Field tests with vertical perforated drainpipes used for beach protection at Southern Holmsland Barrier on the Danish North Sea Coast

Second year report of 1 July 2007

By

Jørgen Fredsøe, Professor, DTU
And
Hans F. Burcharth, Professor, AAU.

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2. Introduction

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 mean for coastal protecting was initiated in a meeting 24 August 2004.
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.
For the evaluation the following two experts were retained

Professor dr.techn., dr. h.c. Hans Falk Burcharth (HFB)
Professor Jørgen Fredsøe (JF)

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, Per Sørensen, KDI
M. Sc. John Jensen, KDI

The present report, authored by the two experts, is the second year report, written as a stand-alone report for which reason it repeats substantial parts of the first year report of 20 November 2006.

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3. Sammenfatning og Foreløbig konklusion efter 2 år.

I denne sammenfatning redegør vi for de vigtigste resultater i det 3-årige forsøg med PEM-systemet ved Skodbjerge syd for Hvide Sande.

3.0 Generelle betragtninger om kysten.

Erosion og aflejring.

Langs den jyske vestkyst er der visse steder erosion, andre steder aflejring (fremrykning af kysten). Disse 2 størrelser er tids-midlede værdier over mange år, og på en kyststrækning, der f.eks. generelt rykker frem, kan der godt i nogle år ske en erosion, d.v.s. tilbagerykning. Dette skyldes, at erosion/aflejring afhænger af bølge, strøm og vandspejls forhold, samt tilførselen af sand.

Når man taler om erosion/aflejring af en kyst, vil man visuelt altid forholde sig til om stranden vokser eller bliver eroderet. Stranden er dog kun en del af det samlede system, idet der også sker store ændringer af bunden udenfor kystlinien, og det er vigtigt at betragte det samlede system. Under en stor storm eroderes en strand generelt. Da storme er hyppigst om vinteren opnår man det såkaldte vinterprofil. Erosionen skyldes først og fremmest at brydende og brudte bølger transporterer sand i en retning væk fra kysten. Under en storm gnaves der derfor af stranden - specielt hvis vandstanden er høj - og sandet transporteres et stykke væk fra kysten. Ofte kan man også iagttage, at revlerne samtidigt bevæger sig en smule væk fra kysten.

I mildere vejr-perioder er bølgerne mindre, og kan derfor nå helt ind til stranden, før de bryder. I disse perioder transporteres sandet ind mod kysten af bølgerne, men mængden af sand der transporteres af disse mindre bølger, er langt mindre pr. dag end den udadrettede transport fra stormbølgerne. De mindre bølger regenerer altså stranden (sommerprofil), men det kan tage meget lang tid, specielt efter en kraftig storm som den vi havde den 8 januar 2005, få uger før rørene blev sat i stranden.

En storm kan altså skabe et reservoir af sand ude i vandet, der kan bruges til at genopbygge stranden på et senere tidspunkt.

Ovenstående beskrivelse er meget simplificeret. Specielt skal det nævnes, at der også sker variationer i sandtransporten på langs af kysten forårsaget af en kraftig ”bølge-genereret” strøm, der igen forårsages af bølgers brydning. Strøm gennem et hestehul i revlen er et eksempel på en bølgegeneret strøm. Bølgerne bryder normalt på revlerne, og er der hul i revlen kan bølgerne her nå helt ind til stranden før de bryder, og herved forårsage lokalt større angreb på stranden. Generelt betyder revlernes opførsel meget for strandens udseende.

Sandfodring

Området er valgt som et kompromis mellem forskellige muligheder, og det væsentligste problem for forsøget på den valgte strækning er den stadige sandfodring, der foregår nord for området: 10

km nord for forsøgsområdets nordlige ende ligger Hvide Sande, hvor der er forbindelse mellem Ringkøbing Fjord og Vesterhavet. Her blokerer molerne delvist for sandtransporten langs kysten. I dette område er sandtransporten sydgående, og har i et gennemsnitsår en størrelse på godt 2 millioner kubikmeter per år.

Hvis man blokerer for sandtransporten får man erosion syd herfor. Derfor er der lagt kystparallelle bølgebrydere på en delstrækning langs stranden syd for Hvide Sande. Nedstrøms dette område kan man forvente erosion, da bølgerne kan gnave af kysten, hvis den samtidigt ikke får tilført sand nordfra. Derfor sandfodrer KDI kysten syd for havnen ved at dumpe sand, dels ude på en revle godt 500 m fra kysten, dels på selve stranden for at kompensere for den manglende tilførsel af sand nordfra.

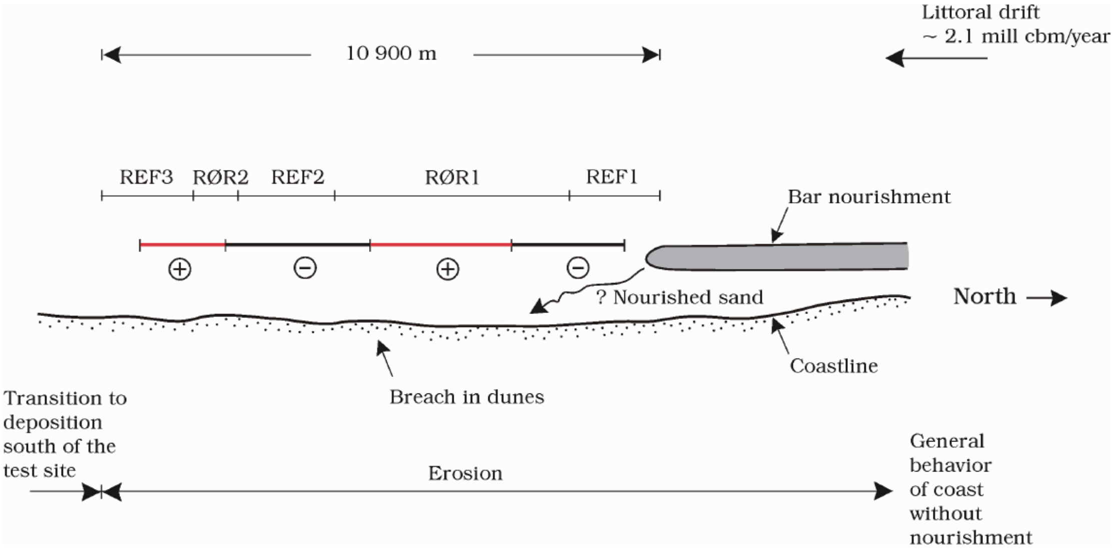
Uden denne sandfodring ville der på forsøgsstrækningen ske tilbagerykning - erosion – af kysten, aftagende fra ca 2-4 m/år i gennemsnit på nordlige del af forsøgsområdet og aftagende til ca 0 i den sydligste del af forsøgsområdet.

De fremsatte vurderinger og foreløbige konklusioner er udelukkende de 2 uvildige eksperters, og der er enighed om konklusionerne.
Da SIC har spillet en særdeles aktiv rolle såvel i de offentlige medier som andre steder i tolkningen af forsøget, har vi set os nødsaget til i denne sammenfatning også at fremsætte SICs påstande, og kommentere dem.

Forsøget, der begyndte i Januar 2005 har nu kørt i mere end 2 år, og de sidste opmålinger af kysten er foretaget januar 2007.
Forsøget består af følgende elementer:

- En opmåling af strand og havbunden 600 meter ud langs hele den 11 km lange forsøgsstrækning.
- En tolkning af kystens adfærd, med speciel henblik på om der er forskel på de strækninger, hvor der er rør sammenlignet med de såkaldte reference strækninger.
- En forståelse af rørets virkemåde.

3.1: Opmåling af bund:



Figur: skitse af forsøgsområdet set fra oven.

Opmålingen af bunden foregår dels på land (Carl Bro), dels til søs (Kystdirektoratet). Vi har opdelt områderne i 5 dele: 3 reference områder uden rør, og 2 områder med rør, se ovenstående figur. Disse områder er i alt knapt 11 km lange. Ændringerne i koten af strand (defineret som fra klitfod og 100 m ud, ligegyldigt hvor bred stranden egentlig er) og af havbunden 600 meter udenfor er gengivet i de 3 tabeller nedenfor.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
”Strand”:100 m bredt bælte fra klitfod	-11	17	-55	93	54
”Hav” (600 m bælte udenfor de 100 m ”strand”)	7	0	-5	4	3

Pålejring(+) eller erosion (-) i cm fra Januar 2005 til Januar 2006.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
”Strand”:100 m bredt bælte fra klitfod	-21	-5	-50	-55	50
”Hav” (600 m bælte udenfor de 100 m ”strand”)	13	3	0.5	-19	10

Pålejring(+) eller erosion (-) i cm fra Januar 2006 til Januar 2007.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
”Strand”:100 m bredt bælte fra klitfod	-33	12	-105	38	104
”Hav” (600 m bælte udenfor de 100 m ”strand”)	21	3	-5	-15	14

Pålejring(+) eller erosion (-) i cm fra Januar 2005 til Januar 2007.

De 2 rørområder
Foran det nordligste 4.7 km lange rørområde har stranden bevæget sig op og ned og op gennem de første 2år. Ved sidste opmåling Januar 2007 var den ca 12 cm højere end da forsøget begyndte. Foran det 900 m lange sydlige rørområde har det også gået op og ned, og for tiden er der pålagt 38 cm sand på denne del af strækningen.

De 3 reference områder
Det nordligste reference område er i dag 33 cm lavere end Januar 2005. Det midterste er betydeligt lavere, nemlig 105 cm, mens det sydlige (reference 3) er betydeligt højere, nemlig 104 cm.

3.2. Kystens adfærd

De i afsnit 1 gengivne resultater synes ikke at give noget entydigt svar:

- Stranden er vokset i begge rørområder, dog ikke særligt meget sammenlignet med strandens naturlige fluktuationer.
- Stranden er blevet lavere i to af de tre reference områder, men i det sydlige reference område er stranden blevet betydeligt højere end i rørområderne.

Går man ikke mere i detaljer vil man på denne baggrund kunne konkludere at rørene muligvis har **en svag positiv effekt.**

3.2A: Tendensen i overgangsområderne

Hvis man går lidt mere i detaljer med opmålingerne (der foretages med 100 meters mellemrum på stranden) vil man se at skiftet fra erosion til aflejring ikke følger overgangen mellem rørområder og reference områder. F.eks . fortsætter erosionen fra det nordlige reference område 1.5 km ind i rørområde 1, og den sidste km af det samme rørområde viser heller ikke tegn på aflejring. Allertydeligst ses det i overgangen fra rørområde 2 til det sydlige referenceområde, hvor aflejringen bare vokser og vokser.

I det midterste reference område 2 uden rør er der tydeligvis erosion, hvilket kunne tyde på at rørene kan have en positiv virkning. Men stranden var her allerede smal på en kortere strækning før forsøget begyndte, og siden hen er havet her godt i gang med at danne skår gennem klitten. Et sådant skår øger vindens mulighed for at transportere sand fra stranden op i baglandet, så den smalle strand fastholdes. Det er derfor svært at relatere opbygning/erosion udelukkende til rørene.

For øjeblikket detailmåler vi revlesystemet ud for dette område for at kunne forstå denne del af kystens adfærd lidt bedre. Et hestehul i revlen giver som tidligere nævnt også anledning til lokal erosion af stranden.

SIC har gået meget aktivt ind i tolkningen af resultaterne, og forklarer de observerede variationer ud fra en mekanisme, de kalder vasket sand. Med dette begreb mener SIC at rørenes tilstedeværelse øger strømningen i stranden, og herved skyller de finere partikler væk. Tilbage bliver kun det grovere sand, der gør stranden mere stabil.

Vi mener at forklaringen ikke har hold i virkeligheden: rørenes drænende virkning er begrænset – hvis den overhovedet eksisterer, se nedenfor. Desuden er tætheden af rør ganske lille: kun ét rør med diameter på 8 cm pr 1000 kvadratmeter strand. At dette skulle kunne skylle hele stranden igennem kan simpelthen ikke passe. I forsøget har vi da også udtaget sedimentprøver ved forsøgets begyndelse og et år senere uden at kunne finde nogen tendens til at strandens sand skulle blive grovere. Som ventet er spredningen på resultaterne dog stor.

Teorien om vasket sand modsiges også af, at der er erosion i slutningen af det nordlige rørområde samt det midterste reference område: her er der erosion, skønt man jo skulle synes at sandet havde fået rig lejlighed til at blive vasket, da rørområdet er 4.6 km langt.

Så hvis man skal konkludere på det detaillerede forløb i overgangen mellem de enkelte områder må man sige at **rørene ikke synes at have nogen effekt.**

3.2B: Udbulinger foran de enkelte rørrækker

Rørene står i rækker med 100 meters mellemrum langs kysten I disse rækker er der 10 m mellem de enkelte rør. Man må derfor formode, at hvis der er en drænende virkning, vil den være størst i nærheden af rørrækkerne, og forsvindende midt imellem rørrækkerne, hvor vi jo er 50 m væk fra rørene. Det må derfor forventes at der sker mest sandaflejring ved rørrækkerne, mindre midt imellem. Med andre ord: der bør dannes en pude af sand ved hver enkelt rørrække, hvis systemet virker.
En sådan opbygning er aldrig blevet observeret når vi har synet forsøgsområdet.

Igen har SIC en forklaring: de enkelte puder (som aldrig er blevet observeret) danner lokale (sand)høfder, der samler sand længere nede ad kysten og derved opbygger stranden også imellem rørrækkerne.

Dette lyder ikke rigtigt: hvis man end ikke kan se lokale udbulinger, hvordan kan de så have høfdevirkning?? I øvrigt har en strand masser af udbulinger, små som store, uden at de ses at fungere som sand-høfder.

Manglen på lokal akkumulation foran hver rørrække er en særdeles stærk indikation på, at rørene ikke har nogen effekt.

3.3. Rørenes virkemåde.

Det er naturligt at spørge om hvorledes rørene virker. Det er naturligvis ikke nødvendigt at forstå funktionen for at bruge rørene, blot de virker, men det er usædvanligt indenfor ingeniørverdenen ikke at prøve at forstå fysikken. Med en sådan forståelse er man jo også i stand til at sige hvor systemet eventuelt kan virke og hvor det ikke kan.

Det drejer sig om at dræne stranden. En drænet strand er mere stabil overfor bølgeangreb end en ikke-drænet strand, og generelt mener man at dette skyldes, at de enkelte bølger bringer sand med ind når de skyller op på kysten. Er stranden vandfyldt vil det meste af dette sand bringes med ud igen, når bølgen trækker sig tilbage. Er kysten derimod drænet vil en del af det vand, bølgen bringer med ind sive ned gennem stranden, så der transporteres knapt så meget sand med ud igen, når bølgen trækker sig tilbage. Denne effekt har været kendt i mange år, og har bl.a. været markedsført af GEO, der har udviklet det såkaldte Beach Management System, hvor man pumper stranden gennem en kystparallel rørledning for at dræne. Selv med en så kraftig fremprovokeret dræning er det dog kun lykkedes at samle moderate mængder sand op. Dette kan forbedre strandkvaliteten (mere sand), men har aldrig været tænkt som en egentlig kystbeskyttelses foranstaltning, i det mindste ikke på en så udsat kyst som den Vestjyske.
Grunden til at vi ikke mener, at rørene kan have en drænende virkning er den simple, at rørene ikke er, som dræn normalt konstrueres, nemlig forbundet med et afløb, så vandet kan komme videre. For eksempel konstrueres et omfangsdræn omkring et hus således, at vandet gennem et hældende drænrør føres hen til en brønd (fagsprog: et område med lavt tryk) og derfra videre. Et markdræn ledes normalt hen til en grøft, hvorfra vandet strømmer videre (fagsprog: et område med lavt tryk). Men PEM-røret ender blot dybt nede i sandet. Vandet der ledes gennem rørene skal jo også videre ud til havet, og der er der intet naturligt afløb (fagsprog: et område med lavt tryk) for enden af røret.

Man kunne forestille sig lag dybere nede med grovere materiale, f.eks.småsten, hvorigennem vandet ville løbe lettere, men det ville vandet så også gøre uden rør. Her må man jo også forstå, at vand ikke har så svært igen ved at løbe gennem sand, hvad man sagtens kan forvisse sig om ved at tømme en spand vand på stranden, vandet forsvinder med det samme. I øvrigt ville mellemrummene i et lag bestående af småsten hurtig fyldes op med sand i en strand, og så ville det ikke være lettere for vandet at strømme her end alle andre steder.

Fra SICs side har der været skiftende forklaringer på rørenes virkemåde: impermeable lag, permeable lag, ferskvandstryk og lignende.

Vi har prøvet at analysere de forskellige muligheder. Hvis sandet overalt er ensartet vil rørene faktisk have en drænende effekt på den af tidevandet skabte bevægelse i stranden. Desværre er den drænende effekt mikroskopisk: alle beregninger viser at det er meget svært at få mere end 0.5 liter/minut til at strømme gennem de enkelte rør, når de er placeret i sand med de trykforskelle, der her er tale om Dette har vi også testet i laboratoriet. Stranden ved Nymindegab skal samtidigt typisk tømmes for 50-100 liter saltvand pr løbende meter strand per minut når vi bevæger os fra høj- til lavvande, så dræningseffekten midlet over hele strækningen er mindre end én promille. Antager man at dræningen kun forgår lokalt omkring de enkelte rør, f.eks. i en radius på 5 m (=halv afstand mellem de enkelte rør) bliver dræningseffekten lokalt stadig mindre end én procent.

Sammen med SIC har vi også udført trykmålinger i stranden nord for test området. Målinger viste at der var en vis trykforskel inde i røret og udenfor –midt mellem rørene. Dette tolker SIC som om rørene virker, på trods af at dette netop var at forvente ifølge vores teoretiske betragtninger, hvor vi påviser at denne trykforskel medfører at dræningseffekten er mikroskopisk.

Vi har prøvet at analysere effekten af høj-permeable områder i stranden, og konkluderer at rørene generelt ingen effekt kan have. Dog kan isolerede lommer af højpermeable lag (ral og lignende) sættes i funktion, hvis de ved hjælp af rørene kan sættes i direkte kontakt med havet. Dette må dog anses for at være et meget specielt tilfælde.

Vi kan ikke se at rørene forbedrer dræningen af strande med impermeable lag, da det stadigt er lige besværligt for vandet at komme ud i havet efter at det f.eks er bragt ned under et sådant lag. Igen kan man forestille sig helt specielle geometriske former af de impermeable lag, f.eks. skålformede lommer, hvor rørene kan have en vis virkning.

SICs hovedforklaring på virkemåden er ferskvandstrykket, og der henvises til en bog, der skitserer hvorledes ferkvand presses ud i havet over noget saltvand uden yderligere forklaringer.
Nu er der faktisk ikke noget rigtigt ferskvandstryk ved Skodbjerge, der jo ligger på en smal tange mellem havet og Ringkøbing fjord. Så ferskvandstilførslen til stranden er på årsbasis højst 20 liter/time per løbende meter strand. Som nævnt ovenfor løber der samtidigt ca 4-6000 liter/time saltvand ud, når vi bevæger os fra høj- til lavvande, altså mindst 200 gange så meget. Så ferskvandet kan kun have en mikroskopisk effekt. Men den har formodentligt slet ingen effekt, da ferskvandet som sådan ikke skaber nye strømninger, blot ændrer vandspejlet lidt på grund af de trykforhold der opstår, når let vand (ferskt) ligger over tungt (salt). Dette prøver vi nu at modellere ved hjælp af en numerisk model.
Sammenfattende kan vi sige, at der **fra et strømningsteknisk synspunkt ikke er nogen indlysende mekanismer, der skulle fremprovokere en betydelig øget dræningseffekt.**

3.4. Afsluttende bemærkning:

Efter 2 års forsøg er der ikke sket synderligt meget i området, der jo helst også gerne skulle være stabilt, da Kystdirektoratet sandfodrer for at kompensere for den erosion, der ellers ville være i området. Det har ikke været muligt for os at se forskel på de områder af kysten, hvor der har været rør sammenlignet med reference områderne uden rør. Der er ikke nogen tydelig signatur af rørrækkerne på kysten, hverken lokalt eller mere overordnet.
Der har samlet været aflejring af sand i forsøgsområdet, men her må man erindre, at forsøget begyndte kort tid efter den store storm 8. Januar 2005, hvor en masse sand blev eroderet fra stranden ud i vandet . En del af dette er siden da igen transporteret ind på land, hvilket forklarer den

relativt store opbygning af stranden det første år af forsøget. I år har der været en del storme i Januar, og havet har igen taget en del sand fra stranden.
Efter 2 år mener vi ikke, at vi definitivt kan sige at rørene har en positiv virkning.

September 1th 2007

Jørgen Fredsøe Hans F. Burcharth

4. Planning of the tests

Project group meetings were held in the autumn of 2004 with the objective of selecting a test size and decide the positioning of the pipes and the methods of monitoring the response of the coast.

4.1 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.

Two potential sites were discussed: A 15 km long stretch at Skodbjerge just south of the part 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 site, 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 structures 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. 4.1. Erosion decrease to the south so that just south of the 15 km stretch the beach is stable. Further south accretion takes place. Beach nourishment will not take place in the three years test period, but nourishment at the offshore bar will continue.

A long groin at the northern boundary of the Skallingen site creates lee side erosion. Erosion takes place over the full length of the actual stretch of coast.

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.

SIC argued that the length was too short as a 10 km stretch was needed. Moreover, SIC regarded the influence of the long groin to be to disturbing for the tests. As SIC refused to use Skallingen it was decided 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.

Figure 4.1 Bar nourishment

4.2 Planning of the test

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 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 and JF 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.
A compromise as shown in Fig. 4.2 was found in which 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 installation during the first two years are shown in Table 1. As seen from the table, drains have been added in some areas where increase in beach width made it possible.

Figure 4.2 Location of the test stretches.

4.3 Monitoring of the test site

4.3.1 Surveying

Profiling per 100 m of the beach including the 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 four surveys have been performed in April, July and October 2005, and January, April, July and October 2006, and January 2007. Carl Bro A/S performs 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.

4.3.2 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 is of importance. The method of instrumentation is under discussion.

As it is generally accepted that ground water outflow in the beachface 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 found that the actual ground water table variation across the land spit had no influence on/or could not enlighten the function of the drains.

4.3.3 Grain size analyses

In order to check the hypothesis of SIC that the drains increase the groundwater outflow through the beachface 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 accreation observed after placement of the drains).

Grain size analyses of the samples have been made and compared with samples taken in May 2006. The relative amount 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 no conclusion on the effect of the drains in terms of wash-out of fine materials could be made.

4.3.4 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 was tried but without success. Aerial photographs have not been of a frequency and quality which allow more systematic analyses.

4.3.5 Pressure measurements in the beach

In order to get some insight regarding the physical functioning of the drain system, a field test program for measurement of water pressure variations in the beach and in the proximetry of the drains were performed in the spring of 2006.

The programme was carried out with additional consultancy of Dr. Peter Engesgaard, Geological Institute of University of Copenhagen. The report of Peter Engesgaard, attached as Appendix 3, concludes that the no effect of the drains on the surrounding water pressures could be detected on the observed pressure variations would be expected also without the drains.

5. Characteristics of the test site

5.1 Geomorphologic conditions

The test site is 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 at present extends approximately 450 m from the foot of the dunes.

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 ranging from 50 m to 100 m.
Grain size analyses of the sand in the foreshore and in the beach top layers shows medium to very coarse sand with grain diameter in the range 0.3-2.5 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.
Several shore parallel bars, typically three, 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 Hydrographic conditions

Water levels

At the coast 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 of 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 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 three storm situations, 1.1.07 max. water level +1.75m with winds just over

21 m/s, 11-12.1.07 max. water level +2.14m with winds over 21 m/s, and 14.1.07 max. water level +1.78 m with winds over 21 m/s.

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

Waves

The prevailing westerly winds cause quite frequently 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 to actual stretch of coastline is West-North West, causing the net sediment transport to be Southbound.

Typical crest levels of the bars in the nearshore zone area round 2 m below MWL, limiting the significant wave height passing the inner bar to be approximately 0.6 x (2,00 + heigh water), i.e. around 3 m during the highest storm water levels.

Because of the protecting effect of the bars against beach erosion it is important to identify positions and holes in the bars, especially related to the inner bar. In case of holes much larger waves will reach the beach. It is therefore important to see if there is correlation between the bar topography and the beach erosion and beach accretion. It is however very difficult to get, within reasonable costs, information about the bar topography. Methods of obtaining this information are investigated. Fig. 5.1 and 2 show the one-year 2005 and 2006 statistics of significant wave height recorded by a directional waverider buoy in 15.5 m water depth offshore Nymindegab.

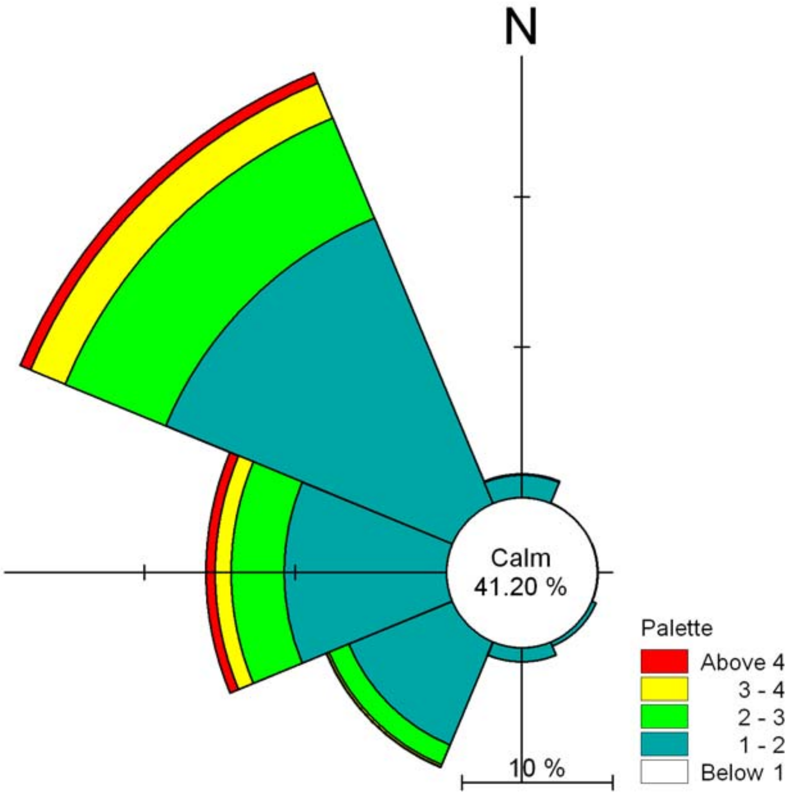


Figure 5.1 Wave rose year 2005

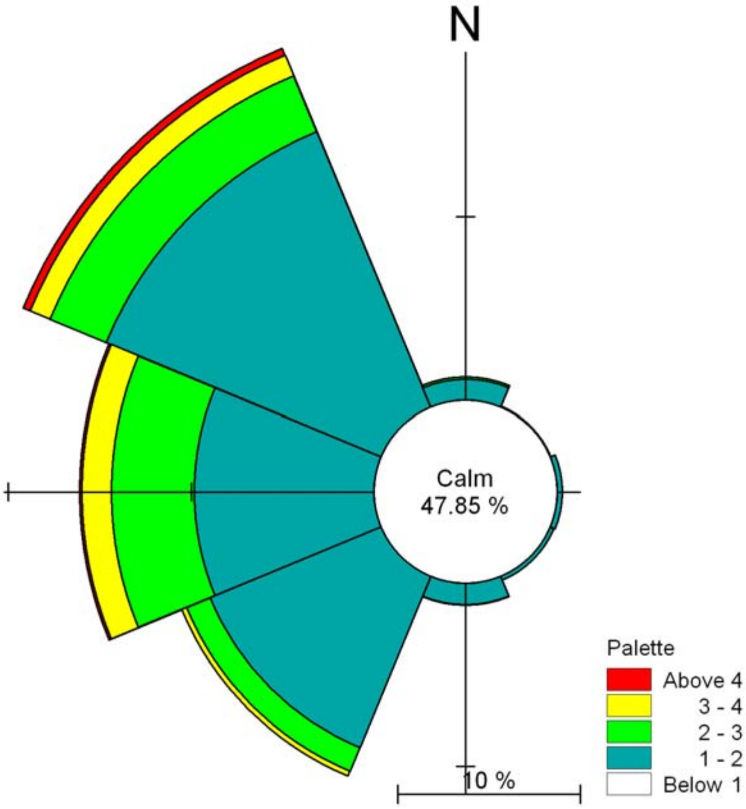


Figure 5.2 Wave rose year 2006

It is seen from Fig. 5.1 and 5.2 that the angle between the coastline and the dominating incoming waves is approximately 45^{o}, thus causing a net-sediment drift in southern direction.

5.3 Former coastal changes and man-made interventions

The natural erosion (retreat of the coastline) 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, calculated as averages over the years 1977-96.

The actual erosion is different due to man-made interventions. Actually the coastline has, apart from fluctuation, in average been stable over the last 5-10 years as documented by the KDI profiling of lines 5700-5810 (chainage 4010000-4021000). Table 5.1 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 5.1. Man-made interventions, 1977-2006

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					
Total	2.246.230	4.556.147	138.855	307.224	1.308.197	1.346.563	385.855	200.255	352.124	614.242

6. The functioning of the tubes

6.1 The near-tube flow.

Introduction

In this chapter we try to study the functioning of the tubes.

PEM stands for Pressure Equalization Modules, so as we understand it there must exist a pressure difference in the beach which can be equalized by the tubes. It is not easy to localize this point.
In the following we restrict ourselves to consider where the tubes may improve the **drainage capacity** of the beach, since no pressure difference can be build up, because sand is able to breath .

It has not been possible for the experts to explain a significant drainage effect of the tubes.

In general a drain works as follows: The flow in the soil will always flow from a higher to a lower pressure. Such difference in pressure can be created within a drain, if this is connected to a low-pressure outlet like a well or ditch or similar. The functioning of a drain in a beach is illustrated by two examples in the section “Other drain systems”.

Next it is explained and illustrated that the PEM-system cannot work in the same manner. For this reason we can not see why the system should have any kind of drainage effect.

The flow in the beach is usually quite complicated, and some simple cases will be outlined below and in the two Appendices 2 and 3 at the end of this report.

Water level variations in the beach.

If the water in the sea is calm, and there is no water supply to the beach from land, the water in the beach will have the same water level as that in the sea.
However, usually the Sea level change with time due to

- Wind waves
- Tide
- Wind set-up and changes in atmospheric pressure (storm surge).

The variation in the sea level will create flow in the beach, where the water level will move up and down with the same frequency as that in the Sea, but with a phase shift in time and with amplitude, which is smaller than the water level amplitude of the sea level.
Figure 6.1a-c shows a number of sequences of the ground water level in the beach:
In figure 6.1a and 6.1b, the effect of the ground water table in the beach caused by wind waves with a period of 1-15 sec is sketched. In such cases, also the groundwater in the beach will oscillate, but this oscillation can only be felt a few meters away from the sea.

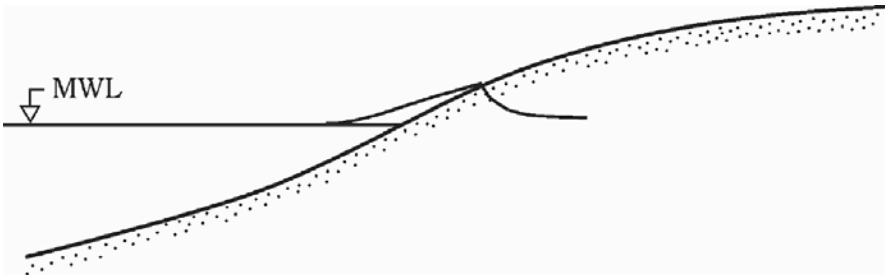


Figure 6.1a: Ground Water Level (GWL) during run-up of wind generated waves.

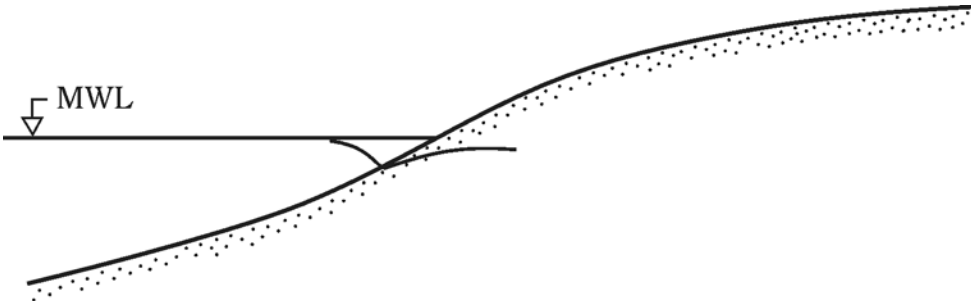


Figure 6.1b: Ground Water Level (GWL) during draw-down of wind generated waves.

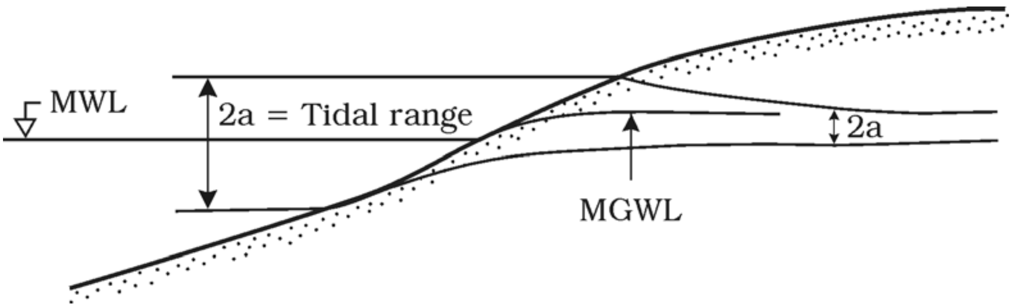


Figure 6.1c: Ground Water Level (GWL) due to long waves (tide)and storm surge. The dampening in the beach (the height of the tidal range 2a in the beach) is much weaker than in the case of wind generated waves.

In figure 6.1c the tidal flow with a very long wave period (around 12 hours) is shown: from this long period motion, the variation in the sea level penetrates much further into the beach, so the dampening of the motion is much smaller than in the case of wind generated waves..

It is of some importance whether the beach is filled with water or not. If there is a lot of water (high GWL (Ground Water Level)), the individual swash will be of equal size in the run-up and in the draw down period, resulting in nearly equal deposition and erosion of sand in the swash zone, figure 6.2a.

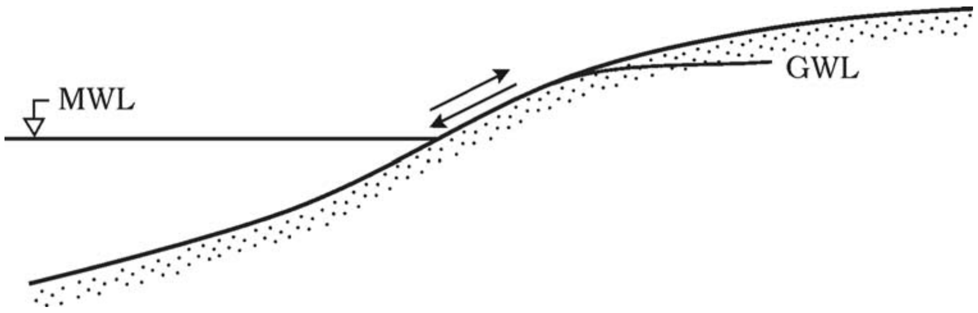


Figure 6.2a: The swash zone flow back and forth is more or less the same if the beach is saturated.

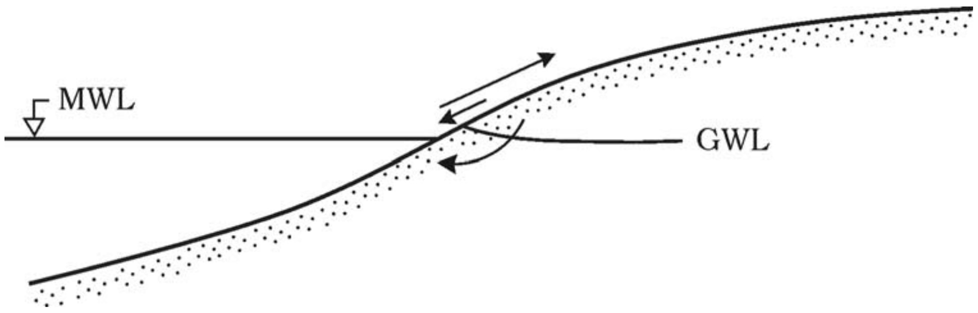
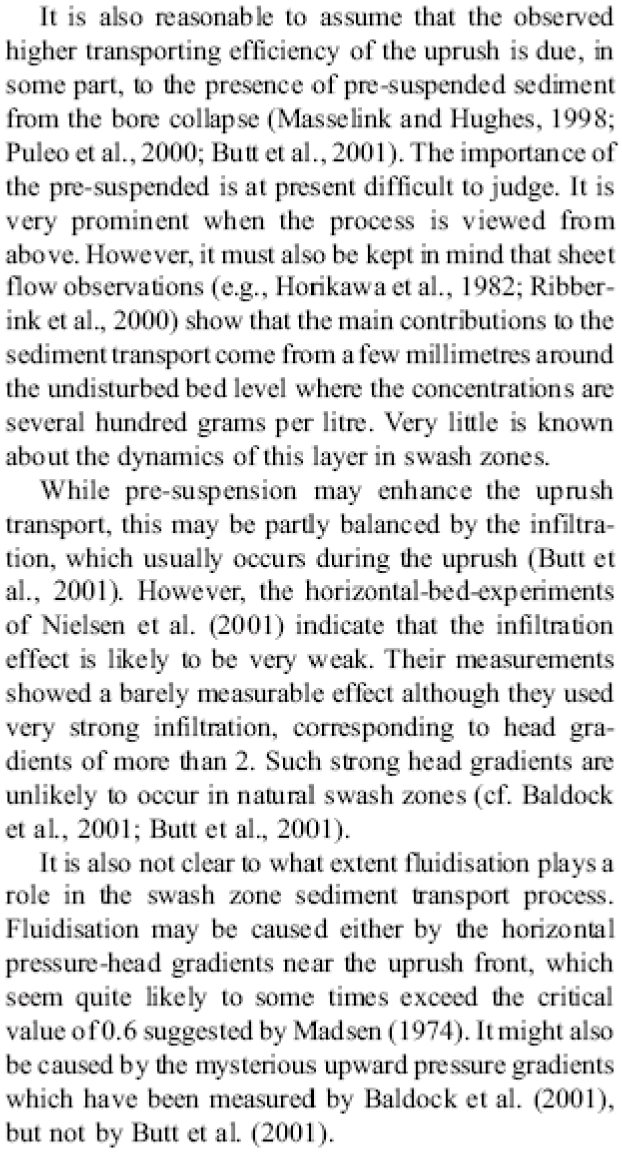


Figure 6.2b: The flow is stronger in the uprush-phase than in the downrush-phase if the beach is drained, because some of the uprush-water can filtrate into the beach..

If the beach is well drained, some of the water transported towards the beach in the run-up phase does not need to run down the slope through the draw down phase, but can instead be drained away through the beach as sketched in figure 6.2b. In this case some deposition of sand may occur leading to a stronger beach profile.
However, the importance of this effect is usually considered to be weak. Below we quote Peter Nielsen, Coastal Engineering 2002:



It is also reasonable to assume that the observed higher transporting efficiency of the uprush is due, in some part, to the presence of pre-suspended sediment from the bore collapse (Masselink and Hughes, 1998; Puleo et al., 2000; Butt et al., 2001). The importance of the pre-suspended is at present difficult to judge. It is very prominent when the process is viewed from above. However, it must also be kept in mind that sheet flow observations (e.g., Horikawa et al., 1982; Ribberink et al., 2000) show that the main contributions to the sediment transport come from a few millimetres around the undisturbed bed level where the concentrations are several hundred grams per litre. Very little is known about the dynamics of this layer in swash zones.

While pre-suspension may enhance the uprush transport, this may be partly balanced by the infiltration, which usually occurs during the uprush (Butt et al., 2001). However, the horizontal-bed-experiments of Nielsen et al. (2001) indicate that the infiltration effect is likely to be very weak. Their measurements showed a barely measurable effect although they used very strong infiltration, corresponding to head gradients of more than 2. Such strong head gradients are unlikely to occur in natural swash zones (cf. Baldock et al., 2001; Butt et al., 2001).

It is also not clear to what extent fluidisation plays a role in the swash zone sediment transport process. Fluidisation may be caused either by the horizontal pressure-head gradients near the uprush front, which seem quite likely to some times exceed the critical value of 0.6 suggested by Madsen (1974). It might also be caused by the mysterious upward pressure gradients which have been measured by Baldock et al. (2001), but not by Butt et al. (2001).

This reference suggests that the deposition in the swash zone is mainly due to presence of pre-suspended sediment, and not due to infiltration.

Other drain systems.
The idea of drainage has for instance been followed in the so-called Beach Management System (BMS), in which a tube is placed horizontally down in the beach as shown in figure 6.3.
The beach water is drained to the tube, and the water is transported further away by using pumps, thus creating a low pressure in the tube. The BMS has demonstrated some success: a small berm of beach sand is accumulated in the neighbourhood of the tube. The size of the berm depends strongly on local conditions, but the magnitude of accumulated sediment is 1-10 cbm per meter beach. Such a berm is good for recreational purposes, but is unlikely to provide a real measure of coastal protection, because it will be eroded away in a very short time during a real storm. A negative part of the BMS system is that it requires electricity and maintenance of pumps.

Beach management system

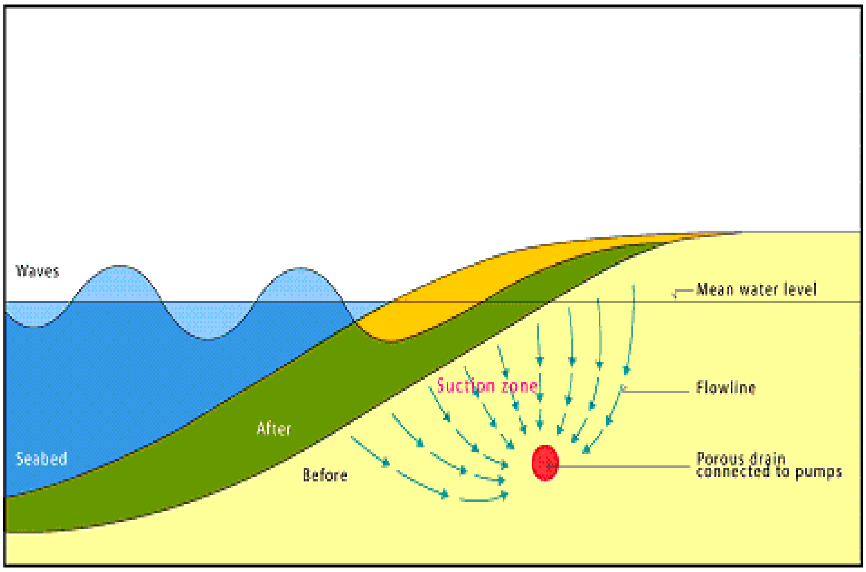


Figure 6.3. The Danish Beach Management System drains the beach by pumping through nearly horizontal tubes located parallel to the shore.

Another drain approach is Japanese, and shown in figure 6.4. In this concept, a permeable layer is placed in the beach reaching from a high level in the upper part of the beach to a level below the lower part of the beach with connection to the sea. In this way the system utilizes the slope of the beach to create a pressure gradient (from high to low pressure) within the permeable layers.

Development of Gravity Drainage System for Beach Protection

Shin-ichi Yanagishima*, Kazumasa Katoh, Naoto Iwasa and Yoshiaki Kuriyama

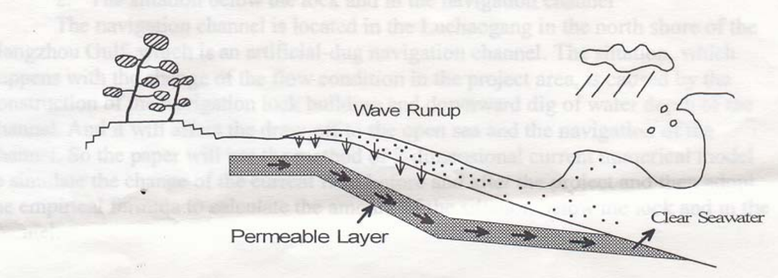


Figure 1 Beach protection system with gravity drainage.

Figure 6.4. Japanese system to drain beaches: here the tubes are replaced by a highly permeable layer, which is emptied by gravity. Please note the sea-connection of the permeable layer.

In the gravity system shown in figure 6.4 accumulation of sand has also been observed, the magnitude being slightly smaller than that obtained by the BMS system.

The PEM-system

The flow in the PEM-system is as follows: A row of vertical perforated tubes is drilled down in the beach sand.
Figure 6.5 shows a single tube used in the present test, and figure 6.6 shows the dimensions of the slots in the tubes. The slots are only 0.2 mm wide in order to avoid penetration of sediment into the tube. From Figure 6.5 it might be noted that slots only are present only in the lower half of the tube. Where the slots are present water is allowed to flow in and out of the tube, so this part is called the “active part” of the tube. In all following drawings and sketches, only this active part of the tube is shown.

Figure 6.5: Photo of tube in full length. The tube is without slots in the upper half, but it is ventilated at top, so air can go through.

Figure 6.6 Blow up of the tube: Slots of 0.2 mm width are cut in the lower half of the tubes, so water can flow in or out.

The distance between each row of the tubes is 10 meter, and the distance in between the rows is 100 meters.

6.2. The homogeneous beach.

Let us consider beaches, which consist of permeable, sand all over, i.e. no impermeable layers are present. Usually the sand is characterized by an average size d and a geometric standard deviation σ. Very graded sand has a large content of sediment, which is much finer than the average size d. This sand is called *natural sand.* If you remove a lot of the fines, this will hardly change d, but will decrease σ. This is what SIC calls *washed sand.*

Because the different flow-resistance in the sand and in the tube, the water level will be different outside and inside the tube if a vertical pressure is present. This might be the case if the ground water motion in the beach is introduced by an oscillatory motion in the Sea. This motion can be caused by wind waves, for which we agree (the experts and SIC) that the PEM-system does not have any impact, The oscillations caused by the tide and storm surge for which much more water will infiltrate the beach because of the slow changes in water level, cf figure 6.1 is therefore considered in the following.

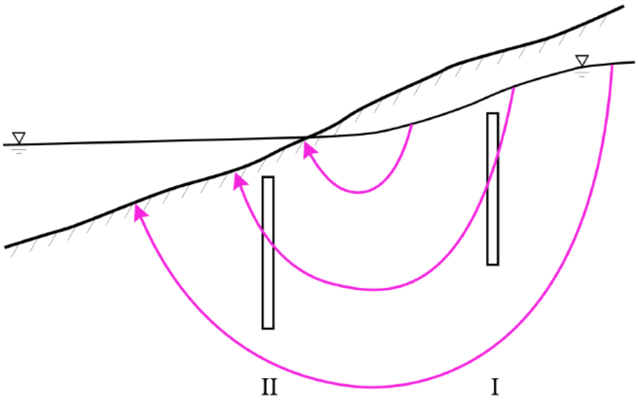


Figure 6.7: The flow introduced in the beach caused by tidal motion in the sea.

The analysis given in appendix 2 consider the tidal situation, where a vertical pressure gradient leads to a ground water motion very different from standing waves in front of a vertical wall, see figure 6.7. This will cause a negative vertical pressure gradient at tube I shown in figure 6.7, and a positive vertical pressure gradient at tube II. It has been measured (see appendix 3) that the water outside the tube has an amplitude in the order of 1 meter, and this result has been used in the analysis to estimate the vertical pressure gradient needed to create this strength of the groundwater flow. The analysis suggests that during *falling* groundwater level the water level outside the tubes must be higher than inside the tubes, leading to flow directed towards the tubes at the top, and away from the tubes at the bottom of the tubes. During *rising* water levels the opposite will be the case, see figure 6.8 a and b.

The reason why the experts don’t understand that the PEM-system should work is that there are no connection of the tubes to a low-pressure region at the end of the tubes,

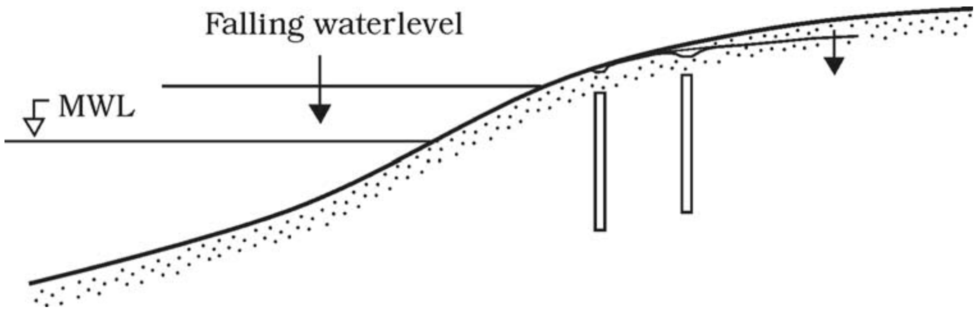


Fig 6.8a: During falling waterlevel (ebb flow or after a storm), the tubes will improve the drainage, the impact is however estimated to be insignificant, actually less than one per thousand.

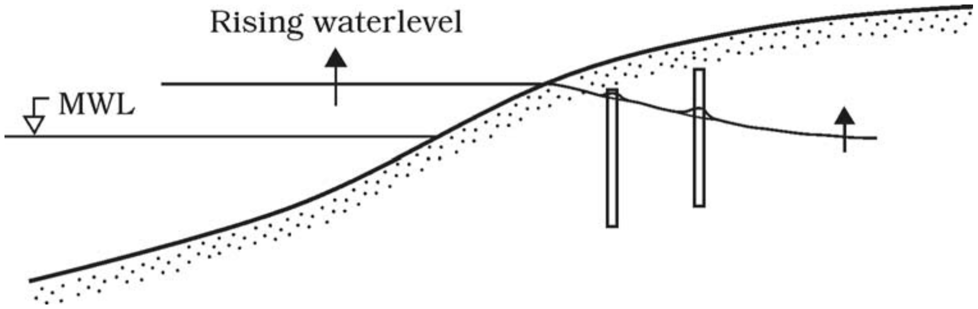


Figure 6.8b: During rising waterlevels (flood or storm surge), the beach is filled slightly faster with water, because the vertical tubes can lead the water easier to the beach. Like in the falling stage, the impact is insignificant; less than one per thousand.

So nearly no driving forces exist to activate the flow near the tubes. A simple estimate of the impact of the tubes is given in Appendix 2, in which it is demonstrated that there certainly is created a flow through the tubes because of the less flow resistance within the tubes than in the soil outside, but this flow is very small, less than 1 mm per second. Even though this is 5-10 times larger than the flow velocity in the surrounding soil if this soil is very fine, it will have no drainage effect because the tubes occupy a very small fraction of the area under consideration. If we assume that the tubes shall drain the surrounding area in a radius of 5 m only around each tube, the improved drainage will be 0.01% (in case of very fine sand, where the impact is largest), corresponding to a change in the tidal range from 1.0000 m to 1.0001m.

6.3. The in-homogeneous beach: presence of permeable layers.

A number of sketches are presented in the following, where the expected impact from the PEM-tubes are discussed. For simplicity only one tube is shown in the beach, and we are considering the case of a falling watertable in the beach.

Figure 6.8 shows the basic case: the water will locally easier flow through the pipe, so you get a faster speed from A to B, but the water still needs to flow from B to C, and there is no trigger for this, so not much has been gained by installing the tube, it is nearly just as easy to flow from A to C as from B to C.

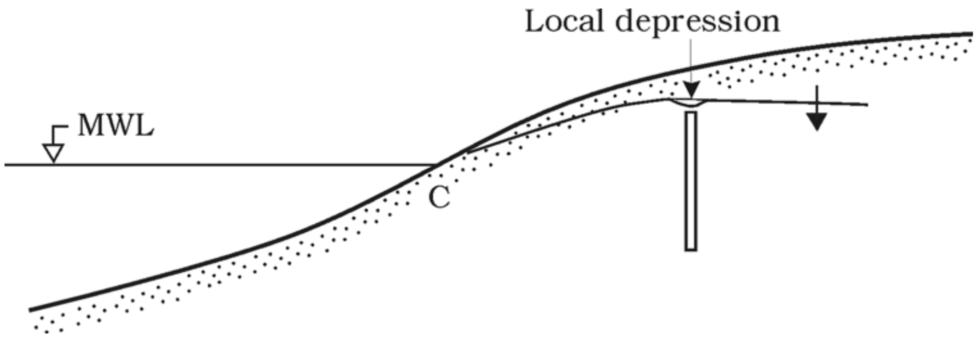
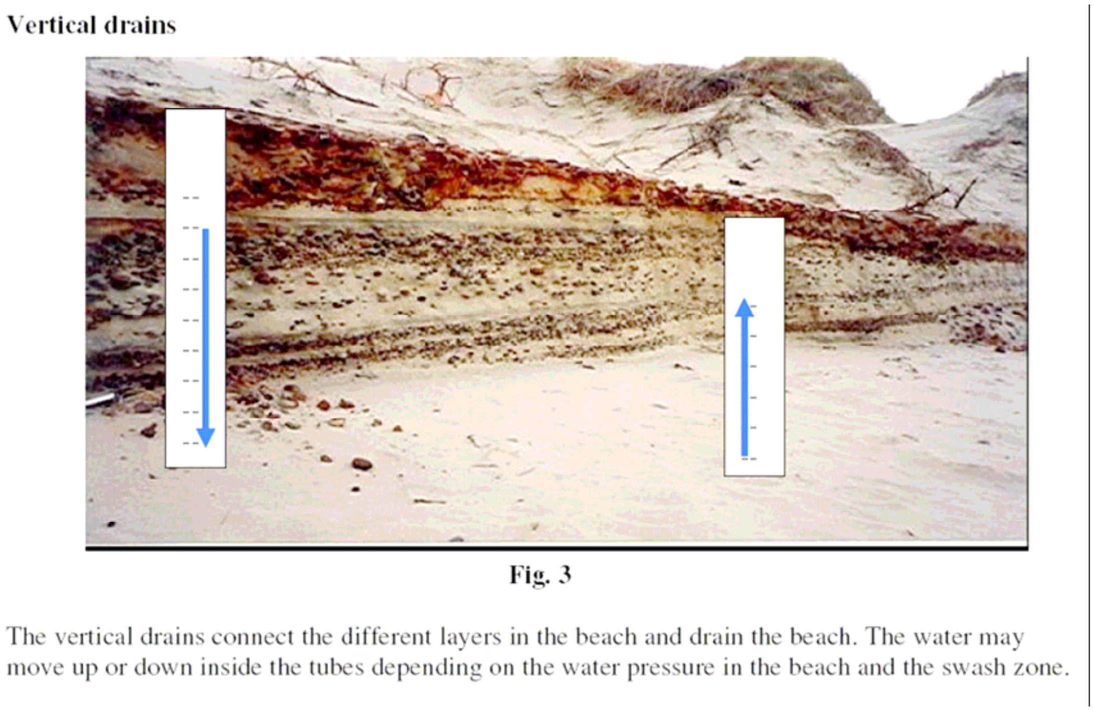


Figure 6.8: The drain will locally make a depression cone around the tube. However, the size of the cone is extremely small, and most of the beach water will flow directly rather through the tube towards the sea.

Figure 6.9 suggests that a trigger for the flow from B to C can be established by the presence of a permeable layer, see also the photo figure 6.9 from a SIC report.



Vertical drains

Fig. 3

The vertical drains connect the different layers in the beach and drain the beach. The water may move up or down inside the tubes depending on the water pressure in the beach and the swash zone.

Figure 6.9 SIC's explanation of trigger

Figure 6.9 illustrates one explanation given by SIC of the functioning of the drains in the presence of permeable layers.

As shown in figure 6.10, the presence of a horizontal-like permeable layer will in all cases improve the drainage of a beach, even without tubes installed. The requirement

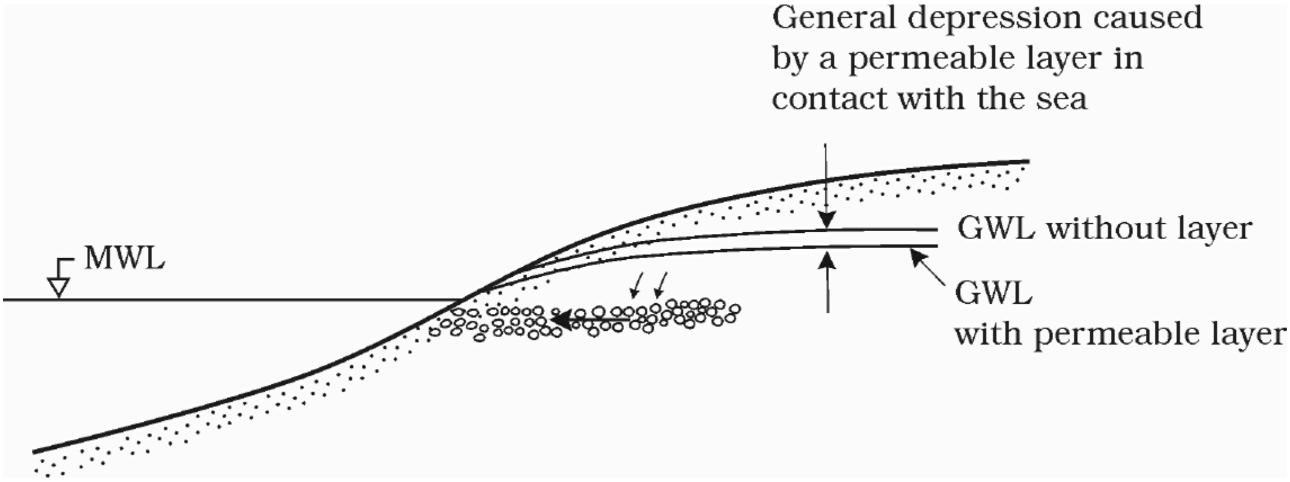


Figure 6.10: The presence of a sea-connected permeable layer will anyway improve the drainage of the beach.

will just be that the permeable layer will be sea-connected, so a low pressure can be established in the permeable layer. The layout in figure 6.10 is slightly different from the Japanese system shown in figure 6.4, because a sloping drain actually is not needed, just a pressure drop, which also can be created in a fully horizontal, but permeable layer.

Figure 6.11 shows the same situation as that in figure 6.10, but with a tube installed. Now

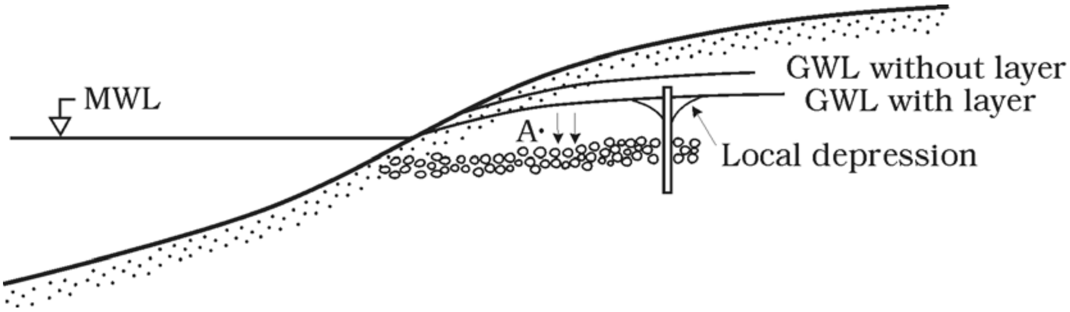


Figure 6.11: Tubes installed in a beach with sea-connected permeable layers will only have a local effect, because most of the water will go directly to the permeable layers.

next to the general lowering of GWL also a small local depression close to the tube is established. This depression can actually be expected to be larger in case of a permeable bed compared to the case of a homogeneous bed as explained in the following.

The presence of the permeable layer will anyway cause a general lowering of the water table, with or without tubes. Or, put in other words: it is easier for a water particle located far away from the tube (like in location A, figure 6.11) to move directly through the sand to the permeable layer, than to move from A to the tube (also through sand), and next further through the tube and the permeable layer to the sea.

Figure 6.12a-c illustrates this a little bit further: In figure a, we have a very permeable layer (like a PEM-tube) connected to the sea, and the drainage capacity is simply determined by the pressure drop ΔH equal to the difference in height between the actually GWL and the Sea Water Level.

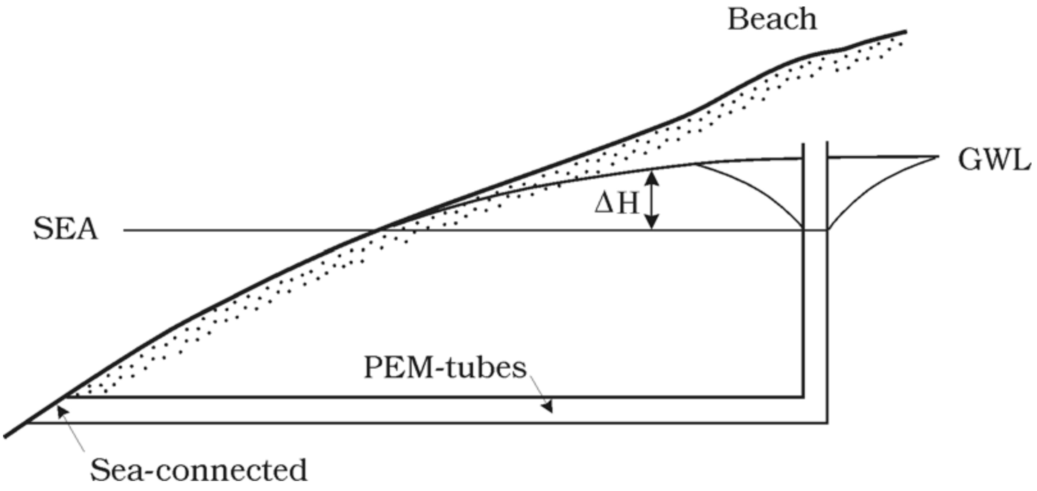


Figure 6.12a: An effective solution to drain the beach, if GWL is higher than Sea Water Level.

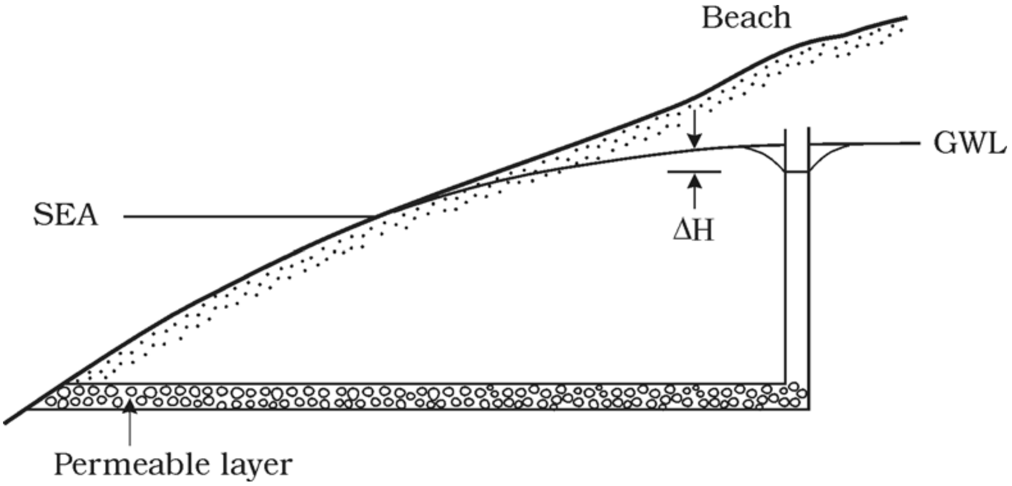


Figure 6.12b: the drainage capacity decreases if the sea –connection get a smaller permeability.

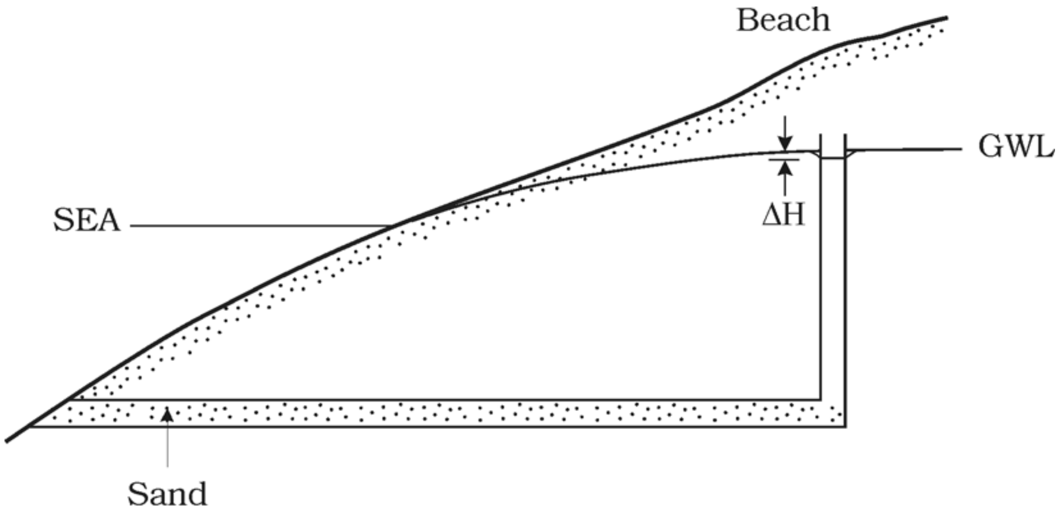


Figure 6.12c: in the case where the horizontal part of the drain simply consist of the same material as the original beach, the drainage effect disappear to be negligible.

In figure 6.12 b the highly permeable layer is replaced by a less permeable layer, but still more permeable than the surrounding sand. In this case there will be a certain energy loss through this layer, so ΔH becomes smaller because the water level in the tube must be higher in order to force the water through the permeable layer. In figure 6.12c the horizontal tube is filled with sand, and we are back to the situation shown in figure 6.8 with a very small local depression.
From the sketches in figure 6.12 it is realized that the drainage capacity strongly depends on the structure and permeability of the permeable layer.
If a permeable layer exists, it will be easier for the water limited within a circular cone around the tube as sketched in figure 6.13 to flow to the tube: The water confined within the dashed line will flow through the tube rather than directly to the permeable layer. This will certainly increase the impact radius, depending on the ratio k/k_p, where k is the permeability of sand, and k_p the permeability of the permeable layer.

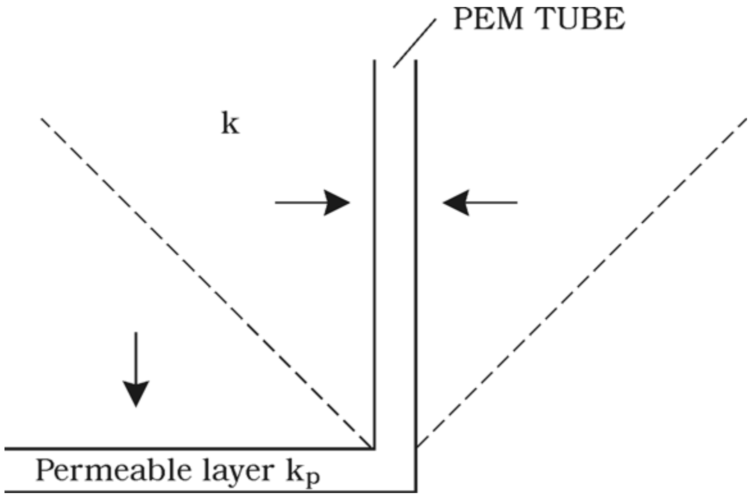


Figure 6.13: The water confined within the 45-degree cone will prefer to flow through the vertical tube if a sea-connected permeable layer exists.

Does permeable layers exists?

Permeable layers might be present in the beach, due to grain sorting by waves and wind. Figure 6.14 is a photo from the site, where layers of pebbles are present in isolated spots on the beach surface. One may ask what happens when these layers are covered by finer material? It is most likely that the voids in between the pebbles are filled with this sand from above, consequently the permeability of those layers will not be higher than that of the surrounding sand.

Figure 6.14: Layers of pebbles on the beach.

"Activation of Permeable layers".

As seen from figure 6.14, the distribution of pebbles on the beach is quiet "patchy" or 3-dimensional in its nature. So the situation as shown in figure 6.15a is a possibility: isolated layers of high-permeable layers (AB) may exist, which through the tubes can be connected to the sea through another high-permeable layer (CD).

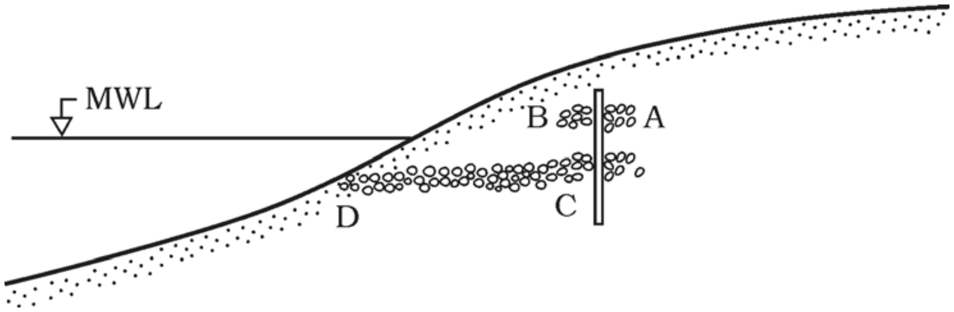


Figure 6.15a: The upper layer A-B will be drained better to the sea-connected layer CD by a vertical drain.

As sketched in the figure, the tubes can act as a vertical link in between the different permeable layers. At least it will mitigate the flow from A to C sketched in the figure, so there will be an improvement if this interconnection continues right to the sea, i.e. DC exists. If the interconnection does not exist, the flow through the tubes will still be very slow.

The row effect: it could be asked whether an interconnection between a number of tubes might improve the drainage as shown on the photo figure 6.9 and in figure 6.15b and c, where it is sketched how more permeable layers are activated. This is possible, but requires the high-permeable layers to be connected to the tubes, and further a connection from one of the tubes to the sea.

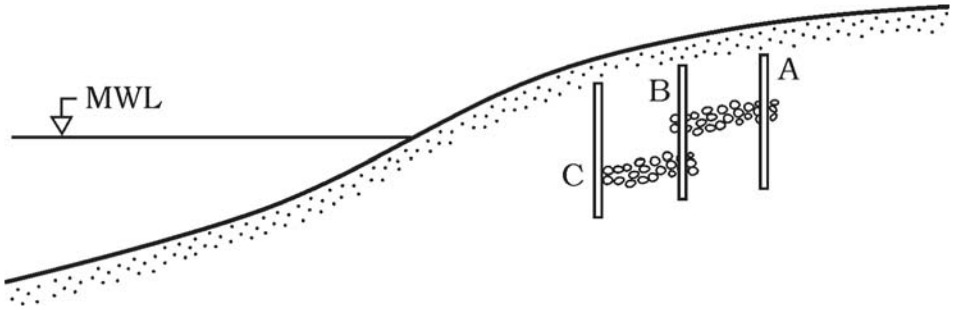


Fig 6.15b: A row of tubes can connect different permeable layers.

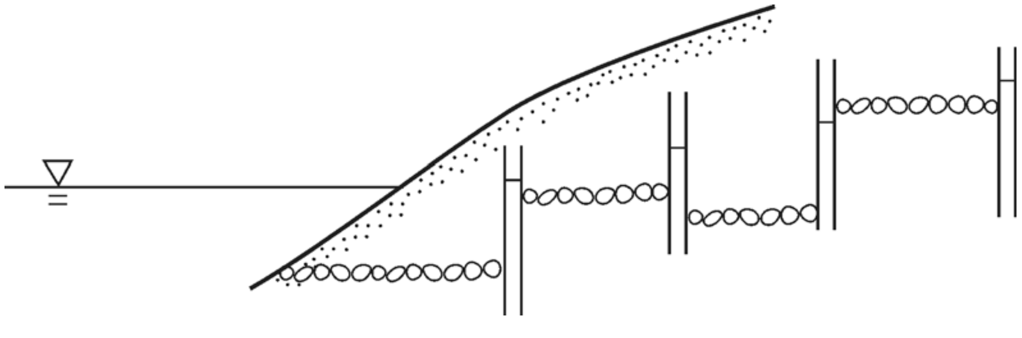


Figure 6.15c: To activate the different layers you need Sea-connection.

Will there be Sea-connection??

The situation with sea connection as sketched in figure 6.15a at point D may be possible. So the system may work, leading to deposition of sand. After this the Sea-connection has gone, the connection is blocked with the deposited sand, and the tube system stops functioning.

6.4. The in-homogeneous beach: Presence of impermeable layers.

Next we consider the presence of impermeable layers, formed be nearly horizontal layers of clay or other fines mixed with the sand.

Now the beach can't be drained as suggested in figure 6.7, because the impermeable layer with a nearly horizontal stratification prevents vertical motion. Instead the water entering the beach during high tide must be drained nearly horizontally to the sea. This will cause a higher average level of the groundwater in the beach as shown in figure 6.16.

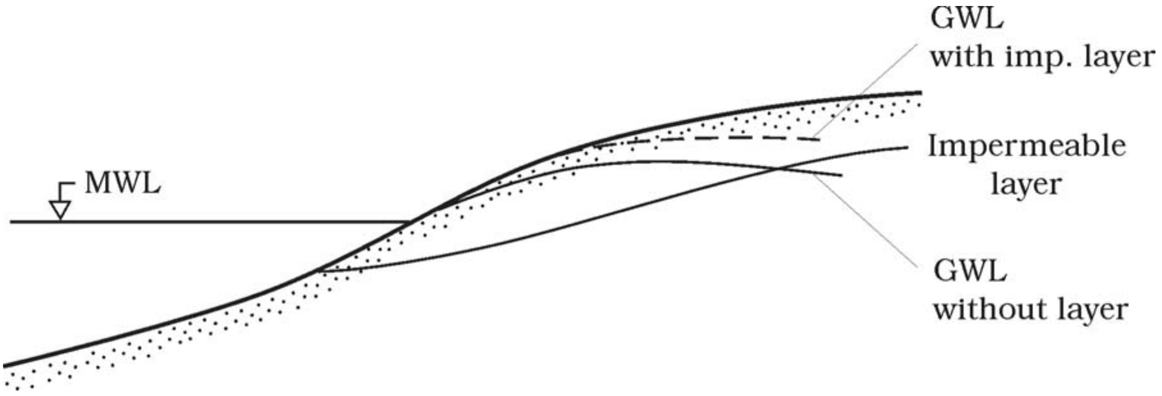


Fig 6.16: Impermeable layers will increase the ground water level in the beach during ebb flow because the flow will be more horizontally.

If a tube is installed, which penetrate the impermeable layer as shown in figure 6.17a, then the water can flow down through the tube if the pressure is lower below the impermeable layer than above.

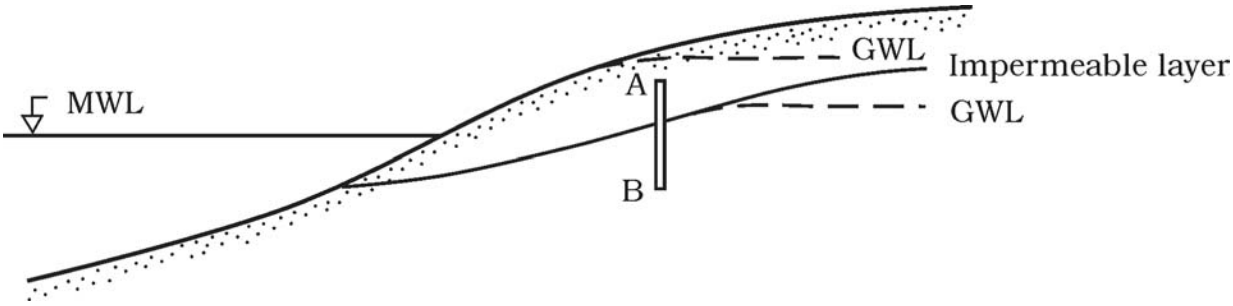


Figure 6.17a: The ground water level can be lowered if a tube penetrates the impermeable layer, and the pressure below this layer is lower than above the layer.

This will require that the extend of the impermeable layer along the coast is large, otherwise there will be a pressure-equilization through the sand outside the impermeable layer.

As sketched in figure 6.17a, the pressure below the impermeable layer is most likely to be lower than above. However, since the water flowing through the pipe still need to flow further from the end of the tube (B) to the Sea, it would be quite helpful for the drainage capacity if the flow below the impermeable layer enter a high-permeable layer as sketched in figure 6.17b. If not the drainage improvement will be insignificant.

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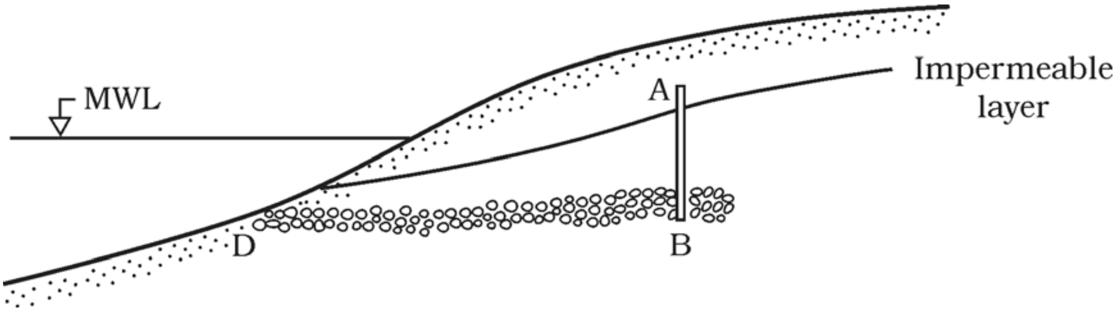


Figure 6.17b: a drain located in a Sea-connected high-permeable layer, and penetrating an impermeable layer above the permeable layer will improve the drainage above the impermeable layer. This will require a very special configuration in the beach.

6.5. Water suply from land.

One of SIC's major arguments for the functioning of the PEM-system is (or has been?) that it drains away the water running from land to the sea through the beach.
The arguments put forward above do not change significantly if the water inflow to the beach does not only originate from tide/storm surge, but also stems from out-flowing water from land. The major difference occurs in the case of the presence of impermeable layers, which in the case of a special configuration as sketched in figure 6.18 can lead to a higher pressure from below than above the impermeable layer. In this case the water will flow *up* through the tube, leading to more water in the beach (artesic pressure) So the most important thing which can be said about the inland water supply will be, that in this case you don't need tidal flow or storm surge to demonstrate any need for draining the beach. One positive effect might be, that if an impermeable layer like that depicted in figure 6.18 exists, this will lead to a concentration of the flow of the fresh water below the impermeable layer. Here a drain might have a small positive effect by reducing this outflow concentration.

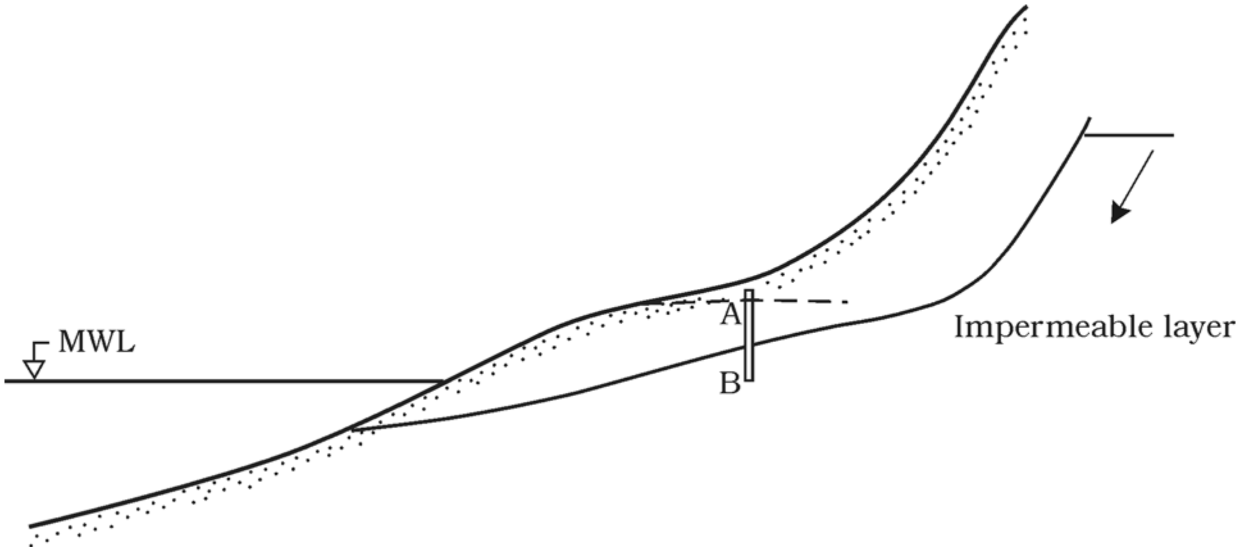


Figure 6.18: Supply of water from land will most probably flow *up* through the tubes (artesic pressure), and thereby make the beach more unstable.

6.6. Final remarks.

Finally here it must be mentioned that in the present test, the tubes only have slots in the lower half part of the tubes, while the upper part of the tube is impermeable. This means that you only have about one meter in height to make it possible to make a shortcut in between the high-permeable layers. Moreover in the case of impermeable layers, this must be located at the perforated part of the tube in order to function.

6.7 Field tests.

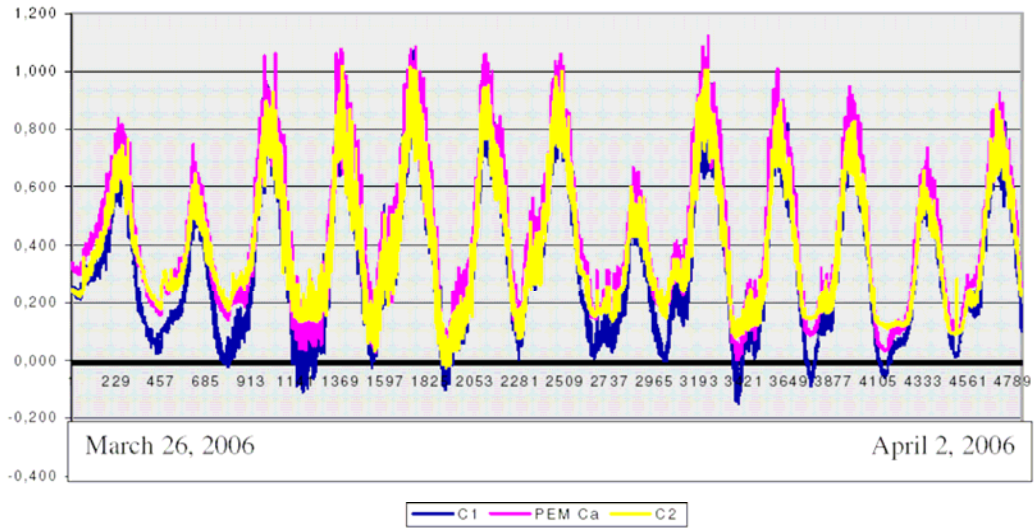


Figure 6.19: Example of Recording from the field test.

A field study just north of the test site was performed as part of this investigation to look at the pressure conditions in- and outside the tubes. The details and an analysis of these tests are given in appendix 3.
The idea behind the test was to measure the groundwater level variation in two lines perpendicular to the coastline in two different environment: in one week without the PEM-system installed, and in the following week with the PEM-system installed. Figure 6.19 shows an example of the pressure variation inside the tubes (pink, PEM) and outside the tubes (black: in between 2 PEM-tubes, C1 is 5 m nearer the Sea than the PEM-tube, the yellow tube C2 is located 5m further landward of the PEM-tube)..
First of all it is seen, that the waterlevel fluctuate partly due to the windwaves (high frequency fluctuations) but more clearly the level is seen to follow the tide. In the present case the tidal range is around 1m, and it is seen that the watertable variation is more or less the same at all three locations, so the flow does not seems to change radically near the tubes. This is certainly not in favour of having a lot of impermeable layers!

Taking a closer look of figure 6.19 it is further observed, that at high groundwater levels, the level is higher in- than outside the PEM-tube (up to 8-12 cm). This means that at high water levels, there is a flow into the tubes in the upper part of the tubes, and a corresponding outflow at the bottom of the tubes. This will improve the vertical drainage. This situation is considered in appendix 2, where the effect of a vertical pressure gradient is considered. Outside the tubes, the Darcy law determines the flow, while inside the tube the pressure is hydrostatic. For fine sand (0.10-.15 mm) it is demonstrated that the flow velocities might be 5-10 times as high as outside, but because the area of a tube is so small as compared to the area to be drained, the increase in drainage will be smaller than .1 per thousand, corresponding to a change in tidal range from 1.0000 m to 1.0001 meter.

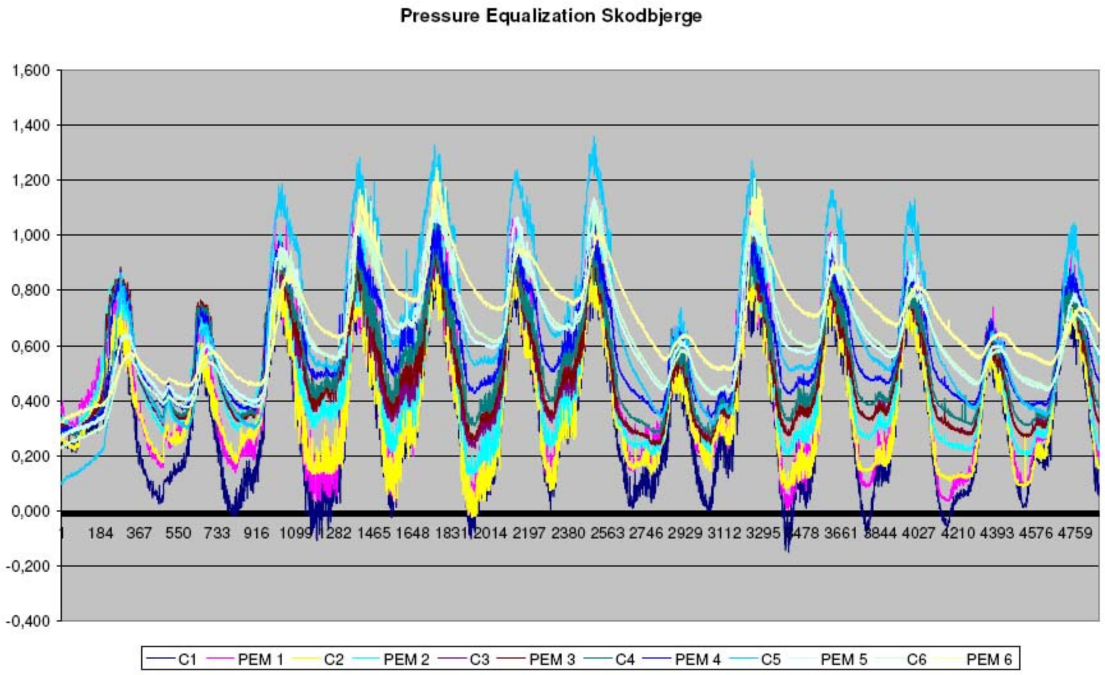


fig. 5

Based on the physical effects of the PEM tubes on the beach described above a separate test was made with water level sensors (Diver), to determine the effect of PEM on the water table in the beach. The test was carried out in the northern part of the Skodbjerge test area in control area 1.

Figure 6.20: Time variation in the whole row of tubes and outside the tubes

Appendix 3 presents the gross-behaviour of the beach, i.e. the dampening of the tidal wave as function of the distance from the coastline as sketched in figure 6.11c.
There was a change in the mean Water Sea Level of 35 cm from the first week to the second, due to changes in the weather conditions. That means that the groundwater flow in the second week incorporated an additional 35 cm thick layer of the beach in its flow domain.
The analysis of the dampening shows that the dampening characteristics of the beach was the same before and after the implementation of the PEM-tubes, which demonstrates that the PEM-system has no significant drainage effect. The inclusion of the 35 cm layer of beach has not changed the characteristics as well, which demonstrates how uniform the composition of the beach actually is.

6.8: The near tube morphology.

It has been claimed by SIC, that due to drainage, accumulation will start to take place. An example provided by SIC is shown in figure 6.21, where a small salient is observed in the neighbourhood of some tubes. However it is just downdrift of other coastal structures (groins), so the morphological behaviour here is a little bit difficult to interpret.

We have observed no individual salients in front of each row in the present test. The coast line passes the individual rows without any local changes in width, and it has been like that from the very beginning of the tests. It has puzzled us a lot, since this should be expected, - at least in the initial state.

The PEM modules create a groin that catches long shore sand transport.

Figure 6.21: Observed accumulation of sand in front of a row of tubes at Skagen.

SIC has suggested two explanations:

- There is an interaction between the initial salients which merge to one bigger coherent structure
- The increased flow velocity in the beach will remove the finer fractions and form “washed sand”. This washing will spread to all sand in the beach because of the down drift of the sand, so the tubes all the times has access to new sand.

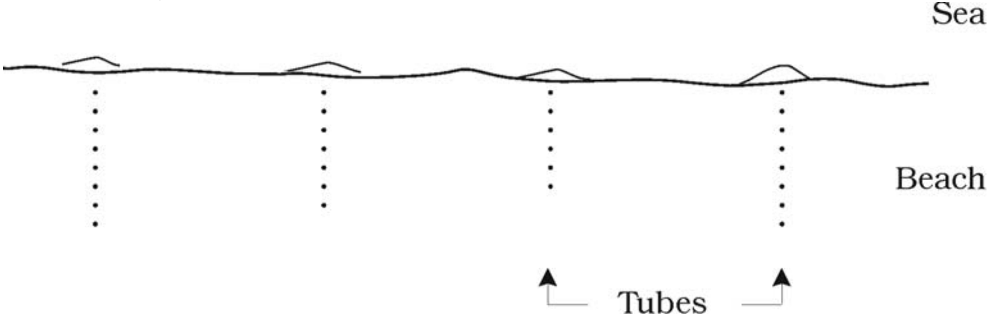


Figure 6.22: No individual salient are observed in front of the tubes just after installation.

If this is true it is still difficult to follow why it is so from the very beginning. The merging of the salient cannot occur before the individual salient has reached a certain size, maybe like the one shown in figure 6.21. And with regards to the washed sand, it certainly must take a long period of time (Years!) before all the fines have been washed away.
Also in Appendix 1, figure 2, an example is shown in which pumping provokes the drainage. In this case a very clear development of a local salient is seen, which demonstrates that such one actually should emerge if draining really took place.

7. Presentation of surveys

7.1 Preliminary evaluation of the accretion-erosion pattern in the beach and offshore

Now after two years, some considerations can be done regarding the behavior of the beach.

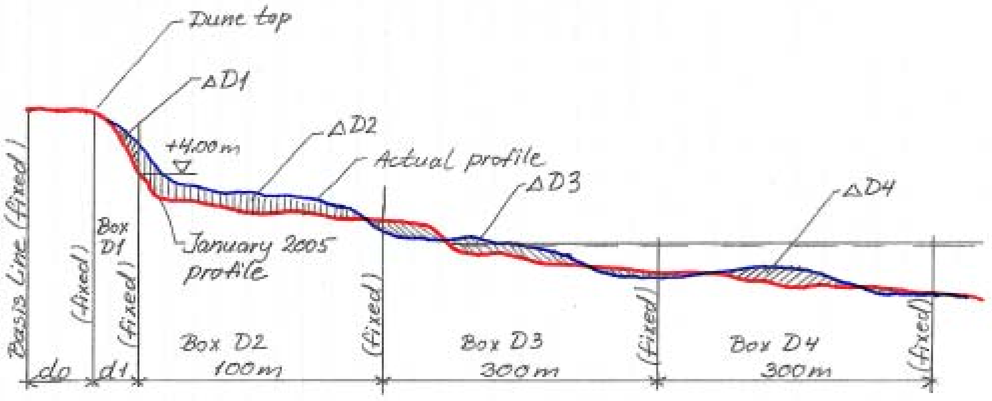
As described in the chapter 5, the idea behind the system is to improve the drainage and hereby get less erosion from the waves.
It must here be mentioned, that if the beach is more well drained, it will be drier, and thus more exposed to wind erosion. Eventually more sand will be accumulated in the upper part of the beach and in the dunes, so it can have a positive effect.

In this chapter we look at the erosion-deposition as measured since the experiment started in January 2005.

It was finally agreed to separate the profile in four fixed boxes and study the volume changes in these boxes. Moreover, it was decided to use parameters which makes it possible to follow the changes in positions of the dune foot and the coastline, and study the volume changes in dune and beach. For convenience the parameters used for the fixed box study are denoted D-parametres while the parameters used for the study of changes in the dune foot and coastline positions as well as dune and beach volume are denoted E-parametres. Both the D- and E-parametres are agreed upon in a project meeting.

Positions of the four fixed boxes of specific widths and fixed positions are related to the positions of the level +4.00m intersection with the first surveyed profile of January 2005, figure 7.1. The changes in sand volumes in each box $\Delta D1, \Delta D2, \Delta D3$ and $\Delta D4$, are calculated. Besides this is calculated the mean surface level denoted MBL in the 100m wide box as well as the changes in this level, Δ MBL. All measured values of D and MBL are included as an appendix after this section.

Professor Hans Burcharth wanted to follow both the changes in the position of the dune foot (dunes are the natural protection against violent storm erosion) as well as the changes in the coastline portion and dune and beach volumes. As the D-parametres do not provide this information it was necessary to supplement with additional parameters, here named E-parametres. The more detailed comments on beach changes are based on the E-parametres, and this is presented in section 3 of this chapter. Since SIC and also one of the consultants feel most familiar with a fixed system of reference, section 2 of this chapter is a very short description of what can be said about the beach behaviour based on the D-profiles. There are some overlap in between section 2 and 3, each section been made by the individual consultant. This only demonstrates the agreements in the evaluation in between the two experts.



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 is denoted ΔMBL

Fig 7.1: Definition of D-parameters.

7.2: Changes in the beach-box (100 m box D2) and offshore-box (600 m box D3 and D4)based on the D-parameters.

The profiles are described in figure 7.1. In section 3 the total figures on volumetric changes for the all individual reaches are given, so tables 7.1-7.3, which are based on the measurements provided in appendix 4, instead provides figures on what have happened per unit length in the separate section with and without tubes ("rør") along the coast.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
Beach-box	-11	17	-55	93	54
Offshore-box, total	43	-2	-31	24	22
Inner 300 m offshore box	-28	25	3	31	13
Outer 300 m offshore box	71	-27	-34	-7	8

Table 7.1: Deposition(+) or erosion (-) in cbm/m from January 2005 till January 2006.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
Beach-box	-21	-5	-50	-55	50
Offshore-box	80	20	3	-115	60

Inner 300 m offshore box	67	-21	58	8	24
Outer 300 m offshore box	13	41	-55	-123	37

Table 7.2: Deposition (+) or erosion (-) in cbm/m from January 2006 till January 2007.

	Reference 1	Rør 1	Reference 2	Rør 2	Reference 3
Beach-box	-33	12	-105	38	104
Offshore-box	123	18	-27	-91	82
Inner 300 m offshore box	39	4	61	39	37
Outer 300 m offshore box	84	14	-89	-130	45

Table 7.3: Deposition (+) or erosion (-) in cbm/m from January 2005 till January 2007.

Since the “beach-box” actually covers everything from the dune foot and 100-meter seawards (see fig 7.1), the average change in “beach-box”-height is found from the data in the row “beach-box” by dividing by 100 m.

7.3 . Method of presentation of surveys based on E-parametres

The E-parameters shown in Fig. 7.2 separates 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.

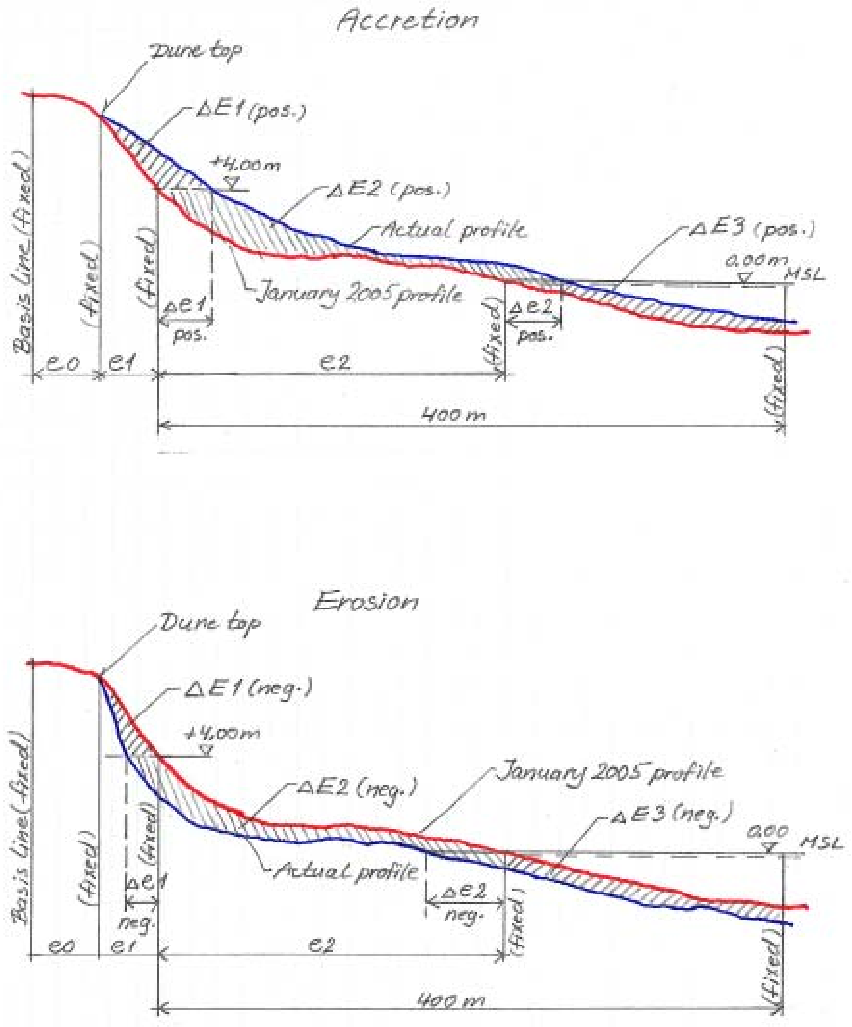


Figure 7.2 Definition of E-parametres

The changes in the position of the dune foot, $\Delta e1$, and the changes in the position of the coastline, $(\Delta e1 + \Delta e2)$, are identified as well as the changes in the dune volume, $\Delta E1$, and the beach volume, $\Delta E2$. Moreover, the changes in volume of the foreshore, $\Delta E3$, is calculated as $\Delta D1 + \Delta D2 + \Delta D3 - \Delta E1 - \Delta E2$.

Because the dunes over level +4.00 m were not fully surveyed in January 2007 it has not been possible to estimate ΔE_1 with high accuracy, as exterpolation has to be made between the highest measured point and the January 2005 measured top of the dune front face. As a consequence Δ MBL is not so well defined. However, the bias introduced by this omission is not very significant and does not change to picture of the development of the coastal profile.

Results of surveys January 2005 – January 2007

7.3.1. Changes in dune foot positions

Seaward changes in dune foot positions are due to transport of sand by the wind from the beach plane to the dunes.

The changes in the dune foot position (defined at level +4.00m) are shown in Fig. 7.3. A shoreward movement is observed for all stretches after the first year. Between October 2006 and January 2007 a considerable retreat of the dune foot took place in Ref. I and Ref. II and in the boundaries of Rør I. In the middle of Rør I and in the entire Rør II there was shoreward movement of the dune foot.

7.3.2 Changes in coastline positions

The evolution in coast line position calculated as the changes in e_2, i.e. Δe_2 observed in the two-year period is shown in Fig. 7.4.

Very large changes are observed in some stations. For example in Rør I a total shift in some lines are app. 80m. After the two-years period there are consistent coastline retreat in Ref. II and retreat in a large part of Rør I. Ref. I and Rør II show both retreat and seaward growth. More consistent seeward growth is seen in Ref. III. In conclusion there is not a clear correlation between movements in coastline position and positions of drains.

The initial beach width e_2 as surveyed January 2005 is shown in Fig. 7.5 together with the beach width e2+ Δ e_2 in April 2005.

Fig. 7.6 shows the beach width in October 2006 and January 2007. From these two figures 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. Only hereafter there has been a change resulting in a more even beach width distribution along the site, but still with the more narrow beaches in parts of Ref. I and Ref. II although limited stretches of narrow beach also exist in Rør I and Ref. III.

7.3.3 Changes in dune and beach volumes

The approximate changes in dune volumes ΔE_1 are shown in Fig. 7.7. After one year there was accumulation in all stations with few exceptions. In the April 2006 survey was seen very large accumulations in the northern part of Rør I and southern boundary of Ref. I. Erosion was only seen in two stations in Rør I and one in Ref. I. After two years the large accumulations were still in the same locations, but erosion was seen in some stations within all five stretches, most severely in Ref. II.

Fig. 7.8 and 7.9 show the changes in beach volume. The first survey in April 2005 showed mainly accretion in Rør I, Rør II and Ref. III. Dominant erosion was seen in Ref. I and Ref. II, but also in the northern part of Rør I. By April 2006 the picture was very different in that deposition was in the middle of Rør I and Rør II and most pronounced in Ref. III, whereas almost no changes from the initial situation in January 2005 were seen in Ref. I and Ref. II. By July 2006 erosion had again taken place in a part of Ref. I and in Ref. II. Fig. 7.10 shows the further development in which the October 2006 situation corresponds closely to the April 2006 situation, but by January 2007 we see a picture of significant erosion in the southern part of Ref. I, in the northern part of Rør I, and in Ref. II. A large deposition in Ref. III is also noted.

Fig. 7.11 shows the total changes in dune + beach volumes, i.e. $\Delta E_1 + \Delta E_2$. By April 2005 there was significant accumulation in most of Rør I, Rør II and Ref. III, but more pronounced erosion in parts of Ref. I and Ref. II. This picture is more or less maintained in the later surveys and is also seen in the July 2006 survey. The only exception is that very pronounced deposition took place in Ref. III. The survey in January 2007 revealed that over the two-years period erosion took place in a part of Ref. I and in Ref. II, while in Rør I and Rør II a more variable picture of erosion/deposition (mainly deposition) is seen. Moreover, a very significant deposition took place in Ref. III.

Table 7.4 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.

Table 7.4. Approximate average dune plus beach volume changes ($\Delta E_1 + \Delta E_2$) from January 2005 to January 2006 and January 2007. Positive values are deposition.

Stretch	**m^3/m coastline**		**Total m^3 over stretch**	
	Jan 05-Jan 06	Jan 05-Jan 07	Jan 05-Jan 06	Jan 05-Jan 07
Ref. I	3	-24	4.620	-42.650

Rør I	44	10	204.577	48.322
Ref. II	-20	-101	-36.011	-181.673
Rør II	126	44	113.472	39.267
Ref. III	80	115	143.110	206.800

For the two years period Jan 05 – Jan 07 it is seen that net deposition has taken place in Rør I and Rør II, but mainly in Ref. III whereas erosion and deposition has taken place in Ref. I. and Ref II. Ref. II shows a significant erosion. The net increase in beach and dune volumes over the total length of the test site amounts to app. 70.000 m^3 in total.

From Table 7.4 it is also seen that for the first year period Jan 05 – Jan 06 the deposition, except in Ref. III, were larger and the erosion smaller. This is because the first-year period was relative quiet with no significant storms. In the second year however, four storms occurred, three of which took place in January 2007.

7.3.4 Dune, beach and nearshore volume

The changes in volume of the near shore zone as defined in Fig. 7.2 (calculated as $\Delta D1+ \Delta D2+\Delta D3 -\Delta E1-\Delta E2$) are shown in Fig. 7.6. As expected for this very dynamic zone there are many shifts between deposition and erosion along the test site and no correlation with drained and non-drained stretches. The only persistent configuration is a deposition in Rør I app. 1 km South of the northern border, seen in all the surveys.

7.3.5 Changes in mean level of a 100 m wide beach zone measured from position of level +4.00 m in the January 2005 profile

The initial values of MBL, in January 2005, shown in the top diagram of Fig. 7.7, are not evenly distributed over the test area. Large values of MBL $\geq$ 2.0 m existed only in Ref. II near the border to Rør II. Values larger or equal to 1.5m were present mainly in Rør I and Ref. II. It is important to notice the extremely low MBL value of app. zero at the border between Rør I and Ref II, as this “hole” is more or less maintained in the two years period as seen in Figs. 7.7 and 7.8. Thus this initial weakness of the beach was never repaired although accumulation took place until October 2005 in the southern part of Ref. II.

The initial “hole” in the northern part of Ref. III was repaired as significant accumulation took place in Ref. III. The changes in MBL, i.e. Δ MBL, are shown in Figs. 7.9 and 7.10. It is seen that after the two years the MBL increased mainly in the northern part of Ref. III and the southern part of Rør II as well as in the middle part of Rør I and northern part of Ref. I. Lowering of the MBL took part mainly in Ref. II and northern part of Rør I and southern part of Ref. I.

7.3.6. Influence of the bar nourishment on the morphological changes in the test area.

With the data in hand it is not possible to analyze the influence of the bar nourishment on the beach development in the test area. Only it can be said that the dumped sand will be transported mainly towards South. The uncertainty it causes for the analysis of the effect of the drains has been known and discussed from the stage of selection of the tests site, cf. §2.1.

7.3.7. Summary of observations including impacts of the storms

In the period January 2005 to January 2006 no significant changes have taken place in the beach planform as the coastline undulations have more or less maintained their positions except that in the southern part of Rør II and the northern part of Ref. III the coastline has significantly moved seaward and there seems to be a tendency that the undulations are moving southwards. It can be observed that significant accumulation of sand has taken place within the two areas with drains, Rør I and Rør II, i.e. the beach level has been raised. The same or even stronger development is however observed in Ref. III with no drains, whereas Ref. I also with no drains exhibit both erosion and accretion. Ref. II generally shows erosion.
This observed development took place in the first year period with no severe storms and extreme high water levels after the very severe storm around 8 January 2005. At that occasion large quantities of sand was probably eroded from the beach. Usually part of this sand will be transported back to the beach in periods with milder wave climate, normally occurring in the spring and the summer. Also sand nourishment might contribute to the accretion of the beach. Actually, the migration pattern of the nourishment sand is not clear, it may go on-, off- or long-shore. However, twice as much sand was nourished as what was accumulated on the beach in the one-year period.

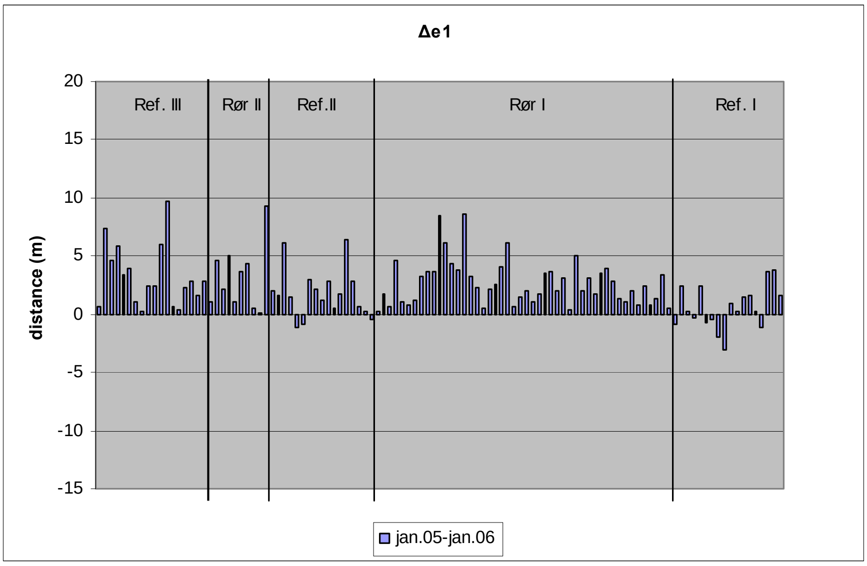
The second year was quiet with no storms until the occurrence of a moderate storm in October 2006 and three more severe storms in January 2007. Until the occurrence of these storms there was no significant changes shown in the beach morphology compared to the end of the first year. In order to investigate the effect of the storms are shown in Figs. 7.6 the conditions/strength in July 2006 of the stretches given by the beach width e2, and the mean level MBL of the 100 m width of beach, together with the erosion of dune and beach (EΔ1+EΔ2) between July 2006 and January 2007 (just after the storms).

From Fig. 7.6 it is seen that severer erosion (say ≥ 90 m^3/m) took place over limited distances in all stretches. It is also seen that - as expected – there is a 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 fact that also other conditions than beach strength influence the erosion in storms. The most likely factor is the nearshore bar formation, as explained in §3.2. The difficulty in dealing quantitatively with the effect of the bars is not only related to lacking information in more quiet periods but indeed to the rapid changes in bar topography during storms.

In order to see if there is correlation between beach erosion and nearshore deposition (assumes that the sediment transport is only in the cross-shore direction which – for sure – is not the case) are depicted in Figs. 7.7 and 7.8 the values of the changes in the box volumes (ΔD1, ΔD2, ΔD3 and ΔD4) averaged over each stretch. The boxes cover beach and sea to a distance of 700m from the foot of the dune, cf. Fig. 7.1. It seems that there is no stringent correlation as expected. Table 7.5 shows the total volume changes in the boxes at different times for each stretch. Although it should not be taken as a good measure of performance it is seen that Ref. III, Ref. I andf Rør I have the best performance and Ref. II and Rør II the worst.

Table 7.5. Total volume changes in Boxes 1, 2, 3 and 4 in m^3.

Period from Jan 05 to	Ref. III	Rør II	Ref. II	Rør I	Ref. I
Apr 05	26	21	16	23	65
Jul. 05	60	67	25	39	35
Oct. 05	86	157	8	37	16
Jan. 06	92	137	-68	34	47
Jul. 06	128	54	-31	41	87
Jan. 07	192	-43	-141	39	102



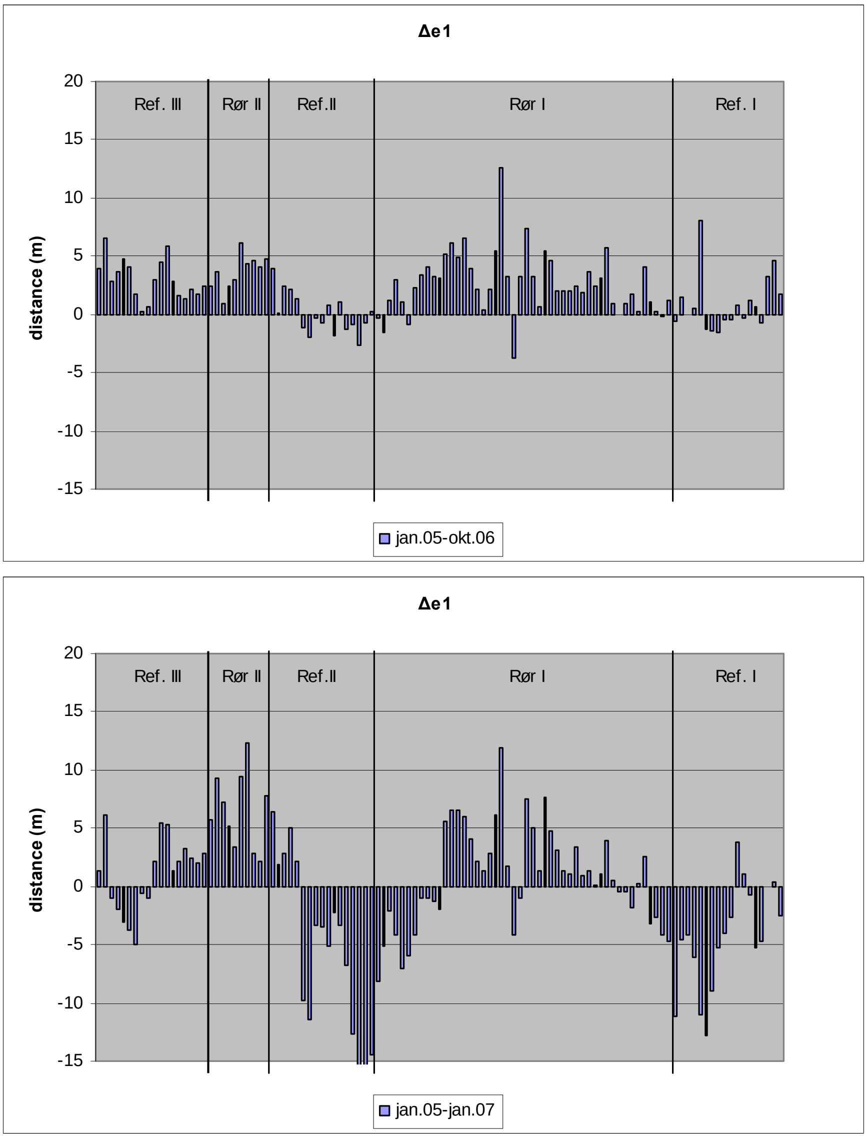
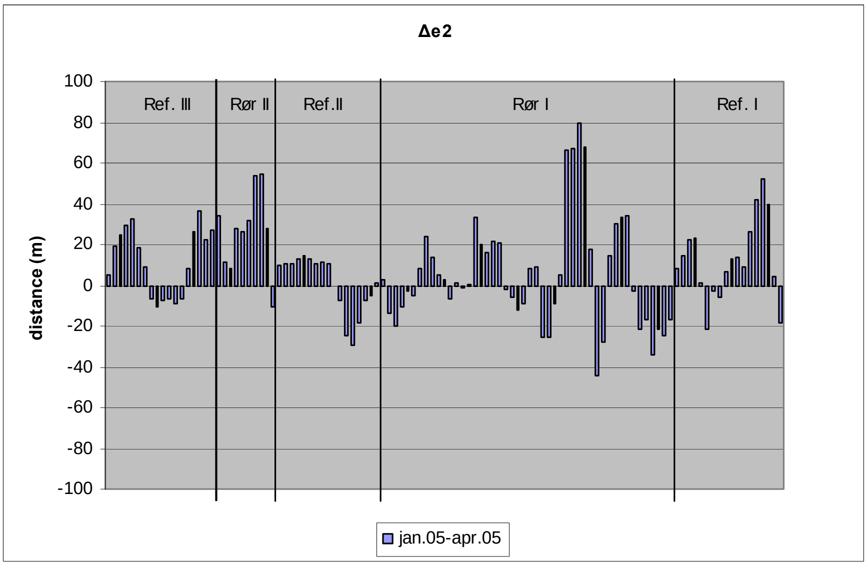
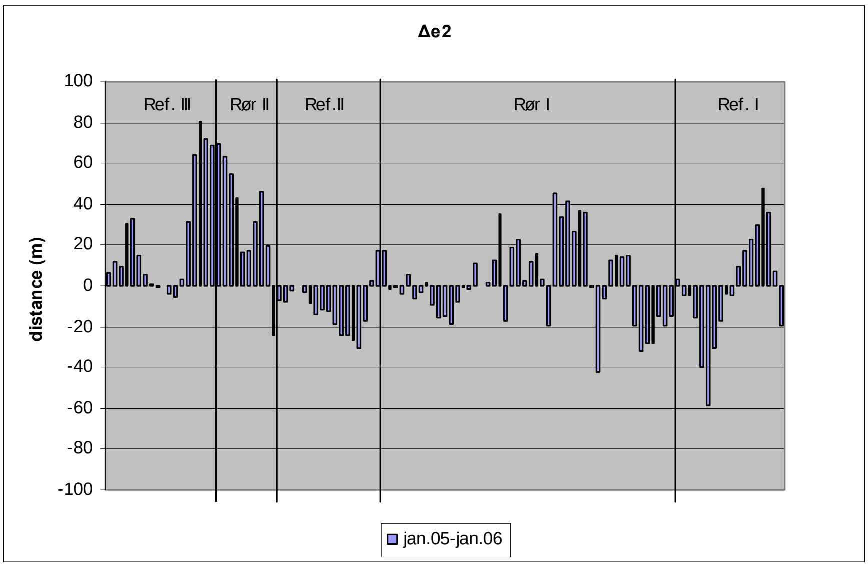


Figure 7.3. Changes in dune foot position


Δe2
100
80
60
40
20
0
-20
-40
-60
-80
-100
distance (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
□ jan.05-apr.05


Δe2
100
80
60
40
20
0
-20
-40
-60
-80
-100
distance (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
□ jan.05-jan.06

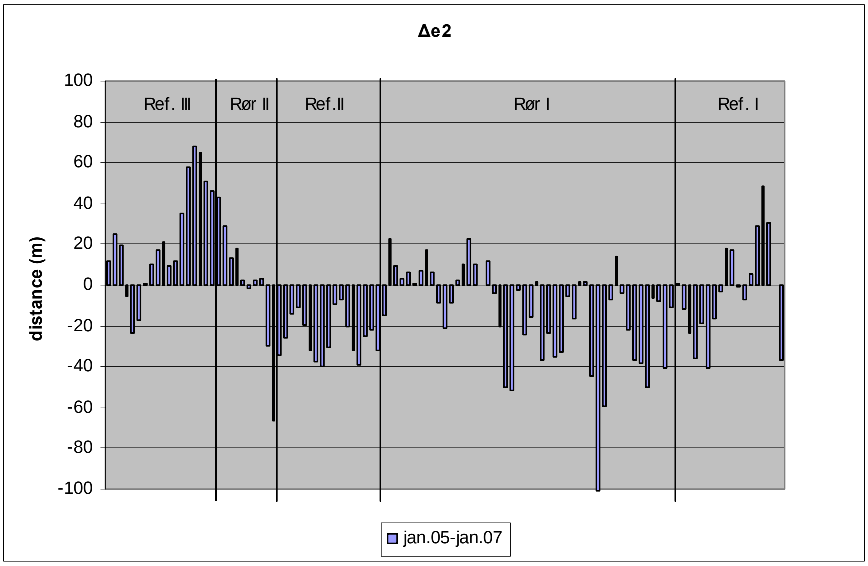
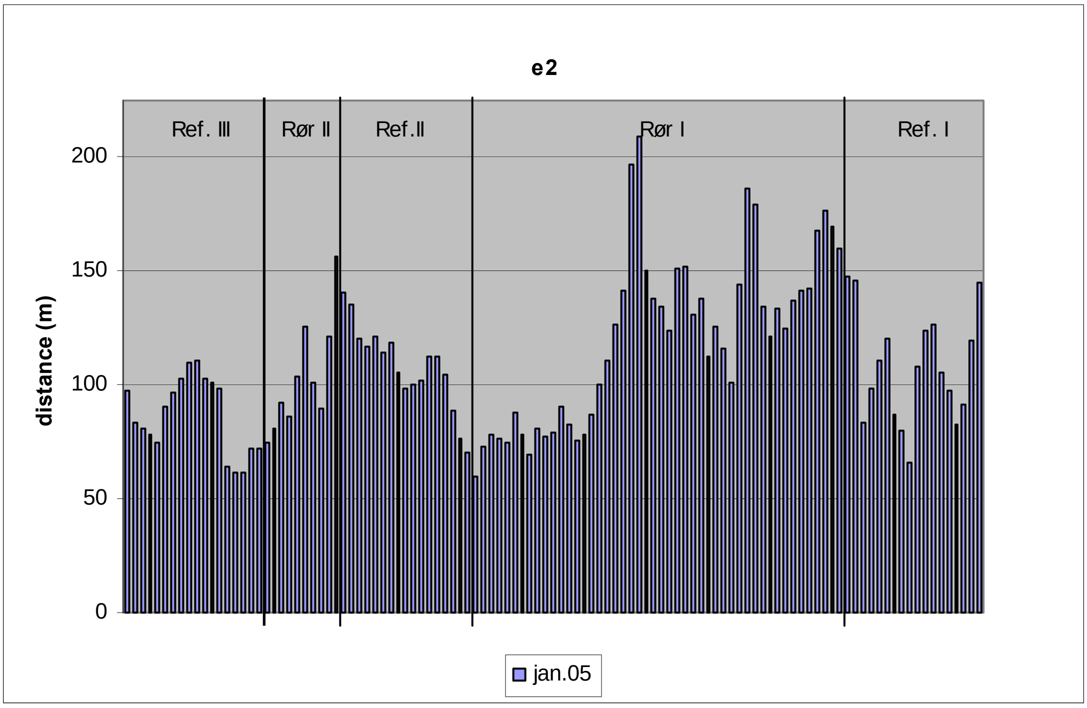

Δe2
100
80
60
40
20
0
-20
-40
-60
-80
-100
distance (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
□ jan.05-jan.07

Figure7.4 Changes in position of shore line (level 0.00m)



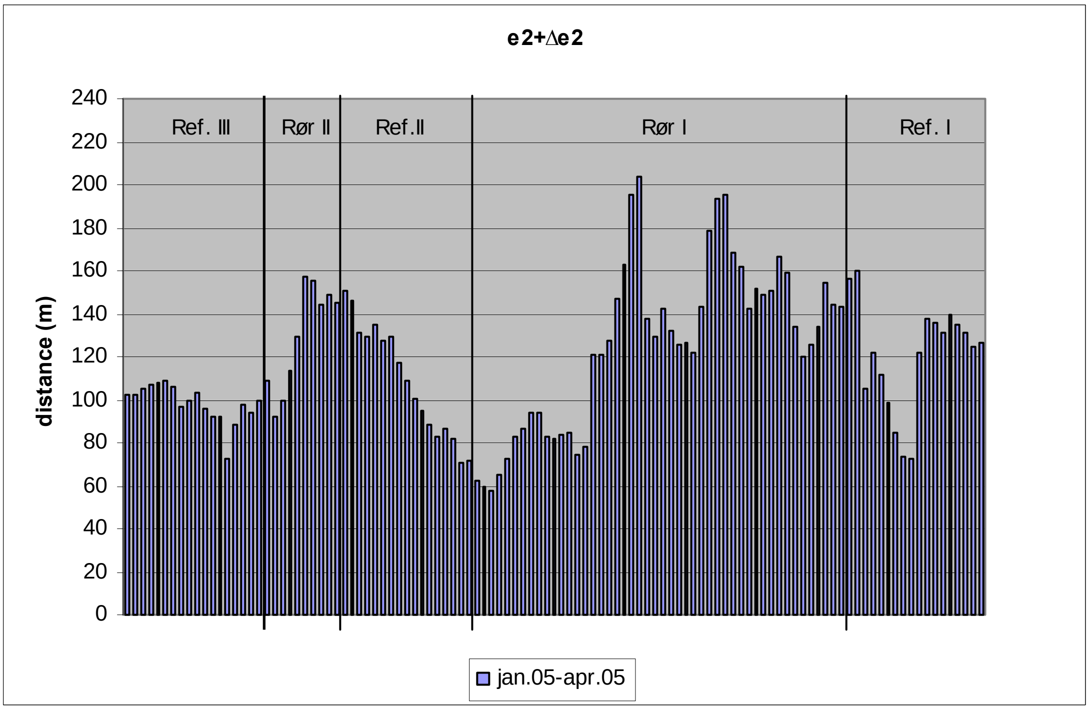
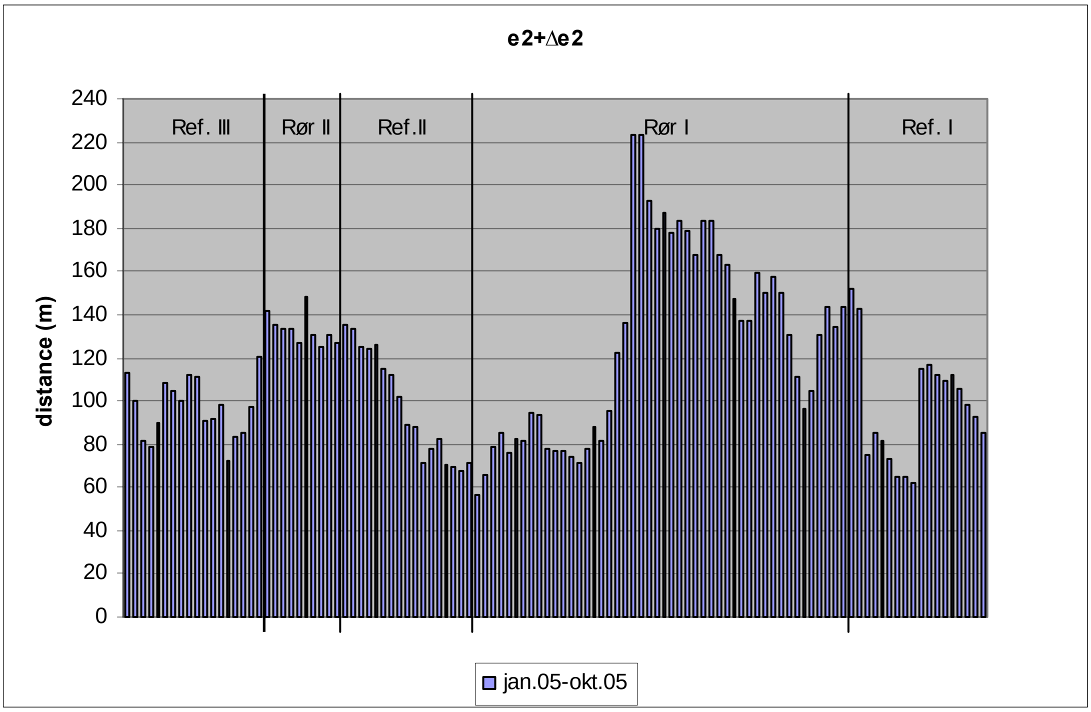


Figure 7.5. Beach widths January 2005 and April 2005



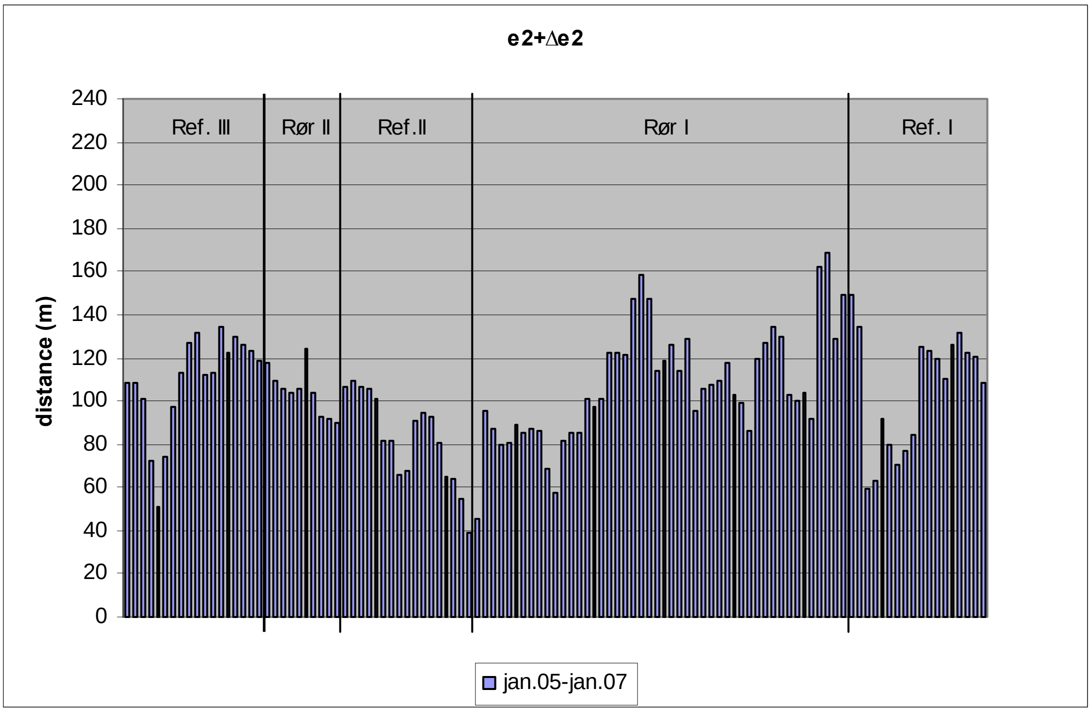
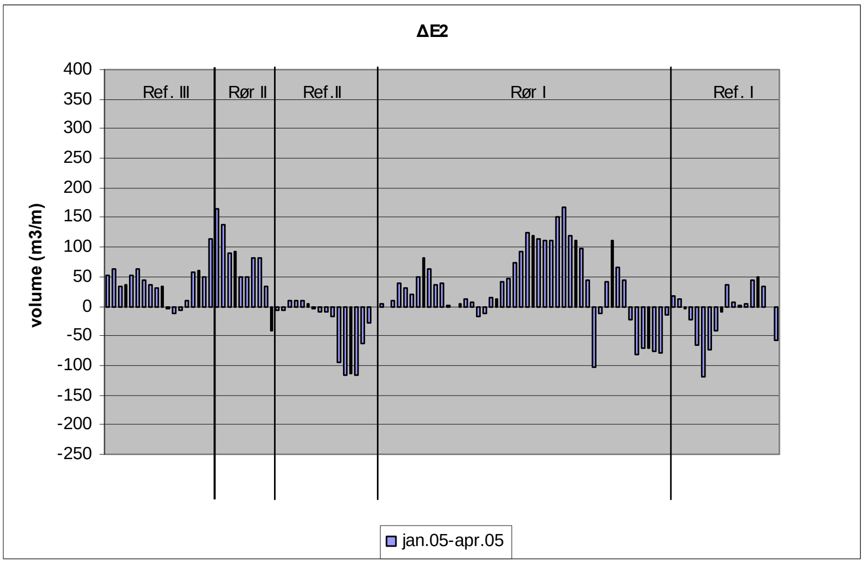


Figure 7.6. Beach widths October 2006 and January 2007



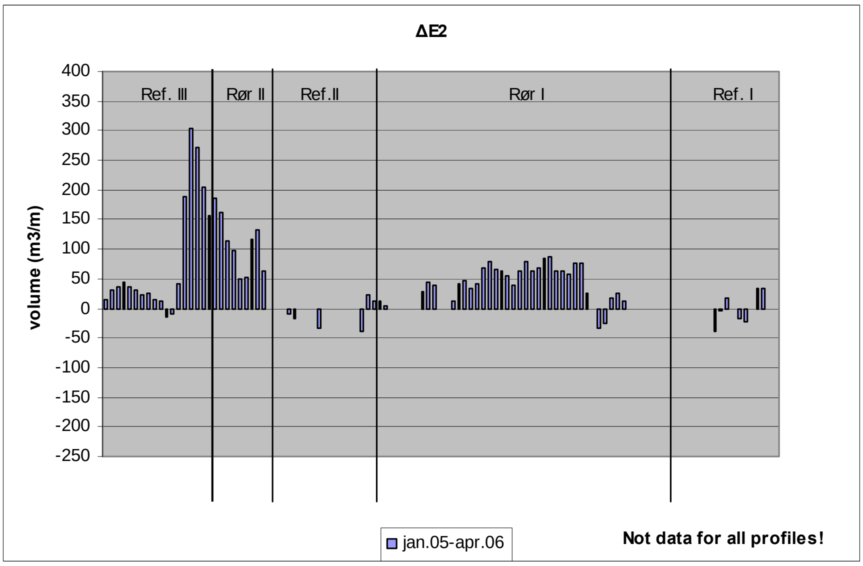
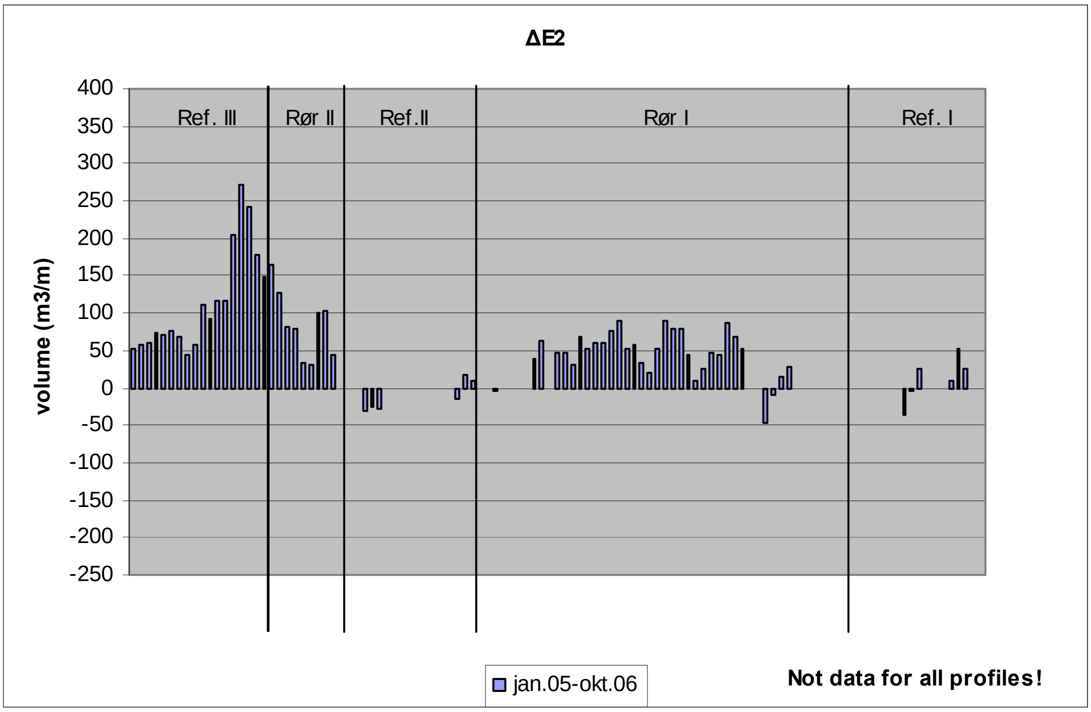


Figure 7.7. Changes in beach volume from January 2005 to April 2005, April 2006 and July 2006



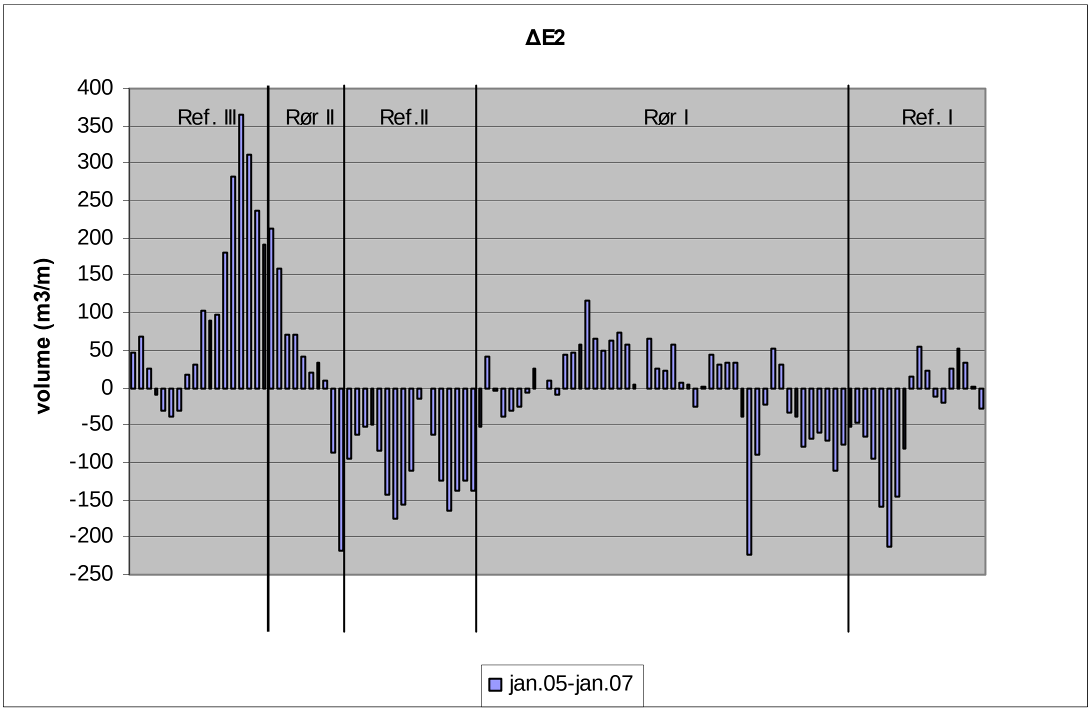
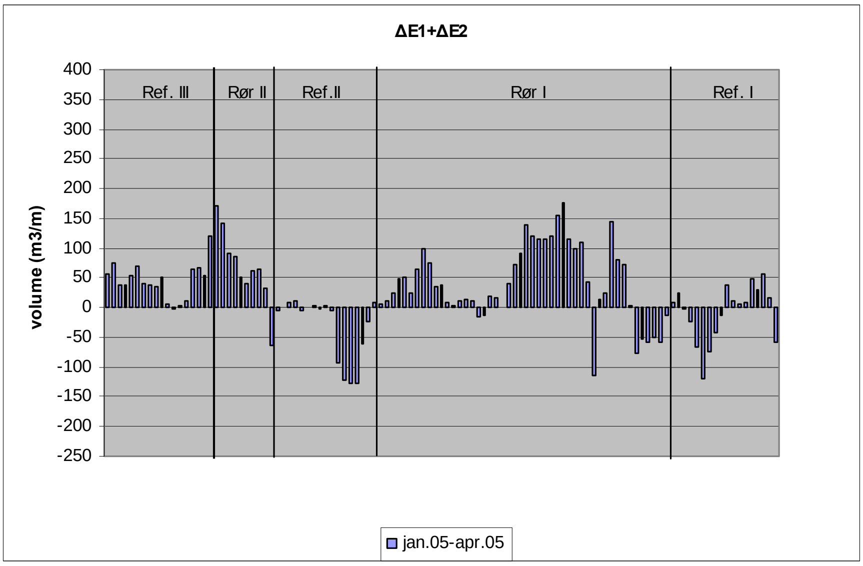
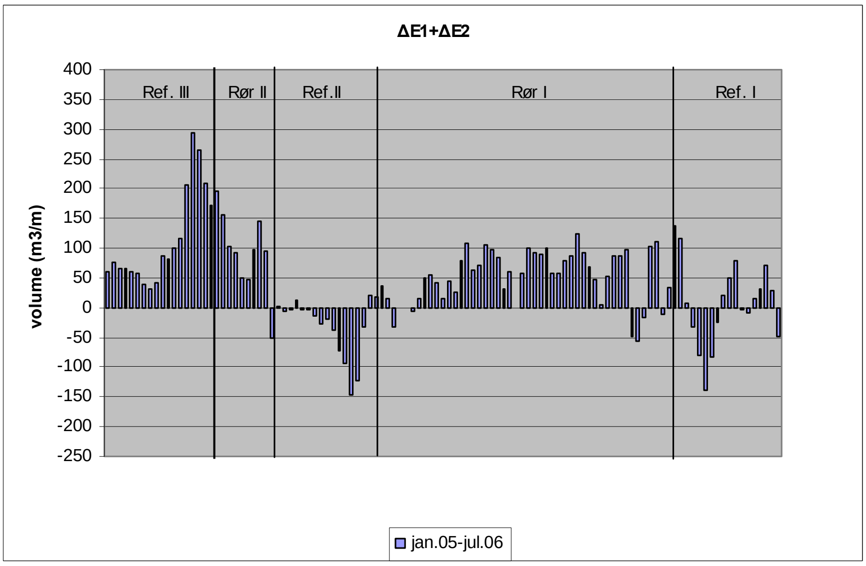


Figure 7.8 Changes in beach volume from January 2005 to October 2006 and January 2007.


ΔE1+ΔE2
400
350
300
250
200
150
100
50
0
-50
-100
-150
-200
-250
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-apr.05


ΔE1+ΔE2
400
350
300
250
200
150
100
50
0
-50
-100
-150
-200
-250
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.06

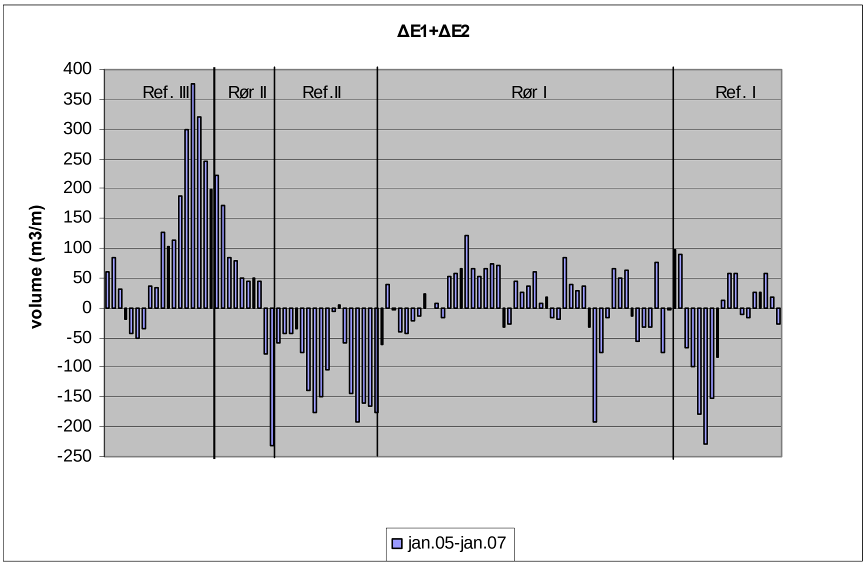
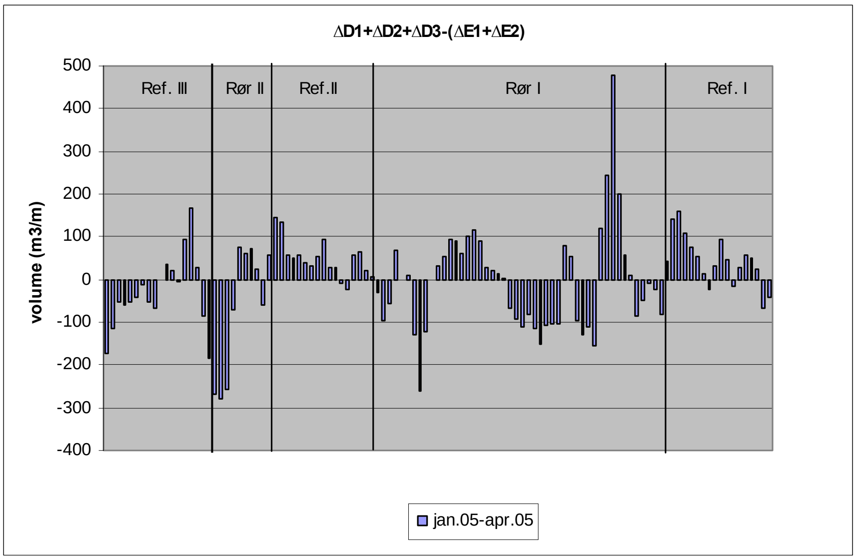
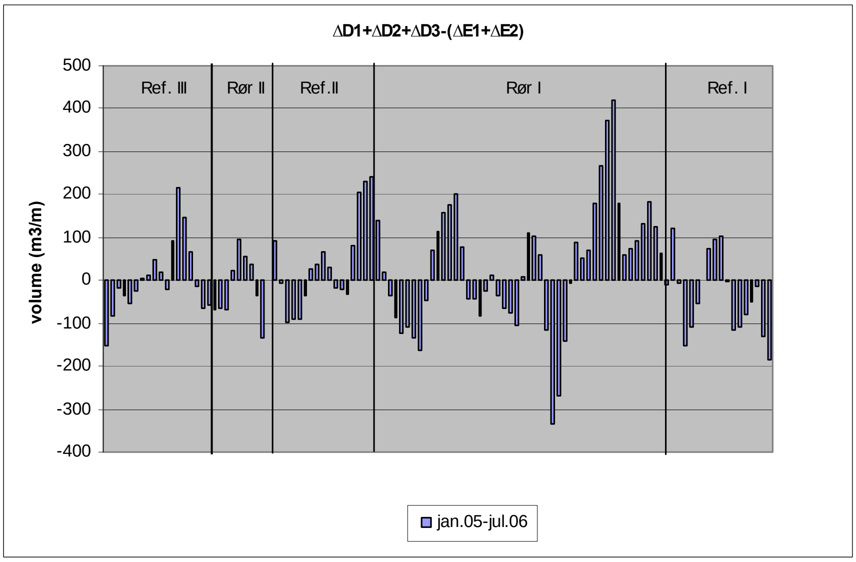

ΔE1+ΔE2
400
350
300
250
200
150
100
50
0
-50
-100
-150
-200
-250
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.07

Figure 7.9 Changes in total volumes of dune plus beach.





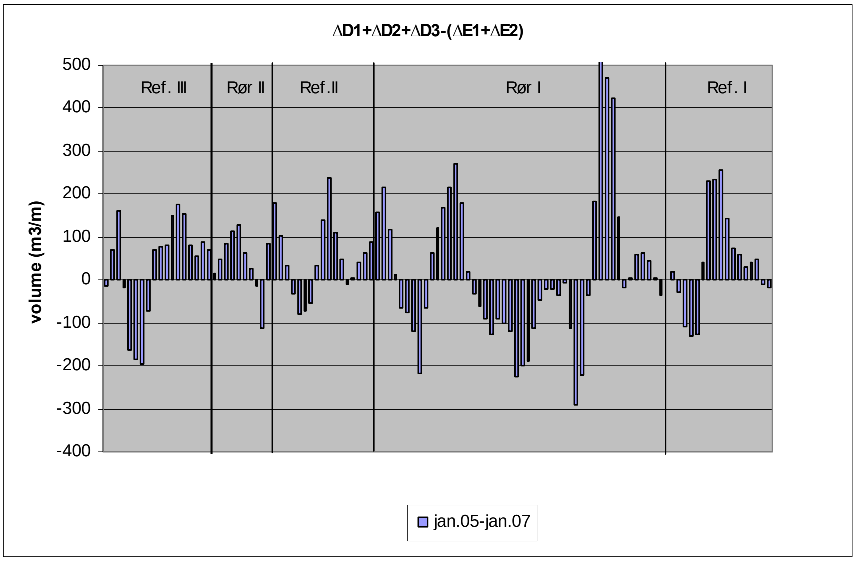
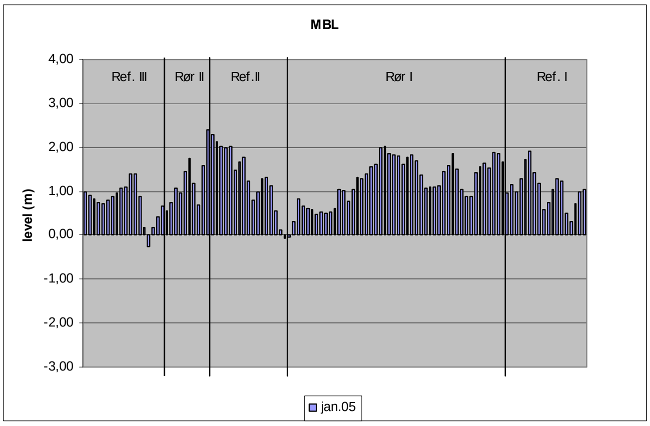
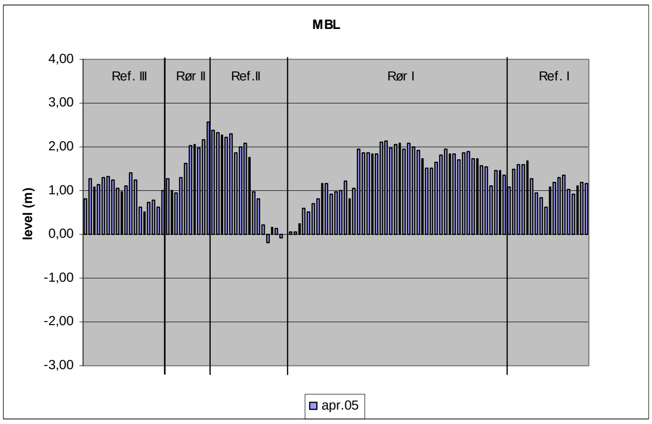
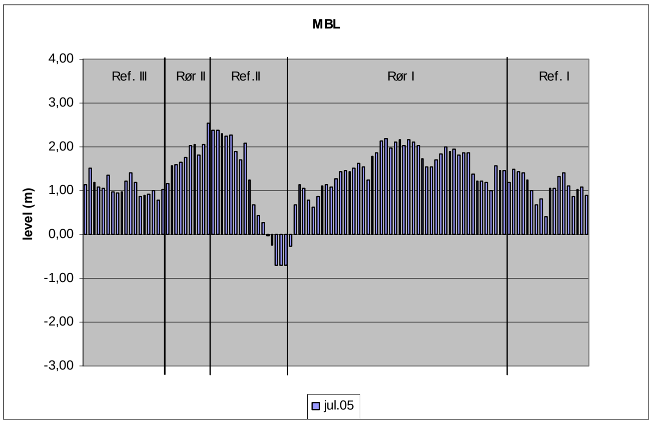


Figure 7.10 Changes in volume of the near shore Zone.







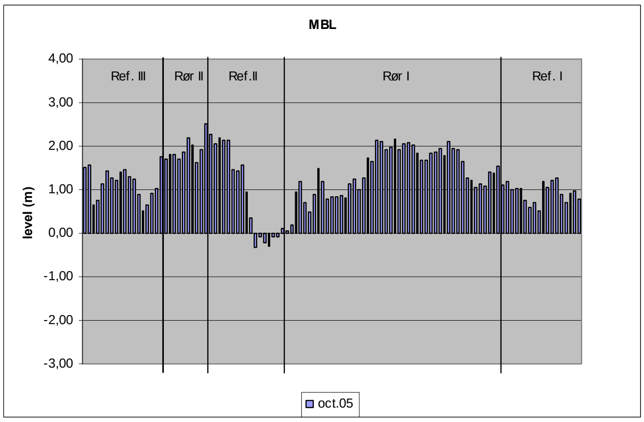
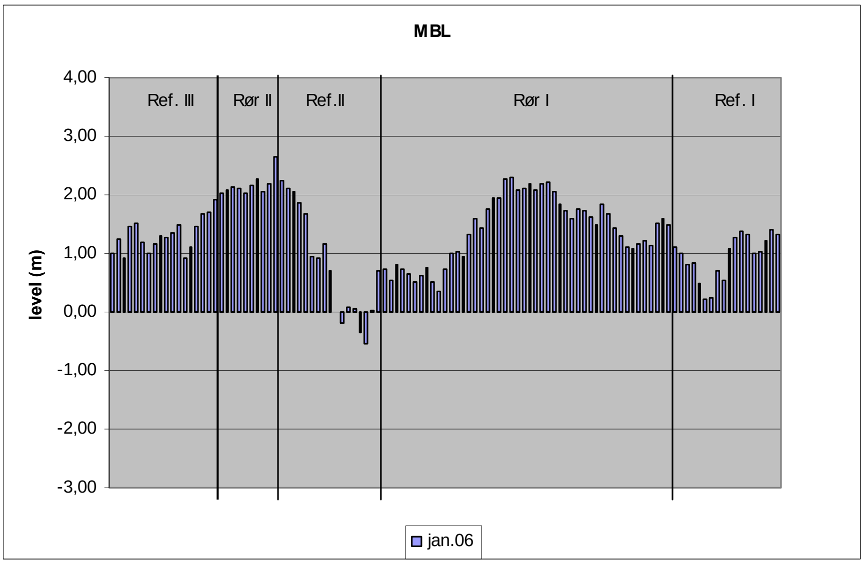
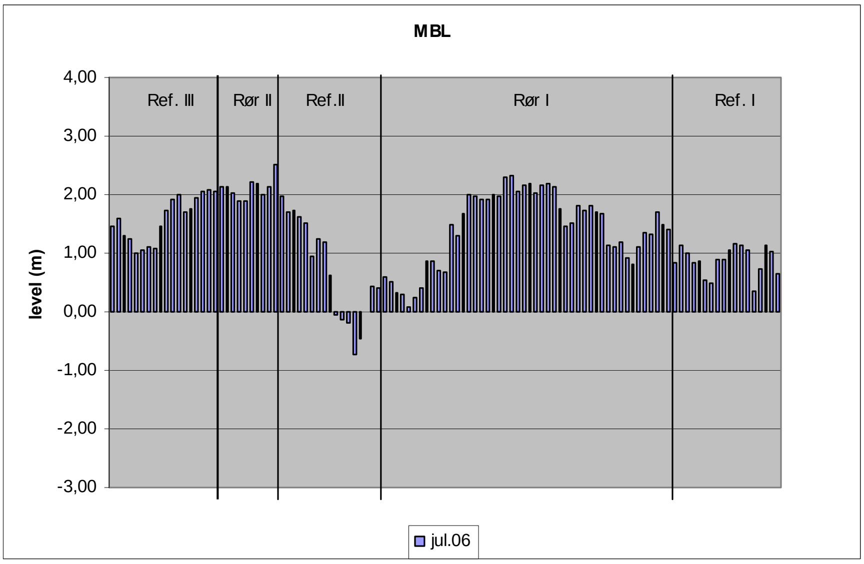


Figure 7.11 Average beach level, MBL, of a 100 m wide zone seaward of the dune foot position of January 2005. January, April, July and October 2005.





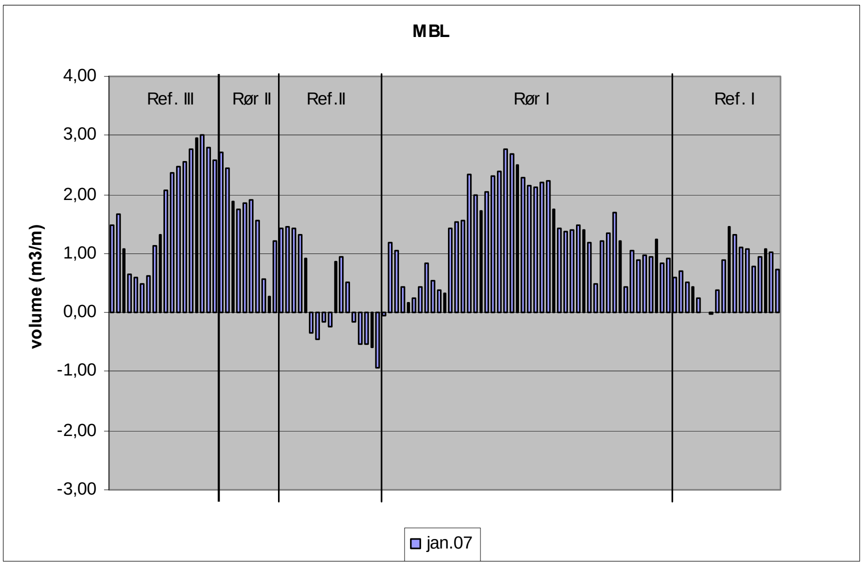
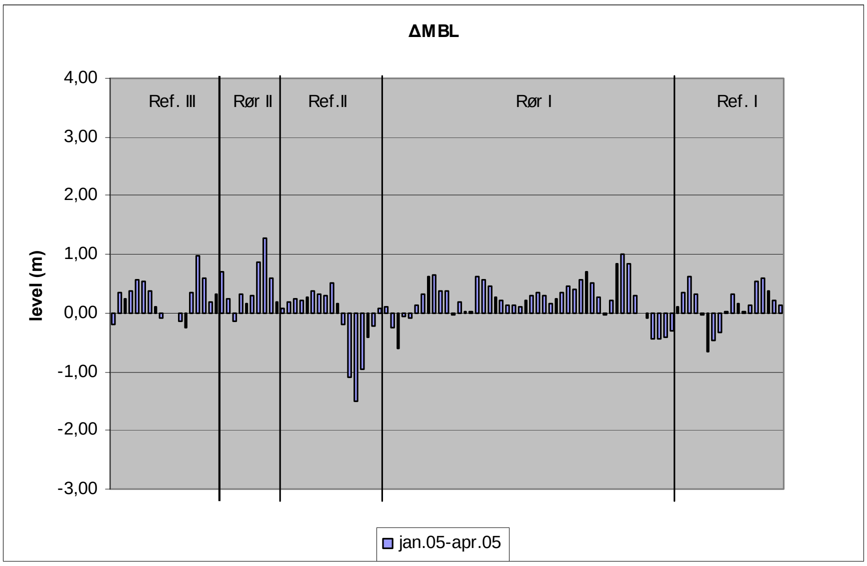
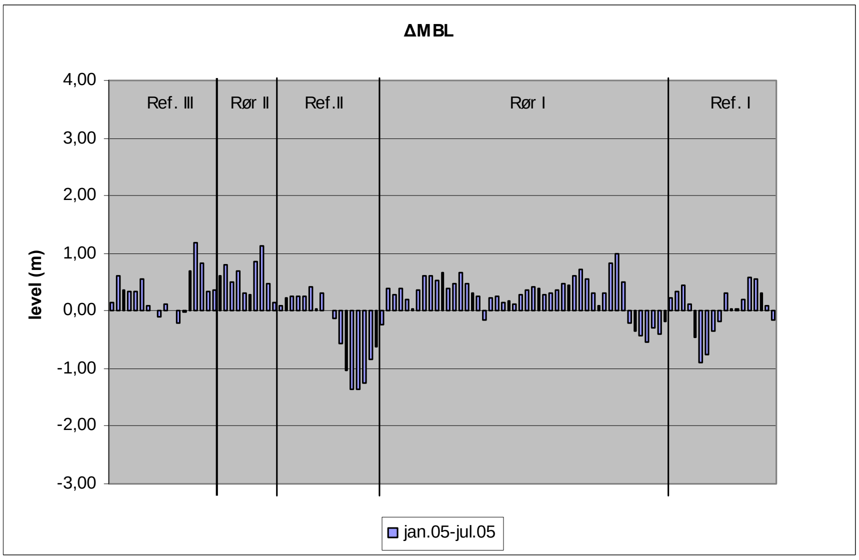
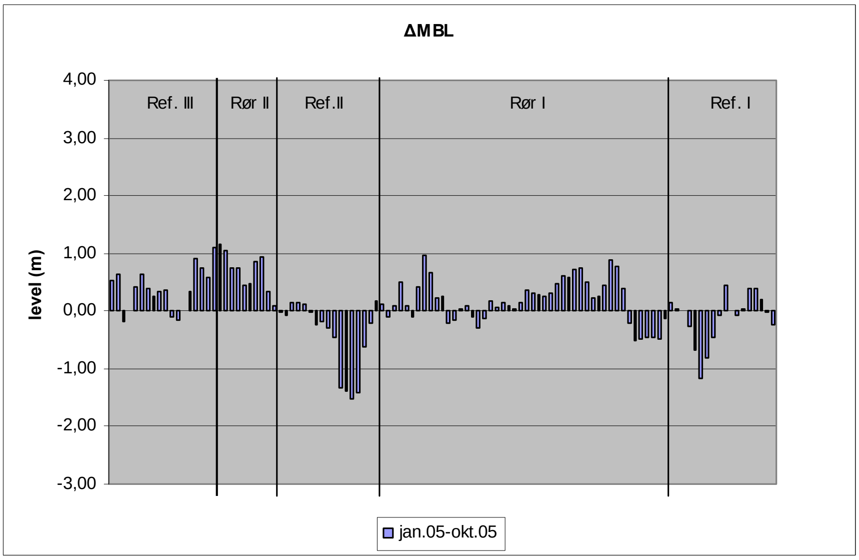


Figure 7.12 Average beach level, MBL, of a 100 m wide zone seaward of the dune position of January 2005. January and July 2006, and January 2007.


ΔMBL
4,00
3,00
2,00
1,00
0,00
-1,00
-2,00
-3,00
level (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-apr.05


ΔMBL
4,00
3,00
2,00
1,00
0,00
-1,00
-2,00
-3,00
level (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.05


ΔMBL
4,00
3,00
2,00
1,00
0,00
-1,00
-2,00
-3,00
level (m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-okt.05

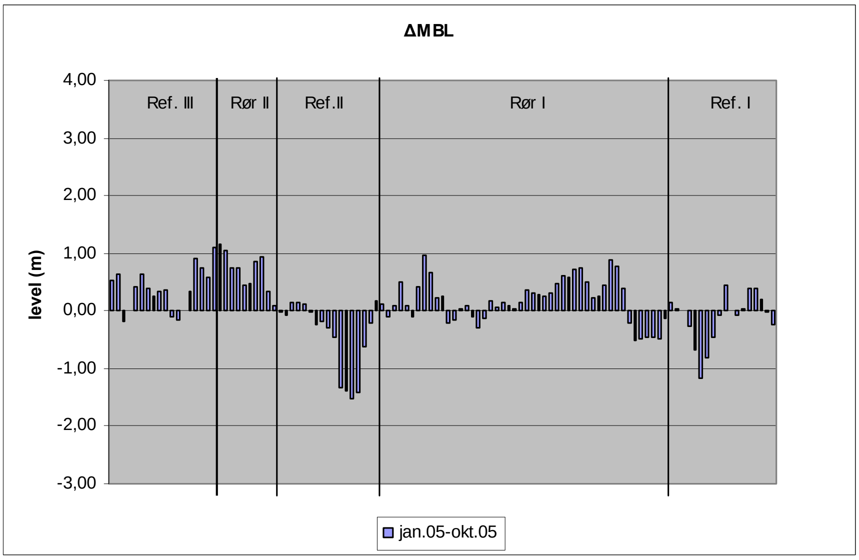
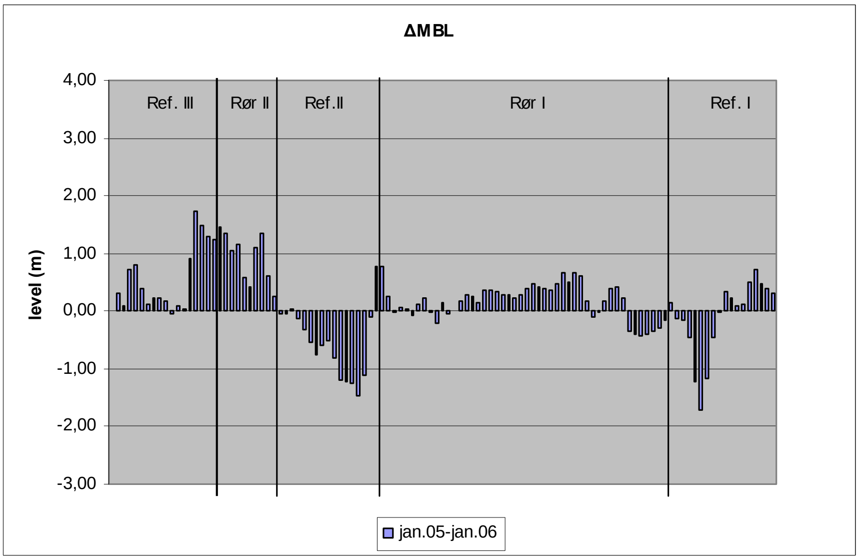
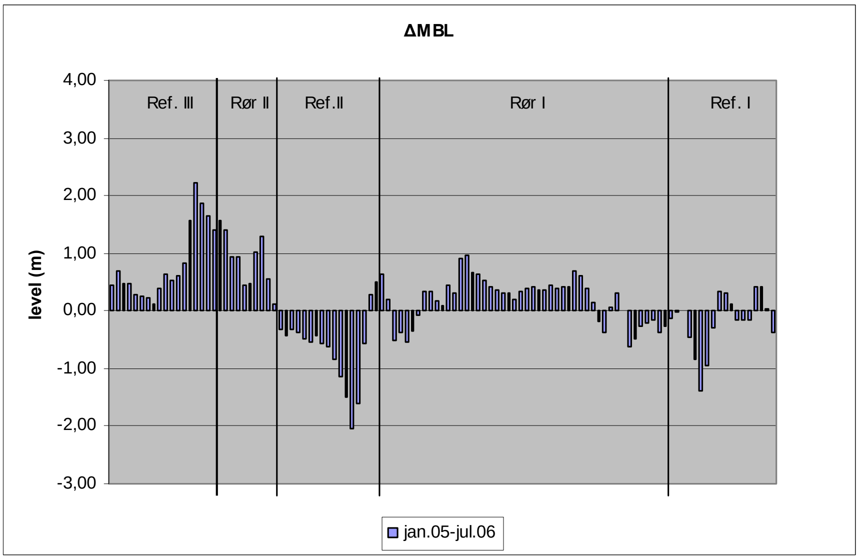


Figure 7.13 Changes in MBL, ΔMBL, from January 2005 to July and October 2005.





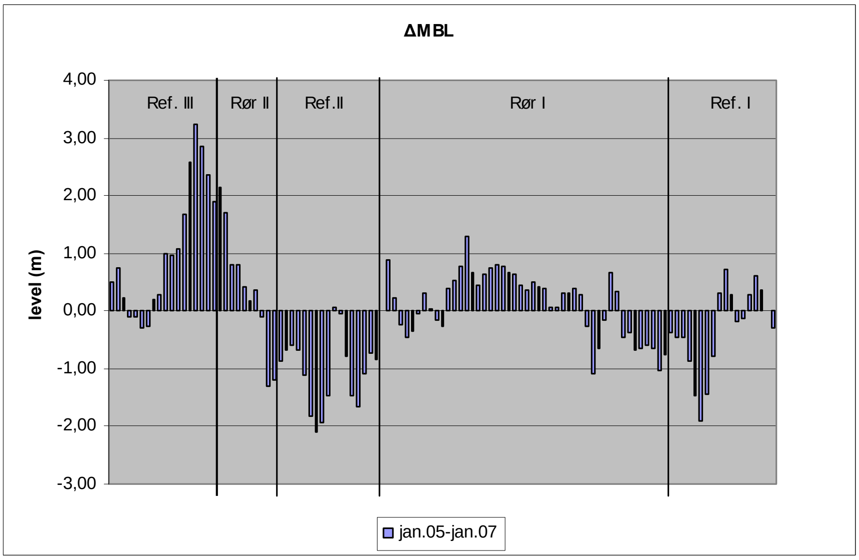
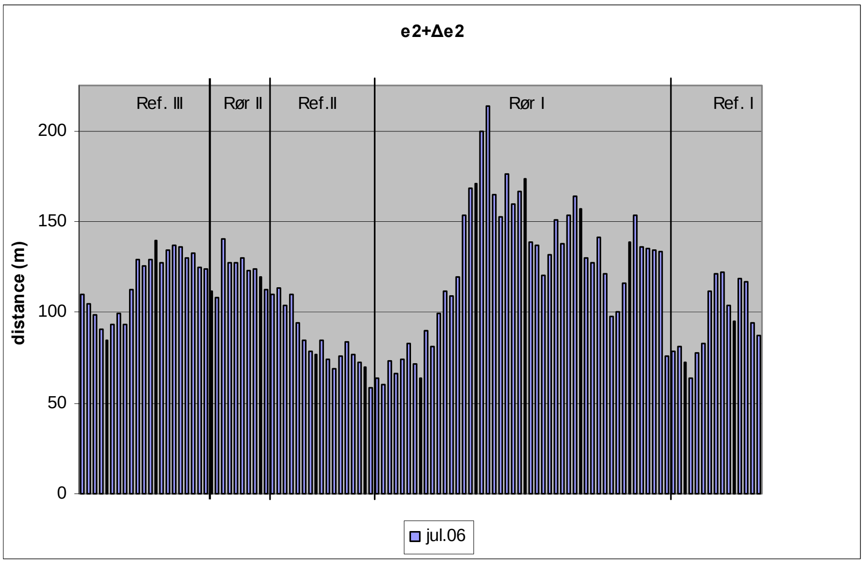
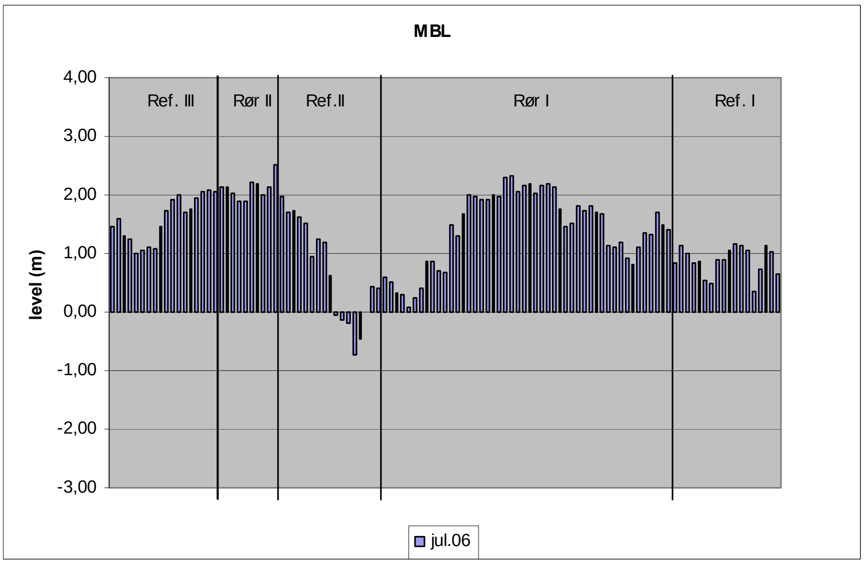


Figure 7.14 Changes in MBL, ΔMBL, from January 2005 to January 2006, July 2006 and January 2007.


e2+Δe2
Ref. III
Rør II
Ref. II
Rør I
Ref. I
200
150
100
50
0
distance (m)
jul.06


MBL
4,00
3,00
2,00
1,00
0,00
-1,00
-2,00
-3,00
Ref. III
Rør II
Ref. II
Rør I
Ref. I
level (m)
jul.06

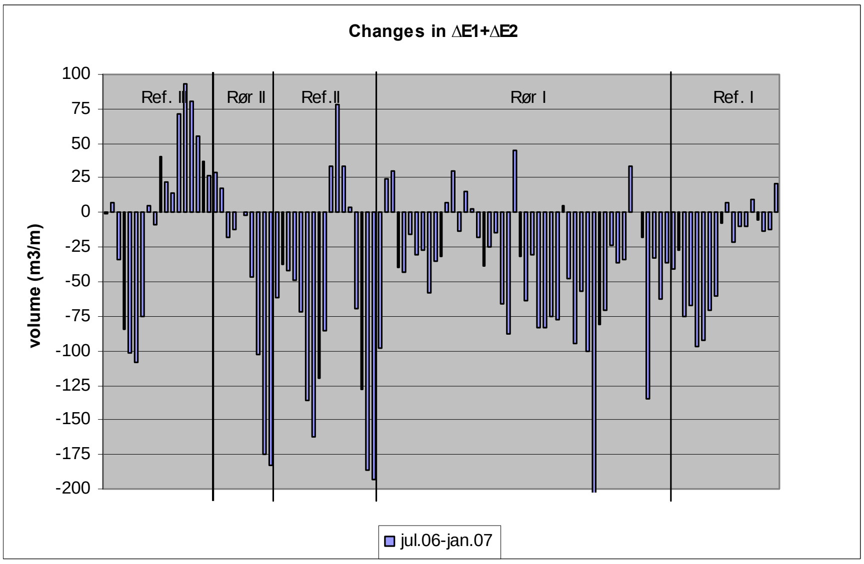
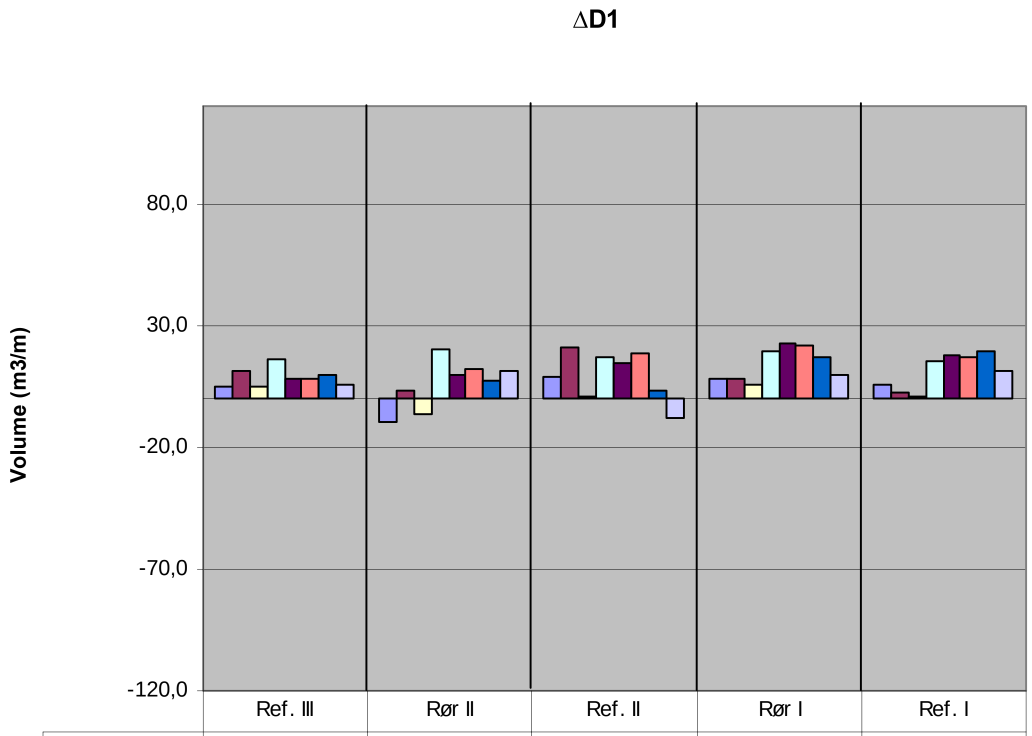
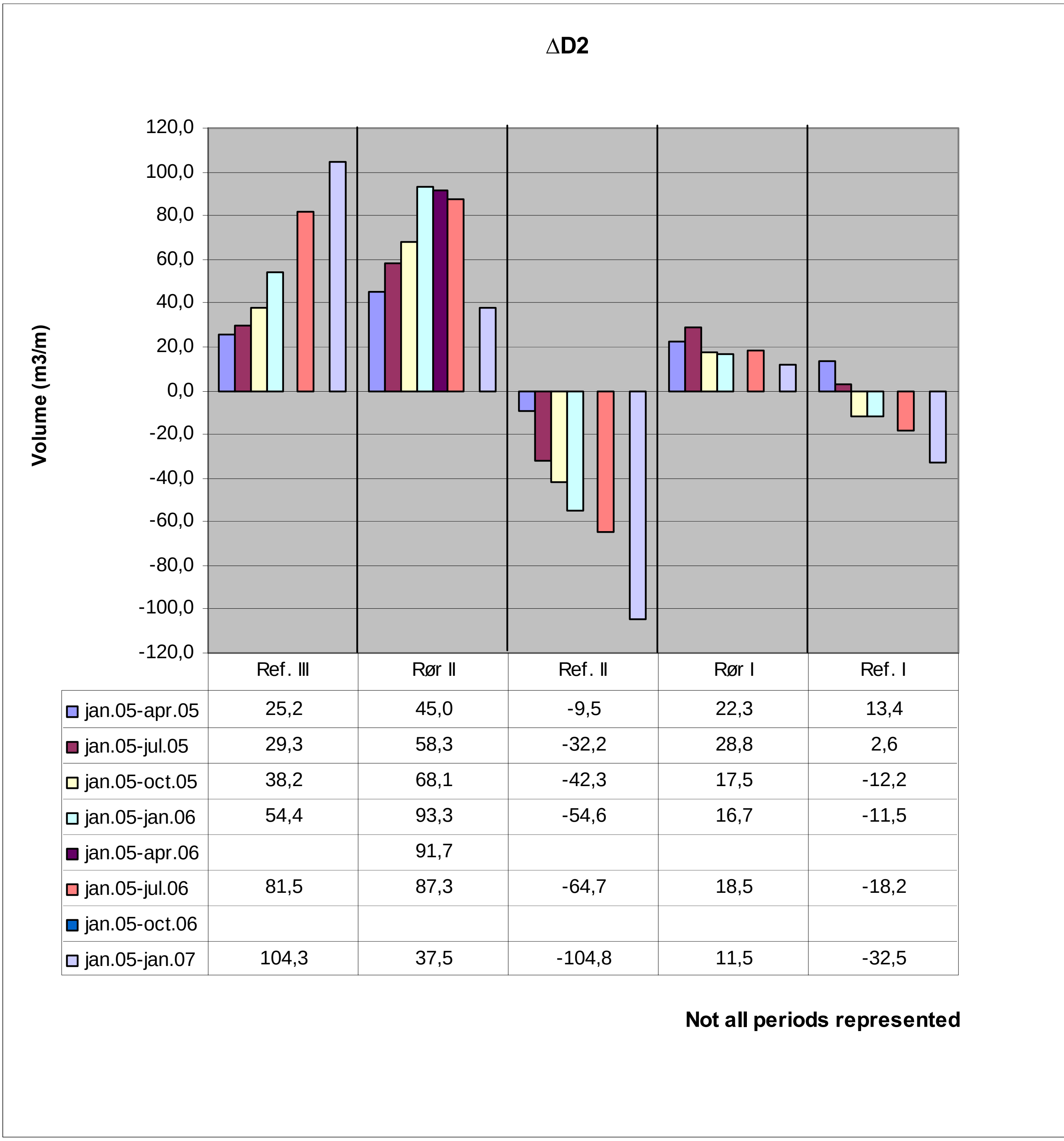

Changes in ΔE1+ΔE2
100
75
50
25
0
-25
-50
-75
-100
-125
-150
-175
-200
Ref. I
Rør II
Ref. II
Rør I
Ref. I
volume (m3/m)
jul.06-jan.07

Figure 7.15 Relative strength per July 2006 of the stretches and effect of the storms October 2006 and January 2007 in terms of dune and beach erosion.



	Ref. III	Rør II	Ref. II	Rør I	Ref. I
■ jan.05-apr.05	4,7	-9,9	9,2	7,7	5,8
■ jan.05-jul.05	11,5	3,5	21,1	8,3	2,8
□ jan.05-oct.05	4,5	-6,8	0,9	5,5	1,1
□ jan.05-jan.06	16,1	19,7	16,9	19,1	15,3
■ jan.05-apr.06	8,0	9,7	14,7	22,5	17,5
■ jan.05-jul.06	7,7	11,7	18,8	21,9	17,1
■ jan.05-oct.06	9,7	6,9	3,5	16,8	19,5
□ jan.05-jan.07	5,6	10,9	-8,4	9,5	11,6

Not all periods represented



	Ref. III	Rør II	Ref. II	Rør I	Ref. I
■ jan.05-apr.05	25,2	45,0	-9,5	22,3	13,4
■ jan.05-jul.05	29,3	58,3	-32,2	28,8	2,6
■ jan.05-oct.05	38,2	68,1	-42,3	17,5	-12,2
■ jan.05-jan.06	54,4	93,3	-54,6	16,7	-11,5
■ jan.05-apr.06		91,7			
■ jan.05-jul.06	81,5	87,3	-64,7	18,5	-18,2
■ jan.05-oct.06					
■ jan.05-jan.07	104,3	37,5	-104,8	11,5	-32,5

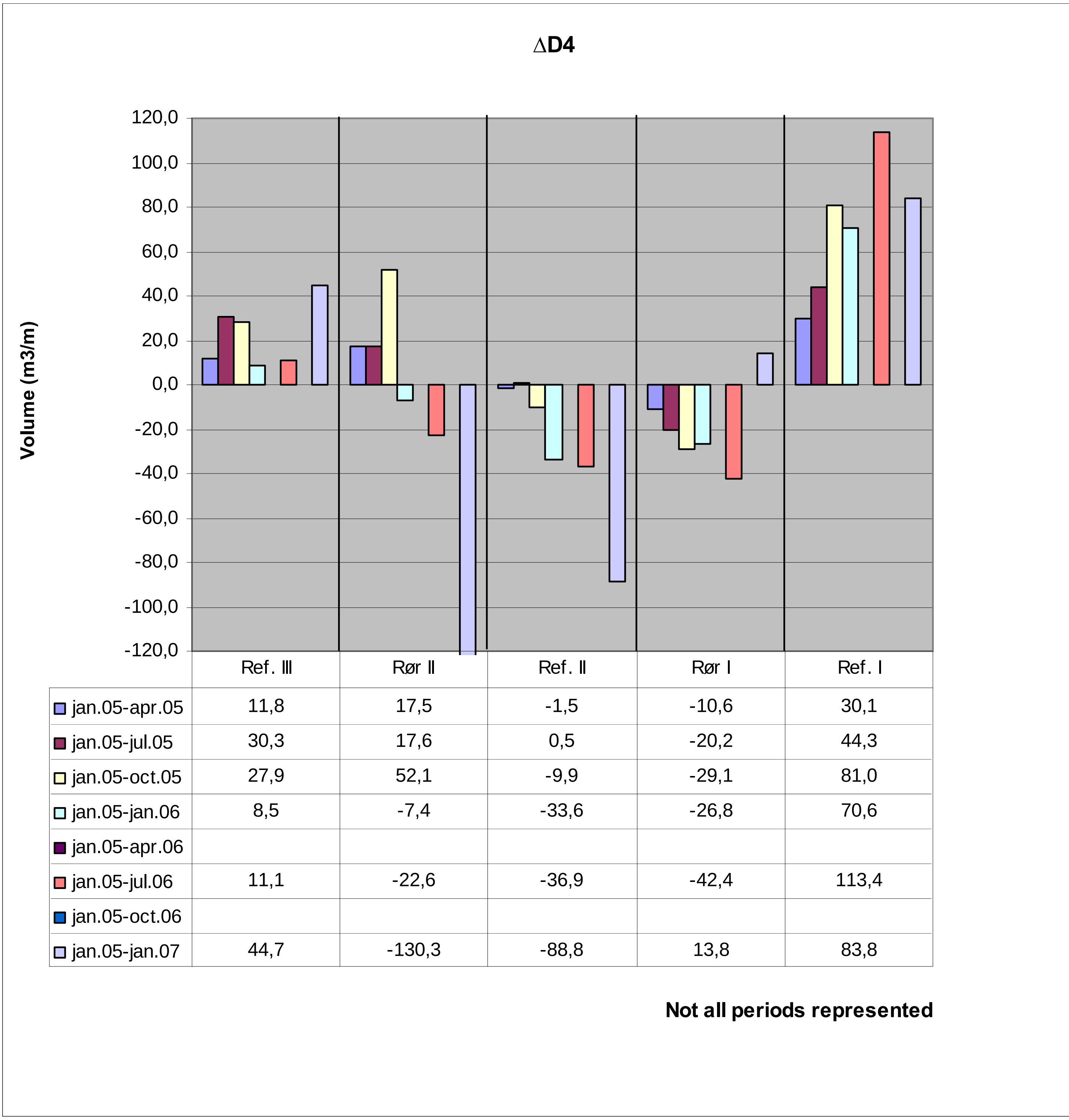
Not all periods represented

Figure 7.16 Averaged volume changes in Box 1 and Box 2.

ΔD3

	Ref. III	Rør II	Ref. II	Rør I	Ref. I
jan.05-apr.05	-15,7	-32,1	18,0	3,6	15,8
jan.05-jul.05	-10,9	-12,2	35,7	21,6	-14,7
jan.05-oct.05	14,1	43,5	58,9	42,6	-54,3
jan.05-jan.06	13,4	30,9	2,9	25,3	-27,9
jan.05-apr.06					
jan.05-jul.06	27,9	-22,9	51,5	43,2	-25,6
jan.05-oct.06					
jan.05-jan.07	37,4	38,7	61,3	4,4	39,3

Not all periods represented



	Ref. III	Rør II	Ref. II	Rør I	Ref. I
■ jan.05-apr.05	11,8	17,5	-1,5	-10,6	30,1
■ jan.05-jul.05	30,3	17,6	0,5	-20,2	44,3
■ jan.05-oct.05	27,9	52,1	-9,9	-29,1	81,0
■ jan.05-jan.06	8,5	-7,4	-33,6	-26,8	70,6
■ jan.05-apr.06					
■ jan.05-jul.06	11,1	-22,6	-36,9	-42,4	113,4
■ jan.05-oct.06					
■ jan.05-jan.07	44,7	-130,3	-88,8	13,8	83,8

Figure 7.17 Averaged volume changes in Box 3 and Box 4.

7.4 Observed trends.

The observed trends are general observations not specifically related to parameter definitions.
A. The beach-box.
After the first year (remembering that a large storm took place January 8th 2005, just before the system was implemented) we had significant accumulation in the beach in Rør 1 and 2 and in Ref 3. We had erosion in the Ref 2, located in between the two reaches with tubes, while ref 1 was neutral(neither erosion or deposition).

In the second year we had significant erosion in all stretches except in Ref 3. In this year severe storms occurred in January 07.

Over the two years we get the picture that we –with one exception – would have anticipated: Erosion at the northern part of the site, and more and more sedimentation as we move south: as mentioned in the introduction, we usually have erosion in the northern part, and accretion south of the southern part, see also the sketch figure 7.18.

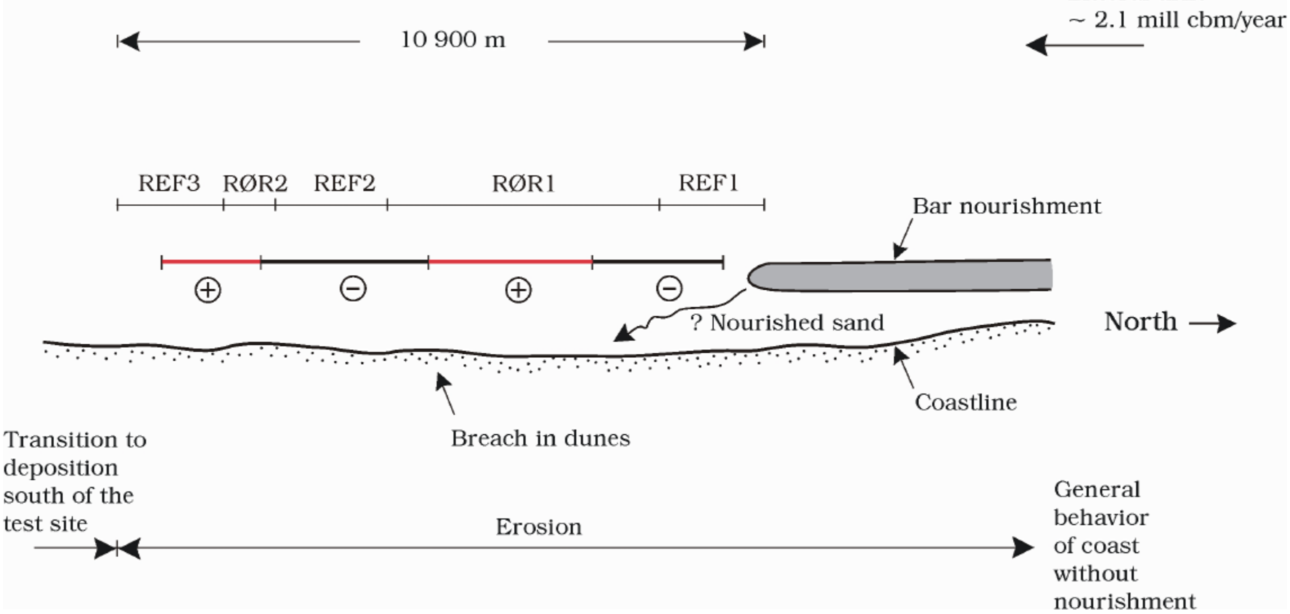


Figure 7.18: General behaviour of the coast at the site.

The exception is the large erosion at reference 2. It could be a proof of a positive impact from the tubes, since the erosion is so large at a location, where there are no tubes.
The question is whether it could be explained otherwise.
We would like to mention at least 4 things

1. Even though Ref 2 in average was quite robust (MBL=1.2In Jan 05), it was very thin and vulnerable in the transition in between Rør 1 and Ref 2, where MBL approximately was zero

over more than 100 meters. Here the waves could attack the foot of the dunes and create a breach in the dunes (which actually happened). Such a lowering in the dune ridge will create a concentration of wind during storms, and this wind will transport a lot of windblown sand landwards and hence accelerate the erosion of the beach locally as sketched in figure 7.20.

Figure 7.19: Observed breach in the dune close to the position between Rør 1 and Ref 2.

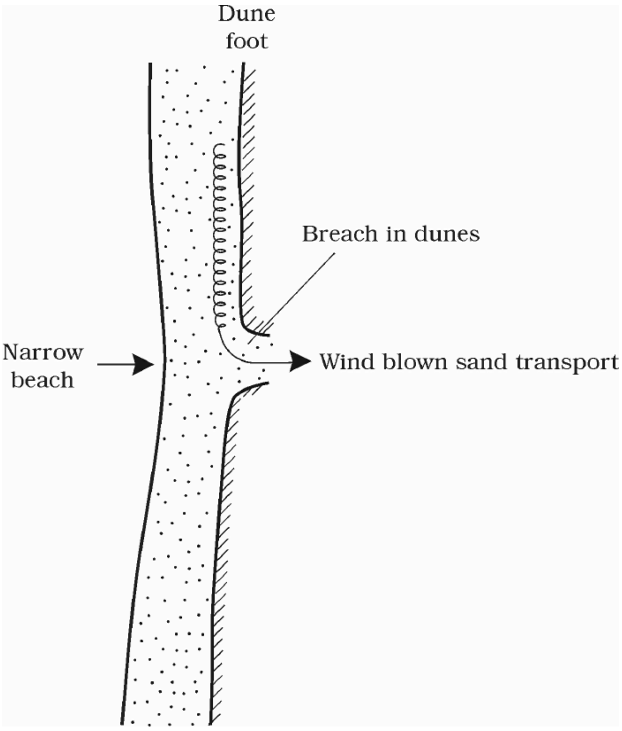


Figure 7.20: A breach will accelerate the wind born sediment transport through the dune system and will result in a narrowing of the beach.

2. The outer bar around 3-400 meter offshore seems to stop just outside the location, where the beach becomes narrow, see figure 7.21. The bar-behaviour in the entire region can be quite strange because of the large nourishment on the bar just north of ref 1, see figure 7.18. A hole in the outer bar or termination of the bar can imply, that waves can penetrate more onshore without breaking (on the bar), and hence be the cause to the narrower beach, see the sketch figure 7.22. Figure 7.23, 7.24 and 7.25 illustrate other possible mechanisms which might be responsible for getting narrow beaches on some locations: concentration of the long shore current behind the bars, presence of rip holes in the bars, and migrating long shore undulations. These possible mechanisms will be studied in more detail during the last year of the project.

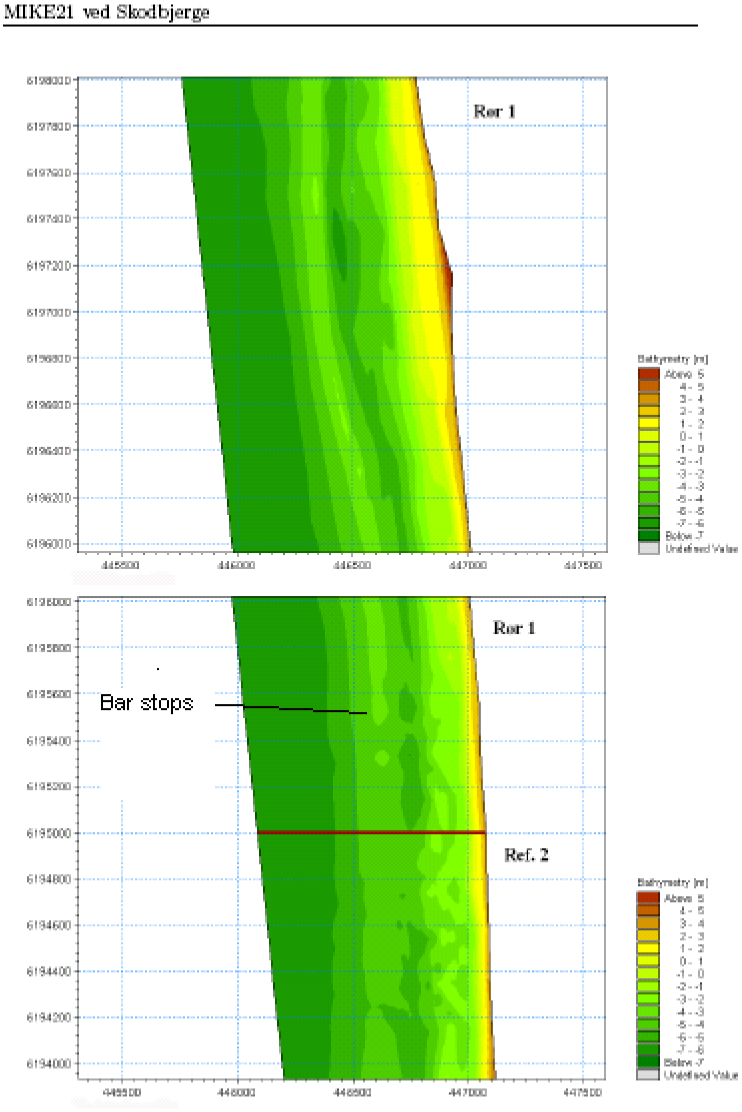


Figure 7.21: The bathymetry indicates that an outer bar disappear just in the transition in between Rør 1 and Ref 2. A more detailed survey will be performed this summer (2007).

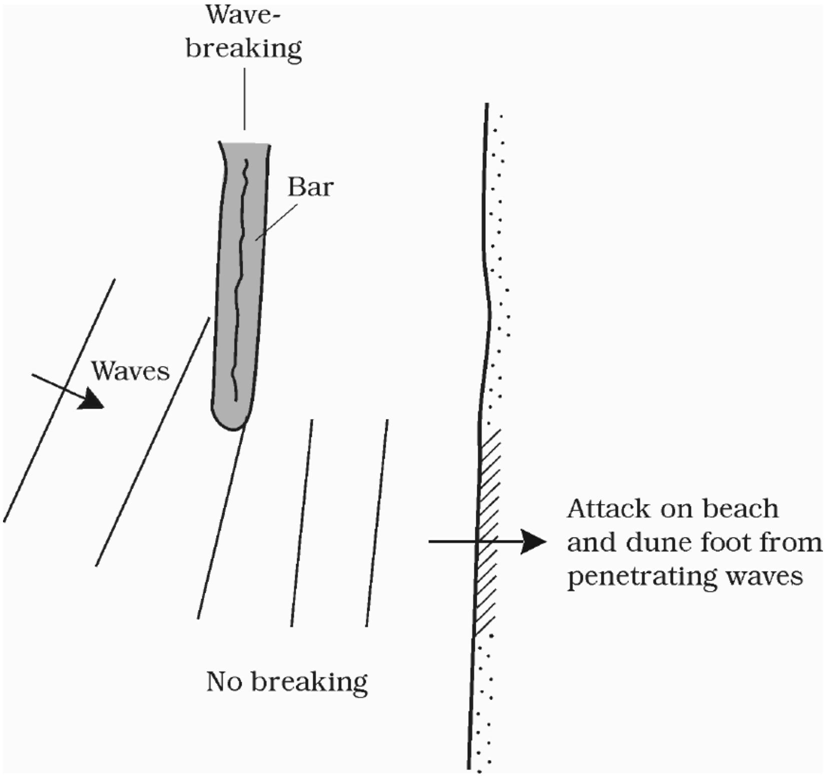


Figure 7.22: If the bar really disappears, the beach will be exposed to a larger wave attack.

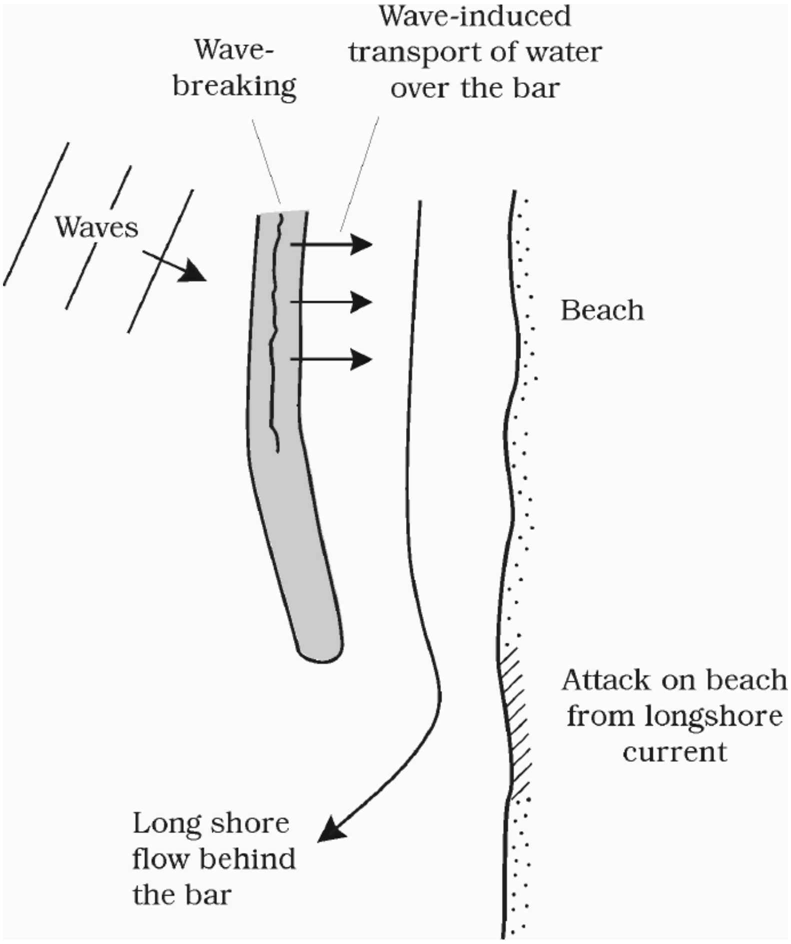


Figure 7.23: Another possible mechanism for a local narrowing beach is a concentration of the long shore current behind a crescendic long shore bar (originally suggested by Søren Knudsen, KDI).

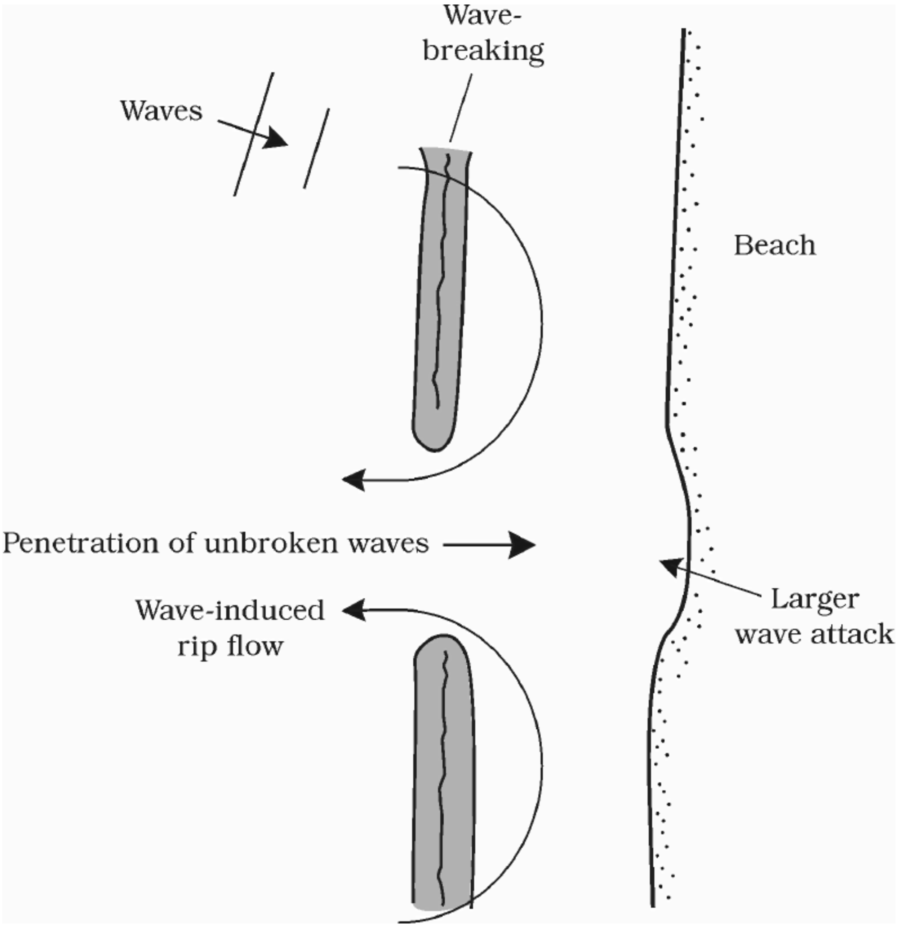


Figure 7.24: Also rip holes allow waves to attack the beach locally.

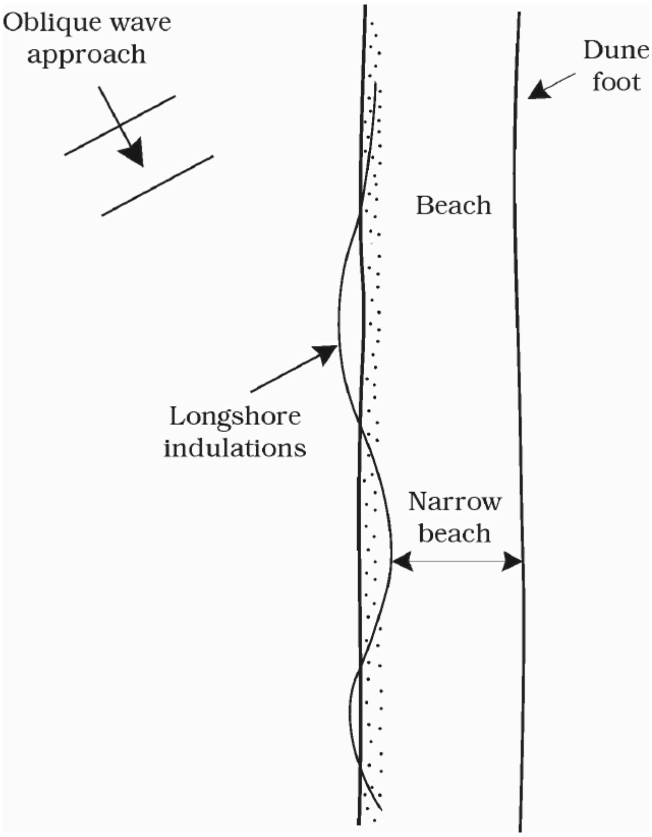


Figure 7.25: Obliquely approaching waves will form long shore undulations as described in the appendix 5.

3. The erosion in the beach is not only significant in Ref2 but also in several places where the tubes are located, see the sketch figure 2 and next section
4. There is no sign of erosion in Ref 3, on the contrary it grows and grows. SIC claims this is due to “washed sand”, but then you can ask why is the sand not washed in ref 2, located down drift of more than 5 km tubes!!

8. List of appendices.

1: Comments on the infiltration into the beach by Peter Nielsen (taken from his homepage).
2: The drainage capacity of a tube in homogeneous sand and exposed to a vertical pressure gradient (by Jørgen Fredsøe)
3: A field study at the site on the flow in the beach (By Peter Engesgaard, KU).
4: D-profiles
5: Undulations along the shore (by Jørgen Fredsøe)

Trouble with magical pipes

INTRODUCTION

A system of vertical pipes connecting the beach groundwater with the atmosphere and marketed by Skagen Innovation Center, e g, Jakobsen (2000) has recently been causing widespread concern among coastal authorities. Through mainstream Danish newspapers, e g, Jyllandsposten October 31, 2000, and articles presented at coastal management conferences, Jakobsen (2000) the system has been successfully marketed to the public and to some coastal managers. Most specialists on beach groundwater dynamics and sediment transport agree however that the system has no physical basis. The following is an attempt to make this clear to a wider group, hopefully including all coastal managers, by drawing on the available physical evidence.

THE (FLAWED) IDEA BEHIND THE SYSTEM

Jakobsen's idea is that the pipes will make the watertable drop and therefore enhance infiltration and sediment deposition. (The pipes are about 2m long with a diameter of the order 7cm, they are perforated near the bottom and allow air to enter at the top. An example is shown in Figure 3 below). The problem with this is, that there is no reason these pipes should make the watertable drop. Jakobsen seems to think that the beach groundwater hangs on a vacuum like the water in the inverted bowl in Figure 1

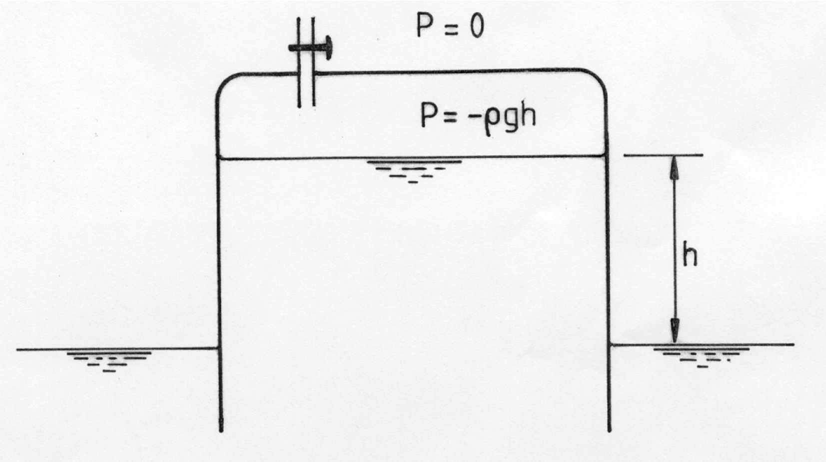


Figure 1: An inverted bowl is able to hold the water at a higher level due to the vacuum ($p= -\rho gh$) inside. If the valve is opened, the waterlevel will drop.

If that was the case, his pipes would have same effect as opening the valve. The beach watertable is however not under a vacuum and there is no reason that the pipes should affect the watertable. Certainly not on the scale of tens of metres which is the usual recommended distance of installation.

EFFECTS OF DRAINAGE ON BEACH EROSION/DEPOSITION

There is evidence that lowering of the watertable in beaches can have some beneficial effects, mainly enhanced accretion in fair weather rather than erosion protection during storms, see Turner & Leatherman (1997). There is also some data, which quantify the underlying effect of infiltration rate on sediment mobility, Nielsen et al (2001). However, the evidence is that a very strong drainage effect is needed in order to give a significant effect while there is no indication, theoretical or experimental that Jakobsen's pipes have any drainage effect.

Davis et al (1992) tested a drainage based system which had a clear and visible effect on the beach watertable but no significant effect on beach erosion/accretion. The drains used by Davis et al were shore normal line drains and their effect on the watertable is indicated by the bend in the borderline between saturated (glassy looking) and drained (matt looking) sand surface in Figure 2.

Figure 2: The seaward peaks in the borderline between drained and saturated surface sand are caused by two of the line drains used by Davis et al (1992) at Dee Why Beach, Sydney.

In contrast, Figure 3 shows one of Jakobsen's pipes installed at Gammel Skagen on the Danish North Sea Coast. There is no indication of a lowered watertable around the pipe.

Figure 3: A "Jakobsen pipe" installed at Gammel Skagen on the Danish north sea Coast.

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Jakobsen, P (2000): Pressure equalization modules for environmentally friendly coastal protection. Proc 10th New South Wales Coastal Management Conference, Yamba, Collated by McLean Shire Council.

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Turner, I L & S P Leatherman (1997): Beach dewatering as a soft engineering solution to coastal erosion: A history and critical review. J Coastal Res, Vol 13, No 4, pp 1050-1063.

Appendix 2.

The functioning of the PEM-tube.

It has been discussed very much – and the discussion is still going on – how the functioning of the tubes are.
The main idea is, that the water table will decrease faster together with the falling water level in the sea in relation to tide and storm surge.
This effect is due to vertical drainage by the tubes.

Let us consider figure 1, which shows the groundwater flow in the sand during falling water level of the sea.
If there is no freshwater supply from land, the flow pattern in the sand is like that sketched in the figure.
Lets consider the pressure conditions at tube I and II:

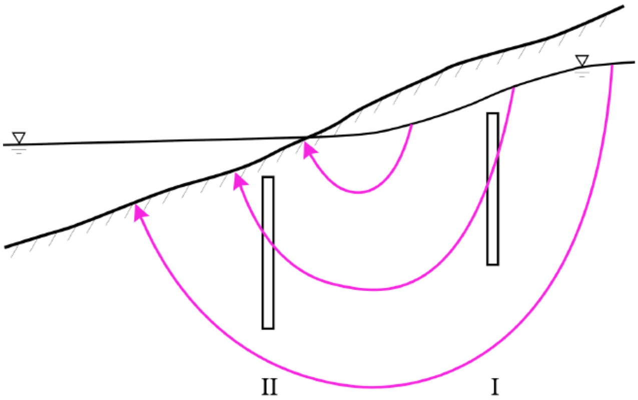


Figure 1: Ground water flow pattern in the beach during falling sea level.

At I, the flow is directed down, and it is easier to flow through the tube than outside in the surrounding soil: in the tube there are nearly no flow resistance, and with small flow velocities, the pressure within the tube can be taken to be hydrostatic.
In the soil you need an excess pressure gradient (in this case negative) to force the flow through the soil, where there is a considerable flow resistance (the Darcy law).

This is illustrated by the schematic pressure distribution in figure 2b. The continuity equation for the tube requires (in a quasi-steady flow) that the flow into the tube equals the flow out. This requirement determines the water level within the tube relative to the water level just outside in the soil. This difference is called Δz (see figure 2a). In the upper part of the tube (from z_0 to $z_1 + \Delta z$) the water pressure in the soil is larger than the pressure in the tube. This will cause a flow into the tube. In the lower part of the tube, the things are opposite: here the pressure is largest within the tube, and there will be a flow form the tube to the soil.

This shortcut through the tube of the near-tube flow will increase the vertical drainage.

The question is how much.

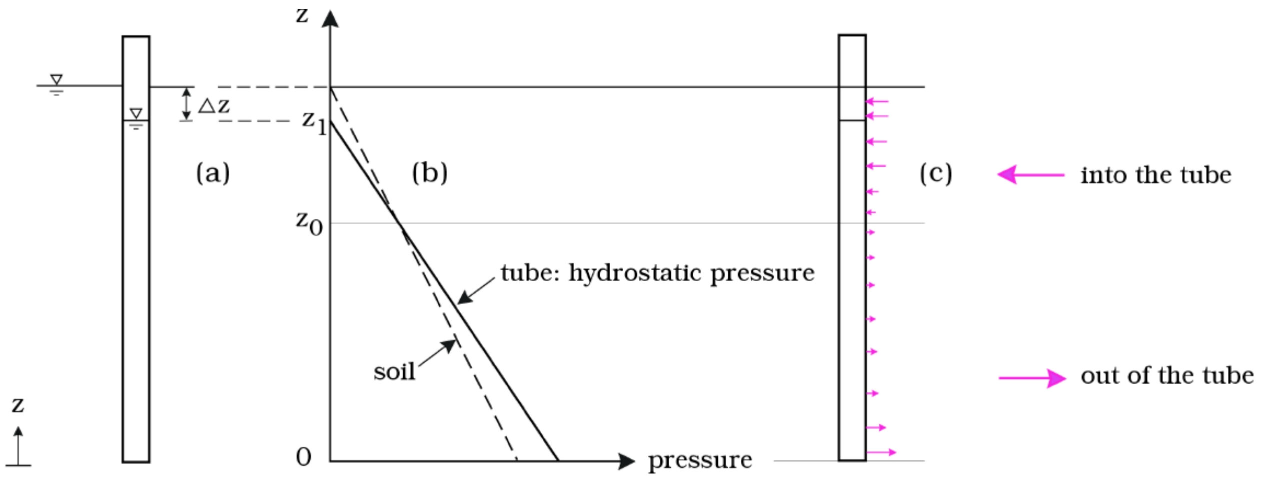


Figure 2.: Pressure distribution along a tube, and the resulting flow pattern to and from the tube located at position I (figure 1) during falling sea level.

Let us consider a well-sorted beach without any kinds of stratification in the sand or water (salt water – fresh water).
Let the permeability coefficient be k=0.005 m/sec (corresponding to 1mm sand).
Without the tubes a typical lowering-velocity of the water table in the beach will be

V= 1m/(3600sec/hour)/6hours)

Or

V~5E(-5) m/sec =0.05 mm/sec.

This corresponds to a lowering of the water table in the beach equal 1 meter in 6 hours.

The hydraulic gradient i to cause this flow is given by

i=V/k = 0.01

Over 2m (the length of the tubes – this is actually exaggerated since there are only slots in the tubes in the lower 1 meter of the tubes) this corresponds to 2 cm loss in energy head =Δz

The next question is how much water will flow through the tube if you have Δz=2cm.

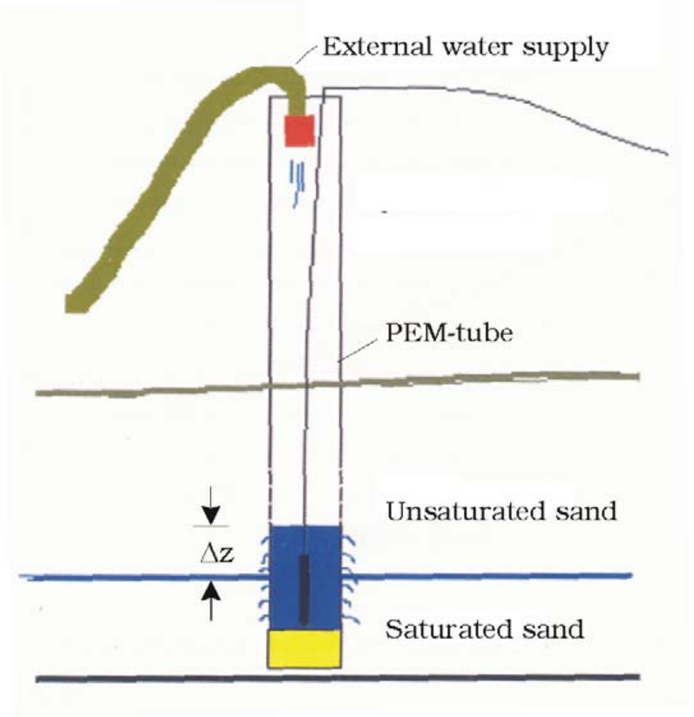
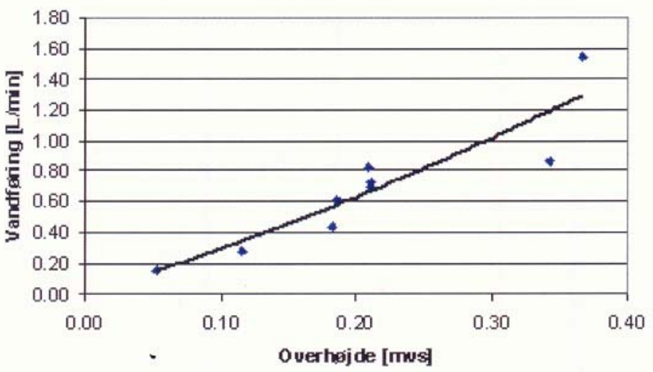


Figure 3: Set-up to determine the flow through the tube. The sand size is about 0.4 mm

For this we did a simple experiment in DTU, where we put the tube into sand as shown in figure 3, and looked at the flow through the tube. With an over height Δz=



Figur 5.4: Grafen angiver sammenhængen mellem vandføringen gennem trykudligningsmodulets filter og overhøjden i filteret.

20 cm, the flow is around 0.6 l/minute (see figure 4), so for smaller heads like Δz= 2 cm, the flow rate is around 0.06 l/minute. This corresponds to a flow velocity of

Figure 4 Relation in between Δz and flow discharge through the tube (diameter 6 cm)

V (tube)= 0.00006cbm/minute/(pi*0.03*0.03)

Or

V (tube)= 0.35 mm /sec.

The flow velocity within the tube is with other words around 7 times higher than outside the tube.

The drained area around the tube is approximately a circle with a radius of 5 meter (since the mutual distance in between the tubes is 10 m), so the area to be drained is

A (drained)= 80 sqm.

The area of the tube is

A (tube)=0.0028 sqm= 3.5 E (-5) A (drained) (0.03 per thousand)

So even with a higher flow velocity in the tube (a factor 7), the impact on drainage will only be 7*0.03 per thousand= 0.21 per thousand increased drainage capacity.

In the table below, the impact of different sand sizes in the beach for the drainage capacity of a tube is given. Lundgren /1/ suggests k to depend on d (10) (10% of the sediment is finer than this size, d given in mm) in the following way:

k=0.0125 d (10)**2

and this expression has been used in the table, all other parameters being the same as used above. For the flow through the tube, figure 4 is applied. Actually, when changing the sediment size, this experiment should be repeated with the corresponding sand size. In this case, the flow through the tube would be smaller for fine sediment, and larger for the coarse. Hence the drainage improvement would be smaller for the fine sand and larger for the coarse.

d (10) in mm	k in (m/s)	Hydraulic gradient i	Δz in m	V (tube) in mm/s	Improved drainage in promille
0.05	3.75E(-5)	1.33	2.66	6.5	0.65
0.1	1.5E(-5)	0.33	0.66	1.6	0.16
0.2	6E(-5)	0.083	0.17	0.41	0.04
0.4	4.68E(-4)	0.0208	0.042	0.103	0.01
0.8	1.17E(-4)	0.0052	0.0104	0.0256	0.0025

It is seen that the improved drainage of an area around each tube is only improved with less than 1 per thousand, even for a beach with a lot of fines. (Please note that Δz in case of fine sand becomes larger than the length of the tubes, which of course is not possible)

Let us finally return to figure 1 and consider the tube II, which is located out in the water, where the flow is directed upwards. In this case the arguments put forward above are exactly the same, and the flow directed upwards outside the tubes will be reduced only with less than 1 per thousand, or much less than required to get any kind of stabilizing effects on the sediment grains moving on the seabed. (This would correspond to a change in tidal range from 1 meter to 1.001 meter)

/1/Lundgren and Brinch Hansen: Geoteknik, Teknisk Forlag, Copenhagen 1965.

Effect of Vertical Drains on Tidal Dynamics in Beaches

Peter Engesgaard
Geological Institute
University of Copenhagen

FINAL REPORT - JUNE 2006

1 Introduction

A low water table in beaches will generally favour infiltration and onshore sediment transport [Horn, 2006]. The location of the water table in beaches is primarily controlled by tidal dynamics. Controlled laboratory experiments have recently demonstrated how a single harmonic tide can generate tidal responses with higher harmonics due to different physical phenomena [Cartwright et al., 2003, 2004]. These may include the non-linear filtering effect of a sloping beach, which also leads to a water table over height [Nielsen, 1990], the effects of the development of a seepage face, and the effects of the presence of a (truncated) capillary fringe near the beach surface. Observations in the field by Raubenheimer et al. [1999] confirm these findings.

The effects of so-called vertical drains on the tidal response in beaches are investigated in this report. The drains are also called Pressure Equilibrium Modules (PEM). The vertical drains consist of a 10 cm drain with a 1 m long screen. The functioning of the PEMs is not known, but one hypothesis is that the effective permeability of the beach is increased. A two-week experiment was conducted at a beach near Holmsland on the west coast of Denmark in order to investigate the hydraulic functioning of the PEMs. Two different experiments were envisaged. A beach-scale experiment where tidal dynamics were monitored in transects with normal observation wells and PEMs, and PEM-scale experiments, where the pressure distribution around a drain was continuously monitored. Unfortunately it was only the beach-scale experiment that was successful.

The experiment was divided into two periods. Period 1 where only 10 cm diameter wells (10 cm screen) were installed with pressure transducers (divers; measurement every 2 minutes) and period 2 where both wells and PEMs were installed, the PEMs also with pressure transducers. Three transects were established. One transect with just wells and no PEMs, which then acted as a reference site, one transect with both wells and PEMs, and then one transect with a few wells and mostly PEMS, which was designed primarily for the PEM-scale experiment. This makes a before-and-after comparison possible, where the tidal response in the wells during period 2 can be compared with the tidal response in period 1 and finally can be compared with the reference site.

The analysis of the data is partly based on the model by Nielsen [1990] and partly on the approach used by Carr [1971]. The model by Nielsen [1990] shows that small increases in the effective permeability of the beach will lead to less reduction in the amplitude of the recorded tidal signal in the wells plus less water table over height (and, thus, lowering of the water table). Carr [1971] used harmonic analysis to interpret the amplitude damping as a function of distance from the sea.

2 Field site

The field site is located near Holmsland on the West coast of Denmark, Figure 1.

Figure 1: Location of field site

Figure 2 shows the location of the installed wells all with divers measuring the hydraulic head and the Pressure Equilibrium Modules (PEMs) also with divers. The North transect acts a reference site, where no PEMs were installed. The Central transect includes wells spaced about 10 m apart, and with PEMs centrally located in-between (i.e., 5 m spacing to wells). The South transect has only four wells, three nearest to the sea, and one at the other end. Otherwise, this transect mainly consist of PEMs.

All wells were installed starting on 8:00, March 20, 2006. This corresponds to Julian day 79.3. The PEMs were installed on March 26, approximately on Julian day 85.8. The experiment ended on April 2, approximately Julian day 92.7.

The experiment is therefore divided into two experimental periods; Period 1 Julian days 79.3-85.5 (6.5 days), where only wells were installed, and Period 2, Julian days 85.5-

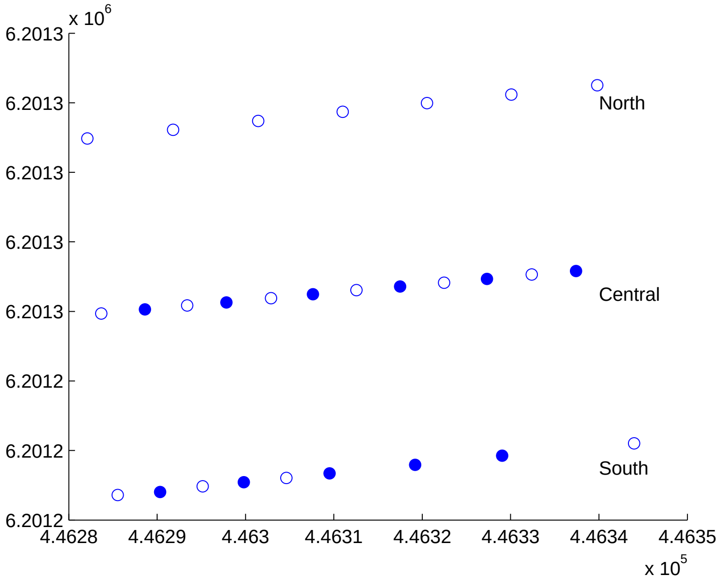


Figure 2: Location of wells with divers (open circle) and pressure equilibrium modules (filled circles)

92.7 (approximately 7.2 days). For reasons discussed below both of these periods will be made shorter.

Figure 3 shows the measured changes in the beach profile (measured on three occasion at every well and PEM). From March 20 to March 26 (i.e., the period without PEMs) there is a change in the beach profile by the addition of sediments to the zone affected by tidal dynamics and waves, which generally causes a decrease in the average slope (inverse of cotβ, where β is the beach slope, calculated as the distance between end points divided by difference in elevation of the beach at the two end points). The exception is the North transect, where a slight decrease in the elevation of the beach profile at the well nearest to the coast line causes an increase in slope. On the other hand, from March 26 to April 2, there is a decrease in the elevation of the beach profile nearest to the coast line, even below that measured on March 20, causing an increase in slope, Table 1. This change likely happened after March 28-29, during which there was an increase in wave and current activity. Notice the possibility of tidal water being trapped in depressions primarily in the Central and South transects in the latter period.

Figure 4 shows the recorded water level at Hvide Sande. By a coincidence the mean water level can be divided into two periods that more or less exactly matches the two

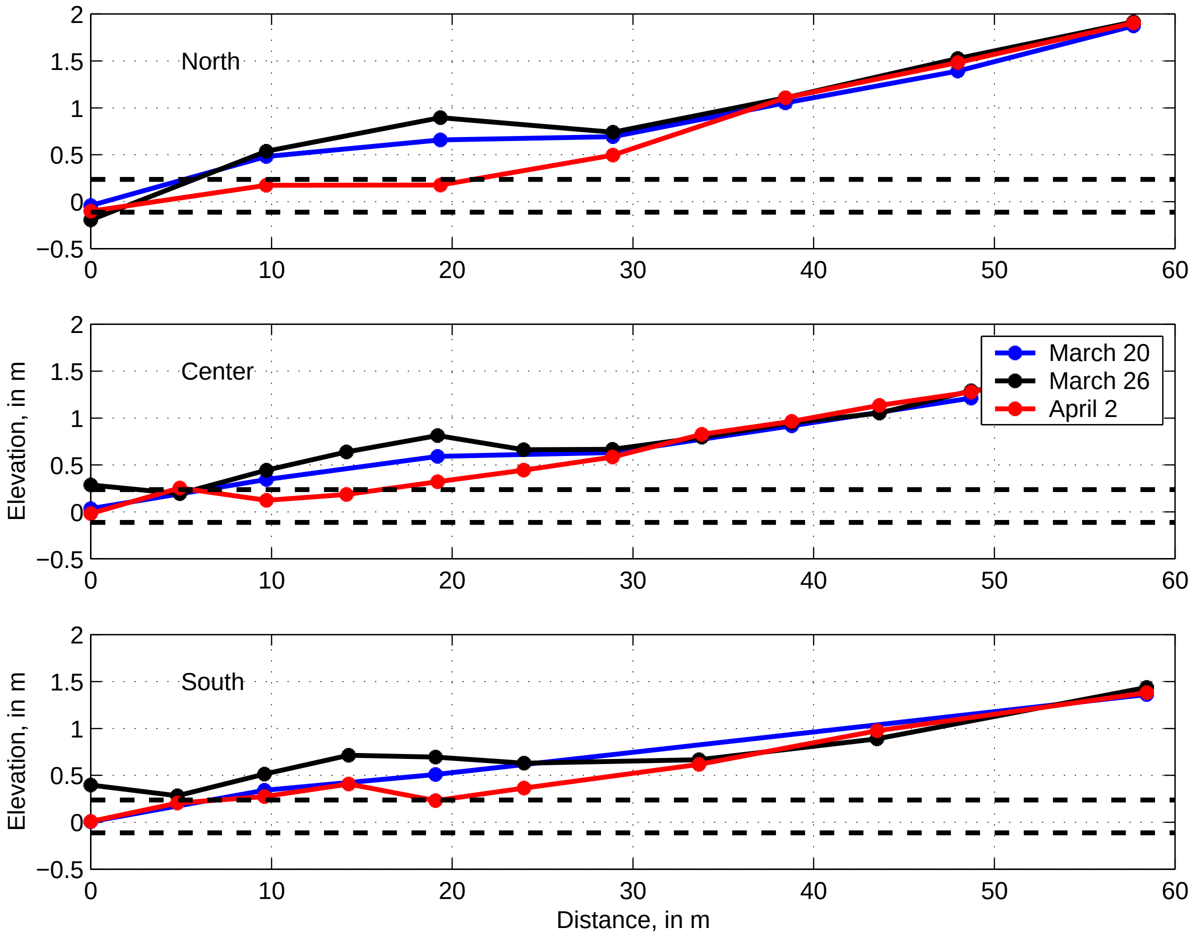


Figure 3: Measured beach profiles on March 20 (blue), March 26 (dark), and April 2 (red) for transects North (top), Central (middle), and South (bottom). The two dashed lines are the MSLs, with the lower and upper lines representing the MSL during periods 1 and 2, respectively.

Transect	Average beach slope		
	March 20	March 26	April 2
North	30.2	27.4	28.8
Central	41.4	48.4	36.2
South	43.2	56.2	42.6

Table 1: Average beach slopes (cotβ) on March 20, March 26, and April 2.

experimental periods. In period 1, the mean sea water level (MSL) is -0.11 m, while in period 2, the MSL is 0.24 m. This will have an effect on the water table dynamics in the beach. The MSLs are shown on Figure 3. The beach profile measured on April 2 is probably representative for period 2 because the small storm started on March 28. Thus, the MSL moved at least 20 m further inland. The mean amplitude of the water levels at Hvide Sande up to day 83 is 0.36 m. After day 86 and to the end the mean amplitude is 0.49 m.

The hydrogeology of the site is not very well known. The beach mainly consists of sand with embedded gravel layers sometimes up to 0.5-1.0 m in thickness. Grain size analysis shows a d_{10} of about 0.2-0.4 mm. Hazens empirical relation for calculating a hydraulic conductivity (K) is;

$$K = Ad_{10}^2 \tag{1}$$

where A=1 if d_{10} is inserted in mm giving K in units of cm/s. Using (1) one can compute K in the range 30-140 m/day.

The other parameter of interest is the drainable or effective porosity, n. For this type of (coarse) sand the drainable porosity is probably close to the total porosity, i.e., 0.2-0.4. However, the effective specific yield (often assumed equal to the drainable porosity) may be much lower due to the presence of the water table near the surface, and, thus, also the capillary fringe, which may become truncated during high tide [Gilham, 1984]. The effective drainable porosity may therefore be less.

Rainfall amounted to about 39 mm over the whole period with the highest rainfall rate of about 14 mm in one day (March 27). At this time of the year recharge is approximately equal to rainfall.

The reported hydraulic heads are based on measurements relative a measuring point found by GPS survey levelling of the wells and PEMS. The precision is about a few cm's

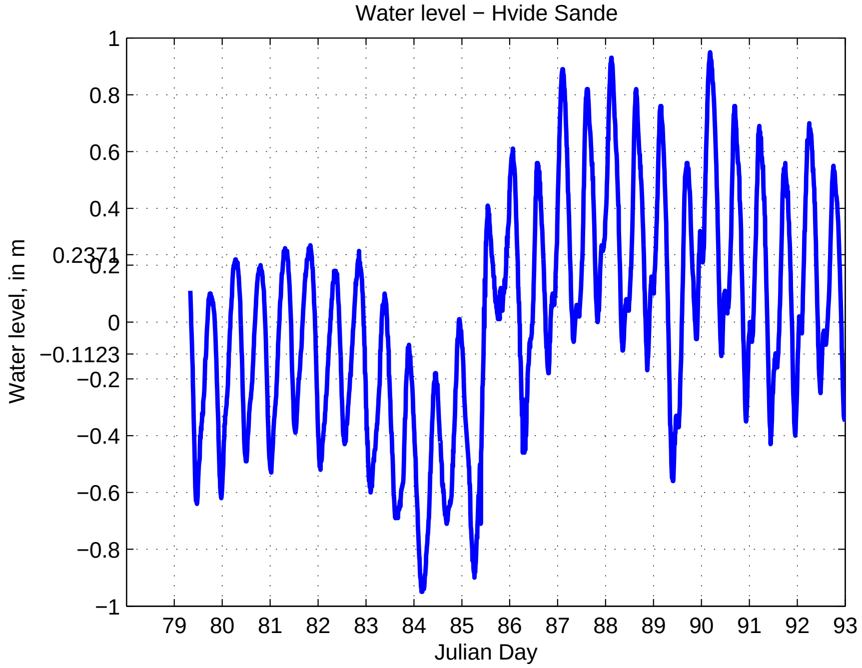


Figure 4: Water level at Hvide Sande

(J. Gregersen, personal communication).

3 Conceptual Model

Nielsen [1990] presented an analytical solution for hydraulic head fluctuations in beaches due to tides. Figure 5 shows a schematic of the considered flow system. The origin of the x axis starts at the intersection of the mean sea level (MSL) and the beach face, and x is positive landward.

The assumptions are;

- A low-permeable layer exist at the bottom of the aquifer. The thickness of the aquifer is D, equal to the distance from the mean sea water level to the bottom.
- The aquifer is homogeneous with an effective hydraulic conductivity, K, and drainable porosity, n.
- The beach has a slope with an angle of β.

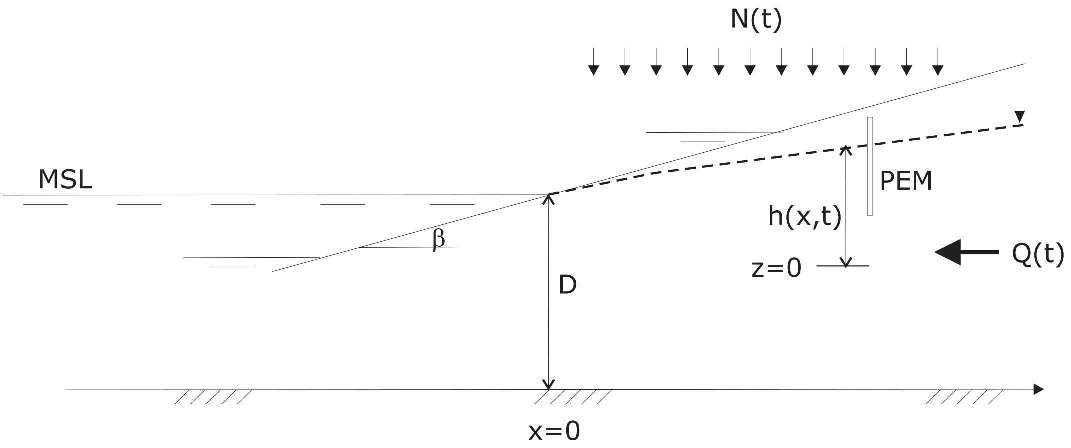


Figure 5: Conceptual model of beach

- Flow is horizontal(Dupuit).
- Single sinusoidal tide with period T.
- Groundwater flow into the coastal aquifer Q(t) is zero.
- Recharge N(t) is zero.
- A seepage face does not develop.
- Capillary effects on water table movement can be neglected.

As mentioned above, the coastal aquifer at Holmsland is likely not homogeneous. However, the analysis will be based on a before-and-after situation, where the PEMs in period 2 may lead to a higher effective permeability because they can lead to an increase in the connectivity between the gravel layers that are known to exist at different elevations.

There is only one harmonics, i.e., the model can not treat low-frequency tides and high-frequency waves at the same time. In the forthcoming analysis the effects of the waves have been filtered out.

Flow is not strictly horizontal at the field site. Raubenheimer et al. [1999] observed that horizontal flow tended to dominate vertical flow, although significant vertical flow did occur during high tide. Likewise Cartwright et al. [2004] found non-hydrostatic pressure distributions in their sand box experiments.

Recharge was not equal to zero during the experimental period. By assuming a drainable porosity of 0.20, then the maximum increase in water table (by neglecting any outflow) from a daily recharge of 14 mm is 0.014/0.2 or 0.07 m. During most of the days the rate of recharge is less than 5 mm, i.e., an increase in water table of about 0.025 m. However, the effect of the capillary fringe extending all the way to the beach surface, at least during high tide, would mean that the effective specific yield is much less than the drainable porosity [Gilham, 1984]. A few mm of rainfall could therefore easily lead to a higher increase in water table. However, there has been no analysis of when rainfall occurred relative to the tide. For example, if rainfall occurs during high tide then it has much less effect.

One of the most critical assumptions is the that related to the formation of a seepage face. A seepage face occurs because of a decoupling between fx. the falling tide and the water table. The seepage face will form in the active tidal region. The analytical solution given below is therefore only strictly valid upstream to the high water mark.

Capillary effects may play a role, but it is generally accepted that this is most crucial for high-frequency signals (i.e., waves).

Despite these simplifying assumptions the analytical model by Nielsen [1990] may still give some valuable insight into which physical phenomena to look for when comparing the tidal response in the beach before and after the PEMs were installed.

The one-dimensional analytical solution is given as;

$$h(x,t) = D + Acos(\omega t - kx)e^{-kx} + \varepsilon A \left[\frac{1}{2} + \frac{\sqrt{2}}{2} cos(2\omega t + \frac{\pi}{4} - \sqrt{2}kx) e^{-\sqrt{2}kx} \right] + O(\varepsilon^2) \quad (2)$$

where h(x,t) is the hydraulic head (m) at position x (m) and time t (days), D is the mean aquifer depth (m), A is the tidal amplitude, $\omega = 2\pi/\mathrm{T}$ is the tidal frequency, where T is the tidal period (days), k is the wave number (see below), and $\varepsilon = \mathrm{kAcot}\beta$, where β is the beach slope. The analytical solution was developed from a pertubation analysis using ε as the pertubation parameter. Equation (2) is correct to first order in ε. Thus, ε must be much lower than 1 for (2) to be valid ($\varepsilon << 1$, in practise it often suffice that $\varepsilon < 0.5$). Also, it is required that the amplitude is small compared to the mean aquifer depth, i.e., A $<< D$. Nielsen [1990] also developed a solution that is correct to second order, however, here it will suffice to use (2) to demonstrate the effects of tides on the hydraulic head fluctuations in a sloping beach.

The wave number, k, is defined as;

Parameter	Value
Hydraulic conductivity, K	50, 200 m/day
Porosity, n	0.2
Amplitude, A	0.4 m
Aquifer thickness, D	20 m
Tidal frequency, ω	$2\pi/0.5$ day^{-1}
Beach slope, cotβ	0, 60/1.5

Table 2: Parameters used to simulate tidal dynamics with Nielsen model. Two values for K and the beach slope are used

$$k = \sqrt{\frac{n\omega}{2KD}} \tag{3}$$

where K is the hydraulic conductivity (m/day) and n is the drainable porosity (-). These are the two hydraulic parameters that govern the effects of tidal dynamics on hydraulic head fluctuations. The ration K/n is also called the aquifer diffusivity. The higher the diffusivity the lower is the time scale for transmitting the tidal signal.

The first term in (2) is the mean aquifer depth corresponding to the mean sea water level. For the case of a vertical beach (cotβ=0) one has ε=0 and the third term cancels out. Thus, the solution represents a pure sinusoidal fluctuation around D, but with a damped signal (Ae^{-kx}) and a phase lag (cos(ωt-kx)). Figure 6 shows two simulations (black solid and dashed lines) with the parameters in Table 2 (the parameters are close to those representing the field site). Notice that it is h(x,t)-D that is plotted versus time. Both simulations give tidal fluctuations around zero. The phase lag and damping increases with an increase in the wave number corresponding to a decrease in K or increase in n. The case with the high K (200 m/day) thus gives tidal fluctuations that are much higher than the case with the low K (50 m/day). For the high K case, the damping is about 0.23/A=0.23/0.4 = 0.58 (0.23 m is the peak value). Likewise, for the low K case, the damping is about 0.18. These reductions are also called the tidal efficiency [Carr, 1971].

The third term in (2) accounts for (i) an extra over height and (ii) an extra, but small damping plus a skewing (asymmetry) of the tidal signal εA$2^{1/2}$/2(cos2ωt). The over height means that the water table is lifted on the mean a factor of 0.5εA above the mean sea level, which is explained by the fact that it is easier for water to seep into a sloping beach at high tide than to drain away at low tide [Nielsen, 1990]. This is also seen in the two simulations in Figure 6 where a sloping beach is introduced (cotβ=60/1.5, red solid and dashed lines). Again the high K case means less damping of the tidal signal, however the mean water level is lifted 0.058 m above the mean sea level. The results are shown at a distance of 50 m from the intersection between the MSL and the beach, i.e., in this case upstream to the high water mark. In the low K case, the water level is lifted 0.11 m. Also, the asymmetry is lower in the high K case.

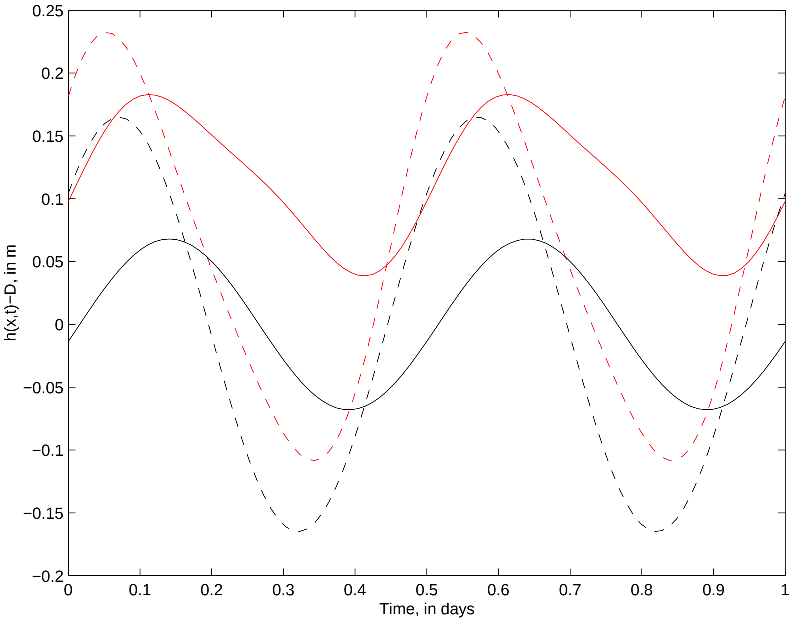


Figure 6: Tidal dynamics at x=50 m for 4 different situations. Black indicates a vertical beach (cotβ=0) and red a sloping beach (cotβ=60/1.5). Dashed lines are with K=200 m/day. Solid lines with K=50 m day.

This leads to the following observations;

- A higher hydraulic conductivity leads to less damping of the tidal signal (and also less phase lag, however this is more difficult to observe)
- A higher hydraulic conductivity leads to a decrease in the so-called water level overheight.
- A higher hydraulic conductivity leads to less asymmetric tidal signals.

4 Presentation of tidal data

Figures 7 and 8 are examples of the recorded tidal signal in wells N1 and N7, respectively. Clearly the signal is composed of low-frequency tide signals and high-frequency waves. At well N7 the high-frequent signals have almost disappeared.

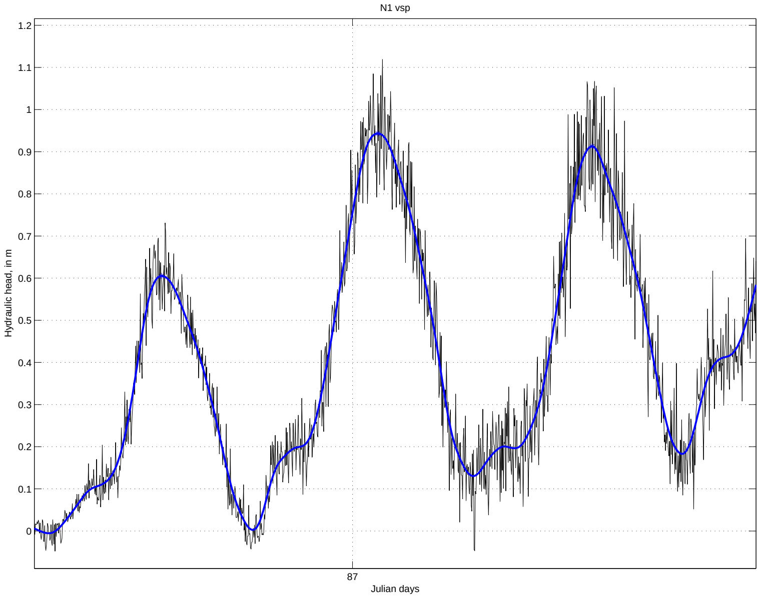


Figure 7: Illustration of the filtering of high-frequency waves in N1. Black line is the recorded signal (every 2 mins). Blue line is the filtered signal.

To make the interpretation easier all recorded signals were filtered using a so-called low-pass band filtering technique, see Appendix A. The results of the filtering are also shown in Figures 7 and 8. The analysis was therefore done exclusively on the filtered signal.

Figure 9 shows the filtered signal in C6. It it clear that during the transition from period 1 to period 2 it is very difficult to pick out low and high tides. This is mainly due to the nature of the sea water level, Figure 4, and the non-linear filtering of the signal due to the beach. The same observation is valid for all wells, except perhaps the wells closest to the sea. The periods of observations have the therefore been changed in order to omit this transition period. Thus period 1 ends at day 83 and period 2 starts at day 86. The amount of rainfall in the new periods 1 and 2 are about 3 and 13 mm, respectively. Thus,

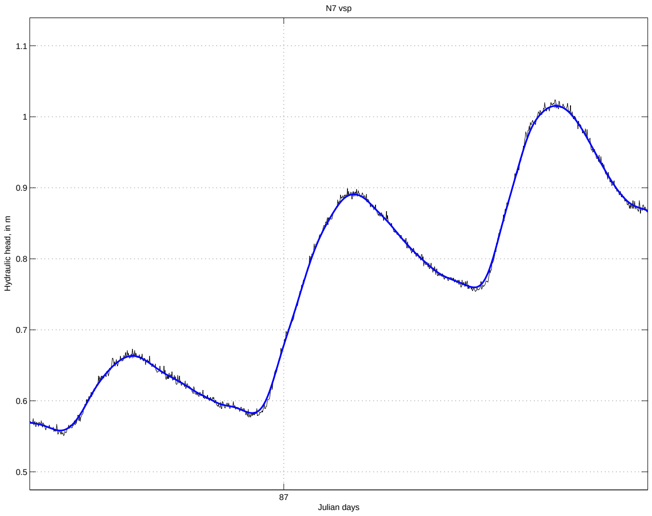


Figure 8: Illustration of the filtering of high-frequency waves in N7. Black line is the recorded signal (every 2 mins). Blue line is the filtered signal.

excluding days 83-86 takes care of the problem with high rates of rainfall with up to 20 mm over 3 days.

The mean hydraulic head in C6 increases from 0.44 m in period 1 to 0.70 m in period 2, which reflects the general increase in mean sea level (0.35 m).

5 Method of analysis

The analysis of the data is performed in the following way;

1. Analysis based on wells only
2. Analysis based on wells and PEMs

5.1 Analysis based on wells

The method of analysis is based on;

- Calculating the amplitude reductions in period 1 and 2 to see if the beach has changed hydraulically by the installation of the PEMs. The PEMs could cause

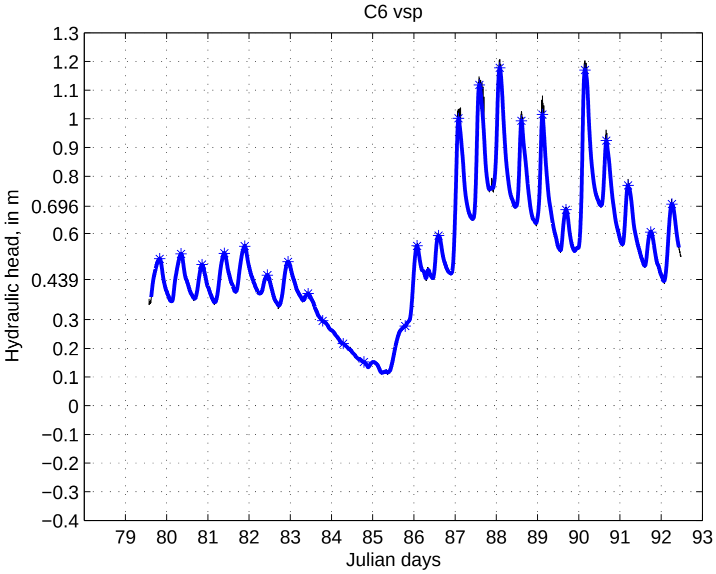


Figure 9: Recorded signal in Well C6. The mean hydraulic head in periods 1 and 2 are, 0.44 and 0.70 m, respectively.

an increase in permeability leading to greater fluctuations during period 2. This approach is similar to that performed by Carr [1971], except that a harmonic analysis is not performed here.

- Calculating the mean water level. The PEMS could cause an increase in permeability leading to less overheight.

The PEMs have not been included in the analysis, i.e., only the wells are included to see how each reacts before and after the installation of the PEMs.

Figure 10 shows the adopted method. For each well the total amplitude has been recorded for each tide. In all there is about 25 low-high tides during the whole period. Because of the exclusion of days 83-86, it amounts to 6 total amplitudes during period 1 and 13 total amplitudes during period 2. Each amplitude is correlated to the same total amplitude in the water levels measured at Hvide Sande, Figure 4. For example, one can have;

$$a_i^r = \frac{a_i^{C6}}{a_i^{HS}} \tag{4}$$

where a_i^r, a_i^{C6}, and a_i^{HS} are the relative amplitude reduction, the total amplitude at C6, and the total amplitude at Hvide Sande for the i'th tide. The amplitude reduction, a_i^r, is also known as the tidal efficiency [Carr, 1971]. Carr [1971] used harmonic analysis to find the tidal efficiency of three primary tidal components. Cartwright et al. [2003, 2004] similarly used harmonic analysis to find both the amplitudes and phase lags of the single tidal component and the higher order harmonics generated e.g. by the sloping beach and the formation of a seepage face.

The mean amplitude reduction and its standard deviation are calculated for both periods 1 and 2. Recall that only 6 and 13 amplitudes are available, so the standard deviation is uncertain especially for period 1. Furthermore, the mean hydraulic head is calculated for period 1 and 2 in each well.

This procedure assumes that the well response time is short [Black and Kipp, 1977; Horn, 2006], i.e., that the observation well responds more or less instantaneously to changes in pressure outside the well.

All the calculations were done semi-automatically using MATLAB, see also Appendix B.

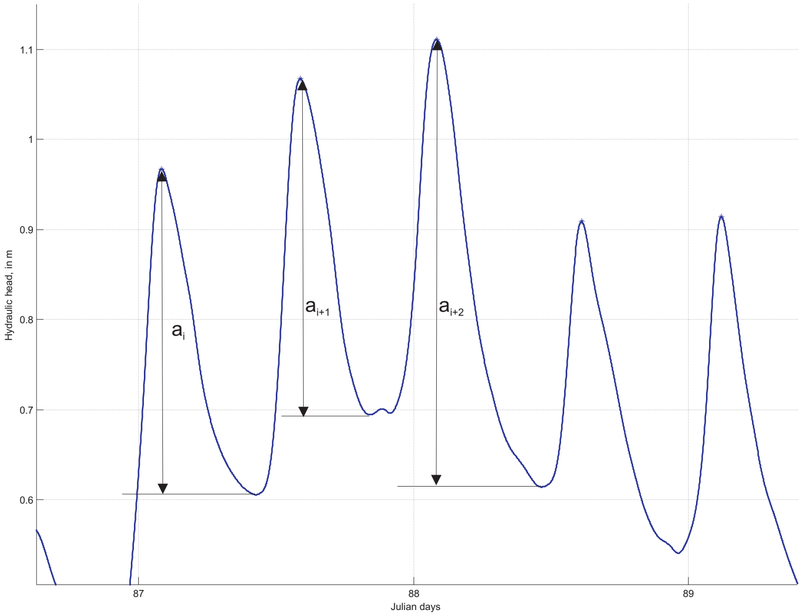


Figure 10: Peak analysis method. Every total amplitude is recorded.

5.2 Analysis based on wells and PEMs

The hydraulic heads were measured with pressure transducers placed near the bottom of the screens of the wells and PEMs, see for example Figure 16. The screen is 1 m long in a PEM and 0.1 m long in a well. The projection of the location of the measurement point of the transducers therefore approximately follows the beach slope.

There are essentially two possibilities for interpreting the tidal response observed in the PEMS;

1. The PEMs act as observation wells with a large diameter and a (relative) long screen.
2. The PEMs act as a drain with water flowing up or down.

Unfortunately it is only possible to investigate the first situation, where the PEMs act as an observation well. Another experiment was designed to closely monitor the head distribution around two PEMs in order to observe significant in/out flows to or from the PEMs. However, this experiment failed. If such a situation is true then inertial effects can become important as has been observed in hydraulic tests of wells. One can not necessarily out rule the possibility of the PEMs draining water from waves in the swash zone, where an analogy to instantaneous hydraulic tests may be made.

The premise for considering the PEMs (and the wells) as observation wells is that the pressure distribution in the well bore is hydrostatic. This means that the hydraulic head inside the well bore represents an average head over the length of the screen. Significant vertical upward or downward flow may exist in the aquifer itself [Cartwright et al., 2003, 2004] although horizontal flow have been shown to dominate at the field scale [Raubenheimer et al., 1999]. The point of measurement is then assumed to be in the middle of the screen, which means that the PEMs measure the hydraulic head about 0.5 m above the wells.

The tidal data from the wells and PEMs have been analyzed to detect possible vertical flows.

6 Results

6.1 Analysis without PEMs

The following analysis is carried out without considering the PEMs. It focuses on a before-and-after situation and a comparison with the reference North transect.

Figures 11, 12, and 13 show the mean amplitude reduction as a function of distance from the first well (N1, C1, or S1). The bars show the plus/minus one standard deviation.

Transect North behaves almost similar from period 1 to 2, although there is a slight tendency to less damping. This may be partly explained by the fact that the MSL moved about 5 m more inland from period 1 to 2 (Figure 4).

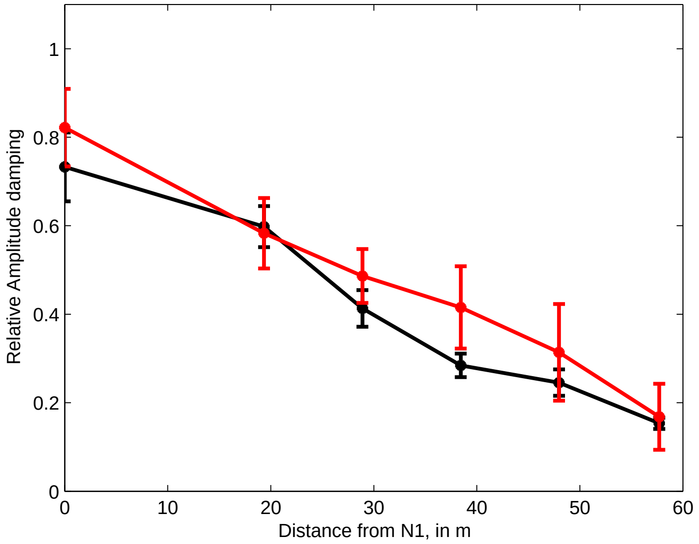


Figure 11: Amplitude damping in transect North. The mean amplitude damping is shown at each well with diver +/- one standard deviation. Black and red lines are period 1 and 2, respectively.

Transects Central and South show a clear tendency towards less damping during period 2, which, again, may be explained by the fact that the MSL moved about 20 m inland, see Figure 3. Figure 22 shows this in another way, where the mean hydraulic heads in the wells have been plotted against the measured beach slope on March 26 and April 2.

Notice that the standard deviation in amplitude reduction is much greater for transects Central and South during period 2. That is, it appears that the beach responds more erratically during this period.

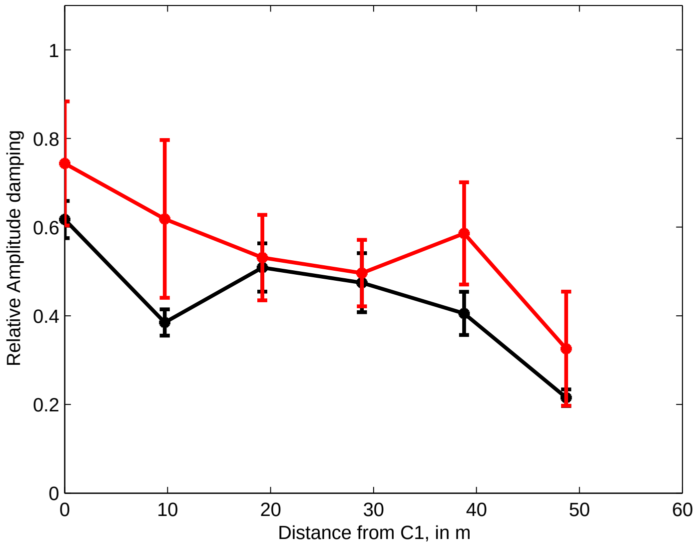


Figure 12: Amplitude damping in transect Central. The mean amplitude damping is shown at each well with diver +/- one standard deviation. Black and red lines are period 1 and 2, respectively.

Figure 14 shows the mean hydraulic head in all wells in the three transects. In all cases the the water table is higher in period 2. This is better seen in Figure 15 where the mean hydraulic head during period 2 was subtracted from the mean hydraulic head during period 1. The three transects show almost identical trends with mean hydraulic heads of 5-35 cm higher in period 2 than in period 1. Recall that Figure 4 showed that the MSL increased by about 35 cm from period 1 to 2. This effectively meant that the MSL moved at least 5-20 m inland. The differences in mean hydraulic heads are less around 10-20 m from the wells nearest to the sea. The reason for this is not known. The micro-topography (Figure 3) would actually trap water in this zone during period 2 and lead to extra infiltration. This would lead to consistently higher hydraulic heads during period 2, and, thus, can not explain the observations. The differences may very well be related to the position of the seepage face during periods 1 and 2. In period 2 the seepage face has likely moved inland. Notice also that the mean hydraulic head is greater during period 2 in the most inland wells up to about 30 cm. This may seem contradictory to the the general behaviour of tidal damping as a function of distance from the coast

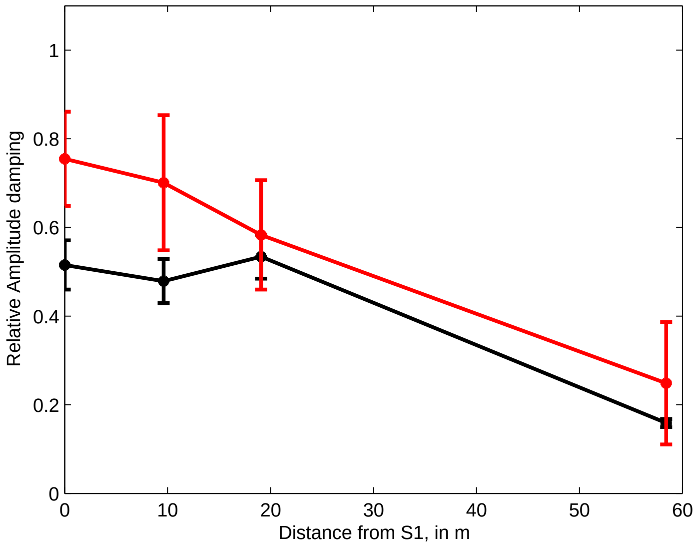


Figure 13: Amplitude damping in transect South. The mean amplitude damping is shown at each well with diver +/- one standard deviation. Black and red lines are period 1 and 2, respectively.

line. This may be explained by the added water table over height or increased inflow of groundwater from upstream areas due to rainfall. The amplitude of the water level at Hvide Sande increases from 0.36 to 0.49 m from period 1 to 2. The water table over height can be computed from $0.5\epsilon A=0.5A^2 k\cot\beta$, where k is the wave number (3). This relation is strictly only valid for computing the extra over height upstream to the high water mark. This condition is only fulfilled for period 1. Using A=0.36 m from period 1 and the parameters from Table 2 together with the estimated hydraulic conductivities (30-140 m/day) gives an extra over height during period 1 of about 11-24 cm. During period 2, with A=0.49 m, the over height becomes 20-44 cm depending on the choice of K. Thus, it is likely that natural physical phenomena can explain the extra observed increase in water level in the most inland wells in period 2.

6.2 Analysis including PEMs

Figures 16-21 show a sequence of measured hydraulic heads in the wells and PEMs during low and high tides. Only transect C is analyzed.

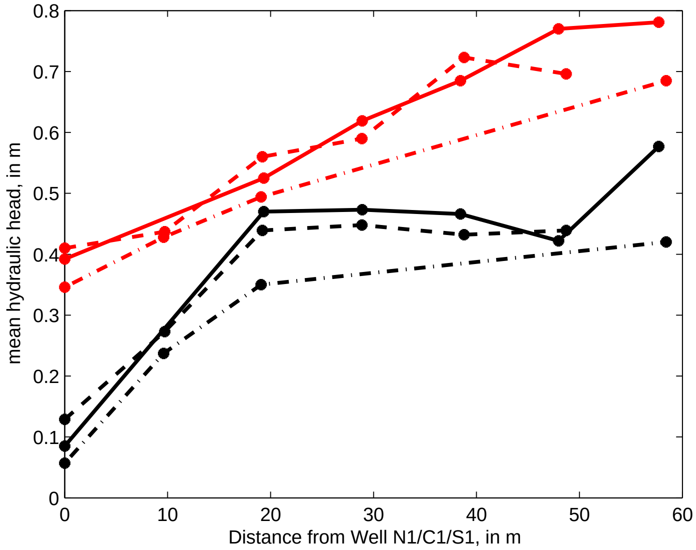


Figure 14: Mean hydraulic head in all wells in the three transect, North (solid), Central (dashed), South (Dash-Dot), with black and red indicating period 1 and 2, respectively

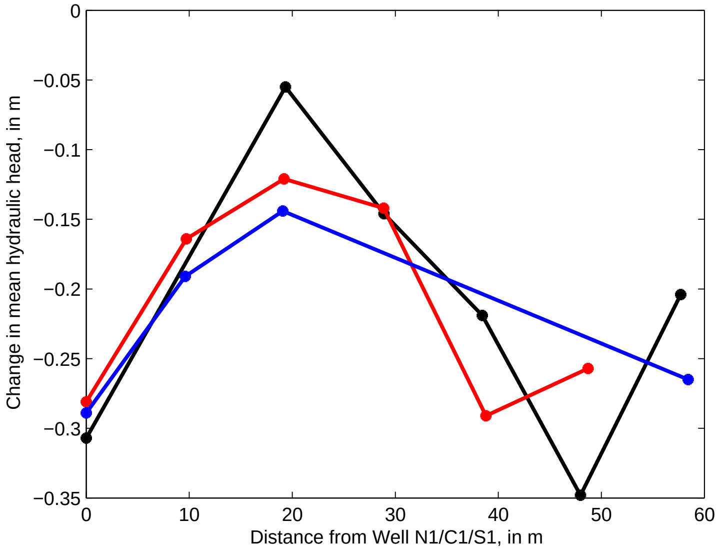


Figure 15: Change in mean hydraulic head in all wells in the three transect, North (black), Central (red), South (blue)

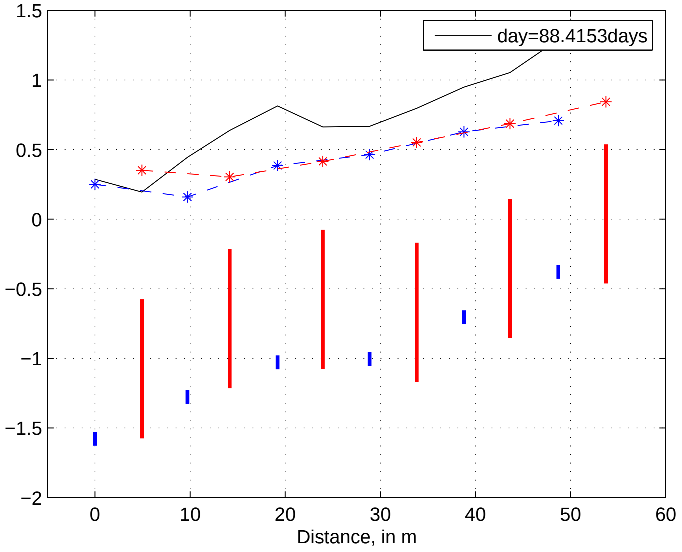


Figure 16: The hydraulic head in period 2 at low tide (88.41 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

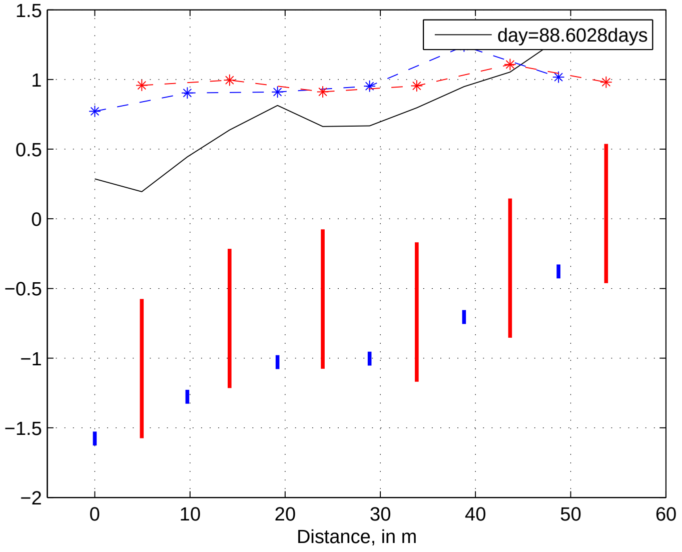


Figure 17: The hydraulic head in period 2 at high tide (88.60 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

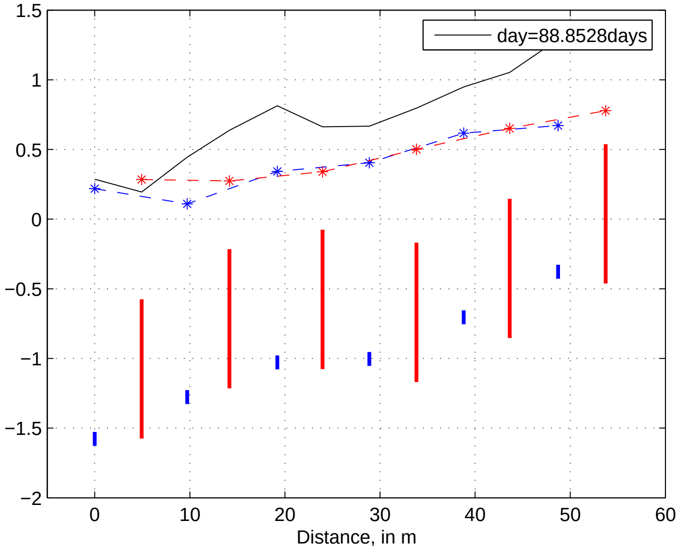


Figure 18: The hydraulic head in period 2 at low tide (88.85 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

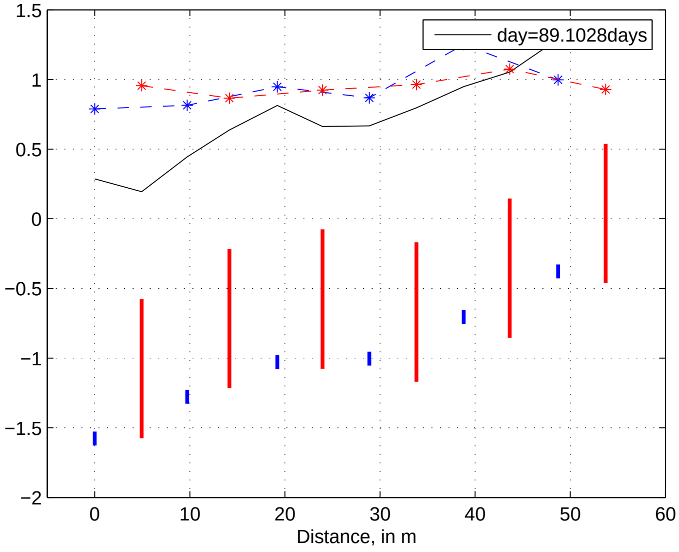


Figure 19: The hydraulic head in period 2 at high tide (89.10 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

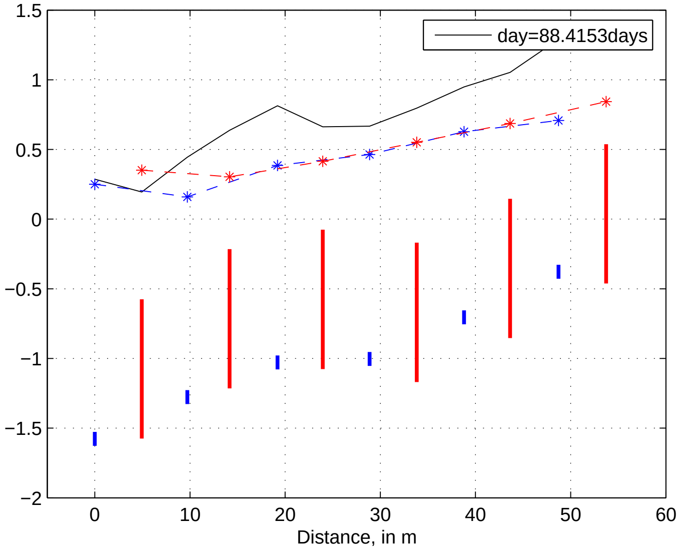


Figure 20: The hydraulic head in period 2 at low tide (89.35 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

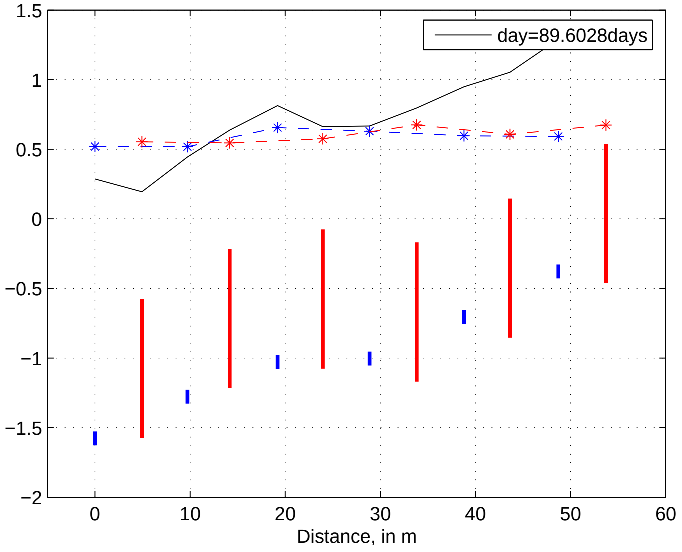


Figure 21: The hydraulic head in period 2 at high tide (89.60 days) for wells and PEMS in transect C. The location of the screens are indicated (red=PEMs, blue=wells)

The hydraulic heads are higher in the PEMs than in the wells in the active forcing zone, 0-20 m, and very similar to the hydraulic heads measured in the wells at distances greater than 20 m (Figures 16-21). Generally, the same pattern is found throughout period 2. Figure 22 show the time mean hydraulic head in the PEMs and wells. Again, the hydraulic heads in the PEMs in the active forcing zone (PEMs Ca an Cb)are higher than in the wells in the same zone (C1, C2, C3). This is consistent with other findings from laboratory experiments [Cartwright et al., 2003, 2004] and field measurements [Raubenheimer et al., 1999], where downward flow was observed during high tide and also as an average over a tidal cycle.

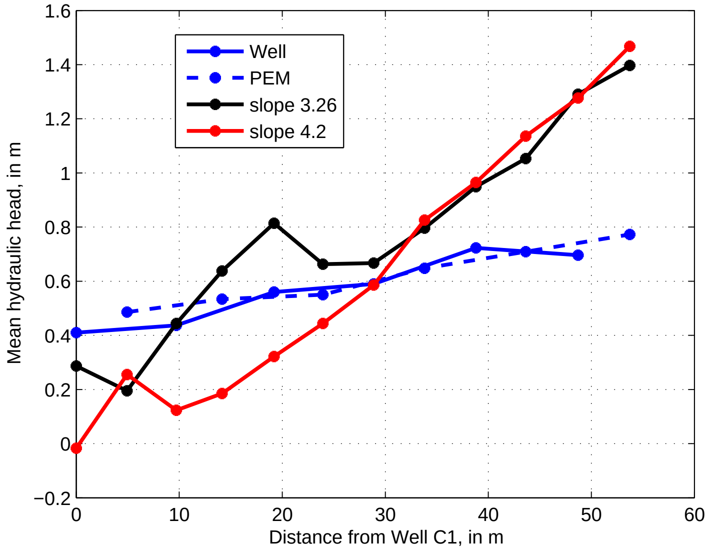


Figure 22: The mean hydraulic head in period 2 for wells and PEMS.

7 Conclusions

An analysis on tidal response in a beach was performed on data from a two-week field-scale experiment at Holmsland.

The analysis is primarily based on a before-and-after situation, where so-called Pressure Equilibrium Modules (PEMs) were installed in week 2. The hydraulic functioning of the beach during week 2 can be compared with week 1 and also compared with a reference site, where no PEMs were installed. The PEMs may result in a more permeable beach because the long screens can intersect several small gravel layers making the whole beach more conductive. Infiltrating water could thereby drain better away.

The analysis is exclusively performed on tidal data where the high frequency waves have been filtered out.

The hydraulic behaviour of the beach in damping the tidal signal was investigated and compared between week 1 and week 2. The analysis is based on similar principles as applied by Carr [1971], model predictions by Nielsen [1990] for beaches of different permeability, and observations in laboratory and field experiments by Cartwright et al. [2003, 2004]; Raubenheimer et al. [1999].

This leads to the following conclusions;

- The damping is less in period 2 (week 2), which is explained by the fact that the mean sea level moved 5-20 m more inland due to a combination of increase in water level at Hvide Sande and a change in beach profile.
- The temporal mean hydraulic heads increased in reasonable correspondence with the observed water levels at Hvide Sande and the fact that a sloping beach leads to an extra water table over height at inland wells.
- A comparison of the mean hydraulic heads in the wells and PEMs suggest that there is a downward flow in the tidal active zone. This is in agreement with laboratory and other field-scale findings.
- In all cases the transect with both wells and PEMs (Central) act very similar to the transect with just wells in both period 1 and 2. Any differences can be explained by the differences in beach profile.

In summary, it is concluded that, for this beach-scale analysis, the PEMs seem to have little effect on the tidal dynamics. The observed differences between periods 1 and 2 and between the Central and North transects can be explained by the physical situation (beach profile) and physical flow processes.

8 Appendix A: Least square filter, FIR

The design of the filter was originally proposed by Bloomfield [1976]. The FORTRAN programs developed by Bloomfield [1976] were rewritten in the MATLAB script language by Boon [2004]. These MATLAB scripts were modified as a part of this project. The method is also known as the Finite Impulse Response (FIR) filter.

The idea of a filter is to smooth a time series by removing all periodic motion oscillating above a specified cutoff frequency while retaining oscillations at or below the exact same frequency unmodified [Boon, 2004]. First of all, the linear filter is based on a weighted moving average

$$h_t^{'} = \sum_{k=a}^{b} w_k h_{t-k} \tag{5}$$

where w_k is a series of weights and h_t and $h_t^{'}$ are the observed and filtered data at time t.

Bloomfield [1976] gives a nice example of how one should choose the weights very carefully. For example, a linear filter

$$h_t^{'} = \frac{1}{3}(h_{t-1} + h_t + h_{t+1}) \tag{6}$$

with w_k=1/3 (constant) and h_t=Acos(ωt-kx), i.e., a pure sinusoidal signal, will produce an output ($h_t^{'}$) that is unmodified for frequencies near zero, whereas frequencies ω=2π/3 will be removed completely.

However, an optimal filter can be designed [Bloomfield, 1976]. Ideally one would like a filter with the following characteristics;

$$\begin{aligned} M(\omega) &= 1(0 \leq \omega \leq \omega_c) \qquad (7)\\ &= 0(\omega_c \leq \omega \leq \pi) \qquad (8) \end{aligned}$$

where ω_c is a cutoff frequency. Ideally, if one could have a filter like M(ω) then it would be possible to filter out all data with frequencies above the cutoff frequency (e.g. high frequency waves). M is also called a response curve.

Without going into details it is possible to show that the weights can be computed as

$$\begin{aligned} w_0 &= \frac{\omega_c}{\pi} \qquad (9)\\ w_k &= \frac{sin\omega_c k}{\pi k} \qquad (10) \end{aligned}$$

where k=1,m, and m is the width of the filter. The width of the filter specifies the steepness of the response curve. The larger the width the steeper the response curve gets.

In practice it is not possible to specify an exactly abrupt response curve and one is left with what is called the transition band. To get a small transition band also requires that one is prepared to sacrifice 2m values, i.e., the first m values and the last m values. Also, it is often found that the response curve can over- and undershoot (oscillate around 1 and 0). This can be reduced by mulitiplying the filter weights with a convergence factor, i.e.,

$$w_k = \frac{sin\omega_c k}{\pi k}\left(\frac{sin 2\pi k/(2m+1)}{2\pi k/(2m+1)}\right) \tag{11}$$

9 Appendix B: Test of method

Figure 23 shows the simulated tidal response in two wells located a distance of 20 and 50 meters from the position of the mean sea water level for the conditions of a vertical beach (i.e., cotβ=0). When the beach is vertical there is no lifting of the water table, and the only difference in the tidal signal is the significant damping at x=50 m.

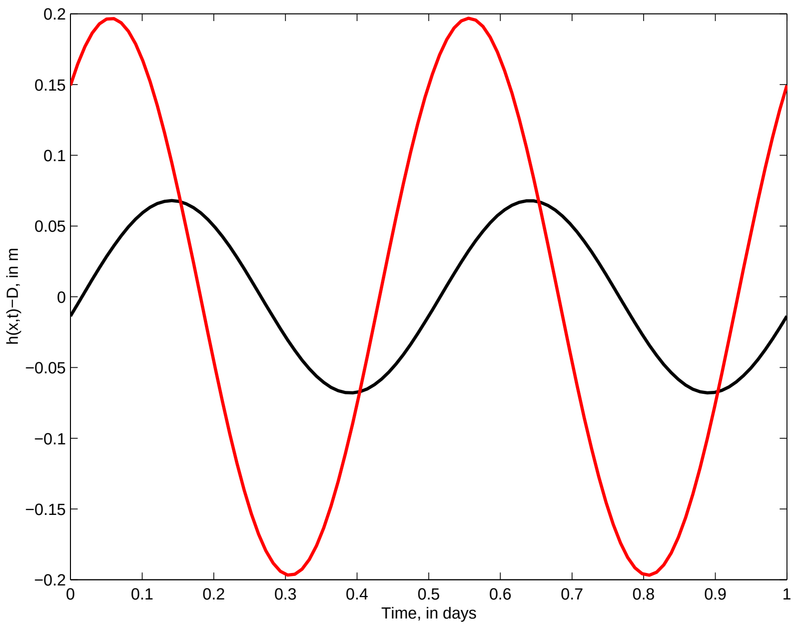


Figure 23: Simulated tidal responses in two wells at x=20 m (red) and x=50 m (black) for a vertical beach. Parameters can be found in Table 2

By use of (2) the relation between the total amplitudes can be given as;

$$\frac{a_{50}}{a_{20}} = e^{-k\Delta x} \tag{12}$$

where a_{50} and a_{20} are the total amplitudes at x=50 m and 20 m, and Δx is the distance between the two wells, i.e., 30 m. The total amplitudes are 0.3927 and 0.1359 m at x=20 and 50 m, respectively, and the wave number can be computed as k=0.0355. By use of (3) it is possible to calculate n/K, the two hydraulic parameters. For example, assuming the porosity n=0.20 is known (i.e. used in the model), one can calculated that K=49.98 m/day, very close to the input value that was used to generate the tidal responses shown in Figure 23.

Figure 24 shows the same type of simulation, but now with a sloping beach, Table 2.

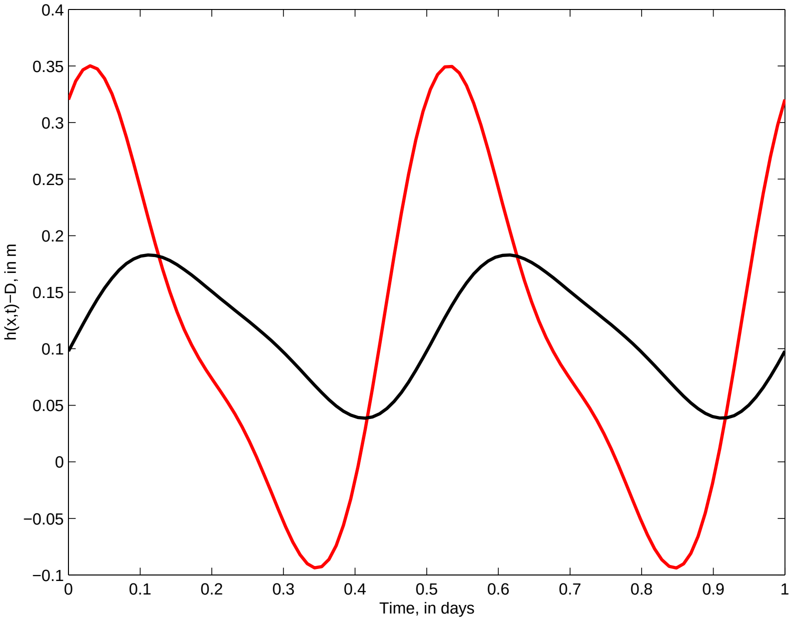


Figure 24: Simulated tidal responses in two wells at x=20 m (red) and x=50 m (black) for a sloping beach. Parameters can be found in Table 2

As mentioned in Section 3 the effect of a sloping beach is to lift the mean water level recorded in the well a quantity $\epsilon A/2$ above the mean sea level, where $\epsilon = kA\cot\beta$. For example, the mean water level in Figure 24 is 0.1133, or ϵ=0.5664. Thus, the wave number can be calculate to be k=0.0354, and by use of (2) the hydraulic conductivity can be computed to be 50.13 m/day under the same assumption that the porosity is known, n=0.2. This value is very close to the value used to generate the curves in Figure 24.

If the same method is used as in the case of the vertical beach (total amplitude) then K is computed to 44.77 m/day, about 10 % lower than the input value. This is because there is a small damping effect in the third term in (2) not accounted for by the simple amplitude reduction equation (12). However, if K=200 m/day was used to generate the tidal signal, then K can be calculated to be 194.31 m/day, relatively closer to the true value.

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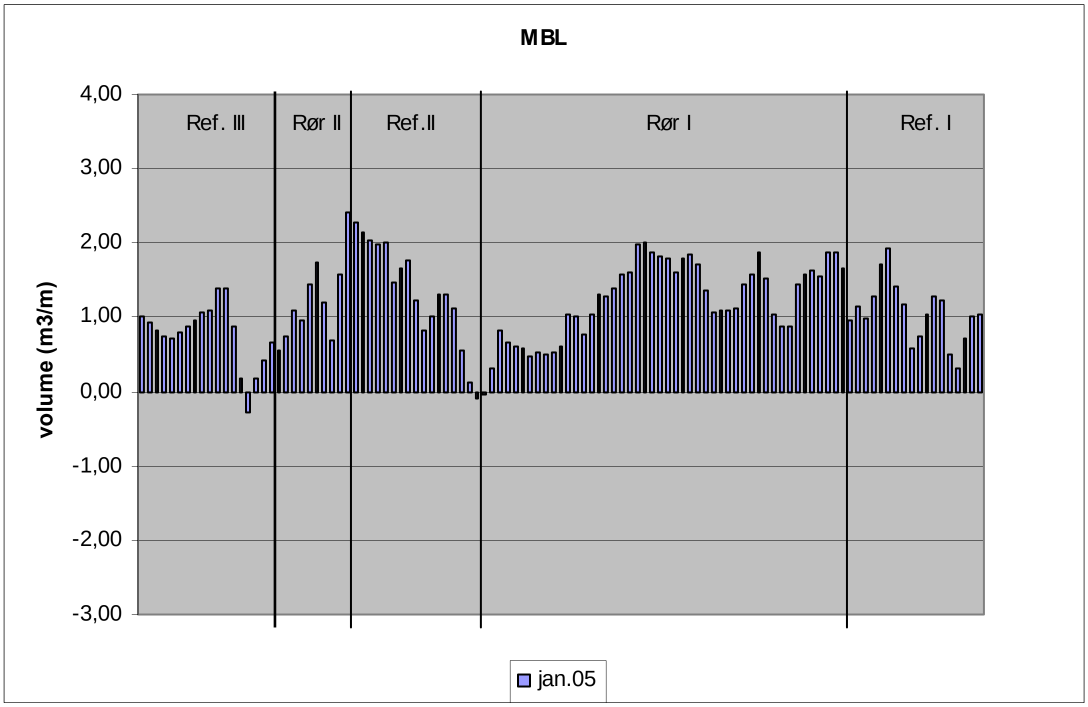
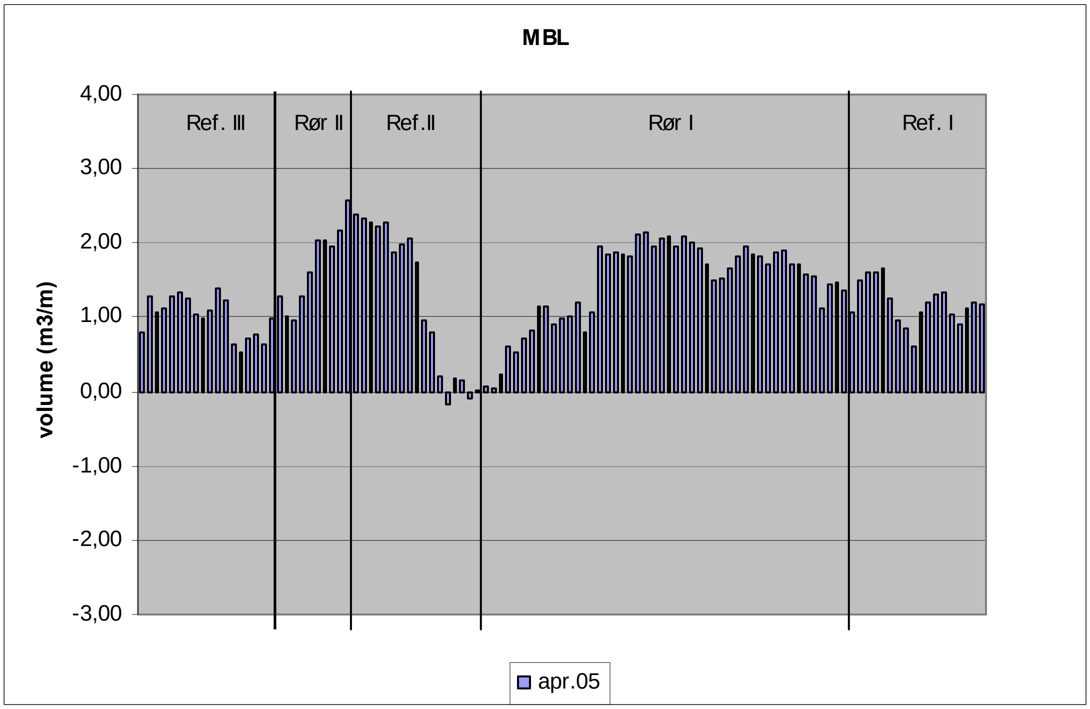
Appendix 4: D-profiles

Changes in mean beach level.

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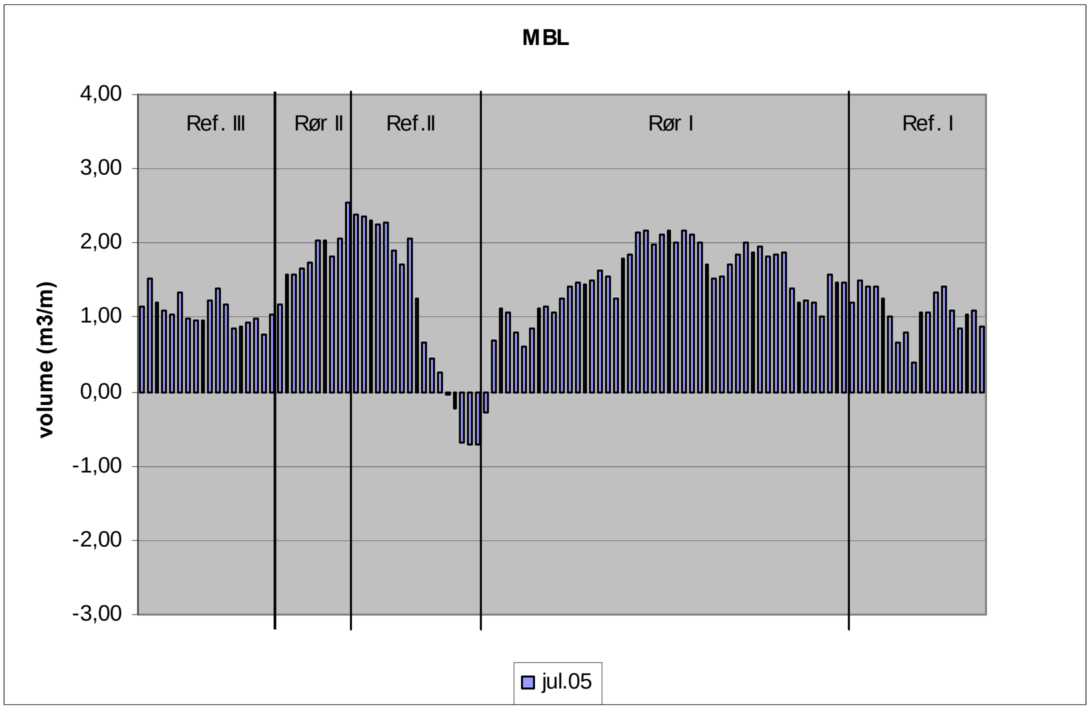
2,00
1,00
0,00
-1,00
-2,00
-3,00
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
apr.05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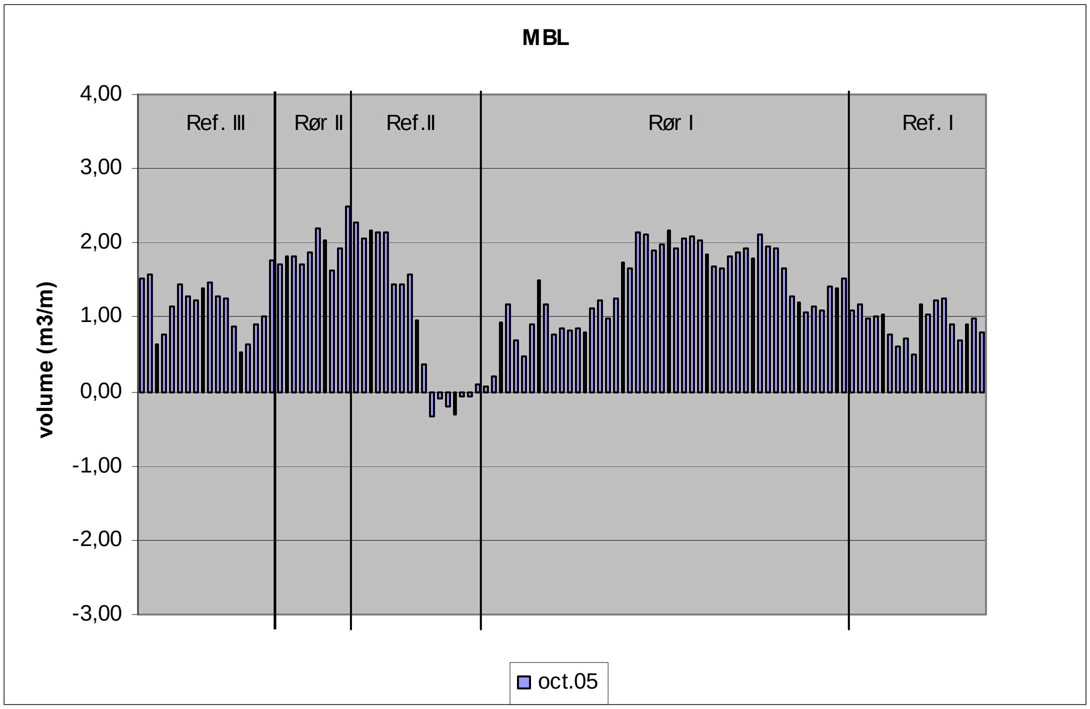
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jul.05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volume (m3/m)
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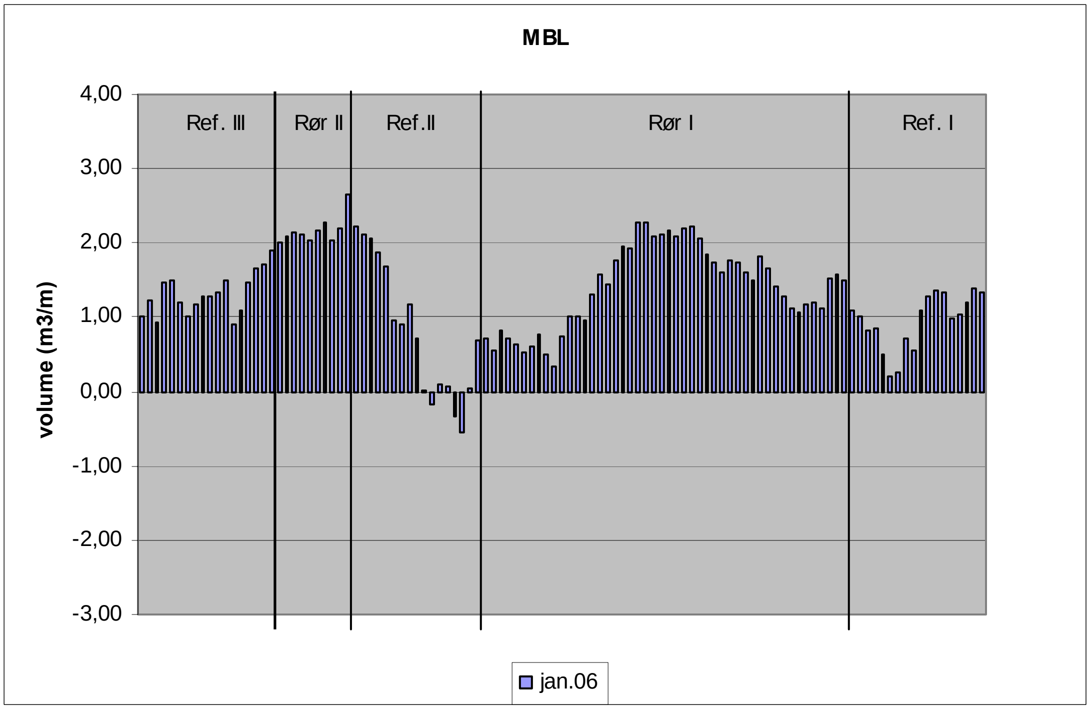
Rør II
Ref.II
Rør I
Ref. I
oct.05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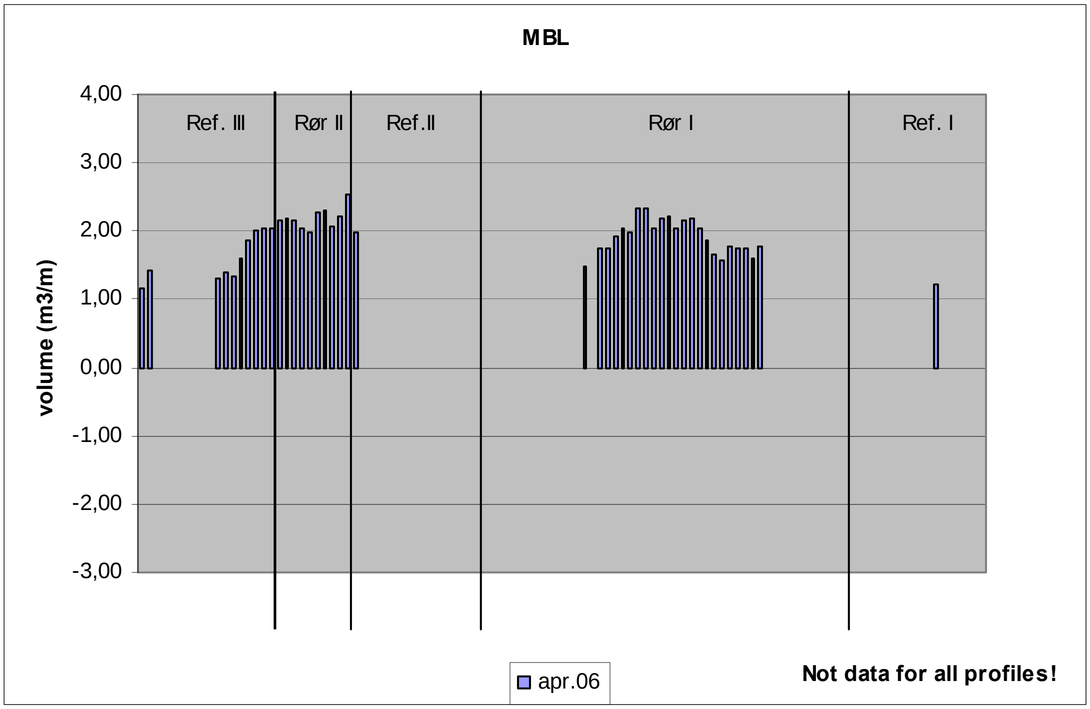
Rør II
Ref.II
Rør I
Ref. I
jan.06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1,00
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volume (m3/m)
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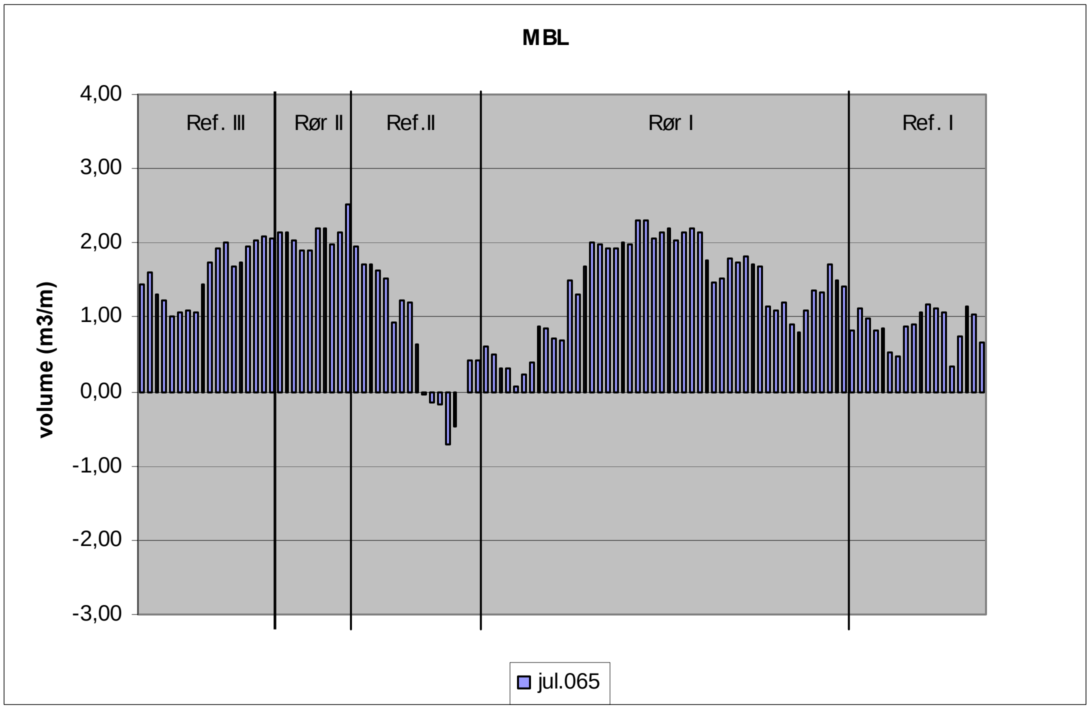
Rør II
Ref.II
Rør I
Ref. I
apr.06
Not data for all profiles!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volume (m3/m)
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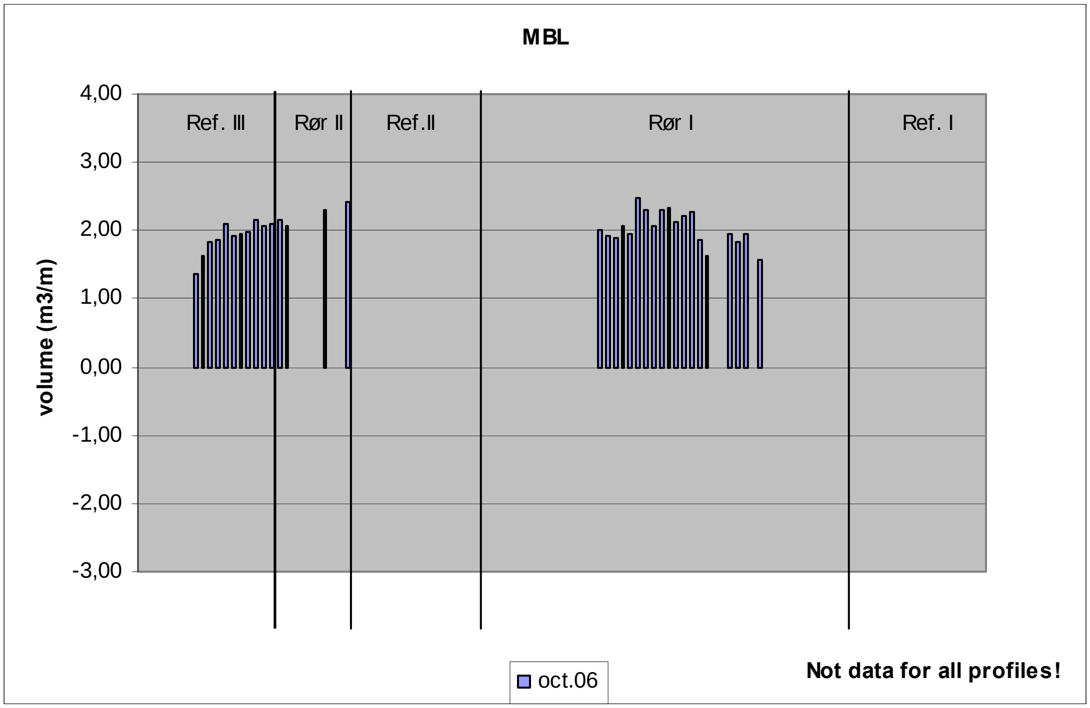
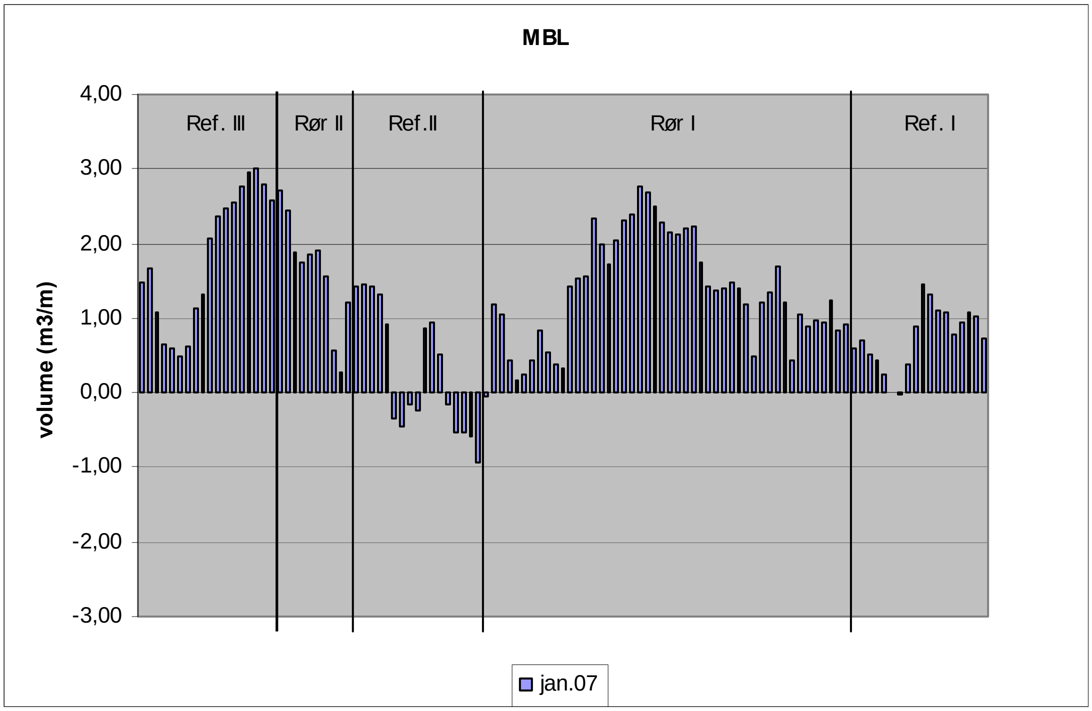
Rør II
Ref.II
Rør I
Ref. I
jul.065



Accumulated changes along the 5 individual reaches.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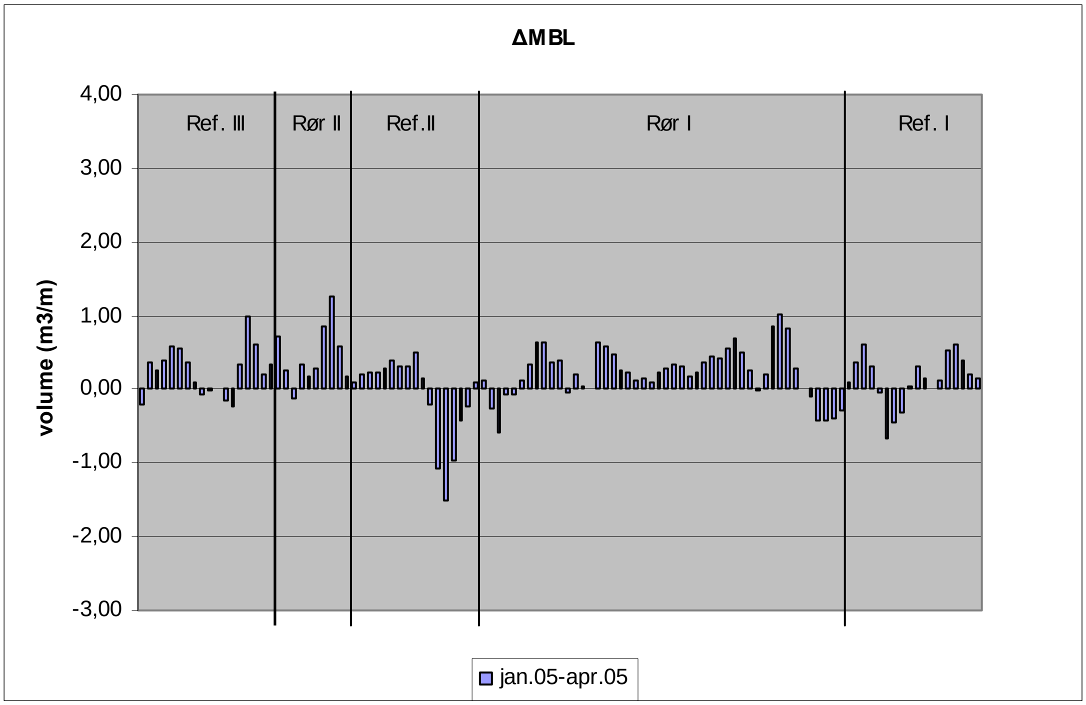
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.07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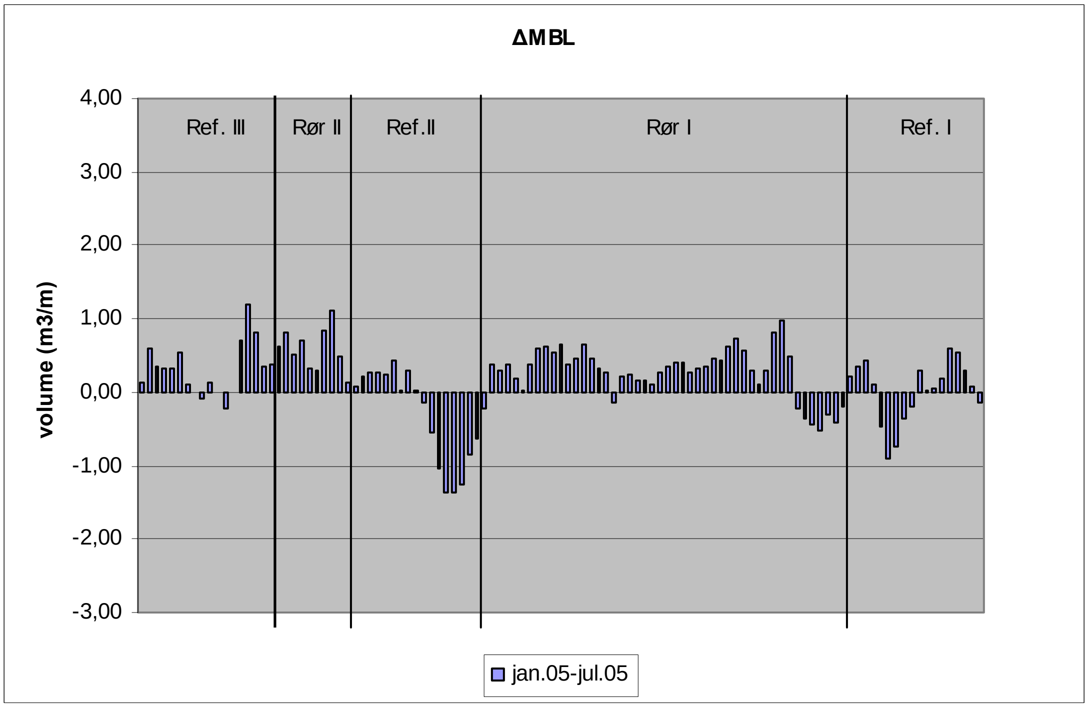
Ref.II
Rør I
Ref. I
jan.05-apr.05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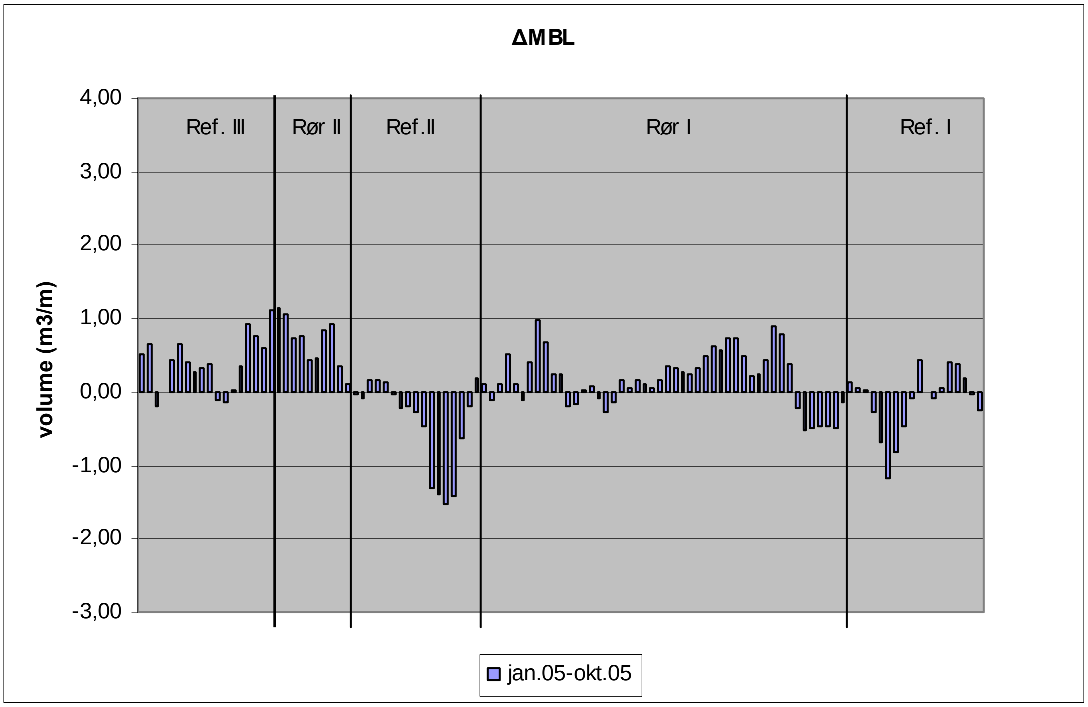
-3,00
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.05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volume (m3/m)
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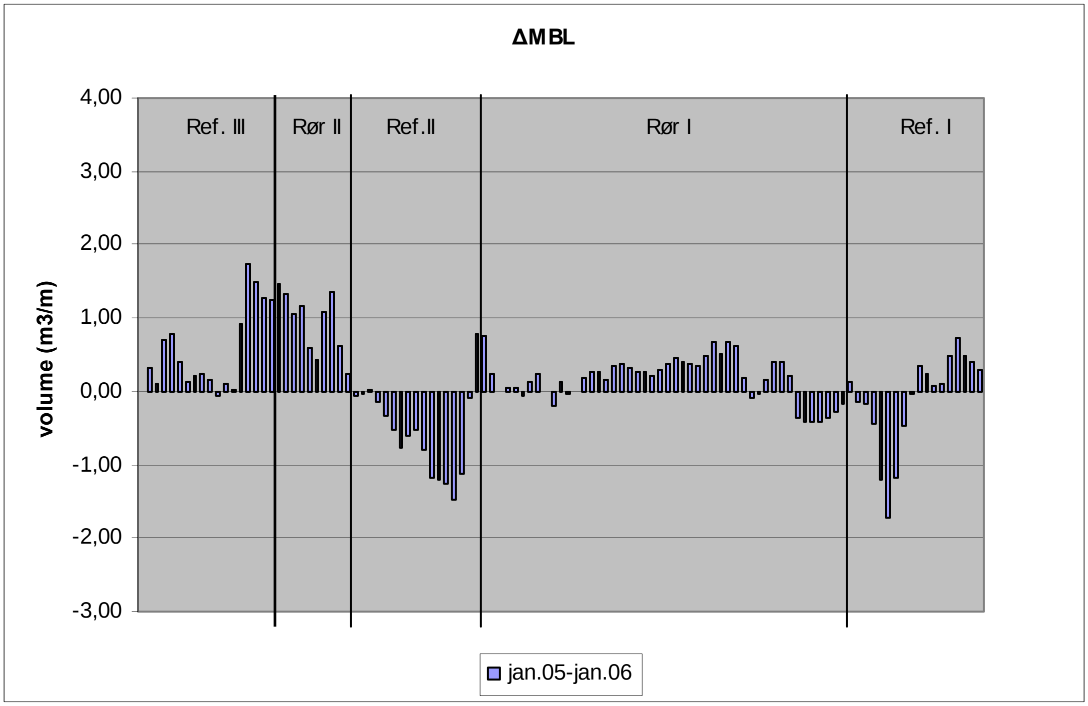
Rør II
Ref.II
Rør I
Ref. I
jan.05-okt.05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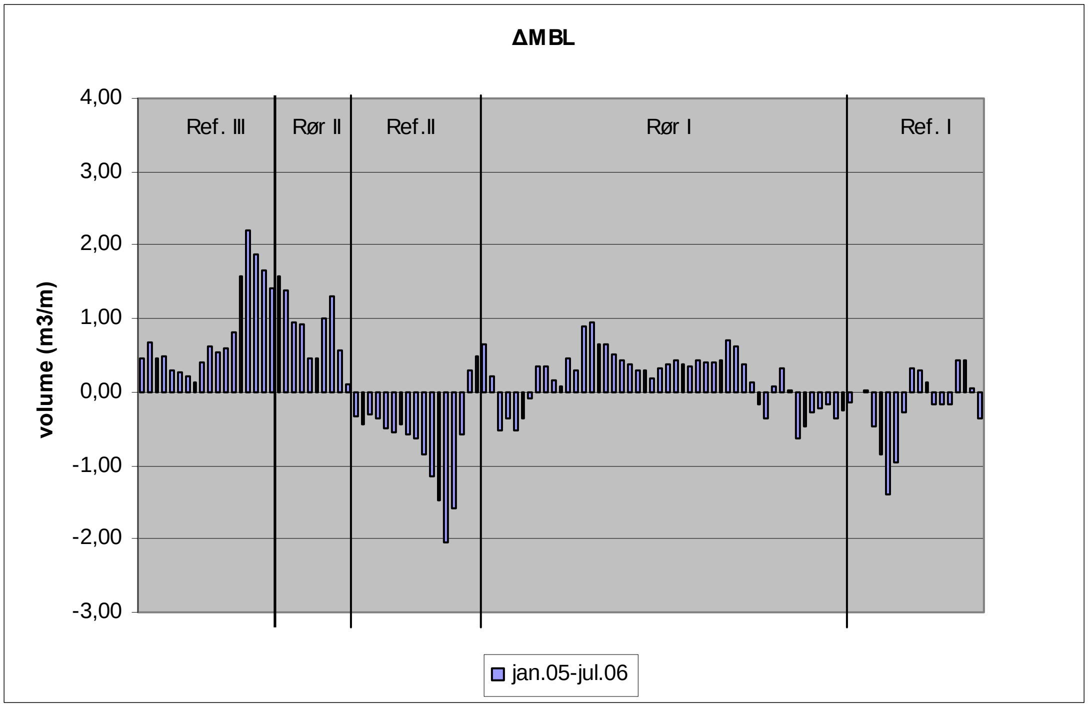
-3,00
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.06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0,00
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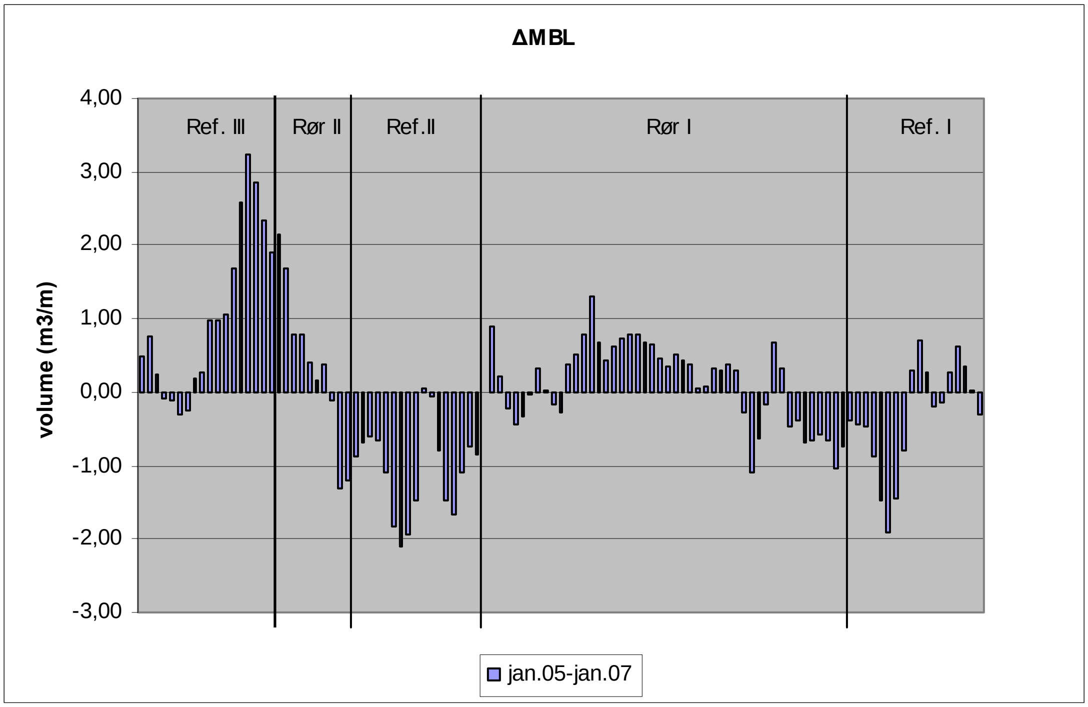
-3,00
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.06


ΔMBL
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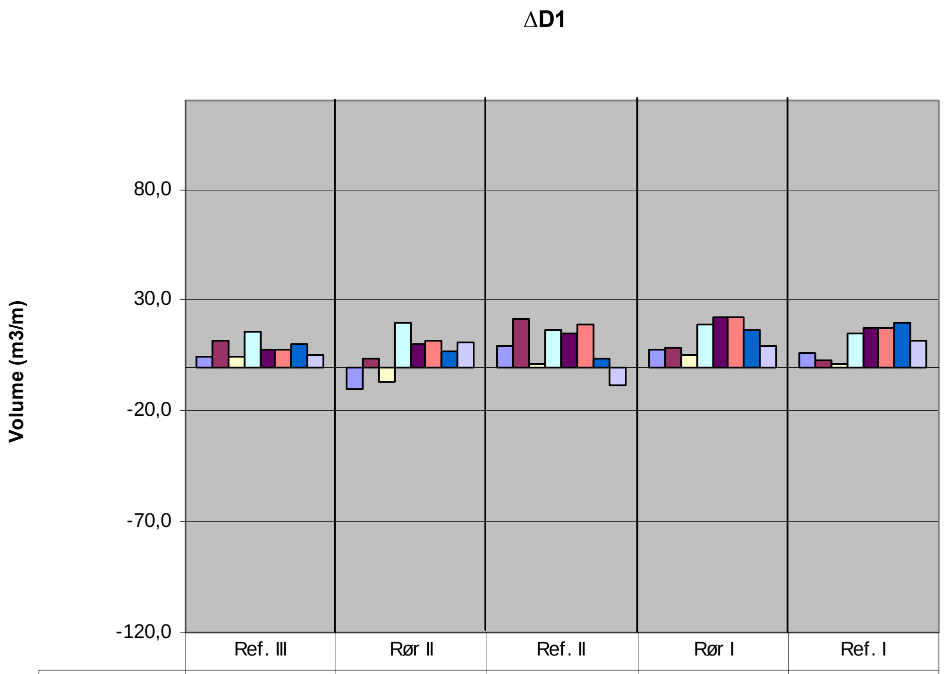
1,00
0,00
-1,00
-2,00
-3,00
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.07

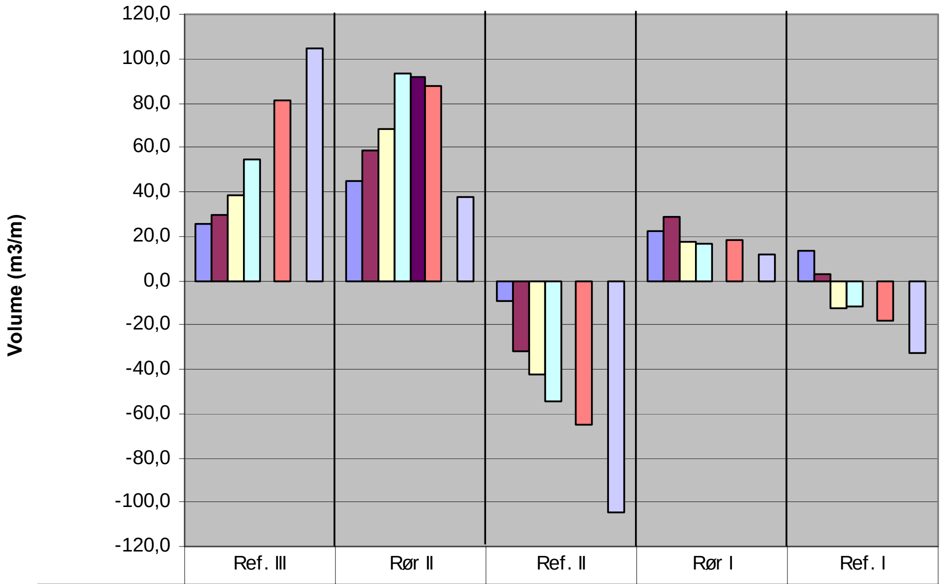
Relative changes in Mean Beach Level.


ΔD1

	Ref. III	Rør II	Ref. II	Rør I	Ref. I
■ jan.05-apr.05	4,7	-9,9	9,2	7,7	5,8
■ jan.05-jul.05	11,5	3,5	21,1	8,3	2,8
□ jan.05-oct.05	4,5	-6,8	0,9	5,5	1,1
□ jan.05-jan.06	16,1	19,7	16,9	19,1	15,3
■ jan.05-apr.06	8,0	9,7	14,7	22,5	17,5
■ jan.05-jul.06	7,7	11,7	18,8	21,9	17,1
■ jan.05-oct.06	9,7	6,9	3,5	16,8	19,5
□ jan.05-jan.07	5,6	10,9	-8,4	9,5	11,6

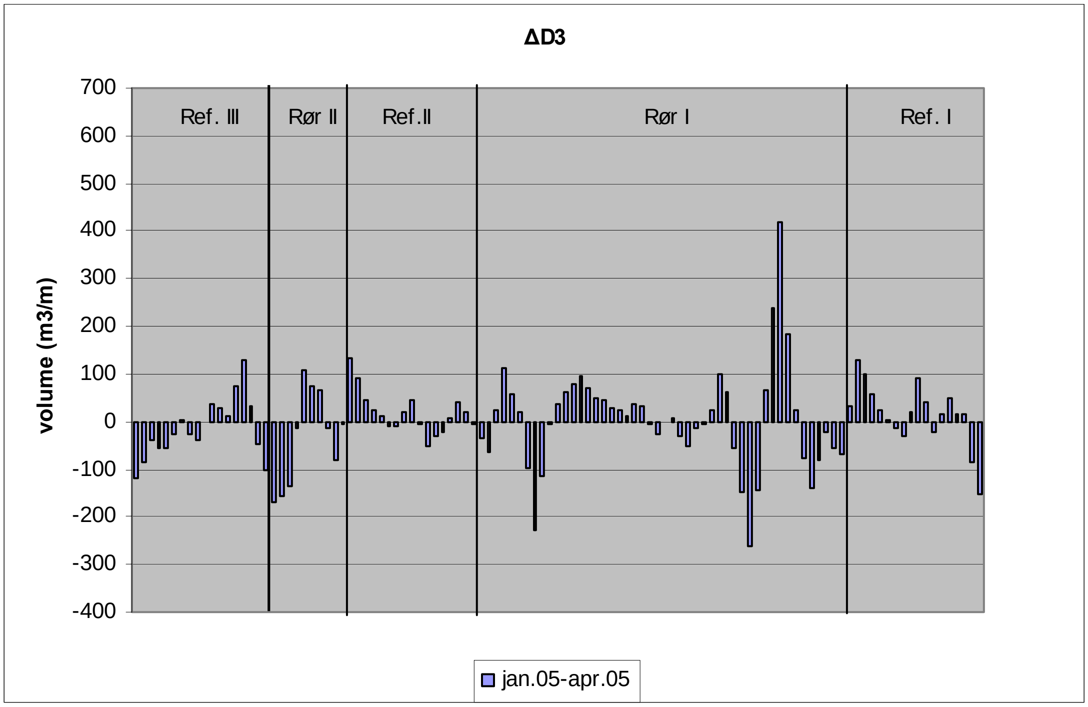
Not all periods represented

∆D2



	Ref. III	Rør II	Ref. II	Rør I	Ref. I
■ jan.05-apr.05	25,2	45,0	-9,5	22,3	13,4
■ jan.05-jul.05	29,3	58,3	-32,2	28,8	2,6
■ jan.05-oct.05	38,2	68,1	-42,3	17,5	-12,2
■ jan.05-jan.06	54,4	93,3	-54,6	16,7	-11,5
■ jan.05-apr.06		91,7			
■ jan.05-jul.06	81,5	87,3	-64,7	18,5	-18,2
■ jan.05-oct.06					
■ jan.05-jan.07	104,3	37,5	-104,8	11,5	-32,5

Not all periods represented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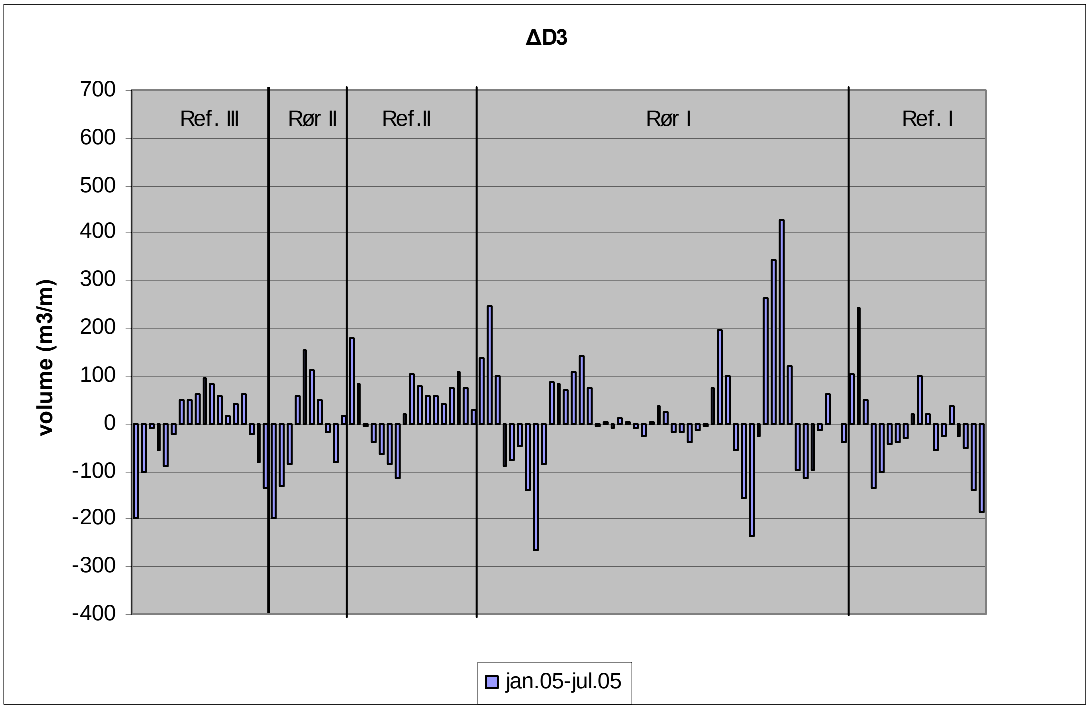
200
100
0
-100
-200
-300
-400
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-apr.05


ΔD3
volume (m3/m)
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0
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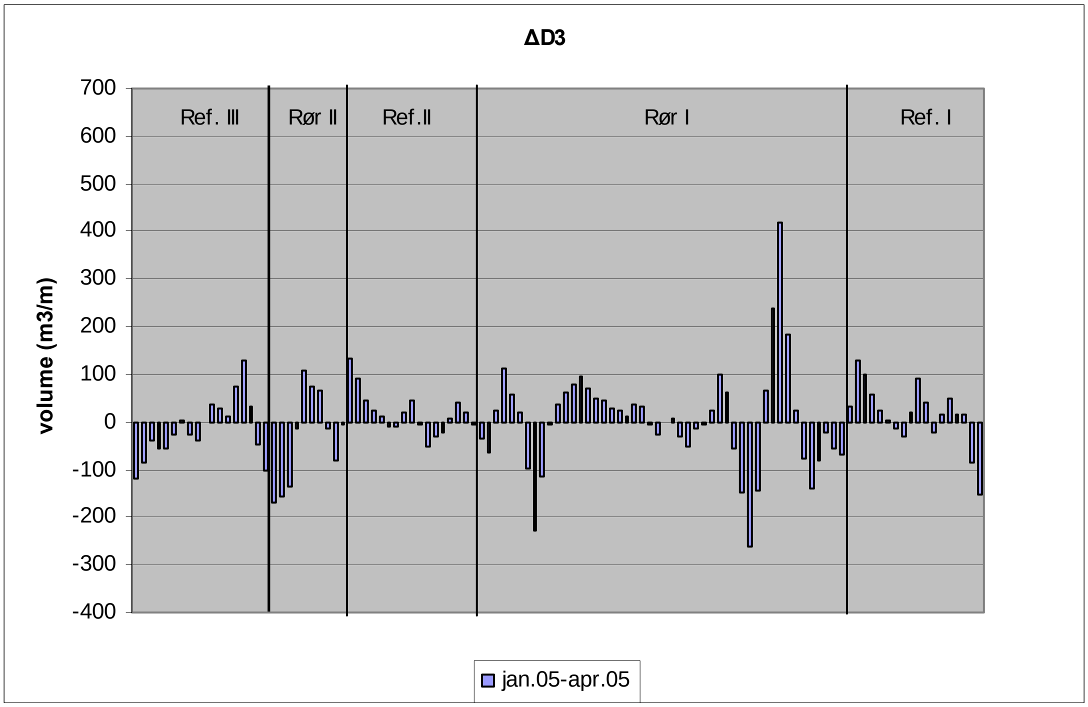
-200
-300
-400
Ref. III
Rør II
Ref.II
Rør I
Ref. I
▨ jan.05-jul.05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volume (m3/m)
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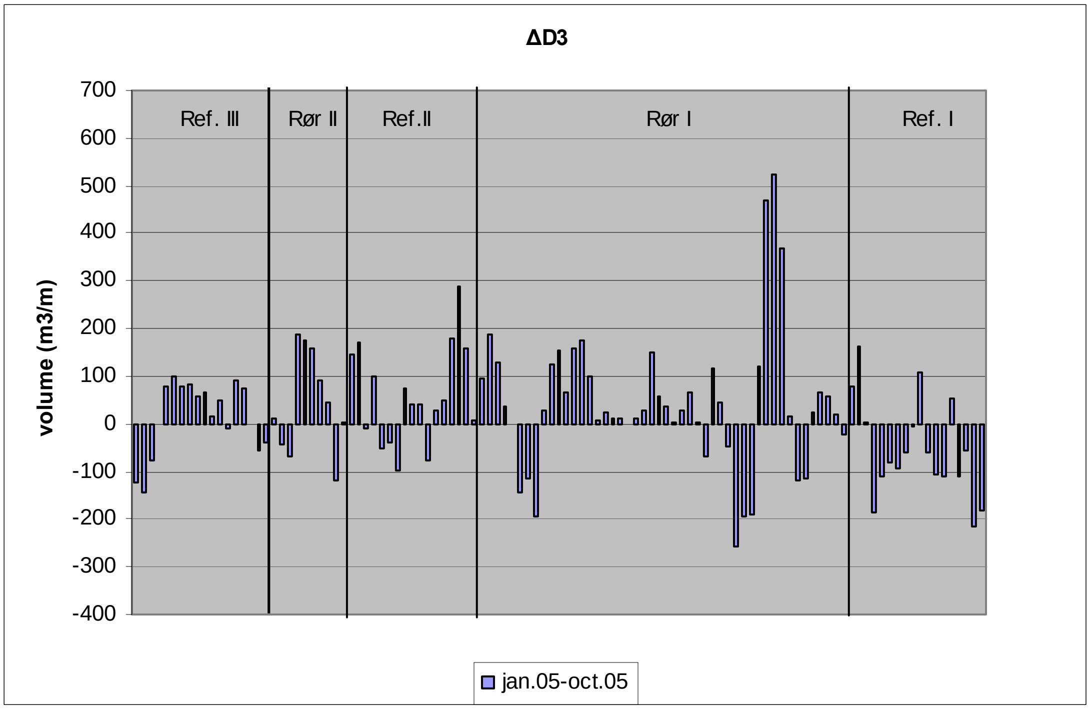
Ref. III
Rør II
Ref.II
Rør I
Ref. I
▨ jan.05-apr.05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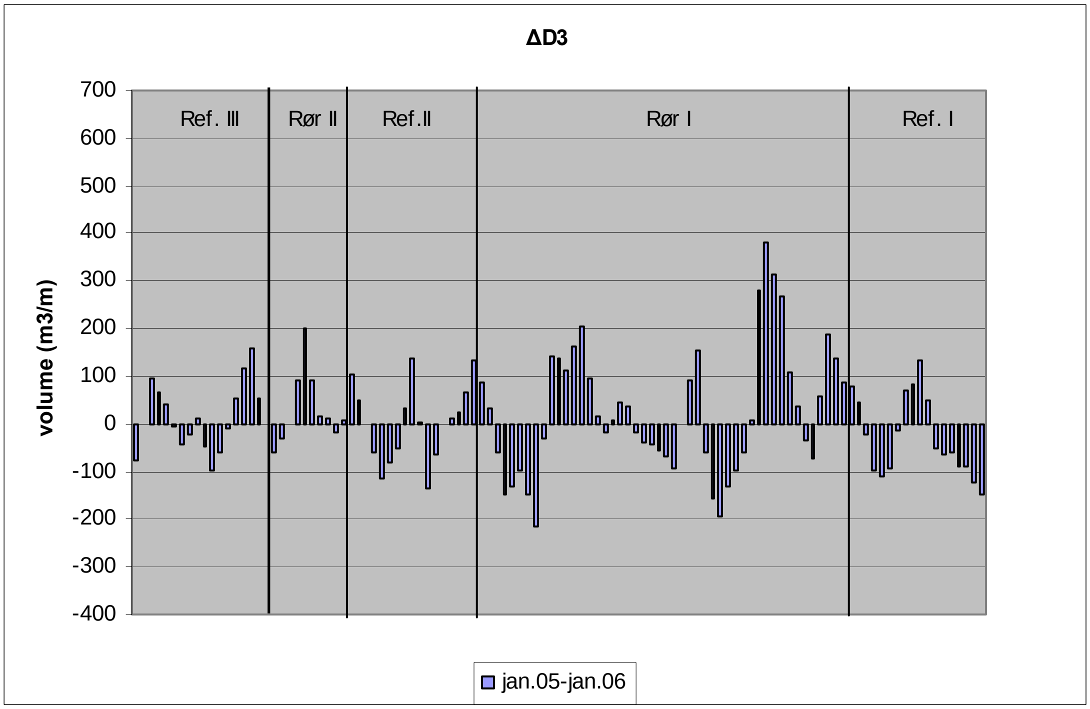
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-oct.05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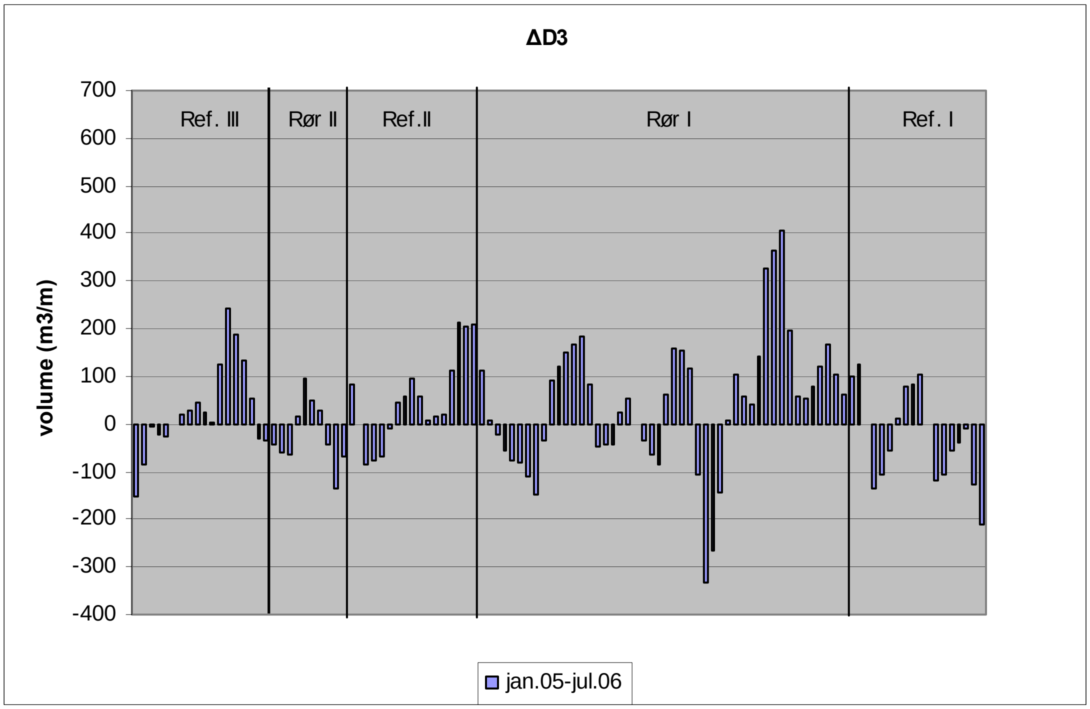
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.06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-200
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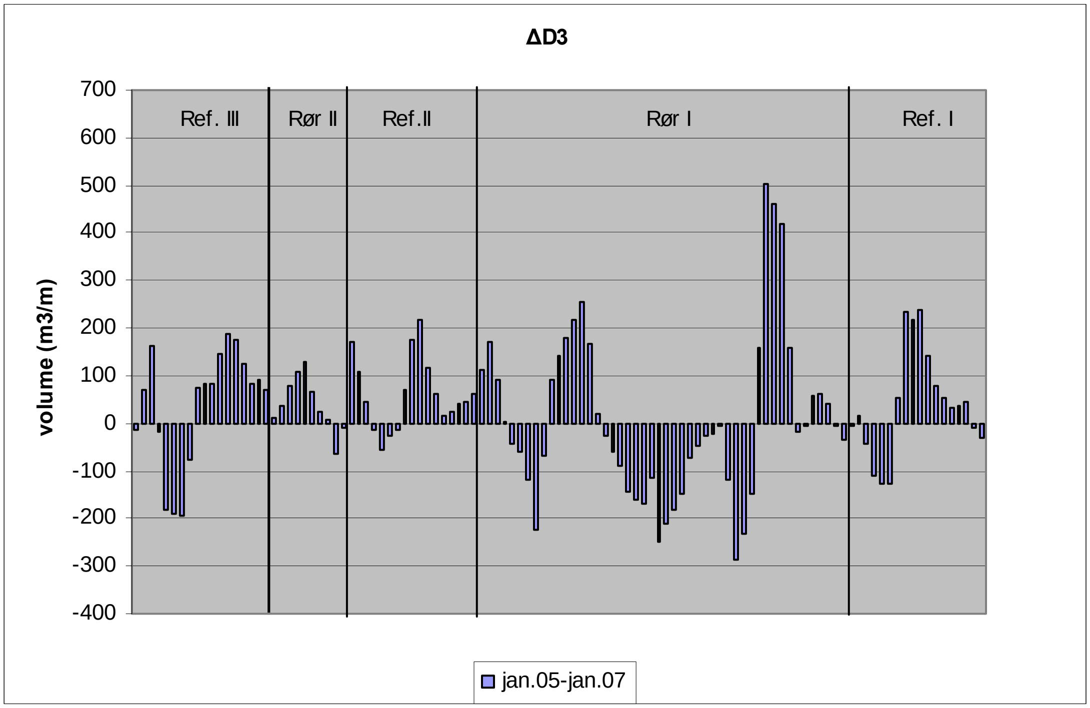
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.06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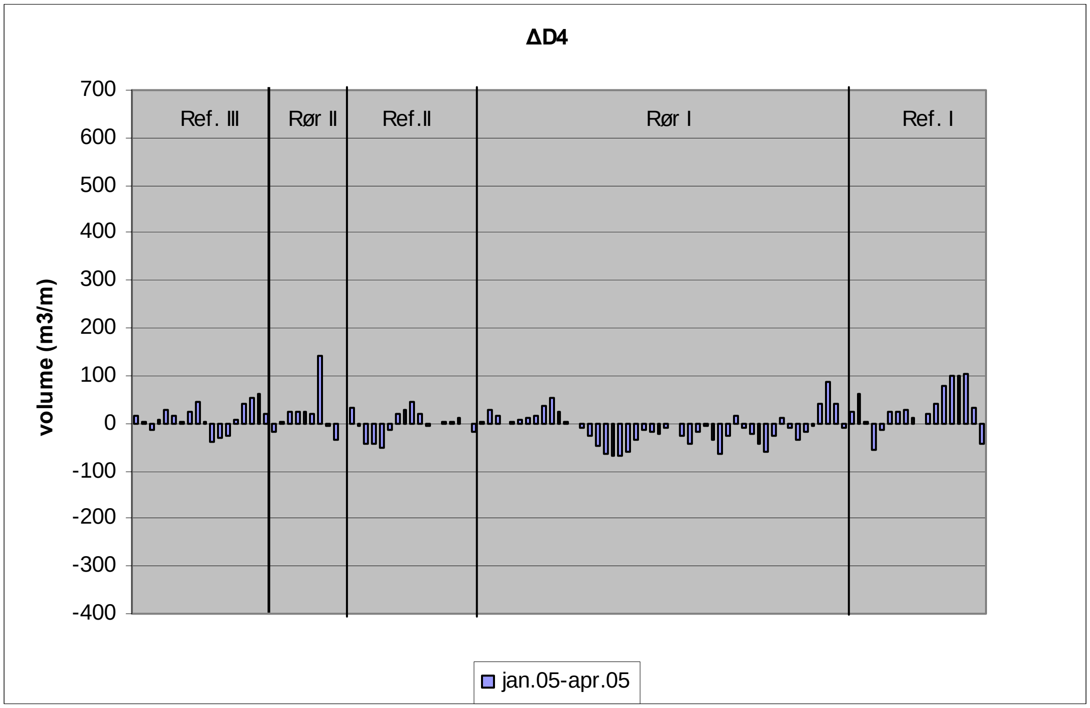
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.07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600
500
400
300
200
100
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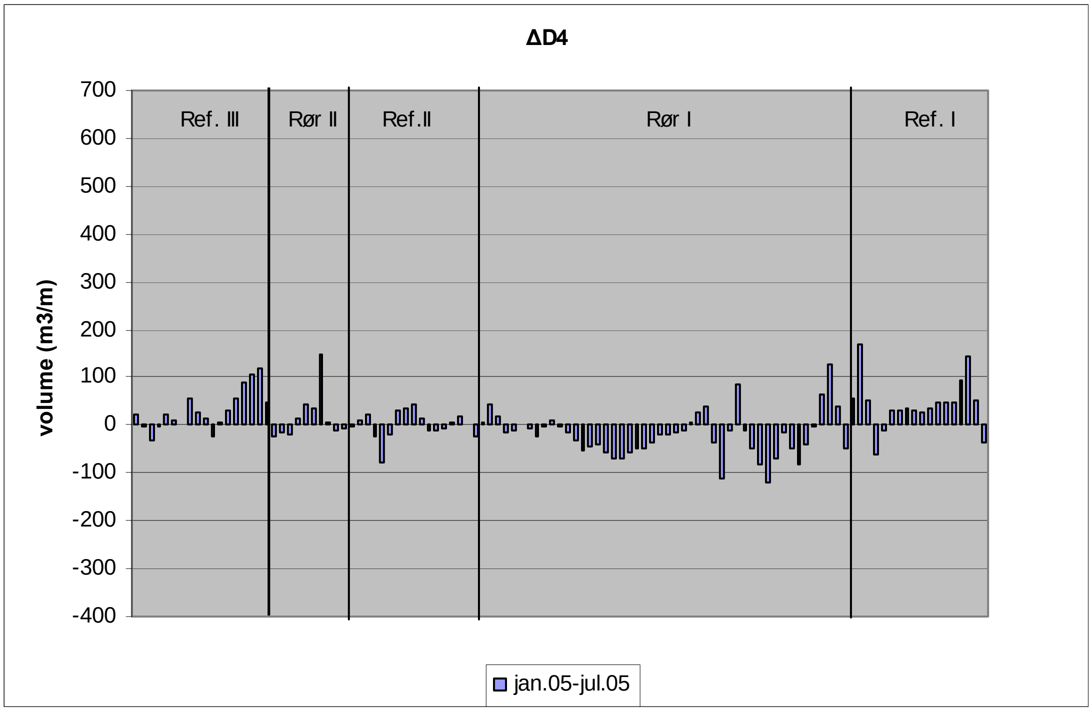
-100
-200
-300
-400
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-apr.05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400
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200
100
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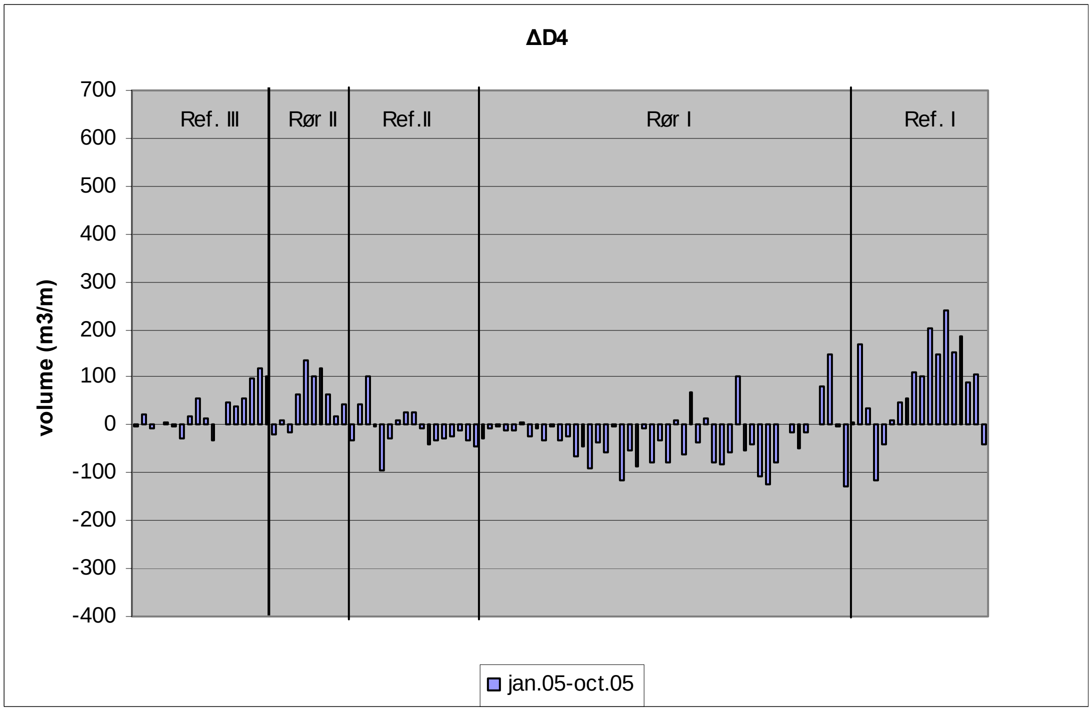
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.05


ΔD4
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400
300
200
100
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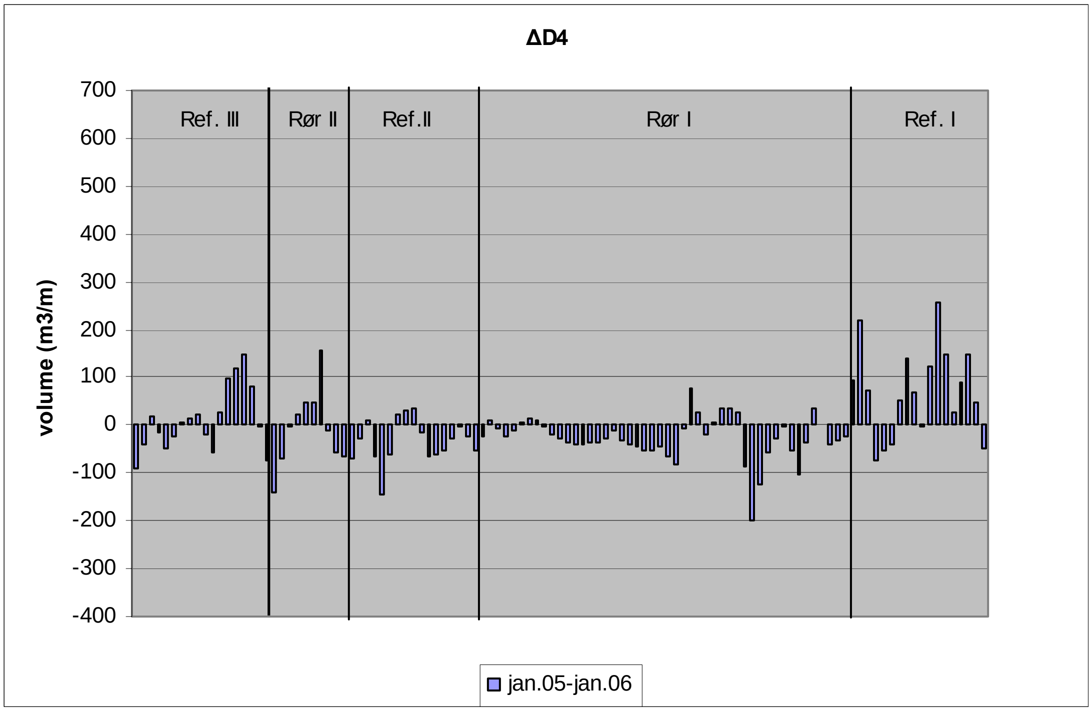
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-oct.05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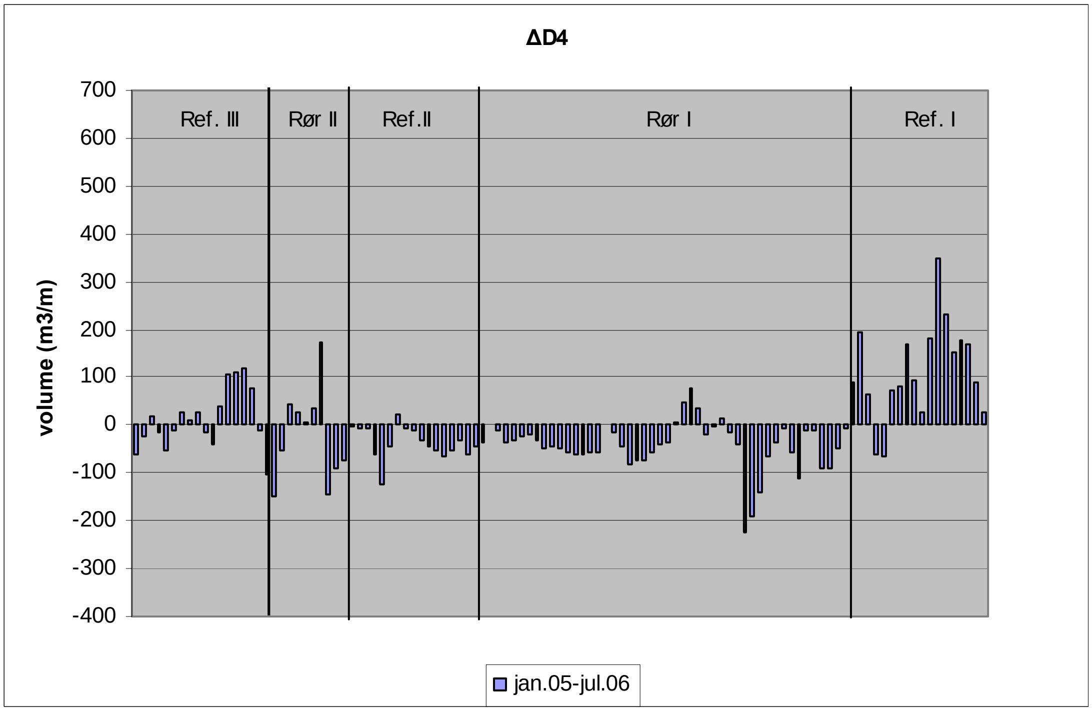
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.06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-200
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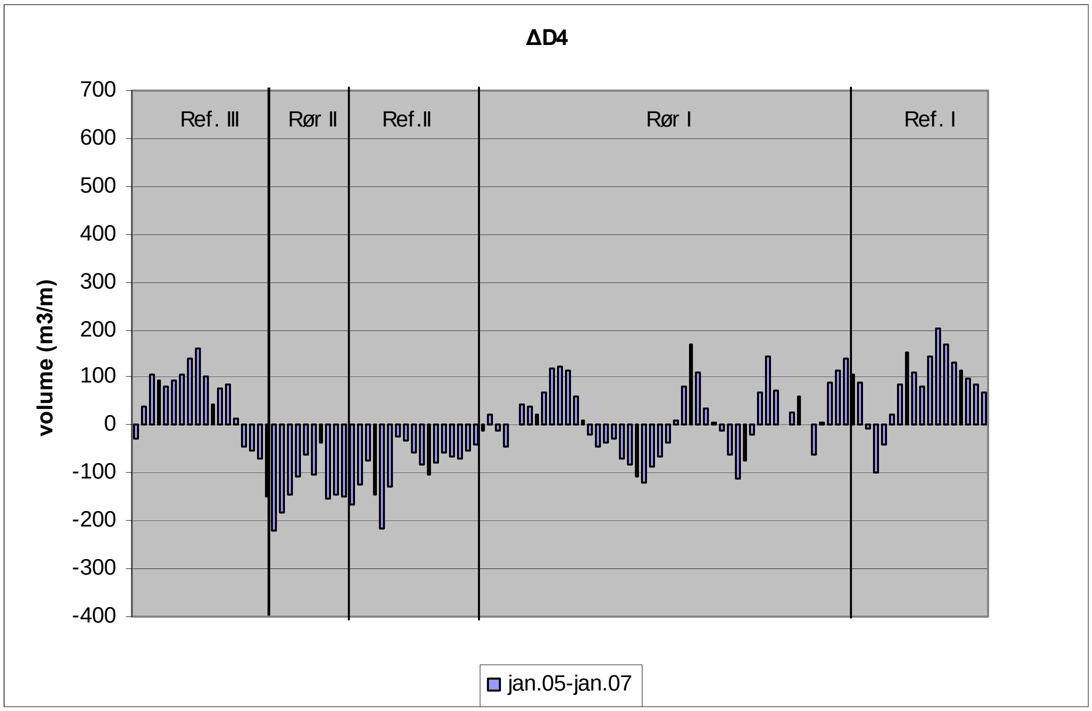
volume (m3/m)
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.06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500
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-300
-400
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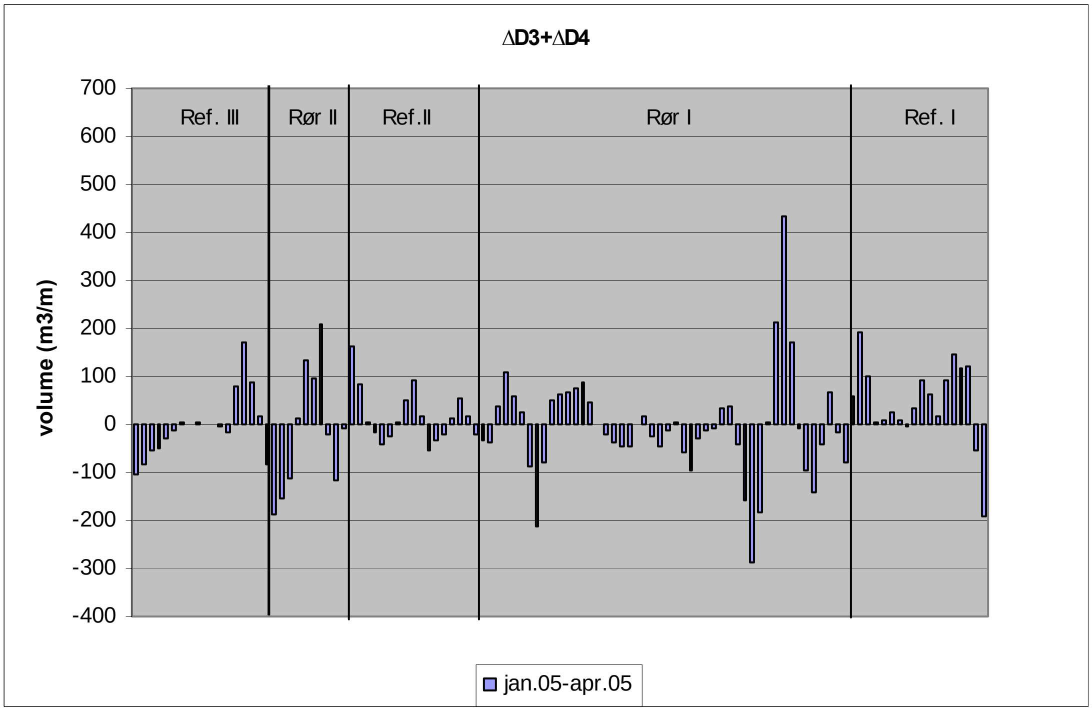
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.07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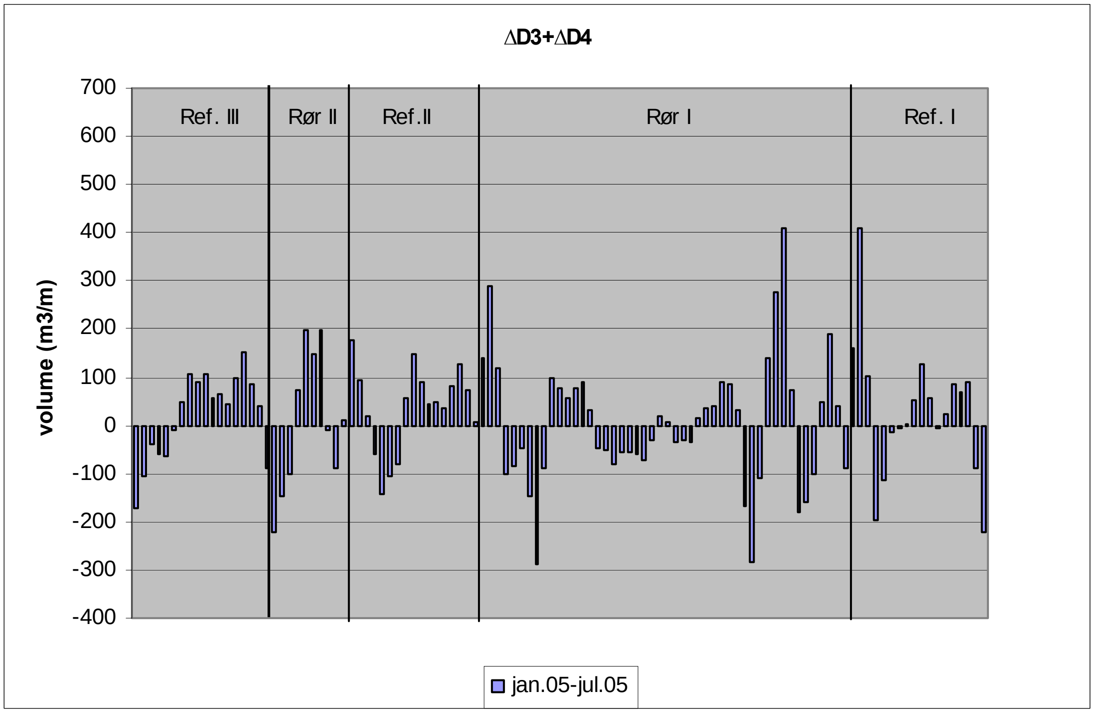
Rør II
Ref.II
Rør I
Ref. I
jan.05-apr.05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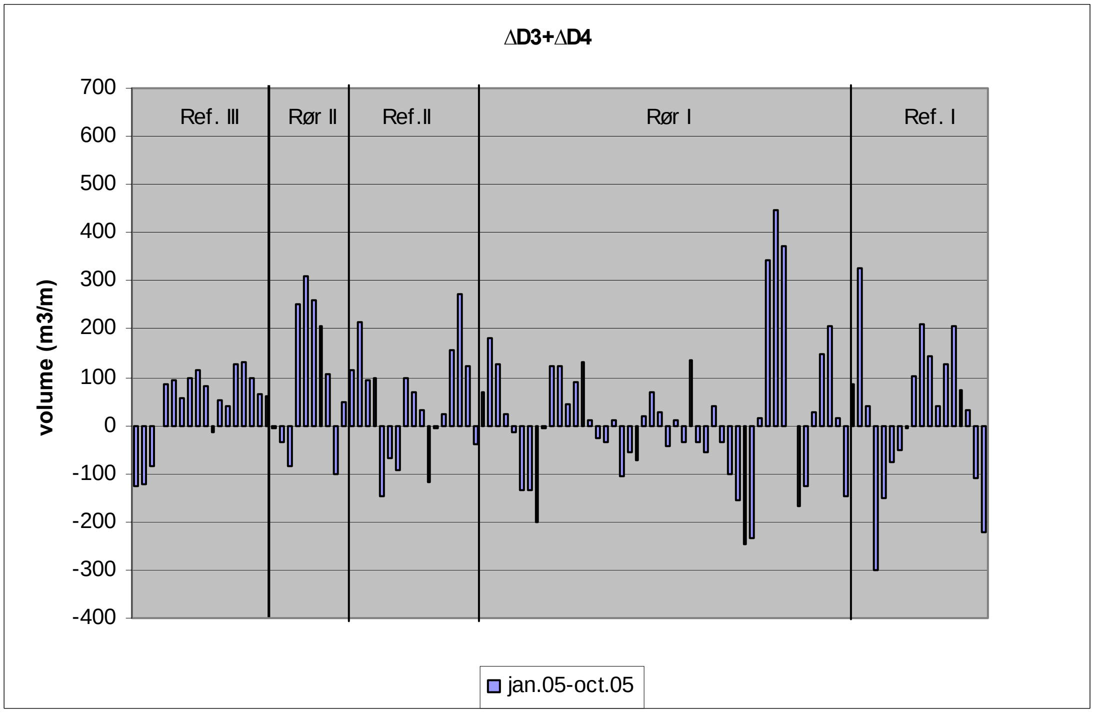
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.05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-400
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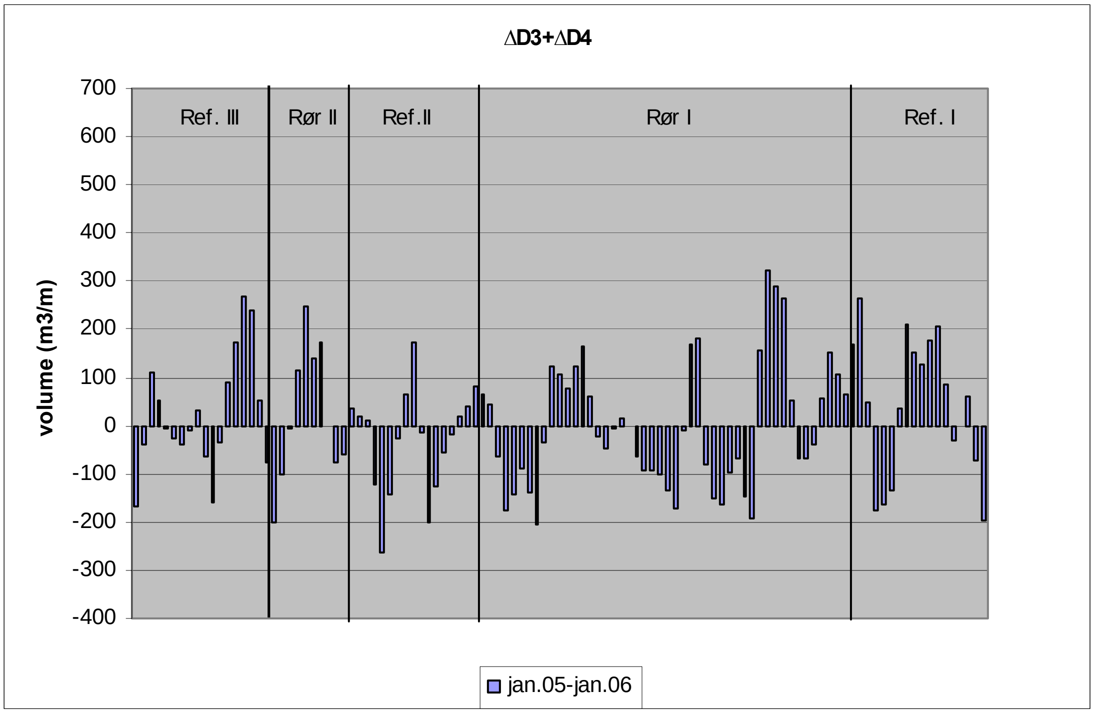
Rør II
Ref.II
Rør I
Ref. I
jan.05-oct.05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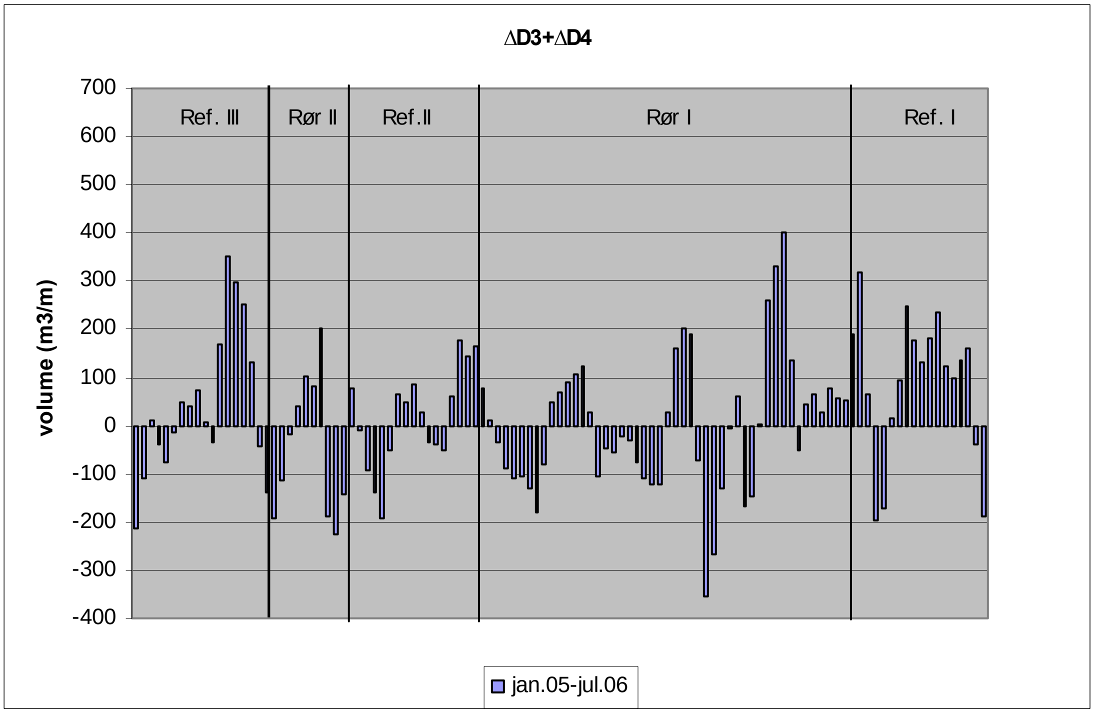
-200
-300
-400
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.06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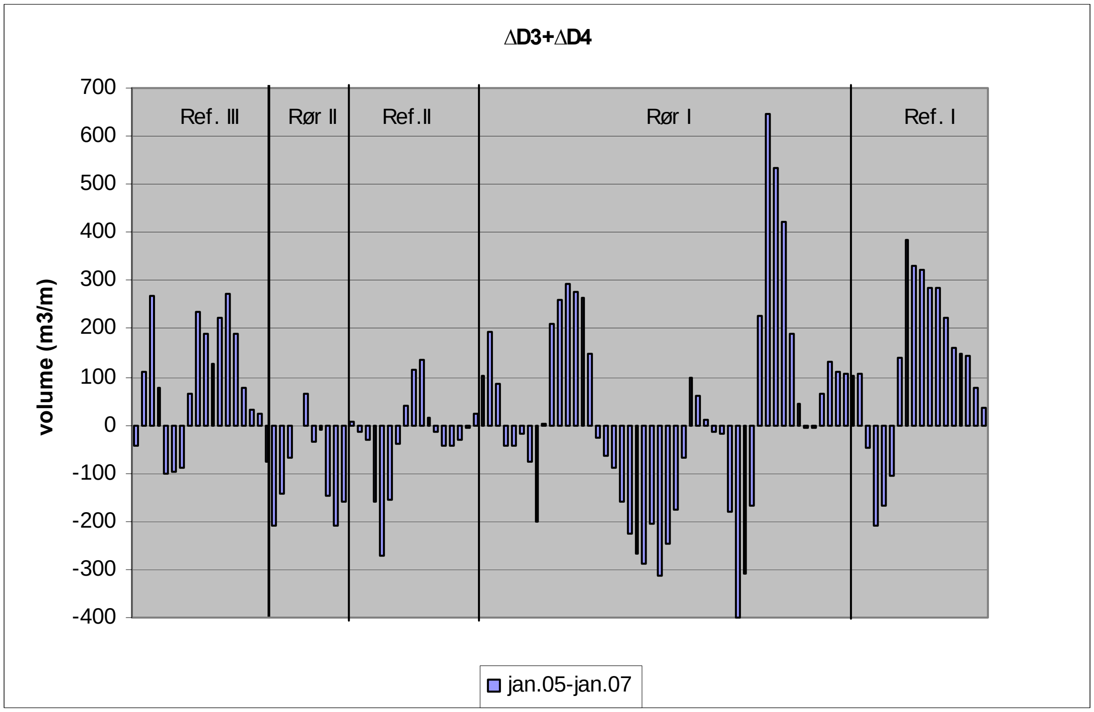
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jul.06

ΔD3+ΔD4
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600
500
400
300
200
100
0
-100
-200
-300
-400
Ref. III
Rør II
Ref.II
Rør I
Ref. I
jan.05-jan.07

Appendix 5. Undulations along the shore.

The observed variation in beach width along the shore may stem either from the PEM-system or be due to natural coastal processes.

1.: Obliquely incoming waves.
It is well known that a coast exposed to obliquely incoming waves can cause undulations in the beach-width. The scale 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 is for instance observed at Uggerby just East of Hirtshals, and at Gl. Skagen at the North tip of Jutland. At both locations, the coast are exposed to very obliquely incoming waves.
At Nymindegab, the angle between the coastline and the dominating incoming waves are in the neighborhood of 45 degrees, see figure chapter 2 figure 3, so also here it is most likely that large-scale undulations will exist.

Figure 1B: wave-rose based on fall measurements in 2005.

From figure 3, chapter 2 it further becomes evident that the sediment drift is in the Southern direction (with the wave climate shown in figure 3, the CERC-formula suggest an annual rate of around 1.2 million cbm, but the wave climate of 2005 and 2006 is actually milder than an average year.).

By inspection of satellite-photos large-scale undulations can be identified, but their behavior (change of shape and migration) is quite stochastic and not so easy to identify during the relative short time of period of the present experiment.

Undulations have been observed at the location of the experiment also before the PEM-experiment was started, so the presence of undulations cannot only be due to the implementation of the PEM-system. Figures 1a and b show the measured long shore variation in beach-width in May 2000 (yellow). August 2002 (blue plus brown) and September 2005 (blue plus dark blue). Also a fit with a polynomial is included in the figures. First of all, undulations can be detected from this figure. Secondly, they seem to migrate in the down drift (Southern) direction, around 1000-1300 meters during the 5 years. The wavelength of the very large undulations is around 6 km, and it is observed that the undulation which in year 2000 had its peak in “rør 1” now has been wider, while the other undulation, which was located on the border between “ref 2” and “rør 2” now has been narrower.

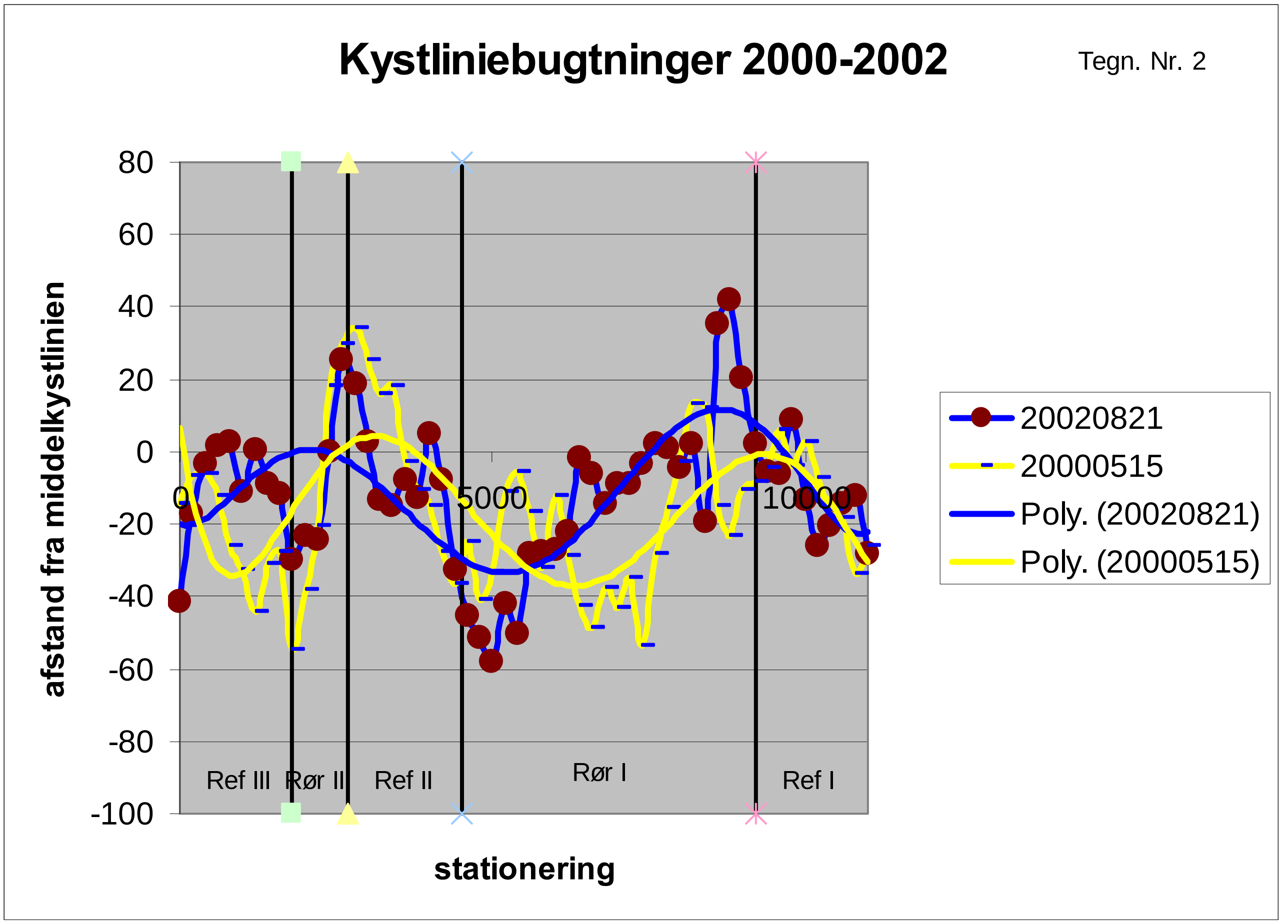

Figure 1a: Variation in beach width from 2000-2002 (Produced by KDI).

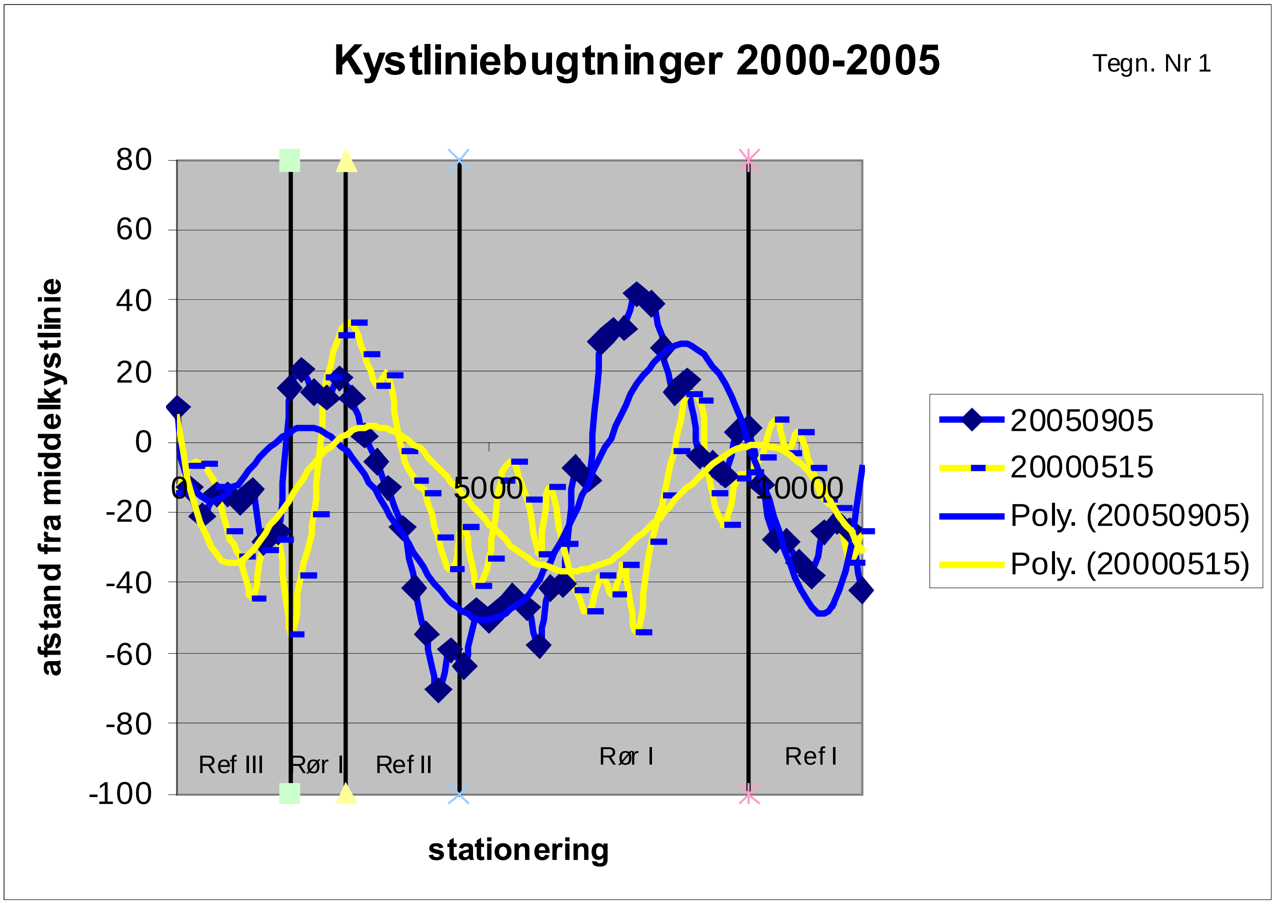

Figure 1b: Variation in beach width from 2000-2005 (Produced by KDI).

The migration of the undulations will cause a rhythmic pattern of erosion and deposition along the coast as sketched in figure 2.

The local variation in sediment transport q along the undulations with the shape h=h (x) is given by

$$\frac{\partial q}{\partial x} = -T\frac{\partial h}{\partial t}$$

where T is the average thickness of the beach.

If we assume the undulations migrate with a steady shape and a migration velocity a, we have

$$h = h(x - at) \text{ and } \frac{\partial h}{\partial t} = -a\frac{\partial h}{\partial x}$$

so

$$\frac{\partial q}{\partial x} = aT\frac{\partial h}{\partial x}$$

If we take a=250 m/year and T=2m, the accretion of the beach will be 100 cbm/year on a location, where the beach widens 10 meter over a 50 meter long distance, which is not unusually on the test site.

The average transport in one “long shore wave” is

$$q = \frac{1}{2}a\Delta WT$$

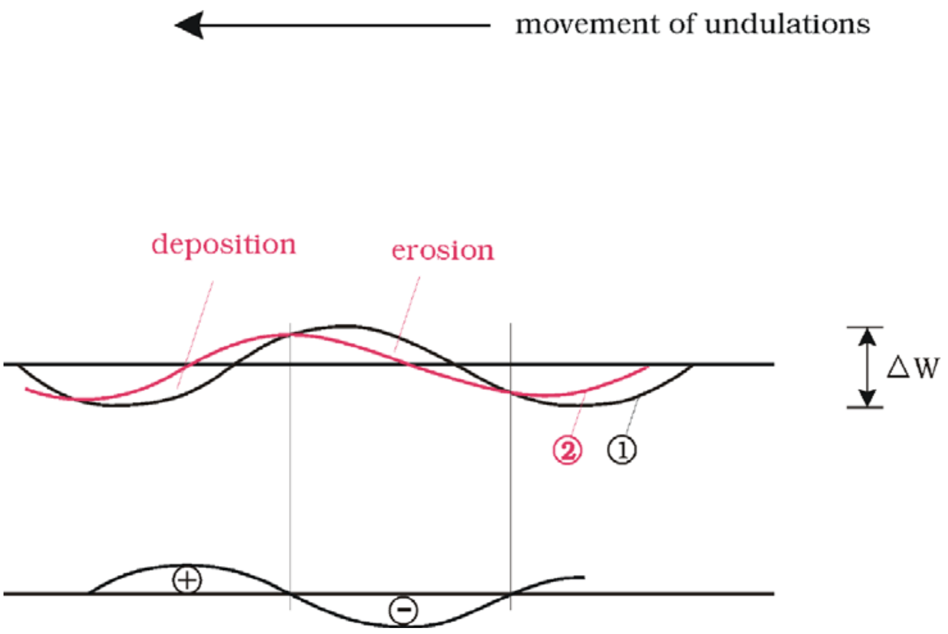

Figure 2: Erosion and deposition pattern caused by migrating undulations.   2 is the undulation to a later time than 1.

where a is the velocity of the undulation and ΔW the difference in the beach width in between where it is widest and narrowest

As an example, let a=250 m/year, T=2 m and ΔW =80 m. This gives an average transport equal 20000cbm/year due to the motion of an undulation.

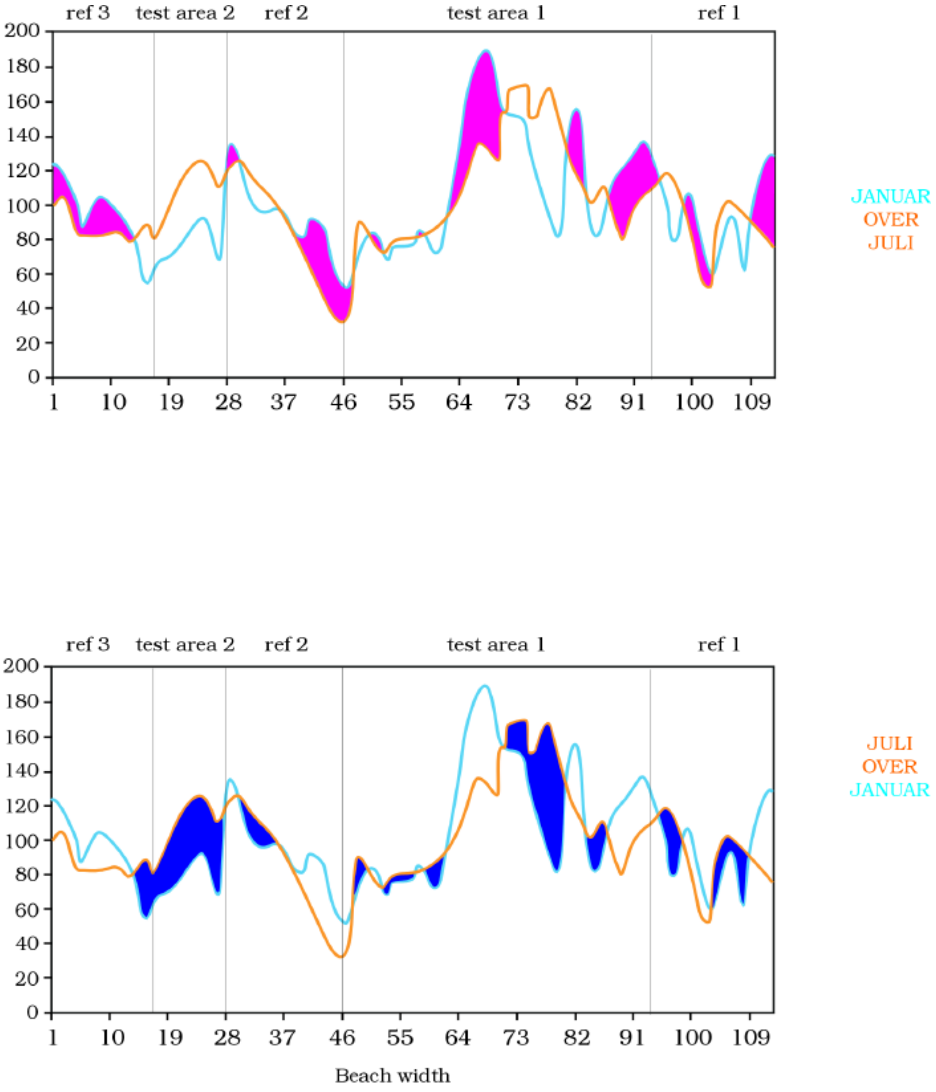

Figure 3A: Variation in beach width from January 2005 to July 2005. Pink: accretion. Blue: erosion.

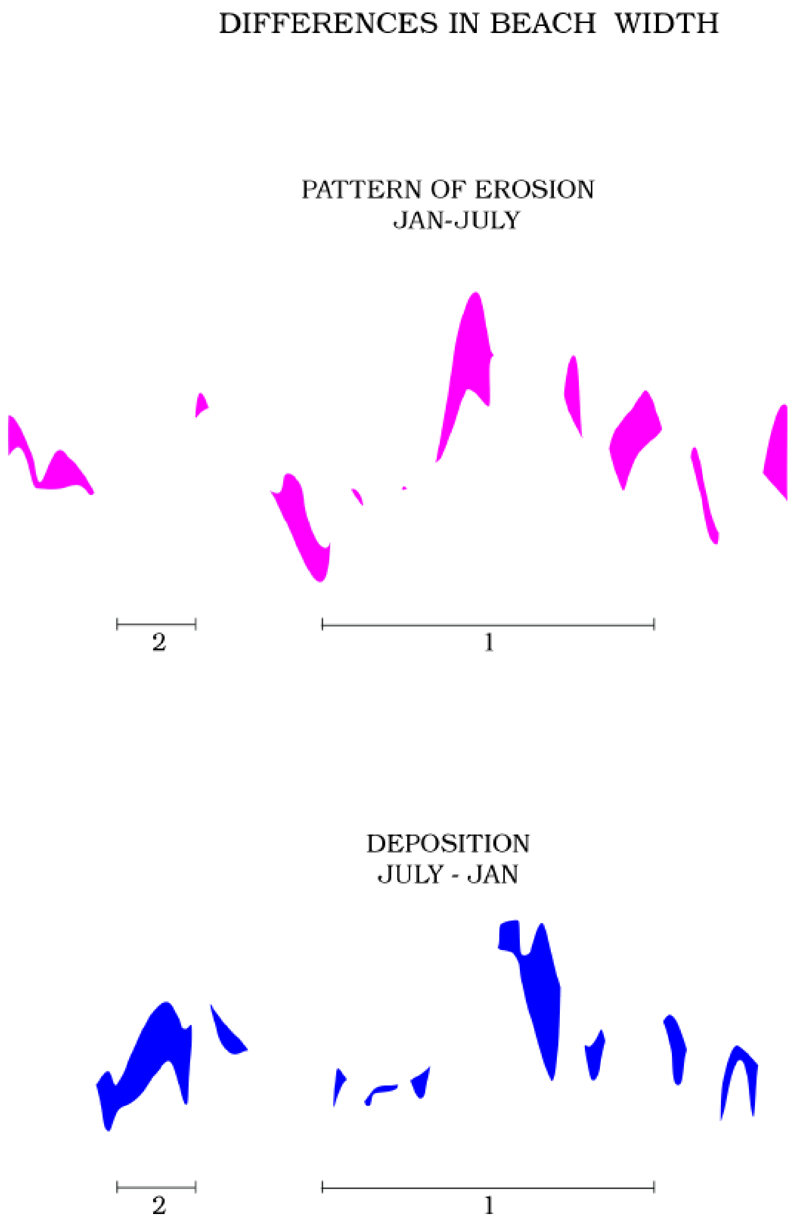

Figure 3B: Like figure 3A, but this picture clearly illustrate that area of erosion is not that different from area of deposition, and everything occur independently of the location of the tubes, at least in the large test area “rør 1”.

A picture like that sketched in figure 2c can to a certain extend be identified in the measurements. Figure 3A and B show the difference in beach width along the site developing during the first 6 months (January to July). The pattern of erosion and deposition is quite patchy due to a varity of different undulations but the tendency is like that sketched in figure 2c.

General observations from satellite-photos June 7 2005 and comparison with measured changes in beach width

Figures 4, 5 and 6 depict the satellite image along the test stretches. From these it is easy to get a visual feeling of the undulations shown in figure 2.

Figures 7 and 8 show the measured changes in the beach width during the first year of the test: Figure 7 shows the changes from January 2005 to April and July 2005 (Second and third “opmåling”), while figure 8 shows the similar change from January 2005 to January 2006.

North of reference 1, figure 4: The beach is quite narrow, and it seems like it has become even narrower during the last 12 months.

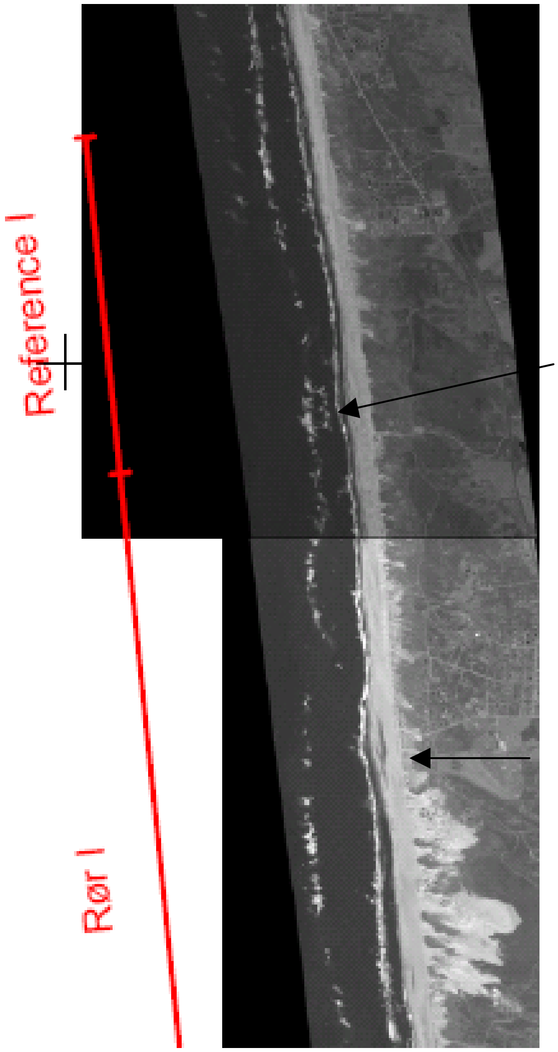

Figure 4: Satellite photo of the Northern part of the test site. The black arrow indicates a pronounced peak in the alongshore undulation.

Reference 1: at least one undulation can be identified in this part of the coast, see figure 4, the upper arrow. The top of this undulation is on its way to move into “Rør-1” during the test period. This will lead to a loss in “ref 1” (see the arrow to the right in figure 8) and a gain in “rør1”. This latter cannot be identified in figure 8.

Rør-1: In addition to the undulation mentioned above, another undulation can be found in this area, se the lower black arrow in figure 4. This undulation does also move, and can be identified at the middle arrow in figure 8. Because it still is contained within the area, only a small flux of sediment is expected to be transferred by the moving undulation from this area to the down drift “reference 2” area.

Reference 2: A very distinct undulation is observed at the border between “reference 2” and”rør-2”, see the white arrow, figures 5 and 8.

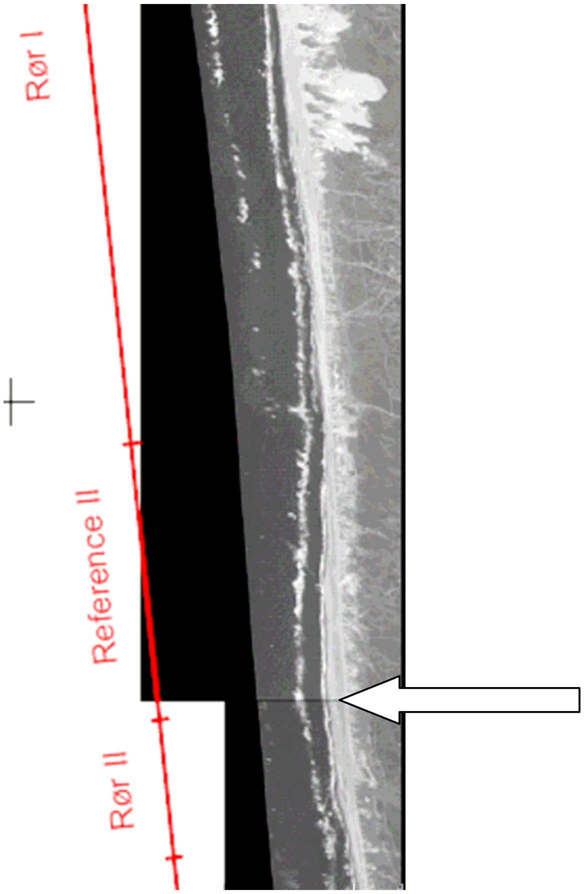

Figure 5: The middle part.

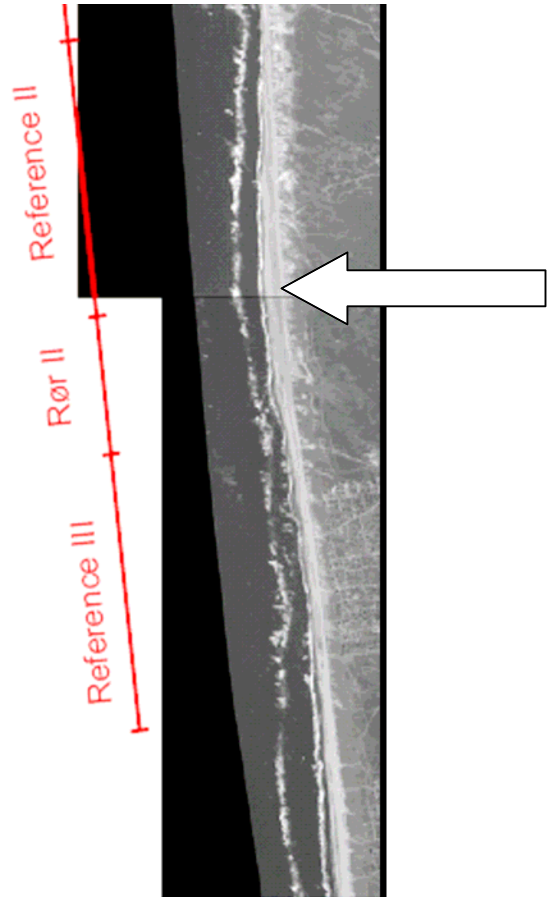

Figure 6: The southern part.

Rør 2 and reference 3: No significant undulations are found here. It seems like the beach widens in the southern direction, indicating the existence of another undulation just south of the test site.

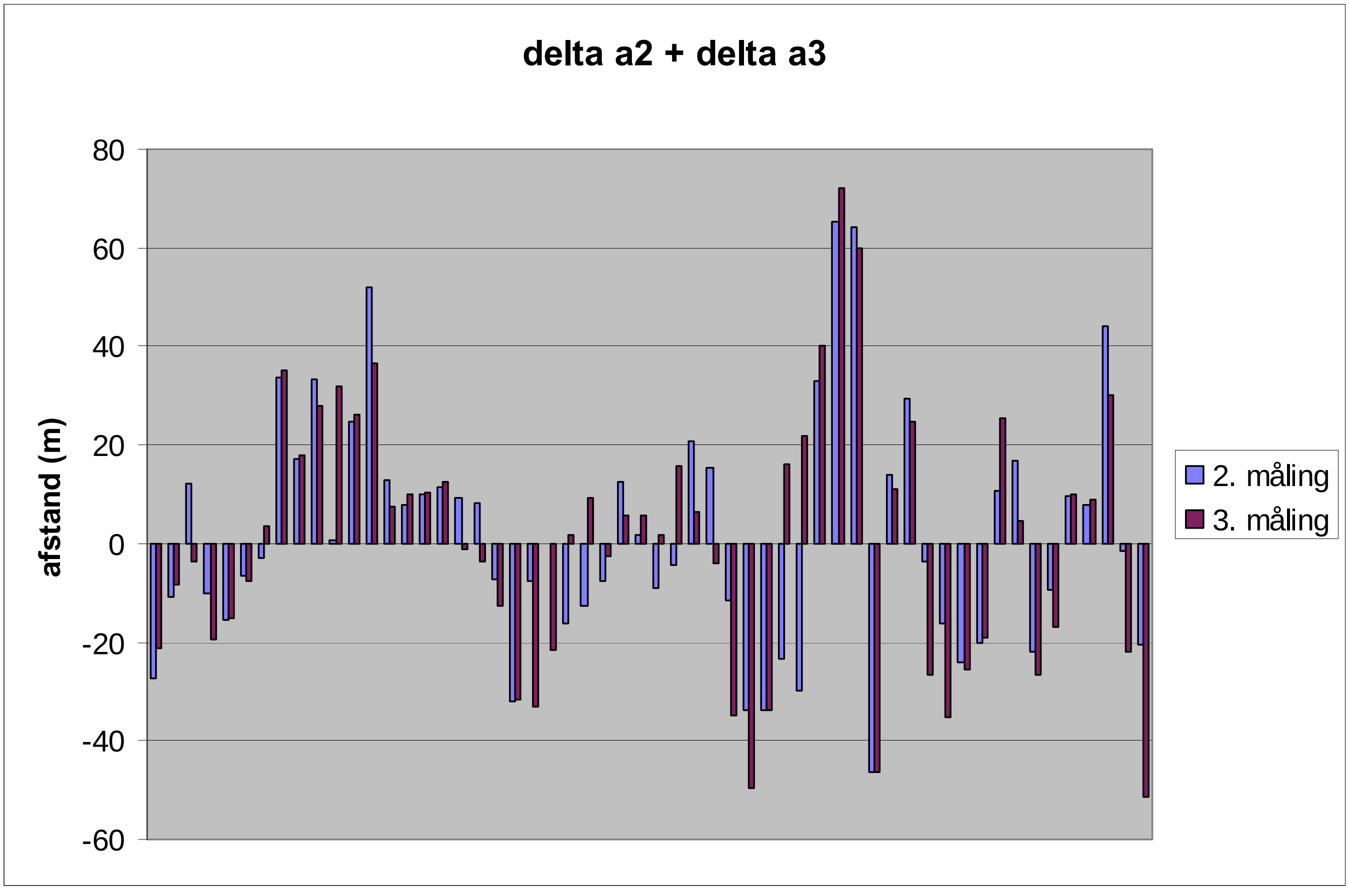

Figure 7 Changes in beach-width from January 2005 to April 2005 (blue) and July 2005 (purple).

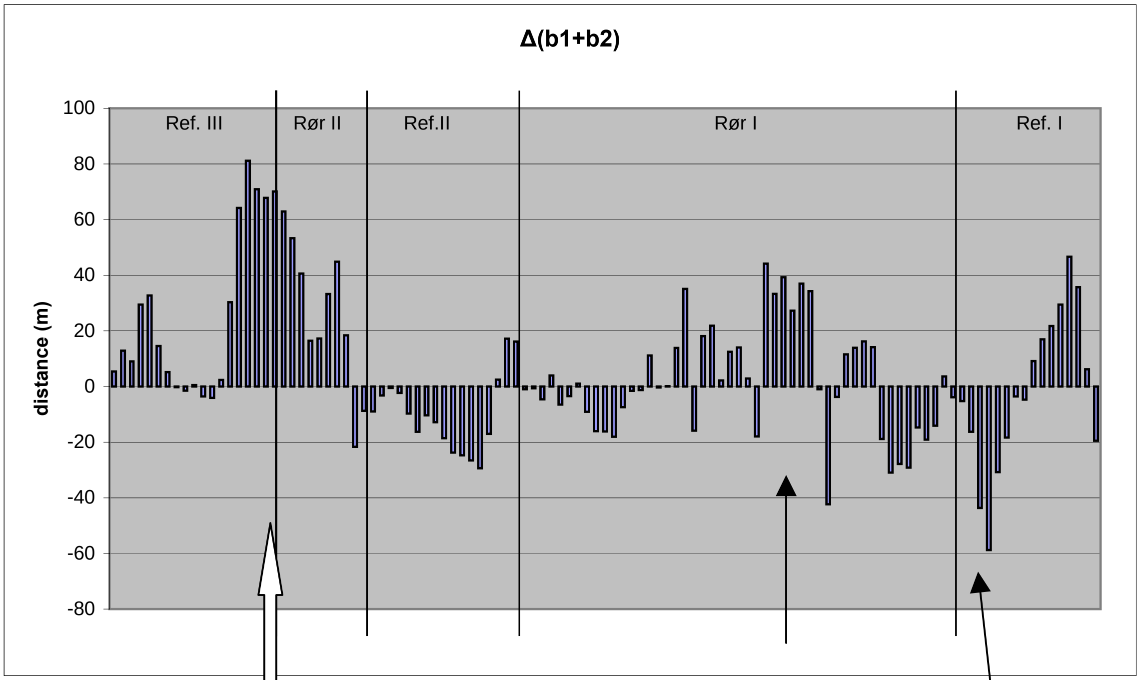

Figure 8. Changes in beach width from January 2005 to January 2006.

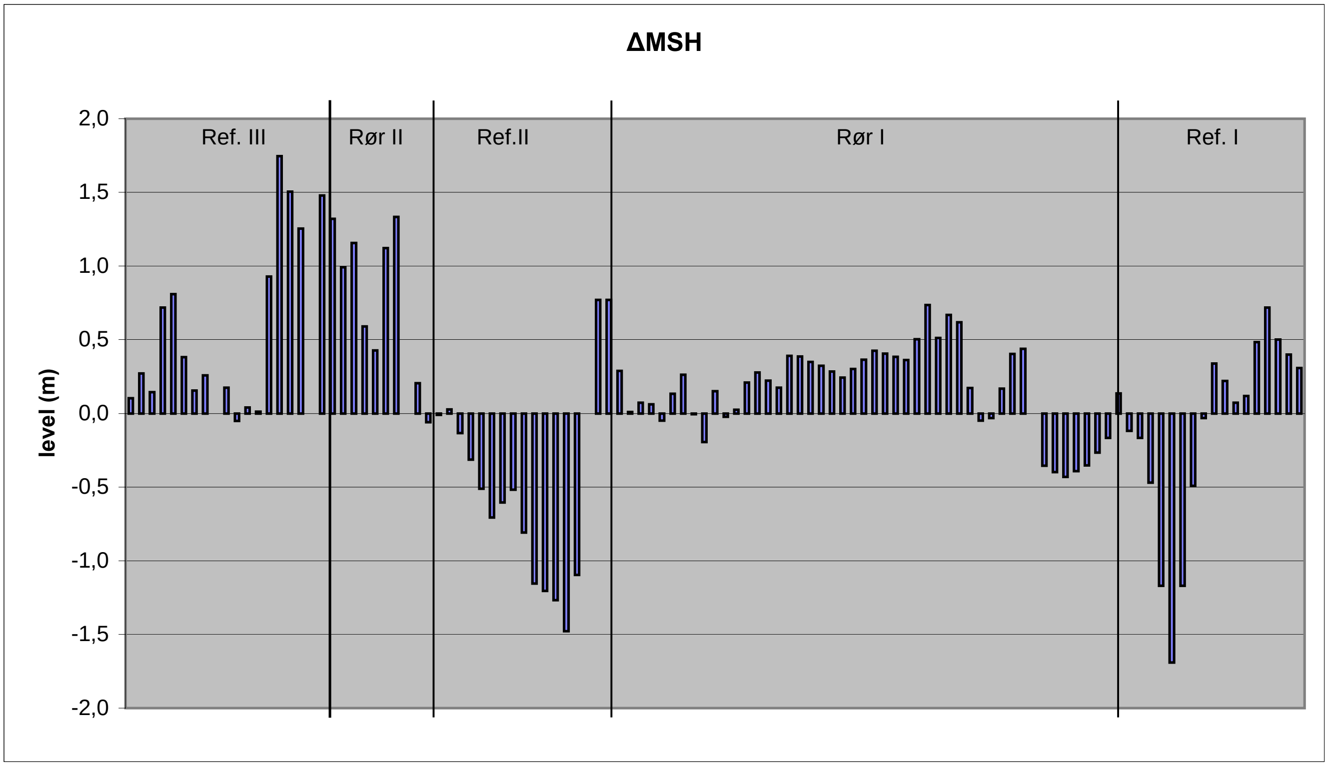

Figure 9: Changes in mean beach level from January 2005 to January 2006. (Volumetric change) The mean level is defined as the average from the dune-foot (level + 4 m) and 100 meter in the seaward direction, independent on whether the actual beach is narrower or wider than 100 meter.