Coastal protection performance of the SIC Pressure Equalizing Modules.

Results of the three years field test at the southern Holmsland Barrier on the Danish North Sea Coast

by
Hans F. Burcharth
Prof. dr. techn, dr. h.c.

Department of Civil Engineering
Aalborg University
Denmark

2 June 2008

Foreword

The author of this report has been involved in the testing of the SIC-Pressure Equalizing Modules for many years after a request from Mr. Poul Jacobsen, SIC. Although it was indeed not from a physical point obvious why such a drain system should have a significant effect, I recommended that the system was tried out in a field study, simply because it was a very inexpensive and harmless method and therefore had a large potential if it worked. My technical reasoning was that if the system could change – even a little – the delicate balance between sand settlement in wave up-rush and the erosion in wave down-rush then the effect could be positive. This pragmatic approach of trial and error was not understood among international coastal engineering specialists who regarded two system worthless.

The former tests, based on very limited financial support ended to my opinion inconclusive with respect to the efficiency of the PEM system.

The financial support of the present field test made it possible to go more in depth with the function of the system.

Front: Photo of SIC-Pressure Equalizing Modules at Skodbjerge (Hans F. Burcharth).

Coastal protection performance of the SIC Pressure Equalizing Modules.
Results of the three years field test at the Southern Holmsland Barrier on the Danish North Sea Coast

by
Hans F. Burcharth
Prof. dr. techn, dr. h.c.
Aalborg University
Denmark

15 May 2008

1. Introduction

1.1. Former investigations

Skagen Innovation Center (SIC) has patented a method for coastal protection by means of spatially distributed vertical perforated drain pipes placed in the beach. The drain pipes are denoted Pressure Equalizing Modules (PEMs) by SIC.

The author of this report participated in planning and evaluation of two earlier tests with PEMs. A five years test (1999-2004) at Skagen Klitplantage is reported in a number of reports, the latest one being Burcharth, 2005. A three years test (2000-2003) at Skallerup Klit is reported in Burcharth, 2004. Both locations are sandy beaches facing the North Sea in the North of Jutland.

The two studies were solely based on comparative analyses of surveyed beach profiles from stretches with and without PEMs. No financial support for more detailed investigations of the function of PEMs was available. Although some signs of a positive effect of the PEMs was observed, it was not possible for the author to present a definitive conclusion because the natural variations of the beaches due to erosion and accretion of sand are very large and hide the effects of any manmade interventions unless these have significant effects. Moreover, the author could not find a physical explanation pointing to a significant effect of the PEM system.

1.2 The present investigation

In accordance with the agreement of 18 August 2004 between Skagen Innovation Center (SIC) and the Danish Governmental Coastal Authority (KDI) a field test with the purpose of demonstrating the efficiency of the SIC vertical drain method as a means for coastal protecting was initiated in a meeting 24 August 2004.
In the agreement is stated that the test period is three years after which a final report has to be presented. The report shall contain an evaluation of the drain system with respect to qualitative and quantitative efficiency and environmental impact, as well as a related comparison with conventional coastal protection methods.
Besides the final report yearly reports have to be presented as well as a report half a year after the start of the field test.

With the availability of approximately 1 mill EURO for the test it was possible, besides surveying of the beach and shoreface, to make more detailed investigations of the function of the PEMs.

For the evaluation the following two experts were retained

Prof.dr.techn., dr. h.c. Hans Falk Burcharth (HFB), appointed by SIC
Prof.dr.techn. Jørgen Fredsøe (JF), appointed by KDI. Participated since 29 October, 2004

The two experts were obliged to take part in the planning of the field tests including selection of the test location.
Besides the two experts the project group consists of

Director, engineer Poul Jakobsen, SIC
Engineer Claus Brögger, SIC
Project manager to 27th March 2006, M.Sc. Christian Laustrup, Senior Consultant, KDI
Project manager from 28th March 2006, M.Sc. Per Sørensen, Head of coastal research KDI
M. Sc. John Jensen, KDI

The present report, authored by Hans Falk Burcharth, is the final report, written as a stand-alone report for which reason it repeats parts of the earlier project report.

Figures and tables marked with KDI as origin have been defined and requested by the author.

The overall conclusions are presented in Chapter 12.

This report cannot be used in citations without the permission of the author.

2. The PEM system

The PEMs used in the present tests are 2 m long circular 63 mm diameter pipes. The upper 0.75 m is metal and the lower 1.25 m is plastic with 0.2 mm slots in the 1 m lowest part. The slots allow water to flow in and out of the pipe, but avoid penetration of sandy beach materials, see photo Fig. 1. The bottom end of the pipe is closed. The top end is semi closed allowing only air and water to go through.

Fig. 1. Photo of PEM pipe (KDI).

The pipes are placed vertically in digged or drilled holes with top level approximately 0.3 m below the beach surface at the time of placing, see Fig. 2. In the present tests the holes were drilled, see Fig. 3. The drilled hole has a diameter of app. 15 cm. The space around the 6 cm diameter pipe (30 cm^2 cross sectional area) is filled with sand. This implies that possible impermeable or less impermeable layers down to app. 2.3 m under the sand surface are penetrated and a 680 cm^2 sand drain established around the pipe.

The pipes are positioned with a distance of 10 m in between them in rows across the beach. The distance between the rows is 100 m, see Fig. 4. The average spatial density is then 1 pipe per 1000 m^2, indicating the low cost of the system. More pipes can be added if the beach gets wider.

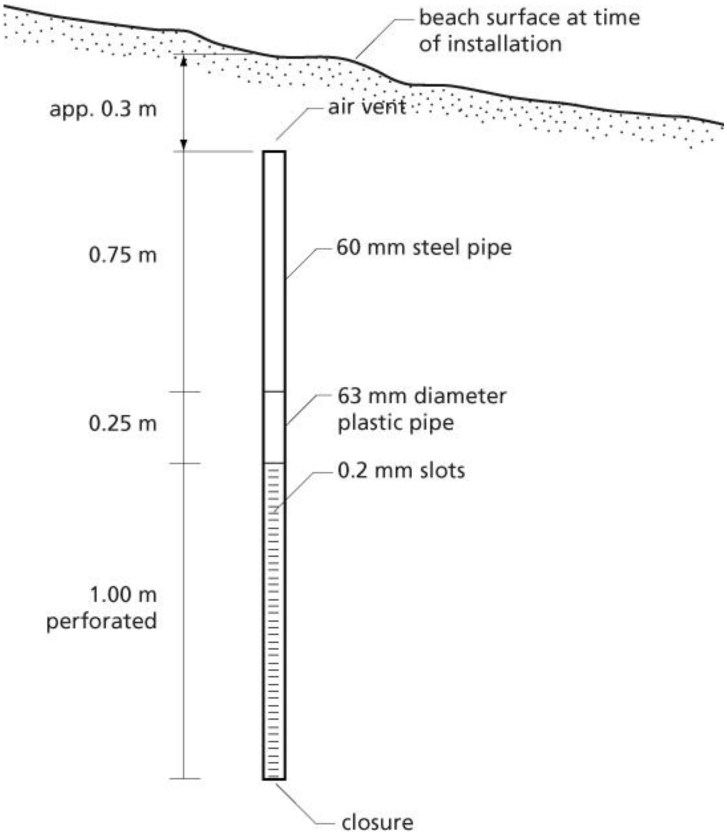


Fig. 2. Installed PEM.

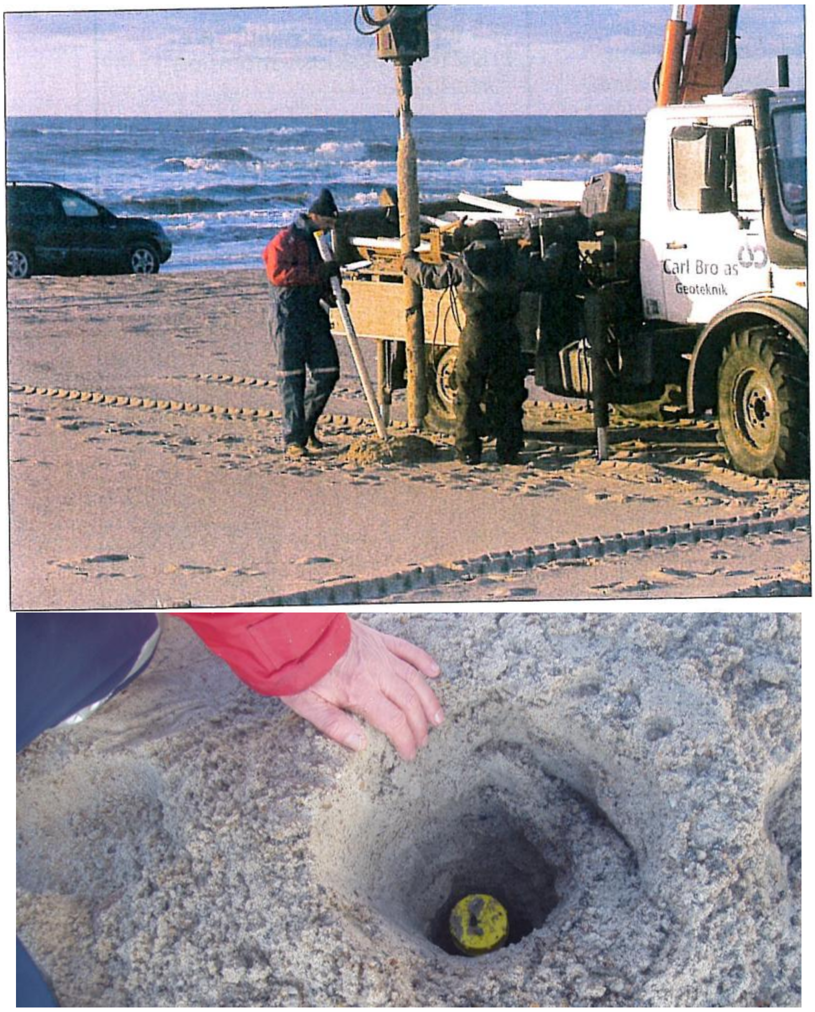


Fig. 3. Drilling of holes for placement of PEMs. (Photo: Poul Jacobsen).

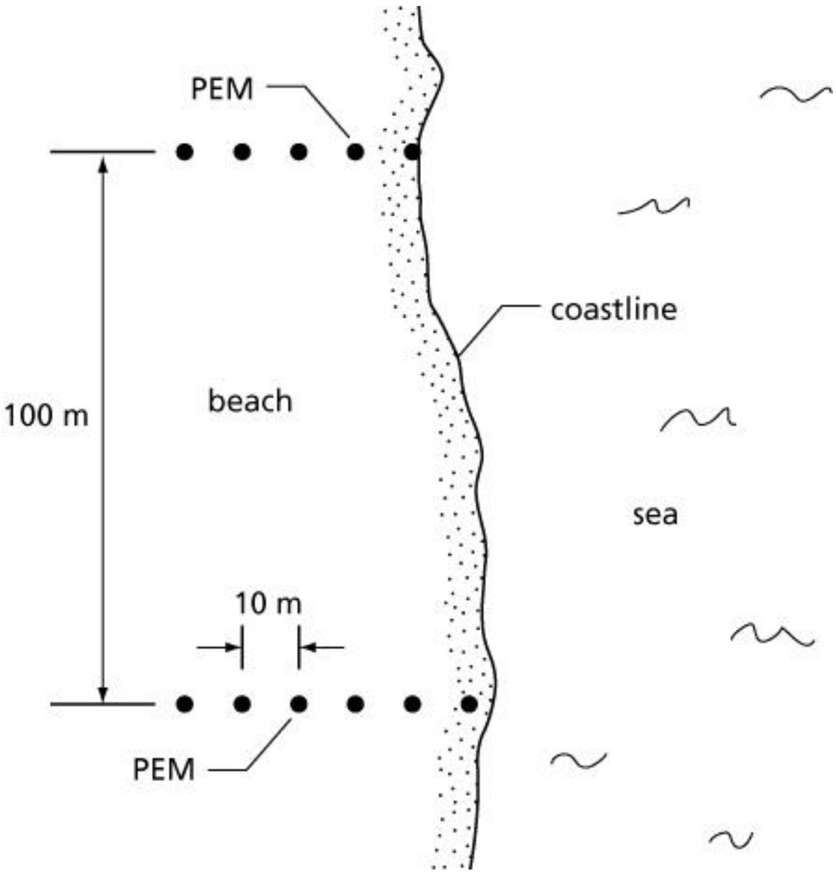


Fig. 4. Lay-out of PEM scheme

3. Selection of test site

According to agreement between SIC and the Ministry of Transport a stretch of approximately 10 km on the Danish North Sea Coast should be selected for the tests.
Conditions with respect to hydrographic and geomorphological conditions should be as homogenous as possible along the stretch. Moreover, influence of man-made interventions should be as small as possible.
Only two potential sites could be found by KDI: A 15 km long stretch at Skodbjerge just south of the port of Hvide Sande, and a 7 km long stretch at Skallingen north of the town of Esbjerg.
The net-sediment transport is southwards at both sites, but much larger at the Skodbjerge site. The Hvide Sande jetties north of the Skodbjerge site create leeside erosion for which reason some beach parallel detached rock breakwaters are placed just south of the jetties. This coastal protection has been supplemented with beach nourishment and nourishment at the offshore bar approximately 600 m from the shore, cf. Fig. 5. Erosion decreases to the south so that just south of the 15 km test stretch the beach is stable. Accretion takes place further south. Beach nourishment would, according to KDI, not take place in the three years test period, but nourishment at the offshore bar would continue.

KDI and JF were in favour of inspecting and most probably selecting the Skallingen site as it seems more homogeneous, and no nourishment takes place. Erosion takes place over the full length of the actual stretch of coast, mainly because former supply of sediments from Blåvands Huk has more or less terminated. A groin system at the northern boundary of the Skallingen site might contribute to the coastline retreat due to lee side erosion. SIC argued that the length was too short as a 10 km stretch was needed. Moreover, SIC regarded the influence of the groins to be to disturbing for the tests. As SIC refused to use Skallingen it was decided in a meeting 16.12.04 to use the Skodbjerge site, despite the not ideal conditions because of the bar nourishment. SIC claimed however that the bar nourishment would have no or marginal influence on the test results.

It should be noted that in 2004, i.e. the year in which the project started, there was placed 600.000 m^3 of sand on the bar, so even if the bar nourishment was stopped, the already placed large amount would have some – but unknown – effect on the coast.

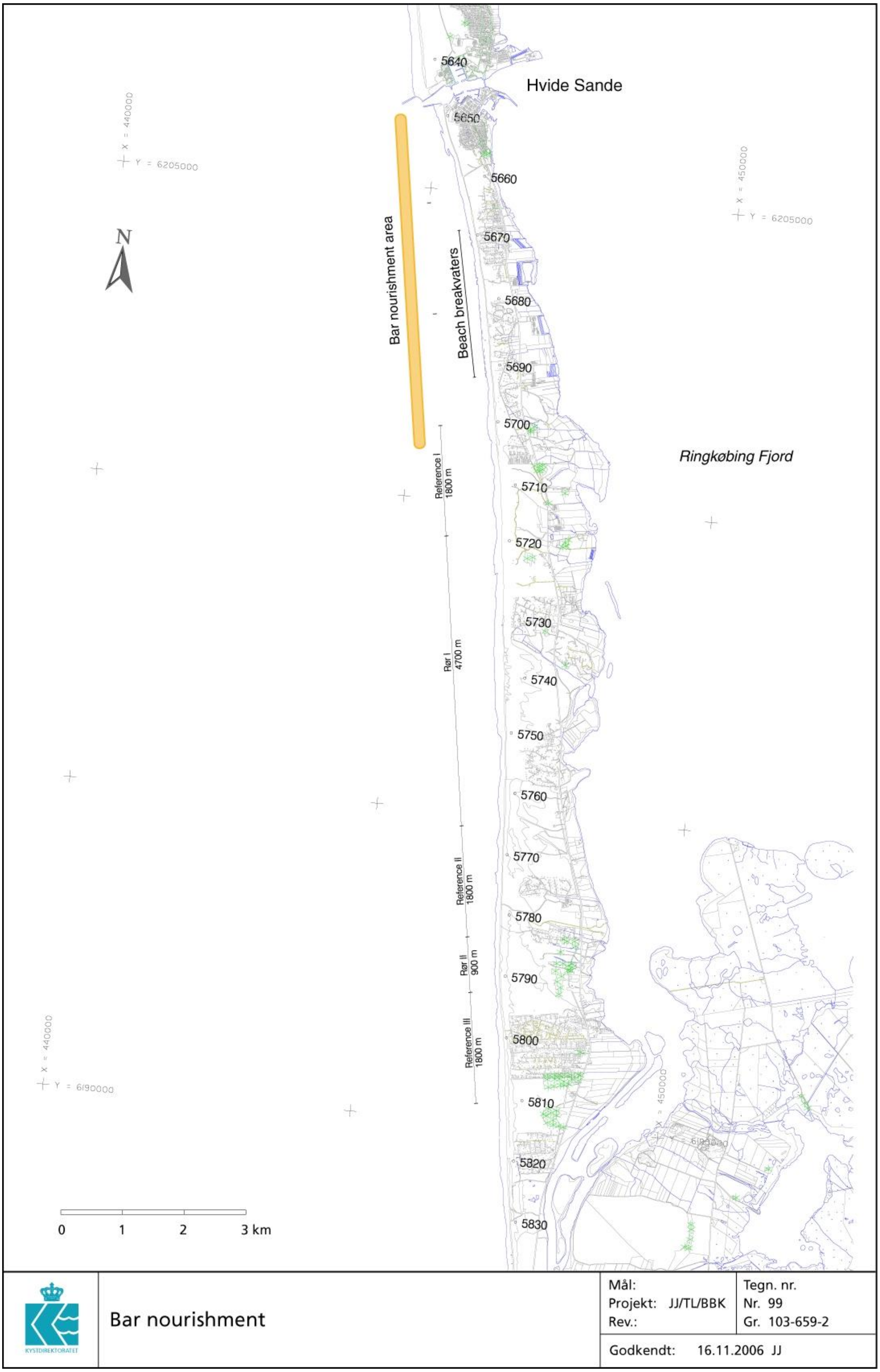


Fig. 5. Position of the bar nourishment 2004-2007.

4. Planning of the test and installation of PEMs

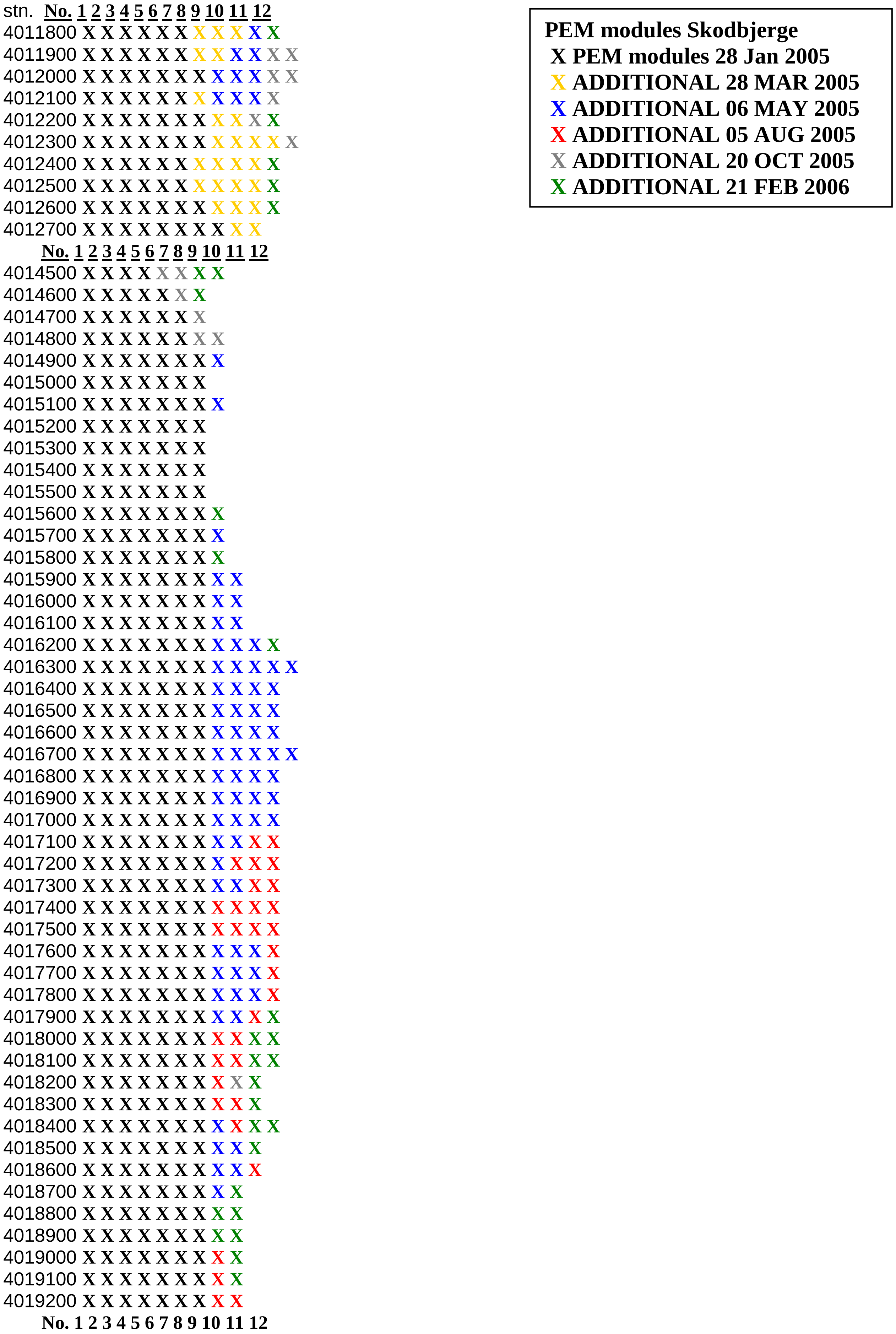
The basis for the evaluation of the tests is a comparison of the morphological changes in stretches with and without drain pipes as well as more detailed investigations and calculations related to the function of the drains.

The total length of the Skodbjerge test site, chosen in meeting 16.12.04, was limited to approximately 11 km in order not to come too close to the beach breakwaters to the north and the accreting coast to the south.
KDI preferred a split of the site in a number of relatively short stretches (say 2 km) with alternating drains and no drains. SIC could not accept this as – based on experience – they wanted longer stretches, basically a 6 km stretch with drains and a 4 km stretch without drains. However, due to the gradient in erosion along the test site this was not acceptable, and HFB proposed as a minimum stretches with no pipes on both sites of the drained stretch.
JF proposed to include an approximately 1 km stretch with drains in the south part of the test site. This proposal was accepted. The final division of the test site in stretches with and without drains as decided in a meeting 16.12.04 is shown in Fig. 6. From this it is seen that two stretches of 4.7 km (Rør I, chainage 4019200 - 4014500) and 0.9 km (Rør II, chainage 4012700 - 4011800) respectively were drained, and three stretches of 1.8 km (Reference I, chainage 4021000 – 4019200), 1.8 km (Reference II, chainage 4014500 – 4012700) and 1.8 km (Reference III, chainage 4011800 – 401000) respectively were left undrained.

The drains were installed in January 2005. The positions and number of the drains and time of installations are shown in Tables 1A and 1B. As seen from the tables, drains have been added in some areas where increase in beach width made it possible, and drains have been eroded after February 2006.

Fig. 6. Location of test stretches.

Table 1A. Positions and number of drains placed.



stn. No.	1	2	3	4	5	6	7	8	9	10	11	12
4011800	X	X	X	X	X	X	X	X	X	X	X	
4011900	X	X	X	X	X	X	X	X	X	X	X	X
4012000	X	X	X	X	X	X	X	X	X	X	X	X
4012100	X	X	X	X	X	X	X	X	X	X	X	
4012200	X	X	X	X	X	X	X	X	X	X	X	
4012300	X	X	X	X	X	X	X	X	X	X	X	X
4012400	X	X	X	X	X	X	X	X	X	X	X	
4012500	X	X	X	X	X	X	X	X	X	X	X	
4012600	X	X	X	X	X	X	X	X	X	X	X	
4012700	X	X	X	X	X	X	X	X	X	X		
No.	1	2	3	4	5	6	7	8	9	10	11	12
4014500	X	X	X	X	X	X	X	X				
4014600	X	X	X	X	X	X	X					
4014700	X	X	X	X	X	X	X					
4014800	X	X	X	X	X	X	X	X				
4014900	X	X	X	X	X	X	X	X				
4015000	X	X	X	X	X	X	X					
4015100	X	X	X	X	X	X	X	X				
4015200	X	X	X	X	X	X	X					
4015300	X	X	X	X	X	X	X					
4015400	X	X	X	X	X	X	X					
4015500	X	X	X	X	X	X	X					
4015600	X	X	X	X	X	X	X	X				
4015700	X	X	X	X	X	X	X	X				
4015800	X	X	X	X	X	X	X	X				
4015900	X	X	X	X	X	X	X	X	X			
4016000	X	X	X	X	X	X	X	X	X			
4016100	X	X	X	X	X	X	X	X	X			
4016200	X	X	X	X	X	X	X	X	X	X	X	
4016300	X	X	X	X	X	X	X	X	X	X	X	X
4016400	X	X	X	X	X	X	X	X	X	X	X	
4016500	X	X	X	X	X	X	X	X	X	X	X	
4016600	X	X	X	X	X	X	X	X	X	X	X	
4016700	X	X	X	X	X	X	X	X	X	X	X	X
4016800	X	X	X	X	X	X	X	X	X	X	X	
4016900	X	X	X	X	X	X	X	X	X	X	X	
4017000	X	X	X	X	X	X	X	X	X	X	X	
4017100	X	X	X	X	X	X	X	X	X	X	X	
4017200	X	X	X	X	X	X	X	X	X	X	X	
4017300	X	X	X	X	X	X	X	X	X	X	X	
4017400	X	X	X	X	X	X	X	X	X	X	X	
4017500	X	X	X	X	X	X	X	X	X	X	X	
4017600	X	X	X	X	X	X	X	X	X	X	X	
4017700	X	X	X	X	X	X	X	X	X	X	X	
4017800	X	X	X	X	X	X	X	X	X	X	X	
4017900	X	X	X	X	X	X	X	X	X	X	X	
4018000	X	X	X	X	X	X	X	X	X	X	X	
4018100	X	X	X	X	X	X	X	X	X	X	X	
4018200	X	X	X	X	X	X	X	X	X	X		
4018300	X	X	X	X	X	X	X	X	X	X		
4018400	X	X	X	X	X	X	X	X	X	X	X	
4018500	X	X	X	X	X	X	X	X	X	X		
4018600	X	X	X	X	X	X	X	X	X	X		
4018700	X	X	X	X	X	X	X	X	X			
4018800	X	X	X	X	X	X	X	X	X			
4018900	X	X	X	X	X	X	X	X	X			
4019000	X	X	X	X	X	X	X	X	X			
4019100	X	X	X	X	X	X	X	X	X			
4019200	X	X	X	X	X	X	X	X	X			
No.	1	2	3	4	5	6	7	8	9	10	11	12

PEM modules Skodbjerge
X PEM modules 28 Jan 2005
X ADDITIONAL 28 MAR 2005
X ADDITIONAL 06 MAY 2005
X ADDITIONAL 05 AUG 2005
X ADDITIONAL 20 OCT 2005
X ADDITIONAL 21 FEB 2006

Table 1B. Drains per 6 June 2007.

PEM modules Skodbjerge
X PEM modules 6 June 2007

stn. No.	1	2	3	4	5	6	7	8	9	10	11	12
4011800	X	X	X	X	X	X	X	X	X	X	X	
4011900	X	X	X	X	X	X	X	X	X	X	X	
4012000	X	X	X	X	X	X	X	X	X	X	X	
4012100	X	X	X	X	X	X	X	X	X	X	X	
4012200	X	X	X	X	X	X	X	X	X	X		
4012300	X	X	X	X	X	X	X	X	X	X		
4012400	X	X	X	X	X	X	X	X	X			
4012500	X	X	X	X	X	X	X	X				
4012600	X	X	X	X	X	X	X					
4012700	X	X	X	X	X	X	X					
No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
4014500	X	X	X	X	X	X						
4014600	X	X	X	X	X	X	X					
4014700	X	X	X	X	X	X	X					
4014800	X	X	X	X	X	X	X					
4014900	X	X	X	X	X	X	X					
4015000	X	X	X	X	X	X	X	X				
4015100	X	X	X	X	X	X	X	X	X			
4015200	X	X	X	X	X	X	X	X				
4015300	X	X	X	X	X	X	X	X				
4015400	X	X	X	X	X	X	X	X				
4015500	X	X	X	X	X	X	X	X				
4015600	X	X	X	X	X	X	X	X	X			
4015700	X	X	X	X	X	X	X	X	X	X		
4015800	X	X	X	X	X	X	X	X	X	X		
4015900	X	X	X	X	X	X	X	X	X			
4016000	X	X	X	X	X	X	X	X	X			
4016100	X	X	X	X	X	X	X	X	X	X		
4016200	X	X	X	X	X	X	X	X	X	X		
4016300	X	X	X	X	X	X	X	X	X	X	X	
4016400	X	X	X	X	X	X	X	X	X	X	X	
4016500	X	X	X	X	X	X	X	X	X	X	X	
4016600	X	X	X	X	X	X	X	X	X	X	X	
4016700	X	X	X	X	X	X	X	X	X	X	X	
4016800	X	X	X	X	X	X	X	X	X	X		
4016900	X	X	X	X	X	X	X	X	X	X		
4017000	X	X	X	X	X	X	X	X	X	X		
4017100	X	X	X	X	X	X	X	X	X			
4017200	X	X	X	X	X	X	X	X	X			
4017300	X	X	X	X	X	X	X	X	X			
4017400	X	X	X	X	X	X	X	X				
4017500	X	X	X	X	X	X	X					
4017600	X	X	X	X	X	X	X	X				
4017700	X	X	X	X	X	X	X	X				
4017800	X	X	X	X	X	X	X	X	X			
4017900	X	X	X	X	X	X	X	X	X	X		
4018000	X	X	X	X	X	X	X	X	X	X	X	
4018100	X	X	X	X	X	X	X	X	X	X	X	
4018200	X	X	X	X	X	X	X	X	X	X	X	
4018300	X	X	X	X	X	X	X	X	X	X	X	
4018400	X	X	X	X	X	X	X	X	X	X	X	
4018500	X	X	X	X	X	X	X	X	X	X	X	
4018600	X	X	X	X	X	X	X	X	X	X	X	
4018700	X	X	X	X	X	X	X	X	X	X	X	
4018800	X	X	X	X	X	X	X	X	X	X	X	
4018900	X	X	X	X	X	X	X	X	X	X	X	
4019000	X	X	X	X	X	X	X	X	X	X	X	
4019100	X	X	X	X	X	X	X	X	X	X	X	
4019200	X	X	X	X	X	X	X	X	X	X		
No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**

5. Characteristics of the test site

5.1 Geomorphologic conditions

The test site is situated on the southern part of a barrier spit separating the Ringkøbing Fjord lagoon from the sea. The spit is formed by sand deposition resulting from a decrease in the rate of southwards longshore sediment transport. The natural southwards shift of the opening between the lagoon and the sea has been stopped by the construction of a permanent sluice and a lock at Hvide Sande where also a fishing port is located. The entrance is protected by jetties of which the longest to the north built in 1962 extends approximately 450 m from the foot of the dunes today.

As to the coastal profile along the test site, the distance from the coastline at level 0.0 m (equal to mean water level) to the 6 m depth contour is approximately 650 m over the full length of the test site, i.e. an average slope of app. 1:100. This slope has remained almost constant during the last 20 years according to the profiling by KDI. The coastline has in the same period shown large fluctuations with changes in position up to 80 m, see Section 5.7.
Grain size analyses of the sand in the foreshore and in the beach top layers show medium to very coarse sand with grain diameter in the range 0.3-2.5 mm often mixed with small stones in the range 10-30 mm. Deeper borings show fine sand down to approximately 10-12 m below the surface. Underneath is very fine sand or silt, and in some places clay.

The beach generally appears stratified as both waves and wind sort the materials. Eroded dunes also reveal stratification of former depositions, see Fig. 7.

Fig. 7. Stratified depositions in dune and on the beach. (Photos Hans F. Burcharth).

Very thin layers of for example high density sand appear in some places in a patchy way. The layers are almost horizontal as they generally follow the mildly sloping beach surface when deposited.

The average permeability of the upper metres of the beach is supposed to be only slightly larger in horizontal direction than in vertical directions, despite the layered structure. This is because the sand is packed in between the stones in the mixed layers (such layers can have reduced permeability compared to homogeneous sand due to the total blocking of the stones), and the layer thicknesses of the very fine materials are very small.

Several shore parallel bars are formed along the coast. The net sediment transport in front of the test site is southwards amounting to approximately 2.1 million m^3 per year in average (ref. KDI). Most of the longshore transport takes place in the bar zones.

5.2 Water levels

The difference between mean high water and mean low water is 0.7-0.8 m. Storm surge caused by strong westerly gales and low pressures can give water levels up to approximately 3.1 m above mean water level. Low water levels down to -2.0 m can occur during easterly winds.
In the Ringköbing Fjord lagoon the water level varies between -0.5 m and +0.5 m, dependent on the operation of the sluices and on the wind set-up.

A very severe storm with westerly winds of more than 26 m/s occurred 8-9 January 2005, shortly before the first survey took place in January 2005. Water levels up to 3.03 m above M.S.L. were recorded at the head of the jetties at the Port of Hvide Sande. Wave set-up might have caused an even higher maximum water level at the beach face of the test site. No severe westerly storms occurred in the first year period. The maximum water level recorded in this period was +1.44 m on the 26.10.2005 in a situation with only moderate wind. In 2006 occurred only one stormy situation on the 27.10 with maximum water level +1.54 m and westerly winds of app. 20 m/s. However, in January 2007 occurred four storm situations, 1.1.07 max. water level +1.75 m with winds just over 21 m/s, 11-12.1.07 max. water level +2.17 m with winds over 21 m/s, 14.1.07 max. water level +1.78 m with winds over 21 m/s, and 19-20.1.07 max. water level +1.82 m with winds over 21 m/s.

The January 2007 survey was performed after this row of January storms.

No stormy situations occurred after January 2007.

5.3 Wind and waves

The dominant directions of stronger winds are from the NW sector. This is also clearly seen from the dominant NW-SE direction of the dune tongues in the hinterland.

The prevailing westerly winds cause quite frequent storm waves with significant wave heights in the range H_s = 3-4 m offshore in 20 m water depth, and related peak periods of approximately T_p = 10 s. During more extreme events, say return periods of 5 years or more, H_s will exceed 6 m and T_p exceed 12 s. It is not often that H_s is less than 1 m and T_p less than 5 s during westerly winds. The waves are strongly seasonal as storms occur mainly in the autumn and during the winter.

The dominating directional sector of the larger waves reaching the actual stretch of coastline is west-north west, causing the net sediment transport to be southbound.

Fig. 8 shows the statistics of significant wave heights in the test period, recorded by a directional Wave rider buoy in 15.5 m water depth offshore Nymindegab.

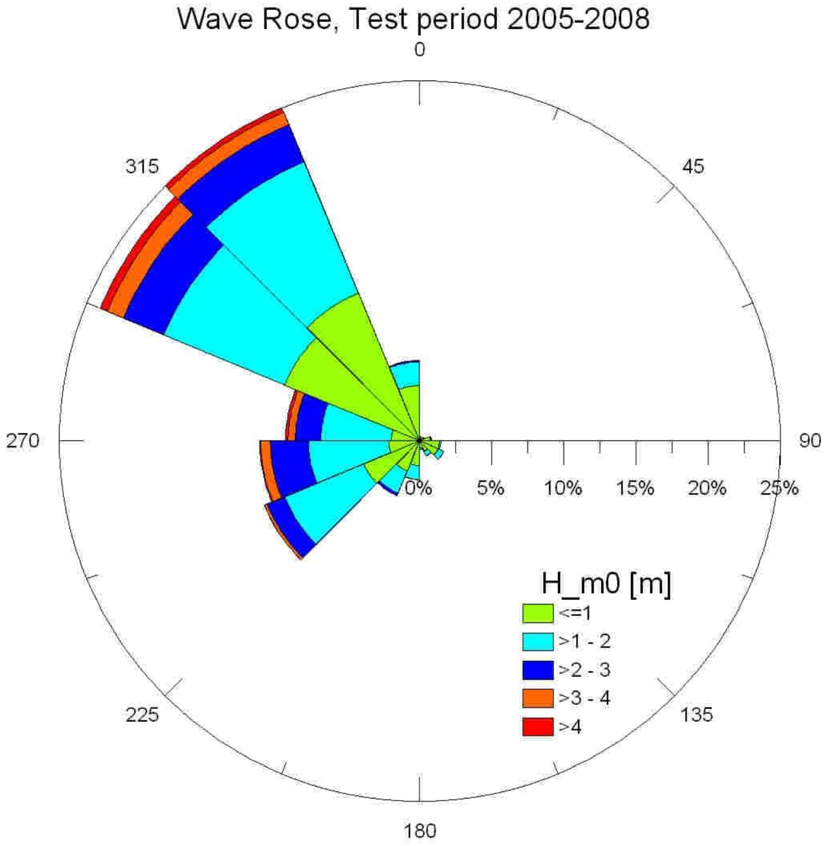


Fig. 8. Frequency and direction of significant wave height for the period 2005 – 2008, recorded in 15.5 m water depth offshore Nymindegab (KDI).

It is seen from Fig. 8 that the angle between the coastline and the dominating incoming offshore waves is approximately 45°, thus causing a net-sediment drift in southern direction.

5.4 Influence of bars on the wave impact on the coast

The protective effect of bars is significant as explained in the following. Bars are formed on sandy coasts with small tidal water level variations as is the case for the test site. The number of bars depends on the steepness of the coastal profile and the amount of sand transport (littoral drift). The flatter the profile and the larger the transport, the more bars are formed. The protective effect of bars is due to the smaller water depth over the bars which triggers wave breaking and thereby reduces the height of the waves approaching the beach. A rule of thumb is that the maximum height of waves passing a bar is app. 0.8 times the minimum water depth over the bar (or a significant wave height of app. 0.6 times the minimum water depth). It follows that the protective effect against beach erosion increases with the bar heights and the number of bars. Fig. 9 illustrates the reduction in wave heights over the bars.

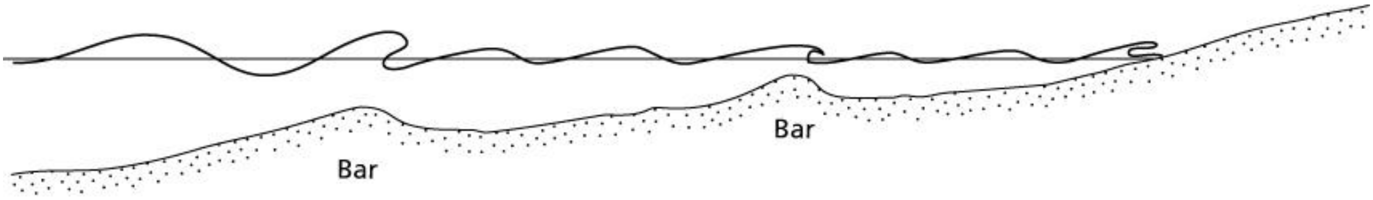


Fig. 9. Illustration of reduction in wave heights by wave breaking over bars.

Typical crest levels of the bars in the nearshore zone area are round 2 m below ML, limiting the significant wave height passing the inner bar to be approximately 0.6 x (2,00 + high water), i.e. around 3 m during the highest storm water levels.
Wave breaking on a bar produces a net transport of water into the trough behind the bar. The water that in this way is “pumped” over the bar must escape back to the sea somehow. Usually this takes place as rather concentrated outgoing currents, so-called rip currents, which create or maintain breaches (rip holes) in the bars, see Fig. 10. The beach is much less protected against large waves and subsequent erosion where holes in the bars are present. This is clearly seen in aerial photos like those in Fig. 11 which shows local retreat where the coast is facing holes in the bars, and local depositions (salients) where sheltered by the bars.

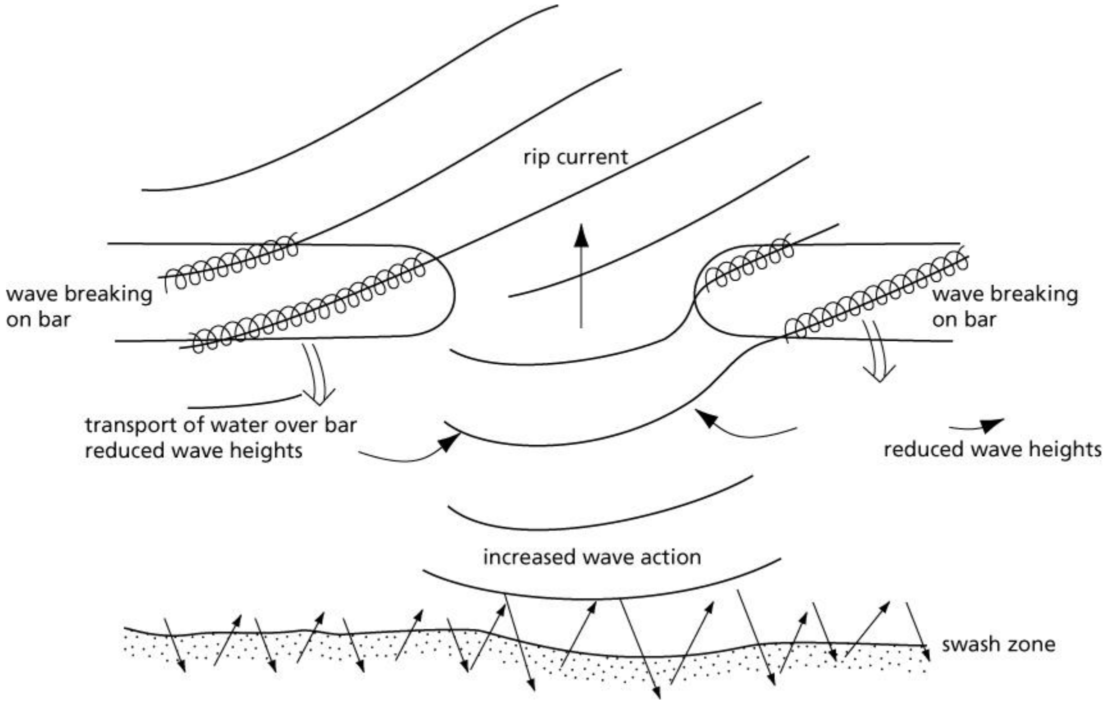


Fig. 10. Illustration of wave breaking over bar and related net transport of water, causing rip current and breach in bar exposing the coast locally to larger waves and erosion.

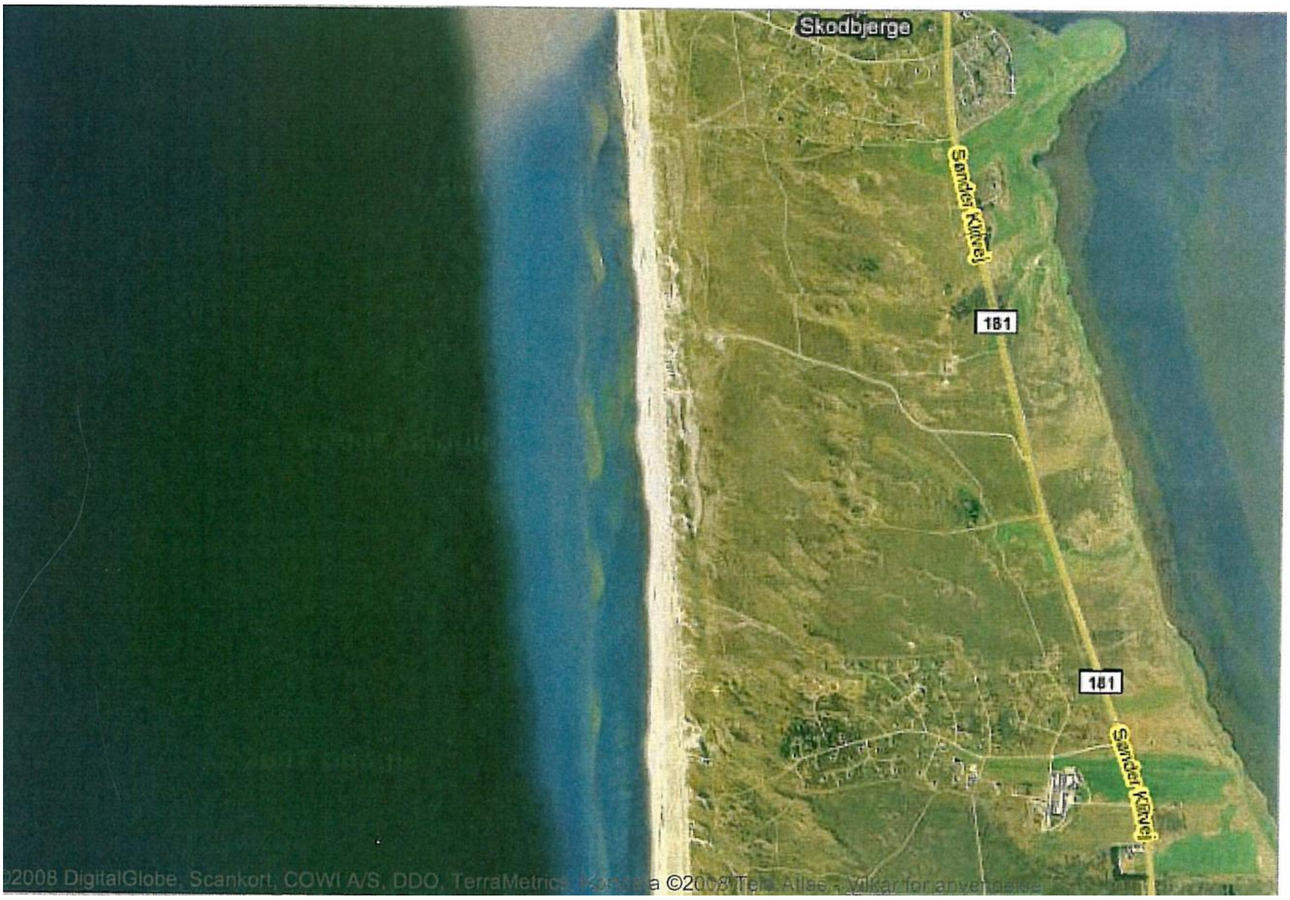


Fig. 11. Aerial photos showing the effect of holes in bars on coastline formation (Digital Globe Scankort, COWI A/S).

The major part of the longshore sediment transport takes place in the breaker zones, i.e. over the bars and in the beach breaker-zone. On the actual coast the net transport is in southerly direction. This causes a tendency to shifting of the holes in southerly direction because sediments are feeded mainly into the northern part of the holes. The shifting of the holes might be slow due to the effect of the rip currents. Anyhow, a given stretch of beach can therefore over the years experience periods with little bar protection – and thereby erosion, as well as periods with better bar protection – and thereby less erosion or even accretion.

Besides the longshore changes in bar formation there are usually also significant cross shore changes in the bar formation. Sediments are transported towards the beach in periods without storms leading to movements of the inner bars towards the shore and built-up of the beaches. Some of the inner bars might connect to the beach and thereby for a period create a very wide beach. On the other hand, storms cause erosion of the beach as sand is transported offshore both from the beach and from the bars. In a following quiet period a large part of the eroded sand is transported back towards and to the beach again. It is well known that built-up of a beach can happen very quickly after an eroding storm. As an example Kriebel et al. (1986) reports that more than 50% of the eroded cross section of a beach on Long Island above MSL returned within two days of fair weather following a storm. It is beleived that the fairly quickly built-up of some beach stretches in the first quarter after the severe Janaury 2005 storm is due to return of eroded sand.

In places like Denmark where severe storms usually occur only during winter we distinguish between so-called winter and summer profiles, the latter ones having wider and higher beaches.

If a coastal stretch with homogeneous soil conditions like the actual one maintains in average over the years a straight coastline and a straight dune front, then it is a sign of rather cyclic shifts in the bar formations as described above. This is the case for the actual coastal stretch.

The time scale of the major cyclic changes is years for which reason a test period of three years as the actual one is the absolute minimum.

5.5 Bar formation along the test site in the test period

Because of the protecting effect of the bars against erosion it is important to identify positions and holes in the bars and to see if there is correlation between the bar topography and the beach erosion and beach accretion.

Fig. 12 shows the bar formations January 05, 06, 07, May 07 and January 08. The outer bar is created and maintained by bar nourishment since 2004, cf. Table 3 in Section 5.7. The outer bar is clearly shielding the coast except in front of the southern part of Ref I and the southern part of Rør I and the whole of Ref II where large waves can penetrate to the inner bar close to the coastline. The fact that the most prominent erosion has taken place during the test periods in these stretches gives a very strong indication of the dominating effect of the bar formation on the changes of the beach. The long outer bar in front of Rør I is attached to the inner bar in the northern end leaving an opening to the south for the escape of the net inflow of water across the bar. The same bar configuration is seen developing both in the north and south parts of the test site. The existence of the holes in the outer bar has been visually observed by the author as stretches lacking wave breaking in windy weather.

On Fig. 13 is seen the bar formations just after the January 2007 storms and two weeks later. A slight growth of the outer bars in southerly direction can be observed. Holes in the inner bars are much more narrow than those in the outer bar, but have in principle the same negative effect.

It is evident that PEMs placed in the beach have no influence on bar formation.

Outer bars existed also before the bar nourishment began in 2004. Surveys by KDI show f.ex. the existence of a very high outer bar in the years 2000-2002 in front of Ref. II, i.e. where today is a hole in the bar, see Fig. 18 in Section 5.7.

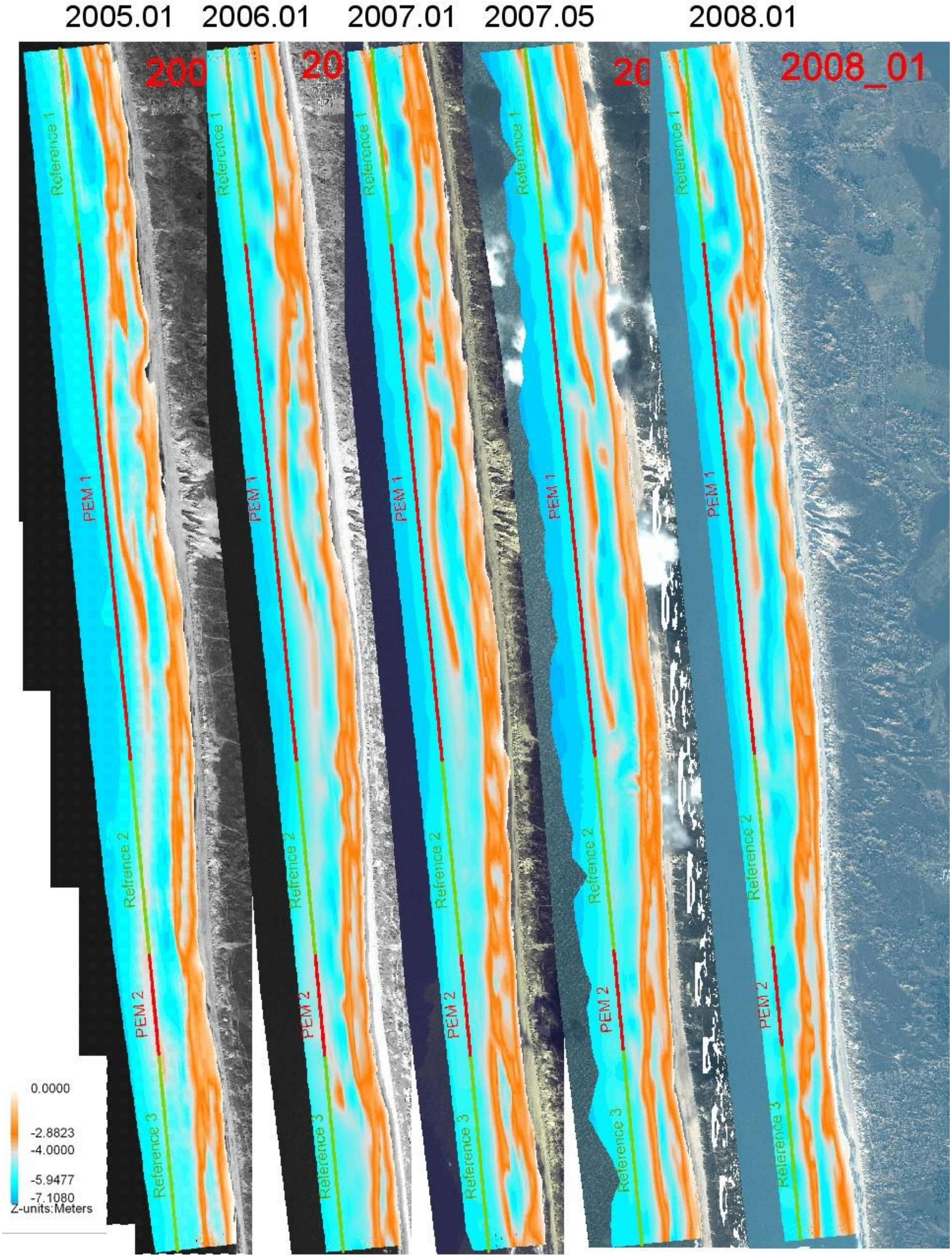


Fig. 12. Bar formations in the test period, (KDI).

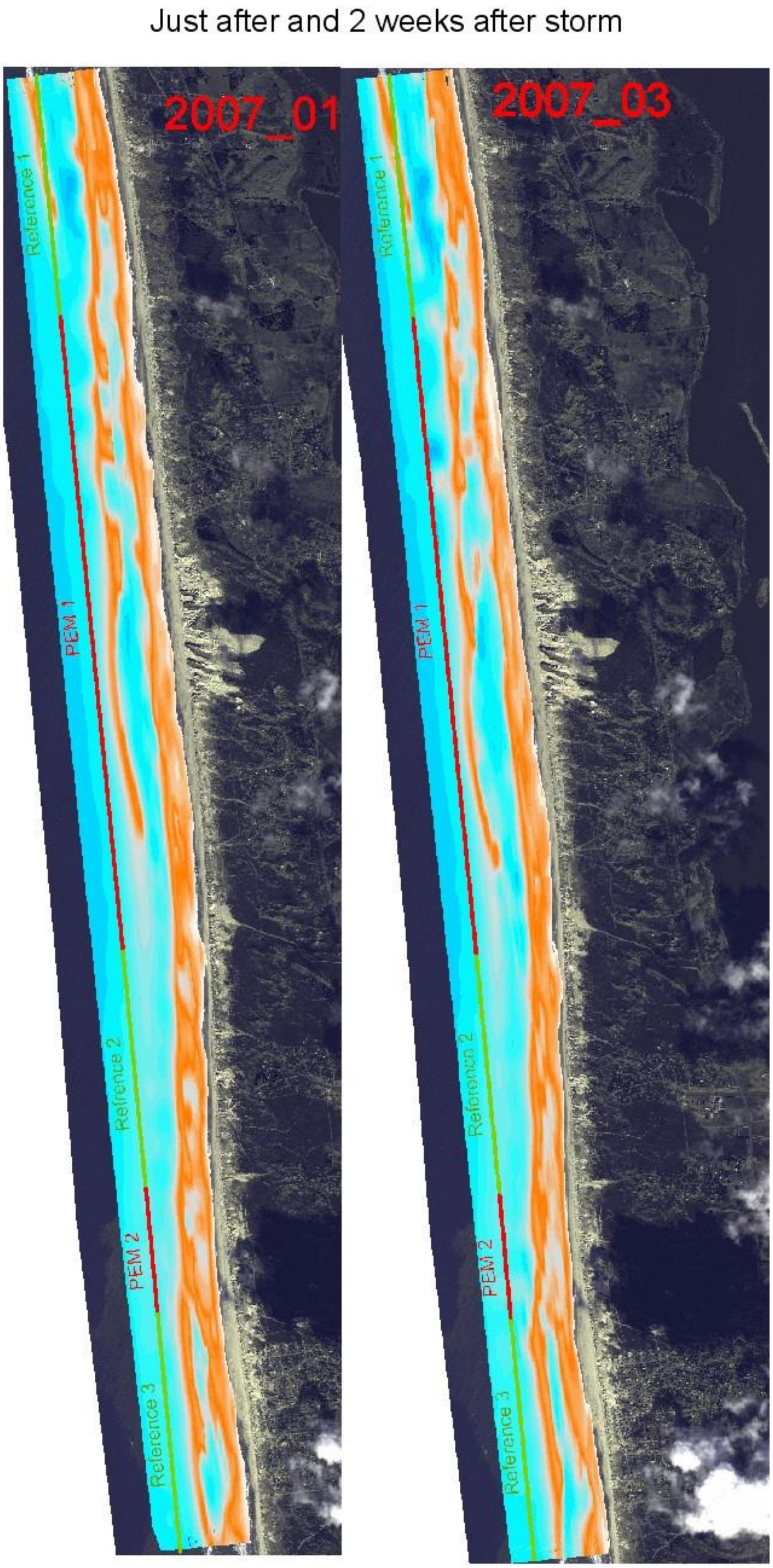


Fig. 13. Development in bar formations over two weeks just after a storm, (KDI).

5.6 Dune formation and protective effect of dunes

Dunes are formed where sand transported by the wind (aerolic transport) meets obstacles which causes the sand to settle. The transport of sand, which can be very large, takes place along the beach or across the beach and dunes, dependent on the wind direction relative to the coastline. The amount of sand removed from the beach surface is part of the sediment budget but difficult to estimate unless very accurate surveys of the complete active parts of the dunes and hinterland are performed.

The wider and the more dry the beach surface, the larger the amount of sand transported by the wind. Drainage of the beach therefore increases the aerolic transport of sand from the beach plane to the dunes.

A low narrow beach in front of the dunes gives little protection of the dunes against wave erosion while a high and wide beach normally prevent wave induced dune erosion. Anyway, the existence of dunes strengthens the coast and is therefore important as natural protection against coastline retreat. For this reason is sand transport by the wind regarded a positive effect although eroded from the beach.

Strong winds can cause local breaches (wind holes or blowouts) in the dunes in weak spots, i.e. without resistant vegetation or where a lowering in the dune occurs due to wave erosion. Sand is then merely transported through the wind holes to the hinterland, normally resulting in a reduced rate of the built-up of the dune front and the upper part of the beach and therefore weakening the beach.

The heights of the dunes along the test site are in the range 10-20 m above mean sea level except for a stretch in Rør I, with heights 5-8 m where three large wind holes were deliberately allowed to develop in order to explore the natural sand formations in the hinterland. No dune erosion took place here in the test period due to a wide and high beach. Actually a built-up of the dunes was observed as a lot of sand was transported from the beach to the hinterland. Fig. 14 shows a photo of one of three wind holes.

Fig. 14. Study area in the middle of Rør 1 for natural development of wind holes in the dunes (Photo Hans F. Burcharth).

Significant dune erosion took place only on a 500 m stretch in Ref II and a 300 m neighbour stretch in Rør I where also large wind holes developed during the janaury 2007 storms. This coincides very well with the lack of outer bar in front of the stretches; cf. Fig. 12, which allows larger waves to enter and erode the nearsshore zone. Fig. 15 show a photo of the wind holes in the northern part of Ref. II, taken after the storms in January 2007.

Fig. 15. Wind holes in dunes in the northern part of Ref. II, taken after the storms in January 2007. (Photo KDI).

5.7 Former coastal changes and man-made interventions

The natural average erosion (retreat of the coastline) over the years 1977-96 is estimated by KDI to vary gradually from approximately 3.5 m/year just south of the Hvide Sande jetties to approximately 1.5 m/year at the southern end of the test site.

Table 2 shows the average retreat velocity in metres per year for the coastline and various water depths for the years 1984-2004.

Table 2. Average retreat and accretion velocities 1984-2004. Retreat is given as positive values (KDI)

		Depth contours				
	Coastline	4 m	6 m	8 m		
VK-line	m/y	m/y	m/y	m/y		
5650	-1,87	1,6	5,1	6,34		
5660	1,35	9,05	4,73	6,92		
5670	-0,21	-0,14	7,05	4,37		
5680	-1,48	-0,61	2,58	5,21		
5690	0,17	1,69	4,06	-1,49		
5700	2,29	4,2	0,65	0,61	Ref 1	North
5710	-0,06	1,85	3,81	2,56	Ref 1	
5720	-1,19	3,59	3,29	2,91	PEM 1	
5730	-1,02	-0,32	-3,23	-0,11	PEM 1	
5740	0,98	2,9	2,46	0,93	PEM 1	
5750	2,46	8,68	3,28	3,04	PEM 1	
5760	0,89	4,31	-0,65	0,68	PEM 1	
5770	-0,27	3,23	-3,27	-1,3	Ref 2	
5780	-1,39	-1,35	-3,8	-2,27	Ref 2	
5790	1,83	-2,46	0,85	-6,72	PEM 2	
5800	1,63	0,21	-0,61	3,79	Ref 3	
5810	2,05	4,8	6,73	4,58	Ref 3	South
5820	-1,23	1,5	2,71	2,86		
5830	-2,01	-1,47	-6,07	-11,37		
5840	-3,3	-1,53	-0,28	1,43		
5850	0,13	-0,6	1,36	1,71		
5860	0,08	3,41	1,51	1,24		
5870	-0,06	0,29	-1,61	-0,47		
5880	-0,13	-0,77	-3,08	0,42		

The average retreat and accretion velocities given in Table 2 show that the coast and nearshore zone have been more or less stable with a slight retreat except in the Ref. III where more noticeable erosion has taken place over the 20 years up to the start of the project. The figures for the coastline are not exactly the same as those given in Fig. 16. The reason is the different periods of averaging. The coastline has, apart from fluctuations, in average been more or less stable over the last 5-10 years as documented by the KDI profiling of lines 5700-5810 (chainage 4010000-4021000). Table 3 lists the man-made interventions for the stretches Årgab (5 km stretch north of the test site), Havrvig (northern half part of the test site) and Skodbjerge (southern half part of the test site).

Table 3. Man-made interventions, 1977-2007, KDI

Volumes (m^3)

	Årgab					**Havrvig**			**Skodbjerge**	
	dumping at dune foot	beach nourishment	beach scraping	foreshore nourishment	bar nourishment	beach nourishment	beach scraping	foreshore nourishment	beach nourishment	beach scraping
1977	158.007									
1978	48.817			34.959						
1979	57.813			29.014						
1980	54.383			17.005						
1981	87.100									
1982	95.342									
1983	84.656									
1984	89.002		21.726							
1985	119.288		17.704	18.491						
1986	85.816		21.604	29.927						
1987	97.542		9.384	25.900						
1988	173.960		750	44.864						26.997
1989	165.361			41.336			4.410			21.182
1990	187.306			7.100			4.418			21.222
1991	177.766			1.318			4.084			24.422
1992	197.907			3.855		21.099			115.669	
1993	82.333	208.099		2.955		152.115	108.904			81.128
1994	60.602	148.455	13.395	1.591		214.945	51.288		82.345	25.123
1995	35.528	184.655	23.848	33.136			58.969			
1996	18.288	395.811		1.973		185.946	11.131			79.873
1997	12.534	187.718	19.001	2.618			36.565			42.875
1998	36.095	504.742		382		326.358	43.637			57.680
1999	17.480	388.036				228.020	8.010	200.255	154.110	41.624
2000	60.256	519.733		10.800		218.080	13.075			56.060
2001	14.342	429.572					4.634			60.900
2002		628.317					12.540			17.188
2003	28.706	527.925			2.632		20.239			42.907
2004		94.800	11.443		600.041		3.951			15.061
2005		192.400			200.419					
2006		145.884			505.105					
2007		180.000			300.130					
Total	2.246.230	4.736.147	138.855	307.224	1.608.327	1.346.563	385.855	200.255	352.124	614.242

Fig. 16 shows the position of the stretches given in Table 3. The figure also shows the average changes in position of the coastline in metre per year, calculated over the period 1987 to 2004, i.e. until the start of the project. A considerable seaward displacement rate of 2.95 m/y is seen in a part of Ref. II. An equal size erosion rate is seen in the South part of Ref. III.

The variability in coastline position is very large along the test site. This is demonstrated in Fig. 17 which shows the positions of the coastline relative to the average position in the years 1987-2007, i.e. the two first years of the test period are also shown.

The very large variability in coastline position illustrates the difficulties in separating the possible effect of PEMs from nature changes.

It is not possible to measure, calculate or predict in a quantitative way the influence of the bar nourishment on the various stretches of the test side. The problem was discussed already when the test site was decided, cf. Chapter 3.

A way to overcome this problem, at least partly, would be to divide the test site in say five or six stretches of app. 2 km length of which every second should be drained with PEMs. By doing this it would be much more easy to identify the effects of the PEMs because it is unlikely that the bar nourishment would not be the same over at least two neighbour stretches. This solution was proposed from the beginning of the discussion about the test site, but SIC would not in any way accept it.

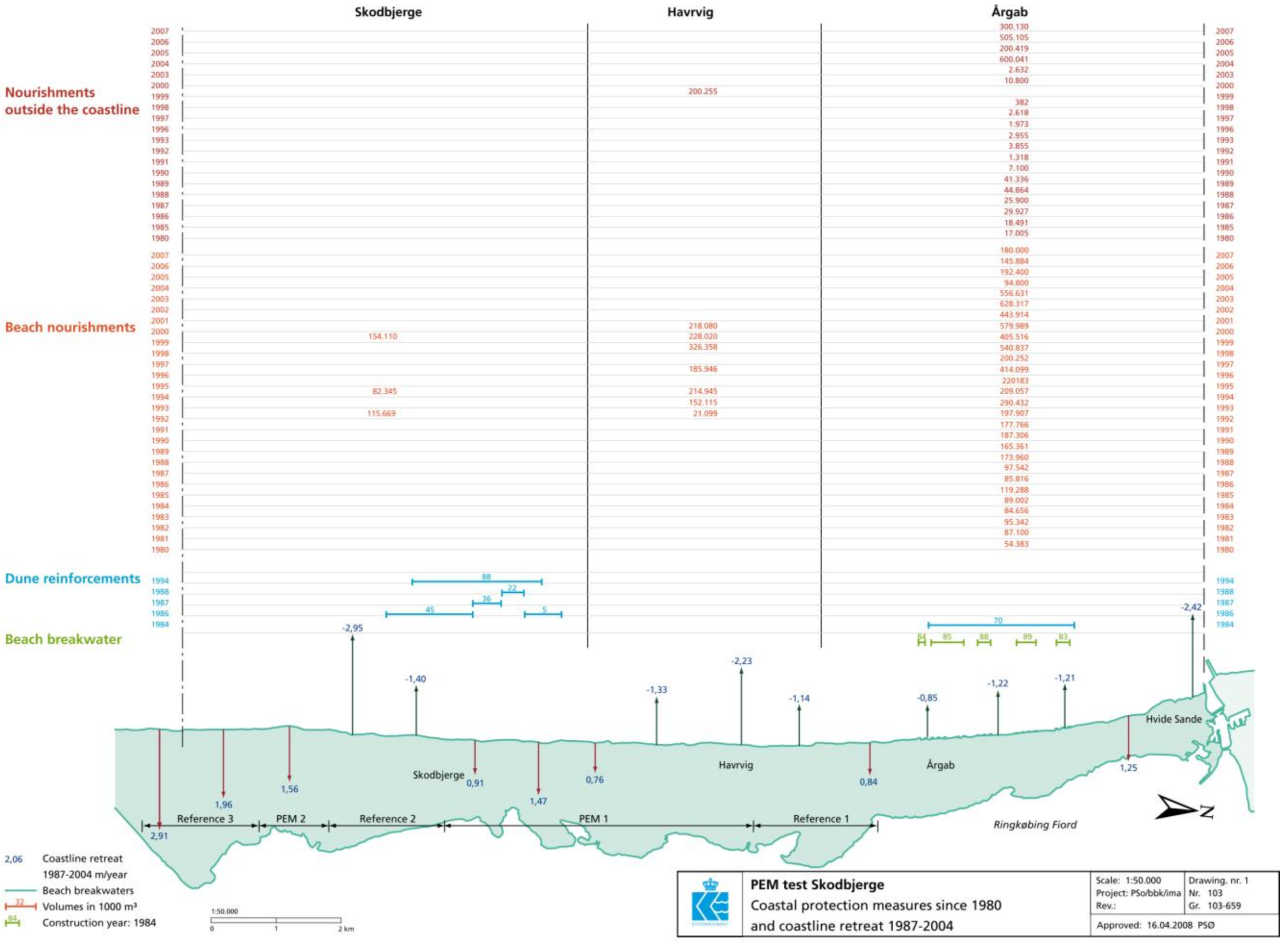


Fig. 16. Man-made costal protection measures since 1980, and average yearly coastline retreat 1987-2004 (KDI).

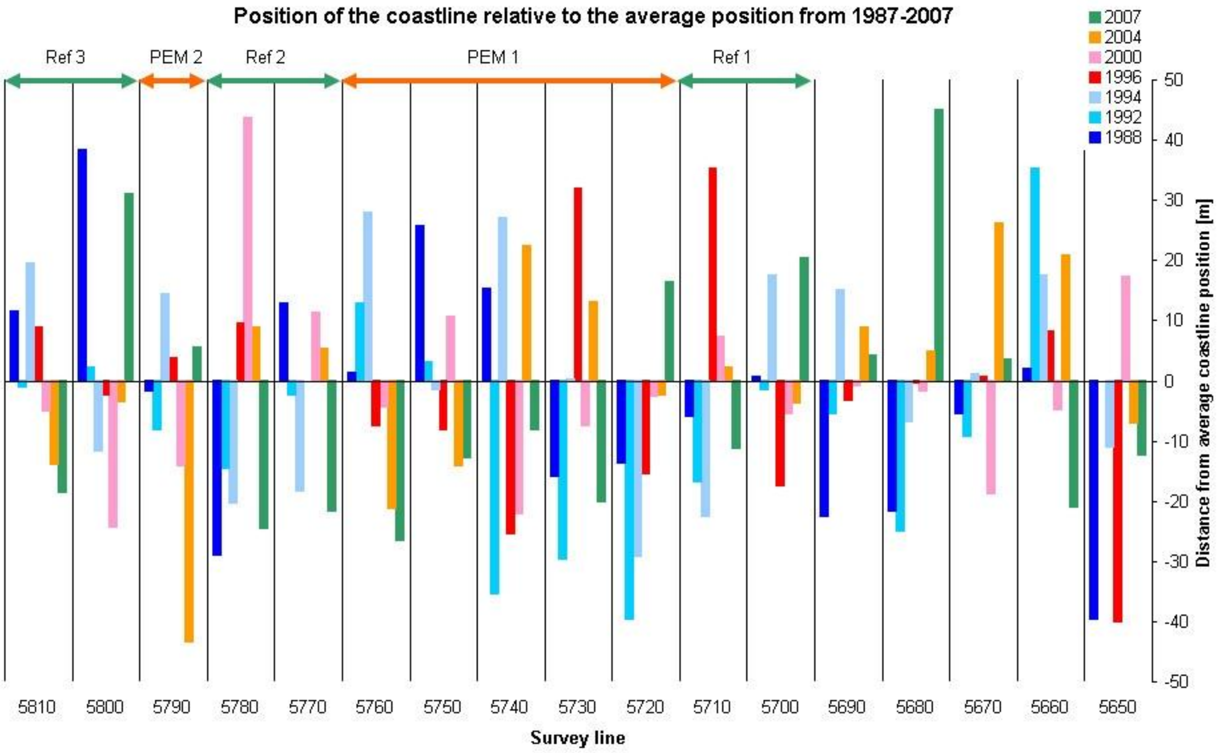


Fig. 17. Coastline positions relative to average position, 1987–2007 (KDI).

The reason for the significant erosion in Ref. II and southern part of Rør I in the test period is most probably the hole in the outer bar. This hole was not present in the former years as is seen in Fig. 18 which shows the coastal profiles in the period 2000-2007. Actually a high outer bar protected Ref. II in the years 2000-2002.

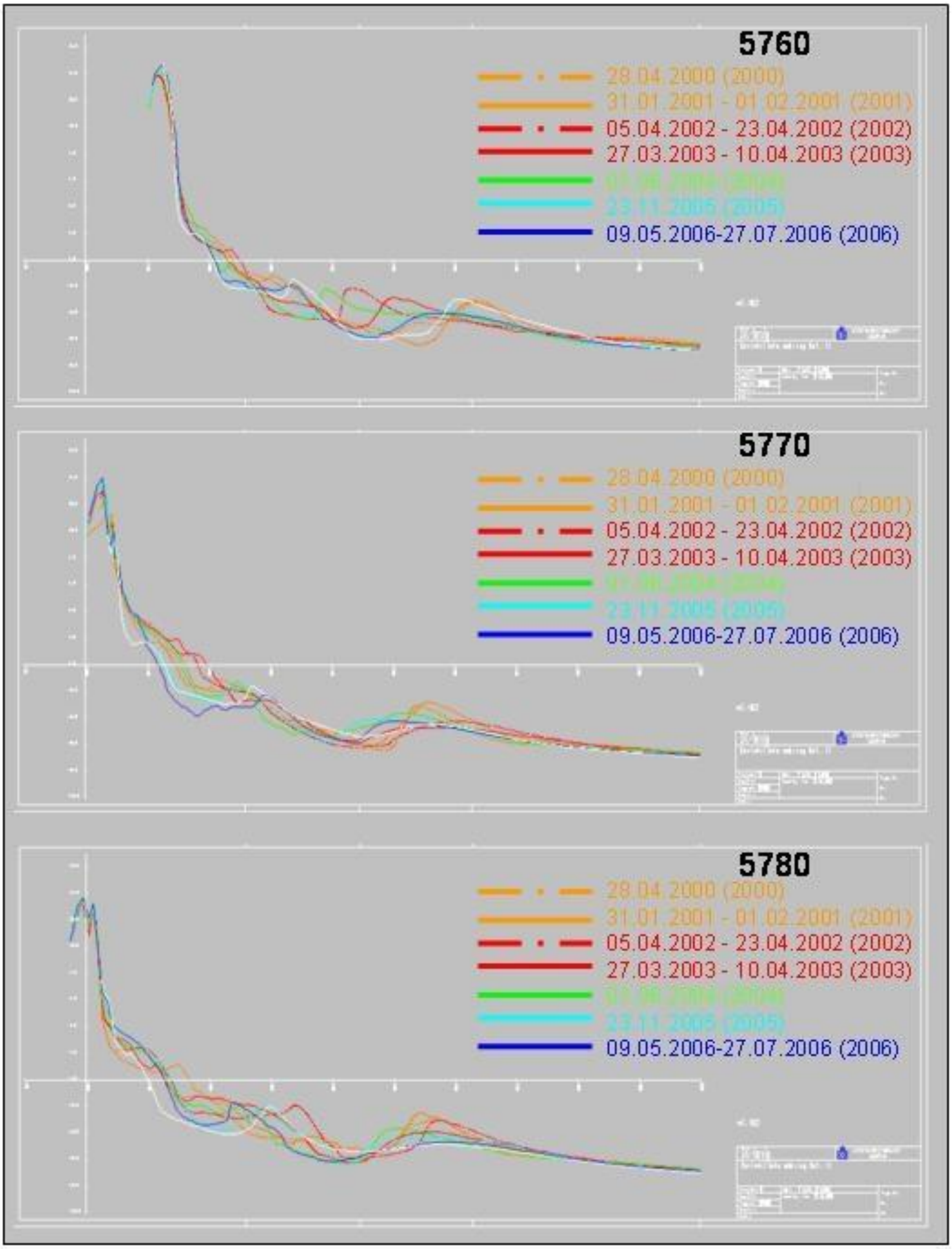


Fig. 18. Depth sounding profiles 2000-2007 in south of Rør I and in Ref. II, (KDI).

5.8 Planform movements along the coast

It is known that obliquely incoming waves can cause undulations in the coastline. The scales of these undulations are typical: wavelength 1-2 km, amplitude 10-50 m, and down-drift migration velocity: 50-500 m/year. Very large undulations along the Danish coast are for instance identified by KDI at Uggerby just East of Hirtshals, and at Gl. Skagen at the northern tip of Jutland. At both locations, the coast is exposed to very obliquely incoming waves. It should be noted that in the period January 1999 – April 2004, during which the coast of Gl. Skagen was surveyed as part of a field test with SIC vertical drains, no systematically moving undulations were identified by the project leader, Hans F. Burcharth, so they are not always present.

Fig. 19 shows an attempt to verify if such moving undulations exist along the test site by depicting positions of the coastline for the period 26.1.05 - 3.1.08. A southbound shift of the coastline planform can be identified. The migration velocity seems to be app. 500 m/year.

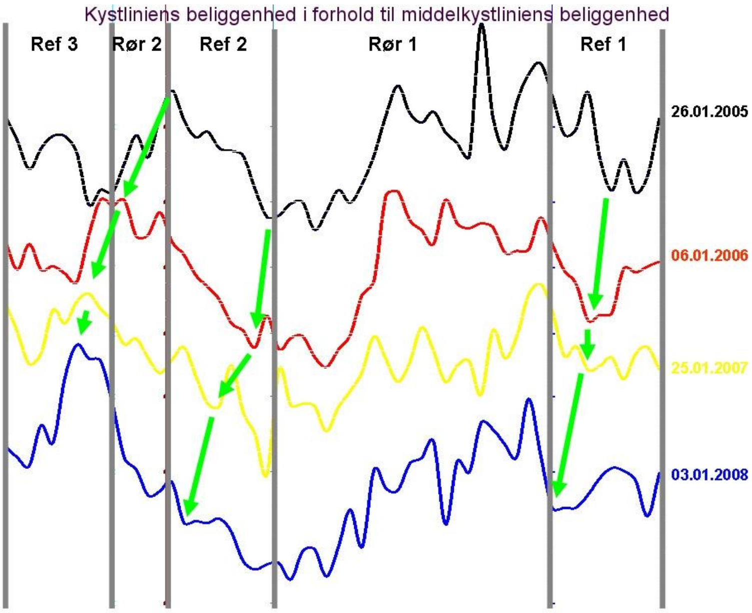


Fig. 19. Movements of the coastline planform, 26.1.05 - 3.1.08 (KDI).

While such movements to a certain degree can explain local shifts between retreat and advance of the coastline, it can be demonstrated that the volume of sand moved alongshore in the undulations is much smaller than the total volumes involved in the process of erosion and accretion of some stretches.

6. Monitoring of the test site

6.1 Surveying

Profiling per 100 m of the beach and dune front four times per year was decided as well as soundings per 200 m of the seabed within 600 m from the shoreline. The first profiling took place in January 2005 just after placement of the drains. Since then a total of 12 surveys have been performed in April, July and October 2005, January, April, July and October 2006, January, March, August and September 2007, and finally January 2008. Carl Bro A/S performed the landward surveying and KDI the depth sounding. Moreover, KDI has performed depth soundings along the North Sea west coast three times every year in lines spaced 1 kilometre and covering the nearshore zone from the beach to app. 8 metre water depth. Five of these lines cover the stretch just south of the Hvide Sande inlet and thereby also the stretch where bar nourishment takes place.

6.2 Monitoring of ground water levels across the barrier spit

According to SIC the function of the drain relates to changes in the ground water flow caused by pressure equalisation in the surroundings of the drains. For this reason a comparison between pressure fields near the drains and far from the drains are of importance. As it is generally accepted that ground water outflow in the beach face affects the sedimentation, it was decided in 2005 to monitor in one line the ground water table across the narrow land spit between Ringköbing Fjord and the test beach. Application for permission to establish wells was forwarded to the authorities. **However, the campaign was stopped in 2006 as SIC now explained that the actual ground water table variation across the land spit would have no influence on/or could not enlighten the function of the drains.**

6.3 Grain size analyses

In order to check the hypothesis of SIC that the drains increase the groundwater outflow through the beach face and thereby wash out the fine beach material, it was decided to investigate if changes in the composition of the beach material take place as a result of the installation of the drains. Five borings were taken app. three month after the installation of the drains in Rør I between chainage 4015500 – and chainage 401540. SIC has raised the question if this was too late compared to the rapid development in accretion observed after placement of the drains. However, the author is of the opinion that the effect should be permanent in order to have influence.

Grain size analyses of the samples was made and compared with samples taken in May 2006. The relative amounts of very fine material with grain size smaller than 0.063 mm were determined from samples taken from each boring in three specific levels. The analysis revealed that in two of the five borings, one being located close to the drains, there was a clear decrease in the relative amount of fine material. In the other three borings, of which one was also close to the drains, there was not a clear picture, but the tendency was an increase in the amount of fine material. On this background it is concluded that a clear sign of decrease of fine material close to the drain was not found.

6.4 Satellite images, aerial photographs, and airborne laser photogrametry

Nine sets of satellite images covering the period 9.10.2004 – 11.9.2006 have been obtained. The varying quality of the images makes an analysis difficult. Airborne laser photogrametry covering the nearshore water area was tried in order to get information about bar formations, but without success. Aerial photographs have not been of a frequency and quality which allow more systematic analyses.

Two airborne laser scannings of the beach and dune topography have been used to estimate order of magnitude of the transport of sand by the wind from the beach to the dunes, see Section 11.5.

6.5 Pressure measurements in the beach

A field study of the pressure in the tubes and in the surrounding sand was performed in the spring of 2006 in order to get some insight in the functioning of the drain system. The location was the northern part of Ref. I, see Fig. 6. The program was carried out with additional consultancy of Dr. Peter Engesgaard, Geological Institute of University of Copenhagen. The details and the analysis of the results are given in the report of Peter Engesgaard: Effect of Vertical Drains on Tidal Dynamics in Beaches, Final Report, June 2006. The idea was to measure the difference in pressure variations and dampening of the tidal wave across the beach in lines with and without PEMs installed, see Fig. 20. A detected difference would indicate a function of the PEMs.

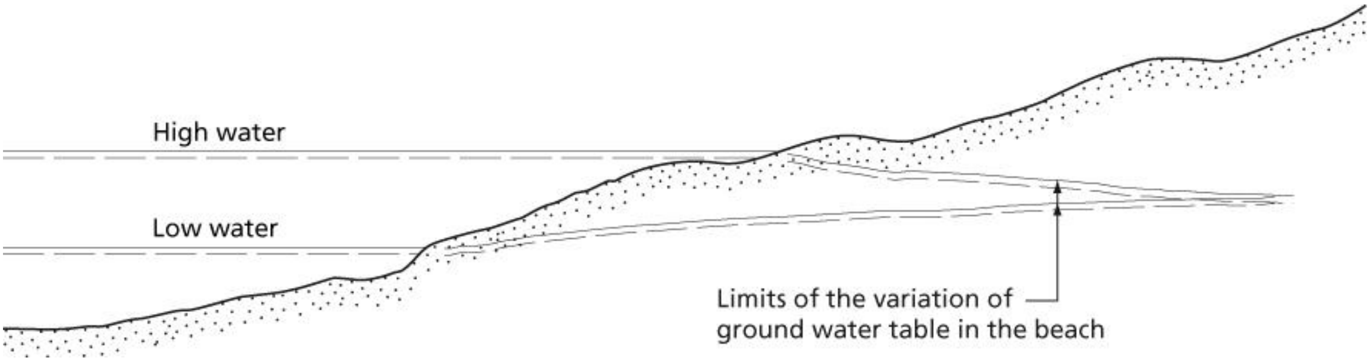


Fig. 20. Illustration of dampening of the tidal wave in the beach, i.e. the ground water table variation in the beach.

The experiment was divided into two periods: Period 1, one week with only wells installed (6 cm diameter pipes with pressure transducers placed in the mid level of a 10 cm screens app. 2.3 m below the beach surface and 10 m in between them). Period 2, one week with PEMs installed in beween the wells, i.e. 5 m distance between wells and PEMs. Pressure transducers were placed at the bottom of the PEMs. The lay-out of one line is shown in Fig. 21.

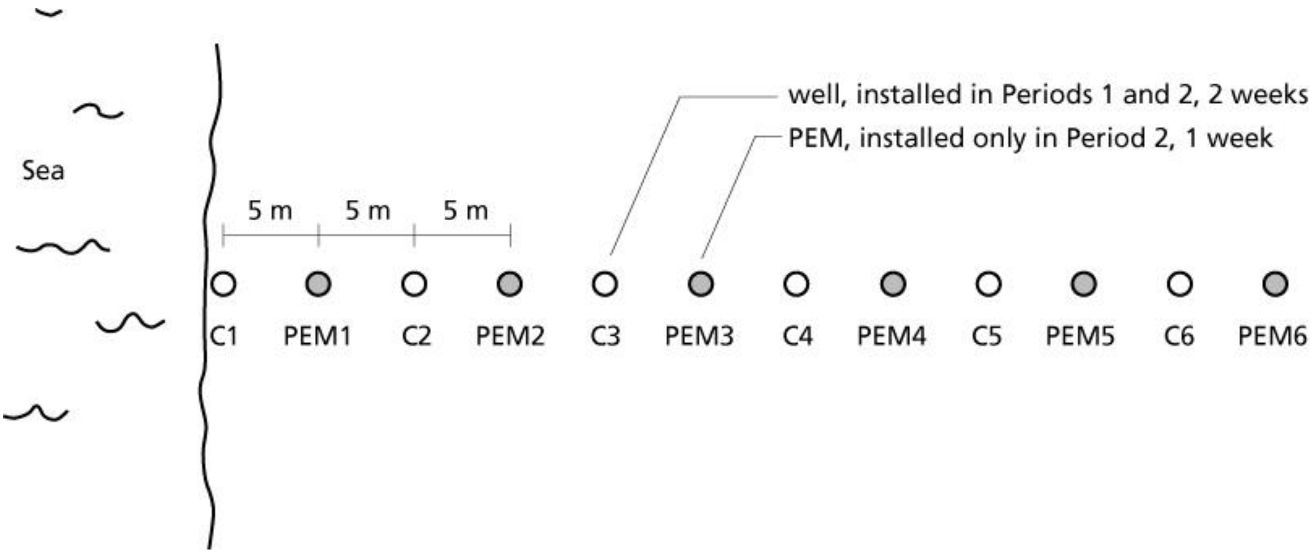


Fig. 21. Lay-out of one line in the field study of pressure variations with and without PEMs.

Pressures were recorded simultaneously from all transducers every two minutes. Fig. 22 shows the variation in hydraulic head (pressure height) during the second week of the tests, Period 2.

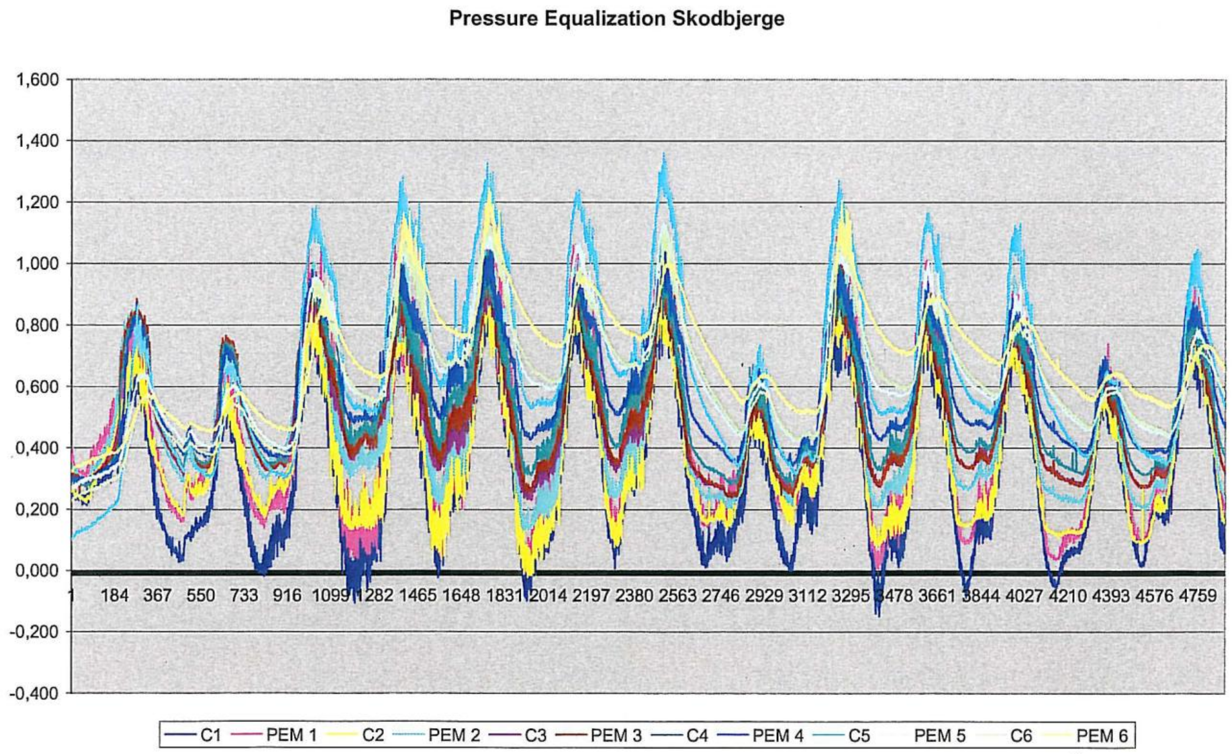


Fig. 22. Variation in hydraulic head in wells and PEMs during Period 2, the second week of the field study (SIC, 2007).

The numbers in the horizontal time scale are 2 minutes intervals. The tidal variation with range up to app. 1 metre is clearly seen. The small variations on top of the major variations are caused by the wave action, but are not representing fully the wave induced pressure variations as the logging interval of two minutes is much too long compared to the wave periods (5-10 sec.). Anyway, the influence of the wind generated waves is not included in the analysis, as was agreed by SIC.

From Fig. 22 is also seen the dampening of the pressure (tidal wave) fluctuations when moving inland. A rise in the mean water level of 35cm from Period 1 to Period 2 changed the flow and pressure conditions, and had to be taken into account in the analyses. Peter Engesgaard (2006) studied and compared the dampening in the two periods and concluded that the dampening characteristics were the same in the two periods for which reason the PEMs did not show a significant drainage effect.

SIC studied the difference in heads between each PEM and the two neighbour wells. Fig. 23 shows an example from the lower part of the beach.

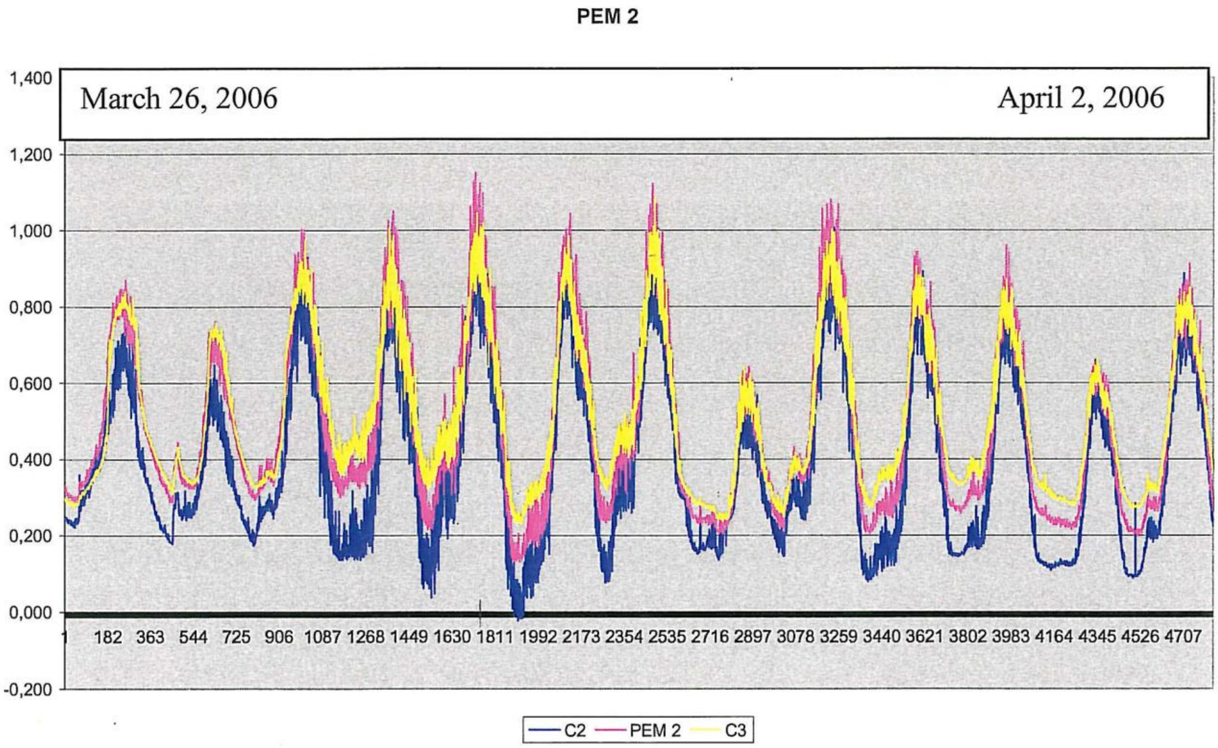


Fig. 23. Variation in hydraulic head in PEM 2 and neighbour wells C2 and C3, (SIC, 2007).

It is seen that the water level in PEM 2 is higher than in the neighbour wells C2 and C3. This indicates an inflow in the upper part of the perforated part of the PEM 2 pipe and an outflow in the lower part. At low tide the picture is not so clear.

The situation around PEM 4 higher up in the beach is shown in Fig. 24.

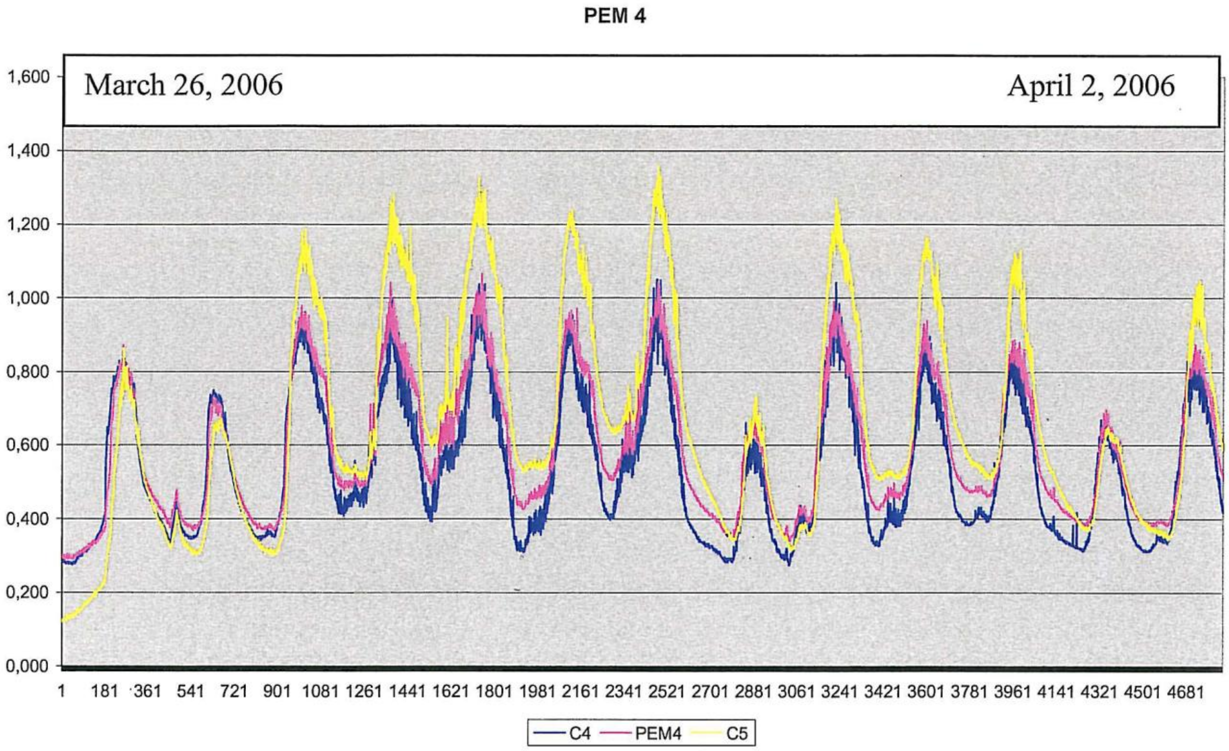


Fig. 24.Variation in hydraulic head in PEM 4 and neighbour wells C4 and C5. (SIC, 2007).

The PEM-water levels are in between the levels of the wells with no clear indication of a net flow direction in the perforated part of PEM 4, except that flow in both directions takes place.

The differences in heads between the PEMs and the average of the two neighbour wells are in the range 0 to ± 10cm.

It is not possible from the differential head measurements to estimate how much more water is actually drained to the sea due to the PEMs. However, an estimate can be made as follows: The head measurements show horizontal head gradients up to $i = 0.01$. An estimated permeability of the beach material would be $k = 0.001$ m/s for medium to coarse sand (CIRIA report 169). The corresponding max. flow velocity is then $v = k \cdot i = 10^{-5}$ m/s = 0.9 m/day. The flow velocities in the 1 m active parts of the 6 cm diameter PEM can be up to app. 200 m/day (Engesgaard, 2008), i.e. app. 200 times higher due to the much lower resistance. However, because the cross sectional area of a pipe would be only app. 0.04‰ of the influence area of a PEM (if this has a radius of 5 m corresponding to the 10 m distance between the modules), it implies that the PEMs increase the drainage by 8 ‰ at the most, and only for a short while by one or two PEMs within a tidal cyclus. The relative effect of the PEMs will be even smaller in the case of coarser and more permeable beach materials.

7. Numerical simulation of the PEM drainage effect.

A desk study of the function of the PEMs was performed by Dr. Peter Engesgaard, Geological Institute, University of Copenhagen (Engesgaard, 2008). The objective was to numerically investigate the effects of PEMs on groundwater flow in a coastal aquifer, especially their drainage effect. A number of simulation scenarios were investigated representing different flow systems including homogeneous sand, inclusion of gravel and clay layers, with and without PEMs. The boundary conditions for the simulations were:

The flow field simplified to 2-dimensional which implies that the PEMs are like continuous *curtains* parallel to the coastline and not 6 cm diameter PEMs placed only per every 100 metre.

Impermeable bottom in level -10.0 m

A fresh water head of 0.3 m from the hinterland

Tidal amplitude of 0.5 m

Hydraulic conductivities of sand, gravel, silt/clay of 25 m/day, 250 m/day and 0.25-2.5 m/day, respectively

No hydraulic resistance in the flow inside the PEMs

All results correspond to the quasi-steady state after 50 repetitions of the tidal cycle shown in Fig. 25.

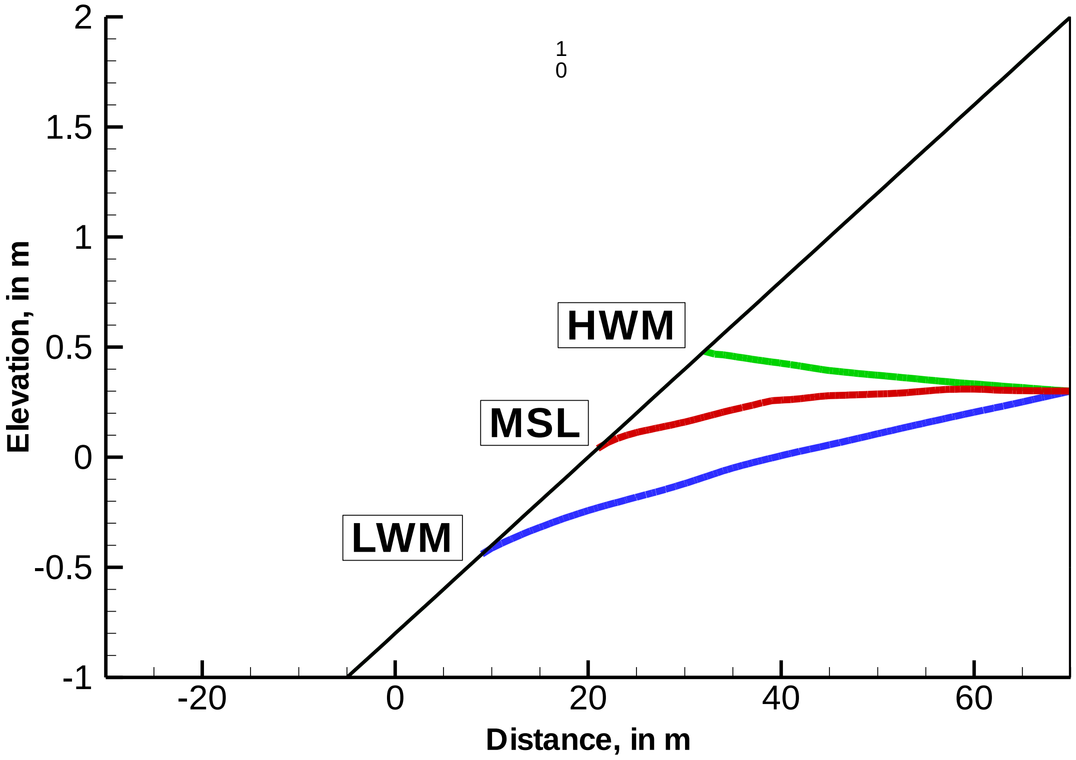
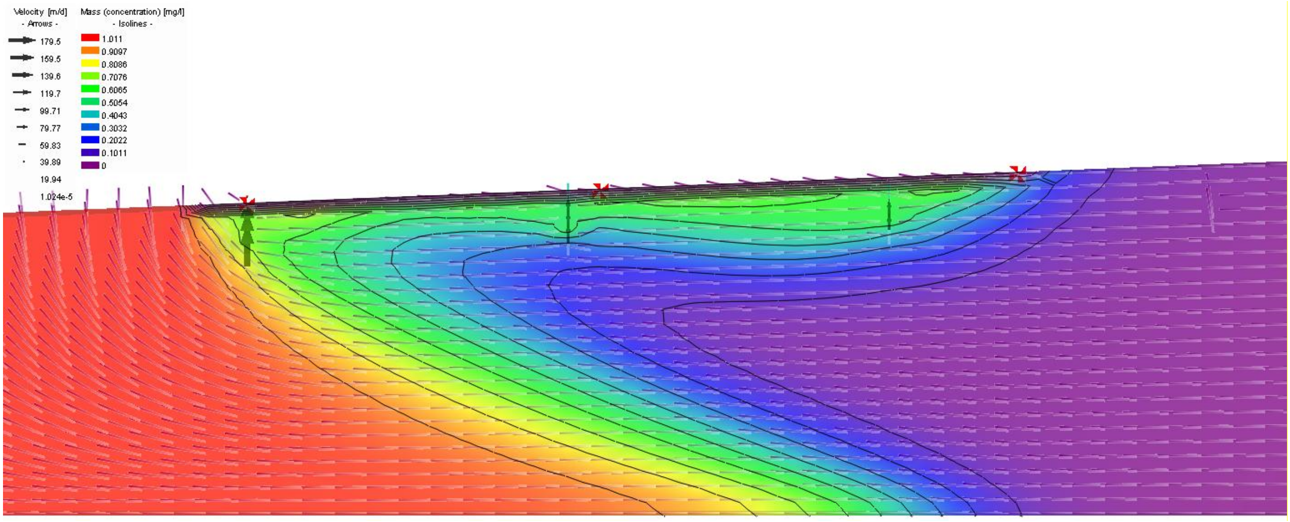


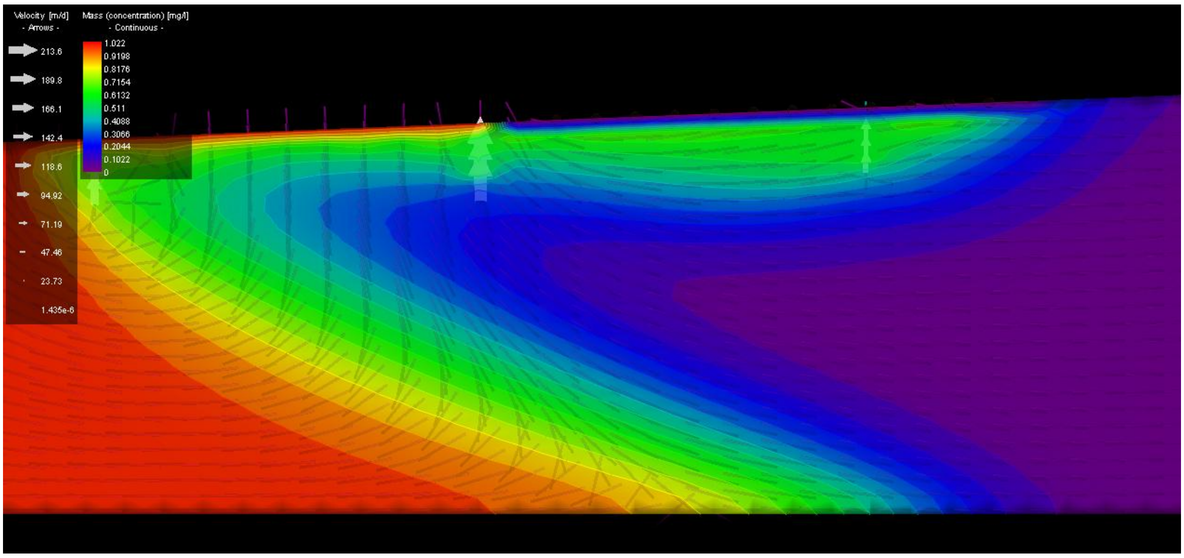
Fig. 25. Simulated water table at low, mean and high tide for the homogeneous case (Engesgaard, 2008).

The following examples of figures from Peter Engesgaards report illustrate results of the simulations:

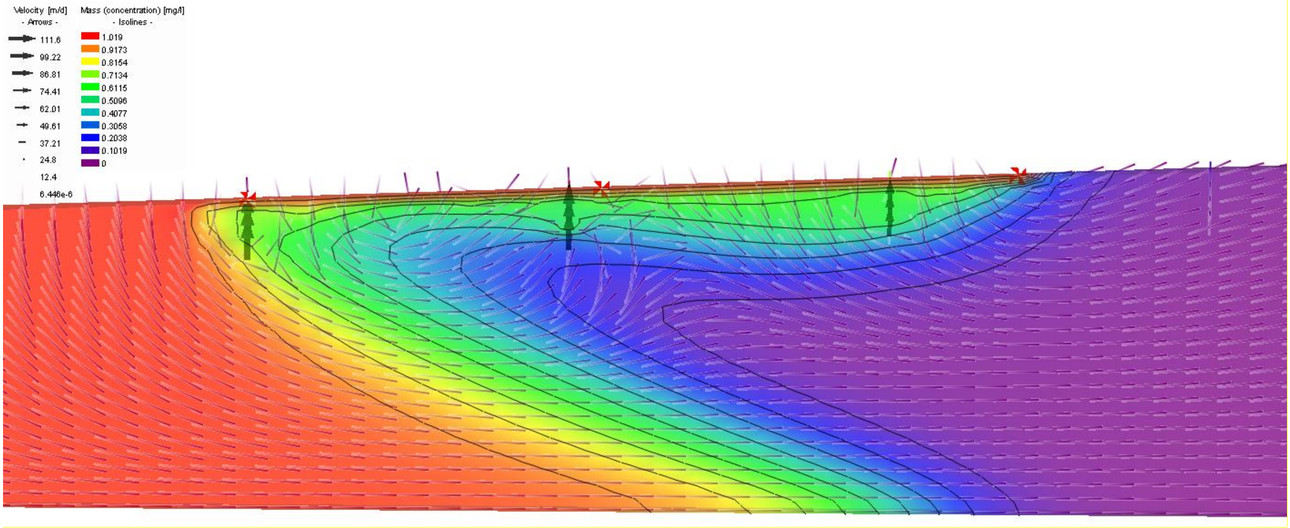
Fig. 26 shows the simulated water mass density and velocities and directions of the flow in the beach for the case of PEMs in homogeneous sand. Low tide, mean sea level (falling water) and high tide are shown.



Low tide



Mean sea level (falling water)



High tide

Fig. 26. Simulation of water mass density and velocities and directions of the flow for the case of PEMs in homogeneous sand. (Engesgaard 2008 and private communication).

Fig. 27 shows the same situations as top and bottom figures in Fig. 26, but the flow is plotted as log-velocity in order better to see the flow directions. The head distributions are also shown. The text below the figures is by Peter Engesgaard.

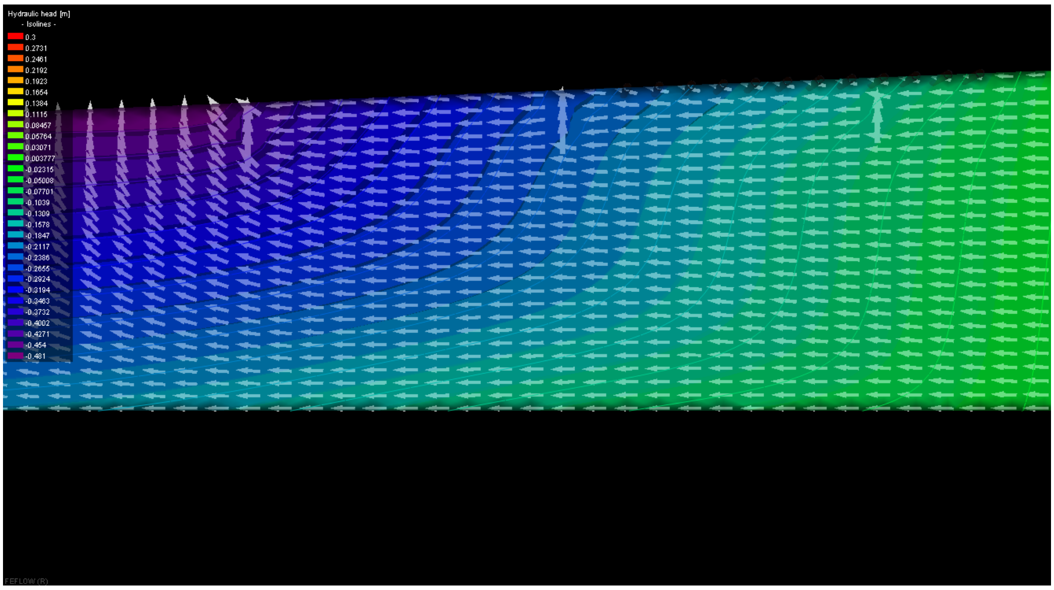
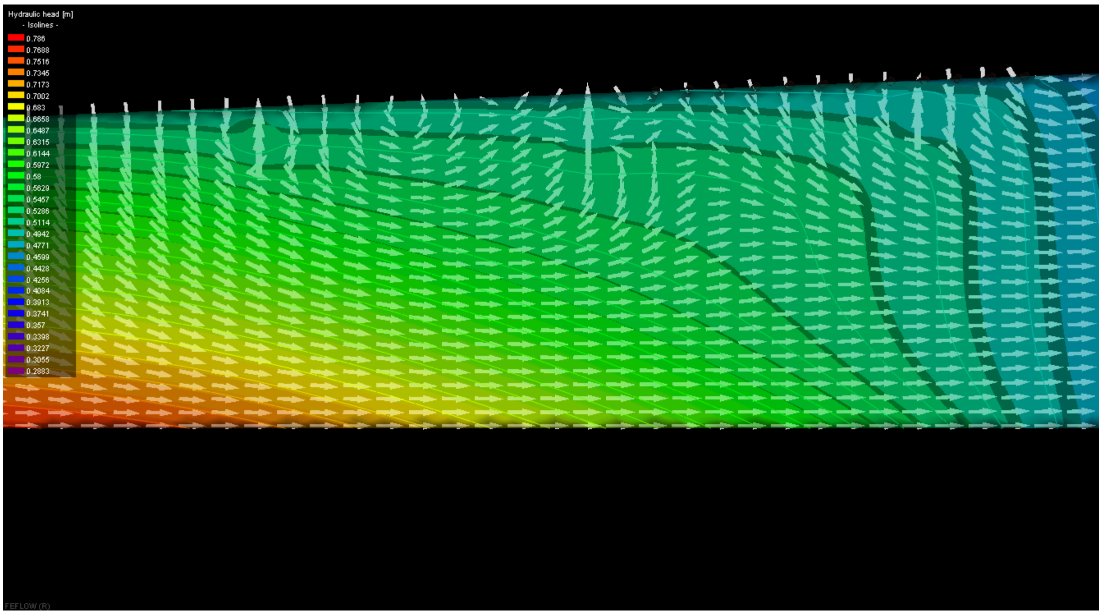


Figure shows low tide with PEMs. Velocities are now plotted as log (velo). The freshwater head is also shown. It looks like the almost horizontal flow velocities are deflected towards the bottom of the PEMs and out of the PEMs near the top. All PEMs show vertical velocities, which seems intuitively correct.



Same figure at high tide. The effect of the PEMs on the velocity distribution is much more apparent. Again all PEMs show upward flow, despite that flow just outside the PEMs can be downward. The left-most PEM actually shows a circulation, where flow moves down the PEMs and back up through the PEMs. The head distribution around the bottom and top of the PEMs confirm that flow is into the PEM at the bottom and out through the PEM. It is not clear why this circulation comes about. Perhaps a combination of buoyancy effects and flow driven by forced convection (head gradients; notice that heads increase toward bottom. At the middle PEM the flow distribution is clearer. Flow diverges upward, enters the bottom of the PEM and exits at the top. At the right most PEM there is again flow down along the PEM, but now part is (apparently) diverted up through the PEM again and parts is flowing towards the landside. Still we see the characteristic head distribution indicating flow into the PEM and out of the PEM.

Fig. 27. Log-velocity plot of flow at low tide and high tide as well as head distribution (Engesgaard, 2008).

Fig. 28 shows the spatial distribution of the flow velocities in a homogeneous beach.

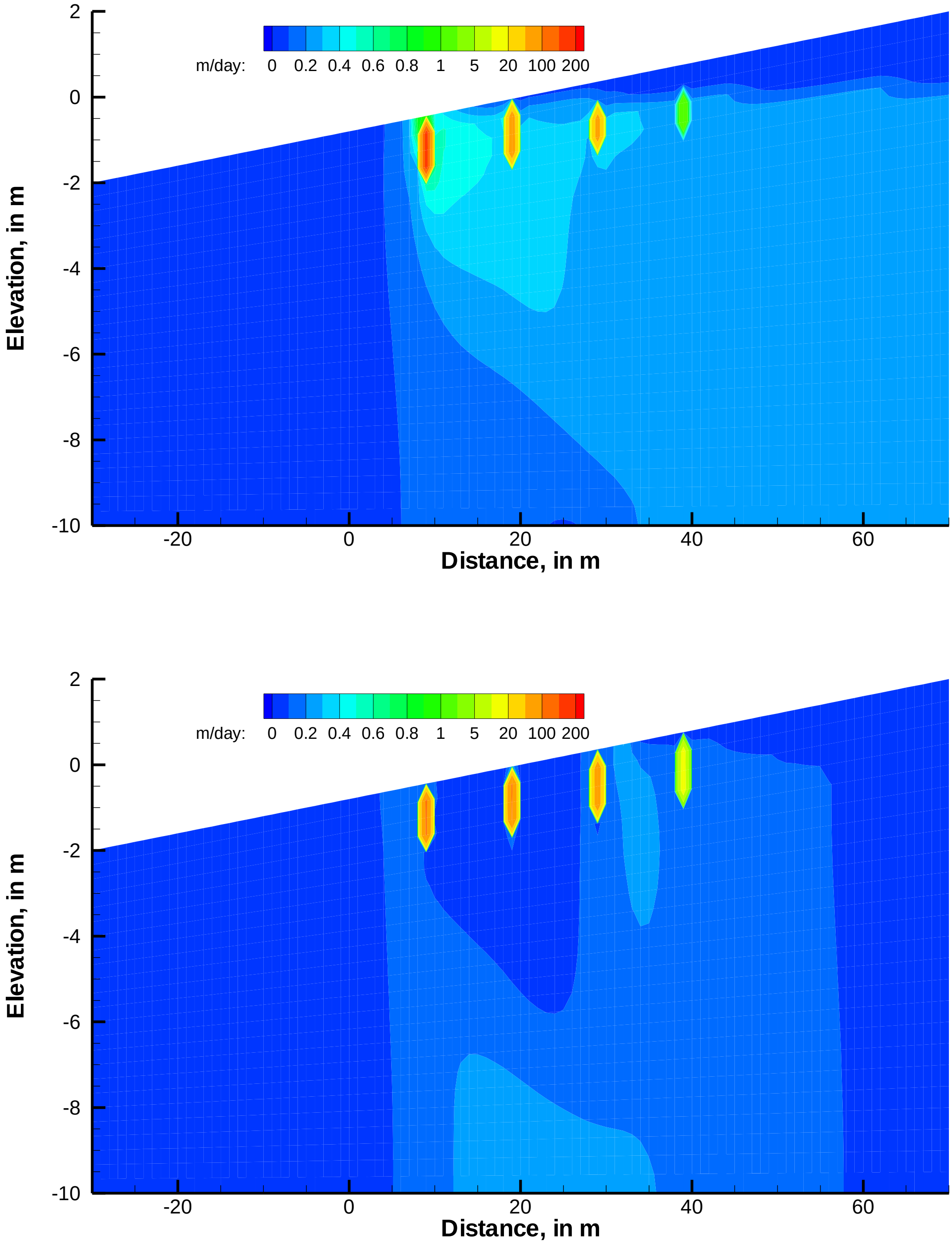


Fig. 28. Distribution of flow velocities in a homogeneous beach. Flow directions not shown. Upper figure low tide. Bottom figure high tide (Engesgaard, 2008).

It is seen that the four PEMs are not active in a uniform way since at low tide it is the PEM near the low water level that is active with upward directed velocities near 200 m/day (2 mm/s) while at high tide it is the PEM near the high water level that is most active. Between the most active PEMs the velocities are in the range 0.2-0.5 m/day.

Fig. 29 shows a close-up of the velocity distribution at low tide with and without PEMs in a homogeneous beach.

It should be noticed that the flow when leaving the PEMs is reduced to the same order of velocities as those in the flow field without PEMs. This indicates that no efficient drainage takes place.

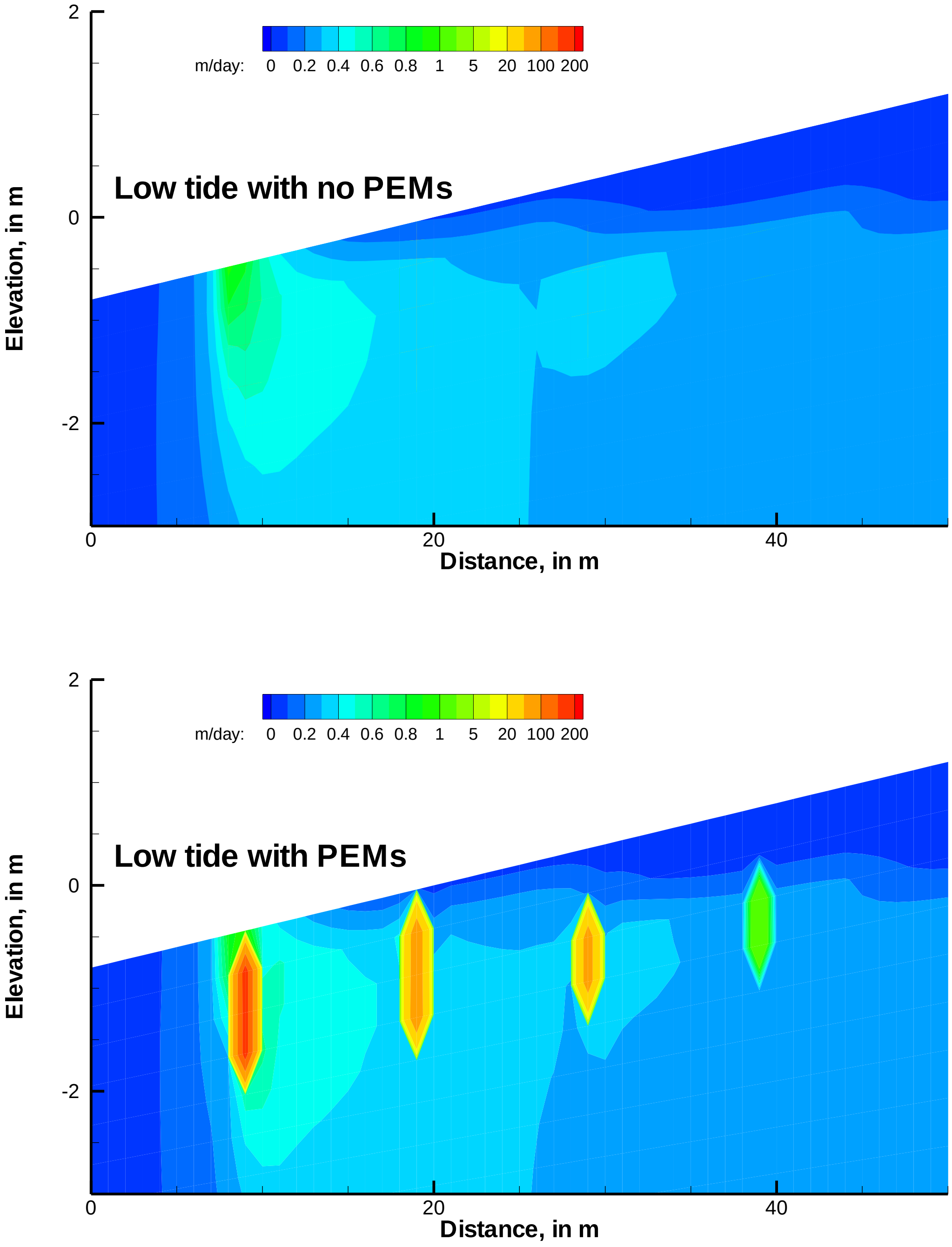


Fig. 29. Velocity distribution at low tide in homogeneous beach with and without PEMs (Engesgaard, 2008).

The effect of the PEMs was studied by Peter Engesgaard (2008) also in situations with a single gravel layer connected to the sea as well as with two overlapping gravel layers connected by PEMs. The difference between inflow and outflow with and without PEMs was marginal, but the PEMs had actually a negative effect.

Simulations with PEM's penetrating a more impermeable layer did show a slightly positive effect.

In the conclusions the following is stated in Peter Engesgaard (2008):

The numerical model still only resembles field conditions in an approximate way:

- *It is a 2D-model, which means that 3D flow phenomena around the PEMs (pipes) are not included. More importantly the pipes will over-represent the effects in the 2D model. The width of the model is implicity assumed to be 1 m, while in reality the diameter of the pipes in the model is only 0.08 m.* (Should be 0.06 m).

- *The discretization in the vertical direction (approximately 0.5 m) controls the thickness of the gravel/silt/clay layers. Whether these layers are thinner or thicker is not known.*

- *The hydraulic conductivity of the gravel layer in the base case is rather high. One may speculate whether this is a reasonable assumption given that small sand grains might fill up the pore space between the gravel yielding a hydraulic conductivity that is more comparable to that of sand. The hydraulic conductivity of the silt/clay layers is realistic.*

- *The PEMs all connect with the layers, which of course is not necessarily the case in a real situation.*

It should be noted that the first approximation leads to a significant overestimation of the effects (positive as well as negative) of the PEMs.

Finally the following is concluded:

- *The PEMs allow water and salt to flow slightly more rapidly into and out of the beach. In most cases they have an effect both ways sometimes resulting in a negative effect of the PEMs, sometimes in a positive effect. Despite this, the effects (positive or negative) are generally small when compared with the integrated outward or inward flux during a tidal cycle. In the base case (homogeneous beach) the extra outflow caused by the PEMs is on the order of 5%. Only in this case with a very low freshwater inflow are the changes in in-and outflow higher than this.*

- *There seems to be a positive effect of the PEMs in the case where a clay layer is present (and connected to the sea and thus acting as a low-permeable barrier to flow). This effect is most pronounced in the high and low freshwater inflow case, while in the base case it is less apparent.*

- *The PEMs do not appear to increase the width of the discharge zone significantly. Only in the cases of steady flow is the width increased from 1-2 m to 2-3 m.*

- *The effects of changing the hydro geological conditions (gravel/silt/clay layer, freshwater inflow) have a larger impact on inflow and outflows across the beach than having PEMs or not.*

8. Other beach drain systems

8.1 The Japanese gravity drainage system

Fig. 30 shows the principle of the system. A very permeable layer is placed under the surface of the sloping beach from a high level to the shoreface in the sea.

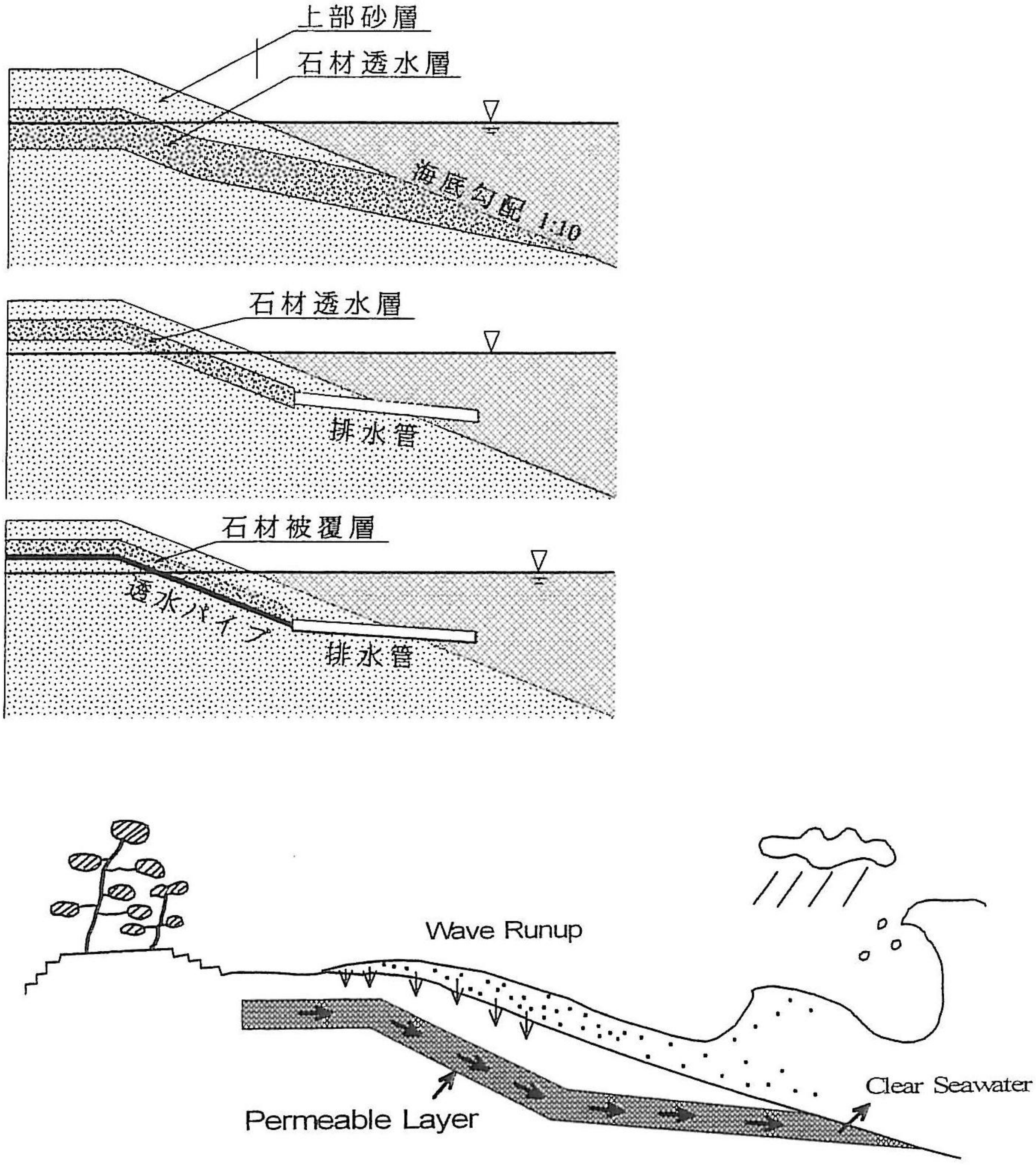


Fig. 30. Principle of the Japanese gravity drainage system for beach protection. (Shin-ichi Yanagishima et al., 2003).

The permeable layer will – at least under certain conditions – represent a zone of relative low pressure as it drains to the sea by gravity. This causes an increase in the infiltration of water from the wave run-up tongue. The down-rush on the beach surface is thereby reduced resulting in more settlement of sediments in the swash zone. The Japanese have linked the effect of the system to the run-up of wind generated waves rather than to the system's ability to lower the ground water table in the beach.

Field tests have demonstrated that the system cause more sand to be accumulated on the beach face, but not in big quantities. Design and maintenance of the drain layer outlet in the shoreface are not without problems.

8.2 The Danish Beach Management System, Danmark A/S (BMS).

This concept which is shown in Fig. 31 has been on the market for many years and is therefore tested at several locations.

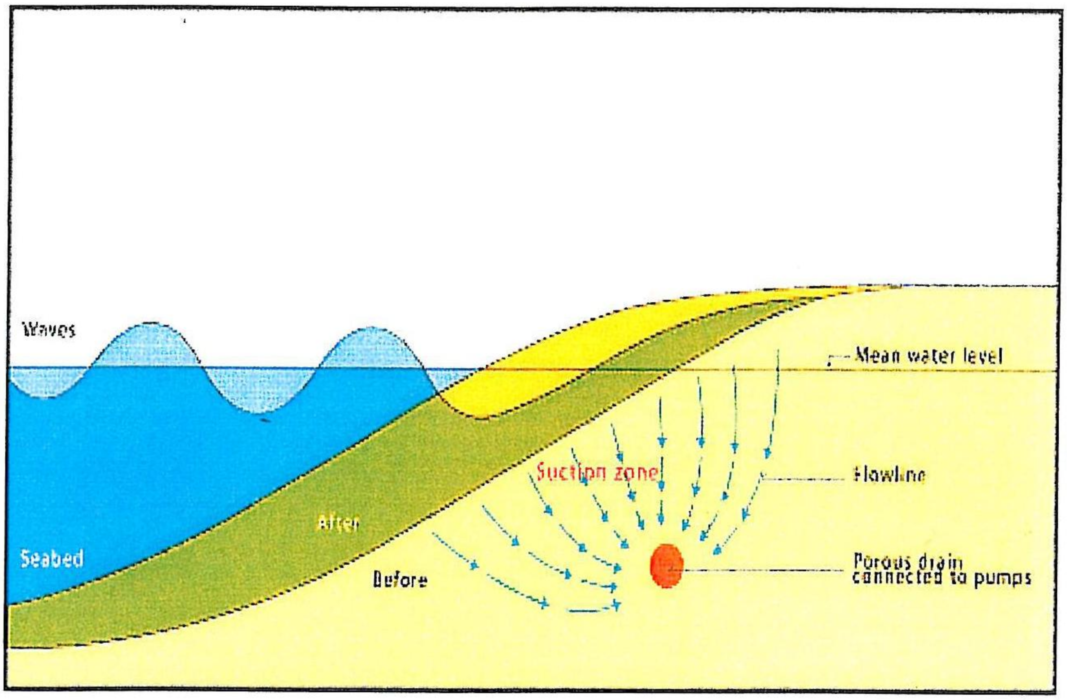


Fig. 31. Principle of the Danish Beach Management System for beach protection.

One or more perforated tubes are placed horizontally in the beach parallel to the shore. By pumping water from the pipes are generated a low pressure suction zone in the beach face which causes increased sedimentation. Field installations have proved the effect although the amount of accumulated sediment is very limited, in the order of 1-10 m^3 per metre beach dependent on the local conditions. Examples of recent installations are given in Bowman et al. (2007) and Ciavola (internet). The effect of the system seems too limited to be a measure of coastal protection. Moreover, the costs of electricity and maintenance of pumps are not negligible. Also, problems due to reduction of the permeability in the suction zone can occur. The system is applicable only on coasts with small tidal range.

9. The function of the PEMs

9.1 The explanation of SIC

SIC explains the function of the PEMs by their ability to lower the internal water table in the beach by increasing the drainage of the beach compared to the natural drainage. According to SIC increased drainage has three effects related to coastal protection:

- The downwards percolation (infiltration) of the uprushing water in the swash zone will increase. As a result less water will flow back to the sea on the beach surface resulting in a reduced sediment transport capacity during downrushing. The balance between the amount of sediments brought on to the beach in the uprush and that brought back to the sea in the downrush is therefore changed resulting in more settlement of sediments. This argument is undoubtly correct for coarser materials where capilary and viscous effects are negligible, but has recently been questioned as being a major effect in case of fine materials like sand (Nielsen, 2002).

- A drier beach – in space and time – increases the amount of sand transported by the wind (aeolic transport) along and across the beach to the dunes and hinterland. This is regarded a positive effect because dunes constitute a natural strengthening of the coast against retreat, although the sand is taken from the beach plane. This explained effect of a drier beach surface is correct.

- The increased drainage enhances the outflow through the shore face. This causes a wash-out of the finer material in the sand which again leads to a more permeable and thereby more stable beach face in front of the row of PEMs. The result is - according to SIC- accumulation of sand forming a small outsticking undulation on the coastline (a sand groin) in front of each PEM row, i.e. for every 100 m along the coast. The effect of the sand groins should be that some of the longshore sediment transport is caught resulting in built-up of the beach also in between the sand groins. Although documented by a photo by SICs from Gammel Skagen, see Fig. 32, it has not been possible for the author to identify such sand groins in front of the PEM rows, neither in the present project or in the two earlier tests mentioned in Section 1.1. However, many undulations (irregularities) at varying positions along the coastline have been observed during the test period.

Fig. 32. SIC photo of sand groin in front of a row of PEM’s (Poul Jakobsen, 1999).

Moreover, it should be mentioned that it was not possible by grain size analyses of sand sampled near and in between PEMs to detect any wash-out effect of the PEMs, cf. Section 6.3.

9.2 Physical analysis of conditions for a drainage effect of the PEMs

Drained water has to go through the drain pipe. In the case of a PEM it implies that the water has to go in and out through the slots solely over the 1 metre perforated height, see Figs. 1 and 2. The upper non-perforated 1 metre of the PEM has no function with respect to drainage and could therefore be omitted without changing the drain capacity. Two conditions in the soil must be fulfilled if a vertical drain like a PEM should function:

The first condition is the existence of a non-uniform deviation from hydrostatic pressure in the soil along the 1 metre perforated part of the pipe. Such deviation exists in homogeneous soil only if the flow has a vertical head gradient. This means no effect in case of horizontal flow of constant density water, and insignificant effect if the flow is nearly horizontal. Fig. 33 illustrates the condition in a downward directed flow field. In an upward directed flow field the flow in the PEM would be in the opposite direction, i.e. upward drainage.

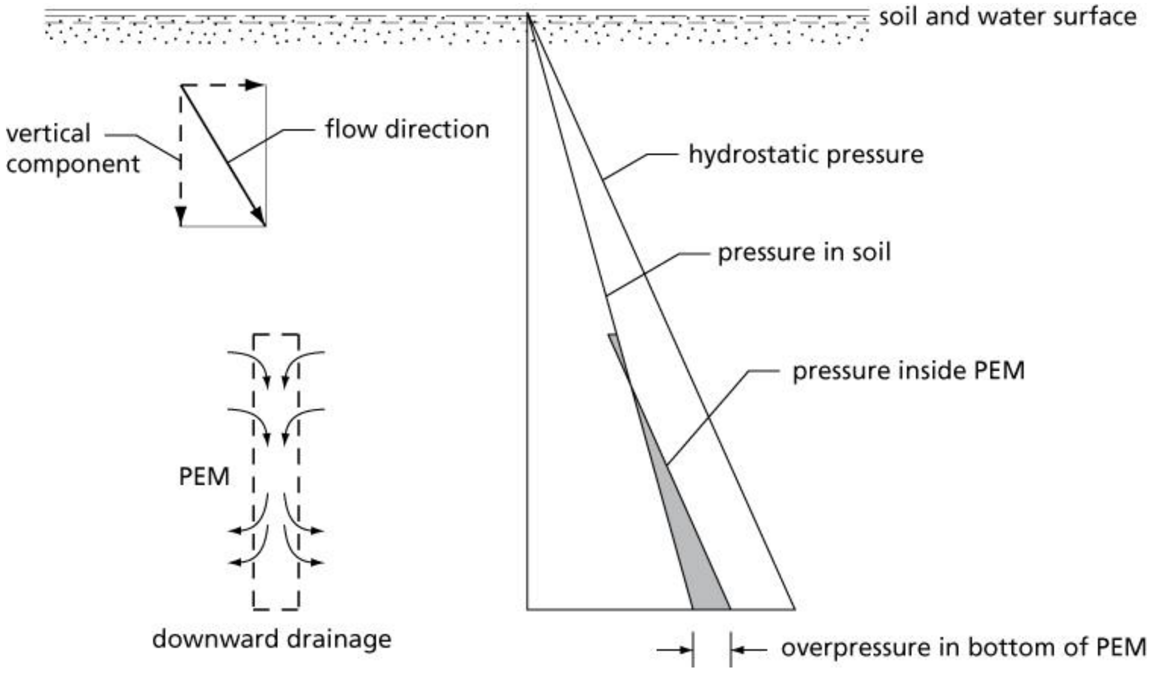
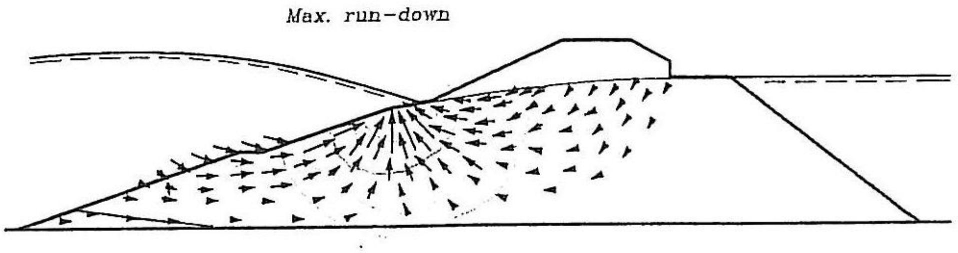


Fig. 33. Illustration of the function of a PEM in homogeneous soil.

The condition of a significant vertical head gradient within the one metre vertical perforated part of the PEM is fulfilled only when the intersection of the sea water level and the beach surface is close to and over the position of a PEM. When outside this the flow around the PEM will be almost horizontal and the drainage effect very limited. The limitation of vertical flows to the region of water level intersection with the surface is known also from wave action on rubble mound breakwaters, see Fig. 34. However, the existence of a fresh water head from land might change the situation somewhat in that vertical flows occur over a wide surface at high tide. This was also found by Dr. Peter Engesgaard in his simulations of flow fields, see Section 7.



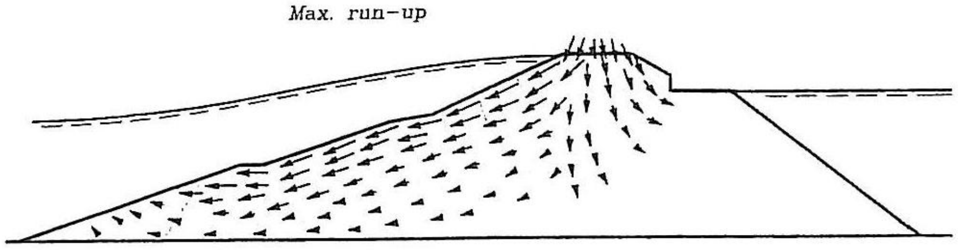


Fig. 34. Flow fields in a rubble mound breakwater showing that the more vertical flow directions are in the regions of interface between water level and structure surface.

In layered soil with different permeabilities in the layers, the condition of a vertical flow component is not necessary because head differences in the vertical direction can exist like in artesian flow below an impermeable layer. Fig. 35 illustrates the situation in a layered soil for the case of upward directed drainage.

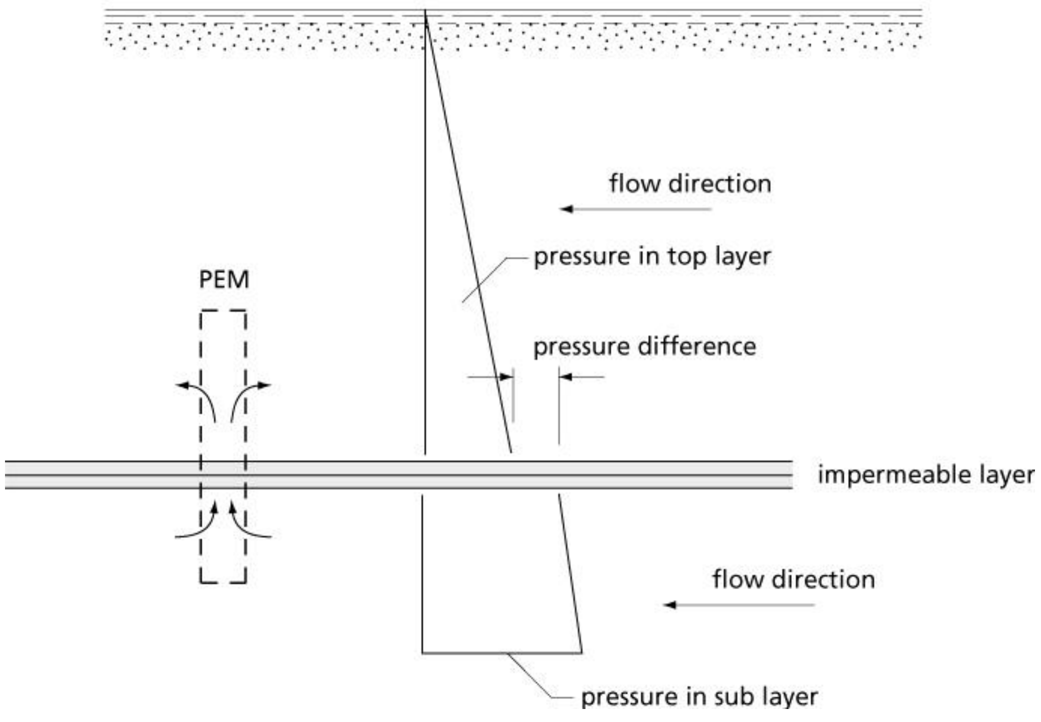


Fig. 35. Illustration of the function of a PEM penetrating an impermeable layer.

The second condition for functioning of a PEM is that the outflowing water from the perforated part of the PEM can be drained to a recipient , i.e. a region with a lower pressure – in this case to the sea during falling water level. Effective drainage can take place only if the perforated part of the pipe reach or penetrate a layer with a higher permeability than that of the surrounding soil. Further, this layer must extend to the shoreface, cf. the illustration in Fig. 36. However, this also implies that water is flowing more easily into the beach during raising sea water level, which has a negative effect.

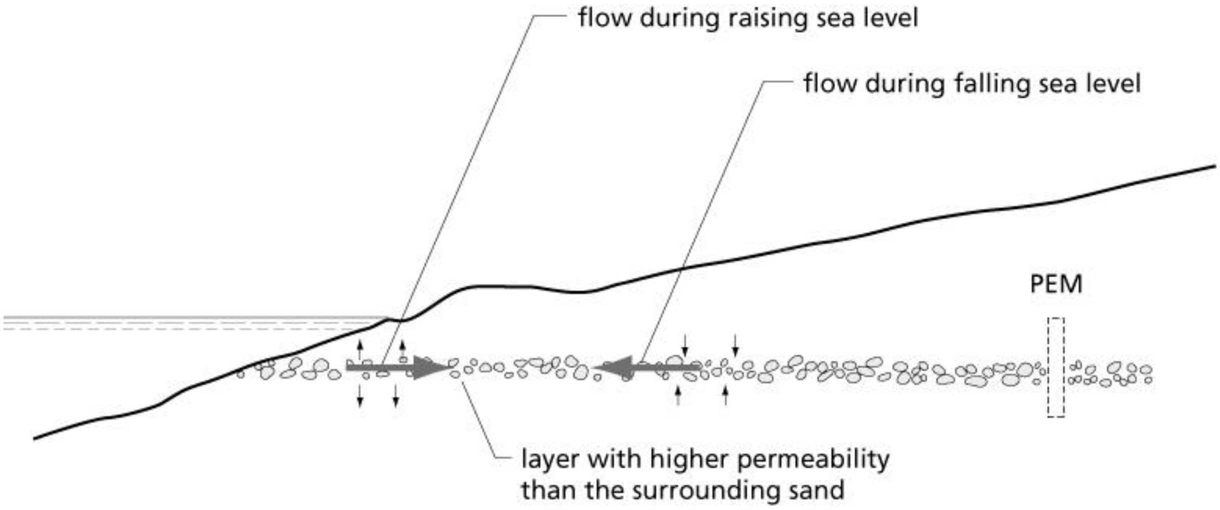


Fig. 36. Illustration of drainage through a permeable layer connected to the sea.

Further, it should be considered that a more permeable layer connected to the shoreface will drain water from the surrounding sand all the way along its surfaces and in that respect be much more efficient than the PEMs which have a very local and thereby relative marginal effect.

When the above explained conditions for a drain effect are fulfilled then the PEM will drain either in the upwards or downwards direction. However, upwards drainage which leads water to the upper layers in the beach might increase the outflow in the swash zone and thereby actually reduce the beach face resistance against the eroding effect of wave down-rush. This counter acts the effect of wash-out of fines in the sand, if any.

According to SIC, see Section 9.1, the increase in outflow should in principle stimulate the wash-out of fine materials in the sand and consequently have a positive effect. However, the wash-out effect could not be verified, cf. Section 6.3. Moreover, it is unlikely that wash-out takes place as the head gradients are very small and the in-situ gradiations wide.

The presented simple analysis of the function of PEMs does not include the effect of salt water intrusion and the inland ground water pressure. These effects have been included in the numerical simulations of Dr. Peter Engesgaard, see Section 7.

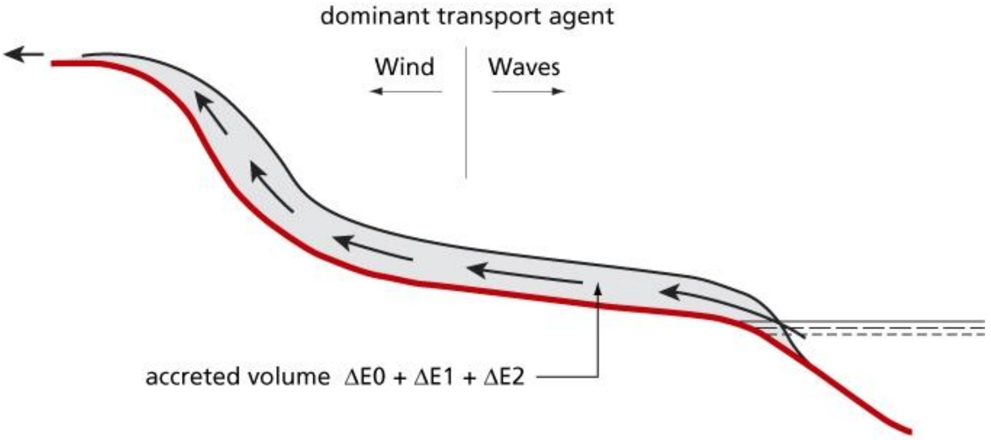
Based on the presented physical reasoning, visual field observations, comparison with effects of other beach drain methods, and the analyses made by Dr. Peter Engesgaard it is concluded that the PEM’s might under certain conditions increase the drainage and the built-up of the beach, but the effect will be small and therefore not sufficient as a coastal protection method on exposed coasts.

10. Method of presentation of surveys

The coastal profiles were surveyed every three months. From such quarterly profiles it is not possible to calculate the total amount of transported sand because balancing accretion and erosion can occur in between the surveys. Solely the net volume changes denoted Δ can be calculated.

The transport of sand is caused by waves and wind, cf. Fig. 37 which illustrates the two typical scenarios of accretion of erosion. The Δ-parameters are defined in Figs. 39.

Accretion in non-stormy periods



Erosion in stormy periods

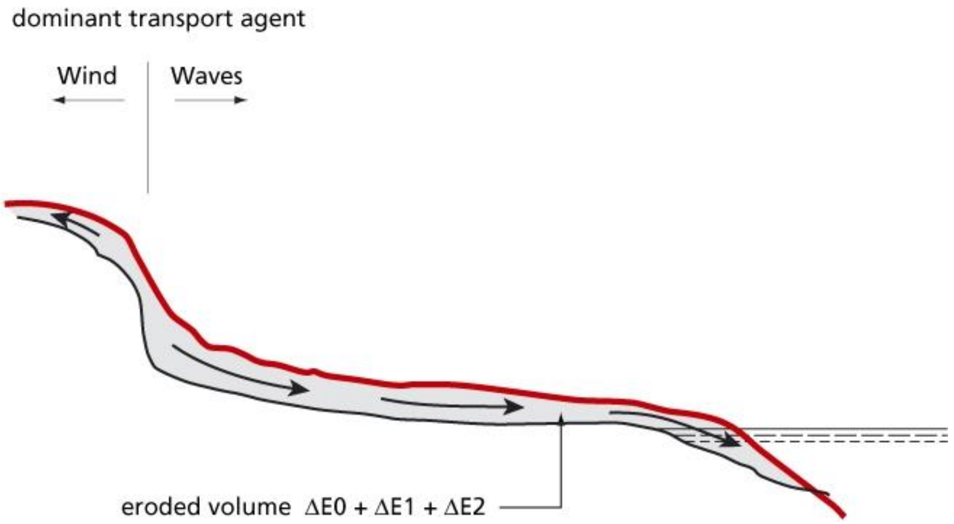
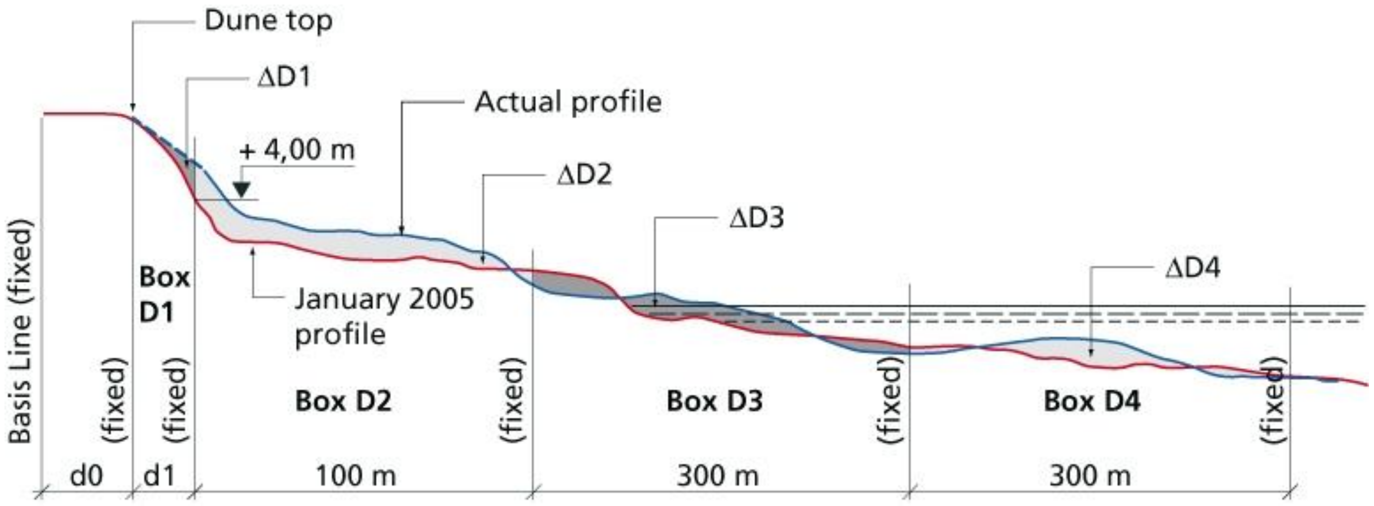


Fig. 37. Illustration of cross shore sediment transport by waves and wind.

No separation and quantification of the wave induced and the wind induced (aeolic) transport can be done as only the total accreated or eroded volumes can be calculated from the surveyed profiles. Moreover, the longshore transport of sediments by the wind from stretch to stretch cannot be estimated.

The changes in the surveyed coastal profiles are identified by use of the parameters defined in Figs. 38 and 39. The so-called D-parameters defined by SIC and shown in Fig. 38 are based on a separation of the profile in four fixed boxes of specific widths and fixed positions related to the positions of the level +4.00m intersection with the first surveyed profile of January 2005. The changes in sand volumes in each box, ΔD1, ΔD2, ΔD3 and ΔD4, are calculated. Besides this is calculated the mean surface level denoted MBL in the 100 m wide box as well as the changes in this level, Δ MBL. MBL is a kind of measure for the height of the inner part of the beach, but as the position of the 100 m is fixed, it will not reflect what is going on outside the 100 m, for example erosion of the dunes and accretion on the foreshore. The volume changes in boxes D3 and D4 have not been analysed in the present report as it is not possible to make a useful interpretation of the results.



Volume changes ΔD within each box are calculated positive in case of accretion and negative in case of erosion.

The mean surface level in Box D2 is denoted MBL.
The changes in MBL from January 2005 are denoted ΔMBL.

Fig. 38. Definition of D-parameters.

The so-called E-parameters defined by the author and shown in Fig. 39 separate the beach profile in three parts: The dune defined by levels higher than the dune foot at level +4.00m, the beach defined by levels between +4.00m and 0.00m (coastline), and the foreshore which is the zone from the coastline to a line in the sea 400m from the dune foot in January 2005.

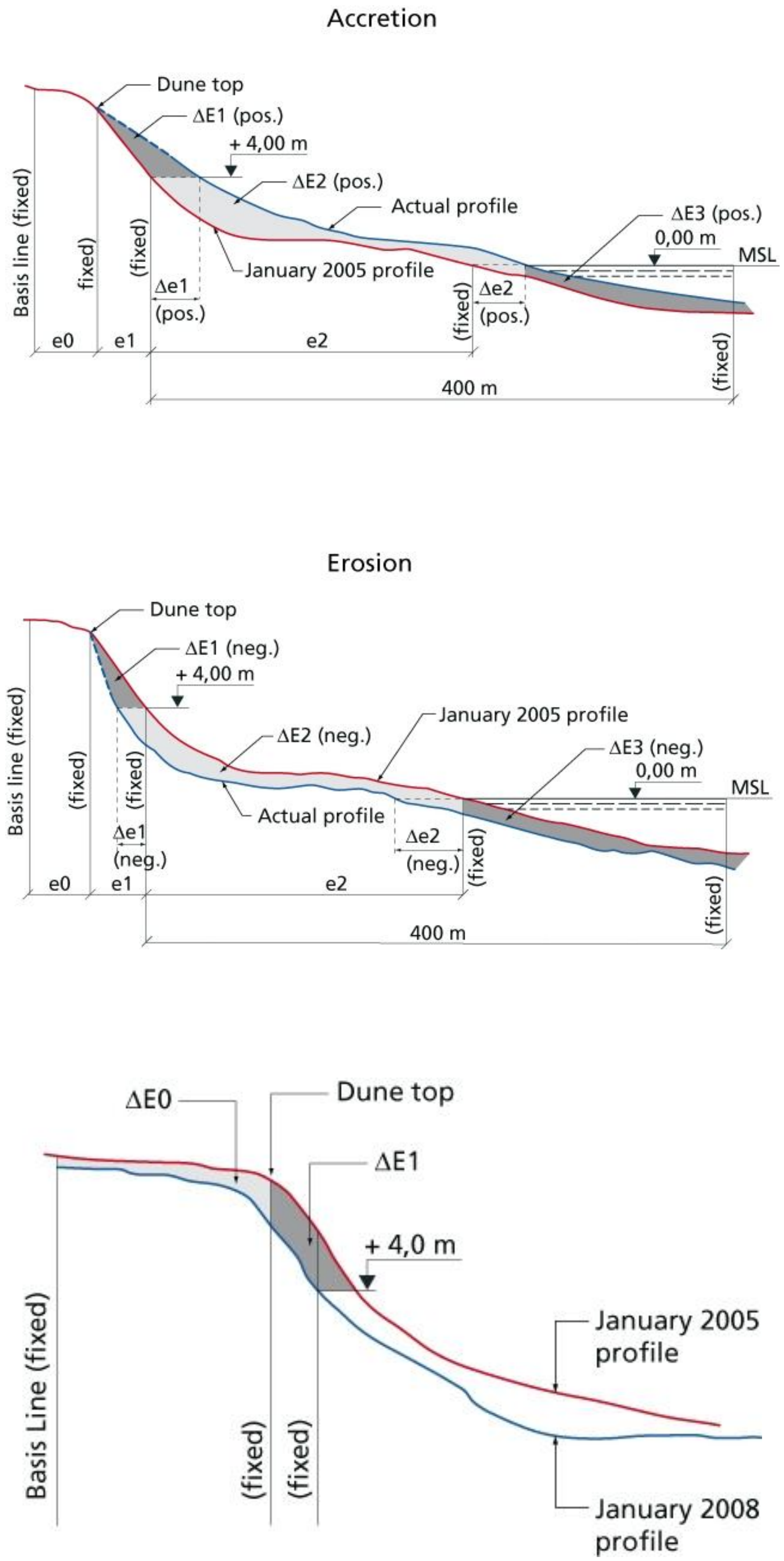


Fig. 39. Definition of E-parameters.

The changes in the position of the dune foot, $\Delta e1$, the changes in the position of the coastline, $\Delta e2$, and the change in the width of the beach $\Delta e2 + \Delta e2$), are identified as well as the changes in the dune volume, $\Delta E0 + \Delta E1$, and the beach volume, $\Delta E2$. Moreover, the changes in volume of the foreshore, $\Delta E3$, could be calculated as $\Delta D1+\Delta D2+\Delta D3-\Delta E1-\Delta E2$. However, $\Delta E3$ is not presented here as it is impossible to make a useful interpretation of the results.

Because the dunes over level +4.00 m were not fully surveyed except in January 2005 and January 2008 it has not been possible for other time intervals to estimate Δ E1 with high accuracy, because extrapolation has to be made between the highest measured point and the January 2005 measured top of the dune front face. It follows from this that the changes in the dune top volume $\Delta E0$ between the basis line and the fixed dune top line could only be calculated for the period January 2005–January 2008 as these two surveys were the only ones which included the profiles all the way to the base line. The error introduced by the extrapolation is very small and does not at all change the picture of the development of the coastal profile.

11. Results of surveys January 2005 – January 2008

11.1 Initial conditions

It is important to verify if any significant differences in the strength of the stretches with and without PEMs existed at the outset of the test. For this purpose is compared first of all the Mean Beach Level (MBL) which is the average height of the upper 100 m of the beach, see Fig. 40. The higher MBL, the more resistant is the beach. The other parameter which is used is the width of the beach, e2, see Fig. 41.

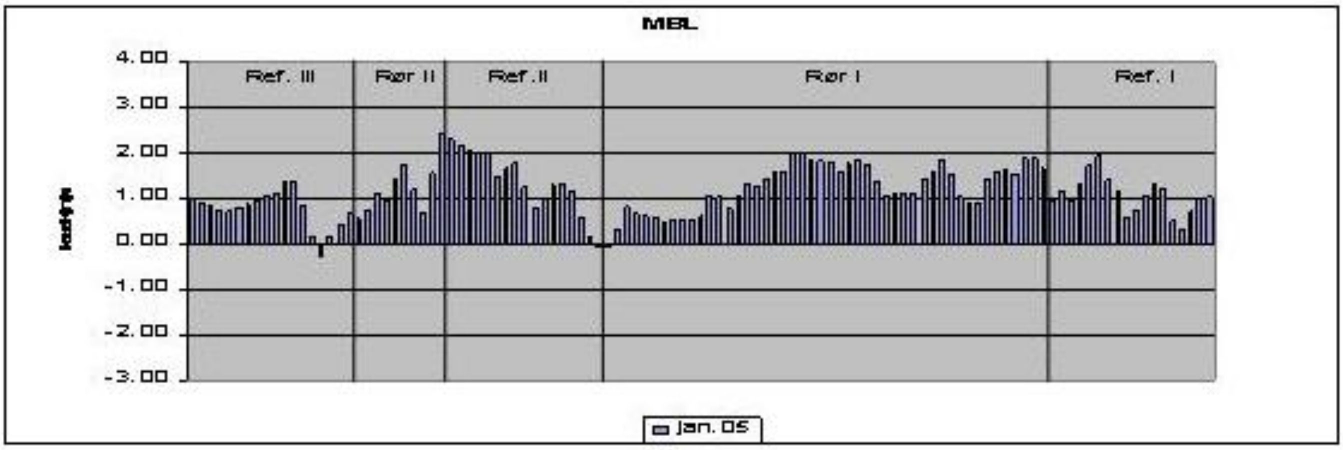


Fig. 40. Initial Mean Beach Levels of a 100 m wide zone seaward of dune foot position, January 2005 (KDI).

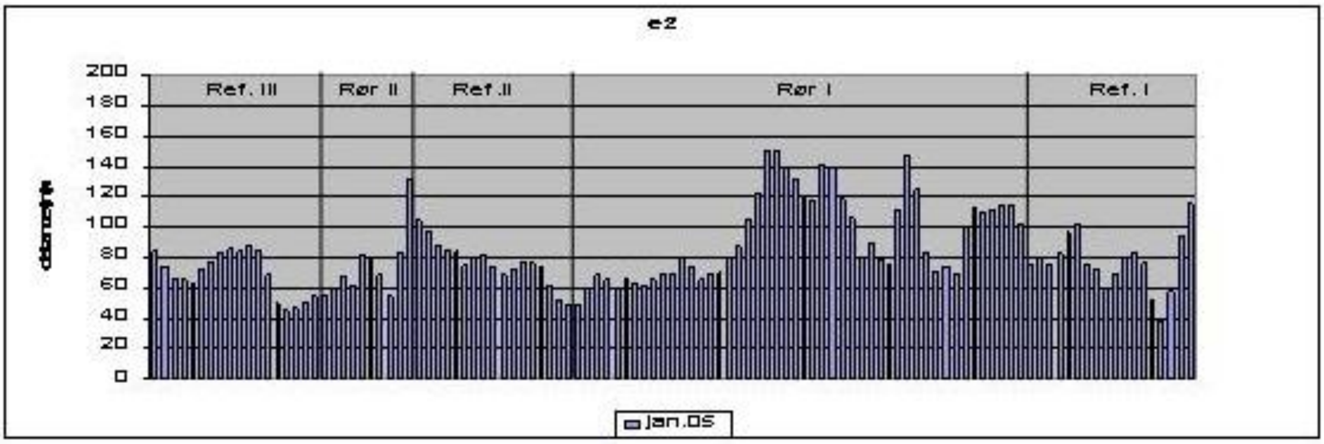


Fig. 41. Initial beach widths, January 2005 (KDI).

From Fig. 40 we observe two very weak stretches, one in the transition between Rør I and Ref. II, and one in the northern part of Ref. III. Even negative values of MBL are seen in a few stations. A fairly week stretch is also seen in Ref. I. More or less the same picture is seen in the variations of the beach width, Fig. 41. The low values of MBL coincide with the more narrow beaches. From this it is clear that at least two week stretches were present from the start of the test period. Moreover, the low beaches coincide with the holes in the outer bar, cf. Fig. 12.

11.2 Variation of MBL

The changes in MBL over the three years test period are shown in Figs. 42, 43 and 44. It is seen that the weak stretches with the low MBL values are maintained throughout the three years with the exception that low MBL values developed also in Rør II, signifying a down-drift shift of the eroded zone. This also coincides with the positions of the holes in the outer bar.

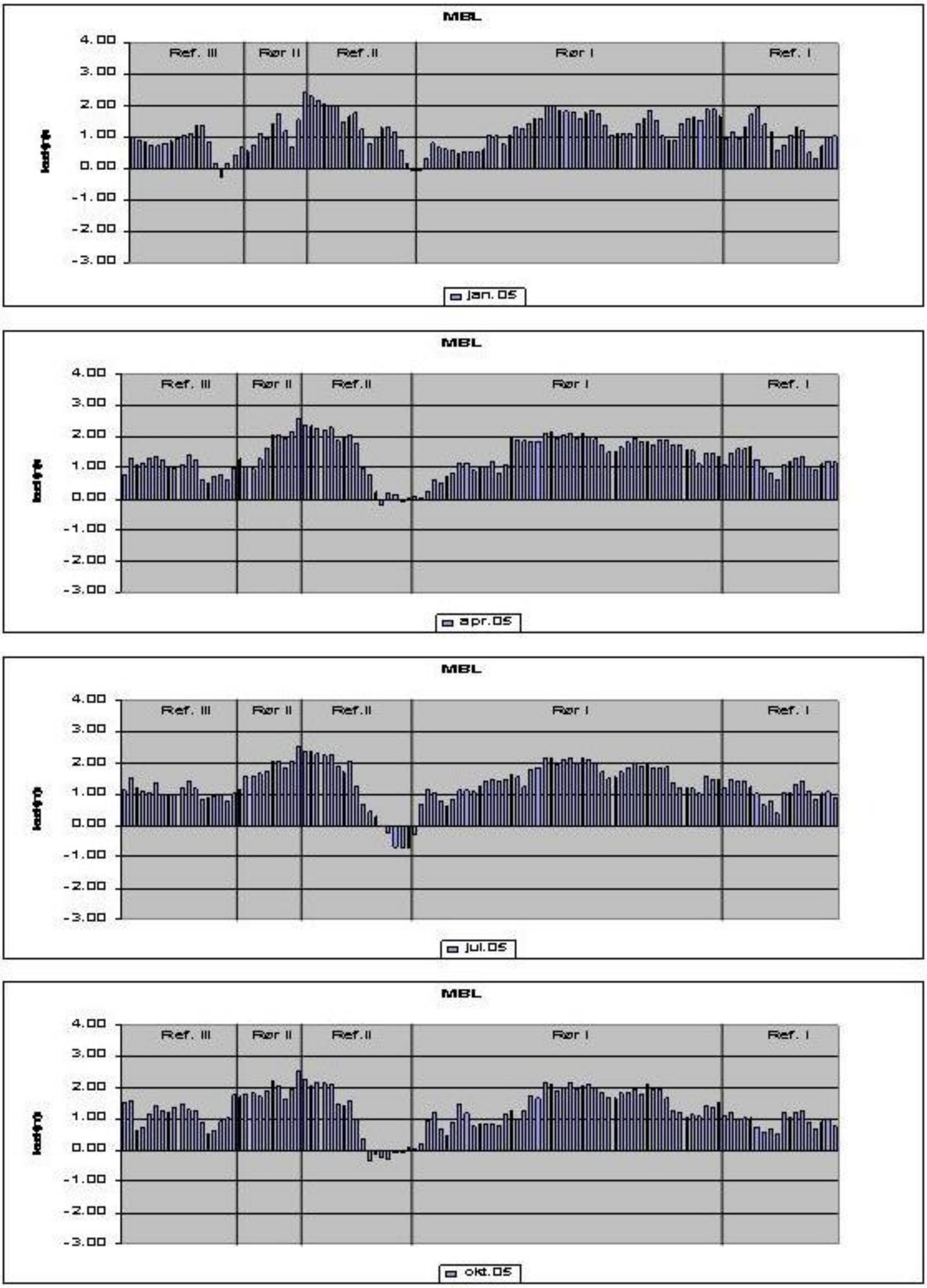


Fig. 42. MBL January, April, July, October 2005 (KDI).

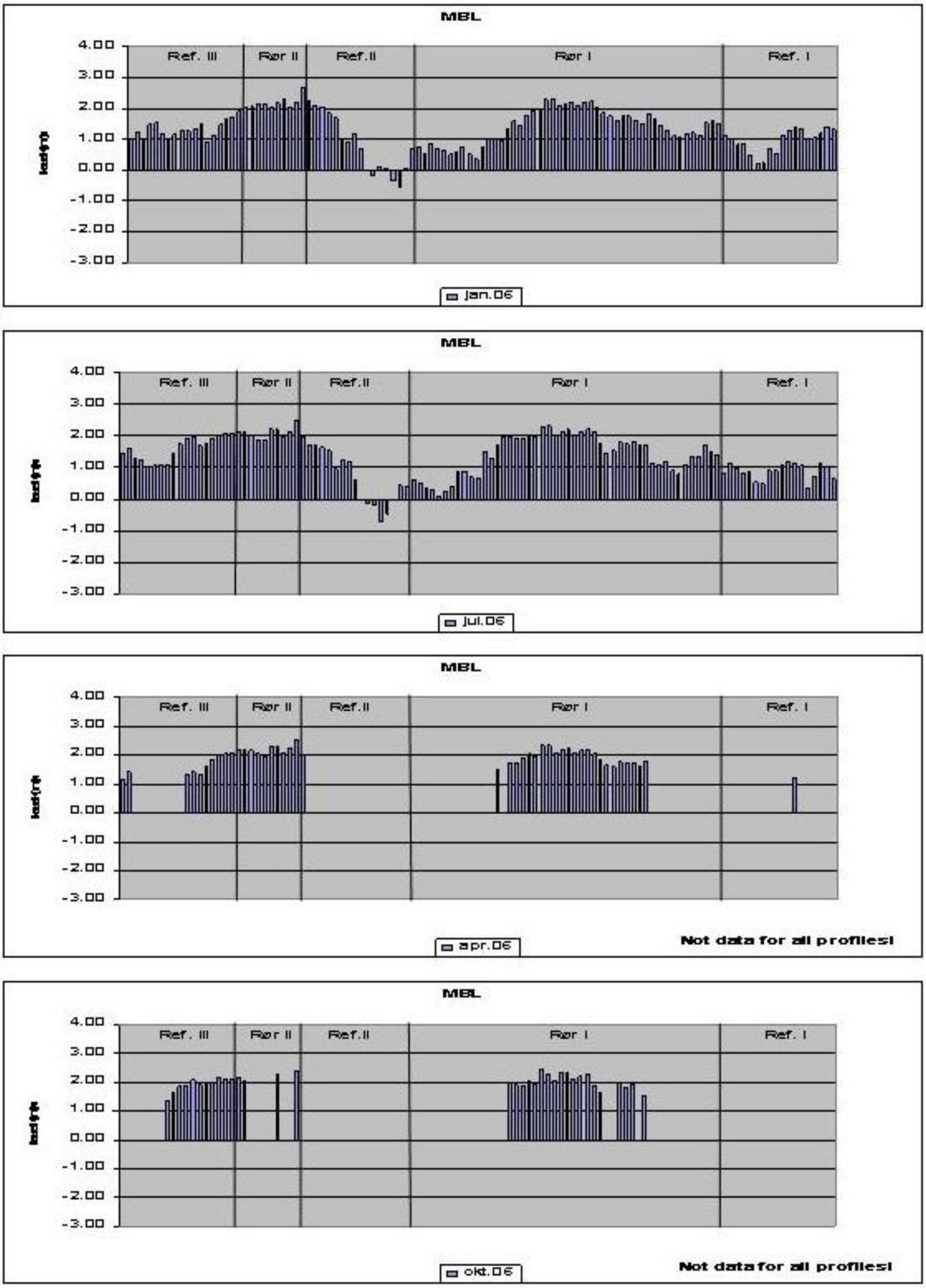


Fig. 43. MBL January, April, July, October 2006 (KDI).

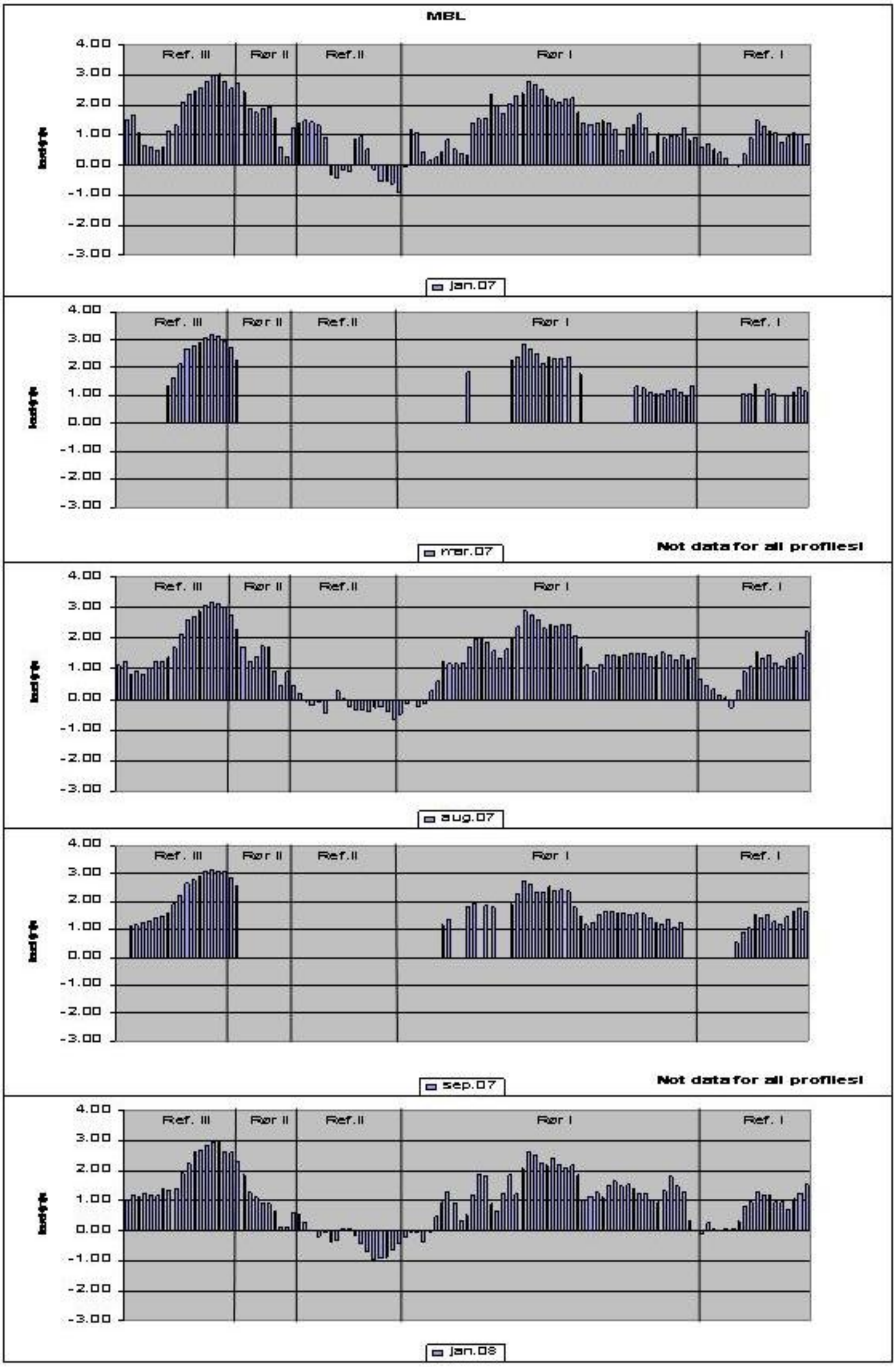


Fig. 44. MBL January, March, August, September 2007 and January 2008 (KDI).

11.3 Variation of beach width and coastline positions

The beach widths, e2 + Δe2, over the three years test period are shown in Figs. 45, 46 and 47. The figures also give a picture of the coastline positions.

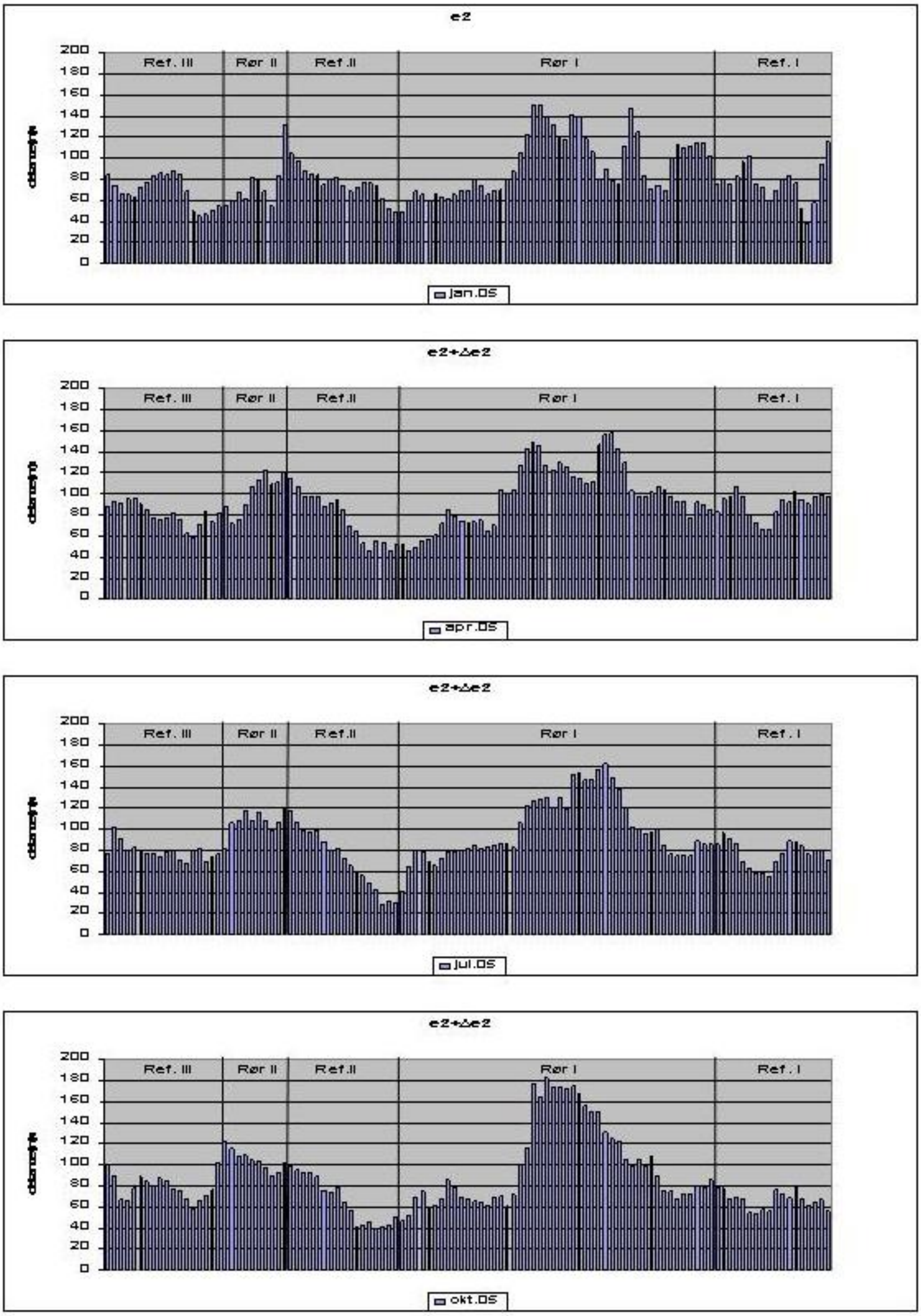


Fig. 45 Beach widths January, April, July, October 2005 (KDI).

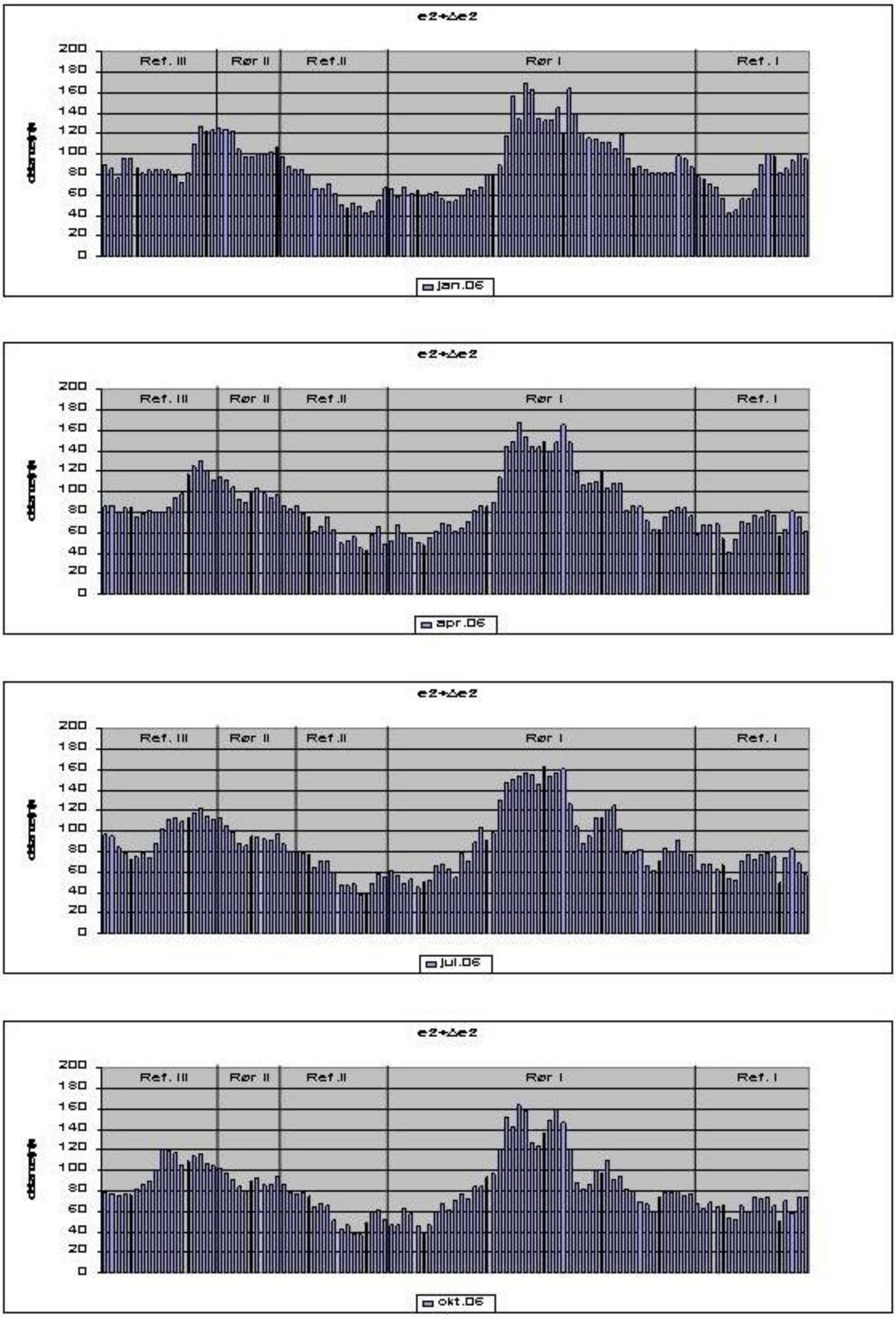


Fig. 46. Beach widths January, April, July, October 2006 (KDI).

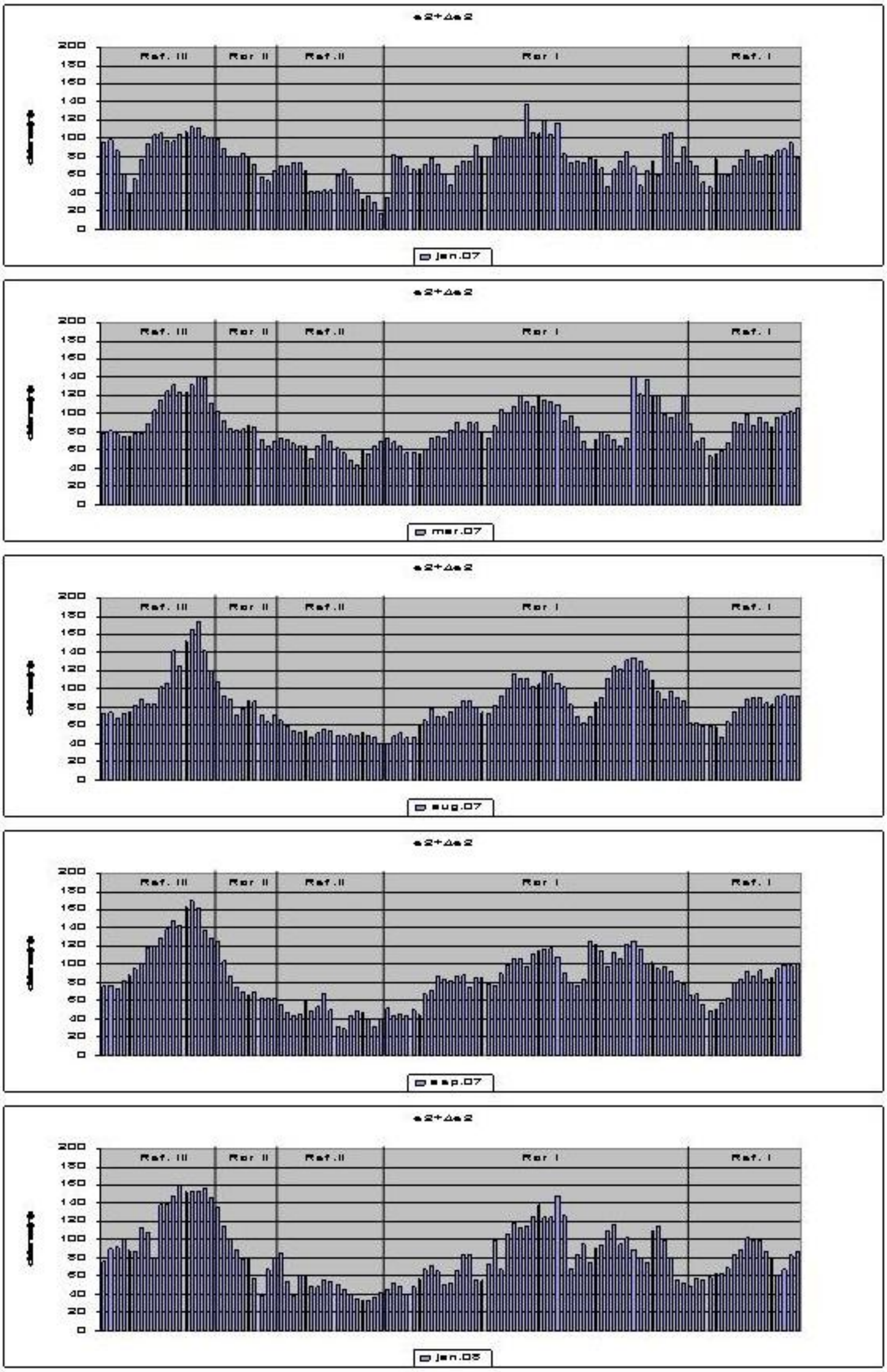


Fig. 47. Beach widths January, March, August, September 2007 + January 2008 (KDI).

From Figs. 45, 46 and 47 it is seen that the variation in beach width along the test site is more or less maintained from the initial survey in January 2005 until October 2006 except that a significant increase in beach width took place in Ref. III. Hereafter there was a general width reduction due to the January 2007 storms, but after that a built-up took place again, still with the more narrow beaches in parts of Ref. I and Ref. II although limited stretches of narrow beach also exist in the southern part of Rør I.

The changes in the coastline positions, $\Delta e2$, over the first half year, the first year, the two first years and the three years are shown in Fig. 48.

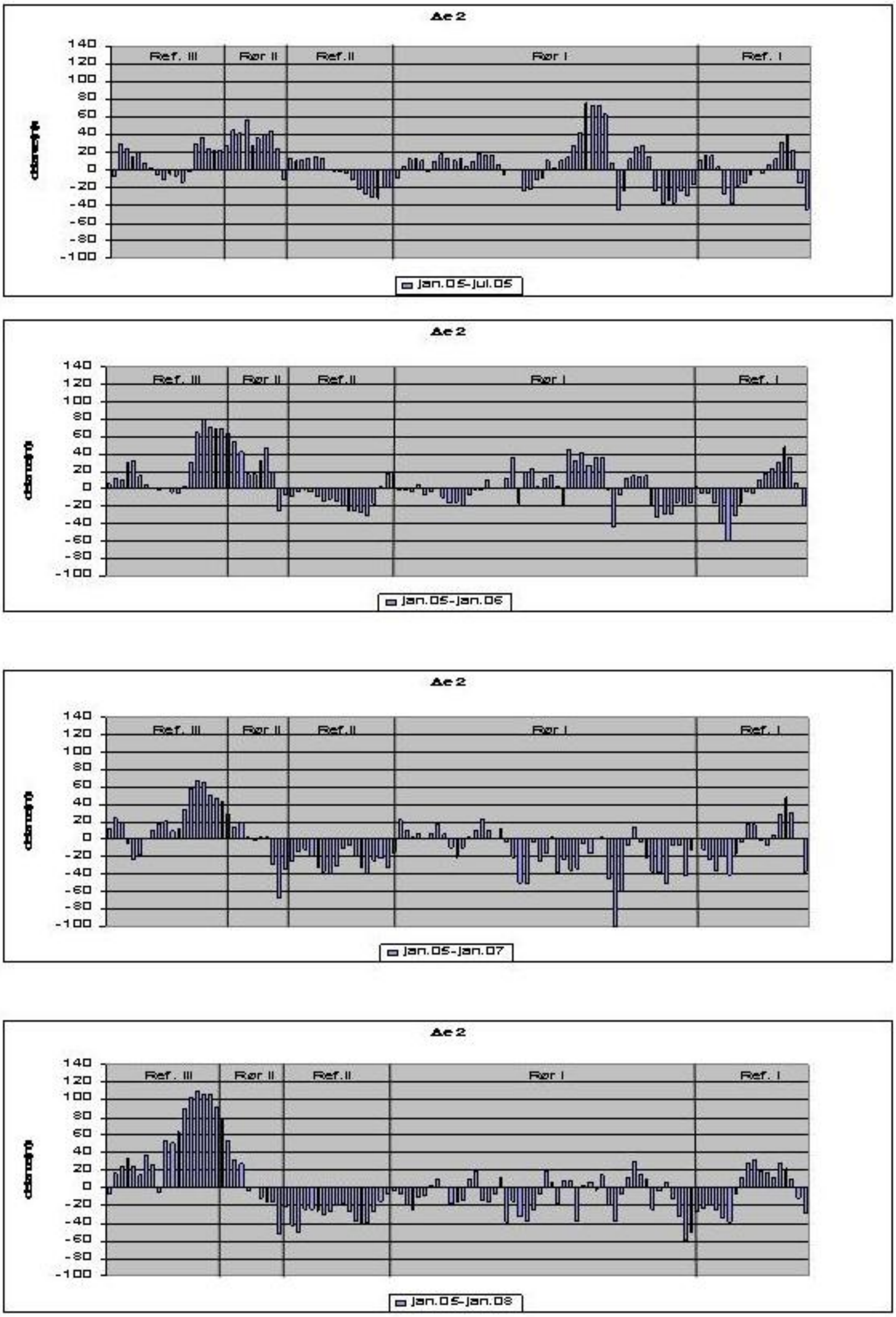


Fig. 48. Changes in position of coastline (level 0.00 m), (KDI).

Very large changes are observed in some stations. For example in Rør I a local retreat in a station is app. 60 m whereas in Ref. III a growth of up to app. 110 m took place. After the three-year period there are consistent coastline retreat in Ref. II and northern part of Rør II and southern part of Ref. I. Rør I shows both retreat and seaward growth. Significant seaward growth is seen in Ref. III. In conclusion there seems not to be a clear correlation between movements in coastline position and positions of drains.

11.4 Changes in dune foot positions

Seaward changes in dune foot positions (defined at level +4.00 m) are due to transport of sand by the wind from the beach plane to the dunes. Landward changes are due to erosion by the waves, cf. Fig. 37. The position of the dune foot $\Delta e1$, i.e. relative to the January 2005 position over the first half year, the first year, the two first years and the three years are shown in Fig. 49. A shoreward movement is observed for all stretches after the first year. The January 2007 storms caused a considerable retreat of the dune foot in Ref. I and Ref. II and in the boundaries of Rør I, i.e. the stretches of low MBL and more narrow beaches. In the middle of Rør I and in the entire Rør II there was shoreward movement of the dune foot until the spring of 2007 where erosion of the dune foot spread to Rør II. In the autumn of 2007 the erosion continued in Ref. II and Rør II.

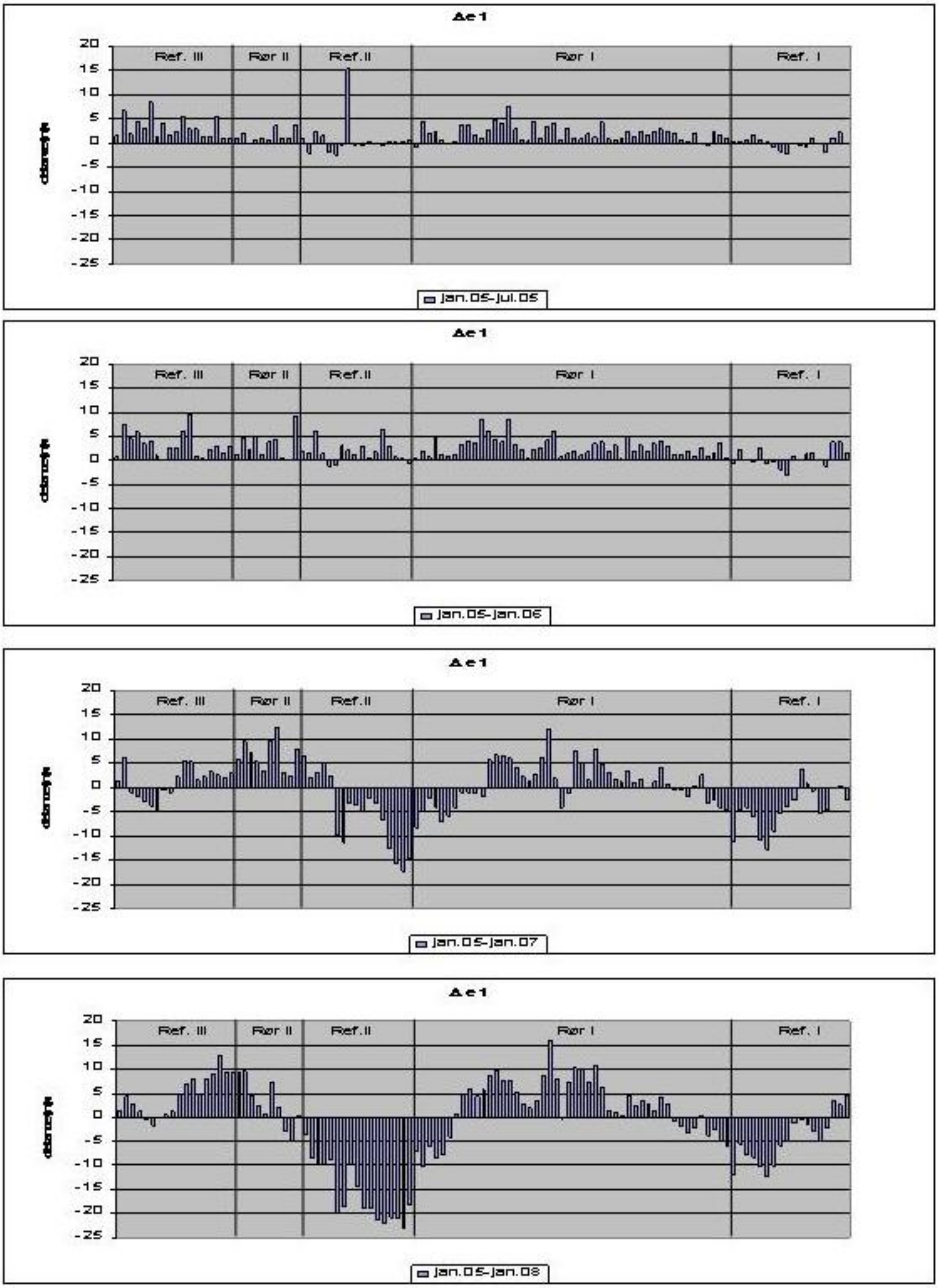


Fig. 49. Dune foot positions relative to the January 2005 position (KDI).

11.5 Changes in dune volumes

The approximate changes in dune volumes ΔE1 (and ΔE0) are shown in Fig. 50. After one year there was accumulation in all stations with few exceptions. In the April 2006 survey were seen very large accumulations in the northern part of Rør I and the very southern boundary of Ref. I. Erosion was only seen in three stations in Rør I (the locations of the natural wind holes) and one in Ref. I. After two years the large accumulations were still in the same locations, but erosion caused by the January 2007 storms was seen in some stations within all five stretches, most severely in Ref. II. In the third year it was mostly in Ref. II and southern part of Rør I that dune erosion took place.

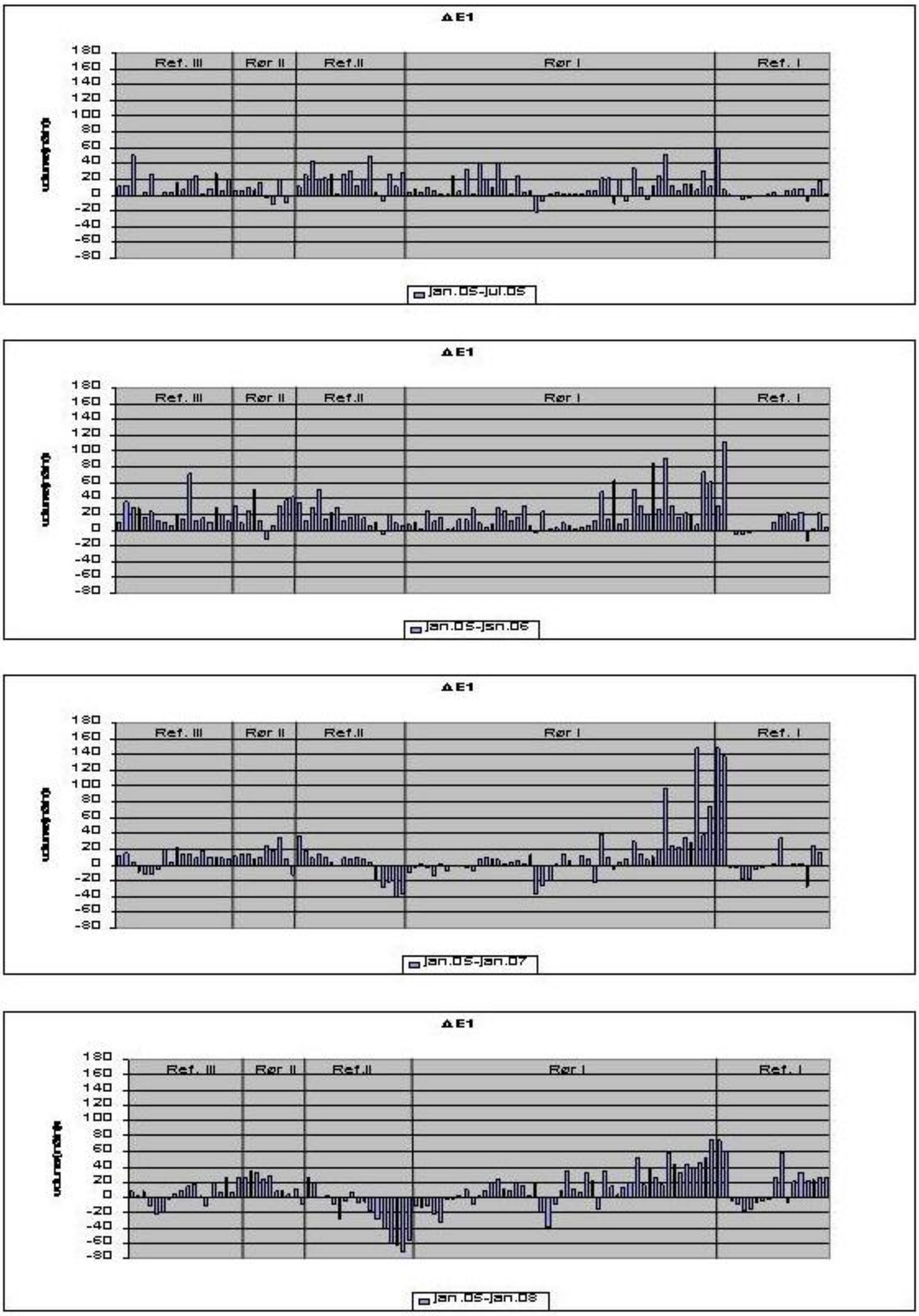


Fig. 50. Changes in dune volumes (KDI).

Related to the sediment budget it is important to include the wind-transported sand volume accumulated in the dune area landwards of the surveyed part of the coast. Two airborne laser scannings 24.5.05 and 27.3.07 make it possible to give an estimate of this volume.

The laser scannings were performed by Scankort A/S with a red laser. Spatial density was app. 1.3 m x 1.3 m and accuracy within 0.15 m. The volume calculations were done by KDI using Digital Terrain Model. The bias is estimated to be 0.1 m.

Figs. 51, 52 and 53 show the results of the scannings in terms of differential topographic levels in a 300 m wide strip landwards of the dune front (defined by level +4.00 m in January 2005).

By subtracting the surveyed eroded dune volumes $\Delta E1$, which are supposed to be eroded almost 100% by the waves, it is possible to get an estimate of the volumes accumulated in the hinterland. It is only a rough estimate because the 300 m wide strip does not cover entirely the hinterland where transport occurs. The results are given in Table 4. The stretch in the middle of Rør I containing the three large windholes is not included as this stretch is not typical.

Table 4. Accumulation of sand transported by the wind in the period 24.5.05 – 27.3.07 landwards of the top of the dune front of January 2007. Stretch with natural windholes in Rør I not included (KDI).

Stretch	Stretch length m	Accumulated m^3	Accumulation m^3/m coastline
Ref. I	1800	27.280	15
Rør I*	4500	120.356	27
Ref. II	1800	17.031	9
Rør II	900	16.053	18
Ref. III	1800	61.452	34

* 200 m with natural windholes not included.

The total accumulated volume is 242.172 m^3 over 22 months. This volume is basically removed from the beach.

It is seen that it amounts to app. 15-30 m^3/m except for Ref. II showing only 9 m^3/m. This figure seems too low although the beach was very narrow here. The explanation could be that the windholes of January 2007 facilitated a large sand transport to the land behind the 300 m strip.

The true accumulated volumes would be somewhat larger than the Table 4 values.

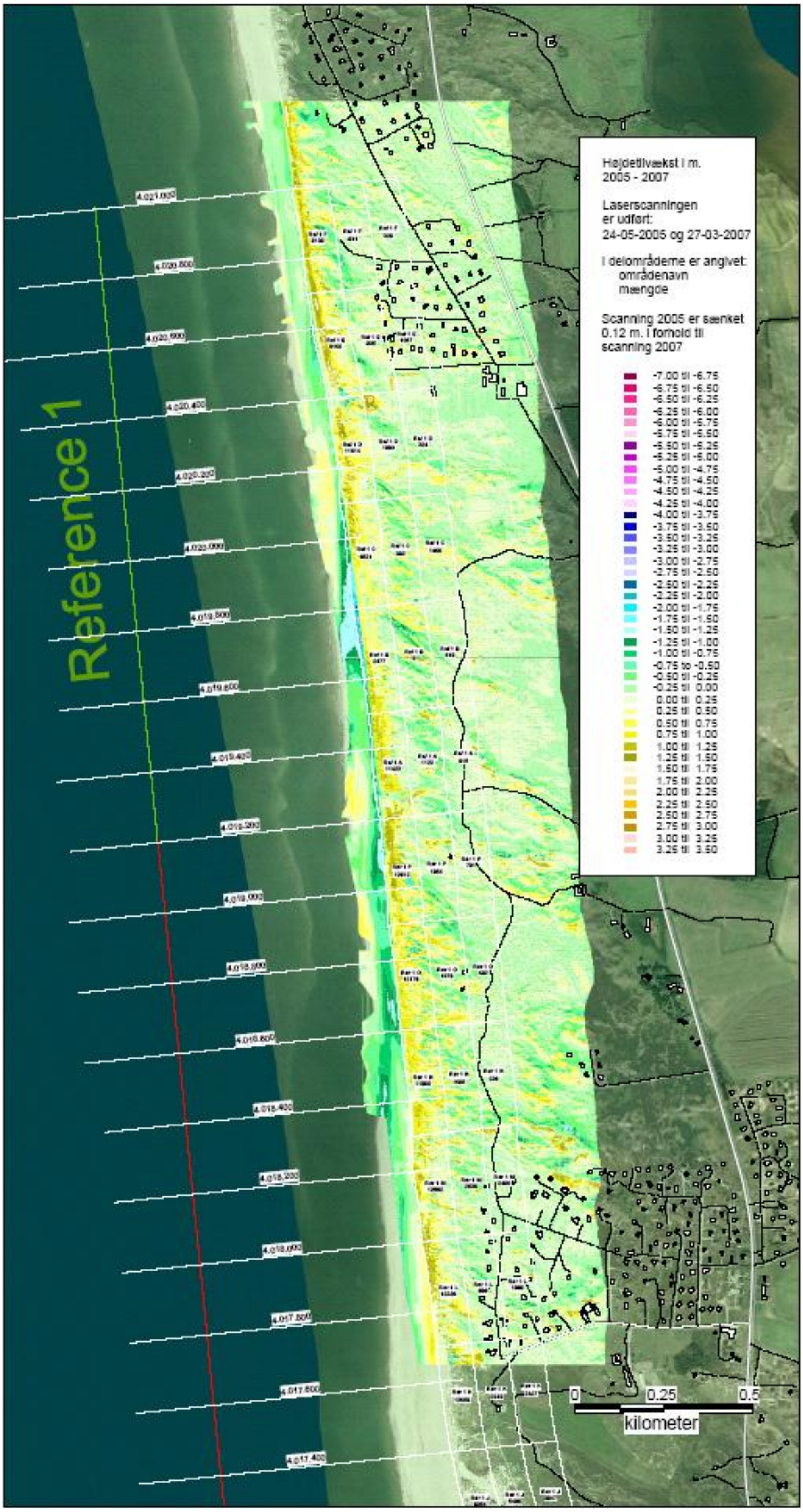


Fig. 51. Changes in dune volumes 24.5.05 – 27.3.07 in Ref. I and northern part of Rør I (KDI).

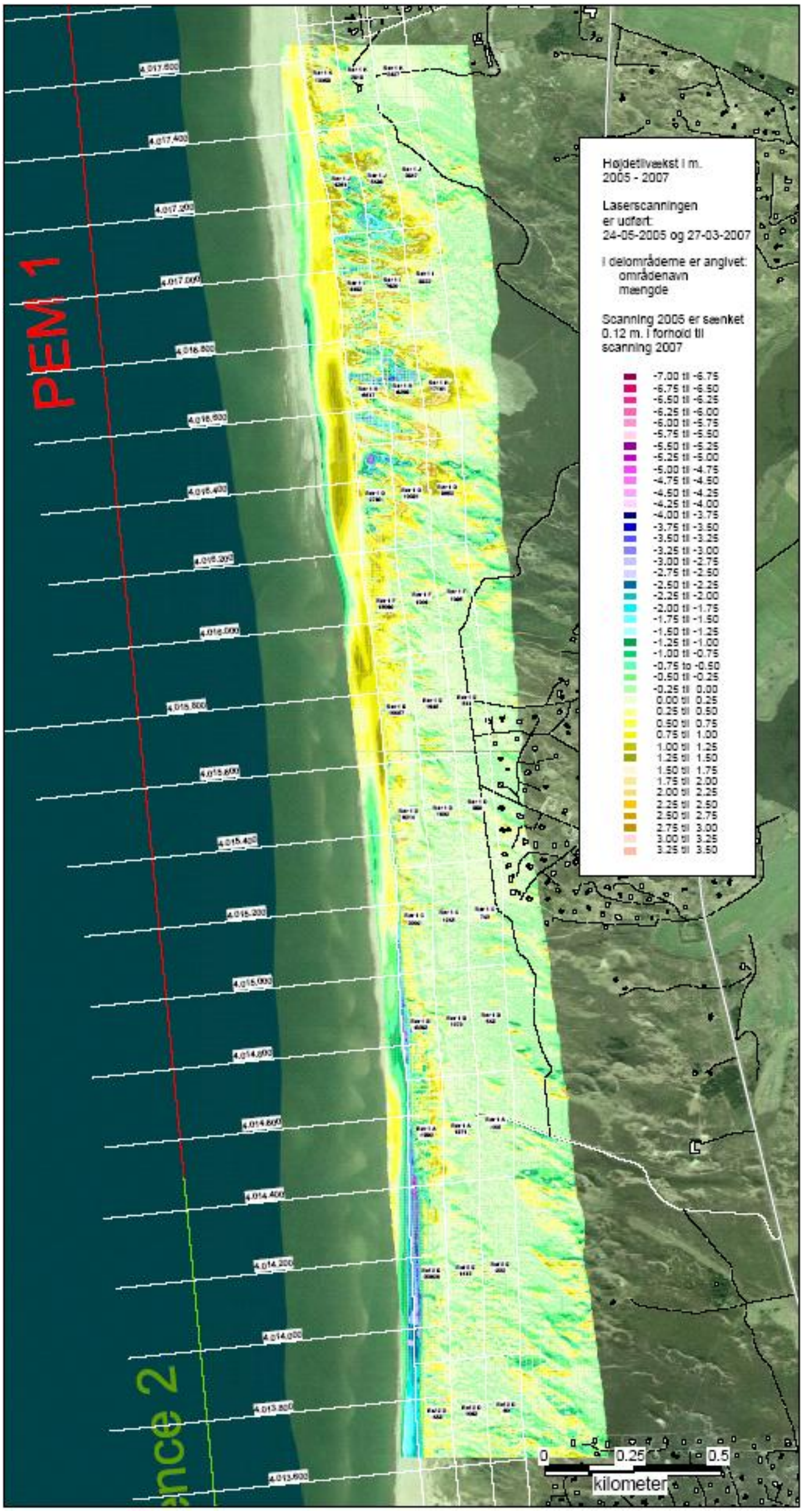


Fig. 52. Changes in dune volumes 24.5.05 – 27.3.07 in Rør I and northern part of Ref. II (KDI).

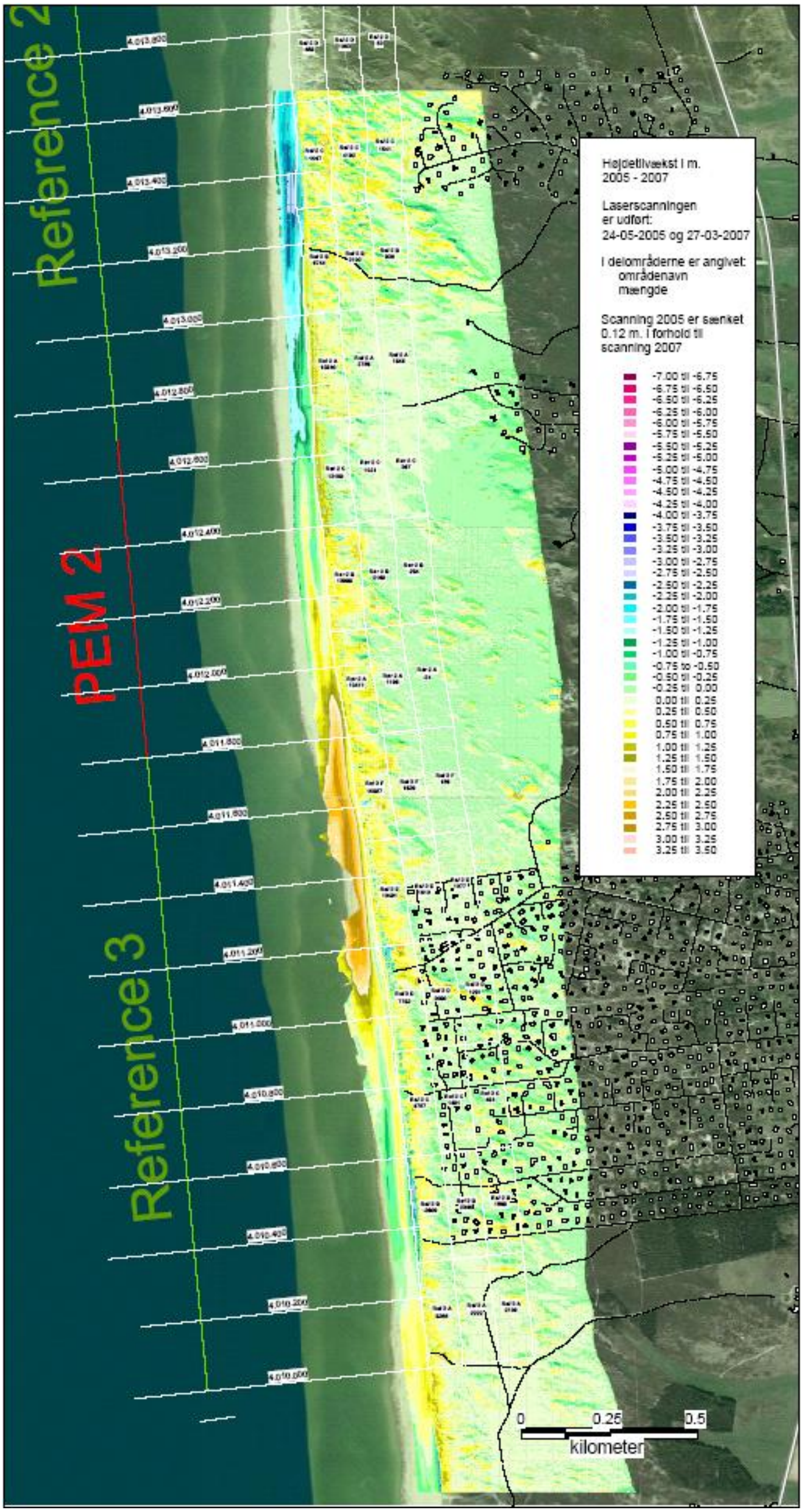


Fig. 53. Changes in dune volumes 24.5.05 – 27.3.07 in southern part of Ref. II, and in Rør II and Ref. III (KDI).

11.6 Changes in beach volumes

Fig. 54 shows the changes in the beach volumes.

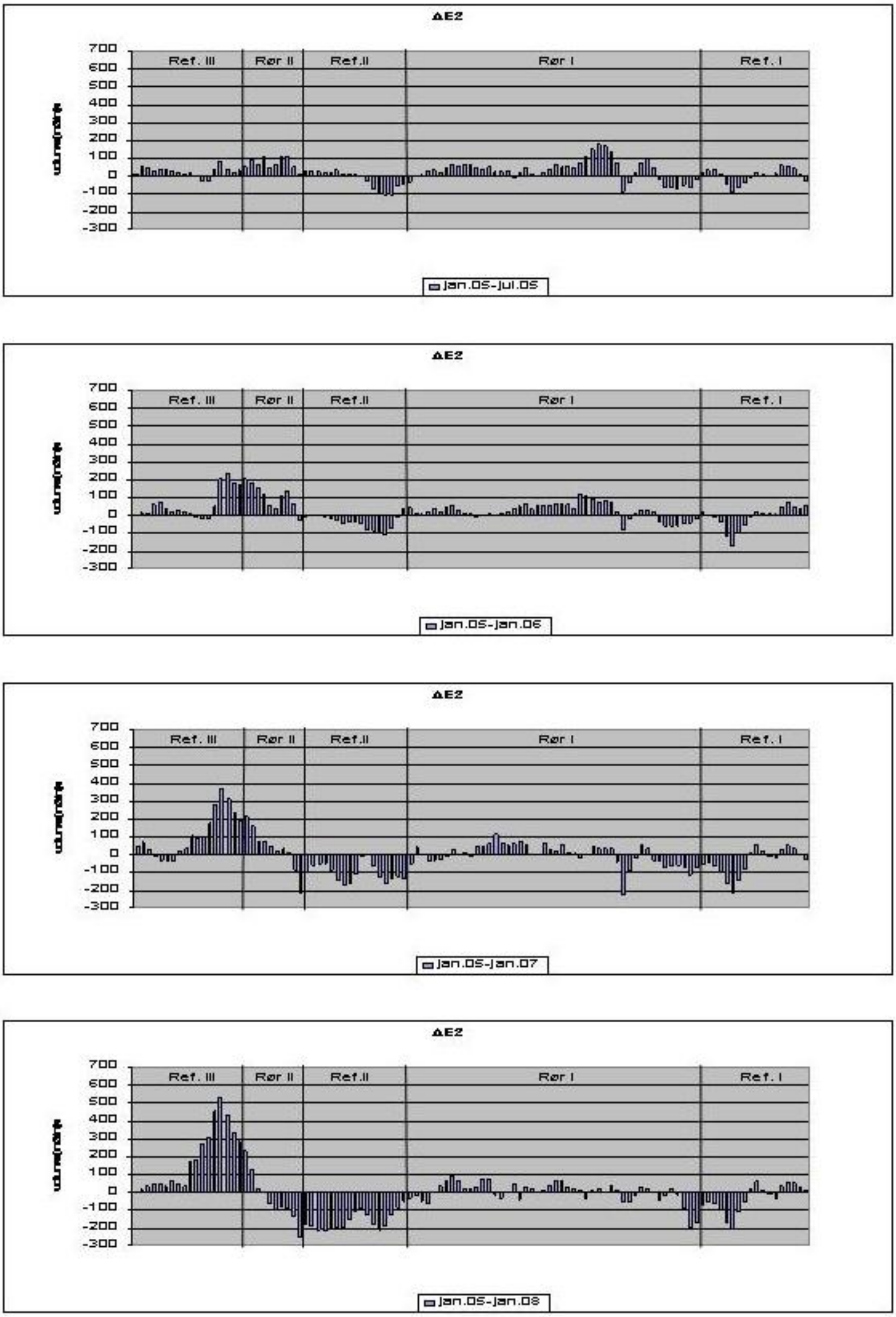


Fig. 54. Changes in beach volumes (KDI).

It is seen from Fig. 54 that increase in beach volume took place within the first half year mainly in Rør II and in some part of Rør I. Decrease took place in Ref. II and northern part of Rør I and southern part of Ref. I. This picture did not change much after one year except that more volume increase took place in Rør II and Ref. II. After two and three years was seen significant volume decrease in Ref. II and northern part of Rør II. Also northern part of Rør I and southern part of Ref. I showed reduced beach volumes. Only the northern half of Ref. III gained a significant amount of sand over the three years period.

11.7 Changes in beach plus dune volumes

The total changes in beach plus dune volumes are shown in Fig. 55.

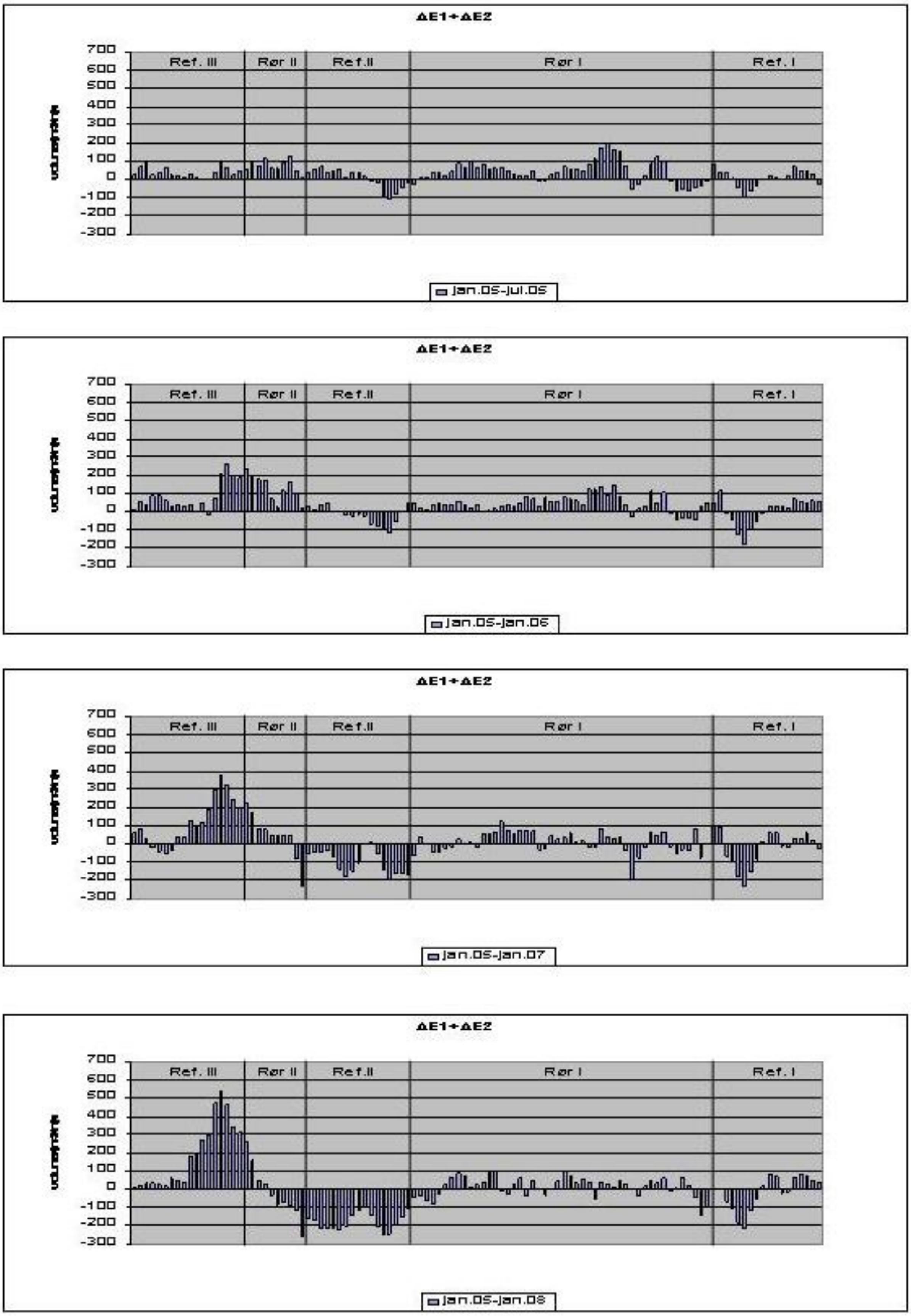


Fig. 55. Changes in total volumes of beach and dune (KDI).

11.8 Dune plus beach volume changes averaged over each stretch

Table 5 shows the approximate volume changes averaged over each of the five stretches. *It should be noted that averaging over a stretch is a significant simplification because large variations occur within each of the stretches, which shows that there is no good spatial correlation between beach behaviour and the presence or not of PEMs*, cf. also the figures in Sections 11.2, 11.3 and 11.7.

Table 5. Approximate average dune plus beach volume changes (ΔE1 + ΔE2) over half, one and two years, and ΔE0 + ΔE1 + ΔE2 over three years from January 2005 to January 2008.

Stretch	**m^3/m coastline**				**Total m^3 over stretch**			
	Jan 05-Jul 05	Jan 05-Jan 06	Jan 05 – Jan 07	Jan 05-Jan 08	Jan 05-Jul 05	Jan 05-Jan 06	Jan 05-Jan 07	Jan 05-Jan 08
Ref. I	7	3	-24	-5	12.640	4.620	-42.650	-9.216
Rør I	43	44	10	24	202.110	204.577	48.322	114.269
Ref. II	2	-20	-101	-187	4.087	-36.011	-181.673	-336.829
Rør II	73	126	44	-2	65.916	113.472	39.267	-2.187
Ref. III	37	80	115	202	66.600	143.110	206.800	363.894
							70.066	129.931

Sand volumes accumulated landwards of the survey base line (defined in Fig. 39) are not included in the volumes given in Table 5.

The most relevant figures in Table 5 are those giving the volume changes in m^3 per metre coastline because of the large differences in lengths of the various stretches.

It is seen that deposition took place in all stretches within the first half year, Jan 05 – July 05, but almost solely in Rør I, Rør II and Ref. III. After this there has been erosion in all stretches except in Ref. III where a very significant increase in deposition of sand took place.

The net result after three years is that the only stretch with PEMs where some deposition took place is Rør I. A very large deposition is seen in Ref. III, whereas very large erosion took place in Ref. II. Rør II and Ref. I both experienced a marginal erosion. No correlation between erosion/deposition and stretches with and without PEMs can be seen.

The net increase in beach and dune volumes over the total length of the test site amounts to app. 130.000 m^3 in total over the three years period.

The differences in wave climate over the various periods given in Table 5 are clearly reflected in the table values.

The table does not give a full picture of the changes in sand volumes on land because the considerable volume of sand transported by wind from the beach to areas behind the base line is not included, cf. Table 4.

The Table 4 values cover more or less the period Jan 05 – Jan 07 and can therefore be related to the Table 5 values for the same period. This is shown in Table 6. It is seen that the accumulated sand volumes in the hinterland are very significant.

Table 6. Approximate average volume changes in the surveyed part of the coast ($\Delta E1 + \Delta E2$) and in the hinterland for the period Jan 05 – Jan 07.

	m^3/m coastline			Total m^3 over stretch		
Stretch	Surveyed part (see Table 5)	Hinterland (Table 4)	Total	Surveyed part (see Table 5)	Hinterland (Table 4)	Total
Ref. I	-24	15	-9	-42.650	27.280	-15.370
Rør I	10	27	36	48.322	120.356	168.678
Ref. II*	-101	9	-92	-181.673	17.031	-164.642
Rør II	44	18	62	39.267	16.053	55.320
Ref. III	115	34	149	206.800	61.452	268.252
				70.066	242.172	312.238

* The Ref. II values probably too low

Table 6 illustrates the importance of catching more of the wind-transported sand and force it to accumulate at the dune front instead of letting it accumulate in the hinterland. Much more use of brush fences or other fences is recommended. The total volume of app. 312.000 m^3 supplied to the coast in the two years Jan 05 – Jan 07 corresponds to app. 40% of the app. 700.000 – 800.000 m^3 nourished on the outer bar in any two-years period over the years 2004 – 2007.

11.9 Correlation between the strength of the beach and eroding effect of the storms in the period October 2006 – January 2007

It is of interest to investigate if weak beaches (low MBL and small widths) are eroded more severely than stronger beaches. Fig. 56 shows the conditions/strength in July 2006 (i.e. before the stormy period) of the stretches given by the beach width e2, and the mean level MBL of the 100 m width of beach. In the same figure is shown the erosion of dune and beach ($E\Delta 1 + E\Delta 2$) between July 2006 and January 2007 (just after the storms).

From Fig. 56 it is seen that severe erosion (say $\geq$ 90 m^3/m) took place over limited distances in all stretches. It is also seen that – as expected – there is some correlation between low strength of beach (low values of e2 and MBL) and larger erosion, but this correlation is not very strong. This points to the fact that also other conditions than beach strength influence the erosion in storms. The most likely factor is the bar formation, especially the position of the holes, as explained in Section 5.5.

This means that a high and wide beach might loose the same amount of sand as a low narrow beach during a storm. However, after the storm the higher and wider beach will of course have larger residual strength.

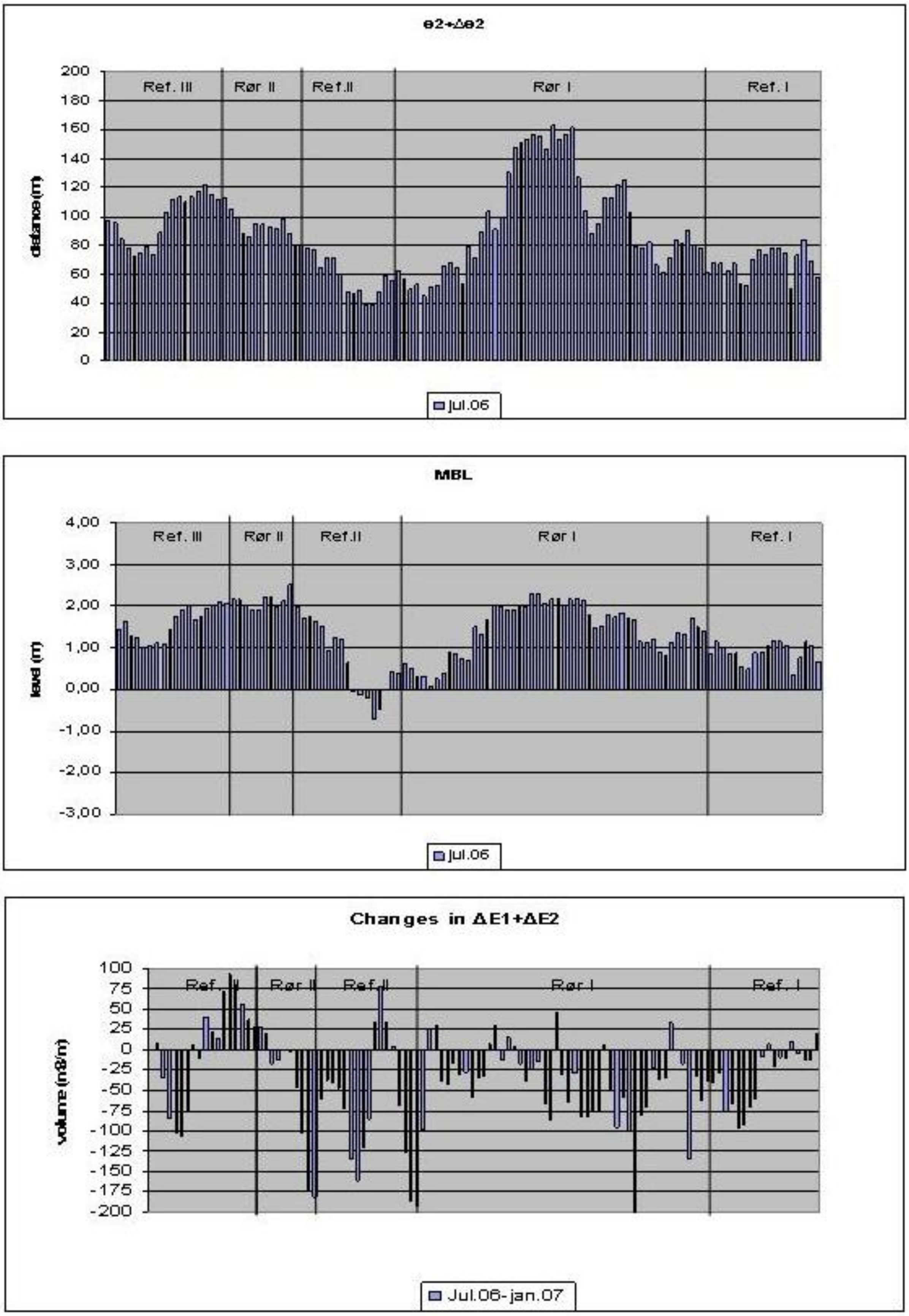


Fig. 56. Beach and dune erosion of the storms October 2006 – January 2007 in relation to the relative strength of the stretches per July 2006, in terms of MBL and beach width (KDI).

11.10 Summary and interpretation of survey results

The most important parameters for detecting the effect of the PEMs are the MBL, the beach width and the changes in volumes of dune and beach. The analyses revealed that over the three years there seems to be only a very weak (if any) correlation between the performance of the beach in terms of the variation of the parameters and the presence or not of the PEMs. On the other hand there seems to be a rather clear correlation between the beach performance and the presence of outer bars in the sense that coastal erosion took place mainly where the coast is badly protected against severe wave action because of holes in the outer bar. The eroded stretches were weak from the start of the project due to these holes, the positions of which did not change significantly during the project. Outer bars along the test site existed before the bar nourishment, but natural variability in the position of the holes prevented long-term erosion of a single stretch from taking place. Actually Ref. II which is the most eroded stretch in the test period was protected by a high outer bar few years before the start of the bar nourishment.

12. Overall conclusions

The objective of the three years field test was to verify the performance of the PEM system as a method for protection of sandy coasts against erosion. The investigations contained the following main activities:

a) Comparison of the performance of stretches with and without PEMs in terms of erosion/accretion, based on quarterly surveying per 100 metre of the coastal profiles.

b) Physical analyses of the function of PEMs.

c) Local field investigations of the function of the PEMs in terms of ground water pressure measurements in the beach in a row with and without PEMs.

d) Numerical simulations of the influence of the PEMs on the groundwater flow.

ad. a. The results of activity a) as presented in Chapter 11 showed no good correlation between stretches with and without PEMs with respect to erosion and accretion. Significant erosion took place both in some parts of stretches with and without PEMs. The largest erosion occurred where the beaches from the start of the study were low and narrow, most probably because of holes in the outer bar at these locations. During the three years study it seems that the spatial distribution of the significantly eroded parts of the beach remained correlated with the positions of the holes in the outer bar. If this is not the explanation of the significant erosion in Ref. II and southern part of Rør I then the only explanation must be a significant leeside erosion effect caused by the middle part of Rør I, i.e. a very negative effect of the PEM-system.

The most significant accretion of sand took place in Ref. III, i.e. a stretch without PEMs. SIC explains this as downstream natural sedimentation behind the sand spit formed in Rør II. The problem with this explanation is that such accretion is not seen in other stretches downstream of even more pronounced sand spits, see Figs. 45-47. Also the beach width in Ref. III increased even after the beach width in Rør II decreased to a size smaller than in Ref. III so this can hardly be due to downstream sedimentation. Moreover, there is no physical explanation of such a phenomenon, which – if true – should also benefit the eroded Ref. II and southern part of Rør I.

The conclusion of activity a) is that it could not be demonstrated that the PEMs had a significant effect on the coastal morphodynamics. The observed changes can be explained by other phenomena.

ad. b. The physical analyses as presented in Chapter 9 show that the PEMs can under certain conditions have a drain effect which might be positive in terms of increasing the accretion. However, the effect – even under the certain conditions – will be marginal. Natural variations in the coastal morphodynamics are very much larger than any influence of the PEMs.

ad. c. The recording of the ground water pressure variations in the beach as presented in Section 6.5 could not and was not meant to quantify the draining effect of the PEMs, but could only demonstrate if the presence of PEMs would change the ground water pressure variation. An analysis of Peter Engesgaard (2006) showed that the PEMs did not change the pressure variation, as the observed variations could be explained by the expected natural variation. SIC compared the pressure variations in a row of PEMs with pressures in neighbour wells and found small differences from an assumed linear variation of pressures between the wells. This assumption together with some uncertainty on the levels of the pressure transducers in the PEMs and the wells (which is comparable in size to some of the pressure differences) makes it difficult to be conclusive about the measurements. Anyway, the measurements cannot show anything about the effective draining effect in terms of more persistent lowering of the ground water table in the beach.

ad. d. The numerical simulations of the ground water flow as presented in Chapter 7 showed that the PEMs under certain conditions can have a draining effect, but this effect will be marginal.

In summary, the PEMs will under certain conditions have a positive effect in terms of increased drainage and perhaps related accretion of sand, but the effect will be marginal and almost impossible to detect on the background of the very large natural morphological changes of a beach like the actual one.

This point of view is supported by the fact that even a system like the BMS, see Section 7.2, based on active lowering of the beach ground water table by pumping, can only create accumulation of a very limited amount of sand which for sure is insufficient as a means against coastal erosion on exposed coasts.

Firnally, it is the opinion of the author that the effect of vertical drainage can be improved by using drains which are active over a larger depth instead of the lower 1 metre of the 2 metre pipes. Moreover, the drains should have a much larger surface than the 6 cm pipes and should be much more densily spaced. It is not clear why pipes are needed. Instead one could for example dig holes of two metre width and depth per every 100 m^2 and fill the holes with cobble stones surrounded by filter stones. The drainage capacity would be much larger than that of the PEM system. Still, the coastal protection capacity will be small.

13. Comparison of the efficiency of the PEM system with other coastal protection methods

13.1 Introduction

In the field of costal protection methods it is common to distinguish between the use of hard structures methods and soft methods. Conventional hard structures are groins, breakwaters, artificial reefs, headlands, revetments and seawalls. Soft methods are nourishment of beach, forshore or bars with sand or pepple gravel, scraping, artificial seaweed, and beach drainage. Combined use of the various methods is common. All methods have drawbacks either with respect to negative influence on neighbour stretches (typically downdrift erosion valid for hard structures sticking out from the coastline), or with respect to high initial costs (some hard structures) or high running costs (nourishment).

The negative aspects related to the various methods are well known today by professional coastal engineers and are therefore considered and dealt with in recent coastal protection schemes.

Coastal protection design is a very complicated field of engineering where the optimum solution varies a lot due to the variability of the environmental conditions. In one case the best solution might be a tapered groin system, whereas in other cases beach breakwaters combined with beach nourishment or maybe pure nourishment might be the optimum solution. It is therefore impossible – and not relevant – to compare the various methods with respect to efficiency and cost except if it is for a given location. The only exception would be if a method or a class of methods can be excluded as non-suitable for protection of eroding coasts. This is the case for the beach drain methods, especially the SIC – PEM system.

Further comparison of the PEM-system with the other proven methods seems therefore irrelevant simply because the PEM-system cannot substitute any of these methods. However, it might be possible to use the PEM-system as a supplement to one of the proven methods although the effect will be limited unless very specific geomorphological conditions and a relatively large tidal range exist.

The PEM-system has the qualification of being environmentally friendly. The only problem is the upsticking pipes in the beach which represent a hazard.

The various coastal protection methods are very shortly mentioned in the following sections. Further explanation of the function, design and experiences would take several volumes. Readers are therefore referred to the literature on coastal protection within the field of coastal engineering.

13.2 Limitations of the PEM-system

The effect of the PEM system is marginal seen in relation to coastal protection. Besides this a primary condition for an effect of the PEM system is the existence of frequent regular sea level variations. This means that there will be practically no effect of the PEMs on coasts with small tidal range like f.eks. the northern part of the Danish North Sea coast and the coasts in inner Danish waters and the Baltic Sea. This kind of limitation does not exist for other coastal protection methods, except maybe to some extend for the Japanese gravity drain method although it is merely based on drainage of uprushing waves in the swash zone rather than sea level variations.

13.3 Groins

Fig. 57 shows a groin system. Each groin catches sediments on the updrift side. The groins must be gradually shortened towards the terminal groin in order to avoid severe leeside erosion. Even so it is often necessary to protect a downdrift stretch of coastline against erosion by a revetment or a seawall.

Fig. 57. Example of groins. (Photo Hans F. Burcharth).

13.4 Detached shore parallel breakwaters

Fig. 58 illustrates a scheme of shore parallel detached breakwaters. Sedimentation in terms of salients or tombolos occur due to the reduced wave action behind the breakwaters. Downdrift erosion, as in the case for groins, might occur if the system is not tapered towards the terminal breakwater. Many different schemes exist with breakwaters sometimes placed almost at the shoreline, sometimes offshore, and sometimes non-overtopped, low-crested and even submerged. The schemes are often combined with beach nourishment.

Fig. 58. Example of shore-parallel detached breakwaters. (Photo Hans F. Burcharth).

13.5 Headland controle

The system consists of establishment of two or more local strong points on the coast, for example made as rubble mound structures. The coast in between will be eroded but in such a way that a bay shape develops. If the strong points mark a coastal cell, i.e. almost no sediment passes the headlands, then a kind of equilibrium shape of the coastline is formed. Beach nourishment might be needed in order to neutralize erosion, or shoreface protective structures might be built in parts of the bay. The eroded depth of the bay can be large, dependent on the distance between the headland and the direction and size of the waves.

13.6 Artificial reefs

The effect of artificial reefs is to dampen the wave action on the coast mainly by forcing the larger waves to break. The effect is basically the same as for bars.

13.7 Sea walls and slope protection

These are hard coast-parallel structures built in the zone between the beach and the hinterland in order to strengthen the upper part of the coastal profile. Scour protection in front of the structures is often needed on eroding coasts as steepening of the coastal profile might occur.

13.8 Perched beach

A coast-parallel threshold structure is placed at the front face of the beach in order to withhole the sand during wave actions. The function is rather limited.

13.9 Artificial seaweed

Artificial seaweed installed in a wide zone in front of the beach has the effect of dampening the waves before reaching the beach. The effect is rather small and maintenance problematic.

13.10 Beach drainage

See Sections 8.1, 8.2 and Chapter 9.

13.11 Nourishment

Artificial supply of sand or pebble gravel is used in many places instead of hard structures, or together with hard structures. There is normally no negative effects related to this method but it can be difficult and expensive to get materials, which has a suitable grain size which should not be smaller than that of the natural beach material.

13.12 Scraping

Sand deposited on the lower and middle parts of the beach in fair weather is moved to the upper part of the beach where it might give more resistance against eroding waves.

13.13 Placement of offshore wave power plants

A prospective method would be to place wave power plants on coasts prune to erosion as the waves behind the plants are significantly reduced. The method has not yet been applied and will only be used if/when wavepower is economically/politically feasible.

Aalborg 2 June 2008

Hans F. Burcharth
Prof. Dr. Tech., Dr. H.C.

References

Bowman, D., Ferri, S. and Pranzini, E. (2007). Efficiency of beach dewatering – Alassio, Italy. Coastal Engineering 54 (2007), pp 791-800.

Burcharth, H.F., 2004. Rapport om kystsikringsforsøg ved anvendelse af lodrette drænrør fra Skallerup Klit til Nørlev Strand i perioden maj 2000 til maj 2003. Hjørring Kommune, teknisk Forvaltning.

Burcharth, H. F., 2005. Evaluering af femårs forsøg ved Gl. Skagen med lodrette drænrør som kyststabiliseringsmetode. Skagen Kommune.

Ciavola, P. The new Beach Management System at Lido Adriano, Ravenna, Italy. (http://www.bms-on-line.com/index.asp).

CIRIA report 169. Manual on the use of Rock in Hydraulic Engineering, The Netherlands.

Engesgaard, P., 2006. Effect of Vertical Drains on Tidal Dynamics in Beaches. Geological Institute. University of Copenhagen.

Engesgaard, P., 2008. Simulation of tidal effects on groundwater flow and salt transport in a coastal aquifer with artificial drains (Pressure Equilibrium Modules). Department of Geography and Geology, University of Copenhagen.

Jakobsen, P. and Brøgger, C., 2007. Evaluation of the function of the PEM system. SIC.

Kriebel, D.L., Dally, W. R. and Dean, R.G., 1986. Beach profile response following severe erosion events. COEL-86/016. <University of Florida, U.S.A.

Nielsen, P., (2002). Shear stress and sediment transport calculations for swash zone modeling. Coastal Engineering 45 (2002), pp. 53-60.

Shin-ichi Yanagishima, Kazumasa Katoh, Naoto Iwasa and Yoshiaki Kuriyama (2003). Development of Gravity Drainage System for Beach Protection. Proc. 2nd Int. Conf. Asian and Pacific Coasts, APAC 2003, Japan.

Shin-ichi Yanagishima, Katsuhiro Sato, Takashi hara, Masafumi Saito, Naoto Iwasa, Kengo Hori, Iwao Hasegawa and Junzo Hasegawa (2003). Development of Beach Stabilization by Gravity Drainage System. Report of the Port and Airport Research Institute, Japan, Vol. 42, No.1 (in Japanese).