers or investigation of erosion risk (scour) and protection. In connection with the mooring of ships in Norwegian harbours, it would usually be relevant to model and observe:

- The stability of the breakwater
- Overtopping
- Wave forces

Occasionally, model tests are not aimed at the items above. This may happen in cases where the waves and hydraulic effects are sufficiently described in numerical models

It is not necessary to carry out physical lab modelling for all breakwater projects. It thus is absolutely necessary to carefully assess whether it can be useful or necessary.

In cases where the circumstances are unusual, physical modelling will contribute to ensuring that the resulting design has the requisite degree of safety. Unusual circumstances could for example be:

- New breakwater project
- Projects with large construction costs
- Designs with large geometrical dimensions or large rock elements, i.e. atypical solutions
- Placements with a highly sloping or irregular sea bottom in front of the breakwater, where the effects of the waves on the breakwater will often be difficult to calculate
- Special breakwater structures or bottom conditions that give rise to a focusing of waves, or other especially vulnerable sections
- Breakwater designs that contain massive structures such as wave screens of concrete, lighthouses, etc.
- Breakwater design with substantial overtopping (stability of armour units on the lee side)

In many breakwater projects, an emphasis is placed on the optimised design of the rock element layer, crest and toe structure. Hence stability will be the most important parameter for most experiments. It may, however, be relevant to measure wave overtopping in cases where it is important to meet an overtopping criterion. See EurOtop (Ref. XVIII). Alternatively, a calculation for overtopping may be sufficient.

The stability of a wave attenuator or similar solid structures in a breakwater profile built of rock may be difficult to calculate. In such cases, measurements of wave forces during lab modelling will be of significant assistance in the design process.

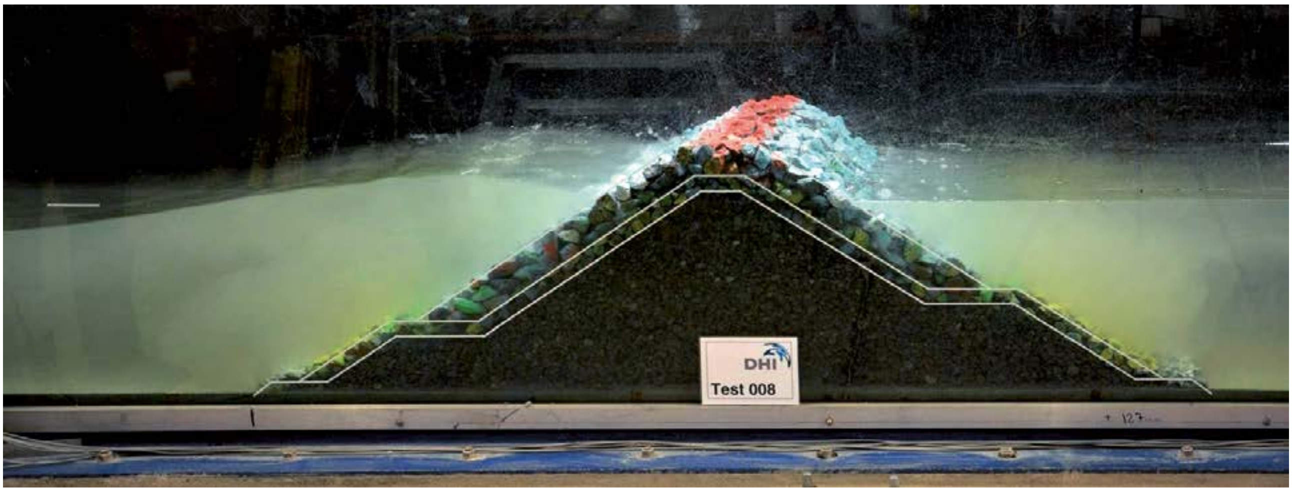



*Figure 5-5 Example of 2D experiment in wave flume, scale: 1:36, crest elevation +6.7 m, sea bottom -13.5 m, water level +1.8 m, $H_{s,100}$ = 3.5 m, $T_p$ = 15 s, $M_{50}$ = 5 t. (Photo: DHI).*

*Figure 5-6 Example of 3D experiment in wave basin, scale: 1:58, crest elevation +7 m, sea bottom -9.5 m, water level +1.5 m, $H_{s,100}$ = 5.1 m, $T_p$ = 16 s, Xbloc® = 10 t. (Photo: DHI).*

#### 5.4.3.2 Lab selection

In cases where it is necessary to conduct lab modelling, the task must be given to one or more experienced and recognised hydraulic laboratories. In a 2D experiment, only a narrow cut-out of a breakwater profile is tested in a wave flume. A 3D experiment encompasses a selected area of the harbour where the breakwater is located. A 3D experiment is performed in a wave basin. 2D and 3D experiments of stability for breakwaters built of rock are performed in accordance with best-practice methods.

When choosing a laboratory, it should be clarified whether it has facilities where it is possible to carry out 2D experiments at a scale of not less than 1:60, and 3D experiments at not less than 1:100. The recommendation is 1:20–1:40 for 2D and 1:30–1:100 for 3D.

The more accurate the information the laboratory can receive from the client, both during the bidding phase, where the individual laboratory gives a description of the model experiments and the corresponding price, and during the subsequent model design, the better it will be. This contributes for example to ensuring that a realistic timetable can be established for the experimental period.

The laboratory usually needs the following information and drawings:

- Drawing of the sea bottom conditions in front of the breakwater
- Overview drawing of the harbour area with the placement of the breakwater
- Drawings of breakwater profiles and proposals for profiles that will be tested
- Design wave conditions
- Criteria for acceptable damage and overtopping
- Salt water density, rock density and concrete density
- Methodology for laying out rock elements

## 5.5 GEOLOGY, QUARRYING

### 5.5.1 Geological assessment

A survey and geological evaluation must always be made of access to material for building the breakwater. This can be of significance in selecting a breakwater design, and ought to be done as early as possible in the planning. Among other things, it must be clarified whether there are relevant quarries in the area, or whether opening a new quarry must be assessed. It could also be an alternative to purchase rock from approved quarries in operation.

The geological assessment determines if the local rock is suitable for breakwater construction. If this is the case, the assessment must also determine the number of armour units of different weight classes that can be produced. Crack planes and weakness zones are essential for this assessment. It is recommended that such an assessment contain in specific the following:

1. Rock types and rock type distributions
2. Mechanical properties of the rock types
3. Chemical composition of the rock types
4. Survey of crack planes and weakness zones
5. Estimates of grading of blasted material, with estimates of distribution percentages for rock element weights
6. Dead load
7. Flakiness/schistosity
8. Available volumes
9. Any possible variations in the quarry's properties as its operation proceeds deeper into the bedrock.
10. Recommended blasting techniques for affecting the weight fractions

### 5.5.2 Investigation methods

For evaluating possible quarries, a number of investigation methodologies can be used. The investigations ought to be performed as early as possible, so that the results and estimates of the block distribution are available before the process of designing the breakwater is commenced. This makes it possible to try to achieve as high a utilisation of the rock elements as possible, and to try to make a breakwater cross-section that utilises what is produced in the quarry to the greatest possible extent.

**Surface surveying**
A surface survey must be performed of the quarry/potential quarry in order to acquire information on rock types, crack planes and weakness zones. If there are rock cuts in the immediate vicinity, then they must be assessed for whether they can provide additional information.

**Total sounding**
The method is based on drilling to find the bedrock surface and loose material covering. MWD (Measuring While Drilling) may be performed in the drill hole in order to acquire an interpretation of the rock hardness, degree of fracturing and water conditions.

**Resistivity measurement**
Resistivity measurements are electrical resistance measurements, and can be used to survey any possible weakness zones in rock as well as loose material covering.

**Seismic refraction**
The methodology is based on measuring changes in the propagation velocity of sound as it moves through different materials. The method can provide indications of the rock quality and covering of loose material.

**Core drilling**
The method is based on drilling out rock samples in order to obtain information on rock types, rock type boundaries, weakness zones and fracturing. Not especially relevant since the method is expensive and most of the information can be obtained by other methods.

**Test blasting**
The intent of test blasting is to find out what composition of stone blocks can be obtained from the quarry concerned, making it possible to have optimum utilisation of the quarry's output.

## 5.6 GEOTECHNOLOGY

As a part of the preliminary investigations, surveying must be performed of the sea bottom in the relevant area for the breakwater.

Initially, this will encompass surveying of the bathymetry. If no sea bottom chart is available for the area, bathymetric soundings are required.


*Figure 5-7 Typical drilling plan for a breakwater.*

Furthermore, surveying must be done of the loose material, both its composition and thickness. If the conditions indicate that there is much rock in the area, then such surveying may be performed with divers or a remotely operated underwater vehicle (ROV).

Surveying of the loose material must be carried out with a geotechnical seabed investigation. Soundings are then made in addition to extracting samples of the loose material.

The recommendation is that the investigations be performed as total soundings.  A total sounding provides information on the composition and disposition of the loose material.

The penetration into soil of this method is good, and a sound indication of bedrock depths can be obtained.

It is recommended that the drill holes be placed in profiles transverse to the breakwater, with 3 or more drill holes in each profile. Figure 5-7 shows a typical drilling plan for a breakwater.

Figure 5-8 shows a typical geotechnical profile from a seabed investigation for a breakwater. The profile shows the sea bottom, the planned breakwater and the sounding profiles. This provides a picture of the composition of the loose material in the area where it is envisioned that the

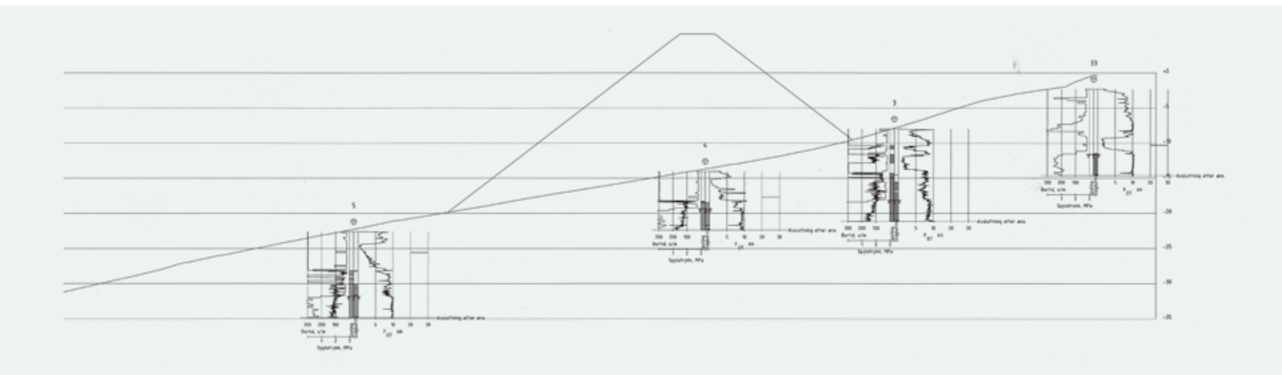


*Figure 5-8 Typical profile with breakwater and drillings.*

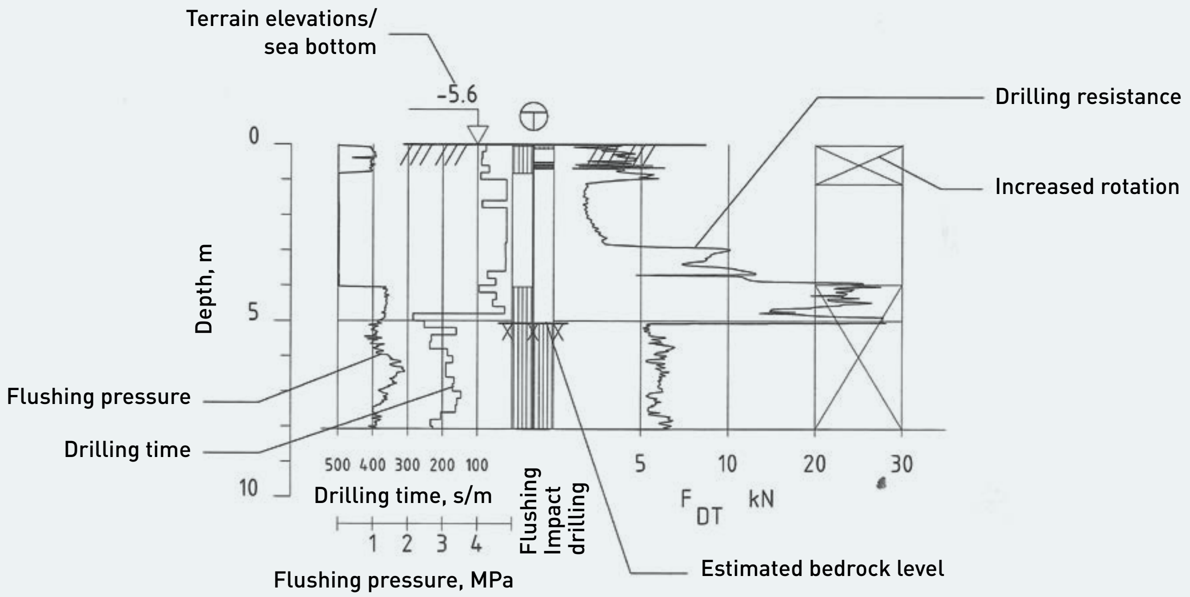



*Figure 5-9 Total sounding profile.*

breakwater will be placed. By presenting the soundings in a profile, it will be possible to get a clear view of any possible permeating soft layers that in turn could pose challenges in terms of the stability.

Figure 5-9 shows a typical total sounding profile. The profile primarily shows the following:

- Terrain elevations/sea bottom
- Drilling resistance
- Estimated bedrock level
- Flushing pressure
- Drilling time
- Flushing on/off – when flushing is on, this is indicated by cross-hatching in the column labelled flushing.
- Impact drilling on/off – when impact drilling is on, this is indicated by cross-hatching in the column labelled impact drilling

The sounding is initially performed with normal rotation and feed force. Low sounding resistance can indicate loosely dispositioned or soft layers, for example clay/silt. When firmer layers are encountered that halt the penetration, measures should be commenced in the following sequence:

1. Increased rotation
2. Flushing
3. Impact drilling

For reliable bedrock indication, at least 3 m must be drilled into the rock. This is in order to ensure that it is not a large block/stone that has been encountered.

Based on the total soundings, an evaluation is made of the test series, pressure soundings (CPTU) and any possible pore pressure meters.

Pressure sounding provides information on the composition of the loose material, layering conditions and soil types as well as an indication of pore pressure and material parameters. The equipment has a limited capability to permeate solid material, and cannot be used for indicating bedrock.

In order to acquire a reliable description of the loose material, sample series must be taken and analysed at a geotechnical laboratory. The scope of the sample series and subsequent laboratory programme will depend on the type of loose material and the specific set of issues that must be assessed for the breakwater. Normally, routine investigations are performed, which include:

- Water content
- Bulk weight density
- Material grading
- For clay:
  - Uniaxial pressure test
  - Cone
  - Consistency limits

| OVERVIEW OF SPECIFIC MAGNITUDES OF MEASURES THAT TRIGGER REQUIREMENTS FOR INVESTIGATIONS AND EVALUATIONS | | | | | |
|---|---|---|---|---|---|
| **Measure** | | **Area** | **Sediment investigations** | **Risk evaluation** | **Nature surveying** |
| **Fills** | Small | <1,000 $m^{000}$ | X | | X |
| | Medium large | >1,000 $m^2$ and <30,000 $m^2$ | XX | (X)* | X |
| | Large | >30,000 $m^{000}$ | XX | X (X)* | XX |

* Requirement for further investigations if there is a need for dredging

*Table 5-3 Excerpt from Table 2-1 in ref. X. Overview of the specific investigations and evaluations that are triggered by fills for establishing breakwaters, depending upon the magnitude of the measure. The number of Xs specifies the degree to which it is relevant to initiate or impose investigations/evaluations. No X = not relevant, X = may be relevant, XX = must be carried out.*

In addition, special examinations may be carried out such as oedometer and triaxial tests in order to obtain the material's deformation and strength properties. These are used further on in assessments of subsidence and stability. The scope of the laboratory programme ought to be assessed in consultation with the geotechnical consultant.

If dense soils are encountered such as clay/silt, it may be relevant to put down pore pressure meters if a pore overpressure would be critical with respect to the fill. This would then provide guides for the filling procedure itself.

Such a geotechnical seabed investigation can be combined with acoustic and seismic seabed investigations. These are, however, not relevant for breakwaters, because they are often very expensive, and thus only ought to be evaluated in special cases.

## 5.7 ENVIRONMENTAL INVESTIGATIONS

### 5.7.1 Requirements of governmental authorities

The Norwegian Environment Agency has prepared special guidelines for the handling of sea bottom sediments. The guidelines present the case flow, code of regulations, requisite preliminary assessments, planning, execution of measures and controls (Ref. X).

The Handling Guidelines apply tiered set documentation requirements, hence the amount of documentation grows with the size of the project, i.e. depending on the scope of the measure, based on the area and volume of sediment that is affected, see table 5-3.

The table shows that sediment investigations would be relevant in most cases. Requirements for the scope of the investigations are described in further detail in ref. XI. The number of sample stations and analyses are often determined based on the size of the measure, local conditions and any possible prior investigations in the area.

### 5.7.2 Classification of environmental condition

The environmental condition of the sea bottom sediments is to be classified with respect to the Norwegian Environment Agency's Classification Guidelines M-608 | 2016. The classification system is shown in table 5-4.

For a more detailed description with respect to requirements for nature surveying refer to chapter 5.11.

It is primarily the bioactive layer of the sea bottom (0–10 cm) that must be investigated for any content of environmental toxins. If there is a need for dredging, for example to obtain satisfactory stability, then the degree of contamination of the sediments must be investigated in the entire area affected by dredging. This may for example be performed simultaneously with the geotechnical investigations.

At a minimum, the sediment samples must be analysed for their content of arsenic, lead, cadmium, copper, chromium, mercury, nickel and zinc, the simple compounds in $PAH_{16}$ (polycyclic aromatic hydrocarbons), simple congeners in $PCB_7$ (polychlorinated biphenyls) and TBT (tributyltin). The analyses must be performed by a laboratory accredited for these types of analyses.

**CONDITION CLASSIFICATION OF SEDIMENTS**

| I Background | II Good | III Moderate | IV Poor | V Extremely poor |
|---|---|---|---|---|
| Background level | No toxic effects | Chronic effects with long-term exposure | Acute toxic effects with short-term exposure | Extensive acute toxic effects |

*Table 5-4 The classification system for environmental conditions in marine sediments (M608, see ref. XIII).*

In addition, the water content, grain size (gravel, sand, silt (63 µm) and clay (2 µm)) as well as the total organic carbon (TOC) must be determined.

Sample extraction methods and analyses are described in further detail in ref. XI.

If results are available from previous environmental investigations, it must be clarified with the pollution control authorities whether the results are still representative or if new investigations are required.

### 5.7.3 Capping of contaminated sea bottom

If the sea bottom sediments in the area where the breakwater will be established contain environmental toxins beyond condition classification II[1] (ref. XIII and Table 5-4), then remedial measures must be proposed to prevent dispersion of contaminated particles. This concerns it leaking out and spreading both during and after the filling.

Such a remedial measure may for example be comprised of laying out an isolating layer with clean material in order to prevent environmental toxins from leaking out as well as to ensure that burrowing organisms do not come into contact with the contaminated sea bottom sediments below.

The authorities grant no general advance approval of capping materials. Their suitability must be documented in each instance in connection with the permit application to implement the breakwater initiative.

Prior to selecting capping materials, the suitability of the materials must be assessed based on:

1. The conditions at the site where the capping will be performed (see ref. X and ref. XII.), for example such as sediment type, fine material content, contamination condition, current-related conditions, bathymetry, state of nature and geotechnical stability (load-bearing capacity and consolidation properties with respect to the capping material).
2. Properties of the capping materials (see ref. XI.) with respect to the capping layer having to reduce diffusion of environmental toxins and leakage due to waves and water currents (advection), as well as preventing bottom-dwelling organisms from burrowing down into the original material (bioturbation). Project planning for the capping layer is described in further detail in chapter7.

## 5.8 ARCHAEOLOGICAL INVESTIGATIONS

The Norwegian Act concerning the cultural heritage (Cultural Heritage Act) establishes that cultural monuments from the Middle Ages and earlier in time (before 1537) are automatically protected. Shipwrecks are defined in section 14 of the Cultural Heritage Act as "boats more than 100 years old, ship hulls, fixtures/fittings, cargo and other items that have been on board or parts of such items".

An assessment must be made based on the Cultural Heritage Act of whether cultural monuments might be found on the site and whether these would be affected by the project. Fills on the sea bottom can for example cause physical destruction or increased deterioration of cultural monuments. A fill for a breakwater construction will make the area inaccessible to later investigations of cultural monuments. The Norwegian county, marine archaeologist and maritime museum having the administrative responsibility for the district involved must evaluate/investigate the area before the project is initiated.

## 5.9 MAP BASIS

The basis for the specification of elevations on the sea bottom are depths below Chart Datum. The distance between Chart Datum and NN2000 or another zero point for land maps can be obtained by inquiring to the Norwegian Mapping Authority. Indicative information can be found at sehavnivå.no, however this does not give the precise elevation specifications that must be used for establishing a breakwater level or quantity settlement with contractors or others for construction projects. The Norwegian

1 In the classification guidelines (M-608) only effect-based limits are specified for TBT. Applicable practice is that administrative-related limits are still used for TBT (Guideline TA-2229/2008, ref. XIV) in the classification of conditions of sea bottom sediments.


Silda Breakwater, Sogn og Fjordane

Mapping Authority was commissioned by the Norwegian Coastal Administration to place bolts out at a number of locations. The bolts have been surveyed with respect to their placement and the elevation difference between Chart Datum and the bolt. Information about them can be obtained by inquiring to the Norwegian Mapping Authority, Hydrographic Service.

The navigational charts primarily show the sea bottom on a 50 x 50 m grid. If available data exists, and a better resolution than what is provided is not needed, it can be downloaded directly from kartverket.no. If more dense data than this is desired, an application must be submitted to the Norwegian Armed Forces to have classified data ("confidential" level) released. The application should include information on the purpose for using such data and the names of the companies/entities that will be using it. Send the applications to the Norwegian Mapping Authority.

Detailed mapping data may then be procured by contacting the Norwegian Mapping Authority, Hydrographic Service, see also kartverket.no for more in-depth information concerning sea bottom data, hydrographic surveying, etc.

If high-resolution data does not exist, surveying must be carried out. It is important that the measurement equipment and the surveying follow the Norwegian Mapping Authority's standards such as "General requirements for organisations performing hydrographic surveying", "Technical requirements specification for hydrographic surveying" and "Terms and conditions for approval of organisations performing hydrographic surveying".

The rules for classification are currently being revised. The specific rules that are applicable at the relevant point in time must hence be clarified.

## 5.10 LAND RIGHTS AND PROPERTY OWNERSHIP RELATIONSHIPS

During the planning, who possesses the ownership rights to the relevant land for the breakwater must be clarified. It must also be established whether other rights or entitlements exist for the land concerned, for example whether others have rights of use that may come into conflict with the development work.

The breakwater will generally become a part of the landed property that the breakwater is placed on, or filled out from. How the landed property rights associated with the breakwater should be managed depends upon what the future ownership rights to the breakwater and breakwater land will be.

If there is a different owner of the breakwater land than the entity that will own the breakwater, then the breakwater land must possibly be split off and deeded to the future breakwater owner (i.e. as a separate landed property with its own cadastral number and title number). In such cases, a process ought to be initiated at an early point in time for the splitting off (possibly with preliminary surveying) and deeding of the breakwater land to the breakwater owner. The property conveyance must be officially registered before construction is commenced if the value of the breakwater itself will not be included in the basis for calculating the document fee. How this process will be carried out, including any possible final surveying after completion of the

Kabelvåg Breakwater. Photo: Norwegian Coastal Administration

development, will be clarified with the relevant municipality in each individual case.

Furthermore, it must be clarified whether it is necessary to establish other ownership rights for the affected landed properties. This may for example concern limited rights such as access rights, use rights in order to be able to perform construction and maintenance work, etc. If the breakwater land is split off and deeded to the breakwater owner, it must also be evaluated whether there are such needs for rights for the remaining property that the breakwater land was separated out from. Agreements concerning such rights must be entered into in writing with the relevant land owner and officially registered with the breakwater property as the holder of the rights.

The breakwater builder, future owner of the breakwater and breakwater administrator should clarify the extent to which it should be at their disposal for future operation and maintenance, both for the breakwater and for any possible navigation installations mounted on it, as well as to insure its future use does not come into conflict with the breakwater's function. Other relevant rights may be access for inspection and maintenance, access to rock material (blocks, filter and core materials) and the possibility for supplying electrical power to navigation installations. Such rights must also be surveyed and have their co-ordinates fixed.

**5.11 NATURAL DIVERSITY**

The Nature Diversity Act (NML) establishes that all parties must act with due care and do what is reasonable to avoid harming natural diversity.

When carrying out a project, sections 8–12 in the Nature Diversity Act must be taken into consideration. These provisions discuss the knowledge base (section 8), the precautionary principle (section 9), total burden (section 10), that the costs of environmental deterioration must be borne by the project owner (section 11) and environmentally justifiable techniques and operating methodologies (section 12). A good aid in these assessments would be the Norwegian Environment Agency's Guidelines for the Nature Diversity Act, chapter II.

Natural diversity includes biological, landscape-related and geological diversity. Biological diversity encompasses all diversity of species (animals and plants), natural habitats and ecosystems. Area changes comprise the greatest threat to the biological diversity. Acquisition of knowledge, surveying and monitoring are extremely important in order to gain an adequate basis for planning and execution of projects in a manner that does not affect the biological diversity negatively.

Information in the species database's red list for species must be procured for actions that will affect or change habitats. This must be done to clarify whether the measure would have a negative impact on species diversity and vulnerable species. Prioritised aquatic species are particularly highlighted. Dwarf eelgrass (Zostera noltei) is critically endangered and has been designated as a prioritised species (2015). All forms of extraction of or damage to dwarf eelgrass are forbidden. Exceptions are only granted by applying to the County Governor.

The species database also addresses blacklisted species. These are alien species for which it must be ensured that the measure will not involve them being spread further.

It must also be clarified whether the measure will affect:

- Spawning and rearing areas for fish
- Nature reserves
- Bird reserves
- Areas without major infrastructure development (INON) Areas without major infrastructure development are defined as areas that are situated one kilometre or more (as the crow files) away from major infrastructure development (see miljodirektoratet.no).

The following natural habitats and key areas ought to be specially surveyed in connection with a project if it has not already been surveyed by the Norwegian Environment Agency and registered at naturbase.no:

- Larger kelp forests
- Strong tidal currents
- Fjords with naturally low oxygen content in their bottom water
- Especially deep fjords
- Small, rounded fjords with narrow inlets
- Littoral basins
- Ice front deposits
- Soft bottom areas
- Occurrences of coral
- Eelgrass meadows
- Occurrences of oysters
- Shell sand
- Larger occurrences of scallops
- Unattached coralline algae
- Spawning areas (cod)

Perform such surveying with for example an ROV or diver with a video camera. If the initial surveying shows that there are vulnerable areas/species that could be affected by the project, a risk assessment must be performed of the consequences of the construction work before the detailed plan is established.

As a part of this surveying of the natural diversity, it is also important to register important commercial interests that are closely associated with natural diversity in the area. Here, it is especially relevant to have the fisheries interests in the area surveyed, especially fishing grounds and aquaculture facilities. Local fishing groups are good informants in this context.

Information concerning nature-related conditions and natural habitats can be acquired from Naturbase (www.naturbase.No), fisheries-related conditions can be acquired from Yggdrasil (the map tool of the Norwegian Directorate of Fisheries) and other coast-related information can be found at Kystinfo (the map tool of the Norwegian Coastal Administration). Areas with important nature-protection interests may be proposed for protection or already be protected. The municipality or County Governor will have the necessary information in this regard.

### 5.12 AQUATIC ENVIRONMENT

The EU's Water Framework Directive provides guidelines for the aquatic environment in all of Europe being improved. The Water Framework Directive concerns freshwater, groundwater and coastal waters. The objective is to improve the water quality and the physical aquatic environment, including the basis of life for animals and plants that live in water.

In Norway, the principles in the EU directive are implemented in the  Norwegian Regulations concerning a framework for water administration (Water Regulations). The purpose of the Water Regulations is to ensure the best possible overall protection and sustainable use of our water resources.

The Water Regulations apply to all surface water, rivers, streams, lakes, coastal waters and groundwater and poses requirements for co-operation in technical evaluations, decisions and execution of measures in order to attain good environmental conditions in the water bodies concerned.

The following environmental goals apply for surface water (Water Regulations, section 4):

- The condition of the surface water must:
  - Be protected against deterioration
  - Be improved and re-established
  - Possess at least good ecological and good chemical conditions.
- Artificial and strongly modified water bodies
  - Be protected against deterioration
  - Be improved
  - Possess at least good ecological potential and good chemical conditions.

During the planning phase, all measures must be assessed with respect to their effect on the ecological and chemical conditions as regards the Water Regulations. This is done by investigating whether information exists on the conditions in the water body that must be taken into consideration in the planning of the project. It is incumbent upon all municipalities via the Water Regulations to perform surveying of the conditions in the water bodies in the territory of their municipality.

The above-mentioned information will also be utilised to evaluate whether the project after its planning would continue to be able to degrade the water quality and have negative consequences for the water body such that remedial measures will have to be initiated.

If no applicable information nor previous investigations of the water body are available, it must be assessed whether the project is of such a degree that it could involve changes to the water body. If this is the case, investigations ought to be conducted in order to gain a better overview and be able to plan appropriate remedial measures.

Environmental data that will form the basis for the planning and execution of measures to ensure good environmental conditions in the water body during and after projects are available at Vann-nett (http://vann-nett.no/portal/).

### 5.13 MONITORING

Measures in seawater, brackish water and freshwater may lead to the stirring up and spreading of sediment particles that could involve contamination if environmental toxins are bound to the particles. In addition, the stirring up of sediments or addition of material may lead to damage to nature-related assets due to accumulation of mud, regardless of whether the particles are contaminated or not. Accumulation of mud is for example harmful for eelgrass, spawning areas for fish and may result in damage to the gills of the fish. Measures involving fills in areas with nature-related assets will on that basis be able to trigger requirements for permits for measures from the environmental authorities even if the sea bottom sediments are clean.

During this phase it thus is important to utilise the information acquired in accordance with the above chapters in order to procure an overview of the need for monitoring in connection with the project before, during and after the construction work. It is recommended that, as a part of the planning work, a proposal be prepared for a monitoring programme.

As a basis for such a programme, background information must be collected on the project area and the surrounding areas that bear an interrelationship with the project area in order to plan the monitoring. Such data collection may for example consist of surveying the direction of the dominant currents, flow speeds, flow variations, influx of freshwater, effects of tides and turbidity.

The most relevant measure in the monitoring of the construction work is the measurement of turbidity. This is performed using turbidity meters that are placed such that they register the stirring up and spreading of sediment particles. It is recommended that the turbidity monitoring be planned and executed with respect to the procedures specified in NS 9433:2017 "Turbidity survey of measures in water bodies". The background information is to be used in order to assess the best possible placement of turbidity meters and determination of limits for acceptable turbidity in the water bodies.

It should also be assessed whether the natural habitats in the area and the measure's effects on them should be monitored and in what manner.

The project owner is responsible for the planning and execution of measures and monitoring, and must ensure that the measure is carried out according to the laws and other regulations as well as the applicable permits. In permits issued by the environmental authorities, specific requirements may be posed for the monitoring if a monitoring plan does not already exist as a part of the application for the permit. In order to ensure locally adapted and effective monitoring, it thus is recommended that the project owner prepare a proposal as a part of the planning of the measure and formulation of the application.

### 5.14 REMEDIAL MEASURES

The reporting that is carried out during the planning phase, especially the impact assessment of natural diversity and the aquatic environment, often contains recommendations on which remedial measures ought to be carried out in order to minimise the negative consequences of establishing the breakwater structure to the nature and environment in the area. This may encompass special monitoring of the construction work, shielding of the construction site or of vulnerable areas if these are local as mentioned above.

If the preliminary assessments during the planning phase specify remedial measures involving the placement and formulation of the planned structure, in order for it to as much as possible not involve changes to the ecological conditions in the relevant water body, then such measures must be taken into consideration during the project planning before the final formulation and placement are established.

In order to simplify the legibility and increase the clarity, it is recommended that a summary overview be made of all remedial measures that should be carried out. It is recommended that such an overview be included in applications to governmental authorities, possibly also in the development plan for the project. It is furthermore recommended that services which it is desired be performed by external entities (consultants, contractors, measurement companies, etc.) be incorporated into tender documents, contracts or other agreements such that the execution of remedial measures is ensured.

Ballstad, Nordland. Photo: Jorun Karlsen

# 6. LAYOUT AND HARBOUR PLAN

## 6.1 PLANNING PROCEDURE

The need for a breakwater can have several different causes, but the most frequent justification is a need for improving harbour conditions at a selected pier or pier area. Those conditions that it is desired to improve may have their causes in one of the following types of loads/problems.

1. Waves; where a distinction is drawn between 3 different types of waves:
   a. ocean waves and swell, i.e. waves that are formed in the open ocean and enter the area as partly attenuated swell.
   b. local fjord wind waves, which may be formed by local winds in a fjord area that lies outside the harbour, but which does not stretch out to the sea.
   c. local harbour waves, which are formed inside the harbour basin by local winds.
2. Seiches, often called long-period waves, are a phenomenon where a partially closed basin experiences resonant fluctuations, where the basin is most often filled by ¼ or ½ wave lengths.
3. Currents from tides or from rivers.
4. Ice, generally in the form of drifting ice floes or brash/slush, i.e. that is sought to be kept outside the harbour.
5. Inflows from rivers, which cause problems when fresh surface water discharges into the harbour and freezes during the winter.

Other reasons for building a breakwater or breakwater-like structure can be the need for a quay, road embankment, industrial area on a landfill, bridge foundation, fill for protection against ship collisions, etc. All these structures can be modelled as a breakwater, and designed in the same manner. In these guidelines we assume as a general basis that the motivation for building a breakwater is to build or improve a harbour. The process for progressing to the detailed project planning for a breakwater is shown in figure 6-1. This figure is not complete, but will show parts of a complete process.

A. It usually starts with an *Outline project and/or a preliminary project* that will describe an issue and different methods for arriving at a desired solution. During this phase, an assessment is made of whether the different solutions are realistic, such as whether it is technically and economically feasible to build a breakwater at the site where it is assumed to have a good effect. This is where limitations such as sea bottom conditions, property ownership relationships, wave heights, adopted development plans, room for manoeuvring, necessary investments, etc., come in.

B. During the phase *Harbour design/Harbour plan* the focus will be on those alternatives that on the overall are feasible technically and economically, and which provide the desired effect. Methods and tools will be used here such as for example numerical wave models that show how the waves propagate inside the harbour and estimates of the portion of time when a pier would be operational. This phase will conclude with a recommended solution describing where a possible new breakwater should be located and how long it must be to provide the desired effect.

C. During the *Design phase* the breakwater must be dimensioned, i.e. the geometric dimensions determined for the breakwater (height, width), block sizes, filter sizes, etc. In order to be able to determine this data, it is necessary to have a set of design criteria and functional criteria, such as for example

- should traffic be permitted on the top of the breakwater?
- is overtopping permitted of the breakwater during extreme storms?

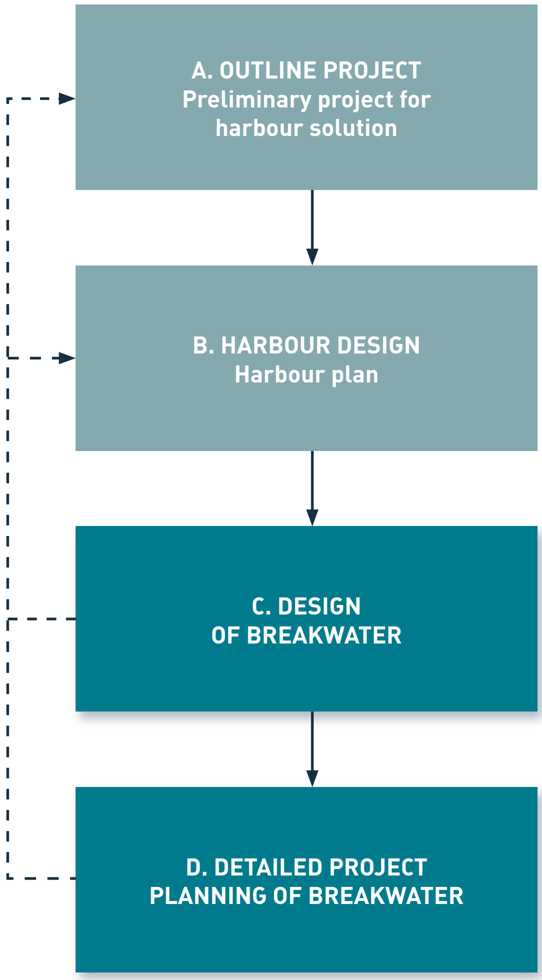



*Figure 6-1 Standard procedure for that part of the activities that lead to the detailed plan for breakwaters.*

D. During the phase *Detailed project planning of the breakwater* detailed drawings and descriptions of the breakwater must be made, for example typically as cross-sections every 10 m. Volume calculations must also be performed of the different material types, so that a contractor will be able to calculate the costs.

We assume for now that this will take place as a linear process, i.e. from Phase A to Phase D. In other words, when Phase C starts it has been decided where the breakwater will be situated and how long it will be. At this point in time, all conditions that affect the selection of the breakwater will have been clarified. This concerns for example sea bottom conditions, availability of rock elements, development plans, etc. Only in exceptional instances should it be possible to go backwards in the process to address a problem in Phase C or D that requires the processes in Phase A or B to be repeated, marked with dashed lines in figure 6-1.

**In this chapter, a brief introduction is given to the themes in Phase A and B. The treatment in this chapter is not complete, and reference is made to other publications and books on this theme for a complete description.**

## 6.2 WAVES IN HARBOURS

Those waves that arise inside a harbour are important for:

- assessing the suitability of for example a pier or a pier area
- design of structures inside the harbour (piers, buildings, etc.)

In order to assess the effects of waves inside a harbour, numerical models with phase resolution are most often used today, i.e. models that show individual waves and how they propagate inside the harbour (the other type of models is spectral models, which only show statistical values at each point, for example such as significant wave height, mean direction, etc. This type of model is usually used across large areas). For a more detailed discussion of the different types of models, see chap. 5.4.2.

Figure 6-2 shows an example of such a model from a Norwegian harbour (Skjervøy) where the sea waves are incident from the northeast upon a breakwater opening that is approximately 200 m wide (adapted for calls by the Norwegian Coastal Express). The wave length in the open area is approximately 250 m, which converts to a wave period of approximately 12.5 s. In the picture, we see that the waves are incident upon the breakwater opening diagonally, and some of the wave energy is going directly into the inner part of the harbour.

Around each breakwater head (the end of the breakwater) we see, however, that the waves are turning around the head and continuing inwards in parallel with the lee side of the breakwater, with a new direction that is approximately 90° to the direction that the waves had outside the harbour. This effect is a combination of two physical phenomena that govern wave motion:

1. A linear refraction effect, which means that a wave propagates faster in deep water than in shallow water. The breakwater is a relatively large structure, and those waves that are closest to the breakwater will be slowed down, whereas those that are further away and thus in deeper water will move more quickly. Hence the waves will swing around the breakwater head.

2. A non-linear effect called diffraction. Diffraction can be compared with a process where the waves hit near the breakwater head, and those waves that pass through the opening will then begin to "spread out" on the lee side of the breakwater. The result is that the wave energy will propagate along the wave crest and transverse to the wave's propagation direction, and have the same movement around the breakwater head as for ordinary, linear refraction. In ordinary wave theory, it is presumed that the wave energy simply propagates perpendicularly to the wave crest, and hence this phenomenon is characterised as non-linear.

The result of these two processes is that the waves have a tendency to round the breakwater head and spread out on the lee side. This of course leads to waves where the one might expect that there was good shelter, but at the same time it also disperses the wave energy that slips in through the opening across a larger area, thereby reducing the wave height.

The refraction effect increases in accordance with an increasing wave period and wave length. Long sea waves will be affected at great water depths (50 - 100 m), whereas short wind waves are affected only in shallower waters (5 - 40 m). This can be seen by comparing figure 6-2 with figure: 6-3, where the latter shows how short wind waves from the east propagate into the harbour. The wind waves are affected by refraction only quite near to the breakwater, and the waves have a much greater tendency to move directly forward. The difference between the two figures is also due to the different angle of incidence to the breakwaters, but we see that the wave height of the wind waves is nearly the same outside the harbour and in a section in the middle of the harbour, whereas the swell is reduced to insignificant values.

A picture produced as a result of numerical calculations of wind waves, as in figure 6-3, can be compared with an aerial photo of the same area after development. An example is shown here of such a picture in figure 6-4 for Hasvik Harbour. Even though they are different harbours, we can see clear similarities between the two pictures.

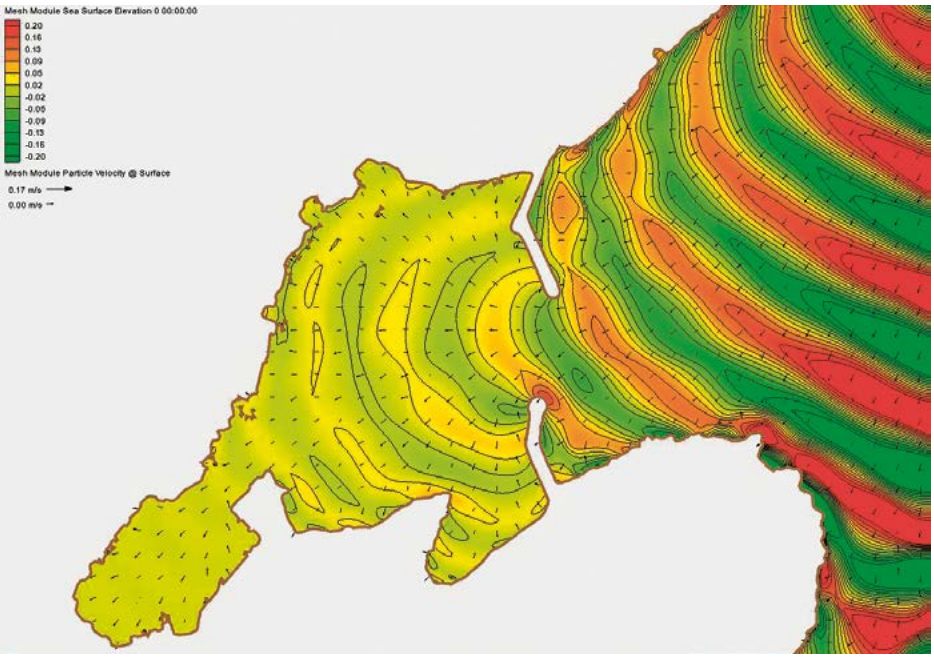



*Figure 6-2*
*Snapshot of sea waves with period Tp = 12.5 s into Skjervøy Harbour calculated using Mild Slope-model. The wave crest is red, and the wave trough is green. (Norconsult).*

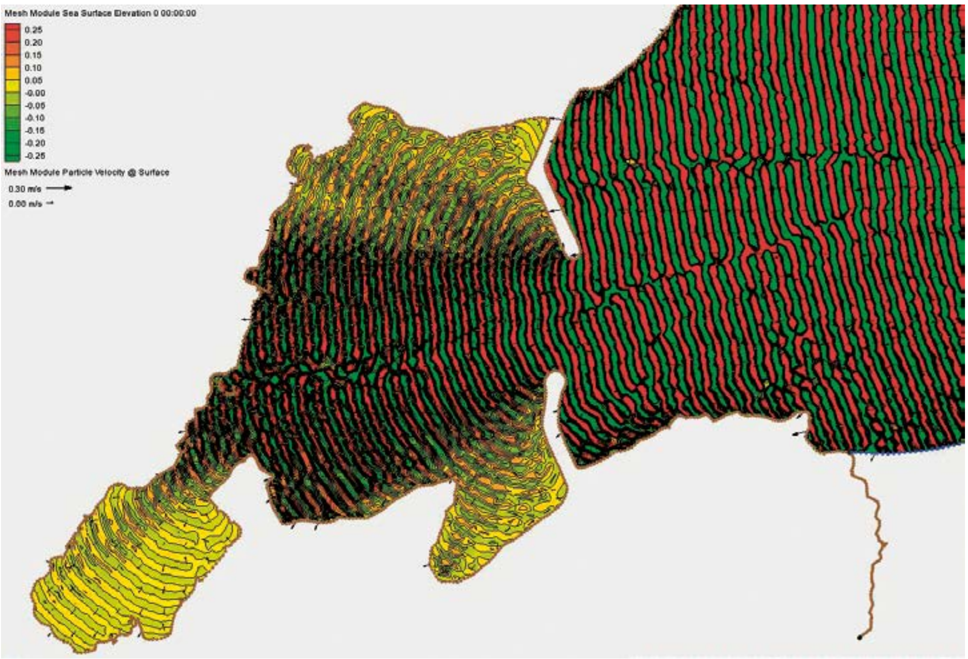



*Figure 6-3*
*Snapshot of sea waves with period Tp = 4.0 s into Skjervøy Harbour calculated using Mild Slope-model. The wave crest is red, and the wave trough is green. (Norconsult).*


*Figure 6-4*
*Aerial photo of Hasvik Harbour that shows incident wind waves from the southwest going towards the harbour. The waves are short, and are hardly affected by the bottom around the breakwater.*

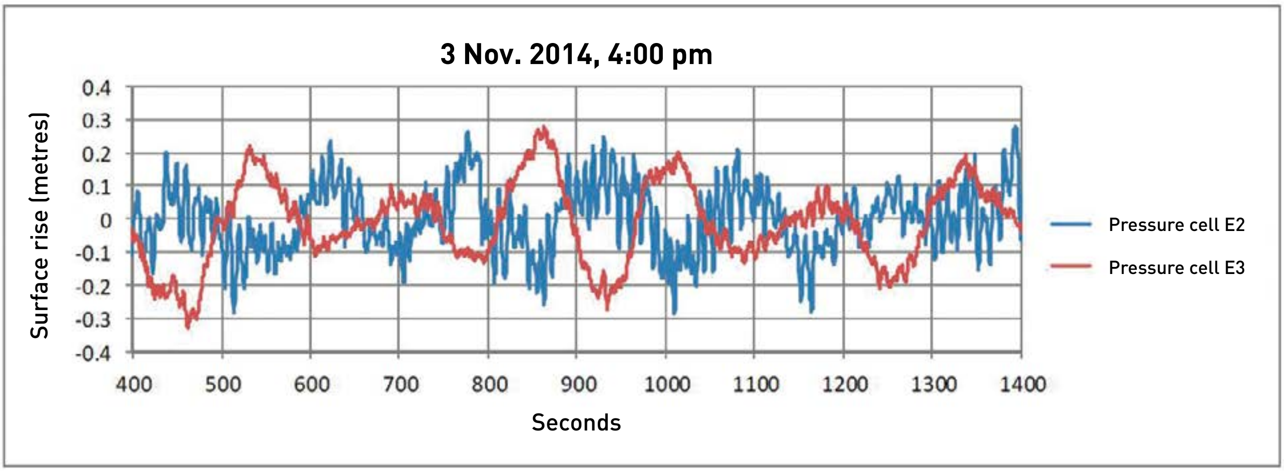



*Figure 6-5 Measurements of long-period waves in Andenes (SINTEF).*

### 6.3 LONG WAVES AND SEICHES

Seiches (long waves) are phenomena that occurs in partially closed basins which can be a harbour basin, a bight/bay, or in some instances entire fjords. The principle is that the entire basin experiences resonant oscillations, and if energy is injected from the outside by waves, and the energy cannot escape, then the oscillations can reach large amplitudes. It is known along the entire coast that in extreme instances the water level furthest inside a bight can vary by up to 1.5–2.0 m, most often with periods from 50 s to 3–4 minutes. Figure 6-5 shows a print-out from a measurement series from two recorders in Andenes, where we see that the harbour fluctuates with a period of approximately 150 s and the two recorders that are shown are in opposite phases, i.e. when one is at the top, then the other is at the bottom. The water level at the recorders can go from -0.3 m to + 0.3 m over the course of several minutes. This type of movement gives rise to oscillating currents inside the harbour, which in turn leads to moored ships and boats tugging on their moorings, with the vertical distance between the pier and ship varying significantly. In the worst case, it can also involve the depth at the pier becoming too shallow.

When planning a harbour solution, it is important to be aware that such fluctuations may be present, or that there in the worst case could be a risk of creating such a problem by the placement of the breakwater.

Figure 6-6 shows a numerical modelling of the harbour at Andenes with a 120 s period, similar to what is currently being experienced at the harbour. The users of the harbour report that they are subjected to long-period waves, especially in the left side of the harbour. We see this in figure 6-6, marked by those areas that are yellow and green (large vertical movements), and those areas that are red (extreme effects).

There is a proposal to expand and change the harbour at Andenes, and an important part of the task is to ensure the elimination of the present oscillation pattern and that new ones do not arise. Figure 6-7 shows a draft for a new harbour plan for Andenes, also with a 120 s period. We see that the large effects have now been eliminated, and that this has occurred by bringing new areas into the harbour, but also by introducing some smaller breakwaters inside the harbour for the purpose of breaking up the oscillation pattern.

Where it is known or suspected that such oscillations exist or may arise, it is important to investigate how a new breakwater could affect the oscillation pattern. In the worst case, there is a risk of introducing a problem that has not been there before.

The best tools for investigating this are phase-resolved numerical models as shown in figure 6-6 and figure 6-7. Physical laboratory models have been used earlier, but such models will not always give a correct answer.

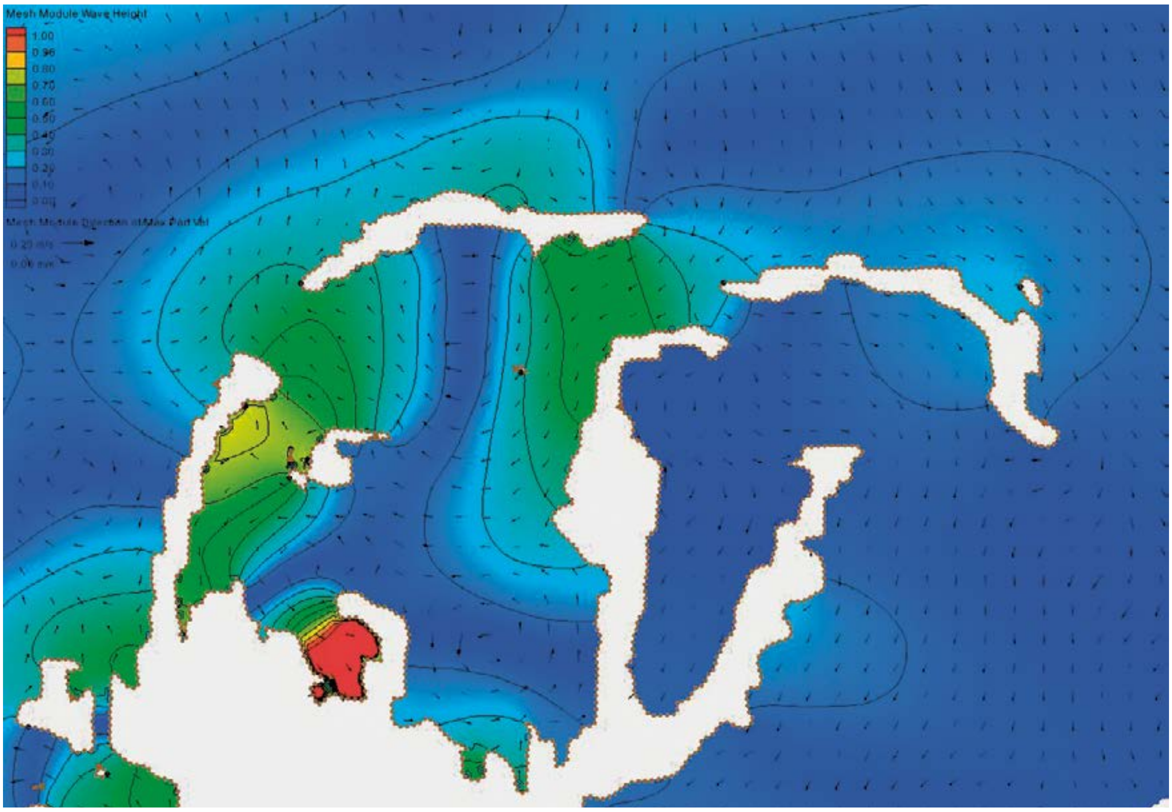



*Figure 6-6  Seiche with 120 s period at Andenes, existing. (Norconsult).*

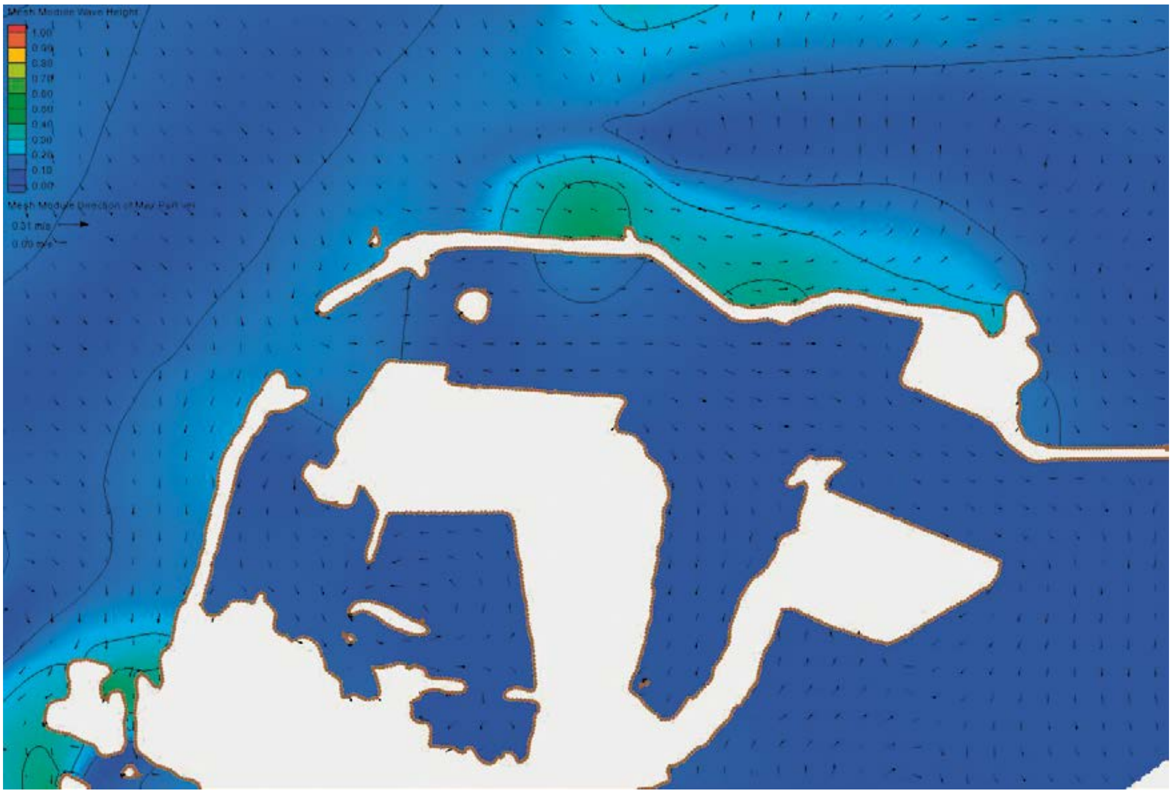


*Figure 6-7  Seiche with 120 s period at Andenes, Alternative 9. (Norconsult).*

### 6.4 CURRENTS

Currents will only in extreme instances comprise a design force on a breakwater. A breakwater can also affect the current conditions in a harbour or nearby areas. Such changes to the currents can lead to increased current speeds with ensuing increases in erosion, or to reduced current speeds and poorer water quality, possibly also the accumulation of sediment.

Measurements of the currents before a breakwater is built can be relevant, and such measurements may be necessary in order to fulfil other requirements for investigations. If measurements are performed, the measurement period should be approximately 35 days or more. A length of 35 days ensures that the observations are made during a complete cycle of the moon, and thus quantifies the dominant and most important driver of the tides, namely the moon.

Measurements at the site will provide information on the conditions as they are *before* the development, however they can also provide valuable information on mechanisms, current-patterns, etc.

If the assessment is that a breakwater structure may result in altered current conditions that are of significance to the water quality, erosion/accumulation or other important interests, then numerical modelling of the currents should be performed. In some cases it can be sufficient to make a local model that only involves the area concerned, but in most cases the currents are connected to the circulation in the sea, and in those cases it is necessary to start with a larger model of the North Atlantic and work downwards in a constantly improving spatial resolution until you are down at the breakwater site.

Figure 6-8 and figure 6-9 show an instance from Kråkvågsvaet between the islands of Kråkvåg and Storfosna at the mouth of Trondheim Fjord. Here, it was desired to build a connecting bridge with a fill/breakwater on both sides and a bridge in the middle. Because the water body is a part of a nature reserve, it was important to build it so that the conditions for fish and birds along the beaches on both sides were undisturbed. The model that was made (by SINTEF) showed the distribution of the current speeds in the inlet before and after construction of each alternative. In figure 6-9 the northern alternative is shown.

See also discussion of the effect of currents on the breakwater trunk in chapter 5.2.

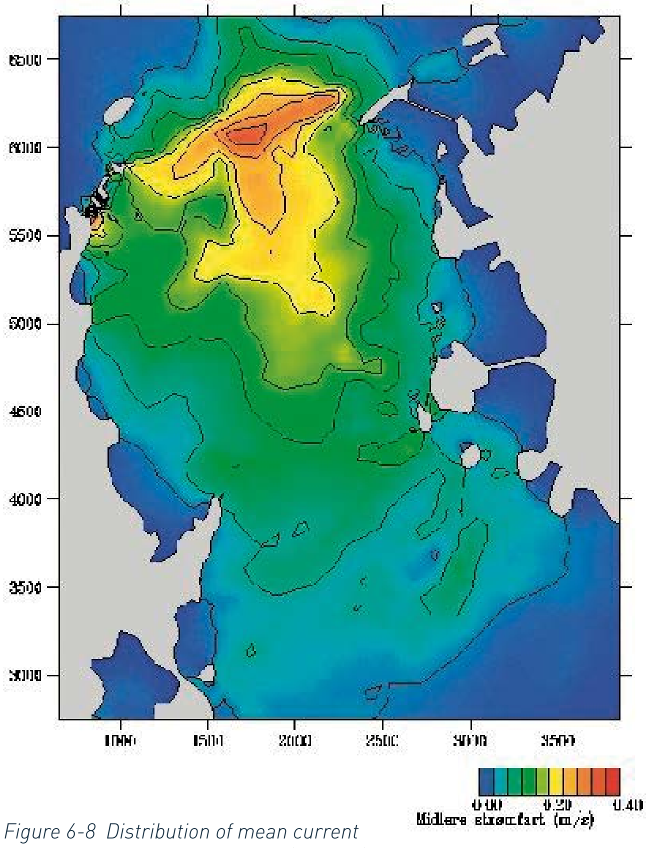


*Figure 6-8 Distribution of mean current speed in Kråkvågsvaet, existing (SINTEF).*

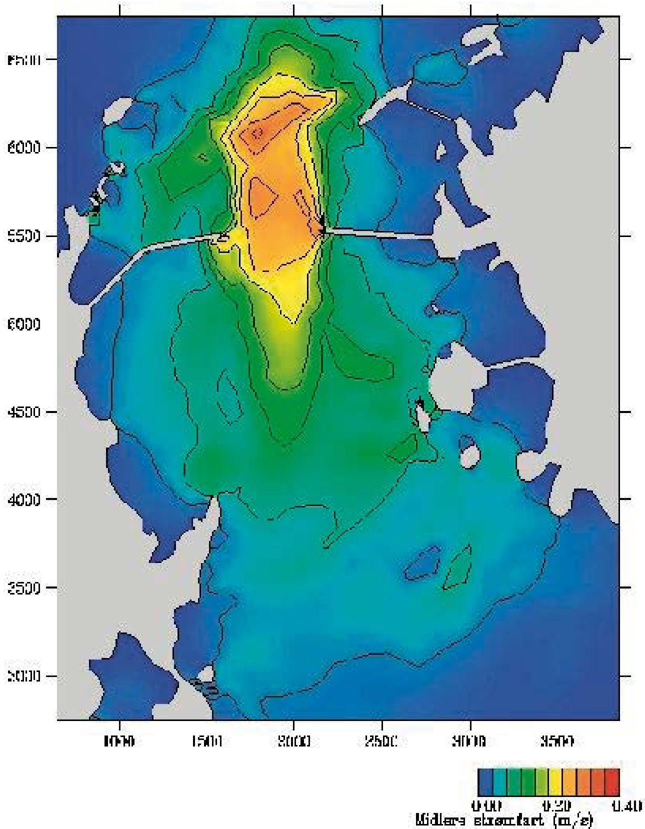


*Figure 6-9 Distribution of mean current speed in Kråkvågsvaet, northern alternative (SINTEF).*

| ATTACK DIRECTION ON VESSELS | WAVE PERIOD Tp (s) | RECURRENCE INTERVAL / FREQUENCY | | |
|---|---|---|---|---|
| | | **50 years** | **1 year** | **Weekly** |
| **Head-on** | $T_p$<2.0 | | 0.31 | 0.30 |
| | 2<$T_p$<6.0 | 0.61 | 0.30 | 0.15 |
| | 6.0<$T_p$ | 0.61 | 0.30 | 0.15 |
| **From the side** | $T_p$<2.0 | | 0.30 | 0.30 |
| | 2<$T_p$<6.0 | 0.23 | 0.15 | 0.08 |
| | 6.0<$T_p$ | 0.23 | 0.15 | 0.08 |

*Table 6-1 Maximum values for significant wave height in a small craft harbour, as per ref. XXI.*

| | ATTACK FROM THE SIDE | | ATTACK HEAD-ON/AFT | |
|---|---|---|---|---|
| | **Wave period $T_p$ s** | **Sign. Wave height m** | **Wave period Tp (s)** | **Sign. Wave height m** |
| **Boat length 4–10 m** | $T_p$<2.0 | 0.20 | $T_p$<2.0 | 0.20 |
| | 2.0<$T_p$<4.0 | 0.10 | 2.0<$T_p$<4.0 | 0.15 |
| | $T_p$>4.0 | 0.15 | $T_p$>4.0 | 0.20 |

*Table 6-2 Maximum values for significant wave height in a small craft harbour, see ref. The specified values may occur one or several few time per year.*

| | SEA WAVES AND SWELL | LOCAL WIND-GENERATED WAVES |
|---|---|---|
| **Spectral peak period $T_p$** | 10–18 s | 2–6 s |
| **Maximum $H_s$ with 1 year recurrence interval** | 0.5 m | 0.7 m |

*Table 6-3 Guideline criteria fir pier conditions for ships with lengths of 50–120 m.*

## 6.5 CRITERIA

Criteria for good harbour solutions are difficult to define, and will vary depending upon the type of harbour, type and size of boats/ships and the use of the harbour (24-hour continuous pier operations, berthing only, sporadic use, etc.). For the majority of harbours, it is sufficient to evaluate the wave height at the pier in order to determine the quality of the harbour. For larger facilities (piers for larger tanker and container ships) analyses should also be performed to specify the ship's movements at the pier, fender forces and mooring forces.

Smaller boats (leisure time boats, fishing boats and small smacks) generally have a low tolerance for waves. This is due in part to these vessels being short, and moving easily in the waves, but also that it would be risky to move between the boat and pier when the boat is undergoing large movements. PIANC (see Table 6-1) has specified some guideline values for small craft harbours. In Norway it is common to use somewhat higher values (up to 20 % above the PIANC values) with the justification that we presume that the population in Norway is somewhat more used to challenging weather along the coast than the population in Central Europe.

For larger boats (fishing vessels of sizes such as trawlers/purse seiners, live fish carriers and boats up to the size of the ships of the Norwegian Coastal Express) the requirements are more unclear, and a clear connection between pier quality and wave heights is difficult to define. In a case where detailed analyses cannot be performed for a specific ship, then guideline values from Table 6-3 can be used.

## 6.6 LANDSCAPE AESTHETICS

### 6.6.1 In general concerning landscape aesthetics

The breakwater's influence on the aesthetics of the landscape has traditionally not been given a great deal of attention when a breakwater is built. This has been the situation despite a breakwater being a large, irreversible initiative that is quite visible in the landscape. Many older breakwaters are, however, experienced as natural continuations of landforms, built with local rock, anchored well and adapted to the existing landscape. There can be favourable side effects due to costs, the construction method selected and where the shallowest areas are.

Today, there is a larger variation in the designs and it is not necessarily local rocks that are most suitable or the least expensive. If a new breakwater is to be adapted to the landscape in a good manner, it requires a certain understanding of the landscape. How the breakwater is designed and adapted to the landscape that it will be a part of will be crucial to the visual harmony of the landscape situation. In the planning of a new breakwater it may be relevant to prepare a report on the landscape situation, which would comprise a useful contribution to the design of the breakwater.

### 6.6.2 Breakwaters as significant landscape elements

Elements that can be relevant to the breakwater's landscape aesthetics:

- Landscape element. Breakwaters always protrude out of the lower-lying sea, with a different material, shape and colour. It places a clearer line in the landscape. This is a significant difference from for example a protective embankment against rockslides and avalanches, which is another large and man-made landscape element. Such protective embankments have tall mountains behind them and thus are less dominating in the landscape.
- Space divider. A visual and physical separator between what is inside and outside the breakwater. A new breakwater will change the view for the people who live there. The sea will be calmer behind the breakwater, which can be experienced both physically and visually. The local climate will be improved behind the breakwater.
- Anchoring. In the planning, it is natural to answer questions such as:
  - How can the breakwater be anchored well to the existing landscape/coastline ?

*Figure 6-10 Breakwater in the fishing village of Ballstad, Lofoten. Delineates the harbour and the location. Photo: Jorun Karlsen.*

  - Can the shape of the existing terrain be used as a point of departure for the breakwater root?
  - What is a natural division of space in the landscape?
  - Are there islands, islets or reefs that can be extended?
- Rock material. It should be clarified whether armour units can be selected of a rock type that is suitable for the landscape, for example with colour nuances that fit in with the location. Light rock is quite visible where the surroundings are darker.
- Idiom. It is important to highlight equality in the idiom across the entire breakwater structure, also where existing breakwaters are repaired or extended. A good result is achieved most easily where a conscious grasp is made of the idiom, either harmony or contrast with the surroundings is chosen.
- Identity. The breakwater is often a partial closing of a harbour. The main function is wave breaking, however the breakwater will become a part of the landscape aesthetics that contribute to defining the location. An example of this is the fishing village of Ballstad in Lofoten, see figure 6-10.

### 6.6.3 Examples of prominent construction projects in the landscape

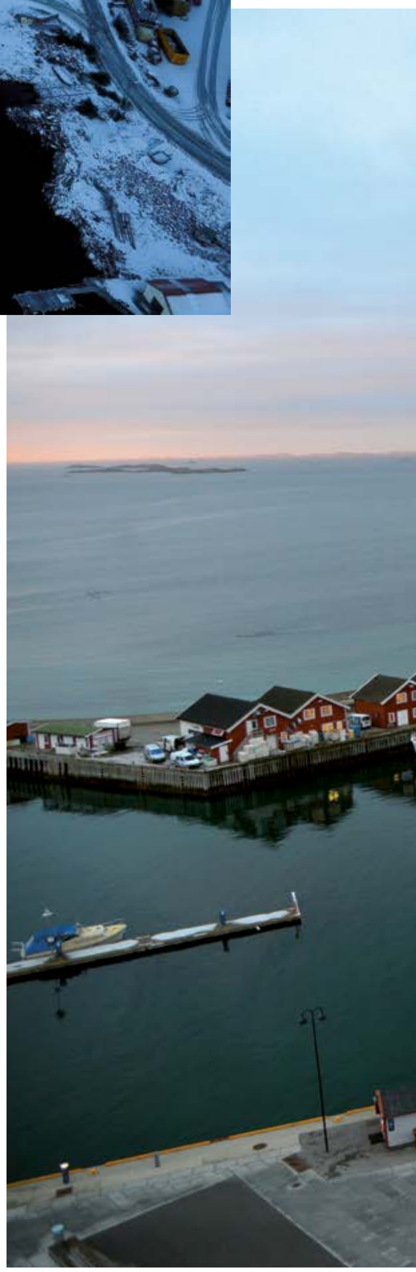
Figures 6-11, 6-12, 6-13 and 6-14 show four Norwegian examples of prominent construction projects in the landscape. These are Vannvåg at Vannøya in Troms, the fishing village of Berlevåg in Finnmark, the breakwater in Mehamn in Finnmark and the breakwater around the Austevoll fisheries harbour in Hordaland.

*Figure 6-11  Vannvåg, Karlsøy Municipality. (Photo: Norwegian Coastal Administration).*

*Figure 6-12  Berlevåg, Finnmark. The breakwater is fundamental for the business activity and thus the existence of the fishing village. (Photo: Tore Larsen, Berlevåg).*

*Figure 6-13  Mehamn, Finnmark – round armoured breakwater. (Photo: GeoNord AS).*

*Figure 6-14 Austevoll fisheries harbour. (Photo: LNS Saga AS).*



### 6.6.4 The breakwater in its local environment

A breakwater will affect the local environment where it will be located. This effect will vary from place to place, but an assessment should be made of the secondary functions of the breakwater. This is especially important if the breakwater is to be located in connection with a town or densely populated area. The breakwater's lee side has the greatest experience value for the local environment.

- **Safe traffic on the breakwater.** Some breakwaters are designed for or arranged for traffic, whereas others are absolutely not suitable for traffic. Those breakwaters that are designed for traffic are often attractive as destinations for outings. Assessments ought to be made of whether arrangements should be made for traffic on the breakwater or parts of the breakwater and whether this could provide new qualities to the area. This is especially interesting in towns or densely populated areas, see the example in figure 6-15. Here, risk assessments during the planning phase are important, cf. item 7.5.11.
- **Sojurns/leisure time arena.** There are many reasons for the breakwater to be an attractive location for traffic: Walking, resting, bathing, fishing, waiting for the ferry/ express boat, local meeting place, etc. For accessible breakwaters it is desirable to have possibilities for sitting, local shelter and universal design.
- **Small craft harbour.** Behind the breakwater, it can be relevant to have a small craft harbour and activities that it involves.
- **Attraction.** Audiovisual experience of wave breaking is a quality that many people desire to experience either in close proximity or at a distance. Here, it must obviously include a safety aspect, cf. safe traffic above.

*Figure 6-15 A breakwater may have multiple secondary functions. The picture shows the breakwater in Bodø, which, in addition to shielding the harbour basin further in, is an attractive destination for outings and shields the small craft harbour. (Photo: The Norwegian Public Roads Administration).*

Bodø Harbour. Photo: The Norwegian Public Roads Administration

# 7. DESIGN OF BREAKWATERS

### 7.1 IN GENERAL CONCERNING DESIGN

Designing a breakwater involves defining which block sizes and rock fractions a breakwater will consist of, and which outer geometrical dimensions each class of rock elements should have. Designing also includes describing how the individual layers should be laid out (for example random placement, controlled placement, walling, etc.) A typical result of such design work is shown in figure 7-1 and 7-2. It should be noted that controlled placement, here, involves the systematic laying out of armour units as described in further detail in item 8.7.

In this chapter, it is a precondition that a decision to build a breakwater with an optimum placement and length has been made. The task consists of designing the breakwater so that it can be built with satisfactory safety, stability and with minimum use of resources.

What is meant here by satisfactory safety is both that the breakwater will function as intended and that it does not represent a new danger to life, health or the environment in either the construction phase nor the operating phase.

The objective of the design is to produce a breakwater cross-section that satisfies the following requirements:

a) solid and with the ability to withstand the design wave conditions that are calculated for the location
b) economical, i.e. least possible use of resources for attaining a completed breakwater
c) safety for life, health and the environment during the construction and operating phases
d) simplicity in its construction, i.e. a design that can be built with available means (construction machines, personnel, site facility areas and internal logistics)
e) good utilisation of rock material (applies especially in those cases where a separate local quarry is opened for the breakwater)

In the list above, items d) and e) may be perceived as a consequence of item b).

When developing the final design, it will be possible to make adjustments to the initial cross-section, so that adaptations to varying loading and availability of rock material can be made. It can lead to the cross-section being more composite in nature, but nevertheless simple to construct. Special attention must be given to the fact that controlled placement with armour units below a level of 1–2 m below Chart Datum are so difficult to carry out that it should not be expected to be possible to implement them. Here, loosely laid rubble should rather be used with the increased requirements for sizes and thicknesses that it involves.

In this chapter, only the design of breakwaters that are built from normal, good blasted bedrock is addressed. This type of breakwater is ubiquitous in Norway, and will in almost all cases be the most economical. Breakwaters of concrete blocks are used at two locations in Norway (Berlevåg and Ferkingstad), and breakwaters of such blocks will have their

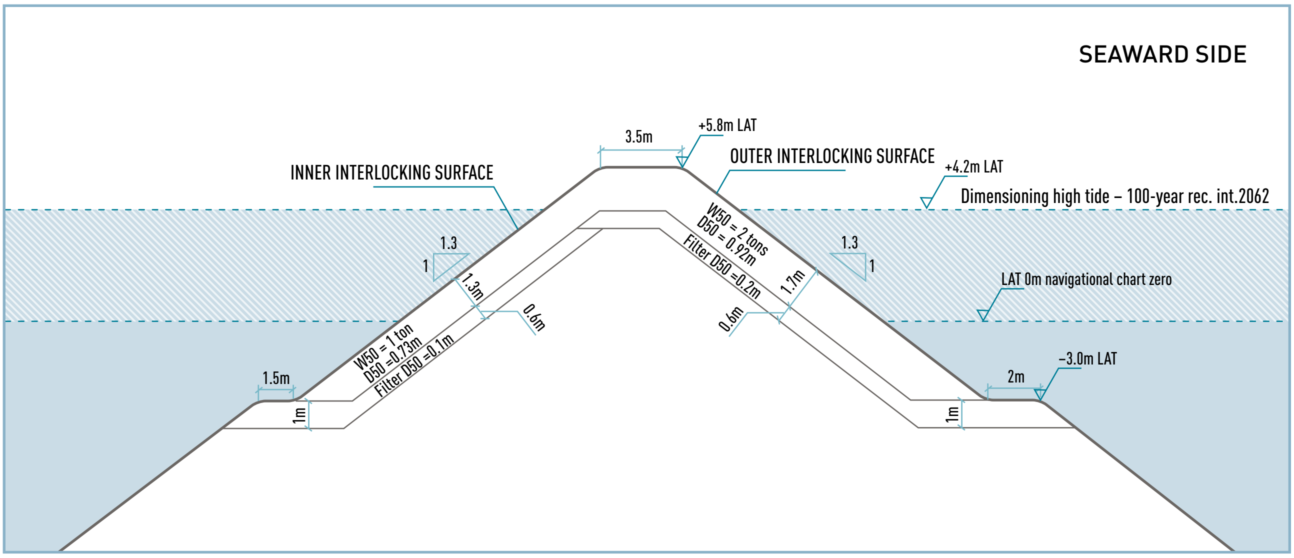



*Figure 7-1 Results of design: Typical cross-section of a conventional breakwater (Torhop, Norwegian Coastal Administration/Norconsult). The breakwater is composed of 2 different block fractions ($W_{50}$ = 2 tons and 1 ton) and 2 different filter fractions ($d_{50}$ = 0.2 m and 0.10 m) and unsorted core. In this cross-section, the termination against the bottom or the underlayer is not defined.*

own design criteria that are determined by the manufacturer of the blocks, and will not be addressed in these guidelines.

## 7.2 DESIGN METHODOLOGIES

### 7.2.1 Relevant methodologies

In the general design of structures, there is a differentiation between two manners of approach.

1. Deterministic analysis
2. Probabilistic analysis

A summary description of the methods is given below. For ordinary breakwater construction, the deterministic approach is ubiquitous. For larger and more complex breakwater constructions, a probabilistic method can provide benefits by making it possible to be more precise when determining the design and thus avoiding overdimensioning.

In these guidelines, deterministic methods are treated and described as standard methods. Project planning consultants may choose to utilise probabilistic methods if these can be substantiated and documented.

It should also be mentioned that at the point in time these guidelines are being launched, work is on-going with the issuing of a Eurocode standard for calculation of loads from waves and currents on coastal structures, EN 1991-1-8. Because the results of this work are still unknown, these guidelines will not take this load standard into account.

### 7.2.2 Deterministic analysis

In a deterministic analysis, a certain level for the load parameters is calculated (for example with a 200-year recurrence interval) and it is postulated that as long as the load does not exceed this level, the structure will be stable.

For example, if a design 200-year significant wave height $H_{s,200}$ for a location has been calculated. At the same location, a 200-year storm surge is also given (including effects of climate change). For the value $H_{s,200}$ a range is then selected for the spectral peak period $T_p$, and this range is preferably several seconds lower and higher than the $T_p$ value that is registered for the highest $H_s$ value. Such a combination of two 200-year conditions can look like the following:

$H_{s,200}$ = 3.0 m (occurs at $T_p$ = 15.0 s), $T_p$ = 12.0–18.0 s at storm surge level $\eta_{200}$ = 3.8 m above the lowest astronomical tide (LAT).

By selecting such a dataset, it is a precondition that the 200-year significant wave height occurs at the same time as the 200-year storm surge. For the majority of cases along the Norwegian coast, high water levels and storms with large waves will be closely interrelated, so this combination

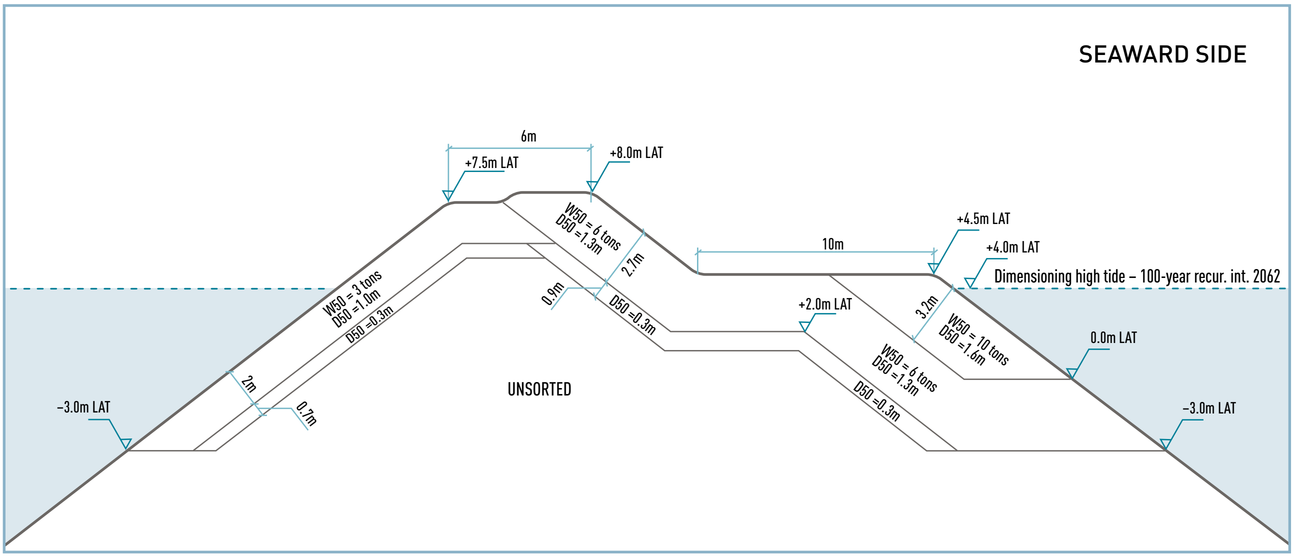



*Figure 7-2 Modification of the cross-section in figure 7-1 in order to adjust for increased loading near the breakwater head (Hasvik, Norwegian Coastal Administration/Norconsult).*

is not unnatural, even though the recurrence interval for the combined event is probably significantly higher.

At the same time, the selection of $T_p$ is based on the highest $H_s$ value (i.e., $T_p$ = 15.0 s in our example). If we choose to go a bit to the side and investigate the requisite block sizes for $T_p$ = 12.0 s, we will ultimately need to take into consideration that the highest $H_s$ value for this period is somewhat lower. I.e. the selected $H_s$ and $T_p$ combination should actually be selected along the ($H_s$,$T_p$) contour with a recurrence interval of 200 years. By also retaining $H_{s,200}$ = 3.0 m for a lower (or higher) $T_p$ value, we will in fact be utilising situations that have a recurrence interval higher than 200 years.

A deterministic analysis for the cases involving breakwaters will thus generally be conservative, and that will compensate somewhat for the lack of safety coefficients, neither for loads nor for capacity.

It should, however, be noted that a deterministic analysis may overvalue the level of safety. Even through the structure will avoid collapse as long as the load does not exceed the selected combination of load parameters, the structure may be worn down or fatigued by repeated events under this level.

For dynamic structures, i.e. structures that have a response that depends on both the size and period of the load, the maximum response may arise for periods other than those that are defined in the load parameters.

All experience indicates, however, that a deterministic analysis for breakwaters will be conservative.

### 7.2.3 Probabilistic analysis

A probabilistic analysis attempts to isolate the characteristic strength or the capacity R from the design load S. These two factors can be described using statistical parameters so that we can compute a probability for a high load occurring simultaneously in time and place with a low resistance capacity. It is common to presume that the parameters can be described by familiar statistical distributions, for example a Weibull distribution for waves and a Normal distribution for block weights.

The most simple form of probabilistic analysis is the use of partial safety coefficients, where a safety coefficient for, for example, wave height and block weight is used. The procedure for this method can be found in, for example, PIANC 1992, ref. VI.

Probabilistic analysis as a basis for designing breakwaters is seldom used for breakwaters of the size and type that are relevant in Norway. The reasons can be many, but the method is generally more complicated than deterministic methods, and it has not been demonstrated yet that it provides considerable operational or financial benefits.

As stated, deterministic design is usually conservative, but it does not give a good description of the breakwater's reliability (or probability of damage). Probabilistic design, in contrast, does this.

### 7.2.4 Selection of methodology

A deterministic analysis is a sufficient basis for determining the design basis for a breakwater.

In the project planning for breakwaters, a probabilistic analysis may be chosen, however a parallel deterministic analysis must always be performed in order to compare the methods.

If a probabilistic method is chosen as the basis, then the two methods must be compared, and the consequences of the choice made clear.

## 7.3 SELECTION OF BREAKWATER TYPE – BREAKWATERS OF BLASTED BEDROCK

The most relevant types of breakwaters in Norway are the following breakwaters built of ordinary, blasted bedrock:

1. **Conventional breakwater** consisting of a core, filter layer and a layer with armour units, where everything is laid out with the same slope, most often 1:1.3–1:1.4. This breakwater type was previously called a rubble breakwater or a single-layer breakwater, however these designations are misleading today and ought to be avoided.

2. **Berm breakwater** is a relatively new variant that has been used at locations with a large wave load since approximately 1985. Such a breakwater consists of an ordinary core with filter, but on the seaward side the front is built out with a "berm" comprised solely of large blocks, without any elements of smaller rocks. The berm can be 5–10 m wide, and will absorb the wave energy of water flowing into the berm embankment.

For design conditions, only the types mentioned in items 1 and 2 will be addressed here. See also the more detailed description of the different types of breakwaters in chapter 3.

In Norway, breakwaters are usually steeper than what is used in many other countries. There are a number of reasons for this, including that the water rapidly becomes deep in Norwegian fjords, and that a more gently sloped breakwater would require a much larger fill volume. The cores of Norwegian breakwaters were previously filled up from dumper/tipper trucks. The natural slide angle after some slope relaxation and reshaping by the waves is then approximately 1:1.3, and in such case the filter and armour units can be placed directly on top. It is also the case that in Norway all breakwaters have controlled placement with interlocking surfaces, whereas in other countries loosely laid rubble mounds and a slope of 1:1.5 are more common.

Slopes steeper than 1:1.5 are not covered in experiments that form the basis for design formulae, however the experience is that a number of breakwaters built using this method are standing up well and without any substantial damage.

## 7.4 DESIGN SEA STATE

### 7.4.1 General

For a deterministic analysis, a design sea state will be defined, which gives the load from the sea that the breakwater must withstand without structural damage and at the same time fulfil its function as a breakwater. The following variables or parameters are included in the design sea state:

- Significant wave height
- Spectral peak period
- Incident direction for the waves against the breakwater
- Design water level
- Recurrence interval

The procedure for establishing the design sea state is based on establishing the highest significant wave height $H_{s,q}$ with a given annual exceedance probability q. Alternatively, the probability can be specified as a recurrence interval. A wave height with recurrence interval $T_R$ (years.) has an annual exceedance probability $q = 1 - \exp\left(-\frac{1}{T_R}\right) \approx \frac{1}{T_R}$ for $T_R >> 1$. For example, a recurrence interval of 1 year corresponds to an annual exceedance probability q = 0.63, whereas a recurrence interval of 200 years gives q = 0.005.

The highest $H_s$ with a given recurrence interval is found by adapting a statistical distribution (usually *Weibull*) to the available wave data. Thereafter, a statistical distribution for $T_p$ is adapted depending upon $H_s$ (for example *log-normal*) (Ref. DNVGL-RP-C205).

For some types of breakwaters, those that are sensitive to changes in wave periods (floating breakwaters, caisson breakwaters) or for design formulas that take wave periods into account (for example Van der Meer), it is theoretically possible that the highest loading does not occur at the greatest wave height. However, this seldom occurs when designing a conventional breakwater.

For design based on $H_s$, the input to the formula for stability of the breakwater $H_{s,q}$ is significant wave height with an annual exceedance probability q. For design based on both $H_s$ and $T_p$, stability must in principle be checked for all $(H_s, T_p)_q$ combinations with annual exceedance probability q. This means that sea states along a contour in the $H_s$-$T_p$ plane are investigated. The design sea state is defined as the most critical sea state in relation to the stability of the breakwater.

Waves and calculation of design wave conditions are described more thoroughly in item 5.6 and in appendix I.

### 7.4.2 Significant wave height $H_s$ or $H_{m0}$

The wave load on a breakwater can consist of locally generated waves that are formed inside for example a fjord, or of sea waves that are directly incident on the breakwater or in a stage where the sea waves have been attenuated and are characterised as swell.

The wave height from a combination of the two wave types in those instances where the waves can arise simultaneously and have nearly the same direction is calculated by:

$$H_{s,total} = \sqrt{H_{s,lokal}^2 + H_{s,havsjø}^2}$$

$H_s$ is significant wave height here, which is defined as the mean value of the highest one-third of all waves in a registration or a storm with a duration from approximately 20 min to 3 hours. $H_s$ is strictly speaking a value that is based on counting of the number of waves in a measurement series. If we do not have such a measurement series, or are utilising model data, we will use the value $H_{m0}$, which is an approximation of $H_s$, and is based on a wave spectrum.

Such a spectrum is shown in figure 7-3, which shows a one-peak wave spectrum (see also the example of a two peak

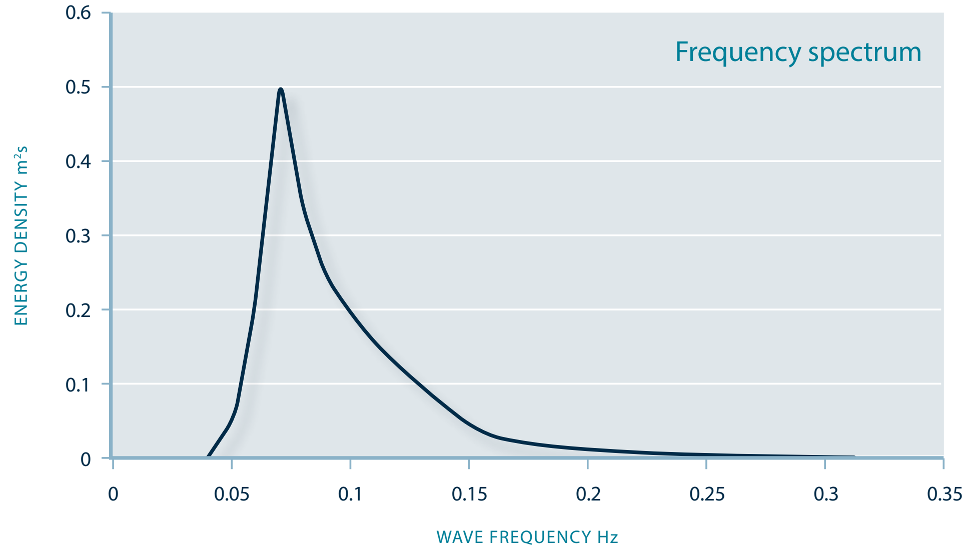



*Figure 7-3*
*Example of wave spectrum.*
*$H_{m0}$ = 0.6 m and $T_p$ = 1/0.07 s = 14.3 s.*

spectrum in figure 5-1). The wave spectrum is an illustration of how the wave energy (in practice $H^2$) is distributed at different frequencies. Long waves from the sea are on the left (low frequency) and short, local wind waves are on the right (high frequency).

The estimate of $H_s$ is now

$$H_s \cong H_{m0} = 4\sqrt{m_0}$$

$m_0$ is the area under the spectrum. This relation actually only applies for deep water, but may be used as a good approximation also for shallower areas.

The spectrum in figure 7-3 is a relatively smooth model spectrum. A spectrum based on measurements would be much more untidy with many local peaks.

Even though $H_s$ and $H_{m0}$ are two values that are not identical, the difference is so small that for purposes of designing breakwaters, they can be used interchangeably.

A design significant wave height must normally have a recurrence interval of 200 years. A robust method must be used for determining the wave height at the location, and the method, can be based on local wind data and/or longer series of waves in the open sea. The lengths of the time series used ought not to be less than 25 years (more on statistical analysis in Appendix I: General Wave theory).

### 7.4.3 Spectral peak period Tp

Tp is defined as the period for the frequency component in the spectrum that contains the most energy, and will normally be the period that is perceived as being dominant when considering the sea. Local wind-generated waves and sea waves/swell normally have respective period intervals of $T_{p,local} \approx$ 4.0–8.0 s and $T_{p,swell} \approx$ 10.0–18.0 s.

In figure 7-3 we see one spectral peak that gives $T_p$ = 1/0.07 s = 14.3 s.

$T_p$ for local wind-generated waves is set equal to the value that comes from the analysis of wind-generated waves from the area.

For sea waves/swell, an interval of a minimum of ± 2.0 s around the most probably value must be selected.

When we use the formula for $H_{s,total}$ above, the associated $T_p$ value must be set equal to $T_p$ for the wave component that contains the most energy. If the contributions are nearly the same, then the highest $T_p$ value will be preferred.

### 7.4.4 Incident direction for the waves against the breakwater

The load on a breakwater is greatest when the waves are directly incident upon the breakwater (normally on the breakwater's longitudinal axis). If it can be demonstrated

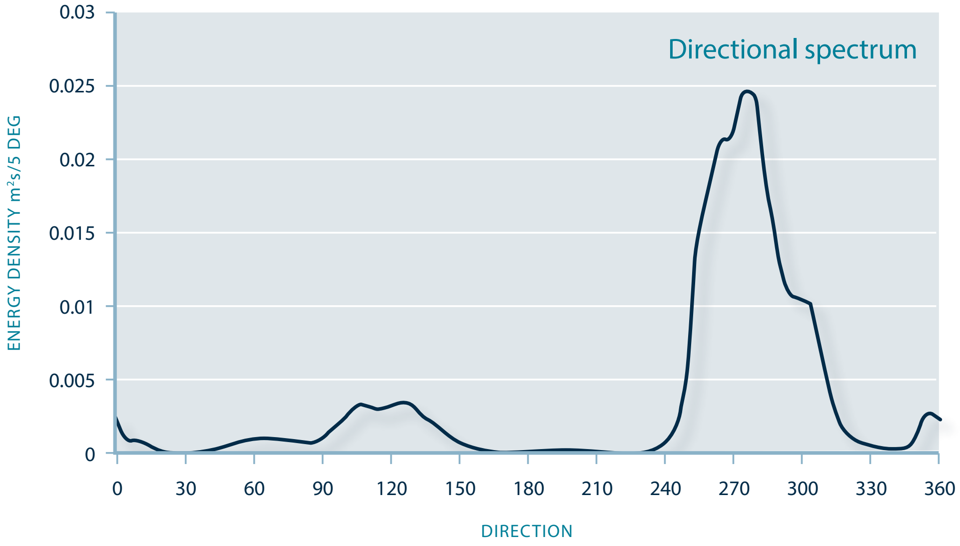



*Figure 7-4*
*Directional spectrum for the waves in figure 7-3. The majority of the wave energy is incident at a direction of 275°, but there are also small components between 90° and 150°.*

that a design sea state has a significant deviation (more than 20°) from a normal attack, then a reduction may be undertaken of the requisite block size, see also the details below.

The direction of incident waves will normally be the direction that the design waves are presumed to have. If necessary, calculations must be performed for multiple directions if the angle of attack for the most dominant direction can lead to waves from a less dominating direction being able to represent a greater load on the breakwater.

Figure 7-4 shows a directional spectrum for the waves in figure 7-3. This figure shows how the energy is distributed by direction. The reason that a small part of the energy separates out at 75° may for example be that a small portion of the wave energy is passing on the eastern side of an islet or a reef and is incident in a more easterly direction.

### 7.4.5 Design water level

#### 7.4.5.1 General guidelines

For every breakwater, the probability of an extreme wave event and an extreme storm surge event occurring simultaneously must be evaluated.

Normally, extreme storms will be connected with extreme water levels, such as was observed during the storm "Berit" in 2011. It is then realistic and slightly conservative to presume that an extreme storm event occurs simultaneously with an extreme storm surge.

For some projects, this connection between the two events can be weak. This will for example be the case for places where the design wave load is local fjord sea from the eastern sector and the open sea lies to the west.

In such cases, the project planner can propose a combination of events with different recurrence intervals, for example a 200-year wave height combined with a 10-year storm surge.

It is also necessary to consider the consequences of anticipated climate changes, see item 5.1. This is discussed below. The Norwegian Directorate for Civil Protection (the "DSB") has prepared guidelines for the country's municipalities to be utilised in the planning. Breakwater structures are in general simple to maintain or expand if the structure needs to be adapted to a higher storm surge level than anticipated at its construction. Choosing values for storm surges can thus be considered that are somewhat lower than those specified by the DSB's guidelines. But if the breakwater must protect an area that is utilised or will be utilised for residences, businesses, industries or other purposes, then people will also be present here during extreme weather. In such cases, the DSB's guidelines are to be utilised directly.

| SAFETY CLASS FOR FLOODING | CONSEQUENCE | LARGEST NOMINAL ANNUAL PROBABILITY |
|---|---|---|
| F1 | small | 1/20 |
| F2 | medium | 1/200 |
| F3 | large | 1/1000 |

*Table 7-1 Safety classes for construction in flooding-exposed area (TEK17).*

| | |
|---|---|
| 200-year storm surge at the point in time of the project planning can be obtained from the nearest / most suitable standard harbour | xxx cm above reference level |
| + estimated net rise in average water level at the project site, with justified selection of variable parameters. | + yyy cm net rise |
| = design storm surge | = zzz cm above reference level |

*Table 7-2 Method for determination of design water level.*

Project planning for a breakwater must follow the rules and regulations applicable at the relevant point in time. As of 2018, Regulations on technical requirements for construction work ("TEK17") applies, see section 7-2 Protection against flooding and storm surges All structures will be placed in a safety class in accordance with table 7-1 taken from TEK17.

Breakwaters are not especially encompassed by the code of regulations in TEK17, but buildings and areas that the breakwater will protect will be encompassed by for example rules concerning protection against flooding and storm surges.

"Largest nominal annual probability" is the annual exceedance probability, so that a value of 1/200 corresponds to a recurrence interval of not less than 200 years.

Flooding is in this connection any permeation of water into locations that are not intended for it, and will for example also encompass water from overtopping of waves.

A recurrence interval of 20 years will be relevant only for breakwaters that are deemed to be temporary. Other breakwaters that are planned for permanent use will naturally fall into class F2 (200 years) because they will normally act as protection for other structures that will be in class F2.

Note that the breakwater could be in a higher class due to the risk to buildings or structures behind the breakwater, or if the breakwater could be subjected to waves generated by slides (rockslides, landslides or avalanches); or if a collapse could involve a danger of loss of life.

#### 7.4.5.2 Tides and water levels

When designing a breakwater, regard must be paid to the highest level that the water could come up to (storm surge). The recurrence interval for a storm surge should normally be set to 200 years.

In the following, the concept of "net increase or change in water level" is used to describe the *change in water level relative to a fixed point in rock at the location (regardless of whether that bolt has also moved).*

Calculations of design, future storm surges must be based on calculated storm surges in the present situation (most recently available data for water level at the point in time of the project planning), with a supplement for the expected local increase in net water level at the location where the breakwater will be built, see table 7-2.

In the selection of values for table 7-2, the following apply:

a) the 200-year storm surge at the point in time of the project planning can be found from suitable calculations (for example the statistics part of the tide tables), or from the Norwegian Mapping Authority's Web pages. If the highest observed water level in a standard harbour is stated to be higher than the 200-year value, then one

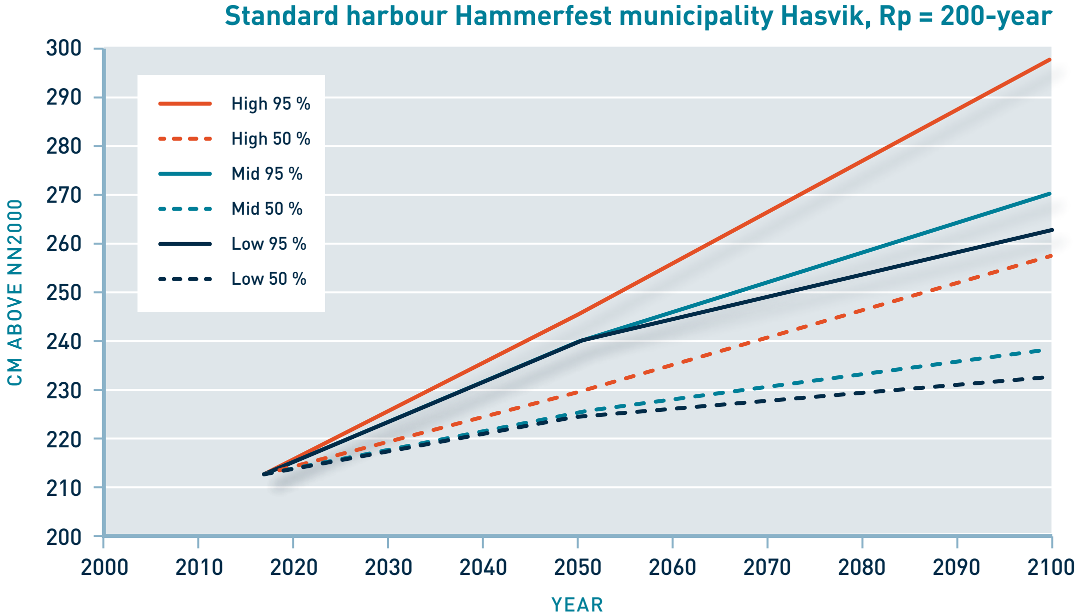



*Figure 7-5*
*Trend paths for 200-year storm surge in Hasvik. The present level is based on data from Hammerfest, and the trend up to 2100 is based on data for the Municipality of Hasvik (Simpson, et al., 2015, see ref. XVI).*

should choose a value equal to the highest observed water level + 10 cm.

b) As a standard harbour, we should select a representative standard harbour, which is not necessarily the nearest one. The tide table contains a list of secondary harbours under each standard harbour that can be of assistance in the selection of a standard harbour.

c) An estimate of climate-related change in the average water level must be fetched from the relevant municipality or the location where the breakwater will be built. The most recently available version of official documents should be used where net change is described.

d) The estimated net rise of the average water level must never be less than zero.

#### 7.4.5.3 Accepted methods

The most recently available publication with estimates of future sea level rises in Norway is specified in ref. XVI. In order to be able to find an estimate of future sea level rises, some input parameters must be selected.

a) Project horizon, i.e. figures up to 2041–2060 (2050), 2081–2100 (2090) or 2100.

b) Assumed emission scenario, choose between low RCP2.6, medium RCP4.5, or high RCP8.5, see below.

c) Ensemble spreading. Because the estimates consist of many models and simulations, the average value of the results (50 %) can be chosen, or a value that encompasses 95 % of all the results. 50 % is the most probable value here.

The scenarios for emissions of greenhouse gasses are summarised as follows:

I. RCP8.5 is a **high** emission scenario. Emissions of $CO_2$ increase to 300 % in 2100, and there is a strong increase in emissions of methane (in comparison with 1986–2005). Global temperatures rise by 3–5° C.

II. RCP4.5 is a **medium** scenario and involves large reductions in emissions. Reduction in total emissions of CO2 before 2040, and the concentration in the atmosphere is stabilised before 2100. Global warming approximately 2°C, which nevertheless leads to a water shortage in large regions.

III. RCP2.6 is a **low** emission scenario. The emissions drop from 2020, and the concentrations in the atmosphere drop from 2040. The scenario requires drastic changes in the consumption of oil and energy in general, and a stabilisation of the world's population at around 9 billion. Global warming is less than 2°C.

An example of what the trend may look like is shown in figure 7-5, which shows possible trend paths for the 200-year storm surge in Hasvik, with a point of departure in the standard harbour of Hammerfest.

*Note that storm surge does not include the effect of short-term water level variations such as for example waves.*

#### 7.4.5.4 Recurrence interval

Recurrence interval or return period is defined as the average time that elapses between a given level being exceeded two times in succession, for example a wind speed or wave height. For example it is expected that during the course of a 100-year period, we will register approximately 10 events where the 10-year level is exceeded, and approximately 5 events where the 20-year level is exceeded, etc.

The safety for a structure is determined by which recurrence interval it has been designed for. For breakwaters, safety corresponding at a minimum to a 200-year recurrence interval should be selected. The design water level can in contrast be kept low by for example choosing a project horizon up to 2050. This means that satisfactory safety is secured up to 2050, and at that time a new assessment must be undertaken, and that the breakwater must possibly be raised.

## 7.5 DESIGN OF A BREAKWATER'S DIFFERENT COMPONENTS

### 7.5.1 Overview of relevant components

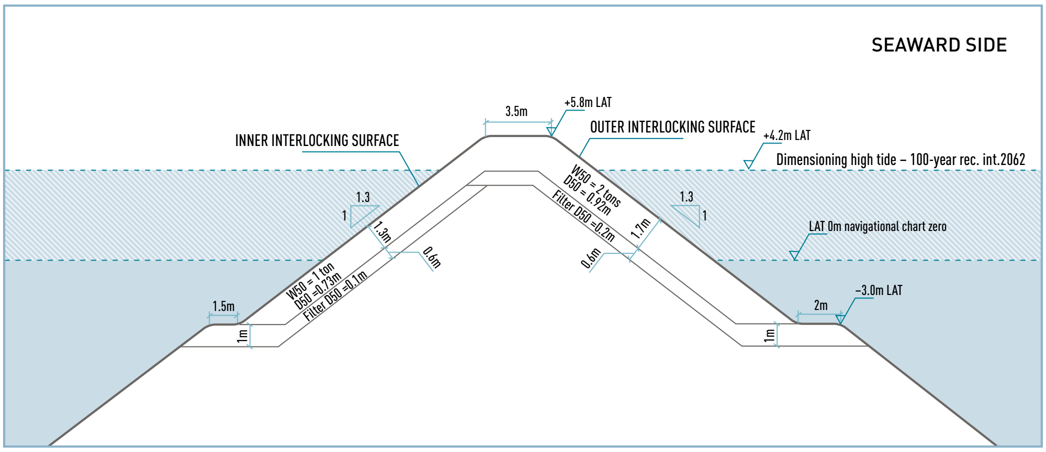



*Figure 7-6*
*The figure shows the relevant components in a typical, Norwegian conventional breakwater.*

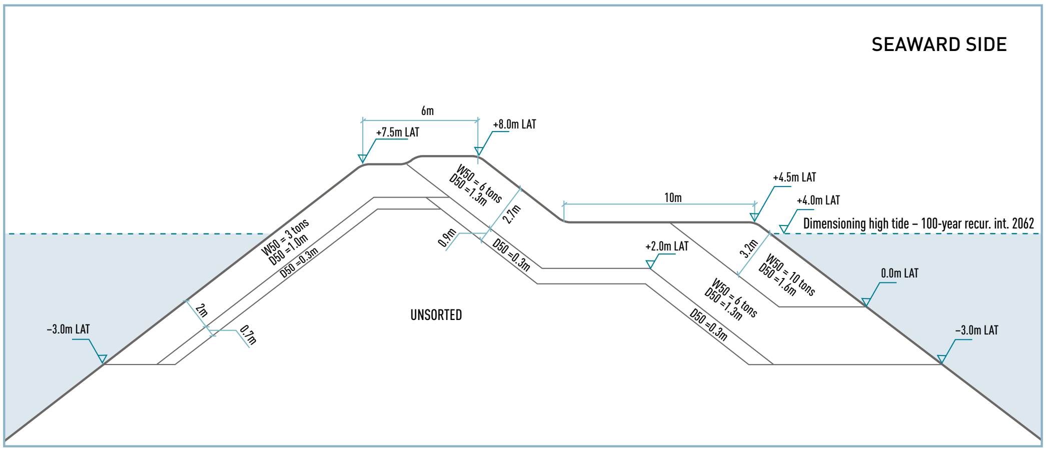



*Figure 7-7*
*Corresponding design for a berm breakwater.*

| MAIN THEME | COMPONENT | TYPE OF BREAKWATER<br>C = CONVENTIONAL<br>B = BERM BREAKWATER | CRITERIA |
|---|---|---|---|
| **Breakwater trunk** | Armour layer | C, B | Requirement for stability during design storm |
| | Filter | C, B | Must prevent flushing out of the core material |
| | Core material | C, B | Primary function is to carry the breakwater |
| | Breakwater height | C, B | Must primarily limit overtopping and prevent impacts on the lee side |
| | Breakwater width | C, B | Will most often be given by construction engineering conditions, but will also contribute to limiting overtopping |
| | Slope angle, Front | C, B | Should be selected so that the slope angle for the core, filter and armour layer are equal; most economical to choose a slope of around 1:1.3 |
| | Reinforcement of breakwater head | C, B | The breakwater head must in many cases be reinforced in order to compensate for lacking side restraint for the blocks |
| | Berm width | B | Selected so that the attenuation of the wave is optimum |
| | Berm height | B | |
| **Details** | Crown wall and roadway | C, B | Must possibly be deigned for overtopping |
| | Securing of breakwater top | C, B | The breakwater top must be secured so that blocks do not fall down during overtopping |
| | Securing of breakwater toe | C, B | Where the breakwater is sitting on a bottom subject to erosion, smooth bedrock or a steep slope, the toe must be secured |

*Table 7-3 Relevant components of breakwaters, see Table 3-2 for more details on the functions for each component.*

### 7.5.2 Armour unit

#### 7.5.2.1 Conceptual model

All design models for hydraulic stability of armour elements are based on one or more forms of equilibrium of forces. A simplification of these forces (without taking into consideration effects such as contact forces or the like.) is shown in figure 7-8.

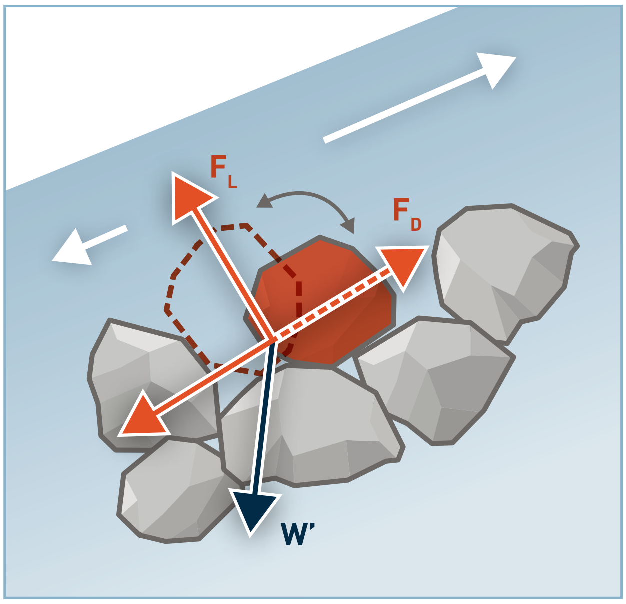



*Figure 7-8*
*Diagram of forces affecting the stability.*
*$F_L$ = uplift force, $F_D$ = drag force, W' = submerged weight.*

In this model, the currents from the waves (the drag force and the uplift force) attempt to move the block. These forces are proportional to the cross-sectional area of the block, i.e. $D^2$. The gravitational force is the force that tries to keep the block in place, and it is proportional to the volume of the block, i.e. $D^3$.

The ratio between the stabilising and destabilising forces, after some algebraic simplifications, give us the stability figure ($N_s$), which is dimensionless:

$$N_s = \frac{H}{\Delta D_{n50}}$$

where there $D_{n50}$ is the nominal diameter of rocks (where 50 % of the rocks have a diameter above $D_{n50}$), H= wave height, $\Delta = \frac{\gamma_s}{\gamma_w} - 1$ and $p \cdot g = \gamma$

This formula says in all its simplicity that the driving forces are represented by the wave height H (presuming shallow water), the stabilising forces by submerged volume weight and the diameter of the rock. We cannot find a mathematical value of $N_s$, but by performing experiments or taking measurements in the field, we can determine a value of $N_s$ for a given breakwater.

#### 7.5.2.2 In general concerning the layer with armour units

The armour layer's thickness and the size of the individual blocks can be determined by a number of different methods. For the design, a recognised method must be used that is based on theoretical studies, laboratory experiments and trials of actual instances. Hudson and Van der Meer are among the most used and recognised models for design of the armour layer, but there are many other formulae (Iribarren, Medina, Losada, Kobayashi, Ryu, or the like.). We must, however, make sure that the fundamental experiments that the model is based on represent both the project conditions and the selected breakwater solution.

The method used shall as a minimum include the following parameters or variables:

a) Wave height, given by a parametric wave height from a spectrum of irregular waves, for example $H_s$, $H_{m0}$, $H_{1/3}$ or the equivalent
b) Wave period, given by a parametric wave period from a spectrum of irregular waves, for example $T_p$ or $T_z$
c) Density of water and rock material used.
d) Slope of breakwater front or the bottom in front of the breakwater
e) Degree of damage permitted.
f) Number of waves in or duration of design storm

#### 7.5.2.3 Conventional breakwater

In Hudson's formula $N_s$ depends on the slope $\alpha$ and a parameter $K_D$ that takes into account the conditions associated with the laboratory experiment the model is based on (type of armour elements, type of waves (breaking/not breaking), part of breakwater (trunk/head) or the like). Note that in *Hudson's* model the reference wave height for natural stone is represented by $H_{1/10}$ and not $H_s$. The expression $H_{1/10}$=1.27·$H_s$ is usually used as an interrelationship between the two:

| PLUNGING BREAKER | SURGING BREAKER |
|---|---|
| $\xi_z < \xi^{'}$ | $\xi_z > \xi^{'}$ |
| $\frac{H_s}{\Delta D_{n50}}\sqrt{\xi_z} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}$ | $\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_z^{\,P}$ |

*Table 7-4 Van der Meer's formulae.*

$$N_s = \frac{H_{1/10}}{\Delta D_{n50}} = (K_D.cot\alpha)^{1/3}$$

*Hudson's* formula is simple to use. It is independent of the wave form (T, L) for regular waves. The method can be used in simple rough estimate calculations. Note that Hudson's formula was developed for regular waves, where the wave period is given by the wave height. The method is thus non-conservative in most instances in Norway, because breakwaters most often are subjected to sea waves that are somewhat attenuated. Thus we receive a (relatively) low wave height without the period being correspondingly reduced, and the forces on the breakwater will be larger than what Hudson's formula specifies.

For conventional breakwaters, Van der Meer's method should be used (see Table 7-4 and Table 7-5). This method is based on comprehensive laboratory studies and has been used ubiquitously in Norway since approximately 1980.

The formulae are written both for instances of *surging breakers* and for *plunging breakers*.

Which type of breaker is occurring is governed by the Iribarren number $\xi_z$, which is defined as:

$$\xi_z = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_0}}}; \quad L_0 = \frac{g}{2\pi}T_z^{\,2}$$

where $\alpha$ is the angle to the sea bottom's slope, $H_s$ is the significant wave height and L is the wave length. The sub-index 0 means that the parameter is calculated for deep water without any effect from the sea bottom and wave form. The Iribarren number determines the type of wave breaker, as shown in figure 7-9.

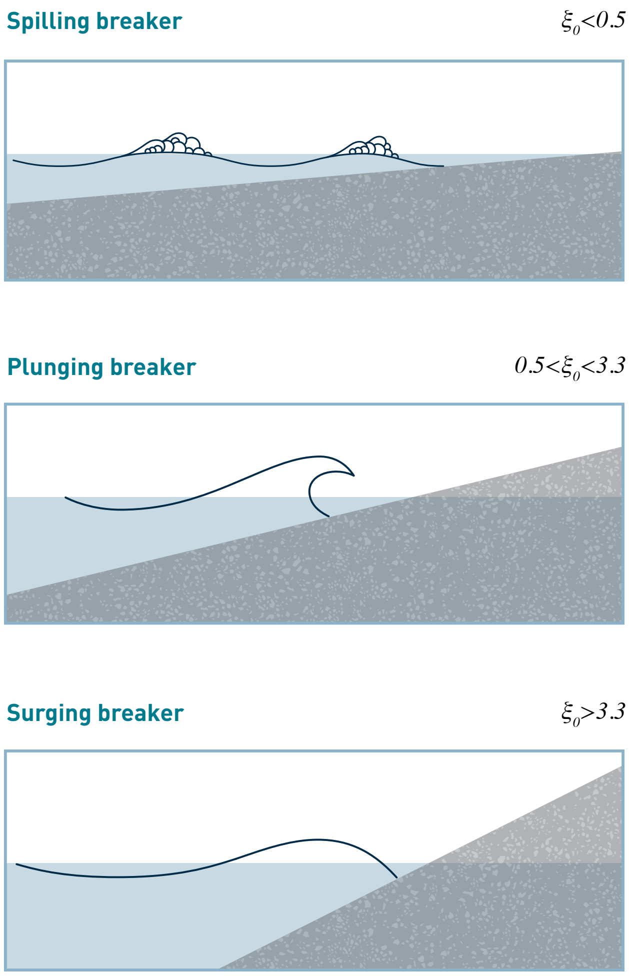



*Figure 7-9 Different types of wave breakers depending upon the value of $\xi$.*

The transition between a plunging breaker and a surging breaker takes place at a specific value of the Iribarren number that is given by:

$\xi' = \left(6.2P^{0.31}\sqrt{\tan\alpha}\right)^{\frac{1}{P+0.5}}$, also called the critical value of the Iribarren number.

**NOTE!** Block weights that come out of these formulae have a maximum point for values of the Iribarren number around $\xi'$, in other words the transition between the two formulae. For a given geometry and wave height, $\xi_z$ will be dependent upon the wave period $T_z$, and hence it is important to investigate the sensitivity by also selecting values of $T_z$ above and below the most probable value.

| SYMBOL | UNIT | COMMENT |
|---|---|---|
| $\xi_z$ | dimensionless | Iribarren number |
| $\xi'$ | dimensionless | Iribarren number at transition between breaker types |
| $H_s$ | m | Significant design wave height of incident waves at the toe of the breakwater |
| $\Delta$ | dimensionless | $\Delta = \frac{\rho_s}{\rho_w} - 1$ ; dimensionless relative buoyant density |
| $\rho_s$ | $kg/m^3$ | density, rock |
| $\rho_w$ | $kg/m^3$ | density, seawater |
| $D_{n50}$ | m | equivalent, average diameter of rock material |
| P | dimensionless | Permeability factor, see the table below |
| S | dimensionless | $S = A_e / (D_{n50})^2$ ; dimensionless damage number |
| $A_{el}$ | $m^2$ | Areal of the location with removed blocks, measured in the cross-section |
| N | dimensionless | (design) number of waves, usually between 1000 and 5,000 |
| $\alpha$ | ° | Slope angle of the breakwater |
| $L_0$ | m | Deep water wave length |
| $T_z$ | s | $T_z \approx T_p/1.35$; mean wave period |
| $T_p$ | s | spectral peak period in the spectrum; design wave period |

*Table 7-5 Symbols used in the calculation of armour unit sizes.*

**The permeability factor, P** expresses how easily the waves permeate into the breakwater, and ranges from 0.1 for an impermeable core to 0.6 for breakwaters without a core, refer to the table below:

| PERMEABILITY FACTOR P | THICKNESS OF ARMOUR LAYER | THICKNESS OF FILTER | CORE |
|---|---|---|---|
| 0.1 | 2 | 0.5 | impermeable |
| 0.4 | 2 | 1.5 | unsorted |
| 0.5 | 2 | without filter | unsorted |
| 0.6 | the entire breakwater consists of blocks, without filter and without core | | |

*Table 7-6 Different values of the permeability factor P for different types of breakwaters. The thickness of the armour layer and filter shown as a multiple of the rock diameter $D_{n50}$.*

For Norwegian breakwaters, an average value for P will lie in the range of 0.3 to 0.4, provided that a core is being used. For breakwaters without a core and only blocks, P= 0.6 is used.

The damage number, S expresses how large the area is of the cross-section where blocks have been removed, measured in terms of the number of blocks. If the damage number is for example 5, and block diameter $D_{n50}$ = 0.75 m, the cross-section of the area where blocks are removed would be the corresponding cross-sectional area of 5 blocks, i.e. 5 x $(0.75)^2$ = 2.81 $m^2$, or 2.81 $m^3$ per running metre of breakwater.

The Norwegian Coastal Administration recommends using a value for the damage number of S = 2.0, which gives a nearly statically stable breakwater. That is in practice no damage, and only a "shake-down" of the breakwater.

#### 7.5.2.4 Berm breakwater

A berm breakwater is a breakwater that has an extra thick layer of only large blocks on its exposed side. This block layer serves as an absorber and attenuates the wave motion and the speed in the water gradually such that the load becomes less, and the block size can be reduced. Due to this internal attenuation in the berm, the surging breakers on a berm breakwater will also be lower, enabling the height to be reduced.

Figure 7-7 shows a typical berm breakwater. This requires a larger volume for the rock fill and requires greater space in the horizontal plane than a conventional rubble mound breakwater, but smaller blocks can be used, which are less expensive to produce.

Berm breakwaters are most often used for the most exposed locations on the coast. Which breakwater type will be selected will be an economic assessment, but in general berm breakwaters will be competitive from a design wave height of $H_s \approx 3.0–3.5$ m and up.

| CLASS | DESIGNATION USED IN NORWAY | CODE | $H_0$ | $S_d$ | $REC/D_{n50}$ |
|---|---|---|---|---|---|
| 1 | Non-reshaping; Icelandic type | HR-IC | 1.7–2.0 | 2–8 | 0.5–2 |
| 2 | Partly reshaping; Icelandic type | PR-IC | 2.0–2.5 | 10–20 | 1–5 |
| 3 | Partly reshaping; homogeneous armour layer | PR-MA | 2.0–2.5 | 10–20 | 1–5 |
| 4 | Completely reshaping; homogeneous armour layer | FR-MA | 2.5–3.0 | -- | 3–10 |

*Table 7-7 Typical sizes for $H_0$, $S_d$ and $REC/D_{n50}$ for breakwaters of varying stability (see ref. XVII).*

In Norway, a large number of such breakwaters have been built, both as new construction and as renovation/repairs of breakwaters that have been damaged.

Experience shows that a breakwater with a berm breakwater head has a greater ability to attenuate waves that are rounding the head (due to diffraction and refraction, see item 6.2) than a conventional breakwater has. This is why composite breakwaters are also being built with a conventional trunk out to the breakwater head, which is then is constructed with a berm.

In the beginning, berm breakwaters were built with an intention for reshaping, i.e. that the profile was expected to be taken down into an S shape as the effect of storms. Such breakwaters typically had a low and broad shoulder. This technique led to large movements of each individual block, something that in turn caused wear and fracturing of the blocks. Hence, there has been a trend towards a profile where the berm is narrower and taller, and it is an intention that the breakwater be statically stable with minimal reshaping ("recession" – Rec) even for design storms.

A berm breakwater that reshapes is called "reshaping-statically stable" or "dynamically stable", because it has a nearly stable exterior shape even through there can be large movements of the rock elements.

At the other end of the scale are berm breakwaters that are "non-reshaping - statically stable", i.e. that the shape and the individual rock elements are unchanging.

The reshaping is measured here as the distance that the front edge has pulled back. A more modern classification is shown in Table 7-7.

Berm breakwaters built in Norway must as a general rule be statically stable, i.e. that the parameter $H_0$ < 2.0 (see also the section in item 3.2.3).

The berm breakwater principle is used most often with the largest structures, and hence it can be relevant to develop a theoretical design that is then tested and possibly verified in a laboratory model.

There is general agreement that a berm breakwater's fundamental design is governed by the following parameters:

$$H_0 = \frac{H_s}{\Delta D_{n50}} \text{; og } T_0 = \sqrt{\frac{g}{D_{n50}}}T_z$$

Less weight is currently being placed on the parameter $T_0$, even though it was previously recommended that $T_0$ < 70–80.

In a publication from 2017, van der Meer and Sigurdarson (see ref XVII) proposed values for the parameters $H_0$, the damage number $S_d$ and factor for reshaping $Rec/D_{n50}$ (dimensionless). The breakwaters are divided into groups of varying stability. The parameter selection is described in table 7-7.

A berm breakwater that is not statically stable will gradually be worn down to the bottom by the movement of the blocks, and the breakwater will attenuate both waves and overtopping more poorly because parts of the berm are gone.

**The general requirement for a berm breakwater in Norway thus is $H_0 \leq 2.0$.**

Table 7-8 shows which dimensions (with the specified symbols) are established during the design, see also figure 7-1.

- Block weight is calculated based on the following expression: $W_{50} = \rho_s (D_{n50})^3$
- The slope angle between the berm and the breakwater crest is set equal to 1:1.5 - 1:3 (1:$S_2$)
- The slope angle between the berm and bottom in front of the breakwater (1:$S_1$) is set equal to the critical friction angle (1:1.3-1:1.25).

| SYMBOL | UNIT | COMMENT |
|---|---|---|
| $W_s$ | m | Berm width |
| $H_k$ | m | Breakwater height above design high tide |
| d | m | Water depth at toe of breakwater |
| $H_s$ | m | Significant design wave height |
| $\Delta$ | dimensionless | $\Delta = \frac{\rho_s}{\rho_w} - 1$ ; buoyant density |
| $\rho_s$ | kg/m³ | density, rock |
| $\rho_w$ | kg/m³ | density, seawater |
| $D_{n50}$ | m | equivalent, average diameter of rock material |
| $W_{50}$ | kN | Block weight corresponding to $D_{50}$; $W_{50} = \rho_s D_{n50}^3$ |
| $T_z$ | s | Zero-crossing period "mean period" = $T_p$/1,35 |
| $T_p$ | s | Spectral peak period in the spectrum |

*Table 7-8 Symbols used in the design of a berm breakwater.*

### 7.5.3 Filter layer

#### 7.5.3.1 In general concerning filter layers

Filter is the material that is placed between the armour layer and the core. The purpose of the filter is to prevent small particles and parts of the core from being pressed out through the block layer. The water flow is restricted when passing through the filter, and the filter is designed to retain the particles of the core. At the same time, the filter must be adapted to the block layer, so that the filter itself is not washed out through the block layer.

It may be necessary to lay the filter out in multiple layers. The following designations are used to specify the rock size in the filter layer:

$$4 < D_{50f} / D_{50b} < 5$$

*$D_{50f}$ = average diameter in filter layer*
*$D_{50b}$ = average diameter in underlying (base) layer.*

In practice, this means that the step in median diameter between the two layers should have a factor of between 4.0 and 5.0. The filter should as much as possible be uniformly graded and not contain fine material.

The filter layer must have a thickness of a minimum of $3xD_{50}$. It is often difficult to lay out filters with such precision under water, and thus a larger thickness will often be specified and a large tolerance given.

For example: If a 75 cm thickness is needed, then a 1 m thickness+/- 25 cm is indicated.

It is believed that such filter criteria, which are relatively simple, will be the only practically usable one during the execution of the construction of the breakwater.

#### 7.5.3.2 Methods

The filter quality will in practice often lie in the size range that applies for crushed blasted rock, i.e. a quality that has passed through a sieve with openings of less than 600 mm. The aim should hence be to try to specify a filter quality that is in the vicinity of some of the commercial fractions that the crushing mills deliver, see table 7-9.

If the desired filter quality is $D_{50}$ = 200 mm, we can arrive at an acceptable material by specifying the quality 0/300 with everything under 120 mm eliminated.

| TYPE MATERIAL/SORTING |
|---|
| Unspecified |
| 0/4 |
| 0/8 |
| 0/11 |
| 0/22 |
| 0/63 |
| 2/4 |
| 4/8 |
| 8/11 |
| 8/16 |
| 8/22 |
| 11/16 |
| 11/22 |
| 16/22 |
| 16/32 |
| 22/32 |
| 22/63 |
| 22/120 |
| 0/300 – blasted bedrock |
| 0/600 – blasted bedrock |
| 0/25 – crushed asphalt |
| 0/60 – crushed concrete |
| Screened soil |

*Table 7-9 Commercially and readily available fractions of crushed bedrock. For example, 16/32 means all material remaining between a grating size of 32 mm and 16 mm.*

#### 7.5.3.3 Filter fabric / Geotextile

In some cases, it may be required that a filter fabric or geotextile be used. In general, a filter fabric is rarely used for breakwaters in Norway. The exception is breakwaters that are used as a containment jetty for deposition of contaminated material. In such case it may be relevant to use a filter fabric in order to prevent leakage of contaminated material. However, in such case the filter fabric will be laid on the inner side of the breakwater, and not on the side that is exposed to waves.

It is not relevant to build breakwaters with cores of contaminated material. It will not be possible to lay out the core without substantial and uncontrolled spreading of (contaminated) core material during the construction phase.

In some instances it is conceivable that a breakwater will border upon an area consisting of (firm) clay or other fine-grained material (sand, silt), and in such case it may be relevant to assess the use of a filter fabric.

If it is decided to use a filter fabric, then in nearly all cases it will not be advisable to place armour units directly on the fabric. The laying out of the blocks would cause damage to the fabric, and there is always a danger that the fabric will be pulled out of position by blocks that roll. If a filter fabric will be used, then in nearly all cases it must be protected by a layer of crushed stone that will have the same characteristics as filter stones.

A filter fabric or geotextile thus is relevant only in very special instances, and the need for a protective layer on the fabric causes there to not be any economic benefit in using the fabric.

### 7.5.4 Core material

The core of the breakwater consists of unsorted material from quarries, crushed stone, gravel, smaller blocks or the like. With the extraction of material from a quarry, what will most often be utilised for the core material are those materials that are not usable for other purposes. Note that it is not desirable that the core be open, and hence large blocks are only permitted to a limited extent. For the inner parts of the core, no special requirements are posed for *grading*, whereas for the outer parts (the outermost 2–3 m facing the filter), the composition of the core material must be checked and possibly adjusted so that it fits in with the filter design.

The following types of material are nevertheless not permitted anywhere in the core material:

- Clay and silt
- Soil, humus or other organic material such as sawdust, wood or the like
- Shell sand, muck or mud
- Plastic remnants from the blasting work
- Loose material containing environmental toxins (contaminated material, cl. II-V)
- Metals

The core material can consist of:

- Quarried rock with ordinary grading. If *good packing* is attained, there are in practice no restrictions on the sizes of blocks that are permissible, but the large blocks will most often be reserved for the armour layer and possibly the filter.
- Blocks and rocks that have a sufficient size to be armour units, but which are deemed to have an unsuitable shape for use in an interlocking surface.
- Crushed stone and gravel from a quarry or natural deposit. If a natural deposit is used, it should be supplemented with quarried rock in order to achieve good grading, because natural deposits of gravel are most often of a single grade.
- Tunnel material, surplus material from quarries for decorative stone or industrial stone.
- Quarry waste and fine material from quarries are permitted in up to 5 % if it is added evenly and continuously and not deposited in pockets or in separate locations.

It is not recommended that any forms of construction waste be used (concrete, asphalt, brick or the like).

*Figure 7-10*
*Breakwaters that may or will be subjected to overtopping must have interlocking surfaces with equally large rock elements on the lee side as well as the sea side. We must also assess the extent to which such a structure should be open to the public. (Gismerøy, Mandal. Photo: Norconsult).*

### 7.5.5 Breakwater height

#### 7.5.5.1 General

The breakwater height will determine the extent to which the breakwater will be in a condition to prevent the highest waves from going over the breakwater. A low breakwater will pose a number of challenges:

- Waves that go over the breakwater will create new waves on the lee side.
- Waves that go over the breakwater will prevent the lee side of the breakwater from being able to be used for example for quay berths.
- Waves that go over the breakwater will prevent the breakwater from being able for example to be used for a roadway, simple houses or buildings.
- Waves that go over the breakwater comprise a problem for the safety of the general public.
- If a breakwater is subject to severe overtopping, then the back side of the breakwater will be just as subjected to damage as the outer side, which means that the lee side must be secured in the same way as the sea side.

A breakwater can be wholly armoured, i.e. with its outer interlocking surface layer being extended up to and over the crest and then down on the lee side to the mean sea level. Such a breakwater will withstand much overtopping, and the structure does not invite ordinary traffic. In such cases, the height can be designed for an overtopping of up to 0.5$m^3$ per second per metre, with the preconditions mentioned in the list above.

If the breakwater is to be used for unspecified general purposes, then the requirement for maximum permitted overtopping will be relatively strict. A maximum overtopping of 10 l/(sm) will apply for structures where the public will traffic or there will be vehicular traffic (at an assumed low speed).

For structures that are prepared for water and some forces from waves, the tolerance is approximately 50 l/(sm).

Calculation of overtopping for conventional breakwaters can be performed using the method described in ref.  XX. Van der Meer, et al, describe a method for the calculation of overtopping on berm breakwaters in ref. XVII.

Guidelines for an acceptable overtopping rate can be fetched from the EuroTop Manual, and are reproduced in table 7-10.

In assessing safety for the general public, the following should be taken into consideration.

A. A breakwater will often attract the public during larger storm events (see figure 7-10). If a breakwater is built with a roadway/walkway it will be perceived as an invitation to safe use. If it in contrast is wholly armoured with large blocks on the entire breakwater, then it is more obvious to the public that this area is not a generally accessible area.

B. Which possibilities the public and other users have to assess the risk *before* they venture out on the breakwater must be evaluated.

C. The retreat possibilities must also be evaluated. This applies in particular for long breakwaters where there is a possibility for people to go out onto the breakwater and to then find out that the waves are too high to get back to safe ground.

D. Measures for signalling possible dangers to the public can be a warning sign (preferably also as a pictogram), wholly armoured breakwaters (no walkway/roadway), ban on traffic and closure with a fence.

| USER GROUP | USERS | ACCEPTABLE OVERTOPPING l/(sm) | | | EXAMPLES |
|---|---|---|---|---|---|
| | | MIN | AVERAGE | MAX | |
| Pedestrians/ public | General public | | 0.03 | | General public without requirements for equipment, ex. city streets, pavement/ sidewalks, etc. |
| | Aware public | | 0.1 | | |
| | Workmen with training and equipment | 1 | | 10 | Fishermen, harbour workers, hikers/walkers and cyclists |
| Vehicles | Low speed | 10 | | 50 | Max. limit applies for heavier vehicles, tractors, wheel loaders, etc. |
| | Moderate to high speed | 0.01 | | 0.05 | Ordinary road without requirement for reduced speed |
| Buildings and structures | Damage to equipment within 5–10 m | | 0.4 | | Buildings and equipment that are not prepared for water and flooding |
| | Damage to building parts | | 1 | | |
| | Smaller vessels sunk | | 10 | | Concerns boats that are moored on the lee side of a breakwater |
| | More substantial damage, medium-sized boats are sunk | | 50 | | |
| Breakwaters and embankments | No damage in unprotected area | | 0.1 | | |
| | No damage to grass and loose material | 1 | | 10 | |
| | Limit for damage to area where rocks are placed | 50 | | 200 | |
| Promenades and traffic areas | Damage to concrete deck | | 200 | | |
| | Damage to asphalt and wooden decks | | 50 | | |

*Table 7-10 Acceptability criteria for overtopping on breakwaters and fills in the sea, specified in l/(sm).*

| Run-up level n% (see def. below) | A | B | C | D |
|---|---|---|---|---|
| 0.10 | 1.12 | 1.34 | 0.55 | 2.58 |
| 1 | 1.01 | 1.24 | 0.48 | 2.25 |
| 2 | 0.96 | 1.17 | 0.46 | 1.97 |
| 5 | 0.86 | 1.17 | 0.44 | 1.68 |
| 10 | 0.77 | 0.94 | 0.42 | 1.45 |
| significant | 0.72 | 0.88 | 0.41 | 1.35 |

*Table 7-11 Coefficients in formula for overtopping calculation.*

#### 7.5.5.2 Methods for calculation of breakwater height

There are two different methods to choose between for calculation of the necessary breakwater height based on a requirement for maximum permissible overtopping. The method with respect to PIANC can be used for ordinary breakwaters where only the stability of the breakwater is to be investigated. In instances where life and health and the general safety of the public is involved, the EuroTop method should be used.

**Run-up calculation as per PIANC**

Run-up can be calculated using the following formulae taken from van der Meer and Stam (1992):
For the permeability coefficient P<0.4 (i.e. ordinary rubble mound breakwater with core):

$$\frac{R_u}{H_s} = a\xi_z\text{; for } \quad \xi_z < 1.5$$

$$\frac{R_u}{H_s} = b(\xi_z)^c\text{; for } \quad \xi_z > 1.5$$

$$\xi_z = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_0}}}$$

For the permeability coefficient P>0.4 (i.e. for breakwaters without a core):

$$\frac{R_u}{H_s} = d$$

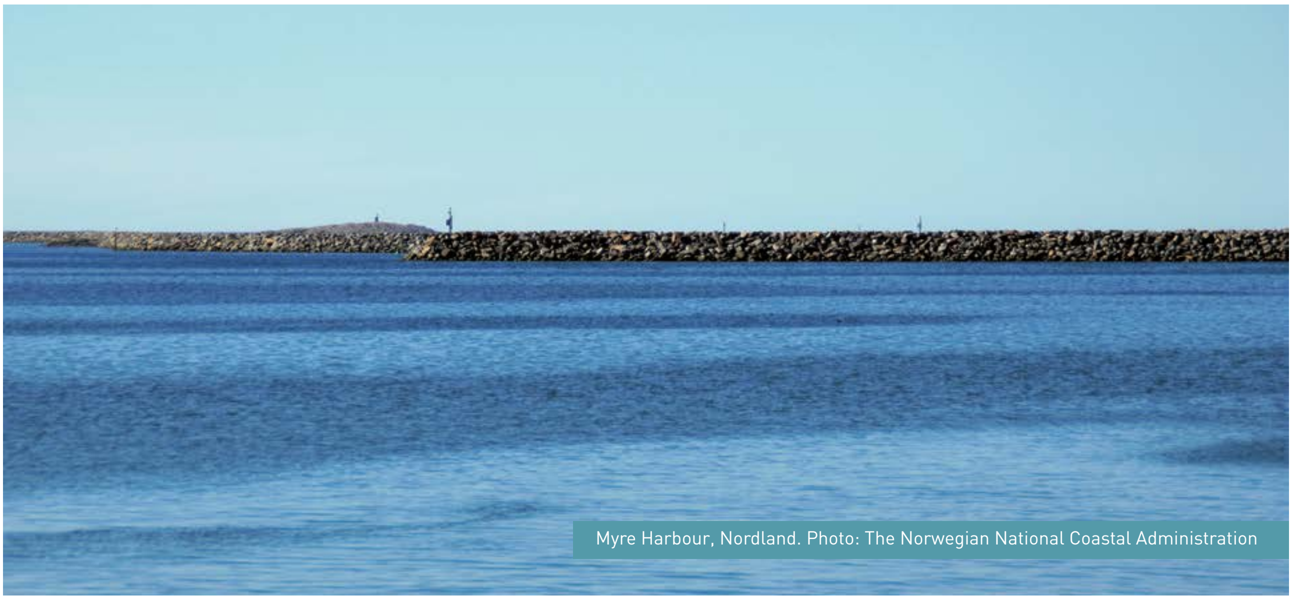
$R_u$ is the run-up height measured vertically from the design highest water level. $L_0$ is the wave length in deep water and $\alpha$ is the breakwater slope angle.

The coefficients a, b, c, d are found in table 7-11 and depend upon the probability level for run-up. The probability level here in % is the percentage portion of the waves that reach higher up than $R_u$ in the design situation given by $H_s$ and $T_p$.

The level of probability is chosen from the following criteria:

- 0.10 % is selected where a particularly high level of safety against overtopping is desired, for example where the breakwater is also a road connection where overtopping is not accepted due to structures or installations in the roadway, airports, or where residential buildings, institutions, etc., are placed directly on the breakwater;
- 2 % is selected where overtopping is accepted only in exceptional circumstances.
- 10 % gives a frequency for overtopping that is in accordance with Norwegian practice. This level can be selected when the breakwater is normally not trafficked by people or vehicles (for example where there is no road on the top), and where there are no structures on the breakwater or on the lee side. Permanent mooring of vessels along the breakwater's lee side cannot be accepted.

If a probability level of 10 % or higher is chosen, then the breakwater must be designed for overtopping, i.e. equal lee side and sea side.



Myre Harbour, Nordland. Photo: The Norwegian National Coastal Administration

**Overtopping calculation as per EuroTop Manual**
The EuroTop Manual (see ref. XX) contains formulae for calculating the overtopping rate given in litres per second per linear metre of breakwater. Together with the values in table 7-10, this method gives a more precise calculation of the risk of overtopping and what consequences it would have. The method is too comprehensive to be presented here, and reference is made to the publication that is openly available, and to a calculation program that can be downloaded.

We can calculate the overtopping rate for a number of different types of structures (for example an ordinary breakwater, berm breakwater, vertical walls and (gently sloping) embankments/dikes, and we can select different types of geometry. One set of formulae is given for deterministic analysis, and another set for probabilistic analysis.

### 7.5.6 Breakwater width

The breakwater width will most often be determined by construction engineering considerations. In the majority of cases, it is desirable to fill up as much as possible of the core material from barges (up to a depth of approximately 3–4 m), and then to fill the rest from dumper/tipper trucks from the land side. A prerequisite in such case is that the core material area which is being filled up be wide enough for vehicles or dumper/tipper trucks to be able to drive out and possibly turn around on the core, which then will be situated 1–2 m above the high tide, depending upon the local wave conditions. On top of this core, a filter layer and block layer must be laid out, and in most cases the core must also be raised a bit such that the top becomes more narrow (note: Besides the construction considerations, overtopping is also dependant on the width).

### 7.5.7 Slope angle, front

The ideal slope angle is approximately 1:1.3 ± 0.05. This angle is close to the natural critical friction angle for the material that is used in the core after the embankment has been subjected to moderate waves. A less steep slope angle can permit somewhat lower block weights, however if for example 1:1.5 is selected, it must be anticipated that some problems will arise with placing the outermost and lowest blocks.

A satisfactory interlocking surface slope of 1:1.3 provides very good stability, and would be quite suitable for ordinary quarried rock. The interlocking placement is achieved by placing one block on top of the one below with a retraction to create the slope. If the slope is mild, then the retraction becomes so great that solid interlocking becomes impossible.

Note that dumping from a dumper/tipper truck can create very steep slopes that are steeper than 1:1, and in some cases with an overhang. This is very dangerous, and during this type of work, the slope must be actively made less steep and the angle achieved must be monitored continuously.

### 7.5.8 Breakwater head

The breakwater head is a potentially weak point in a breakwater, where the breakwaters point straight out into

*Figure 7-11*
*Solid crown wall implementation. Upright blocks with back edge reinforcement of concrete. (Svolvær. Photo: Norwegian Coastal Administration).*

the sea. This is due to the individual blocks here sitting with somewhat weaker side restraints than the units on the straight part of the breakwater, and that these blocks will be subjected to a lateral force from the side when the waves are diffracted (see 6.2) around the breakwater head.

In some cases the breakwater head will be curved away from the waves, leaving it less exposed.

The planning personnel must in each case evaluate the need for reinforcement of the breakwater head. In those cases where the breakwater head is subjected to the same waves as the rest of the breakwater, it is recommended that the block sizes be increased locally at the head by 25 %, and in some cases even more.

It is an alternative to use a berm breakwater at the head. In such case a completely new design must be developed for the breakwater head.

### 7.5.9 Berm width

The berm width is part of the calculation of the breakwater height, and contributes to limiting the overtopping to an acceptable level. The berm width must be chosen to be large enough that a calculated reshaping ("recession") after a design storm is less than half the berm width:

$W_{min} \geq 2\ Rec(H_{s,dim})$

### 7.5.10 Berm height

For berm breakwaters, the height of the berm must be designed as follows:

$d_b \geq 0.6\ H_{s,dim}$

where $d_b$ is the berm's height above the design storm surge, and $H_{s,dim}$ is the design significant wave height (estimated at a 200-year recurrence interval).

### 7.5.11 Crown wall, roadway and navigation installations

If a roadway is to be built on the breakwater, then most often it will be relevant to have a crown wall. The crown wall must then be designed to be so solid (upright rock elements or reinforced concrete) that it actually comprises the top of the breakwater. In addition, the roadway must be placed on a draining underlay (corresponding to the filter), and the termination facing the lee side must be designed so that water that flows over the roadway is drained in free fall into the sea. The pictures in figure 7-11 and figure 7-12 give examples of successful implementations of a crown wall.


*Figure 7-12*
*Design of crown wall in very exposed breakwater (berm breakwater). The crown wall consists of bare blocks, nearest, and a concrete wall in the background. In front of the crown wall there are 3–4 blocks for protection. (Berlevåg. Photo: Norconsult).*

*Figure 7-13*
*Solid crown wall design of stacked natural stone. The blocks are secured against tumbling down by permeating vertical bolts. This crown wall is so tall that a person on the roadway does not have the possibility to assess what the situation on the sea side of the breakwater is. The breakwater must thus be so tall that the overtopping in practice is equal to zero. (Killingøy, Haugesund. Photo: Norconsult).*

*Figure 7-14  Picture from Kabelvåg during the storm Berit, which shows that the police have blocked off the breakwater. (Photo: Bjørnar Larsen, Lofotposten).*

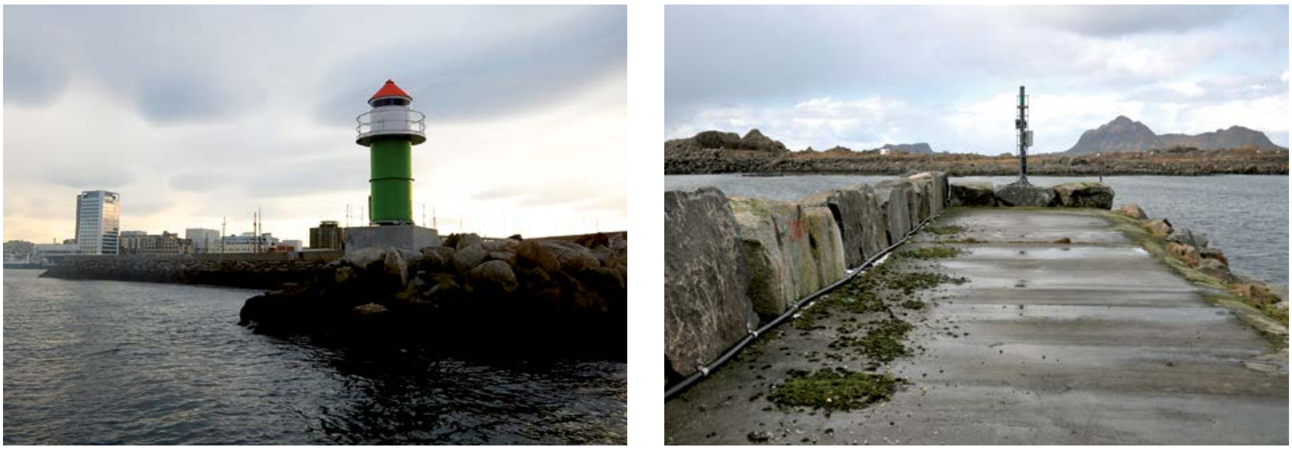


*Figure 7-15  Example of concrete foundation and cable attached to breastwork. (Bodø and Steinesjøen. Photo: Norwegian Coastal Administration).*

Where there is a significant height from the walkway/roadway to the top of the crown wall (in practice more than approximately ½ x the block size or approximately 1 m), the blocks that sit at the top must be secured against tumbling down on the walkway/roadway. The blocks that are sitting at the top can be affected by both wave forces as well as the wind.

A breakwater will always be an attractive location for recreational fishing and persons who are out to look at the weather, as in figure 7-10. In connection with the development plan work, risk and vulnerability analyses must be performed, cf. the Norwegian Planning and Building Act, section 4-3. An evaluation of societal safety in relation to unintended and intended actions must be a part of the risk and vulnerability assessment. In those cases where there is no requirement for a development plan, the RAV assessment will be performed in connection with the preliminary project work. In such an analysis, an assessment must also be made of whether the general public should have access to the breakwater, all the way up to the breakwater head or possibly just parts of the breakwater. Such an analysis may affect the design of the breakwater crest and may also say something about the need for an informational sign stating the hazards that could be encountered by entering the breakwater.

Placement of navigational installations on breakwaters must be performed in consultation with navigators at the Norwegian Coastal Administration. Foundations for the installations will be executed either as concrete (poured in situ or prefabricated), see figure 7-15 to the left, or as a large rock element that is placed such that it can comprise the foundation for the installation. The size of the foundation will depend on what type of installation will be established.

Routing of power can be done by laying an open cable clamped to rocks that are not expected to move, for example rocks in breastwork, see figure 7-15 to the right. This is the simplest method. It is also possible to lay a cable on the sea bottom out to the breakwater head and from there clamp it to the armour units up to the navigation installation. Another

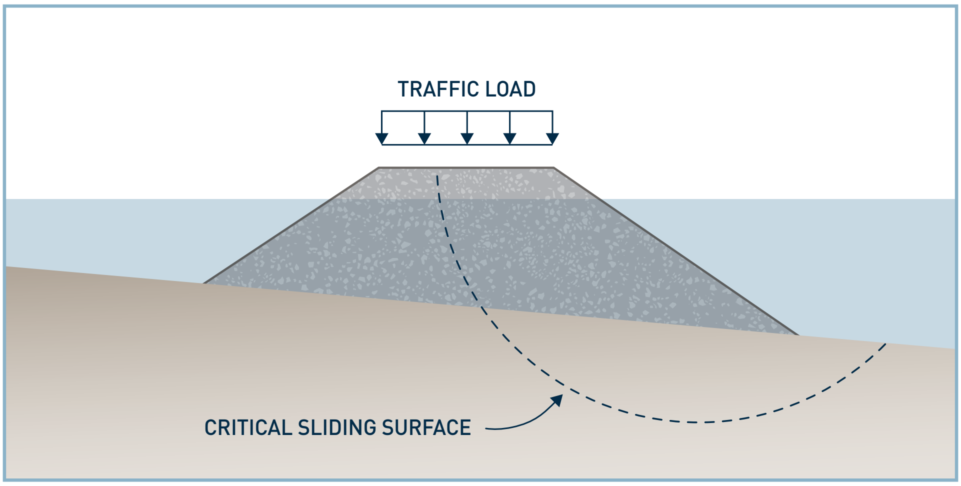



*Figure 7-16*
*Typical profile of breakwater with estimated critical sliding surface.*

possibility is to lay cable conduits in trenches and cisterns for later drawing/routing of cables, but this is less common. Depending upon the specific type of navigation installation that will be established, a solar cell system could also be relevant as an energy source. Project planning of the cable routing must occur in close co-operation with the Norwegian Coastal Administration.

## 7.6 GEOTECHNICAL STABILITY

### 7.6.1 General

Stability of the breakwater will depend much of the bathymetry and the composition of the loose material in the area where the breakwater will be established. It thus is important to have performed surveying of the bathymetry with soundings and seabed investigations before the project planning of the breakwater commences.

We primarily divide the stability calculation into 2 parts, total stress analysis (short-term) and effective stress analysis (long-term).

Total stress analysis encompasses fills on dense, impermeable soils, such as clay (cohesive material). With fills on such materials, the supplemental load from the breakwater will create an interior water pressure in the clay. The stability will thus be the lowest in the period immediately after the filling, because the strength of the clay will gradually increase as the water pressure is drained out of the soils (consolidation).

Effective stress analysis encompasses fills on open frictional soils, such as silt, sand and gravel (friction material).

Variations in tides must also be taken into account in the stability calculation. It is normal to use the lowest low water as the outer water level in stability calculations. The pore pressure in the sea bottom soils will vary with the tide, depending upon the composition of the loose material. Dense soils will be able to have a misaligned pore pressure with respect to the tidal variation, whereas open soils will have pore pressures corresponding to the outer water level.

The requirement for satisfactory stability is given in :
*NS-EN 1997-1:2004+A1:2013+NA:2016*
*Eurocode 7: Geotechnical design*
*Part 1: General rules*

The standard specifies the following requirements for safety factors:

- Total stress analysis: 1.4
- effective stress analysis: 1.25

Sufficient safety must be demonstrated both by total stress analysis and effective stress analysis.

Figure 7-16 shows a typical profile for a breakwater, with the estimated critical sliding surface. Possible loads (traffic load or the like) on the breakwater must be evaluated based on its planned use.

The stability of the breakwater itself is denoted as local stability. In areas where there are contiguous soft layers that stretch in over the land, the area stability must also be evaluated. This especially concerns loose material with brittle fracture characteristics or quicksand. In such cases a breakwater fill can trigger an initial slide that in turn causes increasing slide development. The slide may then spread in over the land, and have substantial consequences for buildings and infrastructure. If the seabed investigations show loose material with brittle fracture characteristics, then the guidelines in NVE's quicksand guidelines must be followed:

*Safety for quicksand slides – Evaluation of area stability by area planning and development in the areas with quicksand and other types of soil with brittle fracture characteristics 7/2014.*

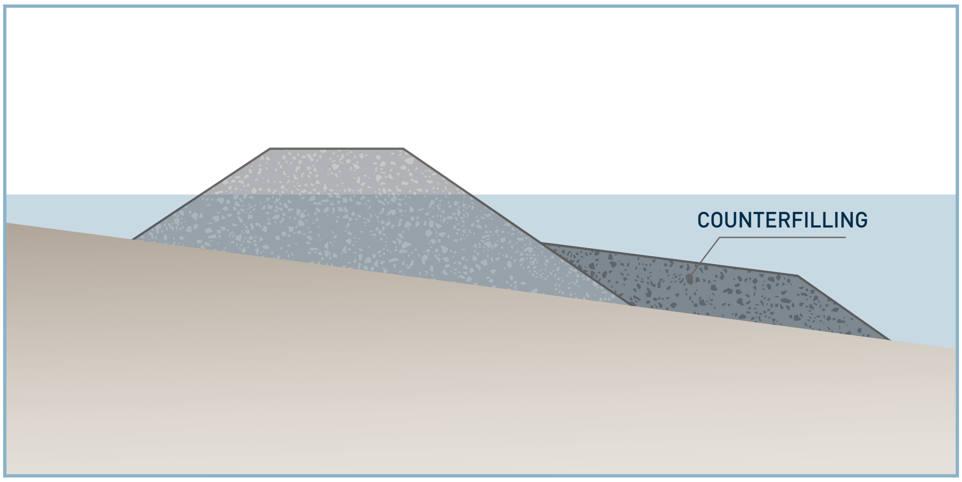



*Figure 7-17*
*Stabilising measures – counterfilling.*

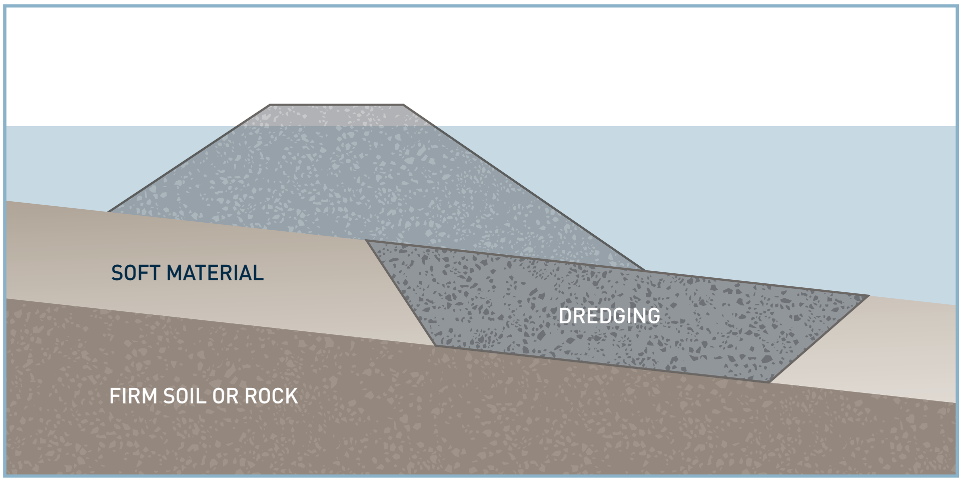



*Figure 7-18*
*Stabilising measures – dredging.*

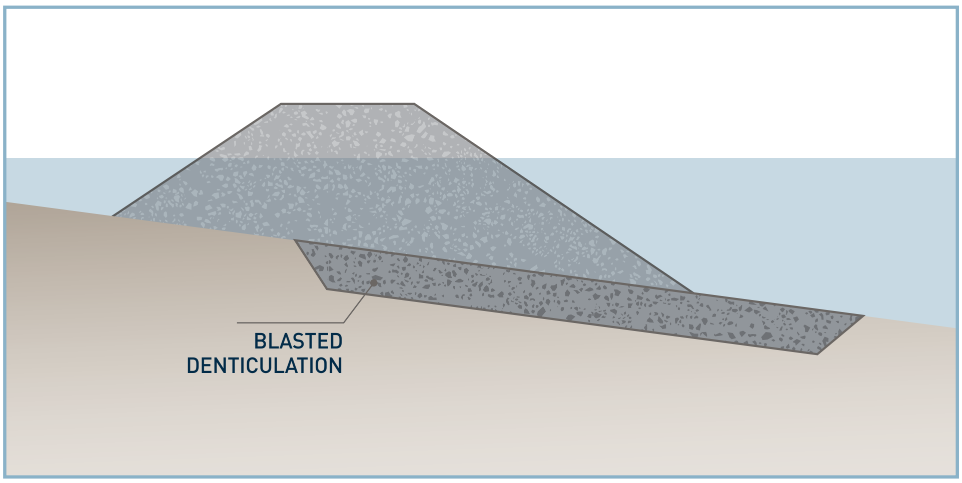



*Figure 7-19*
*Stabilising measures – foothold trenches.*

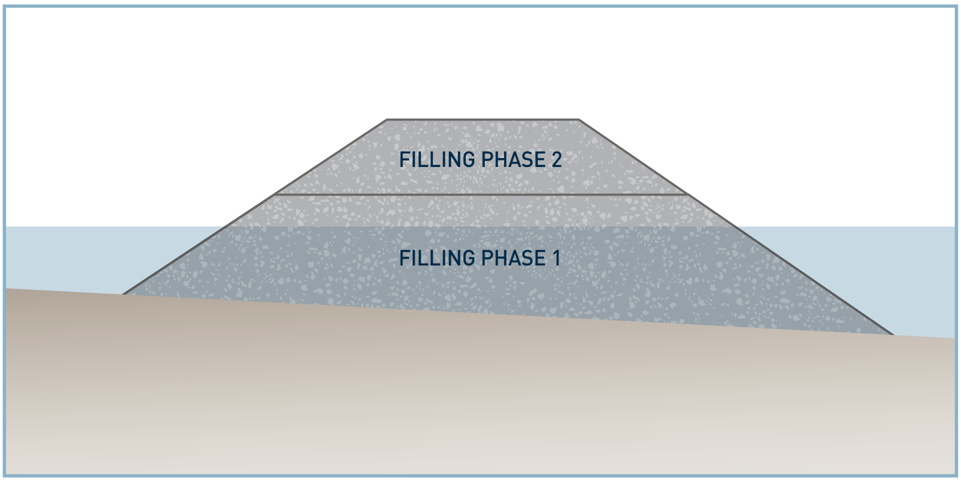



*Figure 7-20*
*Stabilising measures – filling procedure.*

### 7.6.2 Stabilising measures

In order to achieve satisfactory stability, it may be necessary in many cases to undertake stabilising measures. Such measures could be:

- Counterfilling
- Dredging
- Blasting of foothold trenches in rock
- Special requirements for the filling procedure

#### 7.6.2.1 Counterfilling

Counterfilling is laid out in order to extend critical sliding surfaces, which in turn increases the stability of the breakwater, see figure 7-17. The length and thickness of the counterfilling itself will depend on the bathymetry and the composition of the loose material in the area.

#### 7.6.2.2 Dredging

In some cases, the stability of the counterfilling can also be critical, for example when there is very soft material on the sea bottom. In such instances, dredging may be relevant. In other words, the soft material under parts or all of the breakwater is dredged away. The dredging goes down to firm soil or bedrock.

It is important that the refilling be performed shortly after the dredging, so that the dredging trench does not silt up again before the fill material is laid out. In addition, the lowest layer (a minimum of 1 m) should consist of open quarried rock material, in order to ensure good contact between the fill material and the underlying firm soil or bedrock.

#### 7.6.2.3 Creating footholds on smooth and sloping surfaces

If a breakwater is to be established directly on rock, then it may be relevant to create trenches for footholds under parts or all of the breakwater, in order to ensure satisfactory stability. Typical conditions for this are where parts of the breakwater are placed on smooth, wave-polished rock formations. An assessment of this must be made for every initiative.

The breakwater toe (the lowest units in the armour layer) must be secured against the units sliding if the rock is smooth and flat and lacking the natural roughness that locks the units in place. Sufficient roughness is ensured by establishing foothold trenches in the rock, for example in the form of a trench that locks the lowest units. *This also applies in those instances where the rock surface is horizontal.* Typical dimensions of such trenches are $W/D_{50}$ = 2.8–3.0 and $D/D_{50}$ = 1.4–1.7, where W is the width of the trench D is the depth of the trench.

The need for foothold trenches is difficult to ascertain in the project planning phase, and its implementation is thus often decided up at the site.

#### 7.6.2.4 Filling procedure

In order to ensure satisfactory stability, the filling procedure may have to be specified in detail. The filling can be conducted in several phases, where a pause is also taken between the phases in order to let the ground consolidate (increasing the strength) before the next filling phase is carried out.

If there is not satisfactory stability for filling the breakwater from land, it will be relevant to use maritime tools, such as barges or the like, during the first filling phase. Once this has been done, it can be followed by filling from land. Limitations can also be posed for filling in relation to variations in tides. During a tide cycle, the stability will be at its lowest just before the lowest low tide. Limitations can then be set such that all filling must occur on rising tides.

It is recommended that stability analyses be performed using geotechnical expertise.

### 7.6.3 Firm ground

On firm sand, gravel or moraine, the load-bearing capacity will as a rule always be sufficient.  Soundings should be performed regardless in order to confirm the presumption of a structurally sound layer. The possibility that a soft layer exists under a layer of firm soil must always be evaluated and possibly investigated.

### 7.6.4 Settlement

Filling that is established on loose material will involve subsidence,  For a breakwater, this subsidence can be divided up into 3 parts.

1. Subsidence of the original sea bottom
2. Self-subsidence of the breakwater body
3. Penetration of fill material into the sea bottom

Added loads on the original sea bottom, due to loads from the breakwater body, will cause subsidence of the sea bottom. The magnitude and duration of this subsidence will depend upon the layer thickness and composition of the loose material. Clay soils normally give larger subsidence, which may also occur over a longer time in comparison with sandy soils where the subsidence is expected to be less and faster.

Self-subsidence will also occur in the breakwater body itself. The size of this will primarily depend upon the layer thickness/height of the fill, and the rate of filling.

Penetration of material into the sea bottom also causes subsidence or deformation. This arises by the fill materials pressing down into the sea bottom, and the sea bottom soil filling voids in the fill materials. The size of these subsidences will then depend on the composition of both the fill materials and the sea bottom soils.

### 7.6.5 Erosion protection of the breakwater toe

Erosion and poor sediment stability is relevant in areas with strong currents and where breakwaters are placed on a sea bottom subject to erosion, for example such as sand or loose moraine. Erosion will also be able to arise where the breakwater toe is subjected to incident waves.

Erosion arises when a large rock element is placed on the bottom and water is forced to flow around it. The speed of the water will then increase and the sand will be removed where the speed increases around the rock, but no new sand come in because the speed has not increased a bit further away. This process will stop after a while, either due to larger particles being uncovered that are stable in the higher current or due to the current locally becoming weaker when the blocks and/or sand sink down; or both.

In order to avoid this, erosion protection must be used, which is a fill of quarried rock of moderate size (often the same size as the filter) which is laid out on the bottom in front of the breakwater. The purpose is for these smaller rock elements to create a smaller local speed increase and thus give less erosion.

Erosion can be very dangerous for a breakwater if the breakwater toe is attacked. An undermining of the lowest blocks will lead to the blocks above sinking down and such a break can propagate all the way up to the top. The following two measures are recommended to prevent erosion:

1. Securing of the breakwater toe with erosion-protecting material.
2. Use of base fill that ensures that a "reserve" of blocks exist that can sink down without the break propagating upwards.

The purpose of erosion protection consisting of smaller blocks is two-fold:
1. The smaller blocks create smaller changes (increases) in the current, and in such case the erosion will be smaller.
2. The erosion protection serves as an inventory of extra blocks that sink down into the sand, and the erosion process stops before this inventory is used up.

In a case where the breakwater's foundation sits on replaced soil, there will always be a transition between the original bottom (most often clay or mud) and the new material (either the toe of the breakwater or a horizontal section with replacement material on a level with the original bottom) which must also be protected.

Erosion protection should be designed using geotechnical expertise and the width and thickness of the erosion protection must be adapted to local conditions.

The erosion protection *must be designed for each individual instance,* but typically consists of quarried rock of size $D_{50}$ = 0.2–0.4 m that is laid out in front of the breakwater toe with a width of 3–4 m and thickness 0.5–1.0 m. Over time, the erosion protection will also be subjected to erosion, and then it will sink down into the sand like a blanket and be sitting at the same level as the bottom in general. Then there not be any speed increase, and the erosion will stop. There will continue to be some speed increase around the actual breakwater and the large blocks that are sitting there, but the top of the erosion protection is sitting here providing protection against erosion.

Erosion protection must be laid out with barges or boats, since it is difficult to obtain the desired distribution by tipping from the breakwater.

## 7.7 CAPPING OF CONTAMINATED SEA BOTTOM

### 7.7.1 In general concerning capping

If the sea bottom sediments are classified as contaminated in the area where the breakwater will be established, then remedial  measures must be carried out in order to prevent the spreading and transport of environmental toxins from the sediments to the surroundings. The most common measure in such cases is a sealing or capping of the sea bottom with a geotextile or other suitable materials before the breakwater material is laid out. The suitability for the capping materials must be documented in each instance in connection with the permit application to implement the breakwater initiative.

### 7.7.2 Project planning of capping layer for contaminated sea bottom

The capping of contaminated sediments is performed to prevent leakage and spreading of toxins to the ambient water body and intake of environmental toxins in organisms that live in and on the sea bottom. The capping layer should be designed using the results of the investigations performed as a basis, both with respect to location-specific evaluations for carrying out the capping and the characteristics of the capping materials.

Project planning of a capping layer is performed based on:

1. The thickness of the capping layer and the characteristics of the capping materials must be evaluated with an eye towards preventing leakage of environmental toxins as well as ensuring that burrowing organisms cannot come into contact with the contaminated sediments.

2. Evaluation of geotechnical stability in the capping area against the characteristics of the capping materials (the load-bearing capacity and consolidation properties of the sediment).

3. Method for laying it out that causes the least possible stirring up and ensures a contiguous capping with the planned layer thickness.

4. Control and monitoring in order to ensure that the materials are placed as planned and have the effect that is intended.

The principles for the planning and execution of capping of contaminated sediments are described in further detail in the Handling Guidelines M-350, see ref. X, and in the Guidelines for test program for capping materials M-411, see ref.  XII.


Silda Breakwater, Sogn og Fjordane

# 8. EXECUTION

### 8.1 PLAN FOR EXECUTION

An overall plan for the entire execution of the project must be prepared by the general contractor. The plan will be based on the contract documents with the requisite documentation of local conditions, the project planning foundation and possibly also other specific information of significance to the project. It is recommended that the plan be reviewed by the client in a start-up meeting and at the latest before the work is commenced. The client's approval should be given before start-up.

### 8.2 QUARRYING AND TRANSPORT

Most breakwaters are built with rock produced in a local quarry. The effectiveness, economics and quality of the final breakwater is highly dependent upon good and effective operation of the quarry. Optimum operating methods for attaining a good distribution of rock at the quarry is not discussed here, but information on geological conditions at the site must be obtained for the specific project, see chapter 5.3 concerning Geology, quarries.

The best results are achieved if one may use intermediate storage of rocks of different sizes and classes before they are transported to the breakwater. Several positive effects are attained with such intermediate storage:

- The rock quality becomes better if the blocks are handled multiple times before being placed, and potential cracks will arise before the rock is placed on the breakwater.
- Blocks that can be used for the armour layer will be set aside and saved for that purpose.
- For breakwaters with multiple sizes of armour units, the blocks can be placed in different piles and used as needed.
- Blocks that are not suitable for armour units can be set aside and possibly utilised in the core material.
- Usually only a smaller portion of the total blasted volume can be used for armour units, usually 5–15 %. This means that much core material must be produced for each block that is produced, and since the core material and filter must be laid out first, it is smart to set good armour units aside in an intermediate storage area.

Where material is to be produced in a designated quarry (as opposed to rock purchased form an external supplier) it is important that the following be included in a plan for running the quarry:

- Blasting plan with possible test blasting
- Area for sorting of materials (armour unit – filter – core material)
- Area for intermediate storage
- Challenges: Large blocks, different block sizes, filter layer, handling of plastic in the quarried rock materials

*Figure 8-1 Intermediate storage of rock requires large areas, but is a good investment because it makes it easier to attain the desired quality for the finished product. (Sirevåg. Photo: Sintef).*

### 8.3 BASE FILL

Many breakwaters are established in areas with such large depths that base fill is required. In most cases full base fill is used, but ring base fill and false base fill may also be relevant, See figure 8-2. The base fill normally consists of unsorted quarried rock materials (usually the same as the core material) that is deposited from a split barge or equivalent. There may be geotechnical reasons for requiring the deposition of the base fill to be performed in special sequence, layer by layer or over a certain time. Furthermore, the project planning must have evaluated and determined how high the base fill should be.

In order to properly manage each dumping, a positioning plan must be prepared to control where every deposition (dumping) occurs. Surveying of the seabed must also be performed periodically for verification purposes as the dumping progresses. Accurate positioning is important because financial compensation for dumping outside the intended profile will as a rule not be given.

Based on the preconditions that are found in the project planning basis, a plan must be prepared for how the base fill should be carried out. Here, it is important that the following are included:

- Positioning for base fill with split hopper barge. Normal depth for dumping. Plan for dumping.
- Satellite positioning (or equivalent) must be used, also for registering each barge load.

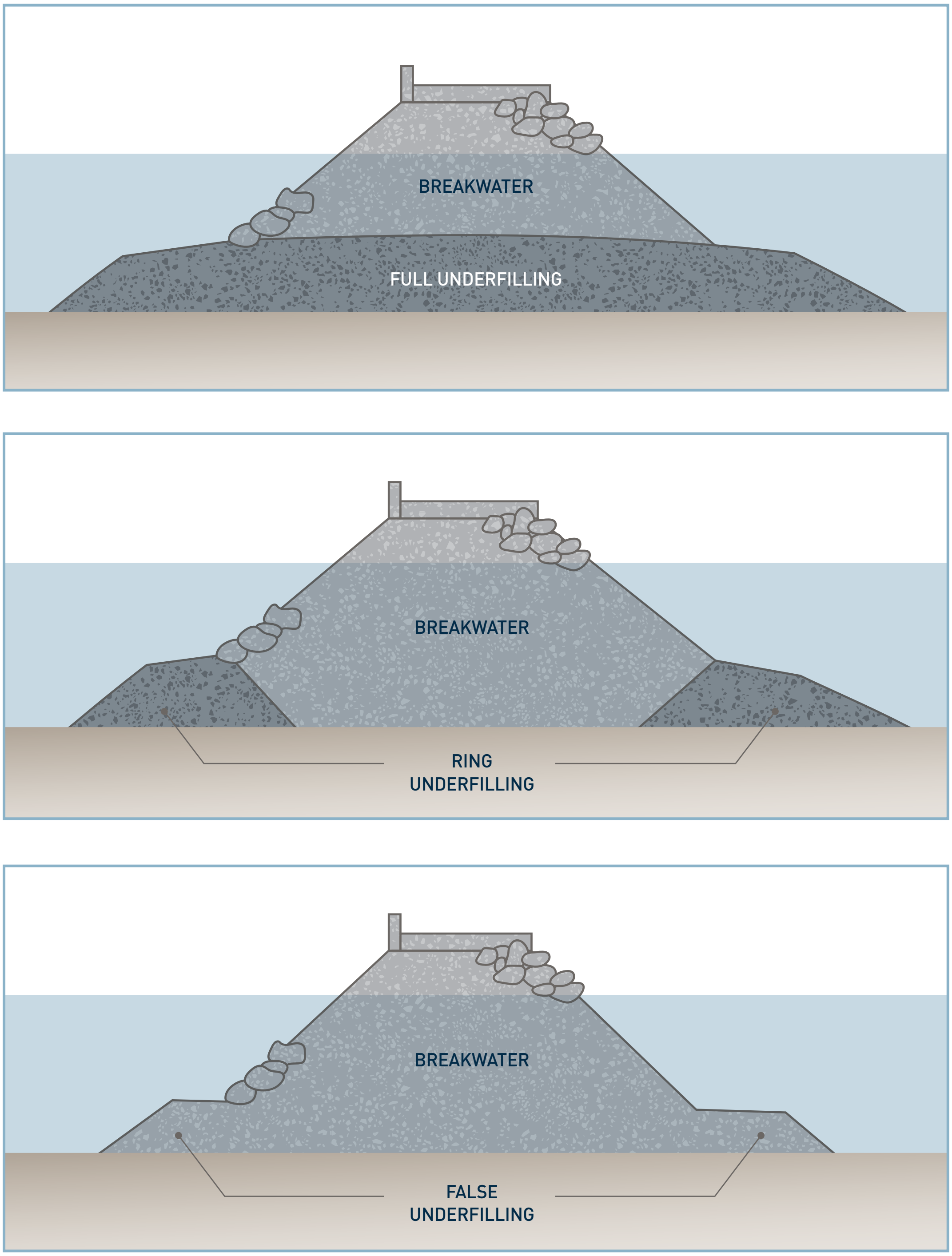



*Fig. 8-2 Different types of base fill.*

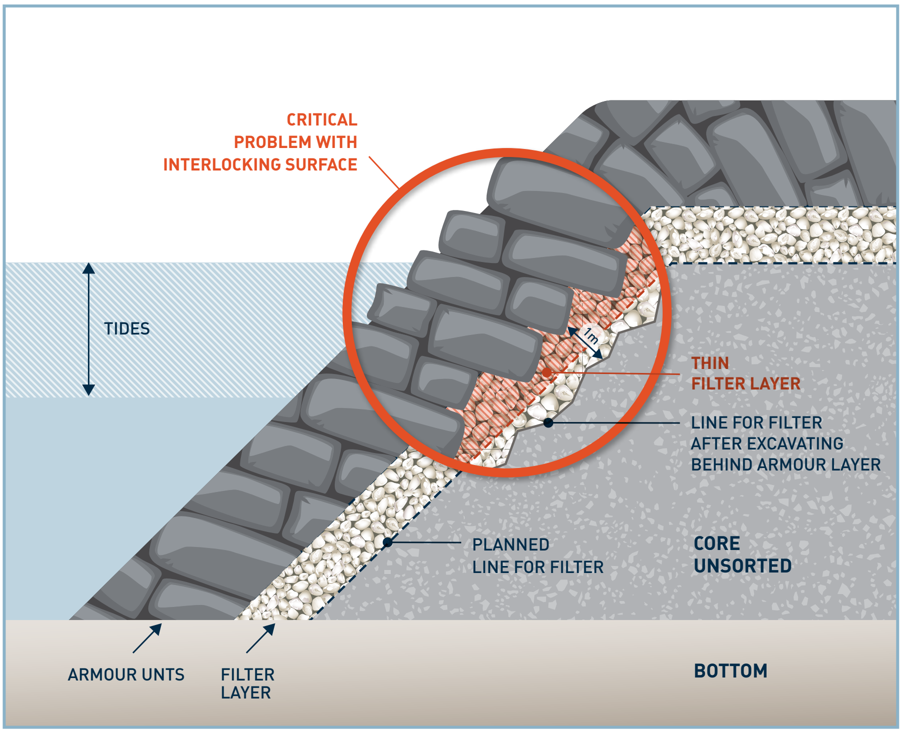



*Figure 8-3*
*The sketch shows the line form the filter from the project planning work and the new, adjusted line after excavation for the filter behind the block layer.*

## 8.4 CORE MATERIAL

Reference is made here to the discussion of core material in chapter 7.5.4.

Because the core material must be dumped, it is important that the final slope of the core material is not milder than the intended slope. This must be checked continuously during filling because the final breakwater's profile is formed during this process.

Compaction of the core material is only relevant where cable trenches and foundations for navigational installations are established. Sufficient compaction of other materials is attained when heavy construction vehicles pass over the unfinished breakwater.

## 8.5 FILTER LAYER

The filter layer is a very important component of every breakwater. See item 7.5.3 concerning the filter layer' function and purpose as well as on the material selection and design. Breakwaters that are built without or with an inadequate filter lead to erosion of the base for the armour units and eventually to a collapse of the breakwater Hence it is important to follow the guidelines given in item 7.5.3.
In general, the filter layer must be continuous, so that the armour block layer does not come into contact with the core material It is important to have high precision in the laying of the filter layer so that the its thickness is minimum what is specified. It may occur that some blocks are so large that they are longer than the thickness of the block layer. In such cases, it is permissible to turn the blocks so that the long axis of the block rests parallel to the axis of the breakwater. (It is nevertheless not permitted to place the block "flat on the breakwater side", i.e. with the shortest axis perpendicular to the slanted breakwater surface. See chapter 8.7 for details.)

Should it be required to place an armour unit with a greater depth than the prescribed armour layer thickness, then space for the filter layer must be made in the core material, see figure 8-3.

It is also important to fill the filter layer to the prescribed depth, in order to ensure stability under wave action. Sorting is done by running the material over two gratings,

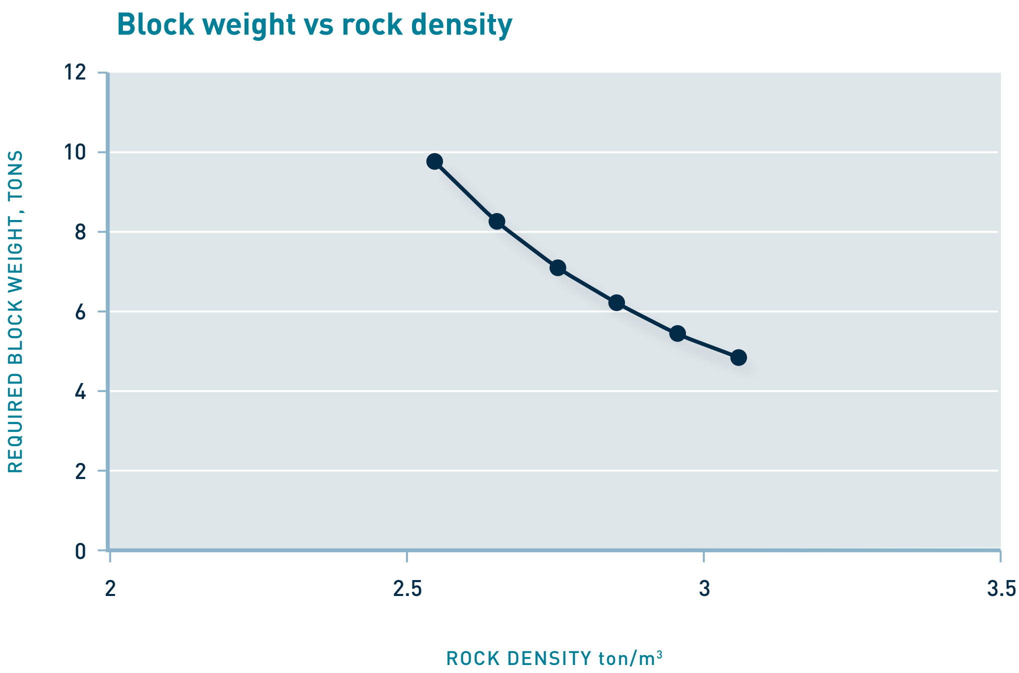



*Figure 8-4*
*Block weight as a function of trock density.*

for example first over a grating with a lattice of 30 cm (0–300) and subsequently a grating with a lattice of 20 cm (0–200). That would then sort the 200–300 fraction out.

## 8.6 ARMOUR UNITS

### 8.6.1 General

Armour units in breakwaters must generally be produced from quarried rock of granite, gabbro, gneiss or similar types of rock with good mechanical rupture strength. Rock elements from moraine, rivers or rock falls may only be used in exceptional cases.

### 8.6.2 Specific gravity

The density of the rock that is used in the armour layer is the most important parameter to monitor besides the weight and shape of the block. A block that has a high density may have smaller exterior dimensions but still attain the desired weight. Because the block is now smaller, it is also subjected to less drag and inertia forces from the water, and thus the size can be reduced further. Figure 8-4 shows the interrelationship between the requisite block weight ($W_{50}$) and the rock density for a case with a significant wave height $H_s$ = 2.5 m. We see that if the specific gravity in the design basis was assumed to be for example 2.75 kN/m$^3$, it then gives $W_{50}$ = 7.2 tons. If the specific gravity that is measured of the actual materials is only 2.65 kN/m$^3$, then it gives a requisite block size $W_{50}$ = 8.3 tons, i.e. over a 1 ton increase. In other words, in this case it is shown that a 4 % reduction of the actual specific gravity involves an increase in the requisite rock size of  15 %.

*The specific gravity must thus be checked regularly in the quarry, and if the stipulated specific gravity is not attained at the site, then changes must be made in the design.*

### 8.6.3 Shape

A good shape for the blocks that are used in the armour layer is crucial to the quality of the results obtained.
A good armour unit has the following characteristics:

- edged, with clear corners and at least one side that is nearly flat.
- moderate flakiness, i.e. a ratio of approximately 2 to 3 between the longest and shortest sides in an enveloping right prism.
- no cracks or failure planes.

*Figure 8-5*
*If there is no equipment for weighing the blocks, the blocks can be measured and the weight subsequently calculated. Shown here by blocks that are well-suited for interlocking surfaces. (Rock for Bykaia, Longyearbyen. Photo: Norconsult).*

*Figure 8-6*
*Examples of blocks that are poorly suited for interlocking surfaces – the blocks are round without a clear flat side, and they are too short in relation to their thickness. (Værnes).*

*Figure 8-7*
*Example of blocks that are well-suited for interlocking surfaces (and possibly walling) – the blocks have an angular shape and at least one flat side, which enables them to work together well with other blocks. (Molde. Photo: Norconsult).*

*Figure 8-8*
*Cracking of blocks is normally not any large problem in Norway, as long as the blocks are handled such that any possible cracks emerge before they are placed. In other countries, however, the durability of the blocks (cracking, peeling) is a substantial problem. (Gabes, Tunisia. Photo: Norconsult).*

Units that are not suitable more or less fall into three groups:

- "Spheres" or "balls", i.e. round units that have no flat side and which will stack poorly.
- "Slabs", i.e. blocks with high flakiness and which will be sitting as flat sheets in the breakwater. (Blocks of this type were previously used on less exposed breakwaters by placing them flat in tile fashion on the sides of the breakwater. This practice gives weak and poorly functioning breakwaters, and hence is not recommended.)
- Blocks of rock types with poor strength, for example mica, shale or porous types of rock, often with clear fracturing. This gives water saturation and thus frost erosion and splitting of the blocks during the winter.

The executing contractor must have a system for sorting out poor and unsuitable blocks. Due to efficient operation considerations, this sorting ought to happen as early as during loading at the quarry. It is also important that all persons involved in the transport from the quarry to the completed placement of the rock have the authority to reject poor blocks.

### 8.6.4 Quality

The quality of Norwegian rock varies from one part of the country to another. In general, rock types such as gneiss, gabbro, granite and other rock types with a stable dead load, high mechanical rupture strength and acceptable shape are preferred (see item above). Types of slate, sandstone and various types of conglomerates are generally less suitable. In many cases the uppermost layer near the surface is cracked and disintegrating, whereas acceptable quality with less crack formation is found a few metres beneath the surface.

The block layer is identified by a specified block size specified by weight as follows:

- **$W_{50}$:** The median block weight, i.e. the size for which half of the total number of blocks is less than the specified weight and the other half is greater. This size will be close to the average block weight, but is not the same.
- **$W_{average}$:** Average block weight, i.e. the total of the weights of all blocks divided by the number of blocks. The average value is normally not used, but if the number of blocks is small it can be difficult to calculate $W_{50}$. In such cases, $W_{average}$ should be 10–15 % larger than $W_{50}$.
- **$W_{min}$**: minimum permissible block size in the block layer. $W_{min}$ is usually approximately 0.7 x $W_{50}$. No blocks smaller than $W_{min}$ are permitted.
- **$W_{max}$:** In practice, there is no upper limit to the block weight. Requirements are, however, posed that there should be blocks up to the size $W_{max}$. In addition, no blocks will be allowed to be counted of a weight more that $W_{max}$ when $W_{average}$ is being calculated.

For ordinary breakwater purposes (armour unit in conventional breakwater and blocks in the berm layer on berm breakwaters) the following criteria are used:
1. $(W_{min}, W_{50}, W_{max}) = (0.7W_{50}, W_{50}, 1.5W_{50})$
2. A minimum of 5 % of the blocks must have a weight $W \geq 1.4W_{50}$

The method for checking the sizes of the blocks may vary, and the contractor can select different systems. One much-used method is to weigh some blocks of different sizes and then mark these with their weights and place them at a readily visible location. Then the machinery operators can use these blocks as a reference when they estimate and log the weight of each block.

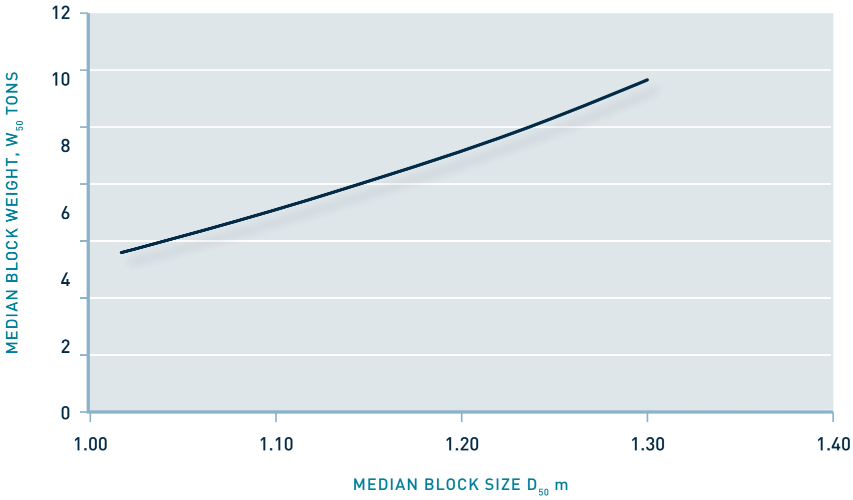



*Figure 8-9*
*Interrelationship between block size and block weight. Note that small changes in $D_{50}$ make for large changes in $W_{50}$*


*Figure 8-10 For larger structures with strong wave loads, it is relevant to weigh and number every single block and store its position electronically (weight 32.4 tons). (Laukvik. Photo: Norconsult).*

In order to make it easier to find blocks of the correct size, the median block diameter $D_{50}$ is sometimes also specified. The idea is then that a block can be modelled as a cube, and the weight is given by $W_{50} = \rho \times D^3$, where ρ is the specific gravity. Note that small changes in $D_{50}$ cause large changes in $W_{50}$. Figure 8-9 shows the interrelationship between $D_{50}$ and $W_{50}$. Blocks of for example 4.5 tons have $D_{50}$ = 1.2 m. A small reduction of $D_{50}$ to $D_{50}$ = 1.15 m gives a reduced block weight of $W_{50}$ = 4.0 tons, and if $D_{50}$ = 1.1 m then we get blocks of only 3.5 tons.

If nothing to the contrary appears in the contract and project descriptions, then the contractor must have scales available at the site. The scales must be calibrated before weighing is started up as specified above.

### 8.6.5 Control of block weights

All agreements concerning deliveries of block stone must be based on a specified weight and an estimated specific gravity. Quantities such as (equivalent) diameter or cross-section may be specified for orientational purposes, but contract fulfilment for block stone must be based on specified weights given by $W_{min}$, $W_{50}$ and $W_{max}$ (see item 8.6.4 above). If the contractor has had a quarry designated, then he cannot be held liable for the specific gravity, but the contractor must report the attained specific gravity, and the client must possibly adjust the design accordingly.

Block weights can be checked and documented in a number of manners. What all the methods have in common is that the documentation must be continuous, and that the client must at all times be allowed to conduct checks (for example by taking random samples) and be able to inspect documentation of the production up to for example the day before the check.

The weighing equipment can be one of the following:

A. Vehicle scales, where the entire vehicle is weighed first without a load and subsequently loaded with the relevant block.
B. Weight on wheel loader. Newer wheel loaders have scales in the bucket, where the weight in the bucket is read in the operator's cab. Note that these are most often based on readings of the pressure in a lifting cylinder, and hence actually measure the moment of the load, which is presumed to be sand or gravel. A block will sit differently in the bucket and thus result in a different moment. It thus is important to calibrate such a weight reading.
C. Crane weight. With this method each block must be strapped and lifted by a crane that has a scale on its lifting cable. With properly calibrated equipment, the method is precise, however it is extremely labour-intensive.

When weighing equipment is not available, it may be acceptable for smaller projects to measure one block and calculate its volume, and consequently its weight. In such cases, the block must be modelled as a prismatic object, and measure the height, width and length in a circumscribed prism. The volume can then be calculated as:

$V1 = (h \times w \times l)\ ) \times C_B$;

where $C_B$ is a block coefficient that is often in the range of 0.5–0.6 (for an ellipsoid $C_B$ = 0.52).

To document the production, each individual block may be weighed and placed in an intermediate storage area. An alternative is to select a sample of the blocks and weigh/calculate them, and then, as mentioned earlier, place them in a readily visible location with their weights written on them and then each individual machine operator must assess which of the classes the block belongs in. The latter method must be supplemented with frequent random checks performed by the contractor.

A recommended method for operating a quarry with intermediate storage is to sort the blocks into weight classes (such as 4 -6 tons, 6 – 8 tons, etc.) Adaptations can then be made to varying project planning $W_{50}$ in the breakwater by combining together a different number of blocks from each weight class to obtain a specified W50.

## 8.7 INTERLOCKING BREAKWATER SURFACES

### 8.7.1 Requirements for execution

Due to considerations involving stability, redundancy and aesthetics breakwaters must have interlocking surfaces, i.e. the blocks must be laid such that every rock element is supported by the rock elements above and on its sides.

The technique for laying out rocks on the outer side of the breakwater (and hydroelectric power dams) is called interlocking. It consists of laying the armour units so that each unit is firmly held and locked by its neighbouring units. This allows the individual rock to withstand a greater load than if it had been sitting all alone.

*Figure 8-11*
*Example of a satisfactorily executed interlocking surface : The blocks are sitting interlocked with an acceptable slope, and are pointing weakly upwards. (Mehamn Airport. Photo: Norconsult).*


*Figure 8-12*
*Example of breakwater/embankment with "structured rubble", see table 3-1. (Photo: Norconsult)*
*The structure does not qualify as an interlocking surface because there are too many small blocks, the blocks are sitting with an incorrect slope and many blocks are sitting without support. MUST NOT BE USED IN BREAKWATER CONSTRUCTION.*

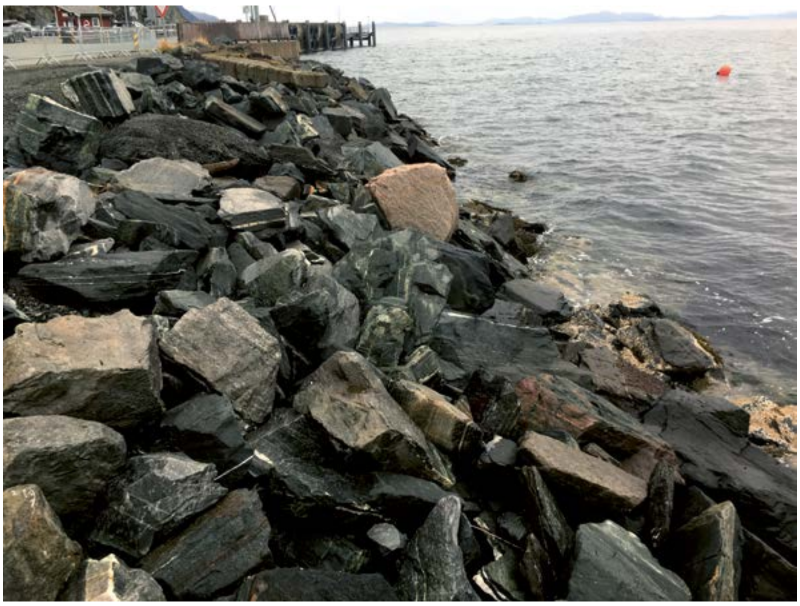


*Figure 8-13*
*Example of construction method "Rubble embankment" from table 3-1. (Photo: Norconsult)*
*MUST NOT BE USED IN BREAKWATER CONSTRUCTION.*

*Figure 8-14 Example of well-executed interlocking surface in protected areas: the blocks are sitting with a good angle and are interlocked well with each other. This is an acceptable method for protected areas, where the waves are moderate. For more exposed areas, a bit more rough surface with greater unevenness would be preferable. The crown wall at the top consists of free-lying blocks that are not secured, something that is acceptable here. (Vikan, Kristiansund. Photo: Norconsult).*

*Figure 8-15 If the rock quality and the shape dictate so, then a staircase front could well be made instead of an level surface. Note that the blocks in this case are extremely long, such that a block continues to provide support for the block above. (Kabelvåg. Photo: Norconsult).*

Interlocking surfaces are built with an excavator/backhoe, possibly equipped with a rock claw. The building of interlocking surfaces should be regarded as a craft that requires great precision, attentiveness and experience of the machine operator.

Important prerequisites for an interlocking surface to be successful and durable are the following:

1. The building of the interlocking surface must begin from the bottom and proceed upwards. Every single rock must be individually placed and must sit at the top of the underlying rock with an insertion that causes the optimum slope to be attained of 1:1.3–1:1.4 (or the slope that is described in the work drawings).
2. Each rock must sit supported with a minimum of support at three points.

A second and equally important prerequisite is that the machine operator or possibly an assistant goes down on the breakwater as low as possible and views the breakwater from the bottom going up. Any possible weaknesses, holes or omissions in the interlocking surface will then easily be able to be observed. Such an inspection ought to be performed several times during the course of a shift, so that it will continue to be possible to correct an error without having to remove too much of the work already performed.

Interlocking surfaces with rock elements should be built from approximately 1–2 m below Chart Datum and up to the top. Going deeper than this is not practical for implementing the interlocking surface, and nor is it necessary. Here, the blocks are simply discharged or dumped (uncontrolled laying out). It must be checked, regardless, that the described thicknesses and quantities of rock elements are achieved below the practical working depth.

The figures 8-11, 8-12, 8-13, 8-14 and 8-15 show examples of breakwater construction with "Interlocking surfaces" and "Structured rubble" and "Rubble embankment", respectively.

### 8.7.2 Foothold trenches

The need for foothold trenches can in certain cases be difficult to ascertain during the project planning phase, and its execution is thus often decided upon at the site during an inspection. Here, the instructions in item 7.6.2.3 must be followed.

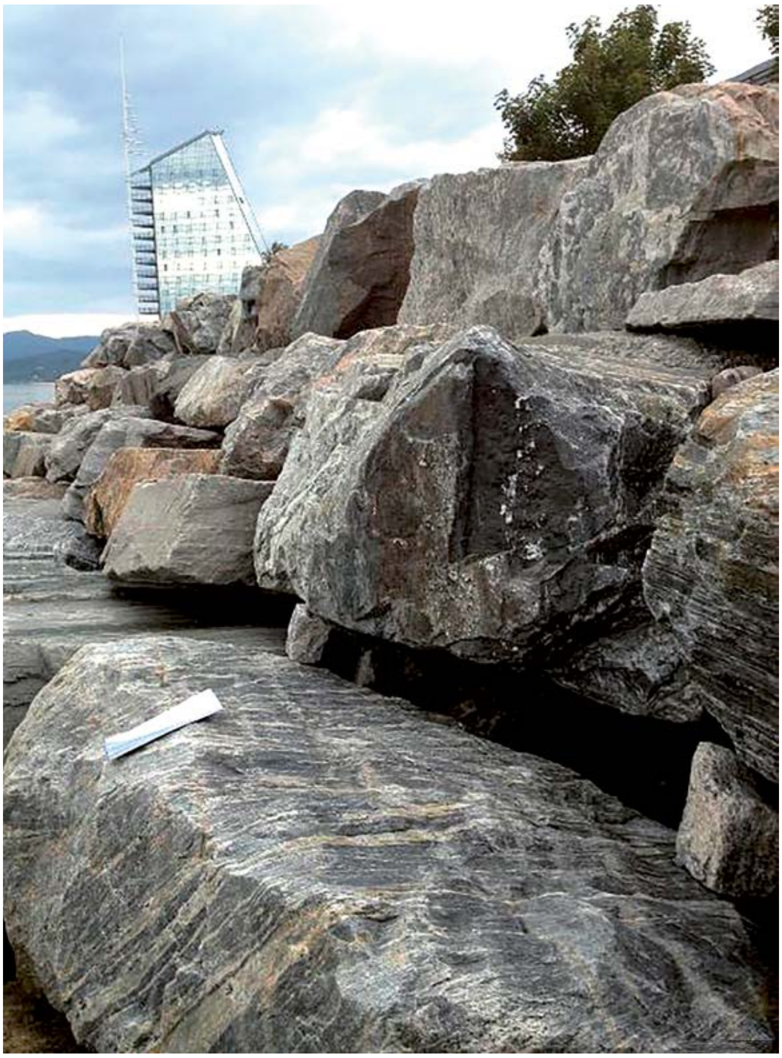


*Figure 8-16*
*Example of interlocking surface that ought to be corrected before completion: Small stones have come between the blocks, something that leads to larger openings and cracks than desired, and to poor contact between the blocks. This can be avoided if the machine operator or an assistant undertake frequent checks of the work by going down on the breakwater and viewing it going up towards the top. This error cannot be detected from a position in the excavator. (Molde. Photo: Norconsult).*

*Figure 8-17*
*Example of interlocking surface that must be corrected: free-laying blocks, large cavities, blocks that are pointing downwards and blocks with a poor shape. (Værnes. Photo: Norconsult).*

*Figure 8-18*
*Example of failure in the underlayer, the breakwater/embankment is placed on filled materials, and subsidence and sinking into the underlayer cause the uppermost blocks to be hanging without good support. At the same time, the interlocking disappears that the overlying blocks should be providing for the blocks below. (Molde. Photo: Norconsult).*

*Figure 8-19*
*Example of interlocking surface that must be corrected: The blocks are partially sitting loose on the surface without support, and the blocks are sitting with an outward slope (pointing downwards). (Mehamn. Photo: Norconsult).*

*Figure 8-20 Examples of interlocking surface that has a low resistance capacity against high waves (but can be OK at less exposed locations): the blocks are sitting with an erroneous orientation, i.e. with the flat side in towards the breakwater. In this case, neither is the interlocking surface executed from the bottom up, and most of the blocks are sitting free without support. (Værøy. Photo: Norconsult).*

Berlevåg Breakwater. Photo: Tore Larsen

**8.8 EROSION PROTECTION OF THE BREAKWATER TOE**
This is described in item 7.6.2.
It is important that the execution occur in accordance with the stated description and/or in accordance with the existing contract and project description.

A long time should not elapse (maximum 3 – 4 months, and before the winter season) between the construction of the breakwater and the laying out of the erosion protection. Preferably, the erosion protection should be laid out before the breakwater construction commences. In cases where the breakwater is sitting on a sandy bottom or another easily erodible material, the breakwater toe must be protected against erosion, i.e. against flowing water removing the sand that the breakwater is sitting on.

**8.9 BREAKWATER DECKS**
A breakwater must have a top or crown that is an integrated part of the breakwater from the commencement of the project planning. If the crown wall is solid, then the top of the crown wall can be reckoned to constitute the height of the breakwater. A breakwater's height determines whether it is tall enough to avoid overtopping, so that the lee side is able to have a reduced block size, or whether the breakwater is designed for overtopping, so that regard must be paid to the water coming over the top of the breakwater.

For breakwaters designed for overtopping, an alternative is to cover the entire top and lee side with blocks of the same size as on the sea side. This may be acceptable in some locations, however the desire is most often to keep the breakwater open for different forms of traffic or public access.

Figure 8-21 shows an example from Steinesjøen, Vesterålen, where a solid crown wall has been formed of upright armour units. At other locations (for example Moskenesvågen in Lofoten) a solution has been found where a longitudinal concrete beam has also been established behind the crown wall in order to keep the blocks in it in place and to distribute the load from the blocks across a larger area. Behind the crown wall, a top deck of concrete is often established, such as figure 8-21 shows. Where such a solution is used with a concrete deck, and possibly with the addition of a beam, it is important that these sit on a substrate of drained material. A good solution is to implement this so that the filter layer is run continuously from the sea side of the breakwater and over the top of the core material.

Figure 8-22 shows an example from Stamsund, where a breakwater was designed without a top deck, but where the decision was subsequently made to build a concrete deck. In this instance, the breakwater itself has endured well, but the concrete deck, which lacks protection and anchoring in the breakwater, is misaligned and ruined.

Another damage scenario is shown in figure 8-23 from Værøy, where there is a breakwater that originally was subjected to wave forces and overtopping that were excessive. The deck, also in this case, was built after construction of the breakwater had actually been completed Here, a relatively large front beam has been made, which has enabled the deck to remain somewhat intact, however the deck is partly overtopped and partly laid bare, with it hanging in free air in certain places.

*Figure 8-21*
*Example from Steinesjøen, Vesterålen, where a solid crown wall was built of upright armour units. (Photo: Norwegian Coastal Administration).*

*Figure 8-22*
*Example of lack of integration between deck and breakwater, where the crown wall is also absent. The deck is poured as a free-laying slab without the protection of a crown wall, and the deck is sitting on non-permeable material. During strong wave action with overtopping, the sea presses on the front edge, and the concrete slabs are lifted by the inner water pressure due to poor drainage. Note that the breakwater is otherwise undamaged. (Stamsund. Photo: Norconsult).*

*Figure 8-23*
*Damage under the deck and on the back side after overtopping and penetration by high waves. The convex shape of the deck and a solid concrete front contributes to preserving the shape of the deck and prevents its total failure. (Tyvnes Breakwater, Værøy. Photo: Norwegian Coastal Administration).*

Floating breakwater, Harstad. Photo: Marina Solutions

**8.10 OTHER WORK**

### 8.10.1 Measures for contaminated sediments

If the area that the breakwater will be sitting on is contaminated, the way to handle this must be clarified during the project planning. Primarily, there are two solutions that are relevant. Either the contaminated materials are removed before the breakwater is constructed or the sea bottom that the breakwater sits on can be covered and sealed by capping.

**Controls:**
When covering it over, it is important to ensure that this is performed in accordance with the tender documentation and work drawings. Keywords in this context are to ensure that the covering materials and any possible fibre fabric are in accordance with the description. Likewise that the layer thicknesses are in line with the drawing and description.

If the contaminated materials are dredged before the breakwater is constructed, it is important to ensure that the dredging occurs with respect to the description and with the permission of the pollution authority. Furthermore, it is important that procedures be drafted for any possible monitoring of the turbidity, material handling and deposition.

### 8.10.2 Securing transitions to land

Where the breakwater has been placed on soil subject to erosion, it may be necessary to use extra erosion protection in the surf zone. The project planning must have clarified this and described how it will be implemented.

If the breakwater traverses bare and smooth rock, there may be a need for undertaking protective measures so that the armour units do not slide. The most common method for locking the lowest armour units is to create a trench for a secure foothold. The project planning must have evaluated this and described its execution. There may be a need for local adaptation that is not captured in the project planning. See item 7.6.2.3 and 8.7.2.

**Controls:**
Ensure that the erosion protection is executed in accordance with the tender documentation and work drawings. For work with foothold trenches, it is important that competent personnel be present so that local adaptations can be made.

### 8.10.3 Temporary marking

During construction, an unfinished breakwater may represent a hazard to seafarers. In periods during its construction, the breakwater may constitute an "artificial subsea reef" and as such pose a substantial danger to

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maritime traffic. It thus is important that the breakwater be marked. Experience shows that it is not sufficient to mark the end of the breakwater with a standard floating marker, but that the entire length of the breakwater must also be marked with buoys with a light. Virtual marking (showing the breakwater on the sea map before it is finished) may also be assessed as a supplement. In addition, it is important to provide information on this to fishermen's associations, fish processing facilities and others who traffic the harbour. It can be beneficial to ask the fish processing facilities to disseminate this information to the fishermen who deliver their catches there. Disseminating information in newspapers and social media has been also shown to be useful.

**Controls:**
Make sure that the marking is established early enough, and that it is moved and supplemented as needed. Check that information is being disseminated via the relevant media and channels. Make sure that this topic is addressed in any possible information plan.

### 8.10.4 Crown wall

Any possible crown wall should be designed during the project planning phase. For the majority of breakwaters, there will be small displacements in the topmost armour units due to subsidence and the effects of waves. The crown wall must be designed so that it does not rest on or use the armour units as a foundation, since these blocks must be allowed to settle a bit without them pulling the crown wall along with them. A crown wall is traditionally built as upright armour units or implemented as a wall of concrete. If armour units are used in a crown wall, they must have a foundation of and be secured with surrounding material, recasting, a concrete beam/concrete footing on the lee side or the like.

**Controls:**
It must in particular be checked that expansion joints are sealed with a suitable material, which allows expansion and contractions. It must in particular be checked that expansion joints are sealed with a suitable means.

### 8.10.5 Cable trenches and foundations for navigational installations

There must be satisfactory compaction upon refilling of cable trenches and before the mounting of foundations for navigational installations.

**Controls:**
Be sure to perform this work at the appropriate time, to avoid disturbing finished work.

# 9. CONTROL, QUALITY ASSURANCE AND DOCUMENTATION

Austevoll Breakwater. Photo: Norwegian Coastal Administration

## 9.1 IN GENERAL CONCERNING CONTROLS AND DOCUMENTATION

It is important that controls be performed on an on-going basis during the execution. This applies to the materials that are being used in the construction, to the execution and the structure as it is gradually built up.

What scope of controls are necessary during the execution in order to ensure that the specified quality is attained must be clarified in the project planning. Hence some controls are also described in the preceding items.

It must also be ensured that there is sufficient documentation that the requirements and preconditions for the project planning have been fulfilled as described.

There are no perfect lines and surfaces in the sea. Hence there must be a reasonable limit for deviations that could possibly weaken the breakwater structure.

Measurements must always be undertaken of the work performed. It is recommended that measurements be performed of cross-sections at a minimum of every 10 m as well as longitudinal profiles of the breakwater crest. Measurements using with drones can also be a good alternative.

## 9.2 THICKNESSES

For the stability and strength of the breakwater it will in general not be negative if some of the quantities or layer thicknesses are exceeded. This is, however, preconditioned on the possible new layers that will be sitting on top still retaining their stipulated thicknesses. If the breakwater is less steep that stipulated, it will lead to an increase in the amount of filter material and blocks. Note that other considerations (for example available depths, development boundaries, etc.) may pose impediments to overfilling being able to be accepted.

- Overfilling of core material can be accepted if the filter and block layer are filled with the thicknesses that were stipulated.
- Overfilling of filter material can be accepted if the block layer is laid out in the stipulated thickness.
- Lacking core material can be compensated for with increased filter material or increased block layer thickness.
- Lacking filter material cannot in general be accepted, but local deviations down to a thickness of $2D_{50}$ can be accepted.
- Lacking block layer cannot be accepted and must be refilled.
- Slope angles that are too steep cannot be accepted and must be corrected. Other remedial measures may be specially considered.

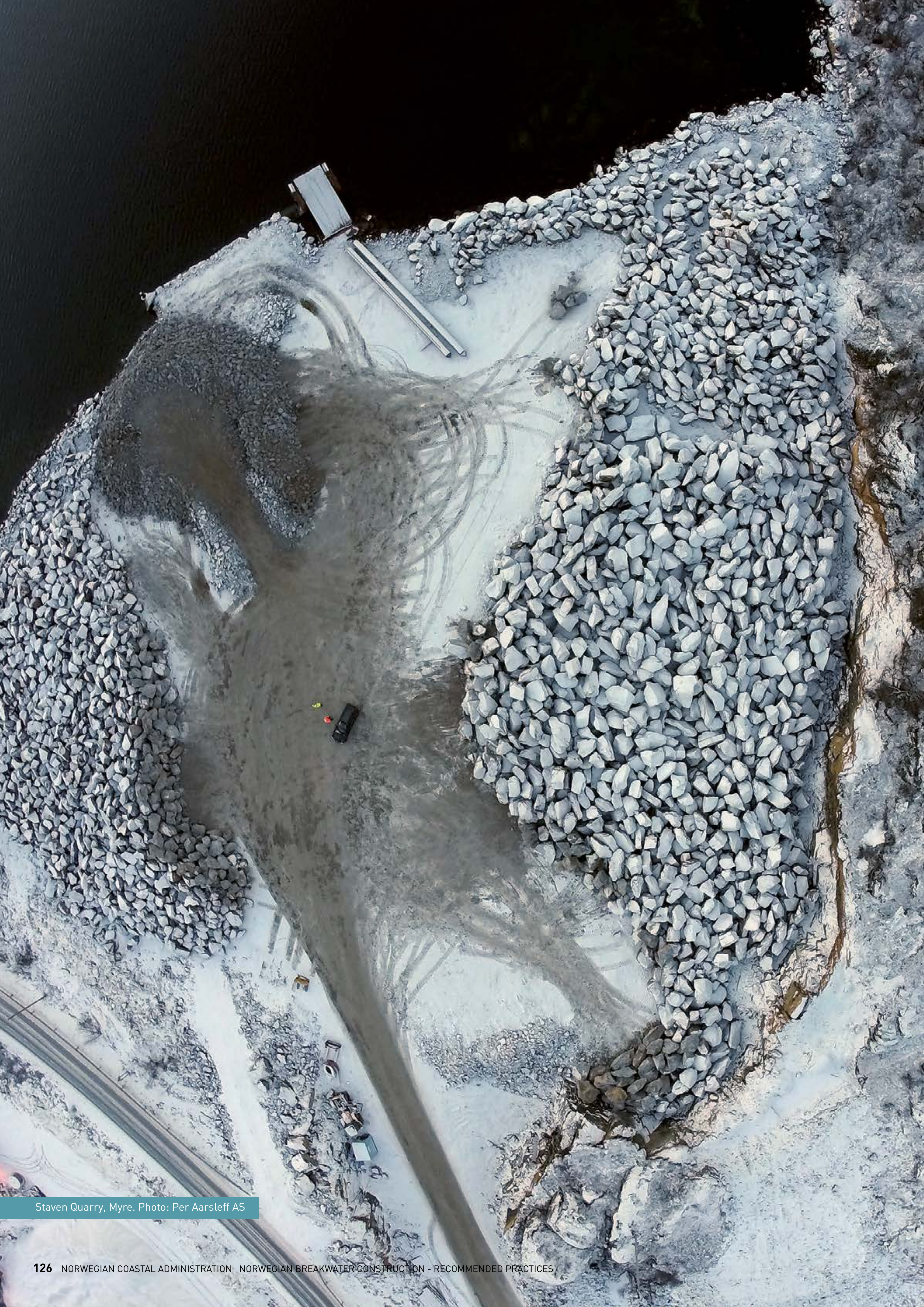
Slope angles that are too mild can generally be accepted, provided that the described layer thicknesses are retained. But it is recommended that this be clarified before the execution commences.

## 9.3 CHECKING AND MONITORING PROGRAMME

In many projects, requirements apply concerning monitoring or environmental-related conditions (turbidity, changes in natural diversity, etc.), see for instance point 5.13.

The specifications in the underlying basis for the work will also show what documentation must be submitted upon completion of the construction. In addition, this often will pose requirements on the client for checking the execution. The contractor must make his own proposals as to what parameters should be monitored in order to achieve the specified and desired result.

Based on these preconditions, a practical inspection and monitoring programme must be prepared that is executable and is included as a part of the daily activities of the contractor, possibly in conjunction with special inspection and monitoring authorities. If this work is distributed across multiple actors, it is important to have good co-ordination, follow-ups and close working relationships between the inspectors, the client's on-site management and the authorities so adaptations are agreed upon and the final documentation will be as the parties have stipulated.



Staven Quarry, Myre. Photo: Per Aarsleff AS

# 10. HEALTH, SAFETY AND THE WORKING ENVIRONMENT (HSWE)

During the planning phase, a plan will be made to ensure that health, safety and the working environment at the construction site are addressed in a good manner for all employees at the site as well as for other parties affected (land owners, neighbours, nearby facilities, passing ships, etc.). This is called an HSWE plan. The legal basis for the HSWE plan is found in the Norwegian Construction Client Regulations. These regulations describe how the client, i.e. the builder or project owner, must address health, safety and the working environment (HSWE) through project planning and execution of building and construction work with an emphasis on the stated conditions for the workers who are performing the work.

The HSWE conditions must be assessed against both the technical and organisational choices. As a part of this work, it must also be ensured that enough time is set aside for project planning and execution of the different work operations.

Evaluations of health, safety and the working environment must be described in all project phases from the outline project to the construction phase. This evaluation must be detailed through the different project planning phases up to when the HSWE plan is available. In order to ensure that this work is performed in a satisfactory manner, the client must appoint a co-ordinator for the project planning.

The HSWE plan is in fact the client's tool for ensuring that the risk conditions connected with the building and construction work in the project are managed in an appropriate manner. The HSWE plan will thus be an important means of assistance for the client's follow-ups and any possible co-ordination of the HSWE conditions during the execution phase. If the client has hired several contractors at the construction site, or is performing parts of the work under its own auspices, then it is necessary to appoint a special co-ordinator for the execution.

For more detailed instructions on how this is done, refer to the Norwegian Labour Inspection Authority's guidelines. A number of private companies and governmental authorities have built up significant competence in this.

It is recommended that the client's HSWE plan be included in the tender documents for the execution phase. Subsequently, this plan must be co-ordinated with the systems of the contractor(s) for HSWE or HSE (health, safety and the environment), which are also used as a designation for this work. The HSWE plan must then be kept up to date on an on-going basis as the project gradually develops during the construction phase. It is important to define who will perform this updating.

Træna Breakwater. Photo: Norwegian Coastal Administration

# 11. EXTERNAL ENVIRONMENT

It is recommended that a plan be prepared for the external environment (EE plan) as a part of the project planning material corresponding to the above-mentioned HSWE plan. The EE plan must ensure that guidelines and requirements for looking after the external environment around the construction site are addressed during the construction phase. The plan should be prepared during an early phase and revised and updated through later project phases. In particular, it is important that this plan be worked on and supplemented prior to the start-up of the construction work.

During the planning work, work must be done on the environmental challenges for the project, including the scope and requirements for remedial measures. Where there is a requirement concerning preliminary and follow-up studies of the environmental effects of the initiative, the need must be clarified at the latest during the detailed project planning and incorporated into the monitoring programme.

The scope and complexity vary depending upon the project's size and environmental effects. It is recommended that the following themes be addressed in the EE plan:

1. Noise
2. Vibrations
3. Air pollution
4. Pollution of soil and water
5. Landscape situation
6. Local environment, neighbours and outdoor life
7. Natural environment and natural diversity
8. Cultural environment
9. Energy consumption
10. Choice of materials and waste management

Berlevåg Harbour. Photo: Tore Larsen

# 12. OPERATION, MAINTENANCE AND REPAIRS

## 12.1 GENERAL

What is meant by the operating phase is that the breakwater has been completed and its final inspection performed. A breakwater will always represent a significant investment in infrastructure and it is necessary to have a conscious awareness of the fact that a breakwater also has an operating phase.

## 12.2 INSPECTION AND FOLLOW-UPS

All breakwaters ought to have periodic inspections and controls, and it is recommended that good inspection procedures be established for observing and reporting damages and abnormal events. Inspections should be conducted by a technically competent person and a written report ought to be drafted and archived by the breakwater owner.

Situation-determined reports should be delivered if conditions exist that ought to be reported.

Table 12-1 specifies the recommended scope of an inspection.

| WHAT SHOULD BE CHECKED | DOCUMENTATION | REPORTING |
|---|---|---|
| Damages and sliding out in the block layer on the breakwater's sea side with exposure of the underlying blocks/filter. | In writing with sketch of the damage and photo in a special report | Immediate |
| Sliding out of blocks at the seaward approach or fairway area and posing hazards to the traffic. | | |
| Other sudden changes that pose a hazard to users of the harbour or the public. | | |
| Damage to mooring arrangements (bollards, bolts) or deck, reported by users. | Verbal report | Immediate |
| Visible subsidence and changes in slope angles or breakwater incline. | Written report with sketches and photo | Semi-annual report |
| Scouring out of filter layer and occurrences of small stones (filter or core) on the breakwater's sea side. | | |
| Changes in depth conditions in the harbour, especially changes in sandy bottoms. | | |

*Table 12-1 Inspection procedures for breakwaters.*

| PLACE | INSPECTION POINT | COMMENTS |
|---|---|---|
| **Block layer** | Size of rock, possible cracking and crushing | Measurement of a representative selection of rock |
| | Number of layers | Visual inspection |
| | Sinking and changes in slope angle | Measured using instruments |
| | Dislocation of marked blocks | Only performed for larger structures |
| **Deck and crown wall** | Sinking and cracking | Visual inspection |
| **Block layer under water** | Sinking in block layer, undermining, overhang | Investigated with diver; photographs taken |
| **Breakwater toe** | erosion protection, if it has been made | Investigated with diver; photographs taken |
| **Filter layer** | Stability | Check that the filter materials are not seeping out on the breakwater's surface (lee side or sea side) |
| **Breakwater trunk** | Mooring arrangements and other equipment | General inspection and controls |
| **Seaward approach** | Investigate whether blocks can pose a hazard to boat traffic | |
| **Breakwater** | Design basis | Check whether there are changes in the use of the breakwater or water level cf. the preconditions for the original design of the breakwater |

*Table 12-2 Status inspection for breakwaters.*

## 12.3 STATUS INSPECTION

Status inspection is a more comprehensive investigation of the breakwater's status that ought to be performed occasioned by the following:

1. after larger damage that causes a need to consider an upgrading of the breakwater (new design)
2. after repairs to any possible larger damage
3. after the first winter after its completion
4. otherwise every fifth year

The status inspection encompasses all design values for the breakwater, and the purpose is to check whether changes have occurred in the breakwater (wear, crushing, sinking, sliding out, erosion, etc.) that were not stipulated when it was built.

The status inspection should at a minimum encompass parameters that are described in table 12-2.

The report from the status inspection must perform a comparison with the original design values, and contain an assessment of any possible measures. It may be relevant to increase the frequency of these inspections.

The report must also contain an assessment of the breakwater's function, and in particular it must assess whether the breakwater is suited to the purpose it is being used for at the moment.

### 12.4 MAINTENANCE

Simple maintenance must always be prepared upon construction of the breakwater. In nearly all instances maintenance of a breakwater consists of replenishment of armour units on the breakwater's outer side in order to replace losses of individual blocks, and in some cases of replenishment of erosion protection.

Maintenance of mooring arrangements and concrete structures is not discussed here.

The scope of planned maintenance will vary for every single breakwater, but it may encompass:

- Preparation of roads for heavy machines also after completion of the breakwater
- Securing of access rights to the breakwater
- A spare parts area of armour units, which are used in connection with ordinary operation and stored in the vicinity of the breakwater (up to 5 % of the block volume of the original breakwater is recommended)

### 12.5 USE OF A BREAKWATER FOR OTHER PURPOSES

The need for a breakwater and the use of it will be clarified in the planning process before the breakwater is decided to be established. The use of the breakwater may, however, be changed without any formal decisions and without the owner being consulted. In particular, there must be an awareness of the possibility that the breakwater will be used to protect other and greater assets than those it was designed for. Examples of such use changes are construction of residential areas, factories and industrial areas, etc.

For berm breakwaters it is especially the case that the blocks could become unstable and undergo movement. Berm breakwaters thus ought to be closed to ordinary traffic, and a sign should be set up giving information about the closure and on the hazards of entering the breakwater.

The municipality and possibly the port authority should be notified in writing of the capacity that the breakwater has been designed for, and what protection exists against collapse, overtopping or functional failure in any form.

### 12.6 RAISING OF STANDARD

Raising the standard of a breakwater means modifying it in order to achieve more comprehensive functionality than before. For example, a breakwater can be extended in order to make for a larger harbour area than before and in order to provide better wave protection in the existing harbour. Such a raising of the standard will be treated as a new construction project as is described in the preceding chapters.

A raising of the standard may be achieved by a simple rebuilding of the breakwater head. If lower wave heights in the harbour are the objective, then this can be a good alternative. See item 7.5.8 concerning special considerations in the design of a breakwater head.

Be especially aware that an expansion/extension of a breakwater can lead to increased loading on the older part, hence it should be checked whether there is a need to reinforce the existing breakwater.

### 12.7 REPAIRS TO BREAKWATERS

#### 12.7.1 Minor repairs

Minor repairs to the breakwater trunk encompass the straightening out of blocks sliding out, sinking in the deck, etc. In order to classify the measure as a minor repair it is a precondition that the damage be small and local, and that it not be caused by design errors, under-dimensioning or erroneously performed work.

#### 12.7.2 More substantial repairs

If the damage cannot be surmised to be local, then there will be a weakness in the entire structure that ought to be corrected. Many older breakwaters in Norway were built without any form of design, and the formulation of the design was governed by experience, available equipment and local conditions.

If the breakwater becomes damaged, it can also be due to the external conditions having changed, for example a greater wave height due to dredging, new piers, etc. In such cases, the repairs must technically be treated as new construction, i.e. that we start with a fundamental design of the breakwater based on the load data (waves, water level, etc.).

In the technical part of the project planning, utilisation of the existing breakwater in the best possible manner should be attempted.

An old breakwater will be so well-consolidated and worked over by the waves that it will be far stronger than the applicable formulae for design dictate.

A good alternative for more substantial repairs is a co-called hybrid berm breakwater: a breakwater where the existing breakwater is used as a core and filter, and the berm is laid on the outside of the old breakwater. The berm in this case may be smaller and have a completely different shape than the traditional berm due to the strength of the old breakwater.

For more substantial repairs, it is relevant to seek support from numerical models and possible lab modelling, because such repairs will be so comprehensive and expensive that it is natural to seek to upgrade or improve the harbour at the same time, provided that the need to do so is present.

## 12.8 INSTALLATIONS WORTHY OF PROTECTION

### 12.8.1 General

The harbour structures that have been built in Norway encompass a number of breakwaters that vary to a large degree in their formulation, design, age and use of material, etc. In many instances the breakwaters represent aesthetic qualities and interesting cultural history that are connected in different manners to the development of their local societies. The qualities are especially closely connected to the technological developments that the Norwegian Coastal Administration and prior Directorate of Harbours have undergone with respect to equipment and construction methods.

Some general guidelines for how to go about repairing and improving structures that may possess such qualities are given below.

### 12.8.2 Administration, maintenance and repairs

It is recommended that the Norwegian Coastal Administration's "National protection plan for marine infrastructure" be used when planning repair work. A number of protected harbour structures and breakwaters across the entire country are presented in this overview.

Some individual older breakwater structures continue to have their original formulations. When undertaking improvements, expansions and maintenance of such structures it is important to have a conscious awareness that the structure possesses an intrinsic value in the first place. Whether a breakwater has such qualities and these ought to be preserved, and possibly in what manner, should be tangibly clarified in each individual instance.

With such a point of departure, it can be said in general that a new project must be planned and built in relation to the existing structure where this is possible with respect to the design, architectural style and use of material. It would for example be considered unfortunate if the replenishment of armour units and use of concrete are done uncritically during repairs to older, beautifully walled breakwaters.

Such evaluations ought to be made during an early phase when a recommendation is to be given or a decision made on measures for repairs, maintenance and further expansions that affect the existing structure.

## 12.9 RECHECKING AND EVALUATION. CONTROLS UPON EXPIRY OF LIFESPAN

It is recommended that a new calculation of the breakwater structure be done when its lifespan is close to expiring. In particular, it is important to utilise that most recently available climate data in such rechecking of the structure.

*Figure 12-1 Part of the breakwater at Silda, Municipality of Vågsøy. The older part was completed in 1889. The structure is a good example of how regard was paid to a certain degree to the architectural style and use of material when further expanding the breakwater.*

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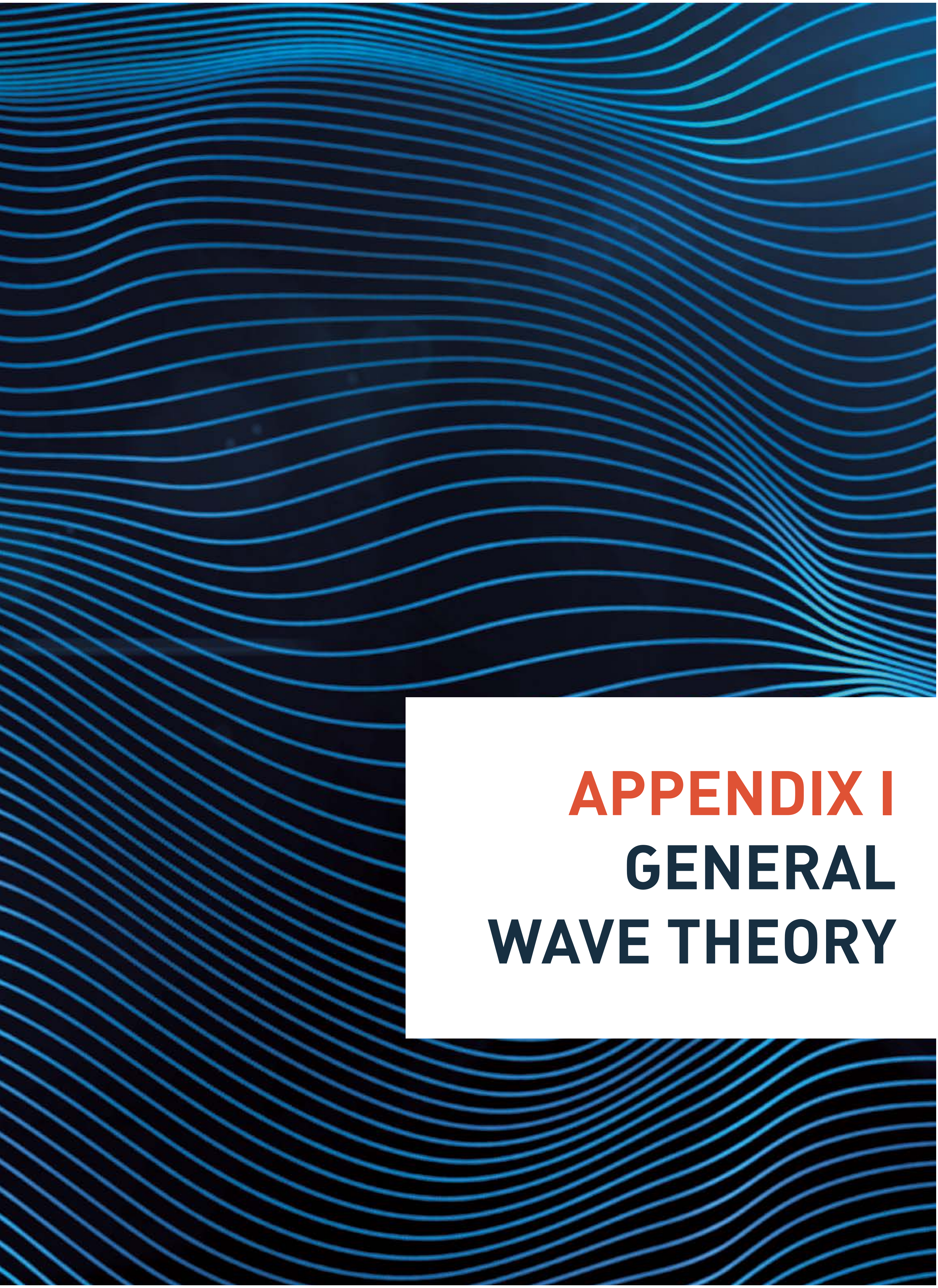


# APPENDIX I
# GENERAL WAVE THEORY

# 1. WAVES

## 1.1 GENERAL

In areas without ice, waves provide the design loads on breakwaters. Ocean waves are irregular and stochastic in shape, height, length and speed of propagation. The designation sea state is used for a wave situation where stochastic wave parameters such as significant wave height $H_s$, mean wave period $T_m$, peak period $T_p$, mean wave direction $\theta_m$ etc. are assumed to be constant within a period of time. The most effective manner of describing a sea state is by the wave spectrum. The duration of a sea state can vary. between 30 minutes and up to 10 hours. It is common to assume 3 hours as a representative duration of a sea state.

Engineering-related design formulae have been developed for different types of breakwaters where the input parameters are $H_s$, $T_p$ and the water depth $d$ at the breakwater location during the design sea state. The design sea state for breakwaters is given as a 200-year condition. I.e., a sea state that is expected to not be exceeded more than once every 200 years on the average. In order to determine 200-year values, we must have access to long measurement series of wave data. Such measurement series are usually not available. We hence base the analysis on simulated sea states using weather data (hindcasting data).

If the breakwater is well-protected against waves from the open sea, it will be sufficient to use local wind data over the open sea in order to simulate the wave conditions. The 200-year wave condition is then calculated based on the 200-year wind, local bathymetry and fetch lengths.

If the breakwater location is open to waves from the open sea, hindcasting data is used as a basis. This is simulated wave data calculated from the historical weather chart for the sea areas, performed for Norway by the Norwegian Meteorological Institute. The hindcasting archive contains more than 60 years of wave data for every third hour for a number of positions on the open sea, however usually not from the breakwater location. It is recommended that measurements be undertaken at the breakwater location in order to calibrate hindcasting data, but these will usually not be long enough for determining 200 year values. There thus is a need for wave models that are produced by taking a point of departure in hindcasting data and simulating the changes in the wave parameters out to the breakwater location.

This chapter is divided up into four sub-chapters:

1. **Wave types** – where the different wave types are described, item 1.2.
2. **Wave description and observation techniques** – where the most significant words and expressions connected with wave description and theory are presented, item 1.3.
3. **Wave statistics** – where short-term and long-term waves are presented, item 1.4.
4. **Lab modelling** – with a brief summary of scaling for lab modelling as well as other useful information concerning physical lab modelling, see item 1.5.

## 1.2 WAVE TYPES

The surface of the sea reflects the effects of astronomical (tide) and meteorological (wind-generated waves, swell and storm surges) forces, as well as the displacement of water (ship's waves, slide-generated waves and tsunamis due to earthquakes). Figure I-1 gives a schematic representation of

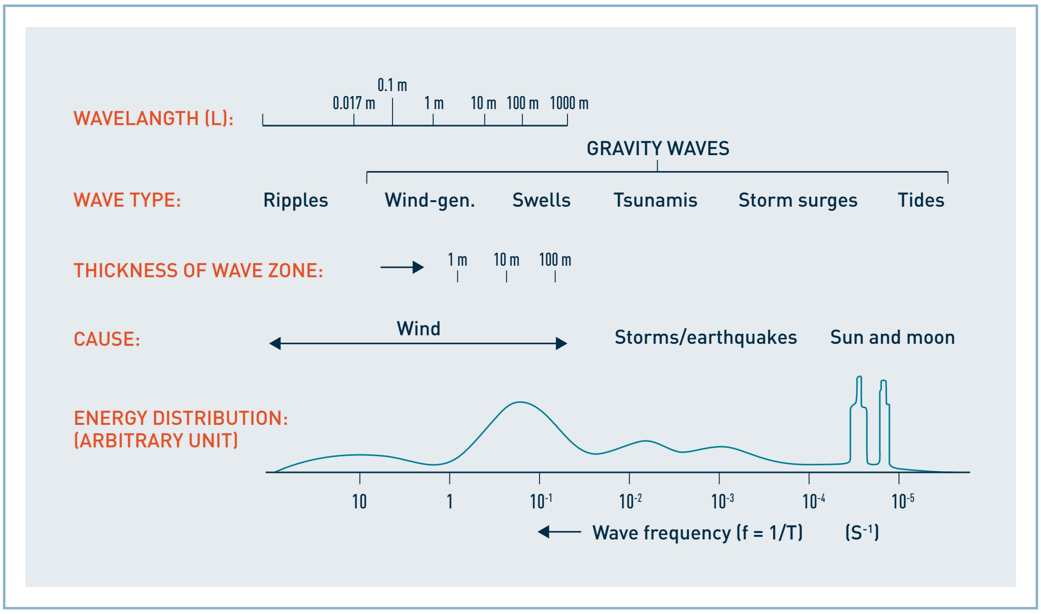



*Figure I-1 Schematic energy spectrum of waves at sea plotted against frequency.*

the different waves observed on the surface as a function of frequency (inverse of wave period). Tides have a well-defined frequency, whereas the other categories are distributed across a broad frequency band. Wind-generated waves are short-periodic short-cresting waves generated by local wind, whereas swell is waves that have propagated out from their generation area in storms far from the location and have a pronounced long-crestedness and relatively long wave period. Wind-driven waves (wind-generated waves) are a stochastic process with periods of 2–20 seconds, whereas swell typically have periods of greater than 10 seconds. A sea state may be purely wind-generated waves or purely swell or a combination of wind-generated waves and swell.

Available data indicates that we can presume that the surface level rise is approximately normally distributed. In the following, only wind-driven waves and swell will be discussed, and these will subsequently be referred to as waves.

## 1.3 WAVE DESCRIPTION AND OBSERVATION TECHNIQUES

### 1.3.1 General

A definition sketch of a regular wave is shown in figure I-2.

There is a unique relationship between the wave length, $L$, and the wave period, T, known as the dispersion relation. For regular linear waves, this is given by

$$L = L_0 \tanh(2\pi d / L);\ der\ L_0 = \frac{g}{2\pi} T^2$$

*(0.0.1)*

An approximation with precision good enough for every practical condition was published by Bob You on the "Coastal list" in 2008:

$$L = L_0 \tanh(\xi_0);\quad \xi_0 = (k_0 d)^{1/2}\left(1 + \frac{k_0 d}{6} + \frac{(k_0 d)^2}{30}\right);\quad k_0 = \frac{2\pi}{L_0}$$

*(0.0.2)*

The advantage of this expression is that the unknown wave length, $L$, appears only on the left side of the equals sign. For steep waves that are coming close to breaking, the steepness will enter into the dispersion relation.

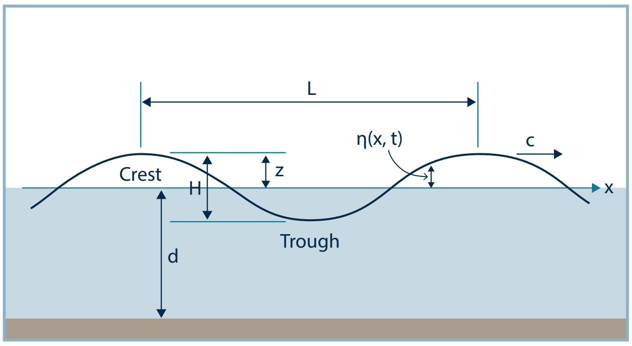



*Figure I-2 Definition sketch of a regular wave.*
*c = L/T is the propagation speed where L is the wave length and T is the wave period.*

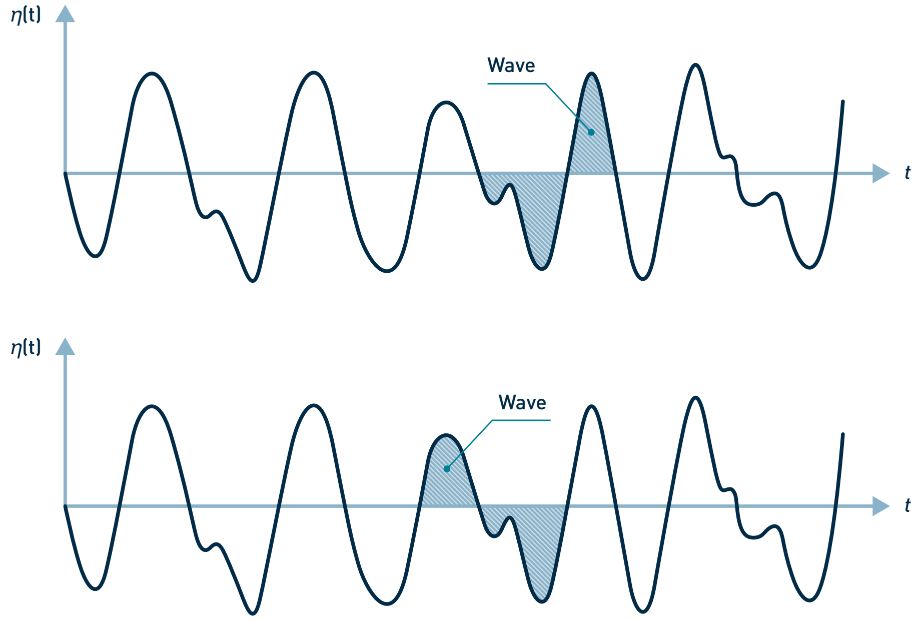



*Figure I-4*
*Definition of a "wave" in a time domain analysis from a wave measurement.*

*Down-crossing analysis (top) and up-crossing analysis (bottom).*

In reality, wind waves are irregular and stochastic. Figure I-3 shows a typical time series of measured wind waves. It is clear that the definitions given above (L and T) are insufficient to describe the measured wave data in figure I-3. The following text describes different techniques used to measure wind waves, and introduces different ways by which to describe such waves.

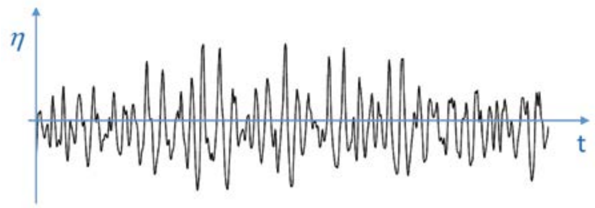



*Figure I-3 Typical time series of measures wind wave.*

### 1.3.2 Observation techniques

A number of observation techniques exist, see Holthuijsen (2009):

- Visual observations are often the only source for wave information available to the engineer. When needed, measurements with instruments will be necessary.
- Measurement techniques can be divided up into in-situ techniques (instruments that are used in the water) and remote measurement techniques (instruments that are used near or high above the water).
- The most common in-situ instruments are wave buoys and wave poles. Other in-situ instruments are inverted echo sounders, pressure sensors and current meters. These instruments are mounted either on a fixed structure or are floating and anchored.
- The most common early warning method is radar, which is based on actively radiating the sea surface with electromagnetic energy and detecting the associated reflection. Radar may be placed at the coast using fixed platforms (for example oil production platforms) or moveable platforms at a relatively low altitude (aeroplanes) or high altitude (satellites).
- Radar can be used to acquire pictures of the sea surface, however it can also be used as a distance meter or for measuring the variation in the surface roughness.
- Each measurement technique has its own special characteristics as regards its operating performance, precision, maintenance, cost and reliability.
- The most common result of a wave measurement is a time series of the height of the sea surface relative to the mean value over the course of the measurement period and in a definite horizontal point or range in the case of radar measurements. It can be supplemented with directional information from, for example, directional wave buoys.

### 1.3.3 Methods for describing waves

#### 1.3.3.1 In general concerning different methods

The usual statistical description of wind waves requires statistical stationarity. I.e. that the expected value, standard deviation and other statistical parameters of the sea surface's movement do not change within a period of time. A time registration of waves (the sea surface's vertical level as a function of time at one place) hence needs to be as short as possible. However, in order to attain sufficient precision of calculated statistical parameters, the measurement series ought to be as long as possible. The compromise is time registration over 15–30 min. If the series is longer,

it ought to be divided up into such segments, possibly overlapping, where each is presumed to be stationary.

The wave parameters are determined by analysing such time registrations, either in the time domain or in the frequency domain.

#### 1.3.3.2 Time domain analysis of waves

The wave condition in a stationary time registration, $\eta(t)$ can be characterised by average parameters, for example such as significant wave height, $H_s$, peak period for waves, $T_p$, and mean wave period, $T_m$. Zero-crossing techniques (either down-crossing or up-crossing) are utilised for identifying individual waves in the time series from a point. The principle is sketched in figure I-4.

Significant wave height $H_s$ is defined as the average value of the wave heights for the highest 1/3 of the waves. If $N$ is the total number of waves in the time registrations:

$$H_s = H_{1/3} = \frac{1}{N/3}\sum_{j=1}^{N/3} H_j$$

Significant wave period $T_s$ is the average of the wave period for the 1/3 highest waves.

$$T_s = T_{1/3} = \frac{1}{N/3}\sum_{j=1}^{N/3} T_j$$

Mean wave period, $T_m$ is the average of the wave period of all the waves in the registration.

$$T_m = \frac{1}{N}\sum_{j=1}^{N} T_j$$

It is interesting to note that the significant wave height is quite well correlated with the wave height estimated visually by experienced observers. This is not the case for significant wave period or mean wave period.

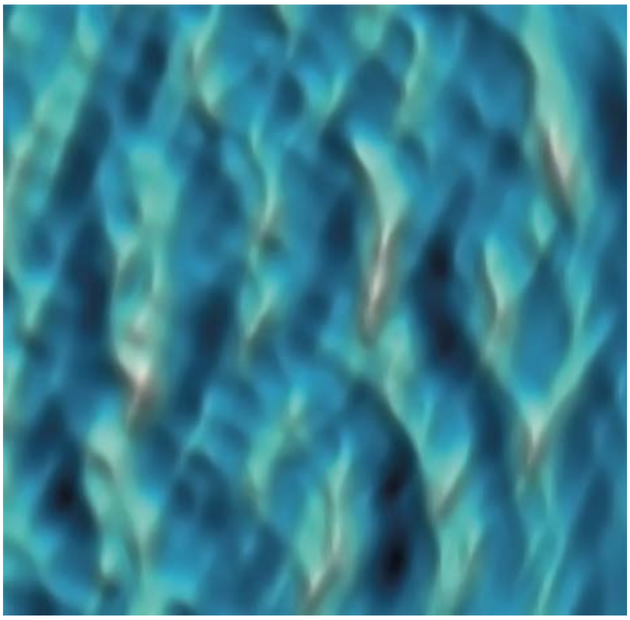
#### 1.3.3.3 Analysis in the frequency domain

A more complete description of the sea state is attained by modelling the time registration as the sum of a large number of statistically independent, regular waves (wave components). This concept is called the "random phase/amplitude model" and leads to the concept with the one-dimensional variance density spectrum (the wave spectrum) $S(f)$), which shows how the variance of the sea surface height is distributed with respect to the frequencies of the wave components ($f = 1 / T$). If $N$ wave components are used, each with a wave amplitude $a_n$, frequency $f_n = \frac{\omega_n}{2\pi}$ and random phase such that

$$\eta(t) \approx \sum_{n=1}^{N} a_n \sin(\omega_n t - \alpha_n),$$

is the variance

$$\sigma_\eta^2 \approx \frac{1}{T_R}\int_0^{T_R} \eta^2(t)dt \approx \frac{1}{N}\sum_{n=1}^{N}\frac{1}{2}a_n^2(t) \approx \int_0^{\infty} S(f)df$$



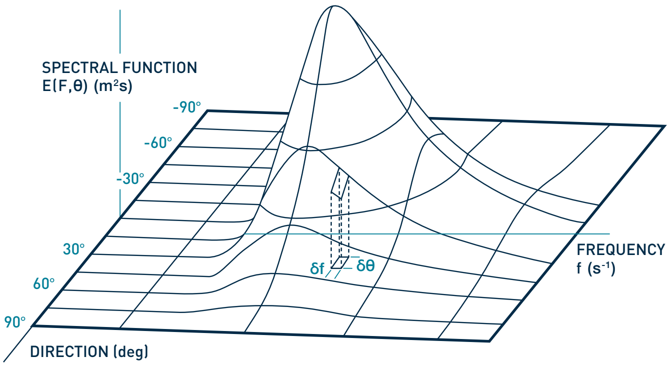



*Figure I-5 Simulation of the sea surface. The picture at the top shows the surface rise viewed from above. The picture at the bottom is an illustration of the wave direction spectrum. The small "volume" $\delta f \cdot \delta\theta \cdot E(f,\theta)$ corresponds to the variance $a^2/2$ for a single wave component.*

If the situation is stationary and the surface height has a Gaussian distribution, then $S(f)$ gives a complete statistical description of the waves.

This concept can be expanded to the three-dimensional, moveable sea surface, which is subsequently viewed as the sum of a large number of statistically independent, harmonic waves that are propagated in all directions across the surface of the sea. The corresponding two-dimensional variance density spectrum $E\,(f,\,\theta)$ shows how the variance s distributed across the frequencies $(f = 1\,/\,T)$ and the directions $(\theta)$ of these harmonic wave components:

$$E(f,\theta) = S(f)D(\theta,f);\ f > 0,\ -\pi \le \theta \le \pi$$

The directional spectrum $D(\theta, f)$ gives the directional spreading of the wave energy. The sea surface can now be simulated as a sum of sine wave components with different frequencies and directions determined by the frequency-direction spectrum. An example of such a simulation is shown in figure I-5.

The one-dimensional spectrum $S(f)$ can be calculated from the two-dimensional spectrum $E\,(f,\,\theta)$by integrating over all directions.

The variance density spectrum also gives a description in the physical sense when it is multiplied by $pg$ ($p$ is the density of water, $g$ is the acceleration of gravity). The result is the energy density spectrum. It shows how the energy of the waves is distributed across the frequencies (and the directions).

Wave parameters such as significant wave height $H_{m0}$ and peak period $T_p$ are determined from the calculated spectrum as follows:

$$H_s = H_{m0} = 4\sqrt{\int_0^\infty S(f)df}$$

$H_{m0}$ is the value of the wave height that is specified in weather warnings, marinograms (ocean forecasts), etc.

Spectral peak period $Tp$ is the inverse value of the frequency $fp$ where the value of the frequency spectrum is the highest. $Tp$ cannot be defined uniquely for multi-peak spectra, for example a sea state with swell and wind-generated waves. We then often operate with two values – one for low-frequency waves (swell) and one for high-frequency waves (wind-generated waves).

Mean wave period $T_Z$ (or $T_{m02}$) is defined via spectral moments $m_i$. $T_z = T_{m02_0} = \sqrt{m_0 / m_2}$ , where the spectral moment $m_i$ is defined by $m_i = \int_0^\infty f^i \cdot S(f) df$. It can be seen that $Tz$ is extremely dependent upon the spectrum's form at high frequencies and thus the estimate can be somewhat unstable.

The interrelationship between $Tz$ and the peak wave period, $T_p$, can in the case where the sea state is described with a JONSWAP spectrum be taken as

$$T_z = T_p\left(0.6673 + 0.05037\gamma - 0.006230\gamma^2 + 0.0003341\gamma^3\right).$$

$\gamma$ is the reinforcement factor (also called the peak enhancement factor or peakedness parameter) in the JONSWAP spectrum. For a PM spectrum the relationship can be taken as $T_p = 1.4 \cdot T_Z$. (ref.: DNVGL-RP-C205, section 3.5.5).

The period associated with the most probably highest wave in a sea state is designated $T_{Hmax}$. The most probable height of the highest wave in a sea state is designated $H_{max}$, and $T_{Hmax}$ is its expected period. $T_{Hmax}$ can be found indirectly from $T_Z$ or $T_p$ from measured time series, or from $H_{max}$ and wave choppiness estimates – usually in order to obtain a number of possible associated periods. $T_{Hmax}$ cannot however be derived directly from the wave spectrum.

For a sea state with a duration of 3 hours, $H_{max}$ can be assumed to be equal to 1.9 times the significant wave height $H_s$.

#### 1.3.3.4 Model spectra

For planning and design it is handy to use model spectra instead of actual spectra. A number of these have been proposed for different purposes. Most used as an estimate of $S(f)$ in Norwegian fairways for wind-generated waves offshore. is the JONSWAP spectrum. This is because storms in Norwegian fairways have a course with a relatively quick increase in wind strength and with the storm passing an area relatively quickly. An example of a storm developing offshore is shown in figure I-6. The figure shows values of significant wave height around the peak of the storm. The data is from the hindcasting archive offshore Mehamn. The measurements support the presumption that a sea state may be assumed to be stationary over 3 hours, which also applies for the storm peak here.

Other spectra are often used for modelling of sea states under more moderate conditions, for example the Pierson-Moskowitz spectrum (the PM spectrum) or if it is necessary to model double peaked spectra, i.e. model both swell and wind-generated waves. Examples of the latter are the Ochi-Hubble spectrum and the Torsethaugen spectrum (Ochi and Hubble, 1976; Torsethaugen, 1996, 2004). Goda (2010) gives the following expression for the JONSWAP spectrum:

$$S(f) = \beta_j H_s^2 T_p^{-4} f^{-5} \exp\left[-1.25\left(T_p f\right)^{-4}\right] \gamma^{\exp\left[-\left(T_p f - 1\right)^2 / \left(2\sigma^2\right)\right]}$$

*(0.0.3)*

where

$$\beta_j = \frac{0.0624}{0.230 + 0.0336\gamma - 0.185(1,9+\gamma)^{-1}}\left[1.094 - 0.01915\ln\gamma\right],$$

*(0.0.4)*

$$T_z = T_p\left(0.6673 + 0.05037\gamma - 0.006230\gamma^2 + 0.0003341\gamma^3\right),$$

*(0.0.5)*

$$\sigma = \begin{cases} \sigma_a;\ f < f_p \\ \sigma_b;\ f \geq f_p \end{cases},$$

*(0.0.6)*

$$\gamma = 1 \sim 7 \text{ (vanligvis 3.3 for norske forhold)},\ \sigma_a = 0.07, \sigma_b = 0.09$$

*(0.0.7)*

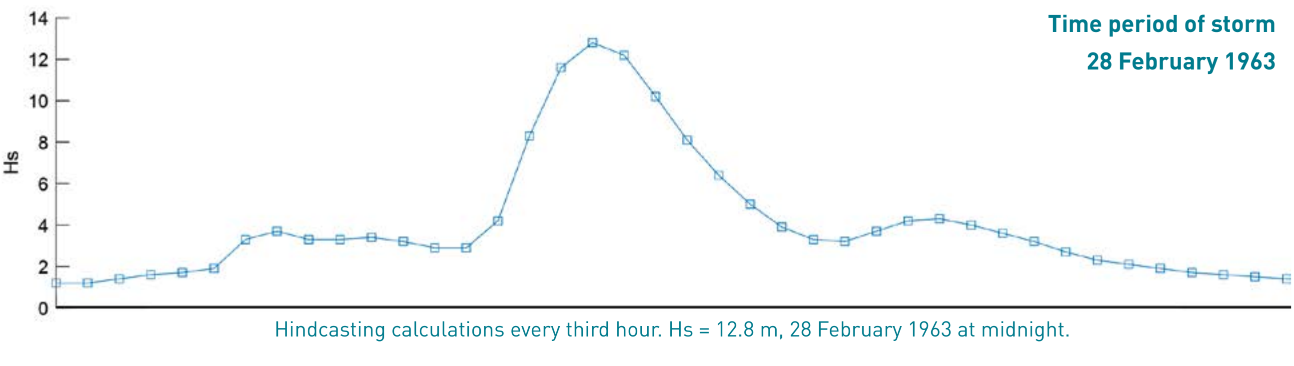



*Figure I-6 Time period of calculated significant wave height during the storm on 28 February 1963. Data from the hindcasting archive WAM10_7119N_2803E.*

A recommended formula for the peak enhancement factor as a function of $Hs$ and $Tp$ is given in DNVGL-RP-C205.

The directional spreading function $D(\theta,f)$ is normalised so that

$$\int_{-\pi}^{\pi} D(\theta|f)\,\mathrm{d}\theta = 1$$

(0.0.8)

$D(\theta,f)$ is often given as a $\cos^{2s(\omega)}(\theta-\theta_m)$distribution where $\theta$m is the main direction of the waves. The normal distribution can in many contexts be just as well-suited and has (especially) the property that it is simple to understand:

$$D(\theta) = \frac{1}{\sqrt{2\pi}\sigma_1}\exp\left(-\frac{(\theta-\theta_m)}{2\sigma_1^2}\right)$$

(0.0.9)

Where the two parameters are mean wave direction, $\theta_m$, and circular standard deviation, $\sigma_1$. When the normal distribution is used together with the JONSWAP wave spectrum, it is usually presumed that $\theta_m$ is the same for all frequencies. $\sigma_1$ usually varies with the frequency. The directional values are specified in radians.

## 1.4 WAVE STATISTICS

### 1.4.1 Short-term statistics

The short-term distribution of individual wave heights, $H$, in a sea state is generally described by the Rayleigh distribution (Longuet-Higgins, 1952) with a cumulative probability distribution

$$\text{Prob}\ (H \le h) = P_H(h) = 1-\exp\left(-\left(\frac{h}{H_{rms}}\right)^2\right)$$

(0.0.10)

Where $H_{\text{rms}}$ is defined by

$$H_{rms} = \sqrt{\frac{1}{N}\sum_{i=1}^{N} H_i^2}$$

(0.0.11)

where $N$ is the number of waves of individual wave height $H_i$.

By using this distribution of maximum values, we find that the expected highest wave among $N$ waves in the sea state can be estimated as

$$H_{\max} = H_{rms}\sqrt{\ln N}.$$

(0.0.12)

If the sea state is narrow-banded then
$H_{rms} = H_{m0}/\sqrt{2} \cong H_{1/3}/\sqrt{2}$.
In this case, for a sea state with duration 3 hours and mean wave period $T_Z = 10\text{s}$ gives $N = 1080$ and $H_{\max} \cong 1.87H_{m0}$. If the sea state is not narrow-banded then the measurements show that or storms $H_{\text{rms}} = 0.65H_{m0}$. This gives for the example above $H_{\max} \cong 1.72H_{m0}$. We see in fact that if the sea state is not narrow-banded and assume Rayleigh-distributed wave heights, the estimate of the highest wave will be conservative. $H_{m0}$ and $H_{1/3}$ are as described earlier both estimates of significant wave height, $H_s$.

### 1.4.2 Long-term statistics

For long-term statistics of significant wave height, $H_s$, a general Weibull distribution is more suitable:

$$\text{Prob}(H_s \le h) = P_{Hs}(h) = 1-\exp\left(-\left(\frac{h-H_0}{H_c-H_0}\right)^{\gamma}\right); h \ge H_0; \gamma > 0,$$

(0.0.13)

$H_0$, $H_c$ and $\gamma$ are empirical constants for a given place. $H_0$ is the locality parameter, $H_c - H_0$ the scaling parameter and $\gamma$ the shape parameter. NOTE! There is no standard notation here. All parameters can have other symbols and can be switched around in other texts.

The Weibull distribution gives the following expression for the mean value, the variance and third order statistical moment for values of significant wave heights:

$$\bar{H}_s = \frac{1}{N}\sum_{n=1}^{N} H_{s,n} \approx \mu_{Hs} = H_0 + (H_c - H_0)\Gamma\left(1+1/\gamma\right)$$

*(0.0.14)*

$$Var\left(H_s\right) = \frac{1}{N}\sum_{n=1}^{N}\left(H_{s,n}-\bar{H}_s\right)^2 \approx \sigma_{Hs}^2 = \left(H_c - H_0\right)^2\left[\Gamma\left(1+2/\gamma\right)-\Gamma^2\left(1+1/\gamma\right)\right]$$

*(0.0.15)*

$$\mu_3 = \frac{1}{N}\sum_{n=1}^{N}\left(H_{s,n}-\bar{H}_s\right)^3 = \left(H_c - H_0\right)^3\left[\Gamma\left(1+3/\gamma\right)-3\Gamma\left(1+1/\gamma\right)\Gamma\left(1+2/\gamma\right)+2\Gamma^3\left(1+1/\gamma\right)\right]$$

*(0.0.16)*

where $\Gamma$ is the Gamma function (see figure I-7).

It is usual to plot data as $\ln\left(-\ln\left(1-\hat{P}_{Hs}\right)\right)$ against $\ln\,(H_s–H_0)$, first by selecting $H_0=0$. If the data does not fall on a straight line, the value of $H_0$ must be adjusted until the best fit of the data to a straight line is obtained. Most often it is satisfactory if a good fit is obtained to a straight line for the highest waves in the distribution $\left(H_s \geq \bar{H}_s\right)$. $\hat{P}_{Hs}$ is the empirical distribution function based on data.

Instead of this plotting method, there are analytical methods for estimating parameters, for example the moment method, which is based on calculating the average value, variance and third order statistical moment, and subsequently insert these in theoretical expressions in which the constants are incorporated. We then have three equations with three unknowns. Other methods are also available.

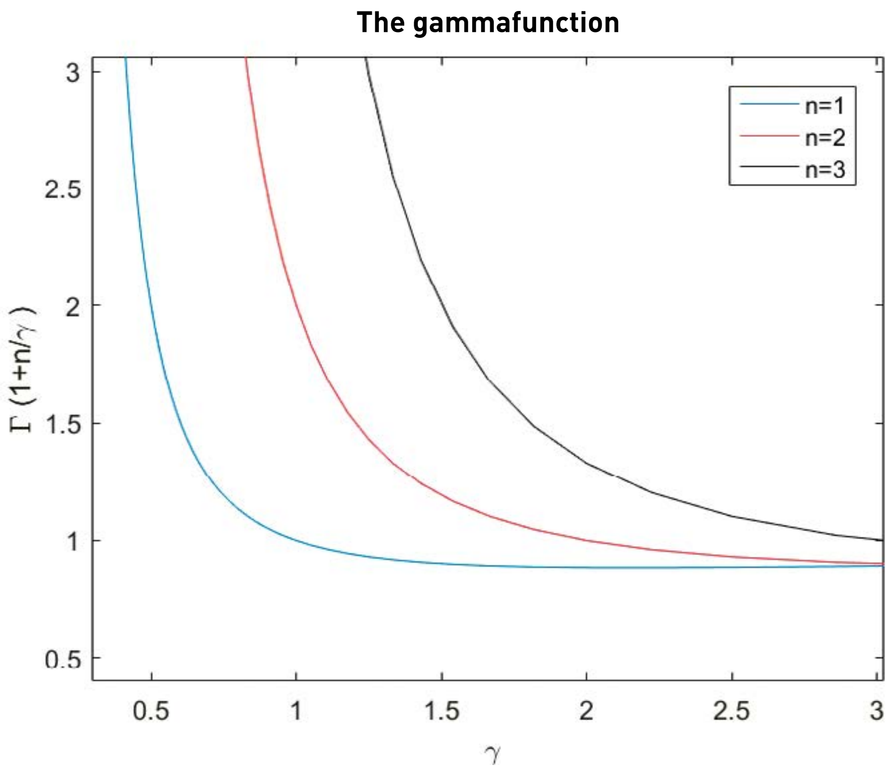



*Figure I-7*
*The gamma function as it is included in the Weibull distribution.*

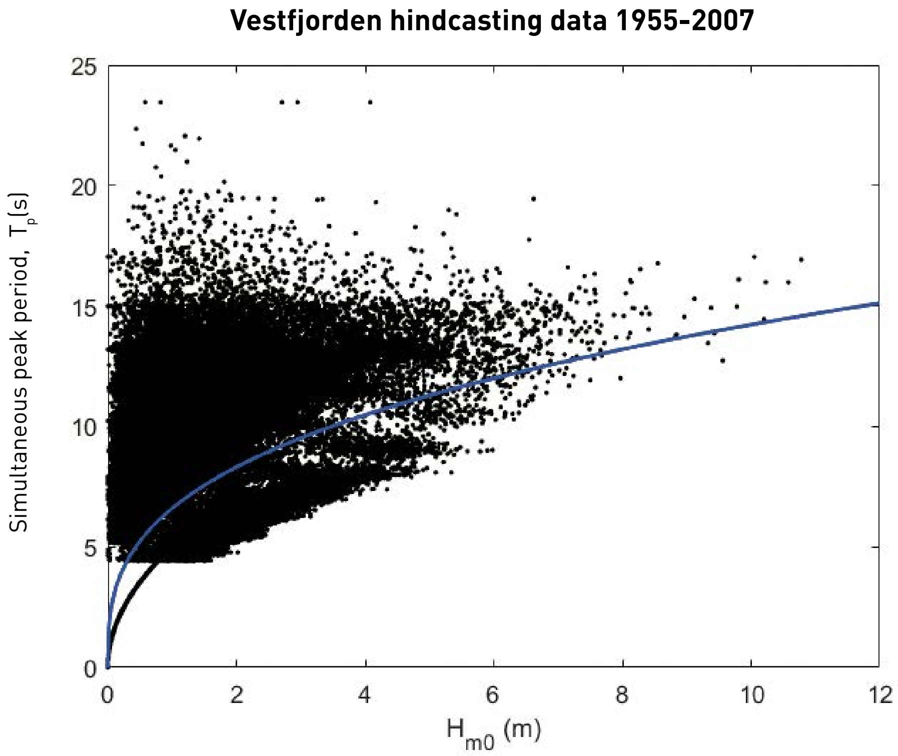



*Figure I-8*
*Simultaneous significant wave height and peak period from Vestfjorden. Hindcasting data. The formula $T_p = 6{,}6 \cdot H_s^{1/3}$ is also shown.*

The procedure for determining the maximum significant wave height $H_{s,Rp}$ over a selected recurrence interval $R_p$ years goes as follows: The recurrence interval is associated with the probability level $P_{Hs,Rp}$ in the distribution through the expression

$$R_p = \frac{\tau}{1 - P_{Hs,Rp}\left(H_{s,Rp}\right)}$$

(0.0.17)

$\tau$ is the average time between observations of the data that was used to determine the distribution function. If all the data is used, then $\tau$ is equal to the time between every measurement. If for example the highest $H_s$ value from different storms is used in order to establish $\hat{P}_{Hs}$, then $\tau$ becomes equal to the measurement period (for example 10 years) divided by the number of storms that maximum values have been selected for. NOTE! Remember that the time unit for $\tau$ and $R_p$ must be the same.

A sea state with a recurrence interval of 100 years – the 100-year sea state – is found by setting $R_p$=100·365·24. If the data is taken from a "Hindcast" wave database with values every sixth hour and all the data is used in the production of $\hat{P}_{Hs}$, then we set $\tau = 6$ hours. The relation that gives $H_{s,100}$ as a function of $H_0$, $H_c$ and $\gamma$, becomes:

$$H_{s,100} = H_0 + \left(H_c - H_0\right) \cdot \left(\ln \frac{R_p}{\tau}\right)^{1/\gamma}$$

(0.0.18)

$H_{max}$ calculated for this sea state will be an estimate for the height of the waves that are referred to as the 100-year wave.

Sea states with the same significant wave height are characterised by a large variation in the peak period $T_p$. Analysis of hindcasting data from Vestfjorden shows this in figure 5-8. For the extreme sea states, the following relation is proposed (Torsethaugen, 1990):

$$T_p \approx 6{,}6 \cdot H_s^{1/3}; \quad \left[H_s\right] = \text{m}; \quad \left[T_p\right] = \text{s}.$$

(0.0.19)

## 1.5 LAB MODELLING

In addition to item 5.4.3, there are a number of detailed considerations that could be useful in connection with physical lab modelling.

### 1.5.1 Selection of model scale

The scale of the model is primarily determined by:

- The rock sizes used in the model scale not being too small.
- The breakwater at the selected scale must geometrically fit in the wave flume or wave basin being used.
- The sea bottom must be correctly represented in order to achieve the correct wave effects in the model.
- The water depth at the wave machine must be sufficient to permit the waves to be generated by the wave machine. It must be possible to generate 120–130 % of the design wave height.
- Once the above limitations have been taken into account, we will typically select the largest possible scale. In some instances it can be relevant to model only a part of for example a new breakwater in a 3D model in order to obtain a sufficiently large scale.

### 1.5.2 Design of model

After selecting the scale, the model will be built. The sea bottom in front of the breakwater will be built as per field measurements in a solid (not erodible) material. The model will be scaled with respect to Froude's scaling law. Some parts of the breakwater ought to be scaled with the use of other scaling methods. Rock elements and toe rocks that are subjected to the direct effects of waves should be scaled with Froude scaling. The block size should, however, be adjusted using Hudson's stability formula, where the density difference between freshwater (model) and salt water (full-scale) is taken into consideration. The formula also takes into consideration the density difference between the rock used in the model and the density of the rock used in the construction.

In instances where the breakwater has a freely laid back side, for example facing a harbour area, and has a permeable core material, then the core material ought not to be Froude-scaled. It ought to be scaled so that the hydraulic gradient through the breakwater during the effects of waves is the same in the full-scale as in the model-scale. There are a number of methods for this (Jensen and Klinting, H.F. Burchart, Vaneste).

At the model scale, the core material and the filter layer will often have small rock sizes. They are so small that the only effective manner of achieving the size is to blend known rock gradings and/or sifting of materials. Rock blendings with rocks at model scale with a weight > 15 gram ought to be hand-sorted where rocks in the scaled grading are individually weighed.

The breakwater will be built up on the laboratory with great precision in geometry and layer thicknesses. When you place the rock elements, it is important to use the same placement method for the full-scale project as in the lab models. For example rocks dumped from a ship, raft or crane can position themselves partly arbitrarily. Alternatively, the rocks can be placed individually with a specific orientation that leads to a denser and often more stable packing of the rocks (which is often done in Norway). The number of rock elements placed in the armour layer and toe in the model must be kept under control so that density can be calculated, and damage during the experiment can thus be specified as percentage damage.

The armour layer and toe structure should be painted in light colours so that it is easy to count the number of rocks that possibly move during the experiment. The active part of the armour layer is defined as SWL (still water level) subtracted from the design significant wave height $H_s$. It thus is recommended to have a colour difference in this area. In addition, the colours should contribute to a visual separation of the structural parts.

### 1.5.3 Definition and measurement of waves

Before lab modelling is initiated, the breakwater will be designed to withstand selected design wave and water level conditions. The design conditions are usually selected on the basis of simulations with numerical models. The waves are determined near the breakwater, but regard is not always paid to wave breaking at the structure itself. It thus is recommended that the waves be defined close to the wave machine in the laboratory such that, with the assistance of the experiment, the wave conditions can be measured directly in front of the breakwater.

Hence the waves ought to be determined at the wave machine by placing a series of wave meters (at least 3) so that it is possible to determine the incident and reflected

wave heights by analysing the measurements. The waves are also measured in the vicinity of the breakwater, but preferably before any possible wave breaking zone. Finally, wave meters may be placed behind the breakwater in order to measure the permeation of the waves through and over the breakwater.

The wave machine in a wave flume must be equipped with an active absorption system so that waves reflected at the breakwater are not re-reflected at the wave machine and in turn affect the breakwater.

### 1.5.4 Measurement of wave overtopping

Wave overtopping is measured in a lab model by gathering up the overtopping water behind the breakwater crest in an overtopping container or box. The water is routed to the container with a minimum width of 3 metres (full-scale). The measurements are run for at least 1 hour (full-scale) so that it is possible to determine the average overtopping rate (q) in litres/metre/second.

If the collected water causes the water level in the wave flume (not relevant for 3D experiments) during the experiment to drop by 0.1 metre (full-scale), an automated system that refills the wave flume should be set up in order to ensure that there is a constant water level during the entire experiment.

The overtopping container can be placed on a load cell so that the amount of overtopping from individual waves can be determined. This may for example be relevant for breakwaters with access for people or vehicles.

It should be noted that measurement of small overtopping rates (< 2 l/m/s) is subject to significant uncertainty.

### 1.5.5 Further conditions in the design of 3D models

For 3D models in a wave basin, it is important to carefully evaluate what wave direction(s) should be investigated. Once the wave direction has been chosen, moveable walls (wave shields) are placed in the tank. These are placed from the sides of the wave machine out towards the model in order to ensure that the waves do not lose height before they reach the breakwater. It may often be visually assessed whether the waves are retaining their height and direction against the breakwater.

### 1.5.6 Execution of experiments

Verification of the stability for breakwaters with interlocking surfaces is typically performed by conducting a series of 4 to 10 individual experiments with the selected design of a breakwater head or cross-section.

Each individual experiment represents a specific combination of an irregular sea state and a water level. Every individual experiment ought to last for a minimum of 5 hours (full-scale, N~2500 waves;). The sea state in each individual experiment is represented by a relevant wave spectrum (JONSWAP spectrum for example), which typically is characterised with a significant wave height, $H_s$, and a peak wave period, $T_p$. The wave height, and the period, is normally increased from experiment to experiment. After each individual experiment, the number of rocks which have moved should be registered. Rocks that move by more than their own diameter are designated to be damage. The

breakwater is not to be repaired between every individual experiment because we normally want to know the accumulated damage.

In cases where a dynamically stable berm breakwater is being tested, the developed profile must be measured between every individual experiment.

Typically, a series of experiments will encompass wave conditions that correspond to recurrence intervals of 25 years and up for the design criteria, which for example corresponds to a 100-year recurrence interval. In addition, an overloading scenario should be tested.

One or more individual experiments are repeated at a low water level in order to assess whether this condition is critical for the foot structure.

### 1.5.7 Criteria for acceptable damage and overtopping

The degree of damage to a breakwater is often specified in a damage percentage, calculated as the number of moved rocks divided by the total number of rocks on a layer, or in a sub-area of a layer. Other damage measurements such a damage level $S_d$ or the damage number Nod can also be used. Overtopping is defined as the average overtopping rate q [l/m/s] per metre of breakwater.

It is absolutely crucial that the acceptable degree of damage (after the design condition has been tested) be defined beforehand by the company and the laboratory. Damage that occurs in an overloading situation should also be addressed. Correspondingly, the acceptable overtopping rate for a given sea state (recurrence interval) must be defined beforehand.

Once these criteria have been determined, it is relatively simple, based on the test results attained, to assess whether the breakwater profile examined is within the limits, or whether there is a need to optimise the design and perform further experiments with a modified version of the breakwater.

### 1.5.8 Quality assurance of experiments

The best manner by which to secure and understand the lab modelling is to be present in the laboratory throughout the entire experimental period. This should either be permanently or during regular visits.

Not only can the presence contribute to detecting misunderstandings before the consequences become serious, but it can also contribute to the time the experiment takes keeping to within the agreed timeframe. In addition, it enables rapid decisions concerning changes in the model design or experimental parameters.

The laboratory ought to report the results and status of the experiment daily. The results from measured wave parameters and observed damages thus ought to be reported as soon as they are performed and quality-assured, or at the latest the day afterwards.

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