

# BEHAVIOUR OF PUMPING PITS IN HARBOUR ENTRANCE ON AN EXPOSED SANDY COAST

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In the summer of 2010 a ferry harbour was opened at the south coast of Iceland connecting the Vestmannaeyjar islands to the mainland. The harbour was constructed on a sandy beach in a dynamic area open to the Atlantic Ocean causing massive longshore sediment transport across the navigational channel to the harbour. Sedimentation in the harbour entrance has turned out to be more than expected causing the harbour to be closed for navigation for large part of the winter time. The volcanic eruption of the Eyjafjalljökull glacier started four months before the opening of the harbour in April 2010. The floods that followed from the river Markarfljot, caused large amount of fine sediments, estimated as 2 million m<sup>3</sup> of sand and 20 million m<sup>3</sup> of mud, to be drifted down through the delta of the river located only 2 km east of the harbour. In the following years the deposition of fine sediments caused navigation problems in the harbour as the necessary navigation depth could not be obtained without heavy dredging. The last two years however the main settlement of sand has been taking place in a limited area between the two breakwater roundheads. In the winter time, it has turned out to be difficult to dredge in the harbour entrance. This is due to the high ambient wave conditions during the winter time where H<sub>s</sub> is larger than 2 m 80% of the time. The limiting wave height for operating dredging vessels in the entrance is less than 2 m. The aim of this study is to assess the possibility of keeping the necessary navigation depth across the harbour entrance by placing a series of static pumps under the seabed in the harbour mouth. Physical model tests were carried out focusing on that problem. First focusing on the effect of a single pump located under the seabed and then on a combined effect of several pumps.

*Keywords: harbour entrance; sedimentation; physical modelling*

## INTRODUCTION

In the summer of 2010 a new ferry harbour was opened at the South coast of Iceland connecting the Vestmannaeyjar islands to the mainland. The harbour was constructed on a sandy beach in a dynamic area open to the Atlantic Ocean. The objective was to shorten the voyage time to and from Vestmannaeyjar and to improve the frequency of transport. The project was divided in two parts to build a ferry harbour on the Bakkafjara coast, named Landeyjahöfn and to build a new ferry instead of a 20 year old existing ferry. The new ferry should be a smaller vessel with lesser draught. The building of the ferry has been postponed for several years and the old ferry is operating the route.

Since the opening of the harbour sedimentation has been causing problems. In this paper, a historical review of the harbour will be given before zooming in on the main topic, behaviour of pumping pits in harbour entrance where model tests were carried out focused on finding a solution of the problem at Landeyjahöfn. The model tests were carried out at the facilities of the Icelandic Road and Coastal Administration (IRCA).

### Research leading up to the construction of the harbour

Preparation for the harbour construction started in 2000 when the Icelandic Maritime Administration (predecessor of IRCA) was handed the task of performing a feasibility study for a ferry harbour on the south coast of Iceland and of a new ferry, in an area known for harsh wave climate.

In the following years, starting in 2002, bathymetric surveys were carried out regularly to follow the bathymetric changes. Historical information on coastline changes was gathered in the area from up to 50 years back as well as wave climate for the same period from the European Centre for Medium-Range Weather Forecasts (ECMWF). Wave buoys were deployed at a location off the coast between the Vestmannaeyjar islands and the mainland, to measure wave height and period, the first one in 2003.

The wave climate was calculated from deep water to the shore using the computer program Mike21 SW from DHI. Based on the gathered information and research the location of the harbour was chosen where the wave conditions were relatively favourable due to the sheltering effects of the Vestmannaeyjar islands. The sand bar outside the coast was in deeper water than further west off the location and the coastline had been relatively stable at least for the last 50 years.

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Figure 1: The sailing route from the Vestmannaeyjar islands to the mainland.

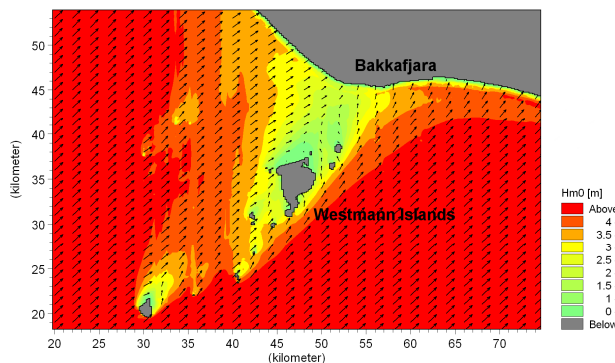


Figure 2: Examples of simulated wave fields in a regional wave model. Waves from SW are sheltered by Vestmannaeyjar

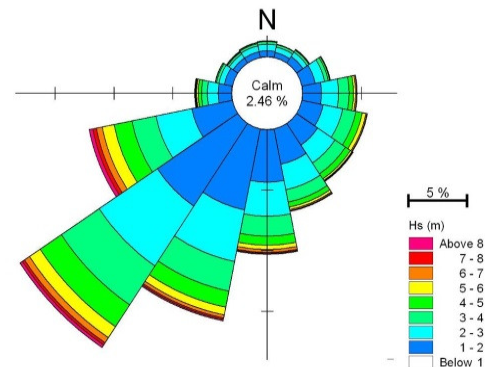


Figure 3: Annual offshore wave statistics, average over period 1979 – 2004.

Hydraulic model tests were carried out to investigate the wave disturbance at Landeyjahöfn ferry harbour and to undertake navigational tests with a remote-controlled model ferry, sailing through the wave breaking zone into the harbour. The main emphasis was to establish criteria for safe navigation of a new ferry. Downtime of the ferry was evaluated based on the hydraulic model test and a wave refraction analysis.

### The harbour layout

The distance between the breakwaters on the shore is 600 m and the breakwaters extend approximately 700 m from the shoreline out to depth of 8 m CD, before construction. The building of a new ferry was delayed due to the bank crisis and in the meantime the existing ferry that requires wider entrance and deeper navigation line is still in use. The entrance width was originally designed as 70 m wide but was increased to 90 m. The critical water depth of 5.5 m CD was defined for the entrance area to ensure that the ferry would be able to navigate safely through the entrance. Approximately 300 m from the roundhead of the breakwaters there is a bar/reef with a water depth from 5 to 8 m CD. Tidal difference at springtide is 2.5 m. From the bar to the harbour there is sometimes a trough of up to -10 m CD and sometimes flat bottom of 6 to 8 m CD. The slope from 40 m depth up to the bar is approximately 1:40 to 1:50.



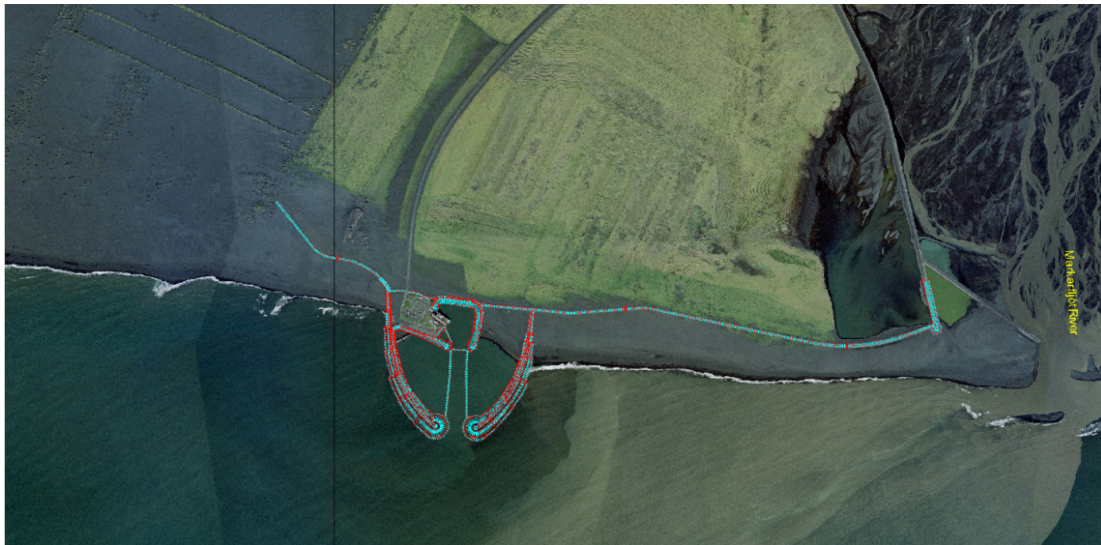


Figure 4: Landeyjahöfn and the surrounding area. The Markarfljót estuary is located about 2 km east of the harbour. An extensive revegetation, the yellow colour, was necessary to decrease eolian sediment transport.

#### Sediment transport and morphology at Landeyjahöfn

Before the construction, the bathymetry outside Landeyjahöfn was characterized by a deep trough up to 10 -12 m CD and a depression in the bar at a water depth of around 5-8 m CD. After the construction the deep trough has filled up once in a while. The proposed location was investigated by detailed analysis of waves, currents, sediments transport and morphological conditions in the coastal water at Bakkafjara in close cooperation with DHI, Denmark.

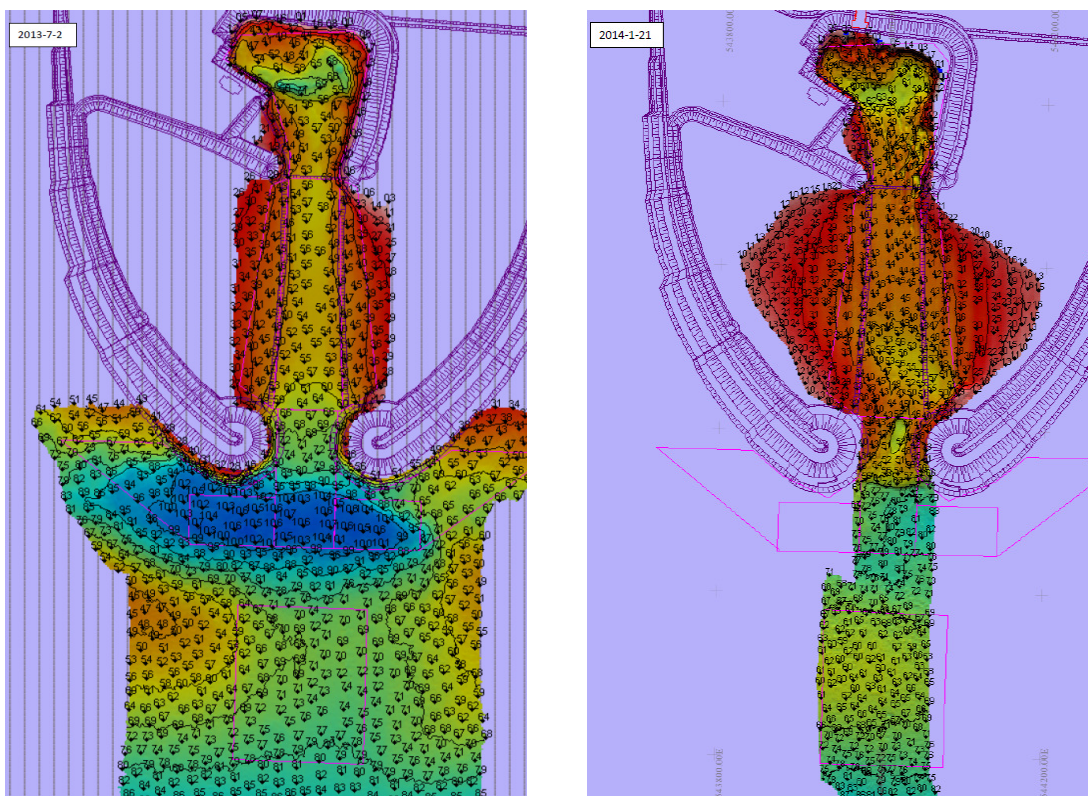


Figure 5: Depth soundings in and in front of Landeyjahöfn harbour from July 2013 and January 2014. Note the depression of the bar in front of the harbour.

The calculated average distributions of net littoral drift, west of the harbour, was estimated 300,000 m<sup>3</sup>/yr east going along the bar and the inner profile. East of the port approximately 2 km, close to the Markarfljót river mouth, the average net littoral drift of 400,000 m<sup>3</sup>/yr was estimated east going, mainly

along the inner part of the profile. This sediment transport pattern is due to the sheltering effect on waves from Vestmannaeyjar as well the different shore orientation and the yearly deposition of sediment from the Markarfljot river.

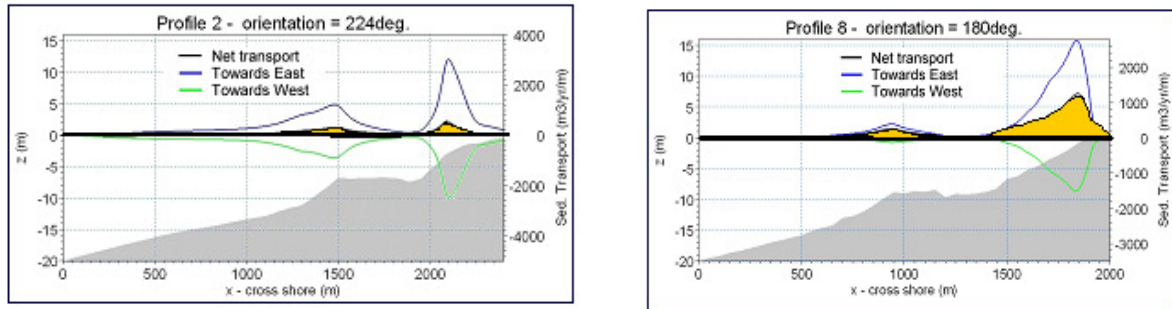


Figure 6: Average littoral drift and the distribution along the coastal profiles at locations west and east of Landeyjahöfn. Note the different scale for sedimentation transport.

### The equilibrium depth on the bar in front of the harbour and in the harbour entrance

The risk of having to dredge the depression through the outer bar was evaluated and found acceptable for the project. It was noted that the required navigational depth at the bar was 6 m CD but the level of the bar has shown to be 5-8 m CD. The rip current will maintain a depression of the outer bar in front of the harbour where the waves are at minimum; however, the dimensions of the depression are a function of the wave height and wave direction. The resulting level of the bar is a continuous battle between the rip current induced erosion and the deposition of sand caused by the long shore current.

Due to the streamline shape and the narrow entrance the majority of the sand will bypass the harbour and it appears that the impact on the adjacent coastline is very small. The equilibrium depth was estimated 5 to 5.5 m CD. After equilibrium is achieved the sedimentation in the harbour entrance will reduce significantly.

The sedimentation of sand into the harbour entrance was estimated as well as sedimentation of fines during the periods where the adjacent river discharges large amounts of fine sediments. The sedimentation was estimated 30,000 m<sup>3</sup>/yr. The annual sediment rate was assumed to change significantly due to the considerable and unusual variability in the wave climate. The sedimentation into the channel was estimated 2,700 m<sup>3</sup> of fine suspended sedimentation.

### AFTER THE CONSTRUCTION AND THE ERUPTION

The volcanic eruption of the Eyjafjallajökull glacier, a volcano that last erupted in 1823, started four months before the opening of the harbour in April 2010. The floods that followed from the Markarfljót river, caused large amount of fine sediments, estimated as 2 million m<sup>3</sup> of sand and 20 million m<sup>3</sup> of mud, to be drifted down through the delta of the river located only 2 km east of the harbour. Westerly waves are more common in this area, dominant 7 out of 8 winters on average. However, in the winter following the volcanic eruption easterly waves were dominant causing large parts of the sediments to drift toward the harbour. In the following months, the deposition of fine sediments caused large problems in the harbour as the necessary navigation depth could not be obtained without heavy dredging. In the opening year the harbour was closed as early as September, with large amount of sediments creating a bar just in front of the harbour entrance, see Fig. 7.

The extended navigational difficulties caused by the volcanic eruption have been decreasing with time and can be considered negligible today. However there are still navigational difficulties in the harbour caused by sedimentation under current conditions in the area and also difficulties caused by the existing ferry, draft of the ferry, shape of the hull and size of the vessel. The navigational limitation of the current ferry in large waves and current environment can be improved by a new ferry designed specifically for those conditions. Another problem concerning the existing ferry is the draught of the ferry, 4.2 m, while it would be desirable to have a ferry with a draught closer to 3 m.

### Sedimentation

Recently the main sedimentation of sand has been taking place in a limited area between the two breakwater roundheads, see Fig. 8. In the winter time, it has turned out to be difficult to dredge in the harbour entrance due to dredger limitation of operating in a heavy weather and large wave height. This



is due to the high ambient wave conditions during that period where  $H_s$  is higher than 2 m in 80% of the time. The limiting wave height for operating dredging vessels in the entrance is close to 2 m.

In wake of the recent development attention has been drawn to local solutions focusing on the area between the breakwater heads. The main topic of this paper is to assess the possibility of keeping the necessary navigation depth by placing a series of static jet pumps under the seabed in the harbour mouth. Physical model tests were carried out focusing on that problem.

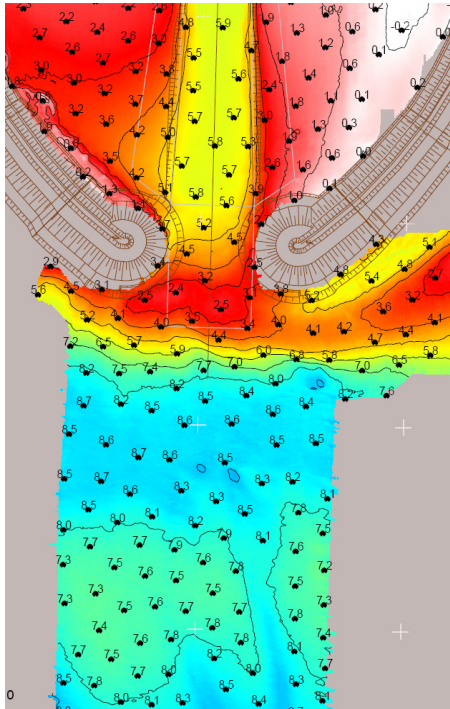


Figure 7 Depth sounding, September 2010. Notice the bar just in front of the harbour entrance and the absence of the outer bar.

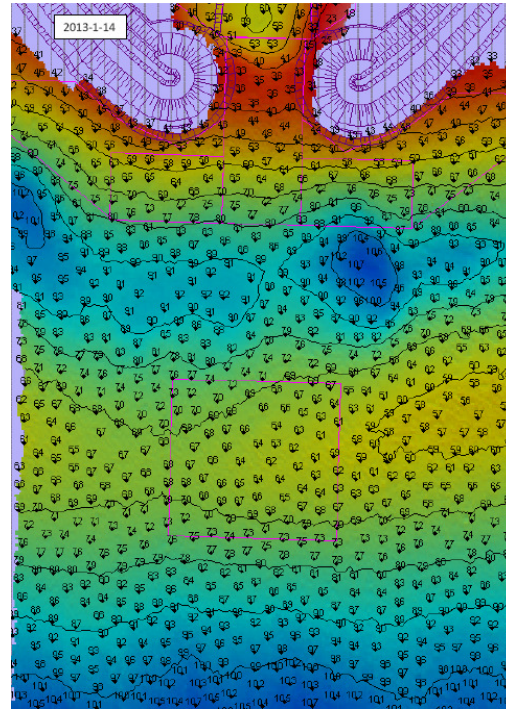


Figure 8 Depth sounding, January 2013. Notice the sedimentation between the roundheads.

**MODEL STUDY - PUMPING SYSTEM**

Physical model tests were carried out focusing on a solution to the recent sedimentation problems between the breakwater roundheads. The focus of the study was to assess the possibility of keeping the necessary navigation depth across the harbour entrance by placing a series of static pumps under the seabed. First focusing on the effect of a single pump located under the seabed and then on a combined effect of several pumps.

**Description of the system**

The function of the system is as follows: suction begins with a certain thickness  $t_p$  of sand above the pump. The jet pump will then quickly remove the sand from above as the sand concentration of the pumping is quite high while there is sand above the pump,  $t_p \geq 0$ . The slope of the pit without wave action depends on the static friction angle of the sediment  $\phi$ , so the radius of the surface circle becomes  $R = t_p / \tan(\phi)$ . The impact of waves is then to move the sand back and forth causing infill of the sand into the pit from the surrounding seabed. In combination with occasional pumping this will cause a general decrease

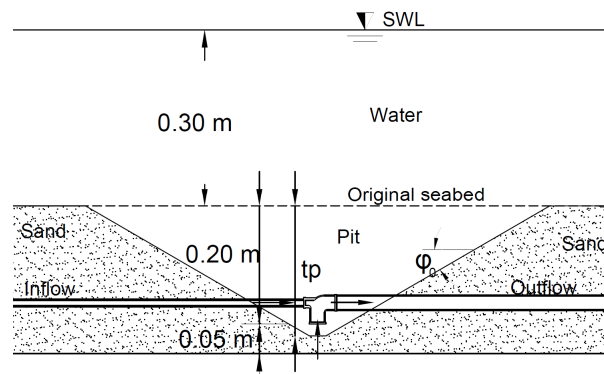


Figure 9 Cross section of the physical model setup of the pumping pit, without wave action. The suction of sand and water mixture is from below.

in bed level at the entrance. For the function of the system it is essential that the depression of the pit will spread laterally so the sufficient water depth in the harbour entrance is not only created locally just above the pumps.

### Model facilities

The experimental investigations were conducted in model facilities of IRCA. The active model area was 5.5 m wide, limited by the width of the wavemaker and 20 m long. In front of the wavemaker there was a 1:20 plywood slope to the sand area that was 5.5 m wide, 6.6 m long and the thickness of the sand layer was 25 cm. Water jet pumps were used in the model with the suction part located 20 cm below the seabed, see Fig 9. On the other side of the active area was a wave absorbing gravel slope. The scale of the model was 1:20 apart from the grain size which was not scaled sufficiently down. Figure 10 shows the setup of the model while figure 9 shows the setup of the pumping pit.

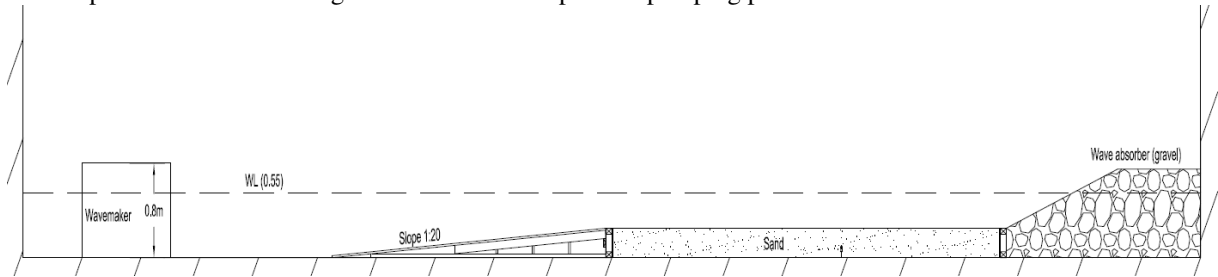


Figure 10 Cross section of the model setup.

### Mechanism of backfilling

The mechanism of backfilling depends on the Shields number, which is a measure of the bed material mobility, the Shields number  $\theta$  is defined by

$$\theta = \tau_b / (\rho g d (s - 1)) \quad (1)$$

Here,  $\tau_b$  is the bed shear stress,  $d$  is the mean grain diameter,  $g$  the acceleration of gravity and  $s$  the relative density of the sand in the bed.

At relatively low  $\theta$ -values, bed load is the dominating transport mode, and in this case, the infill is caused by the simple mechanism, that it is easier for a sand grain to move downhill than uphill, so the sediment becomes trapped when being moved by the waves.

At larger  $\theta$ -values, suspended sediment becomes an increasingly important transport mode, and the settling of suspended sediment should be included. This is far more complicated than the bed-load mode, but in the present case, the Shields number is that low, so we only need to consider the bed load.

The most widely used bed load equation is the Meyer-Peter formula

$$\Phi_B = 8(\theta - 0.047)^{3/2} \quad (2)$$

where

$$\Phi_B = \frac{(1-n)q_B}{\sqrt{(s-1)gd^3}} \quad (3)$$

This value is valid for steady flow, but can also be applied for waves. Most important for this study is, that the gross rate variation of sediment transport with the Shields parameter in waves is approximately proportional to the similar variation for the steady values of  $q_b$ .

#### Calculation of Shields parameter and bed load:

In this section, the estimated transport of sand by waves on the original bed, is outlined. The model test is in scale 1:20, using basalt sand with a density of  $s=2.85$ , and a grain diameter,  $d_{50}=0.175$  mm.

In prototype, the sand has the same density, but usually larger grain size, though with a large scatter. In this calculation example, we apply  $d=0.3$ ,  $0.5$  and  $1.1$  mm respectively. These different grain sizes are chosen due to the large variety in grain size at the site. Regarding the waves in the prototype, we use significant waves of  $H_s=3.0$  m, wave period  $T_p=8$  seconds and a water depth in the harbour entrance equal 6 m.

*Bed friction:* to calculate Shields parameter, it is needed to calculate the bed friction. The friction factor in waves is much larger than that in the current (because the small thickness of the wave boundary layer), which is reflected in the bed shear stresses  $\tau_b$ , responsible for sediment mobility. For the wave alone case we use the formulae (Fredsoe 1984)

$$f_w = 0.04 \left( \frac{a}{k} \right)^{-0.25} \quad (4)$$

for the friction factor where  $k$  is the bed roughness, which is taken as 2.5 times the grain diameter of the bed sediment or 2.5 times 0.175 mm, equal 0.44 mm in the model tests. From knowledge to  $f_w$ , the bed shear stress is given by

$$\tau_b = \frac{1}{2} f_w \rho U_{1m}^2 \quad (5)$$

so the Shields number now can be rewritten to

$$\theta = \frac{1}{2} f_w \rho U_{1m}^2 / ((s-1)gd) \quad (6)$$

By calculating the wave kinematics using linear theory, the following numbers are obtained for Shields parameters and bed load transport in model and prototype.

**Table 1 Calculation Results for the model regarding the bed load transport in steady flow.**

D [m]	Hs [m]	T [s]	a [m]	$U_{1m}$ [m/s]	f [-]	$\theta$ [-]	$q_{br}$ , d=0.175 mm [m <sup>2</sup> /h]
0.3	0.15	1.79	0.11	0.37	0.01	0.21	0.033
0.4	"	"	0.09	0.31	0.0106	0.16	0.019

**Table 2 Prototype, like table 1 for fine bottom sediment, d=0.3 mm.**

D [m]	Hs [m]	T [s]	a [m]	$U_{1m}$ [m/s]	f/ $\theta$ , d=0.3 mm	$q_{br}$ , d=0.3 mm [m <sup>2</sup> /h]
6	3	8	2.13	1.67	0.0055/1.40	1.81
"	"	6	1.42	1.48	0.0061/1.22	1.46
"	"	12	3.46	1.81	0.0049/1.46	1.93
8	"	8	1.75	1.38	0.0058/1.00	1.07
"	"	12	2.93	1.54	0.0051/1.10	1.23

**Table 3 Prototype, like table 1 for coarser bottom sediment, 0.5 and 1.1 mm.**

D [m]	Hs [m]	T [s]	a [m]	$U_{1m}$ [m/s]	f/ $\theta$ , d=0.5 mm	$q_{br}$ , d=0.5 mm [m <sup>2</sup> /h]	f/ $\theta$ , d=1.1 mm	$q_{br}$ , d=1.1 mm [m <sup>2</sup> /h]
6	3	8	2.13	1.67	0.0062/0.96	2.15	0.076/0.53	2.69
8	"	8	1.75	1.38	0.0065/0.69	1.27	0.008/0.38	1.53

**Up-scaling of the results from the physical model test**

The physical model tests were performed in a scale of 1:20, with different (smaller) sediment transport intensity than that in prototype as the sand could not be scaled down sufficiently. The physical model is not distorted so the shape of the pit is assumed to be similar in shape in model and in prototype. This is similar as in other standard tests like scour-around structures investigations etc. The timescale for the development of the pit and the subsequent infill is proportional to the bed load rate.

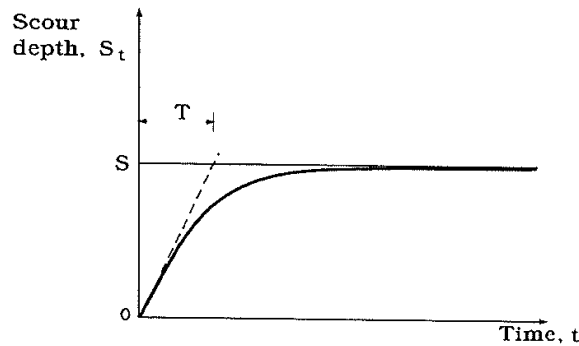
The timescale for the development of the pit, and the subsequent infill, is therefore governed by the following rules:

- The removal of sand is determined by two things: the rate of water pumped in combination with whether the pump is covered by sand or not. Assuming that pumping only takes place when the pumps are sand-covered, measured volumetric sand concentration of pumped mixture was 20% in the model, similar concentration is expected in prototype.
- The infill of sand into the pit is proportional to the bed load rate  $q_b$ .
- The infill of sand into the pit is proportional to the periphery of the pit, i.e. proportional to the length scale.
- The volume of the pit is proportional to the area of the pit (length scale squared) multiplied by depth (length scale).

From this we get that the timescale for infilling up to a certain fraction of the total pit volume in prototype versus physical model test is given by

$$\frac{T_p}{T_m} = \frac{q_{b,m}}{q_{b,p}} \lambda^3 = \frac{q_{b,m}}{q_{b,p}} \lambda^2 \quad (7)$$

in which index p and m stands for prototype and model, respectively, and  $\lambda$  is equal to 20 (the model scale). The timescale is a typical measure of how far a process is reached on its way to equilibrium. In our case, the equilibrium is a sea bed with a totally backfilled pit (i.e. a plane bed), while at  $t=T$ , only around 80% of the total volume has been backfilled, see Fig. 11.



**Figure 11 Definition of time scale: here S is the final depth,  $S_t$  is the instantaneous scour depth**

*Numerical numbers regarding backfilling:*

1. Fine sand, 0.3 mm sand in nature: in this case, tables 1 and 2 suggest that the sand transport rate in nature is 55 times larger in nature than in the model, so the timescale in this case will be around 7-8 times larger in nature than in model, cf. eq. 5 with  $\lambda = 20$ .
2. Medium sand, 0.5 mm sand in nature: in this case, table 3 suggest that the transport rate in nature is 65 times larger in nature than in the model, so the timescale in this case will be around 6 times larger in nature than in model.
3. Coarse sand, 1.1 mm sand in nature: now  $q_{b,m}/q_{b,p}$  decreases to 1/80, and the time scale ratio becomes 5.

In nature, the sediment consists of a mixture of different sand sizes, so the best estimate is that the timescale for backfilling the prototype pit with a certain percentage of the total pit volume will be 6-7 times longer than the similar backfill in the model.

**Infill rates in the model**

The infill rate in the model was found from runs without pumping. Figure 12 shows such run where the infill was measured with small time variation. It is seen that that backfilling initially is very quick, when looking at elevation simply because the volume of the central cone just above the pump has a very small volume. Only at larger times, the variation with time looks similar to the usually observed timescales. This run was conducted when the sand level was slightly higher than in the general setup



(0.23 cm above pump instead of 0.2 m). Therefore the time scale for backfilling for wave conditions in prototype ( $H_s = 3$  m,  $D = 6$  m) should be 7-8 times longer than in the model.

The time of backfilling can be estimated for different conditions, wave conditions and tidal variation. Large part of the data needed, apart from accurate grain size and water depth, is available from real-time measurements as well as rather accurate wave and tidal predictions, therefore the infill of the pits could be estimated and even predicted rather accurately few days ahead of time. For economical use of the pumping system cumulative infill of the pit can be estimated and the pumps be turned on when the pits are 50-70% full and turned off again when the concentration of the output slurry drops significantly.

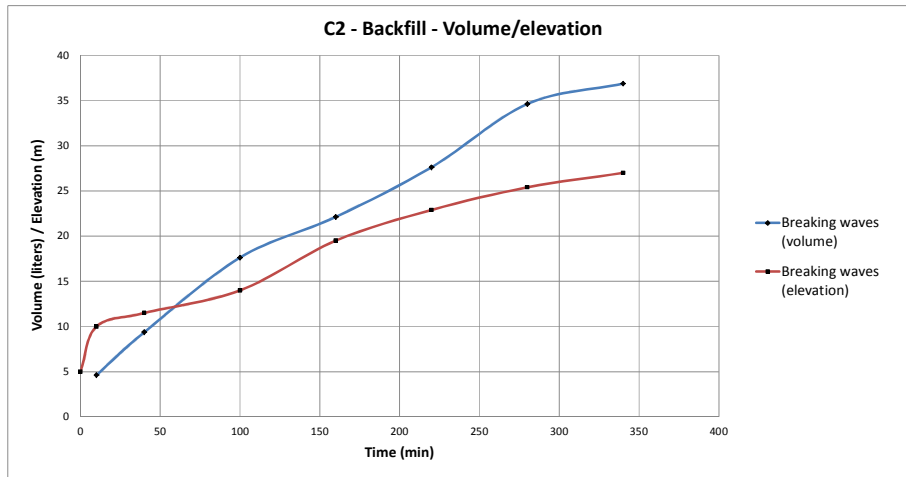


Figure 12 measured backfilling of the pit above the pump.

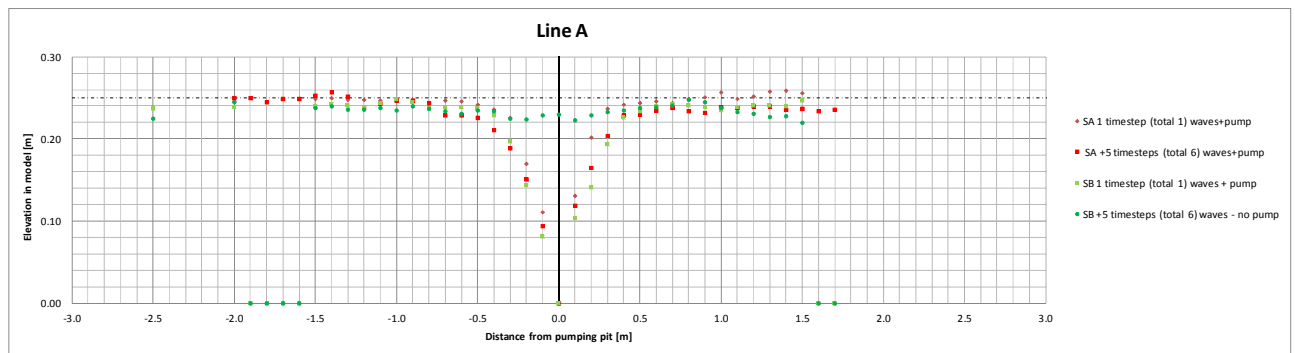


Figure 13 Measured development of the pit with time Series C: After 1 time step, the pump was turned off.

**Horizontal Extension of the pit, and its impact on number of pumps**

The results of the model tests with one pump in the centre of the sand bed indicate that the pit will spread laterally and in the wave propagation direction with time. Parts of the decrease in bed level reflect a general decrease in bed level due to general erosion of the sand bed in the facility. However, figures 14 and 15, suggests that a local lowering of the sea bed is at least 0.5 m away from the pump can be observed from the model test with a single pump, corresponding to an impact radius around each pump equal 10 m in prototype. This will correspond to at least 4 pumps across the harbour entrance to maintain a sufficiently wide navigation channel. However the measured profiles showed indication that the lowered area reaches much further away from the pump, although the general lowering is lesser further away from the centre of the pit.

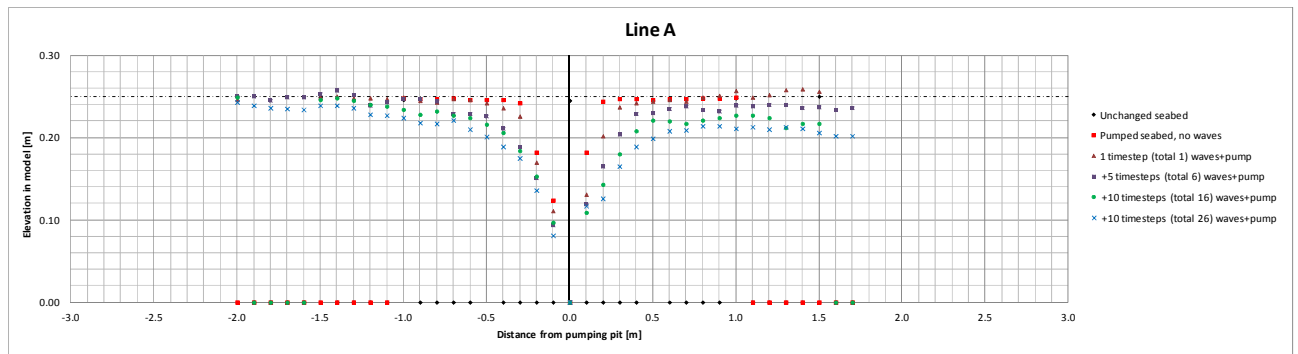


Figure 14 Measured development of the pit with time, waves and pumping simultaneously, line A is a line across the entrance.

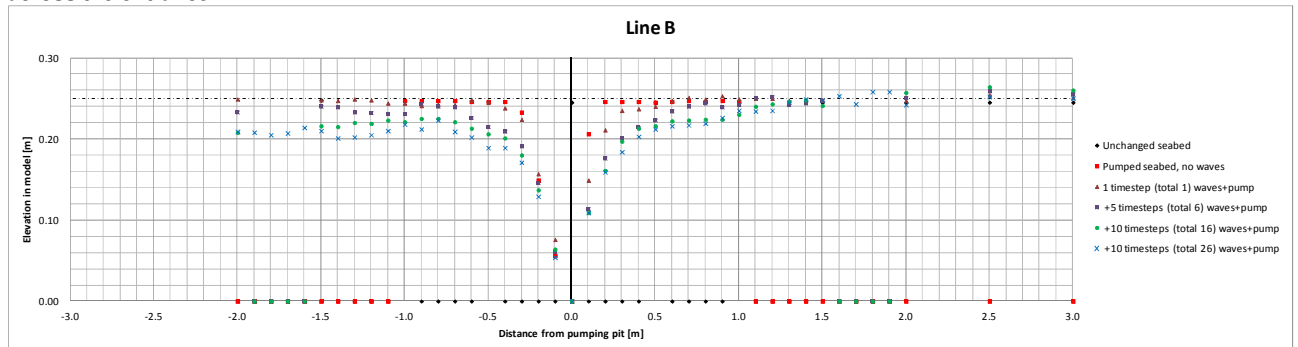


Figure 15 Measured development of the pit with time, waves and pumping simultaneously, line B is parallel to the direction of wave propagation. It is seen that the pit has nearly same dimension in both directions.

In order to study the effect of a system of several pumps on the seabed a system of four pumps was tested in the model, P1 to P4, see Fig 16. The spacing of the two rows of two pumps in the model was chosen to ensure that, if the model results gave sufficiently coherent lowered area between the pumps, it could be applied for the entrance at Landeyjahöfn. Across the entrance the spacing was chosen wide enough to have two pumps across in prototype without being too close to the breakwater roundheads. In the wave direction the spacing was chosen so two pumps in the wave direction could keep the main sedimentation area between the breakwater heads clear, see Fig 8. The chosen spacing in the model was 1.75 m across the entrance and 1.25 m in the wave direction. Profiles were measured in the directions parallel and transverse to the wave direction, across the pumps as well as between the pumps, see Fig 16.

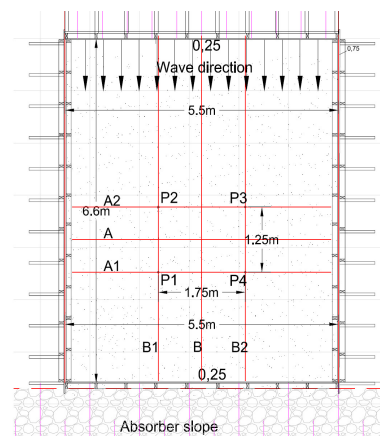


Figure 16 Sand area of the model with a system of four pumps (P1 to P4).

Figures 17 a and b show photos of the lowered seabed as the water is being drained from the model after completion of test with four pumps, graphs of the measured profiles can be observed from the appendix. It is seen, that multiple pumps create a coherent lowered area in between the pumps, the lowering being more than from what could be expected from superposition of the lowering from the individual pumps. The lowering of the seabed between the pumps in the model is 2-3 cm compared to the results of tests with similar setup with waves and no pumps. Should that be considered acceptable lowering in prototype (0.4-0.6 m), two pumps across the entrance are recommended. However adding the third pump across the entrance would give increased lowering of the seabed in the navigation line.

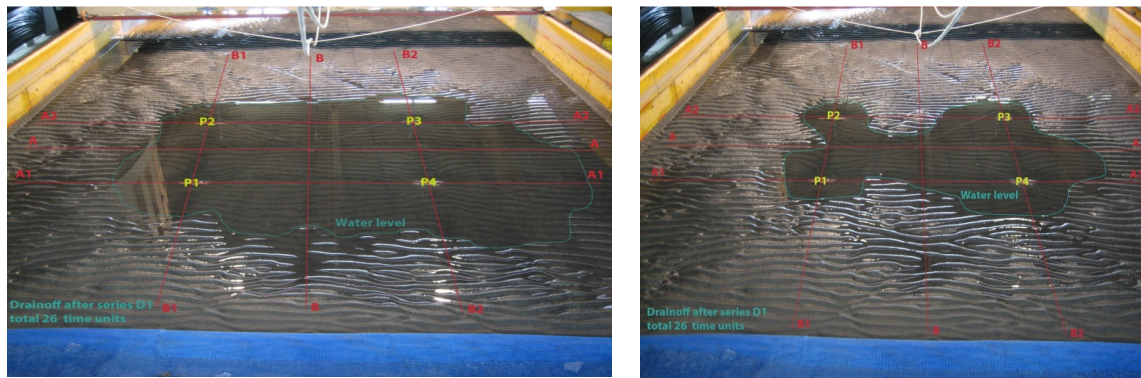


Figure 17 a and b. Test with 4 pumps (P1 to P4), spaced 1.25 m in wave direction, and 1.75 in the transverse direction.

### Conclusions and discussion of results

The main aim of this study was to assess the possibility of keeping the necessary navigation depth across the harbour entrance at Landeyjahöfn harbour, by placing a series of static pumps under the seabed in the harbour mouth. Physical model tests were carried out focusing on that problem. Horizontal extension of the pumping pits is essential for the system to work properly and the model tests showed promising results in that direction. In a case of four pumps with optimum distance between them the results show that the combined effect of the pumping system exceeds the effect of four individual pumps. For the case of Landeyjahöfn it has to be decided if the combination of four pumps, two rows of two pumps, is considered giving sufficient coherent lowering of the seabed or if an extra pump should be added in each row across the entrance resulting in two rows of three pumps.

The process of pumping away sand is a faster than the backfilling, how much faster depends on the wave climate and the capacity of pumps. Applying pumps with a capacity of 1000 m<sup>3</sup>/hour, the removal of sand is 4-6 times larger than the backfilling rate for wave height of  $H_s=3$  m. Therefore smaller pumps might eventually be used. Larger number of pumps, each with a smaller capacity, but placed with a larger density is also an option in order to get a more evenly decrease in the seabed.

From the gathered data the rate of backfilling can be estimated for different wave conditions and tidal variation. Large part of the data needed for Landeyjahöfn, apart from accurate grain size, is available from real-time measurements as well as rather accurate wave and tidal predictions, therefore the infill of the pits could be estimated and even predicted rather accurately few days ahead of time. For economical use of the pumping system cumulative infill of the pit can be estimated and the pumps operated when the pits are 50-70% full and turned off when the concentration of the output slurry drops significantly.

The conditions in the entrance of Landeyjahöfn harbour are still improving slowly with time. Therefore the decision on the future of this project has been put on hold. It is expected that the installation of the pumps is rather expensive and such system would have to be designed with a possibility of maintenance. The risk of clogging the jet pumps is also high and would need to be taken into account in the design phases of the system.

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APPENDIX

The following graphs show measured development of the model seabed with four pumps, spacing 1.75 across the entrance and 1.25 m in the wave direction:

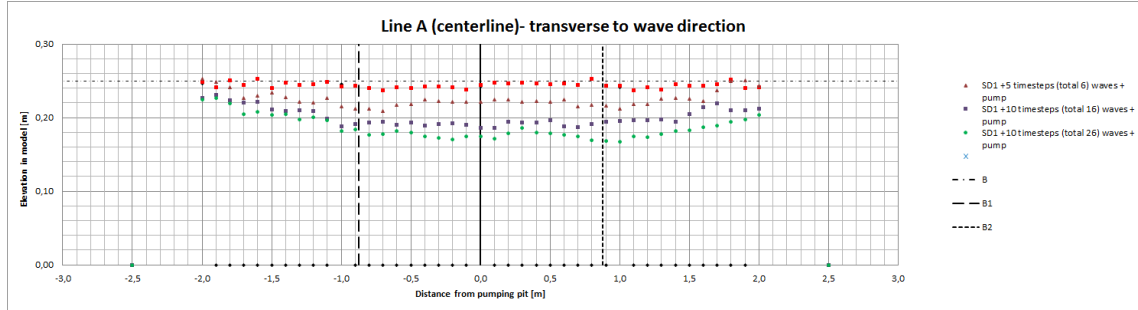


Figure 18 Measured development of the pit with time, waves and pumping simultaneously, line A is between the rows of pumps transverse to wave direction.

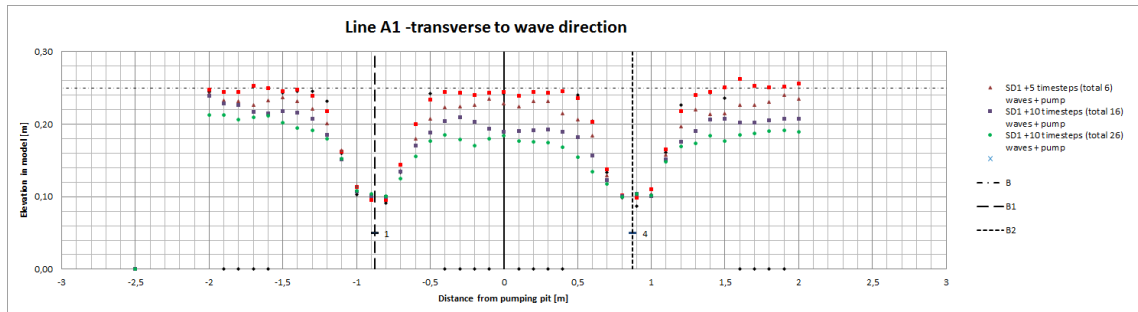


Figure 19 Measured development of the pit with time, waves and pumping simultaneously, line A1 is a section through pumps 1 and 4 transverse to wave direction.

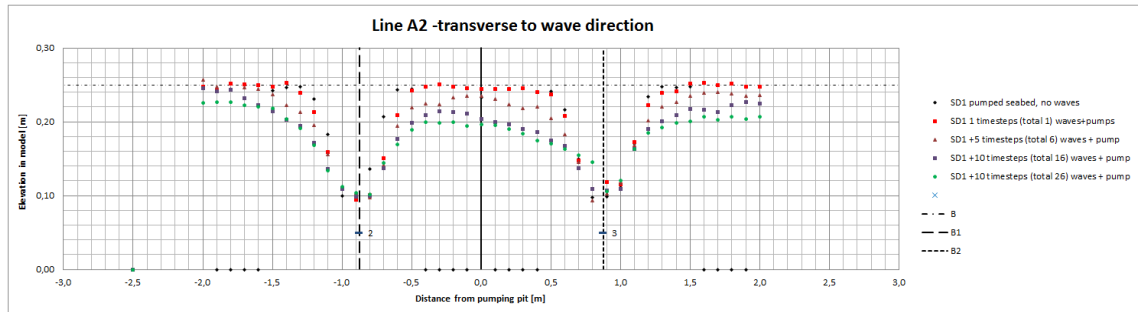


Figure 20 Measured development of the pit with time, waves and pumping simultaneously, line A2 is a section through pumps 2 and 3 transverse to wave direction.

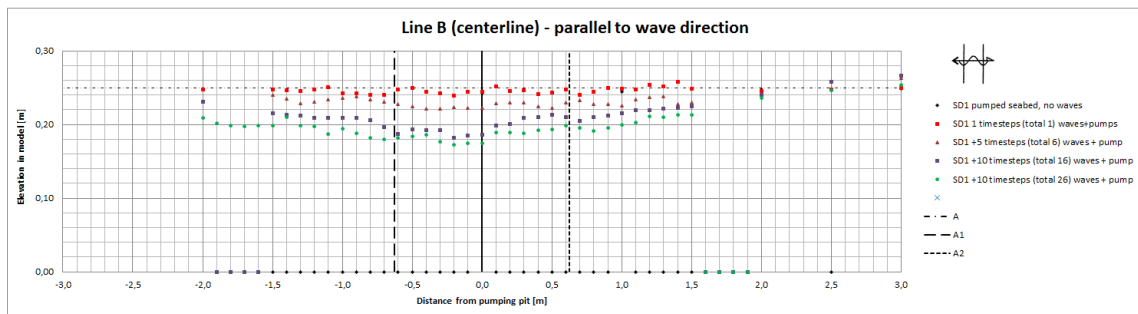


Figure 21 Measured development of the pit with time, waves and pumping simultaneously, line A is between pumps parallel to wave direction.



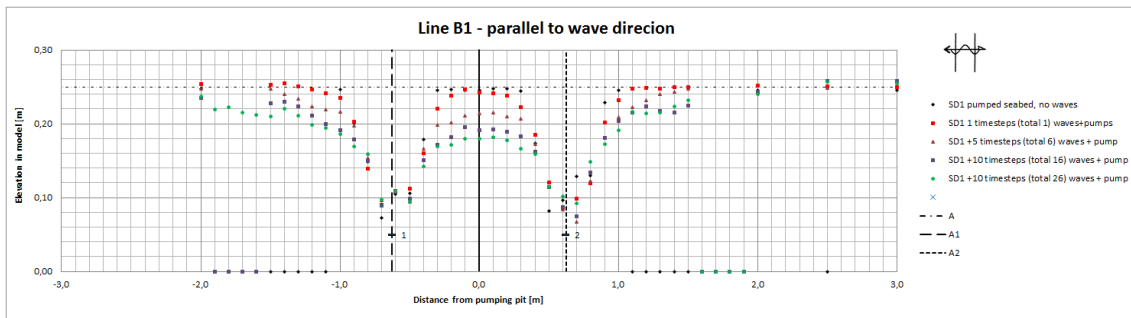


Figure 22 Measured development of the pit with time, waves and pumping simultaneously, line B1 is a section through pumps 1 and 2 parallel to wave direction.

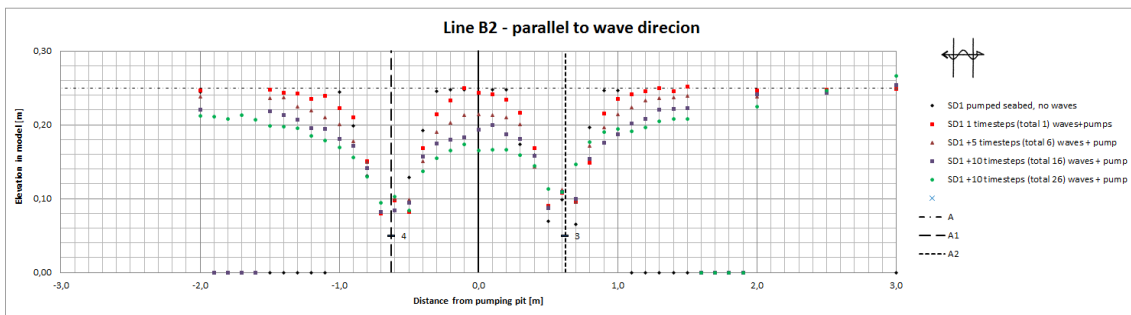


Figure 23 Measured development of the pit with time, waves and pumping simultaneously, line B2 is a section through pumps 4 and 3 parallel to wave direction.