NEARSHORE SAND EXTRACTION AND COASTAL STABILITY

Miriam Moyés I Polo

May 2000, Universitat Politècnica de Catalunya

This thesis has been prepared under the responsibility of the Polytechnical University of Catalunya, Barcelona, Spain. The work has been done during a stay of the author at Delft University of Technology, Delft, Netherlands. Work has been done under the guidance of:

- prof. José. A. Jimenez, UPC Barcelona
- prof. Marcel J.F. Stive, TU Delft

PROJECTE O TESINA D'ESPECIALITAT

Escola Tècnica Superior d'Enginyers de
Camins Canals i Ports de Barcelona

Títol:

Nearshore sand extraction and coastal stability

Autor:

Miriam Moyés i Polo

Tutor:

José A. Jiménez

Especialitat:

Ports, Costes i Enginyeria Oceanogràfica

Data:

Maig 2000

UPC

ACKNOWLEDGMENTS

ABSTRACT

During the second half of this century offshore sand extraction has become commonplace. There has been an important increase of sand demand to use on construction and coastal protection as well as for industrial purposes. On the other hand, the development in the dredging field has helped to the expansion of the process.

Since nearshore dredging is an alteration in a very dynamic environment with important consequences, it is sought to find the minimal depth where a trench can be dredged without affecting the coastal stability and neither considers an excessive restriction. The trivial solution would be to extract in deep bottoms, but the price of these operations (they become more expensive with increasing distance offshore) and also sediment availability makes it unrealistic.

Is in this context of uses and resources where this work is done. First, trying to recognise which are the potential effects of nearshore dredging on coastal stability and next, looking if there is data enough to prove them. Then, after identifying the main effects, a methodology to prevent these impacts is proposed, i.e. some engineering rules to minimise the impacts and select the zones to be dredged.

Theoretically nearshore dredging and the resulting trench can affect the hydrodynamics, the sediment transport and control the coastal behaviour and, as a consequence, the coastal stability, through different mechanisms: the interaction with longshore sediment transport, the beach drawdown, the interception of the onshore sediment transport in the inner shelf, the modification of wave characteristics, the modification of wave's field of velocity and the trench propagation to the coast.

There are different depth criterion to prevent each of these impacts but at the same time, the ones associated to some of them are exceeded by others. Thus, the way to select the depth criterion to prevent all of them must be based on the most restrictive one. On the other hand, it has been observed that other impacts just would verify in particular kind of beaches or coasts (e.g. the interception of onshore transport) or within long time scales (e.g. trench propagation) and although potentially they would be able to affect the coastal stability, they will mainly verify in very specific coastal stretches and, in consequence, they are not very common.

Finally, and after analysing several reported coastal responses, three effects have been identified as the most likely to happen and therefore, some rules are necessary to be sure they will be avoided: beach drawdown, trench propagation and wave's modification.

To avoid the called beach drawdown, trenches should be done seawards of the depth of closure d_l. Therefore it is necessary to find this threshold depth of significant vertical changes, but since it has been demonstrated that it is time and space dependant it is not a trivial task. However, a first assessment of the minimum depth can be done using Hallermeier's equation fed by extremal wave conditions, selected to be representative of a return period according to the life period of the trench, e.g. $T_{return} > 25$ years.

It has been tried to represent trench propagation with a process-based model, but the results have been just qualitative and not quantitative. However, it can exist so to keep sediment transport gradients small and avoid it, trenches should have little depth or be in depths where the transport rates are small. A general criterion could be to dredge seawards of the depth that represents the beginning of significant sediment transport, which at the same time would avoid the interception of the onshore transport in the inner shelf.

The third effect is the modification of wave characteristics. It induces longshore sediment transport gradients that suppose a change in shoreline development as it has been observed in nature. This effect will be significant in long coasts such that their behaviour is conditioned by the longshore sediment transport, but there it will be less important if trenches are shallow and not very wide (less than 400m, Viguier et al., 1984).

INDEX:

1. INTRODUCTION:

During the second half of this century offshore sand extraction has become one of the first sand sources worldwide. There has been an important increase of sand demand to use on construction and coastal protection as well as for industrial purposes (e.g. cast iron and glass industry, Anctil & Ouellet, 1990, see chapter 3.9; tin mining, Vongvisessomjai, 1994, see chapter 3.10). On the other hand, the development in the dredging field has helped to the expansion of the process, in such a way that at present it is possible to dredge in deeper bottoms at reasonable cost and with a high efficiency (e.g. Paris et al. 1995).

The constant increase in the need for aggregate material forced industries to look for other sources of sediment and in some countries as England, France and Japan, sea deposits has been exploited. In other countries as in the U.S.A and Spain, this kind of exploitation is not so generalised (in fact it is forbidden in Spain) although important dredging works have been carried out for beach nourishment, which is the only final destiny allowed for sea extracted sands (see Fig.1.1).
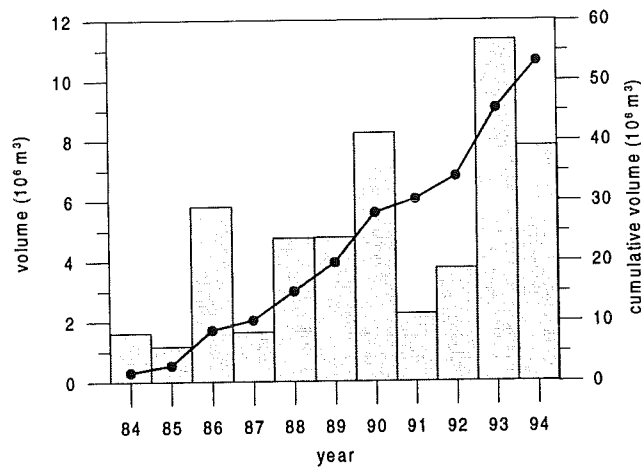
Figure 1.1: Sediment volumes used in beach renourishment by the Dirección General de Costas (Coastal General Direction in Spain) during the years 1984 and 1994 (Jiménez, 1997).

Beach nourishment is one of the most common protection strategies whereas at the same time, it permits to mantain the quality of touristic beaches. These nourishments imply big amounts of sand that because of its availability and quality are usually dredged from the sea bottom. For a beach renourishment not any kind of sand is suitable since

1

beach users require a specific sand quality and also due to work stability reasons the sand must fulfill some quality criteria.

Nowadays dredging is an usual and well documented engineering work, but it is still an alteration in a very dynamic environment with potential important consequences. Extraction of sand is a local change that can be easily propagated to adjacent zones; the extraction can be considered as a withdrawal of sand from the sand budget of the shoreface and adjacent beaches and dunes. Sand extraction leads to a local disturbance of the sea bed topography that, under the combined action of waves and currents, can directly affect the hydrodynamics, sediment transport and morphology of the nearshore zone (see Fig.1.2).
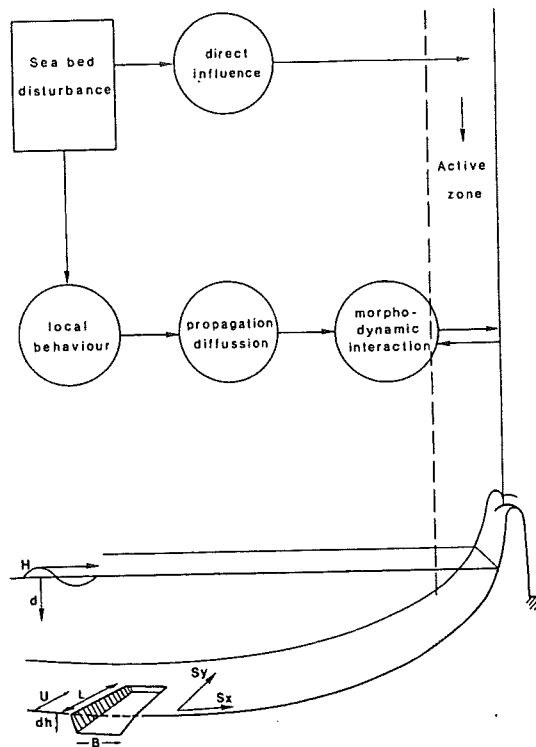
Figure 1.2: Schematisation of relevant processes (H=wave height (m), d=water depth (m), dh=depth of extraction (m), B=width of extraction (m), L=length of extraction (m), U=tidal current (m/s), Sx, Sy= cross-shore and alongshore component of sediment transport) (adapted from van Alphen et al, 1990).

Thus, due to the above mentioned potential influence, there is the willingness to control these actions to avoid the system's degradation. In this context it is possible to use concepts as vulnerability, sustainability, uncertainty, precaution, etc. (e.g. Dovers & Handmer, 1995) or just apply rules as the "Ley de Costas" (1988) in Spain that is the condition that dredging projects must satisfy. According to this law, it is needed to

2

evaluate the impacts on the "dominio marítimo y terrestre", not only where the dredging will take place but also on the zone potentially affected.

In this way a lot of studies have been done to prevent irreversible damages and, in different countries, a threshold depth has been considered as a limit to dredge assuring negligible impacts on the shore. These depths are showed in Table 1.

In this table different values appear due to the different specific hydrodynamic regime of each country as well as according to the precautionary level taken there. These depths usually range between 15 and 40 m.

Location	Depth	Reference
Fukuoka (Japan)	>20 m (until 1981)[1]	Hashida et al. 1992
	>40 m (since 1981)[1]	
Nagasaki (Japan)	>30 m[1]	Hashida et al. 1992
Saga (Japan)	>20 m[1]	Hashida et al. 1992
Genkai sea (Japan)	>35 m[2]	Kojima et al. 1986
Phuket (Thailand)	>15 m[2]	Vongvisessomjai 1994
New Zealand	>25 m[2]	Hilton & Hesp 1996
Holland	>20 m[2,3]	van Alphen et al. 1990
England	>18 m[1,4]	Price et al. 1978
United States	no previous restriction analysis of each case	Hands ("com.pers.")?
Spain	no previous restriction analysis of each case	
General Lab in France (Atlantic)	>21 m[2,5]	Migniot & Viguier 1980

Table 1: Minimal allowed or recommended depths to dredge offshore according to the impacts accepted on the coastal stability ([1] minimal depth required to get the permission; [2] calculated minimal depth to induce negligible erosion; [3] the estimated value was 16 m but it was increased till 20 m because of the calculation uncertainty; [4] detailed analysis is required as well; [5] after laboratory tests local changes on trenches were detected for height waves ($H_{1/10}$) over 9 m) (Jiménez, 1997).

It is sought to find the minimal depth where a trench can be done without affecting the coastal stability and neither considers an excessive restriction. The trivial solution would be to extract in deep bottoms, but the price of these operations (they become more

expensive with increasing distance offshore) and also sediment availability makes it unrealistic.

Is in this context of uses and resources where this work is done. First, trying to recognise which are the potential effects of nearshore dredging on coastal stability and next, looking if there is data enough to prove them. Then, after identifying the main effects, a methodology to prevent these impacts is proposed, i.e. some engineering rules to minimise the impacts and select the zones to be dredged.

In chapter 2, the theoretical or potential impacts of a dredging trench are presented and discussed.

In chapter 3 there is a literature review of several studies where some of these effects, at least the considered more important or the ones that have appeared in site, were reported. The idea is to see if any significant data set to assess the effects of dredging exists.

After this literature review the impacts considered the most likely to occur or at least, or at the same time, the most restrictive, are developed in more detail to know them better and to set boundaries on the future dredging works to avoid or reduce them (see chapter 4).

At the end some final conclusions are presented trying to sum up all the results and answer the objectives.

It is important to say that although nearshore dredging is a global problem here just its influence on the physic system is considered. There are other impacts, as ecological and socio-economical, that should be studied as well to get a whole view of the real effects. Nevertheless this work just look at the topographical shore development.

2. THEORETICAL OR POTENTIAL IMPACTS OF NEARSHORE DREDGING ON COASTAL STABILITY:

Theoretically nearshore dredging and the resulting trench can affect the hydrodynamics, the sediment transport and control the coastal behaviour and, as a consequence, the coastal stability, through different mechanisms.

In what follows the most probable interaction mechanisms are presented and described in general terms. Those considered the most important ones will be further analysed in next chapters (see chapter 4).

2.1-Interaction with the longshore sediment transport:

When the waves break at appreciable angles to the shoreline, the resulting longshore current flows parallel to the shore and is largely confined to the nearshore between the breakers and shoreline. This type of current is particularly significant in causing a longshore sediment transport that can involve hundreds of kilometers of sediment displacement along the coast and that also act to mold the nearshore topography.
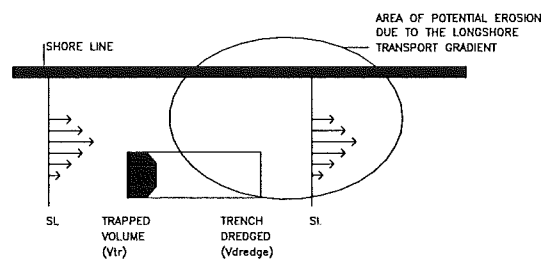
Figure 2.1.1: Infilling effect of a trench for the sediment transported alongshore (adapted from Jiménez, 1997).

If an extraction pit is located close to the coast in the zone where longshore sediment transport occurs, the trench will effectively interact with it. This will lead to a progressive infilling of the trench in the upcoast edge and, simultaneously, a positive gradient in S_l (transport rates increasing in the direction of the current) will be induced downcoast the trench. This gradient will tend to erode the downcoast edge of the trench and the combination of both effects will result in a migration of the trench in the current direction.

5

This alteration of the local sediment balance can also affect to the coastline in the lee of the trench at the downcoast edge, in such a way that the migration of the hole can be accompanied by a local shoreline retreat (Fig.2.1.1).

This impact is easy to avoid if trenches are made seawards of the zone where longshore sediment transport occurs. To do this, it is necessary to know the cross-shore distribution of this transport for the given wave climate. There are many formulations to estimate the local longshore transport and its cross distribution (see e.g. Bodge, 1989), but here a simplified method to obtain this depth is presented.

The active depth for longshore sediment transport (d_{lt}) given by Hanson & Kraus (1988) is used as the depth bounding the width of the zone where the longshore transport takes place, which is given by:

$$d_{lt}=1.6H_{s,b} \qquad\qquad (Eq.2.1)$$

where $H_{s,b}$ is the significant wave height at breaking.

By using this simple relationship it is possible to estimate the d_{lt} associated for each incident wave height of the characteristic wave climate and to obtain its probabilistic distribution. In addition to this, it is also necessary to estimate their contribution to S_l, which can be approximated considering waves coming from the same direction by $H^{2.5}f$, where f is the frequency of occurrence of this kind of waves, and where it has assumed that the transport rate can be estimated by a CERC-like formula.

Afterwards, the cumulative contribution of all the waves is calculated and the minimum depth where the trench should be done to guarantee negligible effects considering this impact is obtained. This depth is estimated after taking a value of the cumulative contribution between 0.95 and 1.00.
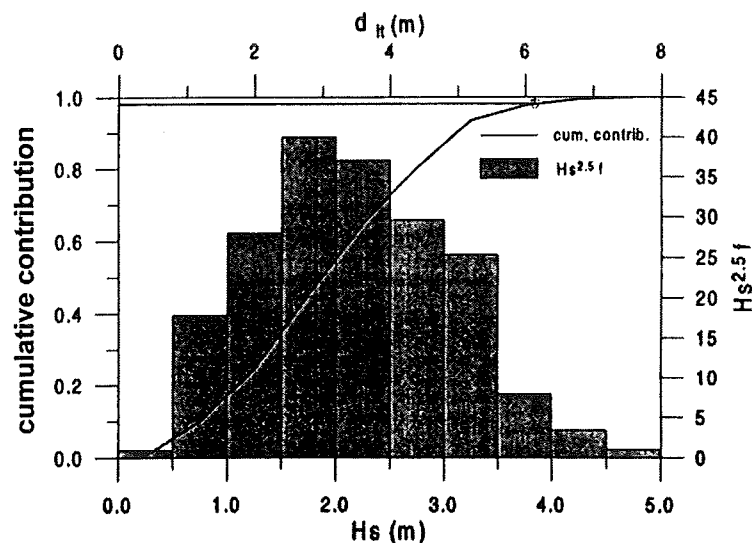

Figure 2.1.2: Height wave contribution to the longshore transport integrated (estimated by using $H^{2.5}f$, where f is the frequency of occurrence of each type) and maximum depth until where the longshore transport is verified, d_{lt}, for each kind of height. The results are

6

presented in contribution of each wave height and its cumulative value. The minimum depth where trenches should be dredged to avoid this effect is also presented using as a criterion the condition that no more than the 2% of the transported sediment will be trapped by them (Jiménez, 1997). (The wave climate used was like the one in the Ebro Delta).

Figure 2.1.2 shows the application of this method for a typical Mediterranean coast (wave data obtained in the Ebro Delta during 2 years), and it gives a value of about 6 m considering a cumulative contribution of 0.98. It has to be stressed that, formally, a more representative long-term wave climate is necessary to preclude the long-term influence of a trench.

Although theoretically this impact can exist, trenches will rarely be done so close to the shore (here it is explicitly excluded the navigation channels which will be the limiting case of trenches close to the coast). There are other effects that force to dig the borrow pits seawards of this zone providing a more restrictive depth criterion than the estimated for the longshore transport. For instance, the critical depth associated to profile changes exceeds $d_{trench,l}$ as it is shown in the next paragraph.

2.2-Beach drawdown:

The impact of a storm on the coast results in shoreline erosion and modification of the profile in such a way that if only the cross-shore transport is considered there is a redistribution of the sediment across the profile. Thus, the sediment eroded from the upper part is deposited offshore, generally in form of submerged bars. As the storm passes and wave conditions become "constructive", i.e. promoting onshore transport, the offshore bar tends to migrate landwards and in the ideal case, the profile will be fully recovered.

If the dredging is executed in a location in the profile where the sediment exchange (between the upper and the lower parts) is verified, there will be an interaction. In this case, the trench will trap part of the sediment eroded from the upper part with a maximum given by the volume of extraction (Fig.2.2). When waves promote onshore transport and recovery processes start, there will be a lack of sediment in the profile equivalent to the trench filling. In the long-term, the profile will tend to "recover" its representative equilibrium shape but with smaller sediment volume and in consequence, this will result in the same profile but landwards of its original position (Fig.2.2). The relative appearance of the new profile with respect to the original one gives the nickname to this effect, beach drawdown.

To avoid this effect trenches should be done in a morphodynamically inactive region (as for this process regards). This impact is one of the potentially most likely to occur when trenches are near the coast and it is analysed in more detail in chapter 4.1.
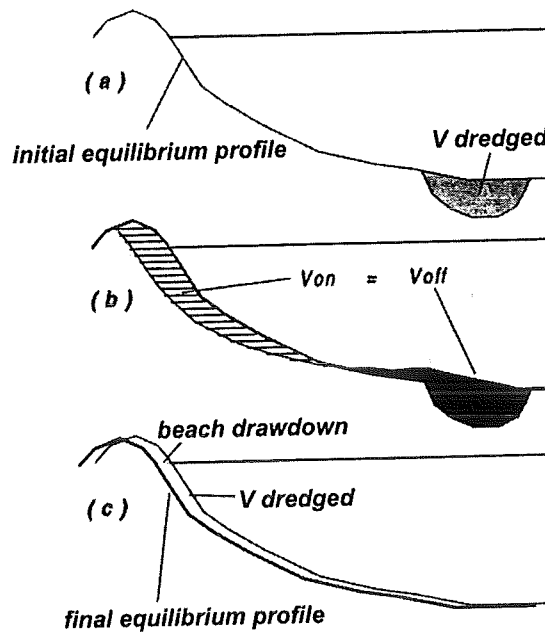

Figure 2.2: Infilling effect for the eroded sediment from the upper part of the profile (Jiménez, 1997).

2.3-Interception of the onshore sediment transport in the inner shelf:

This effect should appear in coasts where a significant net on-shore sediment transport in the inner shelf exists and it is cut by the trench. When onshore sediment transport appears continuously, there is natural nourishment of the beach. Usually, this transport is weak (in comparison with the one verifying in the surfzone), so morphological results are visible at the long-term (from decades to centuries). Its importance has been observed in some beaches as in the south of California where the sediment in the beach has been estimated to be formed by original shelf sand in a range between 15% and 69% (Lee & Osborne, 1995).

If there is a trench, part of the sand moving shorewards can be intercepted. In beaches where this onshore transport is controlling the long-term equilibrium, the effect of the pits will induce erosion at this scale. However, long-term prograding beaches do not need this feeding to maintain equilibrium. In these kind of beaches the accretion will continue but weakly, and the time scale needed to notice any variation in its morphology would be also very long. Whatever the kind of coast is, the effects due to this infilling will appear after decades. In any case it has to be stressed that shorewards of the trench onshore

transport will be also verified so, the modifications in the morphology will be mainly reflected in the submerged profile.

Trenches will be filled by this sediment from the lower shoreface (Fig.2.3.1). If this infilling is compatible with no erosion it will become an "ideal" process; it means there will be a natural "re-filling" of the hole, after some years.
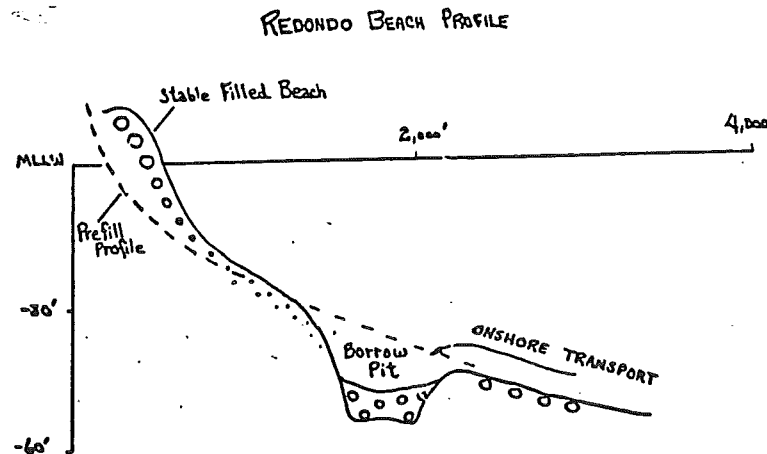

Figure 2.3.1: Interception of the sediment coming from deep waters by the trench at 15 m depth in Redondo Beach, California (Saville, 1981) (Jiménez, 1997).

If this effect has to be avoided, trenches must be placed seawards of the depth that represents the beginning of significant on-shore sediment transport (d_i). This depth should be representative of "normal" conditions and it can be estimated by studying the probability of the initiation of sediment motion for a characteristic wave climate and properties of the sediment.

A simple way to calculate this d_i due to waves action during an average year is through the Hallermeier's equation (1980):

$$d_i = H_{sm}T_s(g/5000D_{50})^{0.5}$$ (Eq.2.3)

where H_{sm} is the averaged significant wave height for the annual distribution, T_s its associated period and D_{50} the medium diameter of the sediment in an approximated depth of $1.5d_l$ (d_l: depth of closure). This equation was derived from physics criterion, i.e. by choosing a critic Froude number for the initiation of motion under wave action, and calibrated by field data (Hallermeier, 1980).

Usually d_i is deeper than d_l, i.e. the depth where sediment motion becomes significant is deeper than the depth of closure (used for beach drawdown) which is not a limit of transport, it is just a limit for vertical changes. However, it is possible to find d_i smaller than d_l because they have been independently derived (e.g.Hands & Allison, 1991).

9

As an example, this criterion is used to determine the depths of significant sediment motion for the wave climate of each of the areas defined in ROM 0.3-91:

Location	d_i	Location	d_i
I (Bilbao)	23.4	VI	9.6
I (Gijón)	19.9	VII (Alicante)	8.7
II	34.4	VII (Valencia)	7.8
III	34.4	VIII (Roses)	11.0
IV (Sevilla)	9.2	VIII (Palamós)	12.0
IV (Cádiz)	11.5	IX	9.6
V (Málaga)	5.1(*)	X (Las Palmas)	17.7
V (Ceuta)	4.9(*)	X (Tenerife)	8.4

Table 2.3.1: Minimum depths where trenches should be done to avoid the interception of the potential onshore transversal transport using Hallermeier criterion (1981) along the Spanish coast, using the wave climate given by ROM 0.3-91 and assuming D_{50} of 0.3 mm. The values marked with (*) are minor than $d_{I,0.137}$, so the minimum depth will be considered as the value of $d_{I,0.137}$ that appears at Table 4.1 (Jiménez, 1996).

Due to the above mentioned inconsistency it seems clear that it would be dangerous to use this criterion and a more "realistic" one should be introduced. Thus, there are others ways to estimate this limit based on predictors of the sediment initiation of motion under oscillatory flux as the Shields criterion (Madsen & Grant, 1976). In this case the coast can be divided in the cross-shore direction as a function of the probability of exceeding the initiation of motion for a characteristic wave climate and the typical sediment of the area.

Both approximations are compared (d_i using Hallermeier and Shields) and represented in Figure 2.3.2. It can be seen that the criterion of Shields gives the probability of exceeding the threshold conditions at different depths which decrease for the deeper part.

Although the probability of excess is of 34.87% at 10 m depth, it does not assure that the mobility of the sediment (net transport) at this depth will be important. As an example, the potential capacity of cross-shore transport at 10 m depth has been estimated to be about 4 m³/m/year by Jiménez (1997). This confirms the idea that an important probability does not imply that necessarily the transport magnitude will be also important.
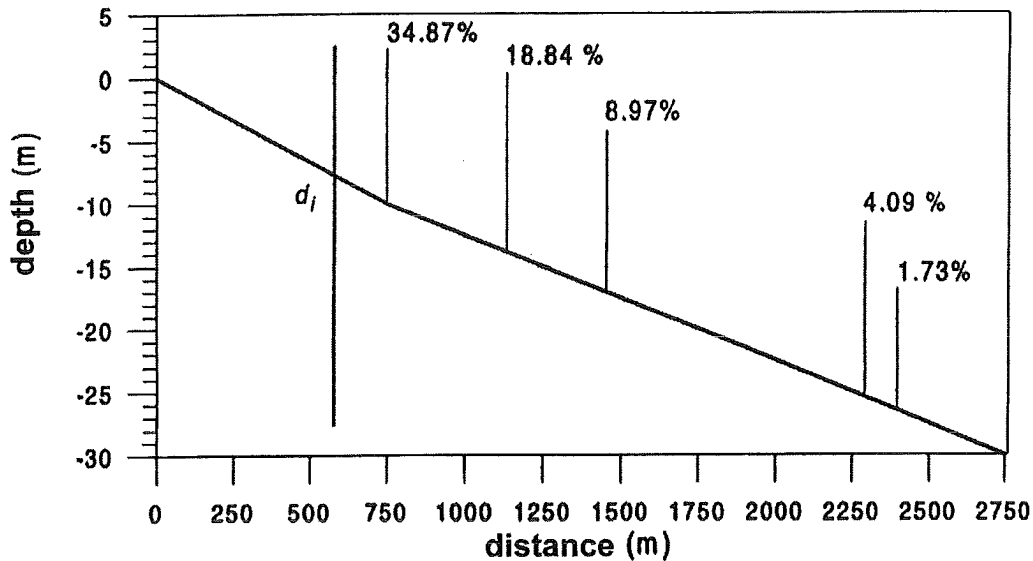
Figure 2.3.2: Determination of the offshore limit for the cross-shore transport by using Hallermeier (1981), d_i, and the criterion of initiation of motion of Shields(?? You said Komar & Miller) in terms of excess probability (Jiménez, 1997). (Sediment $d_{50}=0.2$ mm and wave climate typical of the Ebro Delta).

The depth where sediment transport starts can be characterised by any of the methods explained before, but sand could be extracted landward of it. Then, the hole of the borrow area would catch part of the sediment that is being transported to the shore, but transport is also taking place landwards of it. Between the trench and the shoreline there is an area where the amount of the transported sediment is higher than near d_i. It is then possible to conclude that the extraction of sand shoreward of d_i does not necessarily imply that no sediment arrives at the beach.

2.4-Modification of waves characteristics:

When waves propagate over a different bathymetry because of the presence of a trench, their height and angle of incidence should modify in such a way that the sediment transport pattern in the lee side of the trench can be affected. This implies changes especially in the longshore transport or, more precisely, in its gradients along the coast.

This effect is one of the most important (if not the most) mechanism of interaction between a trench and coastal stability, and it has been identified in real situations as responsible of coastal erosion (the most clear situation of its effect happened in Grand Isle, Louisiana, which is explained in chapter 3.6, Combe & Soileau, 1987). As an example, significant changes in the pattern of alongshore transport of beach material caused by the refraction of waves over the trenches have been found and also analysed theoretically (e.g.Motyka & Willis, 1974). These authors recommended the use of the

deep-water criterion (d/L>0.5) to place the trench and to avoid the modification of wave propagation.

Usually these conditions are not practical because this would mean to extract the sand at deep waters. Consequently, a more detailed study for each particular case is needed. However it is known that there are beaches where this impact can be potentially more important than in others. In long coasts where its evolution is contolled by longshore sediment transport, changes can be easily induced and it is possible to find the formation of tombolos (Horikawa et al., 1977; Combe & Soileau, 1987).

In chapter 4.3 this effect is analysed in depth.

2.5-Modification of wave's field of velocity and trench propagation to the coast:

Waves and currents can experience a modification of the induced velocity field over a trench, with the intensity of the modification depending on the dimension of pits and the characteristics of the local hydrodynamics.

By knowing the variation of wave's parameters (height and direction) due to the trench presence, it is easy to determine the new velocity field. The modification is due to the variation of the wave's height over the hole that leads a gradient of velocity at both ends. To estimate the new field, a first guess can be done by using a wave theory with the modified wave characteristics.

Currents need to be analysed case per case, but their alteration will always depend on the direction of the current in relation with the axis of the trench. In the simple case of a current parallel to the hole, it would appear a reduction of its intensity caused by the increasing of the depth. But this modification would be basically local; i.e. it means that it would disappear rapidly as currents move away from the hole. As an example, Van Alphen et al. (1990) found, in the Dutch coast, that the most important variation was in trenches of 5 m depth in the 14 m isobath and, it was a reduction of 10% of current intensity that disappeared "twice the width of the trench far away".

When the incidence between currents and trenches is oblique the final result depends on this angle of incidence and it makes more difficult the analysis. However a circulation model for the different configurations of the pits can estimate it.

But the most important effect due to this modification of velocities is the possible propagation of the trench towards the coast (see chapter 4.2 and Migniot & Viguier, 1980, in chapter 3.2).

Because of the change in the local hydrodynamics, sediment transport gradient can be induced which should induce a change in the trench morphology. A simple outline of this propagation in the dominant flow's direction is showed in Figure 2.5.
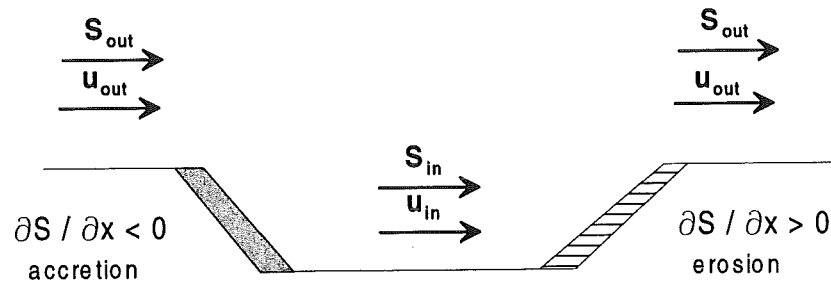
Figure 2.5: Outline of the propagation of the trench in the direction of the flow due to the generation of a gradient in the sediment transport.

3. STUDIES OF REAL CASES: LITERATURE REVIEW ON DREDGING AND COASTAL STABILITY

In this chapter, reported coastal responses to nearshore dredging as well as the evolution of the trench are presented and commented. Each case is presented in a similar way (as far as the published data permits it): location, objective of the analysis, dredging works, type of analysis and data used, climate conditions, results and conclusion, comments.

3.1- South of England (Price et al, 1978):

- Location: in the south of England, between the Isle of Wight and Brighton, and in front of Harwich (Fig.3.1).

- Objective: to find out how dredging might affect the coastline and, from there, to select a depth that assured negligible impacts.
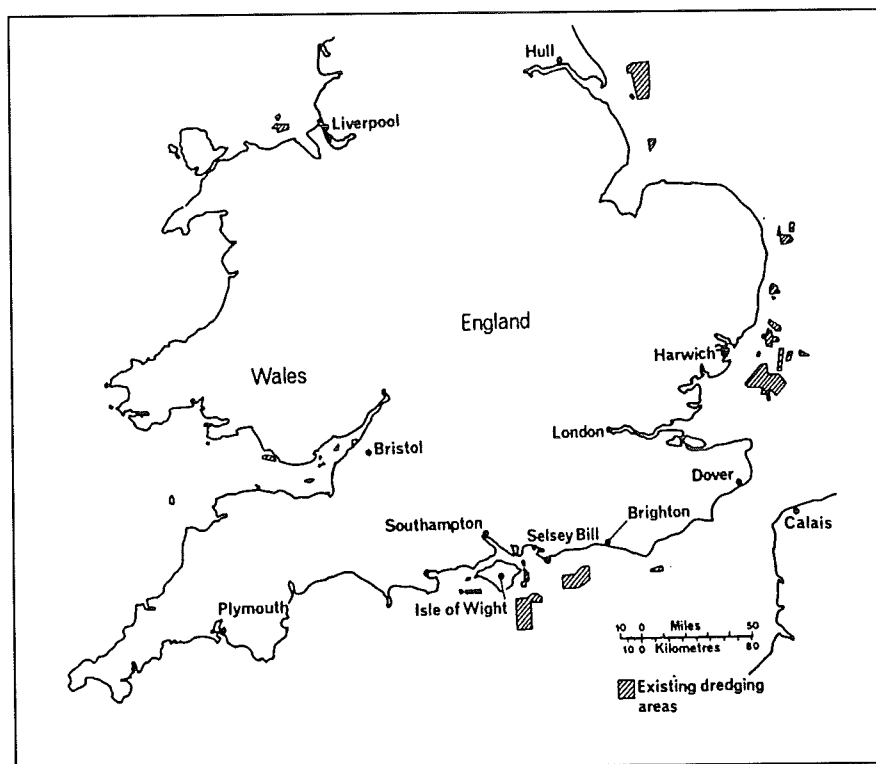

Figure 3.1: Map showing location of areas dredged (Price et al., 1978).

- Dredging works: it is basically a theoretical study, but based on borrow areas in the south of England (Fig.3.1), where dredging works were increasing because of the demand's increasing and the amount of material there was.

- Wave climate: there is no data about it.

14

- Kind of analysed data: they used a lot of information from other papers and investigations, bibliographic data. However, they also used divers to examine the sea floor; radioactive tracers to investigate the movement of shingle; a numerical model of shoreline changes due to wave refraction over dredged areas; and theoretical approaches to calculate the shear stress at the sea bed due to the combined action of waves and tidal currents.

- Results and conclusions:

To explain and study the dredging licensing procedure, they tried to assess the effects of sand extraction to the shoreline and identify values or criterions that let avoid them. Finally they conclude:

1- Two criterions can be used to know the limit for trenches to avoid the beach drawdown (chapter 4.1). The limit for onshore-offshore movement off the south coast of England is 10 m (Inman & Rusnak, 1956). The second was taken, adding other considerations a part of coastal changes, from Watts (1963): it was recommended an offshore distance of 600 m. Although the existence of both criterion, they pointed out that usually other considerations over-ride them.

2- Shingle would not move in depths greater than 18 m if just wave action were considered. If also tidal currents were included, this criterion was of 22 m. It supposed that if a trench were dredged shorewards of these depths, shingle would have been trapped by it.

3- Trenches would not be permitted where banks exist. Just under special conditions, as when the rate of accretion at the coastline is so high that its possible reduction would not damage the shoreline stability, it would be accepted.

4- The effects of wave refraction were insignificant when dredging took place in water depths greater than 14 m (Motyka & Willis, 1974) according to the beach mathematical model developed by HRS.

- Comments:

This paper can be considered as a classic because it was one of the first studies where the likely dredging impacts were analysed. They realised about the necessity of some rules to avoid important changes of the coast, changes that might break coastal stability.

It explains all the potential impacts (chapter 2) and also gives some criterion. Besides the effects analysed in the previous chapter, they looked at the areas with sandbanks (3rd conclusion). However, it does not contribute with any real case. It is just a theoretical study.

3.2- France (Migniot & Viguier, 1980):

- Location: in the *Laboratorie Central d'Hydraulique de France* (L.C.H.F.) they worked with data from the Bay of Biscay.

- Objective: to predict the evolution of sand extraction dredged between shore and 25 m isobath and its influence in the profile evolution.

- Dredging works: two kinds of analyses were carried out in the laboratory, in the flume and in the wave tank. In both of them trenches represented real sand extraction pits with trapezoidal profiles and parallel to the shoreline. In the flume they were 5 m depth and had wide of 80 and 140 m. In the wave tank trenches had 6 m of depth, 200 m of width and were 800 m long (in the flume trenches extended in all the flume's width). Actually these were not the sizes of the trenches done in the lab; there, reduced models were prepared so everything was affected by scale factors (Table 3.2).

	Parameters	Symbols	Values
Geometry scale:	In plan:	$1/m$	1/200
	In height:	$1/n$	1/75
Hydraulic scales:	Longitude	$1/m$	1/200
	Height	$1/n$	1/75
	Horizontal surfaces	$1/m^2$	1/40000
	Vertical surfaces	$1/mn$	1/15000
	Volumes	$1/m^2n$	$1/3.10^6$
	Hydraulic times	$n^{1/2}/m$	1/23,1
	Horizontal velocity	$1/n^{1/2}$	1/8,66
	Wave: Period	$1/n^{1/2}$	1/8,66
	Height	$1/n$	1/75
	Longitude	$1/n$	1/75
	Current	$(m/n)^{1/3}/n^{1/2}$	1/6,25
Sediment:	$D_{model}=1,5D_{nature}$		

Table 3.2: Scales adapted in the reduced models.

- Wave climate: the tests were done using the waves of the Bay of Biscay, the most likely to appear, but reduced by scale factors (Table 3.2). For the flume it was used a cycle of increasing waves that wanted to represent a year (see chapter 4.2 and Fig.4.2.4) although it was not a real wave climate. These waves were frontal waves and acted with sinusoidal tidal. The general currents and either the tidal current were not represented. In

16

the wave tank the same kind of waves were used but in a different cycle; highest waves acted the last ones too, but during a shorter period of time than the others. Firstly trenches were studied just for frontal waves during 2 years, afterwards for oblique waves during also 2 cycles and finally the deepest trench (-21m depth) was subjected to a cycle of 7 years.

- Kind of analysed data: at the beginning of the report some theoretical and field results (using radioactive tracers) were presented. After the laboratory reduced models in the flume and the wave tank, it was possible to check visually the local evolution of trenches and the general development of the profiles until the shoreline.

- Results and conclusions:

After studying the evolution of sea's bottom and the variations of the sediment's size along the time directly, theoretically and by radio-active tracers, it was concluded that for frontal waves of 7 m of maximum height, sediments movements could start between –20 and –30 m depth. But these movements became important shorewards 15 m isobath. But to really understand the coastal behaviour after a dredging, the laboratory tests were executed.
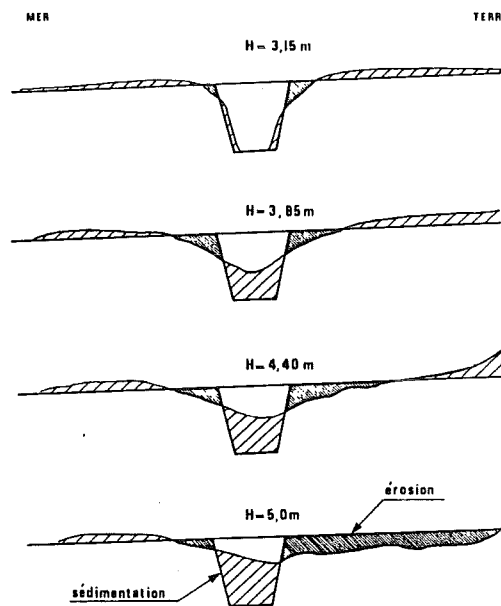

Figure 3.2.1: Example of bottom evolution in the flume after what supposed one-month of action of these different wave heights. Heights are $H_{1/10}$ and the trench is at 12 m depth (Migniot & Viguier, 1980).

The flume studies were basically qualitative. They were 2-D but they let see the infilling of trenches as well as define the wave critical height that supposed the beginning of this process: $H_c=0.25d_f$. The 4 studies made (each one with one trench on different depths of

17

–7 m, -12 m, -15 m and –20 m) showed the same 8 steps of changes on the trench geometry and the same sediment dynamic, with the difference that these steps were reached for different waves depending on each pit. Thus, deposits on the trenches appeared always by softening of the landward trench slope and eroding in shallow bottoms and consequently, on the beach (Fig.3.2.1).
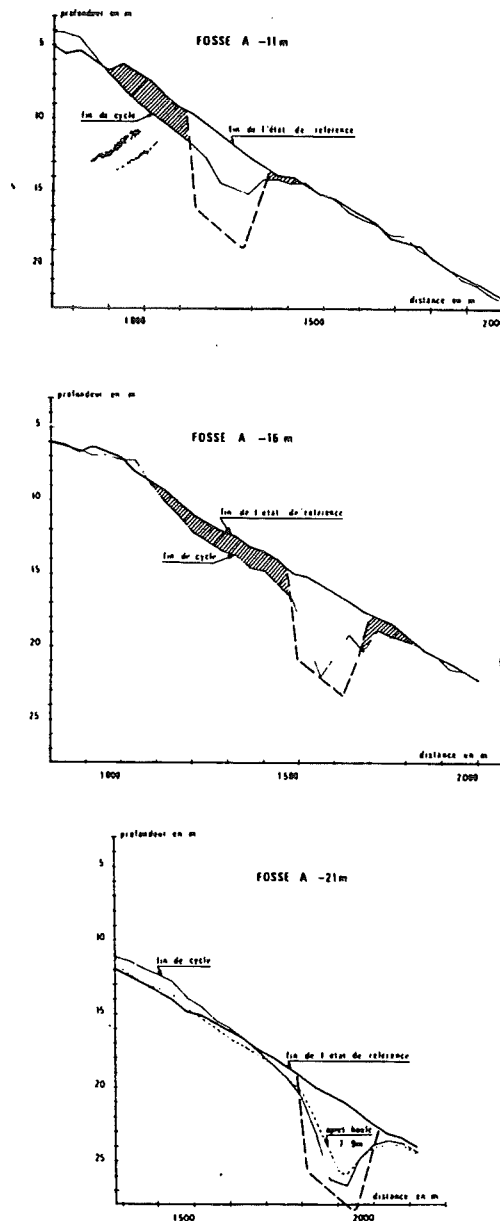

Figure 3.2.2: Evolution of trenches at different depths obtained in the wave tank after 2 annual cycles of frontal wave action (Migniot & Viguier, 1980).

With the 3-D tests in the wave tank, the results about the evolution of the pits were similar (Fig.3.2.2). For trenches on 6 and 11 m isobath a rapidly erosion was observed, while for trenches at –16 m depth the action over the shore was slow and almost negligible for trenches at –21 m. As it can be seen in Figure 3.2.2, trench at –11 m and

18

after 2 annual cycles, was filled by the sand of the 5 m and 8 m isobath without any significant change on the shoreline. As trenches were dug further from the coast than this depth, the sediment that filled them came from deeper isobath (for trench at −16 m the sand was from between the 8 and 11 m isobath). For trenches at −21 m depth there were needed waves higher than 7 and 9 m ($H_{1/10}$) to have some sediment deposition in the hole.

The global development of the coast could be extended thanks to the 3-D results. It was observed less erosion in the lee of the trenches while in the ends it was more important due to the refraction of waves.

When oblique waves were used, it was not seen any important difference in the filling of the holes, just that the seaward trench slope was gentler (Fig.3.2.3).
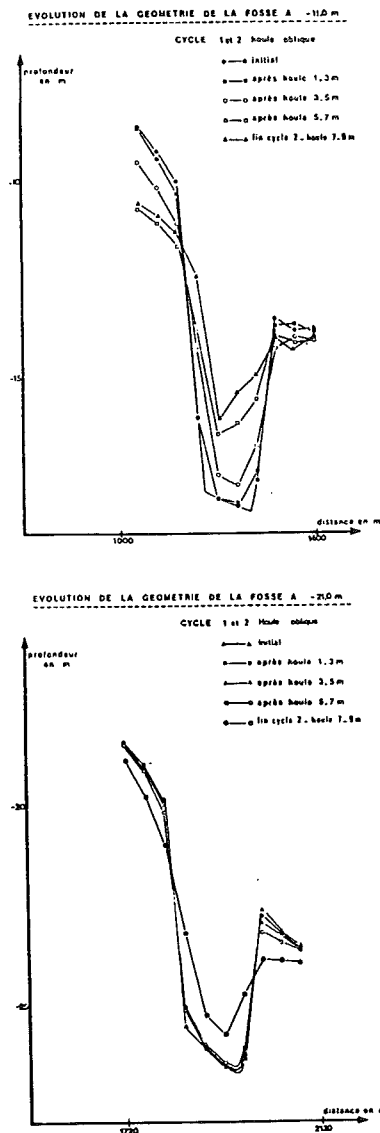
Figure 3.2.3: Evolution of trenches at different depths obtained in the wave tank after 2 annual cycles of oblique wave action (the angle of incidence was 15° from the shoreline) (Migniot & Viguier, 1980).

The filling rate of the dredging trenches was determined as:

$$V \text{ in } m^3/m/day = (H_{1/10}-H_c)^{1.5} \qquad\qquad (Eq.3.2)$$

Finally it was concluded that if the dredging trench was further than –21 m under low water level, its action on surroundings bottoms and on the shoreline was practically non-existent as long as the maximum wave height did not go beyond 9 m ($H_{1/10}$). This was based on the results obtained after waves acting for 7 years over a trench at this depth. It was observed that with storms with waves of 9 m the rate of deposition of sediment was not more than about 5-6%, but it increased to the 17% if a storm with waves of 13 m during 7 days/year was included.

- Comments:

It is important to remind that although they were trying to better understand real behaviours, all was subjected to scale factors. Moreover, they were laboratory studies for a specific kind of waves that, specially in the case of flume tests, were not used as they act during a mean year, so the results can not be considered exactly the real ones. However, they were qualitatively interesting and helped forecast filling conditions of dredging trenches and to fix limit depths for their location with respect to local oceanographical conditions so that consequences on the shoreline were negligible.

These studies might be considered a good first approximation since the results obtained agreed with the theoretical and field ones. At least it seems that the reduced models were well done.

However, as the authors said, to generalise the results and to give the license to dredge, it would be preferably to complete them examining the possible influence of currents (both, general and tidal) added to wave action (see next chapter, 3.3) and consider the shape of trenches, their execution, the material that form the bottom and the morphology.

3.3- France (Viguier et al., 1984):

- Location: as in the previous report (3.2) they worked in a laboratory (L.C.H.F.) with the wave climate of the Bay of Biscay.

- Objective: to know more about the evolution of dredged trenches located at different depths landward from the 25 m isobath, considering or not currents and different shapes of these extraction pits.

- Dredging works: in this case they studied the behaviour for 3 different types of trenches parallels to the shoreline but more or less wide and long. See Figure 3.3.1

But again these studies were done in a wave tank so all the sizes were affected by scale factors.
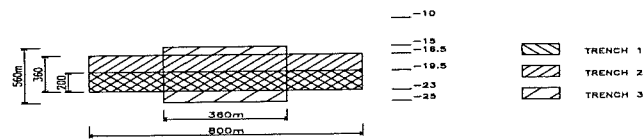
Figure 3.3.1: Outline of the three types of trenches, with their shape and magnitudes.

- <u>Wave climate</u>: tests were carried out using wave climates, tidal and in some case currents, of the Bay of Biscay, during 8 annual cycles. For the study of the first trench it was included a storm with an $H_{1/10}$ of 9 m that lasted 7 days in the 3rd year. For the other 2 trenches two storms were included: one with $H_{1/10}$ of 9 m during 2.5 days in 2 of the years of the cycle (the 4th and the 8th), and the other one with $H_{1/10}$ of 11 m during 1.5 days in other 2 years (the 2nd and the 6th). The incidence of wave was between + or − 25° in such a way that the resultant alongshore transport was non-existent.

- <u>Kind of analysed data</u>: theoretical and some field data were analysed as well as the laboratory studies (in a flume and in a wave tank).

- <u>Results and conclusions</u>:

The previous theoretical analysis they made were useful to realise that dredging should be far from the surfzone and about the indirect effect on the coast because of the refraction of waves. Therefore, they focused the laboratory test in these subjects.

The zone of direct influence of a trench, erosion around it, was about 200-250 m in 8 years and no changes were observed landward the 7 m isobath. Effects of wave refraction over trenches were observed when these ones were wider than 400 m, as the trench number 3. There was more erosion in both ends of the trench and more sedimentation in the lee of it. To predict if this effect would appear, they defined a new parameter, A_r, which represents the relative progress of wave:

$$A_r = 100X(1/L_o - 1/L) \tag{Eq.3.3}$$

where X is the width of the trench (cross-shore longitude), L_o the wave longitude on the depth of the trench d_f and L the wave longitude on the centre of the trench (d_f+the height of the hole). When this parameter was minor than 20-25% there was not any effect on the shore due to refraction.

On the other hand, the filling of trenches was more important for the narrowest trench. The infilling always started by the extremes and because of the effect of their slopes and

21

the gravity, it was more important in the landside. As time went by, the infilling was decreasing and the slopes were getting gentler until they achieve their equilibrium.

Thus, it was clear that the shape of trenches influenced the coastal behaviour.

A part of confirming some relations found by Migniot & Viguier (1980), they also analysed the development when there were currents too. If currents were lower than 0.5 m/s the same results as without currents were obtained. But with highest currents there was an asymmetrical sedimentation in the pit (more sedimentation on the slope from where the current came) and also a tendency to move in the currents direction (Fig.3.3.2).
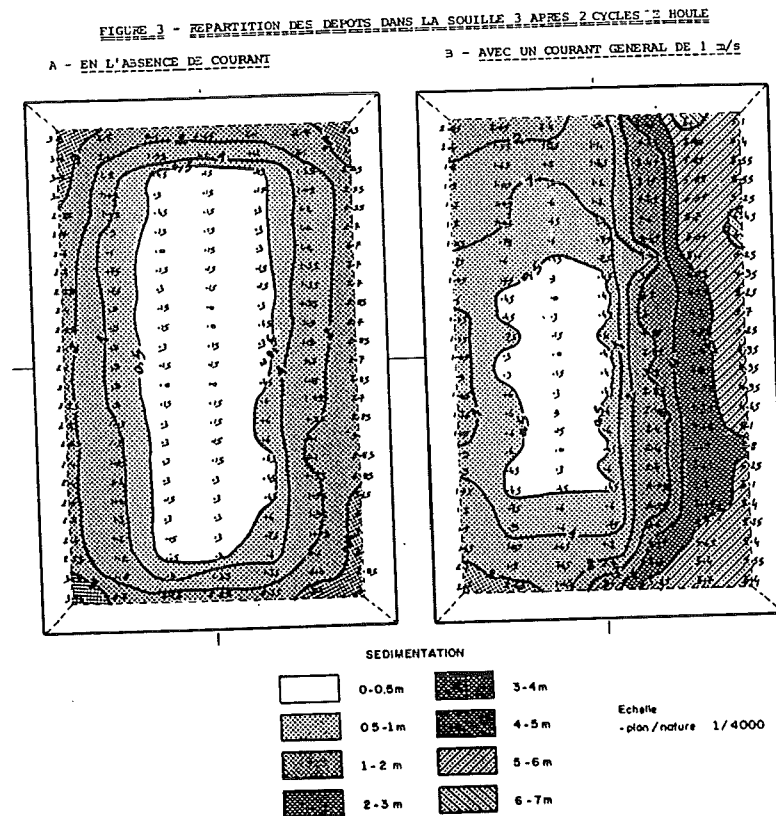

Figure 3.3.2: Distribution of the sediments inside the trench number 3 after 2 cycles of wave action; (a) without currents, (b) with a general current of 1 m/s (Viguier et al, 1984).

- Comments:

These studies can be considered the next step in the studies of Migniot & Viguier (previous chapter 3.2). Migniot & Viguier had already pointed out that more considerations should be taken into account as the presence of currents. In these last studies Viguier et al. keep on working with flume and wave tank so they also worked with scale factors that can introduce errors or deviations from what really could happen. The values they found can not be exactly these ones, although they worked with the "real" wave climate of the Bay of Biscay, and they can be taken just as a first approximation. However, as it was

22

said about the Migniot & Viguier results, they are useful to predict behaviours and reactions at least qualitatively.

About the different shapes of trenches they analyse, it is interesting to remark that while there will be less impact on the shoreline due to wave refraction if trenches are narrower, they will be more filled than wider trenches.

3.4- Genkai Sea, Japan (Kojima et al., 1986):

- Location: in the Genkai Sea, in the northern part of the Kyushu Island and between the Onga river at the east and Hakata Bay at the west. Pocket bay-shaped beaches separated by headlands, form the part of the coast studied (Fig.3.4.1).

- Objective: to assess the relation between beach erosion and offshore dredging and to propose guidelines for offshore sand extraction.

- Dredging works: 8 different borrow areas along this part of the coast (Fig.3.4.1). Most of them were dredged between depths of 15 and 20 m, but H-6 was extracted at 25 m depth.
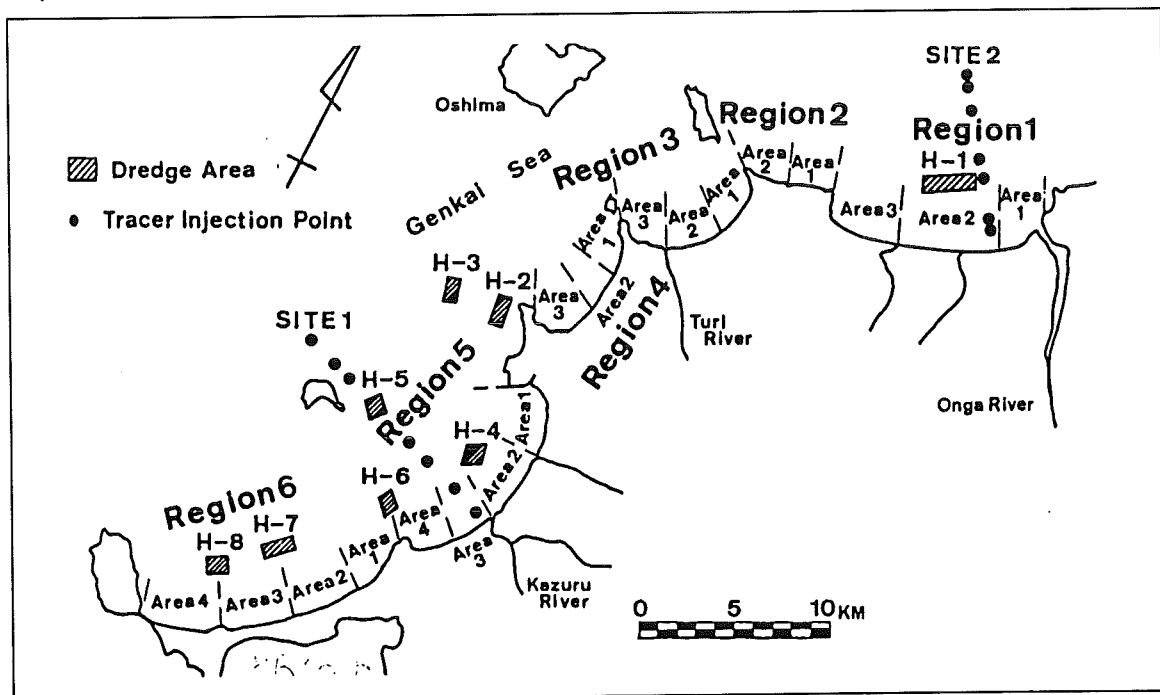

Figure 3.4.1: Locations of areas, dredging sites and fluorescent tracer injection points (Kojima et al., 1986).

- Wave climate: the prevailing high-energy waves in the Genkai Sea are from northwest during winter months. In offshore conditions: wave height of 4.58 m and the associated wave period of 9.20 sec.

- Kind of analysed data: meteorological surveys, offshore wave data, allowed volume of sand extraction, aerial photographs to determine historical shoreline changes,

hydrographic surveys to obtain profile changes and fluorescent tracer studies and sea bed level measurements to acquire data on sediment movement.

-Results and conclusions:

They started trying to find out if which they thought were the possible causes of significant shoreline changes really were.

Through historical studies they concluded that the cause most likely to be was the exceptional intensity of wave impact. The marked beach erosion that can be seen in Figure 3.4.2 between 1947 and 1961, agreed with a remarkably high frequency of storm winds during the same period of time.
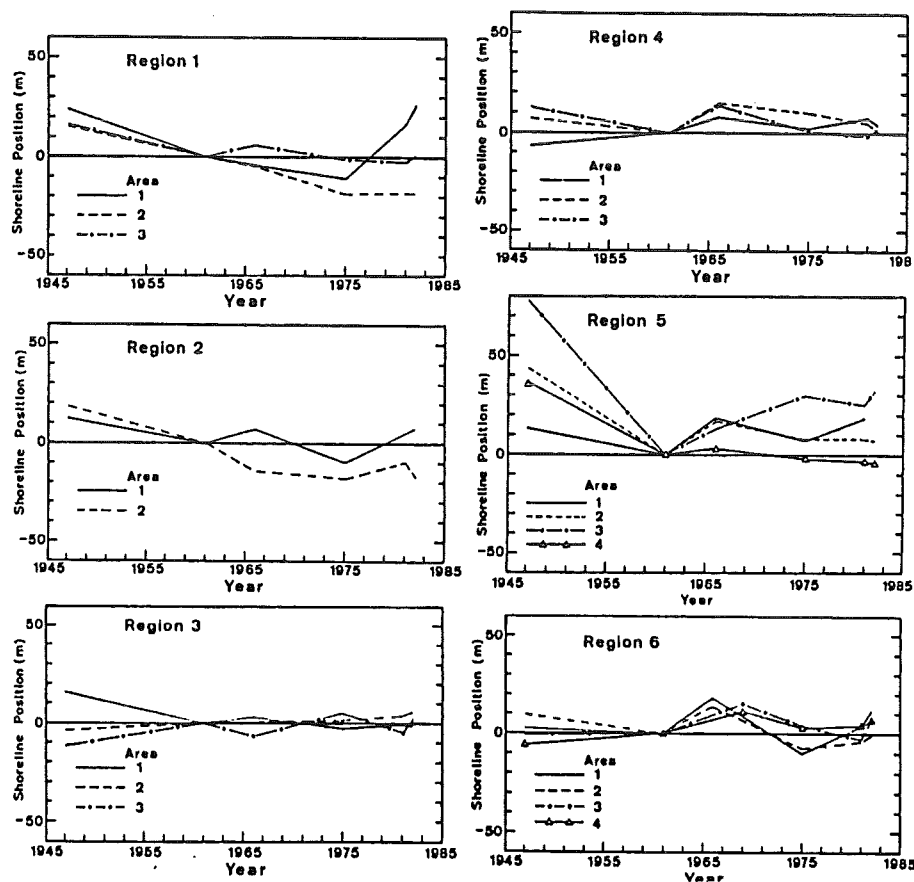
Figure 3.4.2: Shoreline changes for each area versus time (Kojima et al., 1986).

Then they looked at the offshore dredging and at the construction of coastal structures. There was not any data that let them conclude that any construction had contributed to coastline changes.

Comparing shoreline changes and dredged volumes of sand, they suggested dredging had had an erosive effect (Fig.3.4.3).

Evolution of the profile of trenches yielded to assure that the threshold depth to avoid significant profile changes was 30 m. They compared this value to the theoretical one

24

obtained from equations which describe the initiation of sand motion and finally concluded that to avoid drastic beach profile changes trenches should not be landwards of 35m.
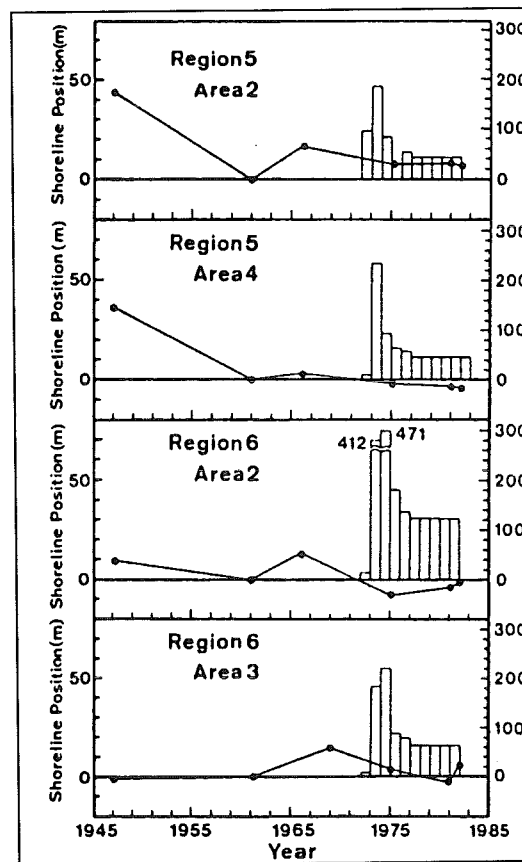

Figure 3.4.3: Relation between shoreline changes and dredged volumes along time (Kojima et al., 1986).

- Comments:

Although they recognised that the available data was not enough to establish a direct cause-effect relation between offshore mining and beach stability, their affirmation on the influence of dredging need some comments. Looking at figure 3.4.3, only the erosion in area 2 of region 6 seems that might be directly associated with dredging works. Before the beginning of dredging in both regions there already was an erosive tend. Moreover, during the last years of constant extraction the shoreline remained stable. The same behaviour that existed in regions where no extraction was done (Fig.3.4.2) can be observed in region 5 and 6. Just in region 1 a different behaviour can be observed (important accretion), that was considered to be the effect of the construction of 6 detached breakwaters in 1979.

Dredging influence might exist but it is not easy to recognise due to the natural impact of the observed number of storms during the study period.

25

On the other hand the profile's evolution of trenches presented is not useful to identify which sediment transport is refilling them. They considered that holes shorewards of 35 m depth would interrupt the beach littoral system trapping the sediment that would travel either on-offshore or alongshore. This 35 m is the depth where sand motion would started, but it does not mean that a trench there has significant changes (depth of closure, chapter 4.1).

3.5- Kochi Coast, Japan (Uda et al, 1986):

- Location: Kochi Coast (in the southern part of Shikoku Island, Japan), facing Tosa Bay and extending from Katsurahama Beach to Hagi Point (Fig.3.5.1).
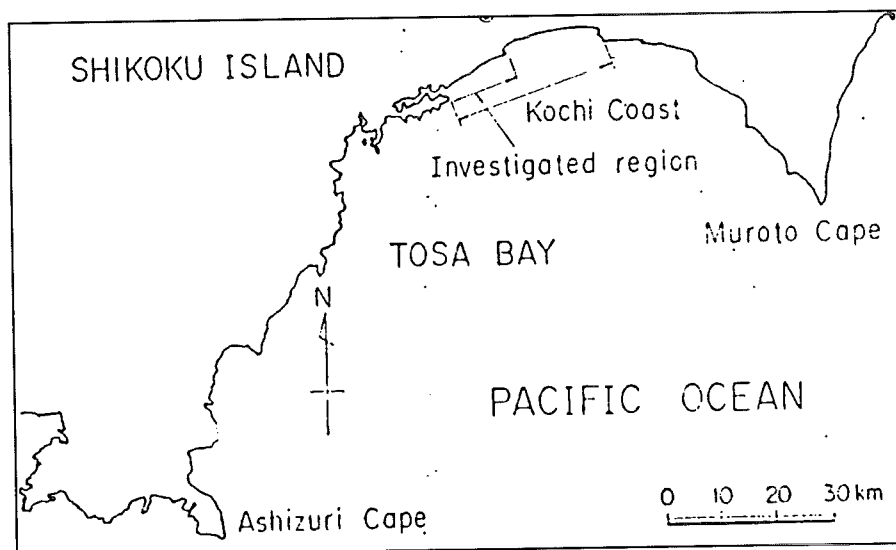

Figure 3.5.1: Location of Kochi Coast (Uda et al., 1986).

-Objective: to analyse beach changes due to offshore dredging.

-Dredging works: until 1975 there were an important borrow area between 300 and 3500 m west from the Katsurahama Beach. But the most analysed work in this paper is a sand extraction that took place 300 m (approx.) east from the Niyodo River mouth in 1981. It was a large hole of 11 m deep at maximum in the nearshore zone; the deepest point was only about 110 m off the shoreline so the slope was 1:10, quite steep.

- Wave climate: there is not enough data about it in the paper.

- Kind of analysed data: aerial photographs and bathymetric data to determine spatial and temporal changes of the shoreline as well as changes on the configuration of the holes.

- Results and conclusions:

26

From the comparison of aerial photographs they observed that the shoreline retreated simultaneously with the formation of the hole, as it was refilled, shoreline kept on being eroded. And it advanced back with the refilling of the pit.
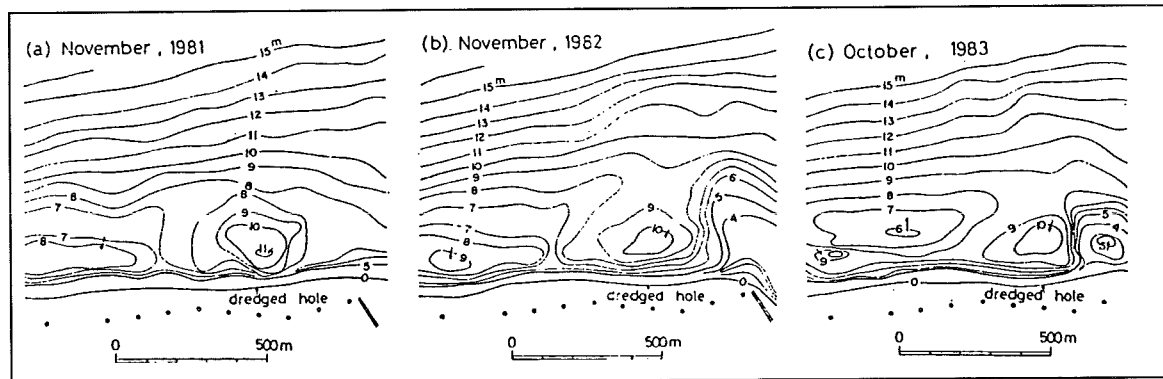

Figure 3.5.2: Contour lines off the mouth of the Niyodo River and eastwards of it: (a) November, 1981, (b) November, 1982 and (c) October, 1983 (Uda et al., 1986).

After the dredging of the large hole 11 m deep in the nearshore in 1981, two floods happened in the Niyodo River in 1982, and then the hole started to be refilled slowly. The way in which it was refilled suggested them that the process was not associated to cross-shore sediment transport and neither beach drawdown. In the first case, a more or less alongshore uniform infilling at the seaward edge must be expected, and in the second case, a suddenly fill from the landward edge would occur (Fig.3.5.3). Actually the hole was progressively filled from the upcoast end, i.e. from west to east, as it is showed at Figure 3.5.2. Because of this, they considered that the infilling of the pit was caused by the deposition of the sediment transported by the littoral drift. They also observed that the shoreline position behind the hole tended to retreat with respect to the adjacent shoreline.

They finished concluding that the shoreline in the lee of the hole retreated because of the wave refraction on the trench. Therefore, they considered that the main driving process in coastal erosion due to dredging, was the change in the nearshore wave conditions.

- Comments:

One of their main conclusions was that the littoral drift caused the refilling of trenches. It is true that while extraction holes are shallower than the critical depth for longshore sediment transport, the hole can be filled by that process (see chapter 2.1). In this case the pit is located at a depth shallower than 8 m, which, according to their analysis, is an active zone for longitudinal transport (S_l). But they based part of this conclusion on the behaviour observed at figure 3.5.2, considering that the sand outflowed from the Niyodo River fell into the dredged hole by the eastward littoral drift. But if the real cause of the

27

infilling is this littoral drift, it would have happened as well without the floods; it would not have been necessary the river's contribution. If the sand from the floods was needed, it only means that the hole was on the river's mouth.

Looking at the trench evolution in the shoreward edge (Fig.3.5.2) it seems that this part has been filled in a more or less uniform manner, which could be due to some offshore transport (i.e. beach drawdown). However, there is not data enough to be sure of it because the time spent between both configurations was one year.

Although these authors associated the coastal erosion in the lee of the hole to a change in wave characteristics, other processes could be relevant too. As it has been pointed out before, there will be a local change in the sediment budget when a trench is dredged within the longshore sediment transport zone. This change would be reflected in a progressive filling of the upcoast end of the trench while erosion would be induced in the downcoast end (Fig.3.5.3). This induced gradient in the S_l can be also reflected in the shoreline; in the lee of the hole it is eroded, especially in the downcoast portion.

CROSS—SHORE TRANSPORT INCLUDED REFILLING

SL OF THE SITE

BEACH DRAWDOWN
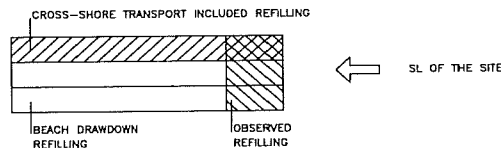REFILLING

OBSERVED
REFILLING

Figure 3.5.3: Outline of the infilling effect of a trench due to both alongshore and on-offshore sediment transport.

That the modification of waves when pass over the trench is the main cause of beach erosion can not be assessed just with the data they published. First, the dimensions of the hole are relatively small although they must be strictly analysed in relative terms, i.e. the hole width versus the wavelength. Thus, for narrow trenches (in relation to wavelength) not too much influence on the wave propagation should be expected.

Second, if the trench is wide enough to modify wave characteristics, a wave height gradient would occur in the lee side in the direction of the wave angle of incidence. This should lead to the formation of a tombolo in this lee side.

Third, there is no wave propagation analysis in the paper to justify their interpretation of the dominant role of wave modification in coastal erosion.

Finally, in the case of wave modification due to the trench, the final response of the shoreline would be a combination of "partial" responses of it on the one hand and longshore sediment transport interference on the other hand.

J.M.Motyka and D.H.Willis (1974) also predicted that shoreline position behind this kind of holes tends to retreat. This is due to the refraction of waves over dredged holes but cross-shore transport still exists too. And in the case of Fig.3.5.2 there was a steep slope that could caused part of the erosion and consequently, of the infilling.

3.6- Grand Isle, Louisiana (Combe & Soileau, 1987):

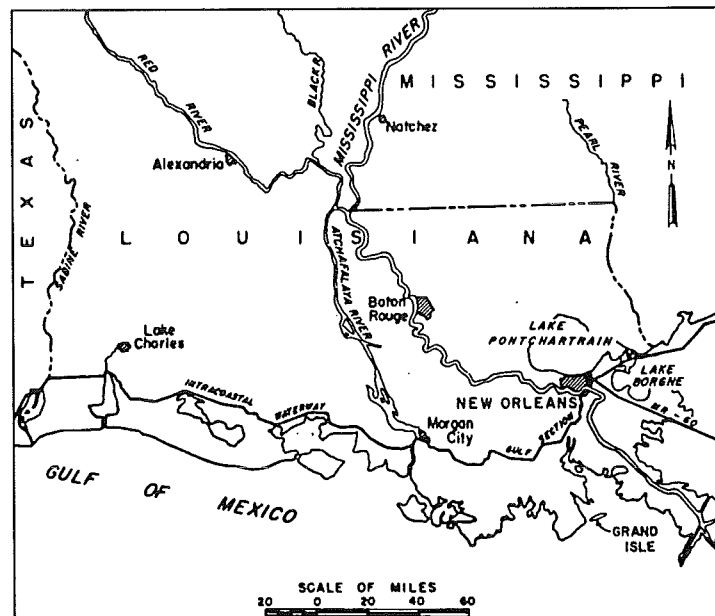- Location: Grand Isle, Louisiana, in the Mississippi Delta (Gulf of Mexico) (Fig.3.6.1).

Figure 3.6.1: Study area (Combe & Soileau, 1987).

- Objective: to analyse the behaviour of the beach and dune, which were built for beach erosion control, recreation and hurricane wave damage protection.

- Dredging works: sand required (2850000 m³) was obtained from two offshore borrow areas located about 900 and 450 m from the beach respectively. The biggest and more suitable borrow material (2250000 m³) was 900 m offshore from the coast, 2740 by 457 m in area extent and 1.82 to 2.74 m thick. There the main trench was dredged. The extraction was not homogeneous; less material was removed from the centre. The depth of excavation below the seabed was 6.16 m near both end and 3.08 m at the centre. Another hole was done 450 m offshore to extract 600000 m³ more of sand.

29

- Wave climate: after the construction in the summer of 1984, there was a severe frontal passage in November 1984 extending through April 1985, and next three hurricanes traversed the Gulf of Mexico between August and October 1985.

- Kind of analysed data: shoreline response through aerial photographs and comparison of surveys of offshore profiles.

- Results and conclusions:

After the passage of the severe frontal waves cuspate bars began to form in the lee of the borrow areas. In the primary borrow area, the biggest, two cuspate bars appeared opposite to either ends of it, with erosion occurring adjacent to and between them (Fig.3.6.2). Another bar like these was formed in the lee of the other trench, but it was not analysed in this paper.
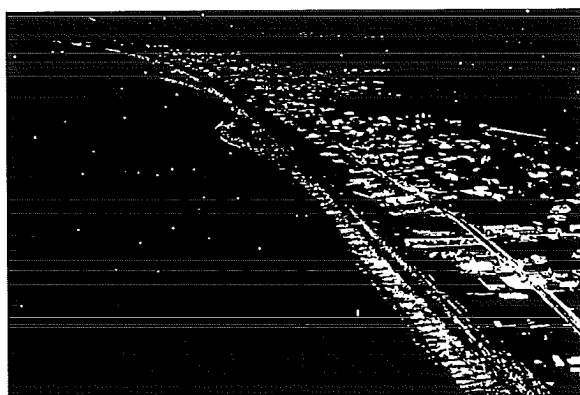

Figure 3.6.2: Two cuspate bars adjacent to the borrow area, 9 August 1985, before the action of the three hurricanes (Combe & Soileau, 1987).

This phenomena just could be explained by assuming an alongshore change in wave conditions. Then, that the dredged areas were of sufficient size to modify the wave climate was proved; diffraction and refraction were taking place. All this was just a redistribution of sand, the losses were only about 8% of the net project volume, but then three hurricanes acted. With the passage of each one the potential for damage to the undamaged portion of the dune was increased. However, despite this severe situation, the project protected Grand Isle from wave action; it prevented millions of dollars of damages to structures and public utilities although losses of sand exceeded pre-construction estimates.

From November 1985 to December 1986, after three hurricanes struck the area, a natural dune was formed over 60% of the project length and the pits showed a trend of in-filling; the deepest areas had filled to about half depth and the centre was filled to the original sea bed elevation. At December 1986 the general situation in the vicinity of the cuspate bars was like in Fig.3.6.3.

30

This natural dune building process led them to think that actually, if storms had occurred later, the project would have had the sand needed to sustain itself. And related to the trenches infilling, they realised that cuspate bars became fairly permanent.

Figure 3.6.3: View of cuspate bars, beach and dune, 30 December 1986 (Combe & Soileau, 1987).

After all this analysis they concluded that the formation of the cuspate bars which, in fact, had already been predicted by other authors as Horikawa et al. (1977), was due to width, depth and proximity of the trenches to the coast.

- Comments:

This article shows an effect that since then had only been predicted or considered as possible, but never proved. Aerial photographs let see the formation of cuspate bars in the lee of the dredging and the erosion next to them, so they confirm the theory that shape and depth of trenches are significant causes for wave refraction and diffraction in the nearshore zone.

It shows a different evolution of the shoreline due to the presence of trenches. Obviously it would have been strong erosion as well if no extraction had been done, but cuspate bars would not have formed and impacts on the shoreline would have been more homogeneous alongshore.

31

3.7- Dutch coast (van Alphen et al., 1990):

- <u>Location</u>: west Dutch coast, in the area that in Figure 3.7.1 appears as Holland coast.
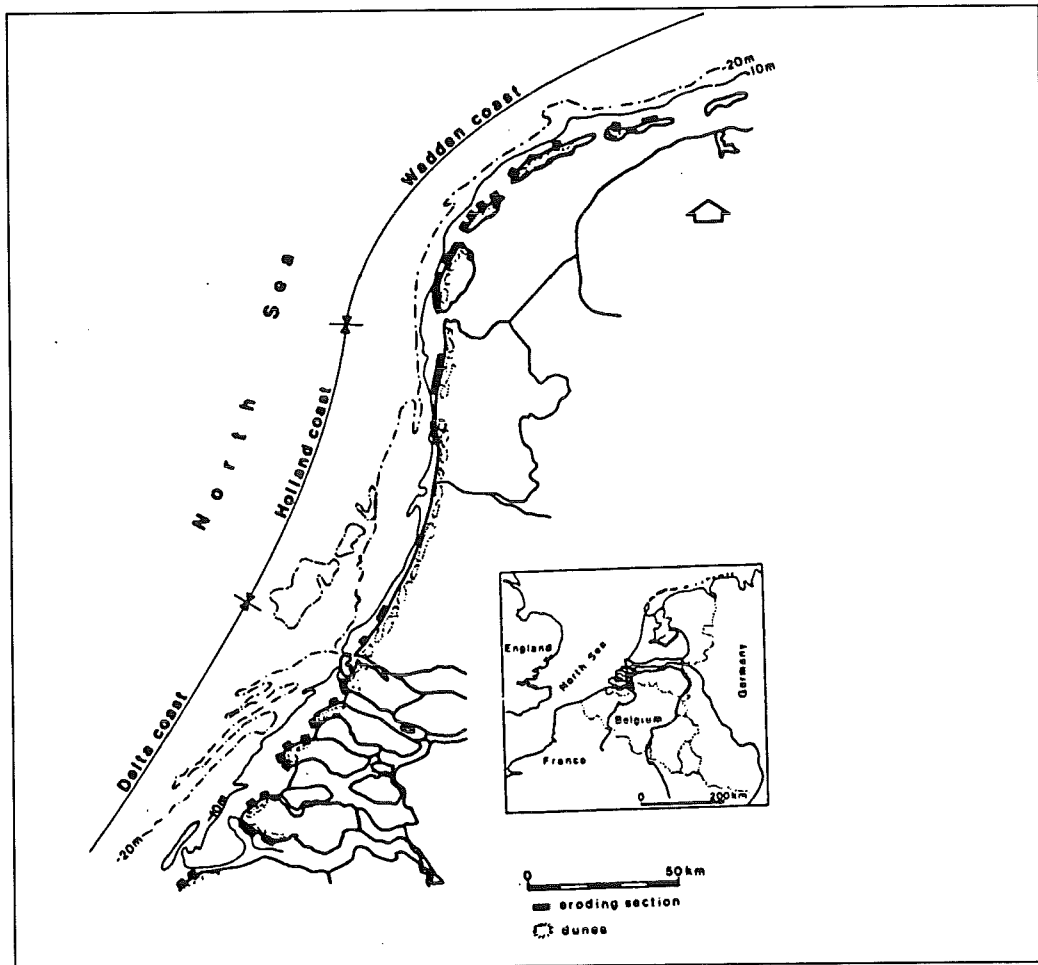

Figure 3.7.1: The Dutch coast (Roelse, 1990).

- <u>Objective</u>: to obtain indications and quantitative estimates of the morphological effects of offshore sand extraction and nearshore profile nourishment (but nourishment results are not included in this chapter).

- <u>Dredging works</u>: different sand extraction schemes between the 10 and 20 m isobath were analysed varying in length, width and extraction depth.

- <u>Wave climate</u>: the year averaged wave height and period is 1 m and 5 s respectively. But the sediment stirring effect of waves was incorporated for $H_s=1.5$ m and $T=6$ s.

- <u>Kind of analysed data</u>: all the study was based on different models. The direct hydrodynamic effects were studied using 2DH models while the direct and long-term morphodynamics of the extraction pits were estimated in longshore and cross-shore direction separately, by a quasi-2DH morphodynamic model (LOMOR) and by a one-dimensional morphodynamic model (CROSTRAN, Roelvink & Stive, 1988) respectively. Morphological changes were found considering both waves and tidal current action.

32

- Results and conclusions:

The results obtained showed that the effects of sand extraction holes on hydrodynamics were local and minor as well. The most important changes existed for a trench of 5 m depth on the 14 m isobath where the wave height increased in a 1% and the tidal current velocity decreased in about a 10%, but these modifications disappeared at 10 m of depth.

Because of these minor direct changes, the instantaneous morphological adaptation was also negligible.

On the other hand, long-term effects were observed. After 40 years there was an onshore migration of a trench on the 16 m isobath (Fig.3.7.2). As a result of a small net onshore sediment transport, deposition on the offshore margin and erosion of the landward boundary led to the propagation of the pit to the coast. But this effect did not appeared for trenches on 20 m isobath. Comparing the cross-shore behaviour of both extractions (Fig.3.7.2) the increasing importance of wave action with decreasing waterdepth and distance offshore was illustrated.

As a consequence, the predicted migration rates of seabed disturbances speeded up rapidly landward of the 16 m isobath. Sand extraction landward of this isobath would affect the coastline within a century due to the approach of the trench to the coast that yields a deficit in the nearshore sand budget.

But these conclusions could not be verified in the field so no dredging were permitted landward of the 20 m isobath. Thus, a security coefficient was included over the 16 m calculated due to the uncertainty of the models.

- Comments:

The main problem of this study is that the results could not be verified in the field, with real data. Because of this uncertainty, the threshold depth landward of which no dredging is authorised was kept on the 20 m isobath. This was the depth that the Dutch regulatory authority already used in the past based on the idea that wave induced onshore sand transport became increasingly important in shallow waters. Moreover, it is the same depth that other studies indicated as the one of Migniot & Viguier also presented in this chapter (3.2) or the one of C.E.R.C. (1984).

Although it did not give a real tested behaviour of trenches, the computations of the morphological development of them showed if their evolution was more or less important and verified the idea that as more nearshore they are, more likely their presence can affect shoreline. But these models were not able to represent the real development of extraction pits yet.
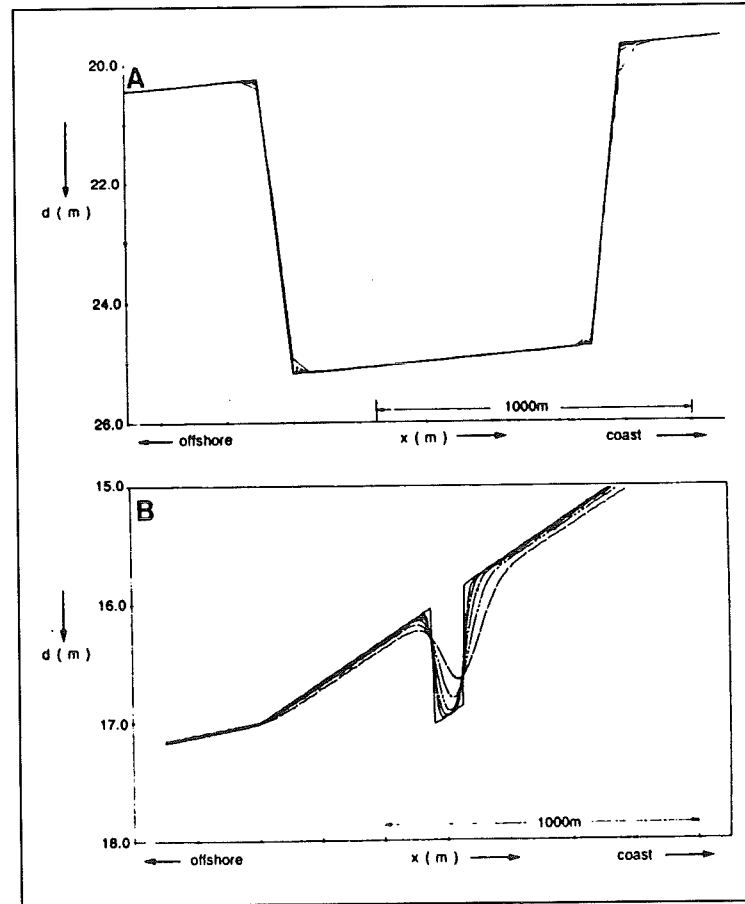
Figure 3.7.2: Predicted cross-shore behaviour of extraction pit at (a) d=20 m (W=1000 m, dh=5 m) and (b) d=16 m (W=100 m, dh=1 m) over a period of 40 years (van Alphen et al., 1990).

3.8- Kirra Beach, Australia (Jackson & Tomlinson, 1990):

- Location: in Kirra Beach, one of the Gold Coast Beaches, in the eastern coast of Australia, south of Brisbane (Fig.3.8.1).

- Objective: to give a qualitative assessment of the impact on coastal processes of the dredging and nearshore nourishment at various depths and to examine the applicability of the modelling techniques for predicting future nearshore nourishment behaviour and options (but nourishment is not studied in this chapter).

- Dredging works: sand was dredged from seaward of the 20 m water depth during 1988 to get 1,5M m³ needed for the nearshore nourishment (between 6 and 10.5 m depth). A plan of the works is showed at Figure 3.8.2.

- Wave climate: as stand-down conditions applied for a significant wave height (Hs) greater than 1.9 m. But heavy sea conditions and storm conditions took place for over a year afterwards. During the worst storm, Hs were 5 to 6 m's for two to three days with wave periods of up to 12 sec.
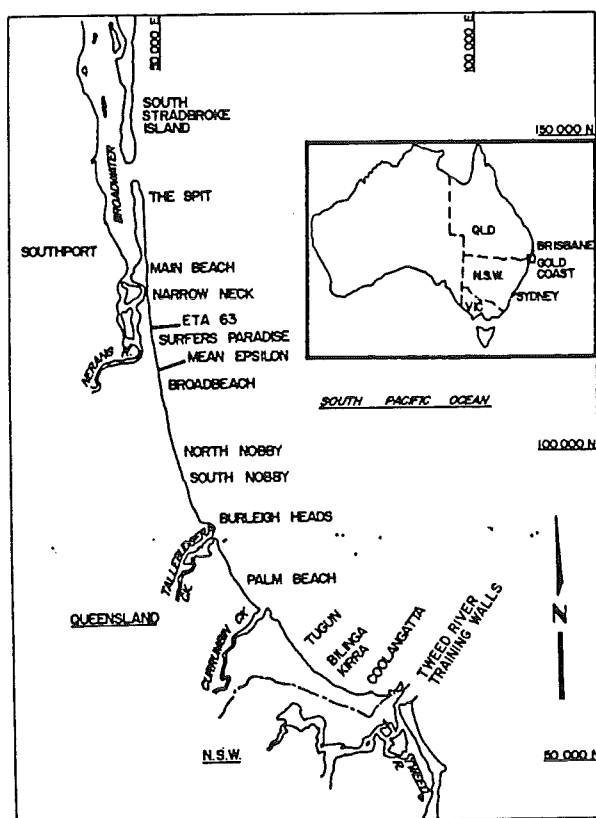
Figure 3.8.1: Study area (Jackson & Tomlinson, 1990).

- Kind of analysed data: they used theoretical studies. Ray wave refraction computer modelling techniques to see the possible effects on shoreline conditions and a sediment transport model (Perlin & Dean, 1983, updated by Scheffner & Rosati, 1987) to predict the rate of migration shoreward of a nourished bar. To test this last model also survey data was used.

Figure 3.8.2: Plan of Kirra nourishment project (Jackson & Tomlinson, 1990).

- Results and conclusions:

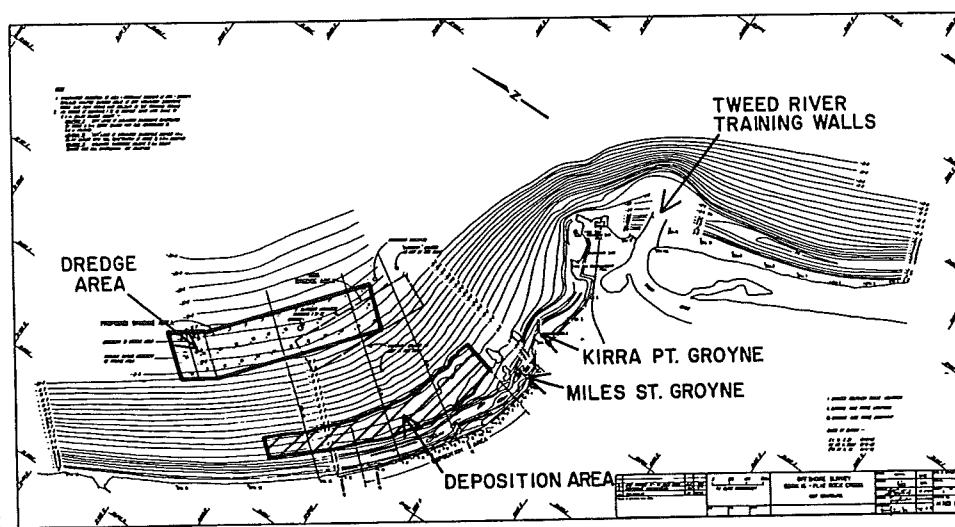Through the wave refraction models was demonstrated that if dredging were in areas seaward of the 20 m depth contour, no significant change on the shoreline wave climate would appear. The criterion used was the change in calculated refraction coefficients and wave angles from the initial bathymetric conditions to the post-nourishment condition. Therefore, they obtained, as could be expected since the dredging operation was parallel to the contour lines, negligible refraction.

Despite adverse conditions during the following year, works were effective in protecting the foreshore and allowing accretion as it was predicted by the model studies.

- Comments:

According to this paper, using theoretical models good predictions of real behaviour might be obtained. But it is always needed to assure if models can represent what is being studied. Then, some field data to test their capacity for predicting real responses should be collected. However, most of the times models will give just qualitative approximations, but rarely quantitative.

In this situation, it was proved that the models used gave good results, but the data analysed do not allow inferring if dredging works actually influenced on the internal coastal evolution.

It is more a study about the functionality of nourishment to avoid beach erosion than about sand extraction. They studied extraction effects just because to carry out the nourishment project on this beach, sand was needed.

3.9- Iles-de-la-Madeleine archipelago (Anctil & Ouellet, 1990):

- Location: in the two spits of Iles-de-la-Madeleine, in the central Gulf of St. Lawrence, Québec, Canada. These tips are Sandy Hook in the south and Pointe de l'Est in the north east (Fig.3.9).

- Objective: to assess the potential impacts of an inner-shelf sand extraction This was theoretically done by analysing the changes in the refraction pattern by dredging.

- Dredging works: there were two dredging zones, one of 16 km^2 located between the Sandy Hook and Entrée Island and the other of about 30 km^2 located off Pointe de l'Est, and both were between the 10 and 20 m isobath (Fig.3.9). Companies wanted to progressively increase the annual dredging volume from 500000 tons to 1 million tons. But inside these areas, just some parts had sediment suitable for glass or iron industries. Considering just the extraction of these materials, 3 different schemes were modelled taking into account 20 years of dredging:

1- 1 m thick layer of sediment extracted over 2.4 km^2 at Sandy Hook and over 7.9 km^2 at Pointe de l'Est. Only sediments suitable for the cast iron industry were considered.

2- 2 m thick layer of sediment for cast iron industry extracted at Sandy Hook over 2.4 km^2 and 1 m thick layer of sand for glass industry dredged over 5.1 km^2 at Pointe de l'Est.

3- 2 m thick layer was removed over 5.1 km^2 at Pointe de l'Est for the needs of glass industry.
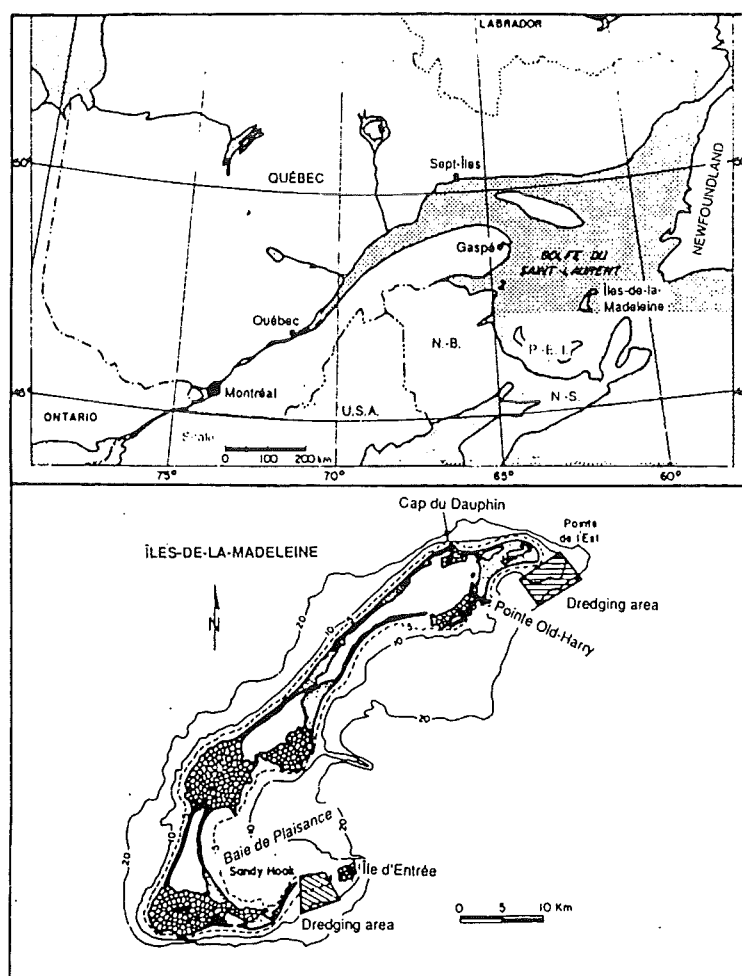
Figure 3.9: Study area. Sandy beaches are shown with a dot pattern and the potential dredging sites with dash lines (Anctil & Ouellet, 1990).

- <u>Wave climate</u>: in the area of study just 3 six-months period waves measurements were available and they were not enough to estimate local wave characteristics. Therefore a model was used to do it from wind data collected (the SMB method, Desjardins & Ouellet, 1984). Wave conditions were selected by using a hindcasting analysis.

It was pointed out that the shore of the archipelago is generally packed with ice from December 15 to April 15. Consequently, only the other 8 months were considered.

- Kind of analysed data: 3 six-month wave data sets collected in 1974, 1975 and 1976, maps, aerial photographs, and some field data as well as results of other authors. Their conclusions were basically based on the use of different models to estimate the wave climate, wave propagation and littoral drift.

- Results and conclusions:

After the adaptation of potential impacts due to nearshore dredging works analysed in other places, they found that there, the most dominant criterion were wave refraction patterns due to bathymetry changes. Using a model that combined wave propagation and littoral drift phenomena they studied different kind of dredging schemes. Contrasting them with the "natural" situation, i.e. without dredged holes, they concluded that 1 m deep excavation would cause negligible or minor impact. But to generalise this result, they said that excavations should be designed in order to limit modifications to wave refraction in such a way as the resulting littoral drift changes were kept within bound specified after the local sediment budget. And of course, as deep as possible since it was known that coastal impact increases inversely with water depth.

They also pointed out that if during the extraction operations bathymetry modifications were as gradual as possible, impacts could be reduced.

- Comments:

Since the results were basically based on model estimates and for a specific area, there is not a complete certainty of them. They can be considered as another contribution to the analysis of the possible effects of dredging works to wave refraction and its impact to the littoral drift, or what is the same, to the evolution of the coast.

3.10- Phuket, Thailand (Vongvisessomjai, 1994)

- Location: Bang Tao Bay, on the west coast of Phuket island, on the west of the peninsula of Thailand (Fig.3.10.1).

- Objective: to find out the causes of observed beach erosion and to establish depth criterion to conserve the beach resorts from erosion.

- Dredging works: offshore tin mining holes, about 300 m of the shoreline and 6 m deep. The analysed dredging works started October 1990 and finished on April 1991 although previous mining in the area are dated since 1987.

- Wave climate: the bay is attacked by waves from May to September, during the southwest monsoon. These waves are mainly from the west so they do not provoke so much alongshore transport.

During the southwest monsoon in 1991 wave height were about 0.5 and 1.5 m with periods of 7 s (Vongvisessomjai & Huq, 1991).
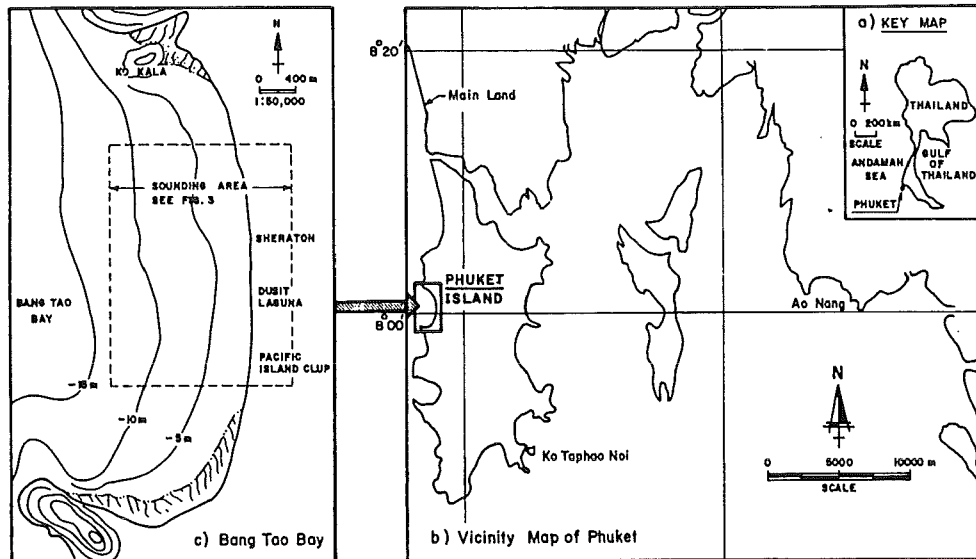
Figure 3.10.1: Study area: (a) key map, (b) vicinity map of Phuket and (c) Bang Too Bay and its bathymetry before offshore tin mining (Vongvisessomjai, 1994).

- Kind of analysed data: there is an important part of literature review that was used to confirm or check some field data and conclusions. Field data on wave climate, alongshore current, wind and resulting beach erosion was collected. The characteristics and behaviour of the beach were observed by field measurements of beach shape and profile, soundings (3 for an area of 2.5 km alongshore by 1.5 km offshore) and bed material sampling.

- Results and conclusions:

Comparing contour lines of soundings in 1990 with those in 1987 for the shorelines, it was found that severe erosion occurred. Average erosion rate was about 7 m per year.

They also pointed that the nearshore holes interrupted the cyclic changes of the beach in terms of erosion by storm waves and subsequent accretion by swells. This resulted in a permanent loss of sand from the beach that was deposited in the dredged holes. Based on beach profiles showed in Figure 3.10.2 and theoretical concepts, they concluded that nearshore deep holes changed the natural beach profile that was like dune with foreshore before the dredging, to a profile like dune without foreshore. Thus, profiles with dredged holes experienced a much higher rate of erosion.

In the final recommendations they said that offshore tin or sand mining should not be made in shallower water than 15 m.

- Comments:

The profile evolution obtained by sounding (Fig.3.10.2) did not show so clearly that holes were refilled by sand from the upper part of the profile after storm events. Trenches behaviour is different for each analysed profile; it does not appear any homogeneous and

similar development of the holes. However, in DL4, where there was the closest hole to the shoreline, it can be observed how the pit drew beach sand.

On the other hand, it can not be concluded so easily that the erosion observed is due to the presence of mining. In fact, the most important erosion was attributed to a storm of severe waves.
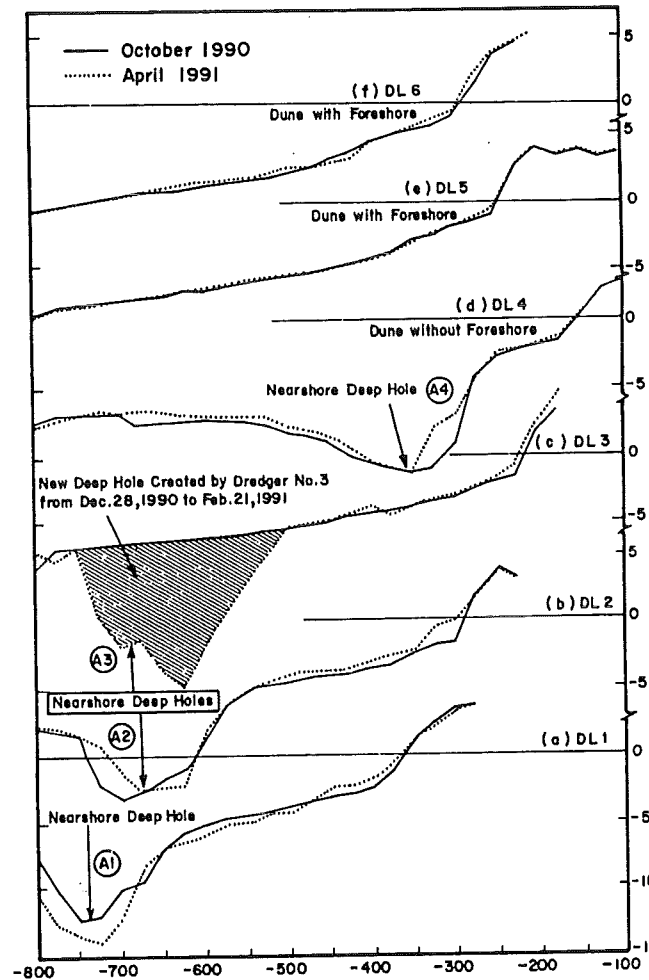

Figure 3.10.2: Beach profiles at different points of the bay, with holes (from DL1 to DL4) and without them (DL5 & DL6) (Vongvisessomjai, 1994).

3.11- Chang-Hwa, Taiwan (Hsu & Chang, 1994):

- <u>Location</u>: in a laboratory they represented situations of the Chang-Hwa industrial area, on the west coast of central Taiwan.

- <u>Objective</u>: to study the effect of offshore sand extraction to be used in land reclamation.

- <u>Dredging works</u>: the trench was at 1.5 km offshore, at −15 m depth that became −16 m in nature, but they worked with this dimensions reduced by scale factors.

- <u>Wave climate</u>: monsoon and typhoon waves in NNE direction.

- Kind of analysed data: results of movable bed model tests. Several test cases were carried out including: tests for determination of time scale, tests for sea bottom changes under initial conditions, tests for sea bottom changes after dredging at –15 m water depth and tests of countermeasures for shore protection after dredging. In Tables 3.11.1 and 3.11.2 some relations between model and prototype conditions are presented.

physical quantity	scale	prototype		model
horizontal length	1/600	24 0km		40m
vertical length	$\frac{1}{100}$	water depth	+3m~-2.0m	+3cm~-2.0cm
		tidal level	+2.38m - -2.13m	+2.8cm~-2.56cm
wave height	$\frac{1}{74.3}$	monsoon wave	2.60m	3.35cm
		typhoon wave	4.50m	6.90cm
wave period	$\frac{1}{8.62}$	monsoon wave	7.76sec	0.90sec
		typhoon wave	11.20sec	1.38sec
Tidal period	$\frac{1}{10}$	12.42hr		1.242hr
submerged specific gravity	$\frac{1}{1.62}$	1.65		1.02
median diameter	$\frac{1}{2}$	0.24mm		0.12mm

Table 3.11.1: Physical quantities in model and prototype (Hsu & Chang, 1994).

quantities wave	scale	prototype	model
monsoon wave	1/5267.4	0.6 year	1.0 hr
typhoon wave	1/10	10 hr	1.0 hr

Table 3.11.2: Time scale of topographical changes (Hsu & Chang, 1994).

- Results and conclusions:

The first tests were used to determine the time scale of topographical changes by longshore sediment transport, which tended to become a constant value after 3 hours.

Under the initial conditions, when monsoon and typhoon waves acted, the littoral sediment transport caused seabed changes because of the limited sand supply from

41

upstream. There was erosion of the sea bottom around the seawall of each reclamation area and recession of the shoreline in the most eastern area (Fig.3.11.1).
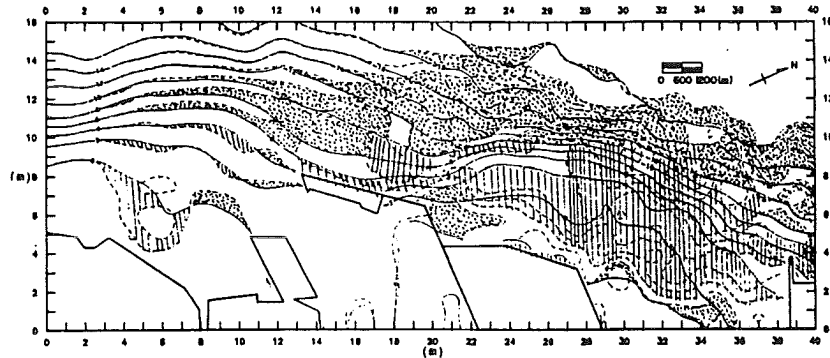

Figure 3.11.1: Seabed changes due to storm wave action before no dredging works (Hsu & Chang, 1994).

When they dredged, all sides of the industrial area were protected with new seawalls and along them severe erosion took place. Figure 3.11.2 shows this and the accumulation of the sediment at deep water. They concluded cross-shore sediment transport dominated topographical changes but there also was a very complicated distribution of the longitudinal transport due to the interaction between incoming and reflected waves.
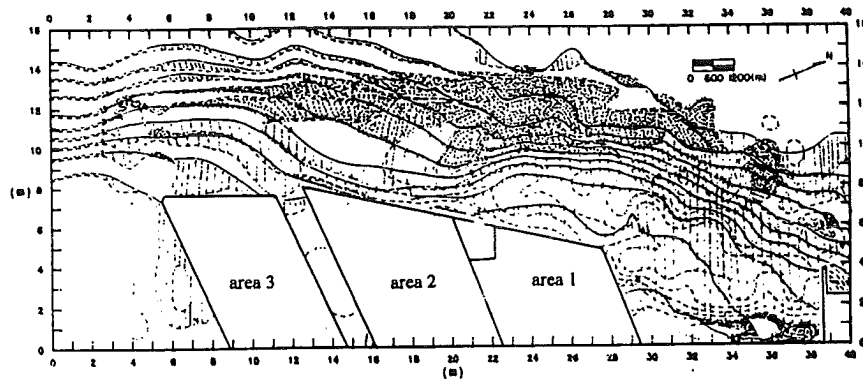
Figure 3.11.2: Sea bed changes due to storm wave action when there was the new reclamation area after dredging (Hsu & Chang, 1994).

Trying to avoid shoreline recession and beach erosion in the vicinity of seawalls, some groins were designed and they really protected shore from the erosion (Fig.3.11.3).

After all these tests, they said the most important effect of dredging was beach erosion, which could be decreased by the construction of groins.
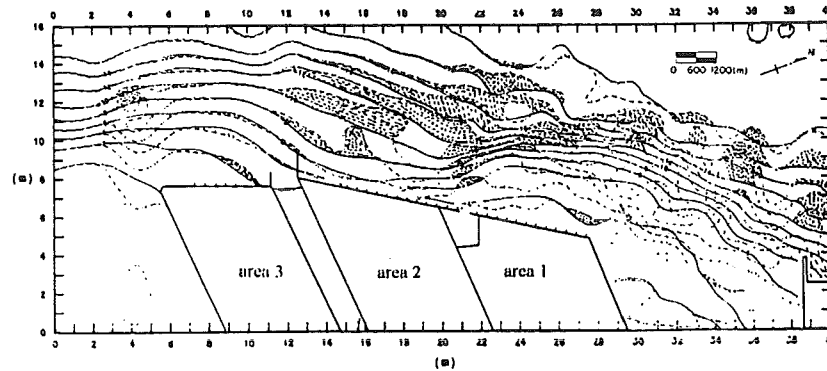
42

Figure 3.11.3: Sea bed changes due to storm wave action when groins protected the new reclamation area (Hsu & Chang, 1994).

- Comments:

By using movable bed model tests they reproduced the actual situation in the reclamation area. They put, in the model, each step as they were suppose to be in the real project, so they built seawalls to protect the new area from wave attack. This made conditions before and after the dredging different, not only because of the extraction. Therefore, the bottom and beach erosion that they observed after dredging, could not be only associated to it. If the seawalls had been constructed before, the complicated distribution of longshore sediment transport would have appeared without the borrow pit too. Moreover, this longitudinal transport seems to be important since the protection done by groins worked. Groins mainly affect this transport, more than the cross-shore, as it was proved with the last tests (Fig.3.11.3).

These experimental studies showed what could happen in this specific project, but they can not be used to predict the effects of dredging offshore in general. They are not useful to determine the real influence of dredging either in this case or other.

3.12- Pakiri-Mangawhai (Hesp & Hilton, 1996):

- Location: Pakiri-Mangawhai, on the headland-bay coast of northeast North Island, New Zealand (Fig.3.12.1).

- Objective: to analyse the impacts of nearshore sand extraction to re-examine the basis that let mining companies extract there for years.

- Dredging works: extraction occur 2-300 m from high water in water depth of 3-8 m, along 9.5 km length.

- Wave climate: the annual mean significant wave height (H_s) is 1.044 m with annual standard deviation (σ_s) of 0.727 m and associated period (Ts) of 12 s. Typical swell is considered as H=1.5 m and T=7 s, and storm waves as H=5 m and T= 12 s.
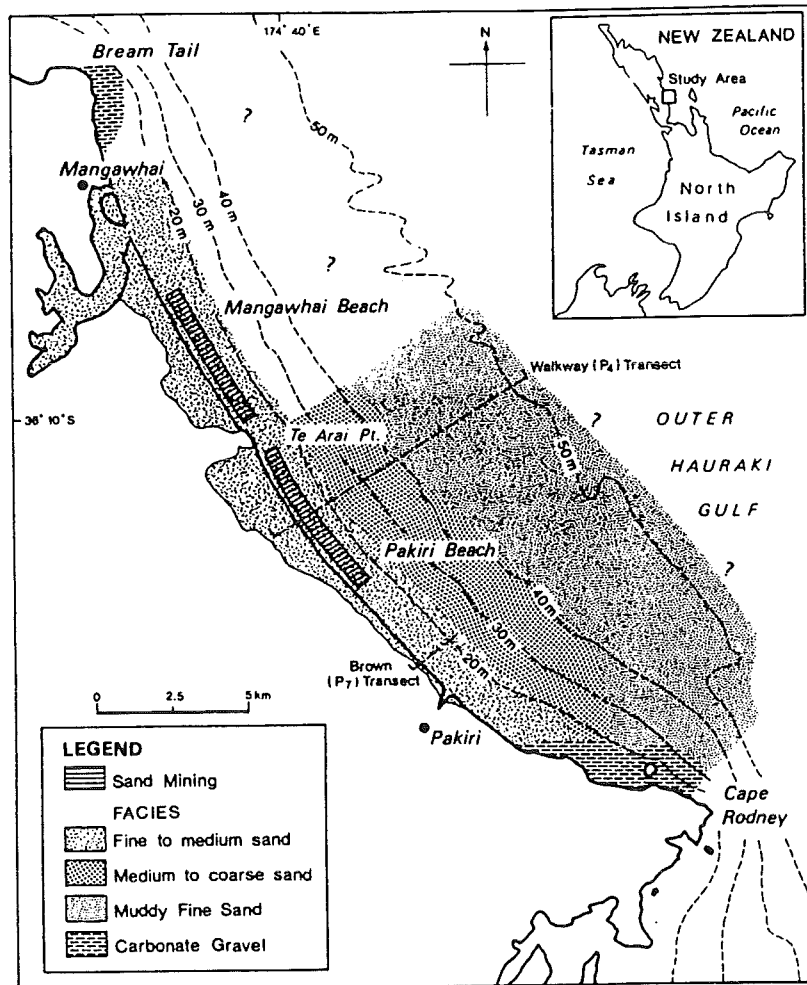
Figure 3.12.1: Study area and its principal sediment facies (Hesp & Hilton, 1996; after Hilton, 1990 and McCabe, 1985).

- <u>Kind of analysed data</u>: they used almost the same empirical and quantitative techniques that can be used to define the limits of the active beach-nearshore system; geological and topographical maps, aerial photographs, some few field observations of stratigraphy, bedforms and benthic fauna, surveys of surfzone and nearshore bed movement (analysis of beach profiles), studies of comparable beaches and theoretical estimates of sediment movement.

- <u>Results and conclusions</u>:

After the interpretation of all these data they found the maximum limit of the modern beach-nearshore sand system for east coast Australian and New Zealand moderate to high-energy beaches around the 25 m isobath. Particularly, they demonstrated that in water depths shallower than -15 m significant bed level variation was possible (Fig.3.12.2).
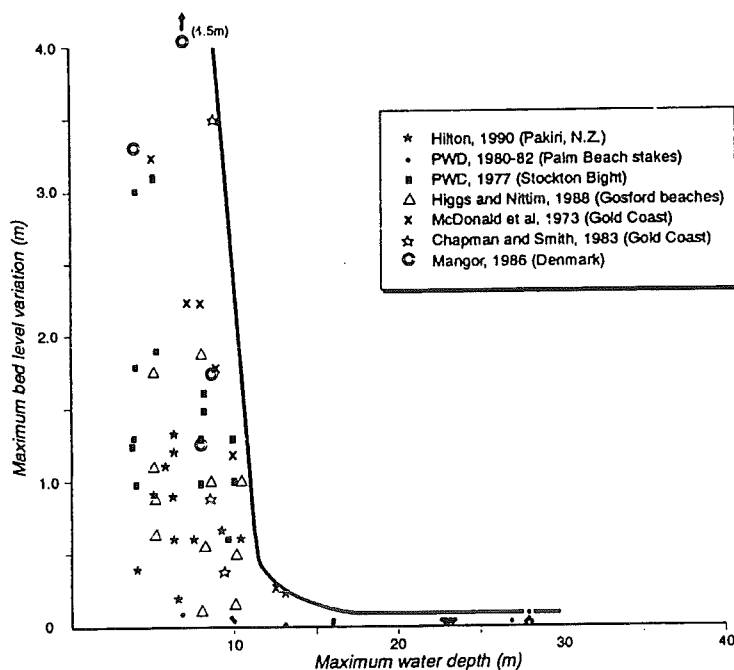
44

Figure 3.12.2: A plot of maximum bed level variation against maximum water depth for various Australian, New Zealand (Pakiri) and Danish beaches (Hesp & Hilton, 1996).

Hilton considered the existing beach profile record was of limited efficacy for determining the impact of nearshore sand extraction because: (i) they were based on time series that began when coast had been severely eroded by 1978 storms and finished when there was a relatively nourished condition, (ii) much of the recorded accretion resulted from artificial foredune nourishment, (iii) geomorphic significance of other anthropogenic disturbances had not received adequate attention and (iv) surveys cover just the landward fringe of the beach-nearshore sand system.

Therefore, other studies in comparable beaches were used and finally they concluded:

-   Pakiri-Mangawhai coast is at best stable and possibly erosional.
-   The beach-nearshore sand system is closed to significant inputs of sediment so sand is a finite resource and mined sand is mainly replaced by internal reworking of nearshore sediments (seawards of 25 m isobath).
-   They expressed the impact of extraction as partial recovery and exacerbated backshore erosion during the next severe storm.

Based on these conclusions they hypothesise that the weak recovery observed on the Pakiri-Mangawhai coast after 1978 storms might have been a consequence of sand mining, otherwise more accretion would have occurred.

However, they admitted that a cause-effect relationship between coastal sand extraction and backshore stability was not proved there.

45

- Comments:

It seems that they based conclusions more on other similar studies than on the available data. Although Holocene record barriers and some fauna confirmed their feeling of considering the Pakiri-Mangawhai coast stable or erosional, it does not give any real relationship between mining and coastal evolution.

After considering this beach no accretional and a closed system, they gave some theoretical consequences and they warned about the weak sustainability of mining so close to the shore.

Although they could not prove the real impacts, their analysis was useful to determine sediment transport thresholds; there, no significant sand movement occurs seawards of the 25 m isobath as it happens in other similar east coast Australian and New Zealand beaches.

According to this paper the mining operations in Pakiri-Mangawhai should not take place at the current locations although other studies had showed there had not been any evidence of the impact of this mining in years.

In summary and as expected (see review in chapter 2), it is logical to assume that any shallow dredging operation, i.e. in the surfzone, done in non sheltered areas will effectively affect the shoreline stability and, in consequence, they must not be allowed.

3.13- Sandbridge Shoal, Virginia Beach, USA (Basco & Lonza, 1997):

- Location: Sandbridge Shoal off Virginia Beach, Virginia, USA est coast, in the Atlantic Ocean.

- Objective: to develop nation-wide management guidelines for dredging beach quality sand in federal waters for all the US coastlines. All this through an assessment criterion for the relative difference found in the numerically modelled water wave transformations before and after the nearshore dredging.

- Dredging works: about the dredged zone there is not a lot of information. Just that is located in Federal Waters, outside the 5 km limit, and that it would last 50 years.

- Wave climate: first was used a large scale-coarse grid model that was calibrated for the wave climate of Hurricane Bertha (July 1996; 3 m, 13 second waves from the southeast direction with relatively light local winds). For the medium scale-fine grid model, the hindcast wave information for the Atlantic coast as updated by Brooks & Brandon (1995) for the 1976-1993 period was used as the wave climate.

It was found that in Federal Waters, where dredging were studied and depths are between 10 and 20 m, ocean wave conditions (T>4 s) had to be considered.

- <u>Kind of analysed data</u>: it was used the parametric, discrete spectral model MIKE21.NSW, and some field data to calibrate it. This numerical model is a wave transformation model developed for the nearshore zone at the Danish Hydraulic Institute.

- <u>Results and conclusions</u>:

All this study was based on the use of numerical models to represent wave transformation. They used a case study example located where some measured wave data existed and could be used to calibrate the model. When the coarse grid model was used the results were not reasonable since the grid scale was too coarse, but they provided boundary conditions along the offshore boundary for the fine grid model.

A part of this case study, a plane beach slope and an equilibrium beach profile shape with realistic dimensions were studied to remove the influence of irregular bathymetry on the results. Also the borrow site volume was varied to find out the influence of trenches' area and depth on the results. For a water depth of 10 m, wave periods greater than about 3.7 s were "transformed" as they propagated landward.

They concluded that the decision of whether a potential borrow site has to be rejected or not should be based on (1) results of a numerical model and on (2) the existence of a sound hypothesis for the acceptance or rejection of the proposed site. But they proposed a preliminary hypothesis: if the change in "mean" (or rms) total wave energy content along the reference line exceeded the natural level by more than two standard deviations of the mean, the borrow site design would be rejected.

- <u>Comments</u>:

In this report they spent more time explaining how the numerical model operates and what they used to calibrate it than providing results.

On the one hand this work mainly recommends to use numerical models of wave propagation to analyse dredging works, but on the other hand, they do not give details on how shape and depth of a trench would affect wave propagation although this is possible to estimate with these kinds of models.

Finally they proposed a method to quantify the importance of the changes in propagation but it is not checked against its implications on coastal evolution.

After reading and analysing all these cases, no clear conclusion can be obtained about the location of trenches to avoid real impacts. In most of the cases final conclusions have not been fully proved and results obtained (and also data used) are so different and disperse that is not possible to obtain any global conclusion or neither to assure that the observed coastal response is due to the dredging works. Just in the 3.6 (Combe & Soileau study of what happened in Grand Isle) a real impact is presented and shown by a

succession of aerial photographs. Knowing the extraction works that were carried out these photos let see how presence of trenches can affect wave propagation and consequently, shoreline development. Thus, changes in wave refraction can be considered as an actual effect of borrow pits nearshore. But this impact, although just in this case is really proved, is presented as one of the most important and likely to happen in several cases. That is why it is more explained and analysed in chapter 4.3, to find some global criterion or knowledge.

Something that also appears in almost all the studies is trenches evolution during storms. As it is known, when storms act beaches suffer erosion and the sand eroded travels seawards. Trenches can trap this sand and induce the called beach drawdown (see chapter 4.1). It is presented in some papers as the cause of more important erosion, but in all of them the authors recognised that any relation cause-effect could be absolutely verified in the field. It was difficult to distinguish if erosion would have been the same or minor if trenches had not existed when storms acted.

As said above, in chapter 4 these two effects are analysed in more detail, but also trench propagation (see chapter 4.2). It has appeared as possible in some modelled and theoretical analysis (e.g.3.2, Migniot & Viguier, and 3.7, van Alphen et al.) and it becomes more important as time scale increases, in long-term situations. Although instantaneously it is negligible, it deserves a deeper study since it can induce trench to catch a dynamic zone.

In some of the studies the depth of initiation of motion is calculated and given as a limit where trenches can be dredged to guarantee no impacts. But as it is also explained in these studies, it is a conservative value.

There are several theoretical studies and although they do not give real results, they give good qualitative analysis and useful patterns of some general behaviour extrapolative to different places. However, it will be always necessary to make specific analysis for each zone to dredge and to have field data to calibrate and compare the results as it is explained in 3.13 (Basco & Lonza.).

4. CRITERIA DEVELOPMENT FOR THE MAIN IMPACTS:

In this chapter the 3 impacts that have appeared as the ones that most easily can affect coastal stability are analysed to find some general criterions to avoid them.

4.1-Beach drawdown and analysis of depth of closure:

As it was explained at chapter 2.2, one of the consequences potentially easier to find due to nearshore sand extraction is the beach drawdown. The sediment eroded from the upper part of the beach due to the action of a storm, falls into the trench leading to a loss of the sand that was supposed to rebuild the beach during next accretionary periods. This lack in the sedimentary balance implies less sediment in the new beach profile's equilibrium, which cause beach erosion.

Considering just this effect, trenches should be done seawards from the threshold depth of vertical changes due to the cross-shore transport caused by changes in incident waves conditions. Sand should be extracted seaward from the depth of closure that represents the limit of "significant" depth change (Hallermeier, 1981).

Depth of closure is seen to act as a morphodynamic boundary separating a landward morphodynamically active region from a more seaward inactive region where vertical changes are shorter than the criterion chosen to define closure (Fig. 4.1.1)
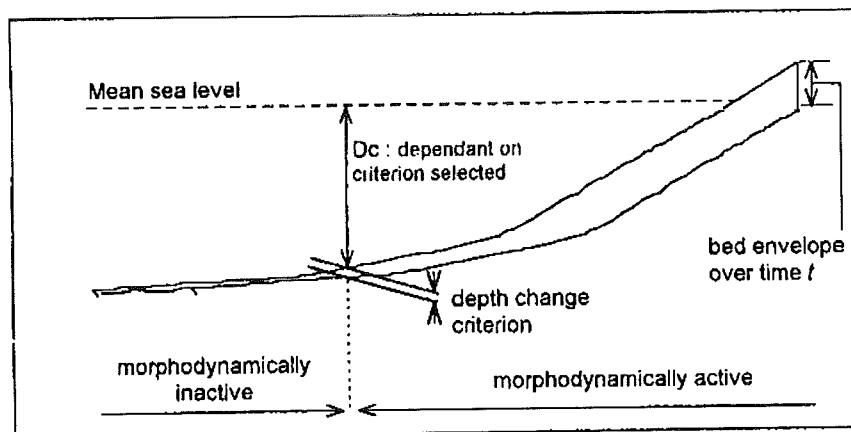

Figure 4.1.1: Zonation of a cross-shore profile over time *t* where Dc represents the seaward limit of significant depth change using the depth change criterion shown (Hinton & Nicholls, 1998, adopted from Hallermeier, 1981).

Depth of closure is a morphodynamic boundary, not a sediment transport boundary. It does not refer to a depth seaward from which there will not be cross-shore sediment transport. It allows estimating how far offshore morphological change is likely to extend.

The right way to find this depth would be by comparing data of profiles from the same alongshore location using methods as the standard deviation of depth change (Kraus & Harikai, 1983) or as the fixed depth change (Nicholls et al., 1996). But this means to have large data sets and that is not always possible. Therefore, models to predict closure are needed. The most used and verified is the one developed by Hallermeier (1981) to estimate the depth of closure on sandy beaches based on the characteristics of the wave's climate:

$$d_l = 2.28 H_{s,0.137} - 68.5 (H_{s,0.137}^2 / g T_s^2) \qquad (Eq.4.1.1)$$

where $H_{s,0.137}$ is the significant wave height that is exceeded 12 hours per year, T_s the associated wave period and d_l represents the limit depth for the changes on the beach's profile.

Short and medium scale studies in wave-dominated, microtidal sandy coasts have shown that closure is time and space dependant (Gracia et al., 1997; Capobianco et al., 1997; Rózynski et al., 1997; Marsh et al., 1998).

d_l generally increases when time scale does it, because the probability of occurrence of a larger storm increases. As a consequence Hallermeier equation was generalised as time-dependant (Stive et al., 1992):

$$d_{l,t} = 2.28 H_{l,t} - 68.5 (H_{l,t}^2 / g T_{l,t}^2) \qquad (Eq.4.1.2)$$

$d_{l,t}$ is the predicted depth of closure over t years, $H_{l,t}$ is the non-breaking significant wave height that is exceeded 12 hours per t years and $T_{l,t}$ is the associated wave period.

The same studies that showed closure dependence on time and space scales, also showed that the generalised Hallermeier equation just provides a limit to real closure for individual erosional events up to the annual periods. It considers cross-shore redistribution of sediment but excludes the effects of beach-nearshore profile translation that becomes determinant for the location of closure at medium scales. At medium scales it has an increasing tendency to overprediction. Consequently the depth given by this equation should be considered as an upper limit to depth of closure, and not as a prediction of it (Nicholls et al., 1998).

Other processes, apart from the wave conditions, can affect the closure depth because alongshore variability has been observed (Nicholls et al., 1998).

To analyse closure and its variation with time and space, the Large-Scale Coastal Evolution concept (LSCE) is used (Stive et al., 1990). Three main scales in coastal morphodynamics can be identified:

i)   Large-Scale: with a morphodynamic length scale of 10s km and time scale of decades. It is the scale needed to determine the long-term effects of changing boundary conditions or of huge interference by man (as dredging).

ii)   Middle-Scale: with a morphodynamic length scale of 1 km and time scale of years. It is the scale used to identify the impact of coastal works on the coastline development.

iii)   Small-Scale: with a morphodynamic length scale of 100s m and time scale of storms to seasons. It is needed to the more detailed design of coastal defence works.

At small scales, it was found the influence of bar behaviour; depth of closure is usually the product of bar migration due to surfzone processes (Nicholls & Birkemeier, 1997; Nicholls et al., 1998). During erosional events the pre-event bar configuration has an important rule; with the same waves acting, deeper closures occur when an outer bar is well-developed (Nicholls & Birkemeier, 1997).

At the annual time scale closure is still strongly related to cross-shore bar movement, but as time increases, beach-nearshore profile translation and shoreface processes come to control the location of closure (Hinton & Nicholls, 1998). These authors found significant changes seaward of the shoreward closure; that some profiles exhibit re-opening. It happens at longer scales than 10 years. A first closure is needed and usually is followed by the re-closure of the profile on the middle/lower shoreface.

At small and middle scales closure occurs on the upper shoreface, the most active zone, while in the middle and lower shoreface morphodynamic changes are weaker (Stive et al., 1990). There, changes are slow and steady, but at decadal scales this area becomes morphodynamically active and significant profile changes appear (Hinton & Nicholls, 1998; Hinton et al., 1999). These significant changes had previously been suggested (Niedoroda et al., 1995; Stive et al., 1990; Roelvink & Stive, 1990; Cowell et al., 1995; Stive & De Vriend, 1995). They were considered as a result of tidally dominated sediment transport at the most seaward boundary of the shoreface, and of wave-dominated sediment transport at the most shoreward of the middle shoreface.

These changes seaward of the estimated closure suppose the re-opening of profiles at time scales that make them important from a coastal management point of view. Over decadal and longer time scales re-opening must be taken into consideration, specially since it is known that it is temporally and spatially dependant (Hinton & Nicholls, 1998; Hinton et al., 1999). This dependence was understood as a consequence of slow steady change, due to cumulative process rather than to infrequent extreme events.

In the majority of cases where re-opening has been observed it could be associated with shoreface erosion (Hinton et al., 1999).

Knowing the importance of large-scale processes related to closure, it can not be forgotten in sand extraction studies. The characterisation of large-scale coastal behaviour is needed to determine dredging long-term effects.

But large-scale coastal behaviour is still a difficult problem. It is needed to identify, analyse and quantify large-scale variations in the position of the shoreline and the nearshore bottom, and this means to go far back into the past to obtain enough data to work with. There are empirical and qualitative techniques that can be used to define this seaward limit at this scale. They do not only include mechanics, but also sedimentology, stratigraphy, geology, climatology and history as it was concluded in the Colloquium on Large Scale Coastal Behaviour (Terwindt & Battjes, 1990).

Since it is still difficult to have information to assure a large-scale depth of closure, its optimal assessment should be done using wave conditions of estremal climate. If the effect of storms is added by considering the wave height associated to a return period of 20 years, the minimal depth where sand can be extracted increases to a value that is approximately twice Hallermeier's depth of closure. If a return period of 50 years is also used this depth still increases more (Table 4.1).

As a final conclusion, if the Hallermeier criterion is used to estimate the limit depth where the trench can be dredged, the minimum wave characteristics that should be used will be those associated with a return period of about 50 years.

Location	$d_{l,0.137}$	$d_{l,20}$	$d_{l,50}$	Location	$d_{l,0.137}$	$d_{l,20}$	$d_{l,50}$
I (Bilbao)	9.6	18.2	22.2	VI	6.1	11.1	14.3
I (Gijón)	8.7	16.3	19.6	VII (Alicante)	5.2	8.9	12.1
II	11.8	21.4	25.8	VII (València)	4.9	8.4	10.9
III	11.8	20.7	25.8	VIII (Roses)	7.6	(-)	(-)
IV (Sevilla)	5.3	10.6	12.9	VIII (Palamós)	6.4	12.0	16.9
IV (Cádiz)	6.5	13.6	17.8	IX	6.1	12.7	15.9
V (Málaga)	5.9	9.5	12.6	X (Las Palmas)	6.5	13.5	17.1
V (Ceuta)	6.1	11.4	17.0	X (Tenerife)	4.3	6.3	7.9

Table 4.1: Minimal depth where trenches should be done considering just the infilling effect for the sediment put into motion by profile's cross-shore changes along the Spanish coast; $d_{l,0.137}$, obtained with Hallermeier's equation (1981) using the wave height that is not exceeded more than 12 hours per year according to the wave climate from ROM 0.3-91; $d_{l,20}$, idem but using the wave height associated to the return period of 20 years; $d_{l,50}$ idem again but for 50 years (adapted from Jiménez, 1997).

With a process-based model that considers cross-shore transport, it could be possible to represent the beach drawdown. At least that is what can be concluded after study the behaviour of beach profiles with trenches by UNIBEST-TC, one of this kind of models. This is explained with more detail in the next chapter, 4.2.

4.2- Trench propagation:

As it was explained in chapter 2.5, the modification of the wave velocity field due to the presence of a trench can induce a modification of the local transport pattern and, in consequrnce, induce the propagation of the hole. In this chapter, this potential effect on coastal stability due to this morphodynamic interaction is analysed by using a process-based beach profile model, UNIBEST-TC, in which the only considered sediment transport is the cross-shore one.

The use of a profile evolution model by only considering the cross-shore transport is a simplification of the real dynamics since in nature, processes generating beach morphodynamics are essentially 3-D. However, by restricting the analysis to a 2DV approach we can solve one of the "component of the process" (e.g. by assuming that processes and changes can be split up into long-shore and cross-shore ones to be afterwards integrated). Moreover, if ambient currents are neglected and waves are considered to be dominant, it is expected that the cross-shore direction will be dominant.

In general terms, the evolution of the beach can be expressed in form of the continuity equation in the cross-shore direction as:

$$\frac{\partial z}{\partial t} + \frac{\partial S_x}{\partial x} = 0$$

(Eq.4.2.1)

what means that changes in the morphology are given by the gradient in the cross-shore sediment transport. This implies to assume that the sandy coast may be considered locally uniform in the alongshore direction.

Here, the capability of the model to simulate this process is assessed by analysing a set of laboratory experiments on the effects of trenches on beach profile stability done by Migniot & Viguier (1980). A detailed description of the experiment including set-up and results is presented in chapter 3.2.

First, a description of the model is presented as well as a comparison between two existing different versions to put in context the selected one.

4.2.1- UNIBEST model:

UNIBEST-TC, which stands for UNIform BEach Sediment Transport - Time-dependent Cross-shore, is a process-based model with two existing versions based on the use of different formulations.

The version 1.10 is based on the model developed by Stive (1985). It uses the approach of Bailard (1981) to estimate the cross-shore transport so it assumes that the instantaneous transport is proportional to some power of the instantaneous near-bottom velocity. Bailard extended the following generic description of sediment transport with the effect of a bottom slope:

$$q(t) = Au(t) / u(t) /^n$$
(Eq.4.2.2)

and distinguished between bed load transport in a granular-fluid shear layer of a thickness in the order of the wave boundary layer and suspended transport in a layer of greater thickness (in the order of several centimetres). The resulting equation was:

$$\langle i \rangle = \rho c_f \frac{\varepsilon_b}{\tan\phi} [\langle /u/^2 u \rangle - \frac{\tan\beta}{\tan\phi} \langle /u/^3 \rangle] + \rho c_f \frac{\varepsilon_s}{w} [\langle /u/^3 u \rangle - \frac{\varepsilon_s}{w} \tan\beta \langle /u/^5 \rangle]$$
(Eq.4.2.3)

where i is the total cross-shore immersed weight sediment transport rate, ρ is the water density, c_f is the drag coefficient for the bed, ϕ is the internal angle of friction of the sediment, $\tan\beta$ is the slope of the bed, w is the sediment's fall velocity and ε_B and ε_S are bed load and suspended efficiencies, respectively.

The above formulation uses vertically integrated equations and it considers that the sediment transport responds in an instantaneous, quasi-steady manner to the near bottom water velocity (i.e. there is no lag between sediment response to near-bottom velocity).

This assumption is probably valid for bed load transport because of the small thickness of the bed load layer that lets sediment respond quickly to the instantaneous shear stress. But the suspended sediment transport is distributed in a layer of greater thickness. The characteristic time constant for this layer is the ratio of its thickness and the sediment fall velocity. Therefore, the quasi-steady assumption is reasonable just for natural beaches with prevailing plane bed conditions and incident wave periods of 5-10 seconds (from the UNIBEST version 1 manual, 1992).

Knowing that this model had some limitations and since there were more studies and formulations in hydrodynamics and morphology, some modifications were implemented. The most important change in the model was due to the study for "Rijkswaterstaat" RIKZ (van Rijn et al., 1995) to predict the yearly averaged cross-shore and longshore sediment transport rates for several cross-shore profiles along the closed part of the Dutch coast.

The results differed from the ones obtained for a similar study carried out in 1989 that was based on the Bailard-Bagnold transport formulation.

Based on this study the version 2.0 was released. In this version there is:

- a new sediment transport formulation according to Van Rijn et al. (1995)

- a consistent treatment of the cross-shore and longshore vertical velocity, using parametric viscosity distributions

    - the inclusion of wind-driven currents

    - the inclusion of the surface roller contribution in the momentum balance

    - the inclusion of breaker delay in wave energy decay model

But the change that yields the biggest difference between both versions is the new sediment transport formulation.

There are used two different formulations to calculate each kind of transport but afterwards total transport rate is calculated as the sum of the bed and suspended ones.

The bed load transport is obtained from the formulation of Meyer-Peter-Muller (1948) in its generalised version:

$$\frac{q_b(t)}{\sqrt{\Delta g d_{50}^3}} = 9.1 \frac{\beta_s}{(1-p)} \left(\left|\theta'(t)\right| - \theta_c\right)^{1.8} \frac{\theta'(t)}{\left|\theta'(t)\right|}$$

(Eq.4.2.4)

where the concept of initiation of motion is used. q_b is the bed load transport rate in volume per unit time and width including pores, d_{50} the median grain diameter,Δ the relative density $((\rho_s-\rho)/\rho;$ ρ_s is the sediment's density and ρ water's density), p the porosity of the sediment, g gravity acceleration, θ' the dimensionless effective shear stress, θ_{cr} the dimensionless critical shear stress and β_s the slope factor.

The suspended sediment flux is approximated by the product of mean current and mean concentration verticals but it is assumed that the wave related suspended sediment transport is small as compared to the current related one so:

$$q_s = q_{s,c} = \int_a^h vc\,dz$$

(Eq.4.2.5)

where v is the time and space-averaged fluid velocity at height z above the bed, c is the time and space-averaged sediment concentration at height z, h the water depth and a the thickness of the bed load layer. With this assumption the onshore wave-asymmetry transport is not considered in the new version so the suspended transport will always appear in seawards direction.

The behaviour of the bed load sediment transport is the same as it was obtained by the old version of UNIBEST as a total sediment transport (Fig.4.2.1). But there is more offshore-directed transport because the total one is mostly led by the suspended that now is computed separately.

With this new version the suspended sediment transport is computed only outside the bottom boundary layer. Then the turbulence that exists in the wave boundary layer is not considered. This turbulence can be considered as negligible in relative terms inside the surf zone but not outside. Under non-breaking waves, processes in the bottom boundary layer are the ones not only inducing bed load transport but also controlling sediment suspension.

4.2.2- Contrasting UNIBEST versions:

If the behaviour of both UNIBEST's versions under the same conditions and without trenches is analysed, some differences can be observed in the magnitude of the transport and in its direction due to the energy dissipation breaking. This will induce a series of different modifications of the inner part of the profile despite the fact that in both representations there appears a bar migrating seawards.

If the transport pattern is observed in detail after the first time-step both two approaches give a different result (Fig.4.2.2 & 4.2.3):

- The Van Rijn's approach (new version) predicts at the initial stage a very large offshore transport in the surfzone with the maximum value at the location where the maximum energy dissipation occurs. This large transport is also inducing a very large gradient that will result in a suddenly development of a bar.

This offshore transport continues with time although there is a decrease in transport rates and, at the same time, the maximum transport is attained in a wider zone where an almost constant sediment transport verifies. However, the transport gradient seawards of this point is still large, so this will imply a continuous bar growth and a seaward migration of the bar following the migration of the zone where the gradient verifies.

The resulting morphological evolution is shown in Figure 4.2.3 where a continuous development of the bar is seen (growing bar and seaward migration).

These changes are due to the difference of magnitudes between the bed load sediment transport and the suspended one (see e.g. Fig.4.2.1). Although there is onshore bed load transport, there is not any significant onshore-suspended transport landwards of the breaking point so the total transport is directed seawards in the surf zone.

This difference between both kinds of sediment transport is due to the slope. In this case a quite steep slope has been used. With milder slope profiles both kind of transports have the same order of magnitude. The difference is mainly due to the suspended transport; the bottom transport just shows some local differences on the bar's head with different slopes.

- The Bailard's approach (old version) gives a similar pattern inside the surfzone although the magnitude is much lower. Moreover, the dimension of the zone where offshore transport exists is narrower and the resulting gradient seawards of it is smaller. This reduction in the gradient results in a lower bar and also slower migration rates. As the bar is formed by sediment eroded from the inner part of the profile, this lower bar represents a smaller erosion of the beach.

Outside the surfzone both two models give very different results. Thus, the old version predicts a slightly increasing onshore transport in the shoreward direction whereas the new one predicts a faster increase of such transport. Moreover, the pattern is also different because the new version predicts a peak in the onshore transport in the lowest part of the profile without having a strong physical reasoning for such behaviour.

4.2.3- Laboratory tests of Migniot & Viguier:

To validate, at least in a qualitative sense, the UNIBEST model the laboratory experiments of Migniot & Viguier (1980) were used.

These studies were done to find out the influence of dredging on the sea bottom equilibrium and particularly, on the shoreline equilibrium as it is explained in chapter 3.2.

Since UNIBEST-TC is a module that just considers the cross-shore development, the French flume test results have been used because they only represent changes due to the cross-shore transport. They are basically qualitative but they let see the infilling as well as define the critical wave height that triggers the beginning of this process.

In the French experiments the profile and the waves typical of the Bay of Biscay were used, but reduced by scale factors. That has not been done for the UNIBEST input data so the real conditions were used. In the cases in which the real values were not known the default and advised values have been used. Just the wave breaking parameter to determine maximum local wave height has been changed in the version 1.10 to use the same as in the version 2.0. This parameter has been calculated with the calibration of Battjes and Stive (1985). Setting the other wave breaking parameter used in dissipation formulation equal to 1:

$$\gamma = 0.5 + 0.4\tanh(33s_o) \tag{Eq.4.2.6}$$

$$s_o = H_{rms} / L_o \tag{Eq.4.2.7}$$

Trenches in the flume represented real trapezoides of 5 meters deep under the sea bottom, a longest base of 140 m and a shortest base of 80 m. This implies an extraction of

550 m3/m linear. These trenches were subjected to a cycle of waves as presented in Table 4.2.1:

Period: Tp (s)	Height: Hrms (m)	Duration (days)
9.91	1.12	32
10.32	1.47	32
10.56	1.68	32
10.76	1.86	32
10.99	2.06	32
11.17	2.21	32
11.60	2.59	32
12.02	2.95	32
12.45	3.33	32
12.86	3.68	32
13.31	4.07	32
13.71	4.42	32

Table 4.2.1: waves cycle used in the flume test without the scale factors applied.

It has to be stressed that this cycle is not strictly representing the real waves cycle that acts in the Bay of Biscay during a year. And in fact, this climate will represent a extremely energetic climate and in consequence, if the trench configuration "pass the exam" it will be possible to assure that no interaction will occur.

These flume tests were really useful to know the behaviour of the sediment deposits in the trenches. These deposits mainly occurred by softening of the landward trench slope and generated erosions in shallow bottoms and, consequently, on the beach. This behaviour was always the same; the changes on the trench geometry and the sediment dynamics for the filling were the same for any trench's depth. The difference was just that for different trenches in different depths the same state was reached for a different wave's height. They distinguished 8 states until the pit disappeared and the sea bottom got a new equilibrium.

The results obtained from these qualitative studies about the filling process of a trench must be considered local results because the influence of the refraction can not be considered.

There are the same limitations with the UNIBEST modules so it seems that the results with trenches should be similar. But even using the same profile, the same tidal and the

same waves (Fig.4.2.4), some differences can be seen between both UNIBEST versions but also between these versions and the flume test's results.

The most obvious difference is the existence of sediment bar. There is no bar in the French results while this is very clear in the UNIBEST's profile development, especially for the version 2.0.

To achieve consistent results with the new UNIBEST, trenches of just 2.5 meters deep and with wide of 160 and 280 have been used since the section's area is the same (its width was increased to let the model "feel" the trench).

In Figures 4.2.5 and 4.2.6 the development of the profile with a trench in -12 meters depth after being subjected to the above cycle, is shown. Figure 4.2.5 is the result with the old version while Figure 4.2.6 is the result with the new one. It is easy to see that the general behaviour is quite similar but the magnitudes are completely different. Since that was known because of the previous comparison with smooth profiles, the local evolution of the trench before the bar arrives on it has been observed and different states at the end of each wave's action have been recognised.

At this depth the bar still has some influence over the trench's evolution with the new version so the behaviour was not the expected, either in the first months of the cycle. Just after the first wave it is possible to see more erosion on the landward side of the trench than on the seaside, while with the old UNIBEST at least the states appear to be comparable with the flume results. Moreover, the beginning of the filling in the trench can be considered at the same point (t131, Hrms=1.86m) while with the new version there is something before. The theoretical starting point is at the same step as the falling of the bar inside the pit in the new UNIBEST.

When the bar is farther from the trench (trench at -20m), there is more time to notice the erosion of the landward slope of the trench and the deposition of sediment in the pit before the bar influences. But, as before, the theoretical beginning of the filling (when there act the waves of Hrms=2.95m, between t230 and t263) is the arrival of the bar in the trench with version 2.20 while is mostly the same moment for the old one (Fig.4.2.7 & 4.2.8).

It seems that the results with the first version of the UNIBEST are more like the theoretical development. But with these trenches, although similar states exist, is not possible to achieve exactly the same results because they are not the trenches they used in the flume tests.

While it was not possible with the new version, the old one lets use the real trenches, narrower and deeper than the ones used in the previous figures. In Figure 4.2.9 the evolution of the test's trench in -20 meters depth is represented. There are the local effects that the French found in the flume tests but not the same deposition of sediment.

Anyway, since these studies were qualitative studies more than quantitative, the old version can be considered a good approximation. By using these kind of models it is tried to represent processes more than exact calibrations. Thus, since the magnitude is acceptable, this version seems to be able to give coherent results, at least from the qualitative standpoint.

4.2.4- Hindcast capabilities of UNIBEST-TC:

The actual cause of trench propagation is the existence of sediment transport gradients that appear due to the local velocity fields modification (see chapter 2.5). This modification is a direct consequence of the arrival of the waves on the trench, of the changes on the local hydrodynamics. The presence of a pit yield local effects in the wave height so there is a variation in the square orbital velocity (Fig.4.2.10), a local reduction of it that leads a local change in the sediment transport. This is something that happens whatever the profile or the waves are, but the question is to find out which conditions can affect the nearshore zone.

If it is studied the evolution of the trench in 20 m depth with the French studies slope (tgα=0.015), the changes are local and the profile's development is just different from the evolution without trench near it. In Figure 4.2.11 there is a comparison of the profile's development in some time points: after one day, when the filling of the pit is supposed to start (according with the French results) and at the end of the cycle. The profile on the beach is not influenced by the extraction, and neither is the breaker bar. The bar is different just when it arrives at the trench because of the falling of part of the sediment in the pit.

But this evolution is just like that because the trench is far from depths where the waves break. The bar catches the pit at the end of the cycle so on one hand this study is good to compare the UNIBEST results with the French ones because there is not bar around the trench, but on the other hand, it is not enough to conclude that a sand extraction will lead only local effects. The result shown in Figure 4.2.11 just lets say that while the bar is not over the trench the development of the profile will be the same as if the pit does not exist. If the development of the same slope is observed with a trench in 7 meters depth (Fig. 4.2.12), it is seen how the erosion of the shore is different because of the pit after the arrival of the bar in it.

The slope used (tgα=0.015) yields a development of the profile that is never able to represent the propagation of the trench. However the model represents quite well the same behaviour (a part from the bar) as the one obtained in the flume tests.

The general behaviour of sediment transport with this slope is always onshore till the breaking point and after, inside the surf zone, it is basically offshore. The bar marks the changing point between both directions. This general development does not change because of the trench, but when the bar falls in it there are some modifications in the profile's evolution: the bar seems smaller because part of it is filling the trench and there is more erosion on the beach.

The behaviour showed by UNIBEST seems the consequence of storms so it is not useful to predict a real evolution of a beach. There some accretion wave climate will act sometime provoking changes that the model can not represent with this slope. It is not possible to assure which will be the real profile in the Bay of Biscay if some trench is done but at least it is possible to know that while trenches are in bottoms deeper than -20 meters no changes will appear on the beach because of them.

As the French results, the ones obtained using the same conditions with the UNIBEST just help forecast filling conditions of dredging trenches in sea and to establish limit depths for dredging with respect to local oceanographical conditions so that consequences on the shoreline are negligible. UNIBEST is useful to know if the pit will affect or not the beach but not to represent the real shoreline erosion. From the beginning it was known that the evolution of the beach profile would not be the same along the entire beach since trenches are not so long and refraction is not represented.

When a gentler slope is used it is possible to see some local trench propagation (Fig.4.2.13) especially with long wave periods. The changes in the extraction pit are as they were supposed and found in other studies.

The unsteady accretion observed on the shore is the same that appeared without trench. After just one month of action of this wave the behaviour is just a local behaviour that, although it is felt along approximately 1000 meters, does not reach the beach. In Figure 4.2.14 the cross-shore sediment transport with and without trench is represented for a wave period of 6 s and it is possible to see this local effect which does not lead a different development of the shoreline. It seems that the immediate effect of extraction on the wave climate is local and minor as it was said by van Alphen et al. (1990, see chapter 3.7). They studied the different morphodynamics of extraction pits on the Dutch coast and they used a one-dimensional morphodynamic model to compute the cross-shore development. This model can be considered the basis of UNIBEST and used Bailard

formulation as well. But these results could not be verified in the field even though they are the same as Migniot&Viguier.

In this Dutch report, which is explained in chapter 3.7, they also studied the long-term effects, at a large time-scale. They found that due to the small net onshore sediment transport that exists on the Dutch shoreface, deposition on the offshore margin and erosion of the landward boundary leads to an onshore migration of the pit. But this is not possible to achieve with the UNIBEST because there is no onshore constant current represented and because a real climate wave has not been used. If waves as the ones they used are tested with UNIBEST (H=1.86m, T=6s) and the gentle slope (tgα=0.005) during 200 days there are no more than local effects because of the trench and no propagation is seen (Fig 4.2.15 & 4.2.16).

It seems that perhaps with more energetic waves a larger propagation at long-term will exist, but when it is tried to compute the profile evolution of this gentle slope with the same wave climate as in the Bay of Biscay, the profile become unsteady after 164 days. UNIBEST shows here its limitation when waves with long periods act over gentle slopes.

It is just possible to confirm that this "propagation" will be faster if the slope is steeper and slower if with the same slope trenches are less deep.

In Figure 4.2.17 the same time steps are represented for trenches in depth -20 meters but with different depth and wide. As it was thought the local variations in the pit are stronger when the trench is deeper, due to the sediment transport gradients that are more important. But again there are not changes on the beach's evolution, the effects are almost local. The amount of sediment that fields the pit, while the bar does not affect it, comes basically from the slopes of itself. Just after the bar's arrival changes are not local anymore.

Anyway, although UNIBEST does not show real propagation of trenches, it is just possible to assume that there will not be an important propagation if they are located around or below the depth that represents the sediment significant movement limit during a mean year (see chapter 2.3).

If there is no sediment motion there is no sediment transport so there will not be significant changes in the bottom height. This restriction is, generally, stronger than the one given by the depth of closure. Depth of closure just assures that there will not be cross-shore changes in the profile, but there is sediment transport.

With the UNIBEST it is possible to see how trenches act as a sink for the sediment that is eroded from the near-shore zone, but with steep slopes and without any onshore constant current, the accretion that could rebuilt the beach profile can not be represented.

Just when there are gentle slopes and smooth waves some accretion appears on the upper part of the profile (Fig.4.2.15). In this sense, UNIBEST can be also used to assess the presence of beach drawdown since it can predict the fill of the trench with sediment eroded from the beach.

4.2.5- Conclusions:

At the beginning of this study it was already known that 2-D analysis can not represent the real behaviour on coastal stability after a sand extraction, but qualitative results could be obtained.

The most important difference between the two UNIBEST versions that have been used is due to the transport formulations. In the second version the wave-asymmetry transport is not considered so the suspended sediment transport is always directed seawards. This, added to the fact that suspended transport is quantitatively major than the bottom one gives a different total transport. As it is shown in Figure 4.2.1 and because of Bailard's formulation, total sediment transport with the old version is like the bottom sediment transport in the new one.

If it is studied the behaviour on gentle slopes a larger onshore and smaller offshore transport is observed with the old version than with the new one. With steep slopes the onshore total transport arrives closer to the shoreline with the old version and afterwards there is smaller offshore transport than with the new version so there is less erosion on the beach (Fig.4.2.3).

The results obtained by the old UNIBEST version have been more similar to the French ones and it was selected as the one to test the trench influence.

Bar appears with the old version too while it does not in the flume tests. Due to this fact, to contrast results with French studies it is needed to look at time steps before the arrival of the bar to the extraction pit. Then the same local development can be seen; deposits mainly occur by softening of the landward trench slope and generate erosions in shallow bottoms and, consequently, on the beach.

Local changes yield changes on the sediment transport and they remain being local until the bar catches the pit and starts filling it. From this moment there is more erosion on the beach than when there is no trench.

But this erosion on the beach is not real. To know the real behaviour a 3-D study should be done. With the UNIBEST there is no refraction and this is an important effect; waves front change because of the presence of a pit and then there is some convergence and divergence of waves that yield different erosions along the beach (see chapter 4.3).

UNIBEST lets see if a trench will lead to erosion on the beach, but it can not determine in which way. It is useful to know if the pit will affect or not the beach but not to represent

the real shoreline erosion. Consequently, this model can not predict the real development of the Bay of Biscay but it is possible to assure that there will be no changes on the beach because of the trench while this was in bottoms deeper than -20 meters.

Using the same slope as in the French tests ($tg\alpha=0.015$) the same different states of the process of infilling have been found and the same wave critical height that triggers the beginning of this process as well. But it is not possible to represent trench propagation. The results seem the consequence of a storm so it can not predict the evolution that it would be even with a real climate wave.

Using a gentler slope ($tg\alpha=0.005$) it appears some trench propagation as an immediate effect that it is local and minor. Without any onshore constant current UNIBEST can not represent propagation of a trench until the shoreline, either in long-time scale.

The profile's development with different kind of trenches has been studied as well. Local variations in the pit are stronger when the trench is deeper but effects keep being local until the bar catches the trench.

As the French results, the ones obtained using the same conditions with the UNIBEST just help forecast filling conditions of dredging trenches in sea and to fix limit depths for dredging with respect to local oceanographical conditions so that consequences on the shoreline are negligible. In some way, and using a depth change criterion, UNIBEST could be used to determine depth of closure in some time-scales, e.g. it could provide a limit to closure during storms. If a real storm is introduced as wave climate in UNIBEST, i.e. high waves during a short period of time, depth of closure could be identified at the end of the bar, in the most seawards part of it or not so seawards depending on the criterion used.

Although it has been proved that UNIBEST can not represent trench propagation, it is an important possible effect when a trench is located nearshore.

By using the criterion of deep water it can be found the depth were waves start "feeling" the sea bottom. Landwards of this depth, the presence of a trench can induce a variation of wave's height that implies a gradient of velocity at both ends of the hole. This change in the local hydrodynamics supposes a gradient in the sediment transport (see chapter 2.5, Fig.2.5), which causes changes in the trench morphology (studied in this chapter through UNIBEST) and can also induce trench propagation.

These depths for different wave periods are presented in Table 4.2.2. Until T=8 s they represent periods of Mediterranean waves ((T_{med})$_{max}\cong 8$ s) and the rest are considered to generalise this approximation to oceans. The relations used are:

$$L_0 = gT^2/2\pi \qquad\qquad\qquad \text{(Eq.4.2.8)}$$
$$d_0 = 0.5L_0 \qquad\qquad\qquad \text{(Eq.4.2.9)}$$

T	L_0	d_0
4	24.9	12.5
6	56.1	28.1
8	99.8	49.9
10	156	78
15	350.9	175.4
20	623.9	311.9

Table 4.2.2: Depth where waves start to feel the sea bottom

These depths mean that trenches landwards of them can potentially propagate to the shoreline, and as trenches go to the coast the propagation is faster and therefore more dangerous for beach stability. For a given rate of migration in a certain depth, the time that trench needs to get the zone of morphodynamic interaction decreases when the slope of the beach profile is becoming steeper. It is due to both the decrease of the distance to go through and the increase of rate transport in the new locations that goes quickly while trench is propagating landward.

Trench propagation in the Mediterranean Sea or in the ocean, will present the same behaviour but with different values since wave heights are smaller in the Mediterranean. But if the same depths for trenches are considered, the velocity on the bottom will be minor than when ocean waves act (it is necessary to remind that $v=v(H,T,d)$). Then the possible propagation will be much slower and almost negligible in depths where for ocean conditions it would be important. In other words, the same rate of propagation will be at shallower depths in the Mediterranean than in the ocean.

If any of the depths of the Table 4.2.2 is compared with some UNIBEST result it is demonstrated that they are deeper (e.g. looking at Figure 4.2.2, where is shown the action of a wave of T=10.76 that it is comparable to T=10 in the table, even after 60 days and considering the new version that gives changes at deeper depths, variations are not observed seawards of 25 m depth, against the 78 m in the table). With UNIBEST something comparable to depth of closure is obtained while in Table 4.2.2 the depth considered is just the beginning of some interaction between waves and sea bottom.

To have an order of magnitude of this problem, in Figure 4.2.18 the sediment transport gradient that can be induced by velocity's field variation is shown. There, it has been

assumed that the transport is proportional with the third power of the velocity as in the Bailard formulation (1981). Then, a reduction of about 10% and 20% in the velocity would induce a sediment transport gradient of about 27.1% and 48.8% of the rate transport that would exist if there were no trench. The morphological effect of this gradient will depend on the transport magnitude, and it will be bigger as larger the magnitude is.
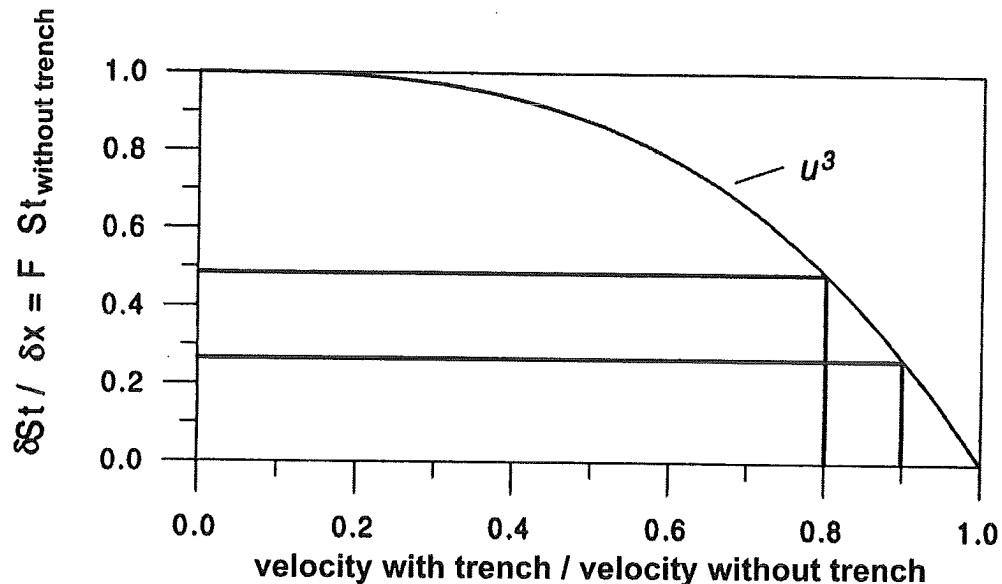
Figure 4.2.18: Sediment transport gradient induced by changes of the velocity field due to the presence of a trench assuming that the transport is proportional with the third power of the velocity (Jiménez, 1997).

Lastly, new and different methods have appeared to analyse the stability of a disturbance on the beach profile. Most of them have been developed to assess the behaviour of nourishment submerged bars, but they also should be useful to assess trenches behaviour since the mechanisms that lead their evolution are the same.

Hand & Allison developed one simple method to predict the stability or migration landwards of a nourishment based on the relations $(d_i-h)/d_l$ and $(d_i-h)/d_i$, where h is the depth where the nourishment is placed and d_l and d_i the Hallermeier (1981) depths for the significant vertical changes and the significant movement respectively.

Another method was proposed by Hands et al. (1996) based on the difference between velocity induced by waves and the threshold velocity for the initiation of sediment movement. Its main characteristic is that uses non-lineal wave theory to estimate the velocity field to characterise the shorewards transport.

Douglass (1995) developed and applied a convection-diffusion equation where the convection coefficient C and the diffusion coefficient D depend on the sediment transport rate in a given depth:

$$\frac{\partial h}{\partial t} + C\frac{\partial h}{\partial x} - D\frac{\partial^2 h}{\partial x^2} = 0 \qquad\qquad \text{(Eq.4.2.10)}$$

and where the movement of the nourishment to the coast, E[C(h)], is determined by:

$$E[C(h)] = \sum p(H,T)C(H,T,h) \qquad\qquad \text{(Eq.4.2.11)}$$

C(H,T,h) is the contribution to the movement at depth h of waves conditions (H,T) and p(H,T) is its presentation frequency.

To avoid trench propagation to the coast it is necessary to dredge such trenches that do not let local hydrodynamics to change more than a little proportion. This should make sediment transport rates variation almost negligible or little enough to assure that if there is trench migration, it will be really slow and it will not affect coastal stability in a century. A general criterion could be to not dredge shorewards of d_i (Eq.2.3; Hallermeier, 1980) which at the same time will avoid the interception of the onshore transport in the inner shelf (see chapter 2.3).

Figure 4.2.1: Comparison different transports
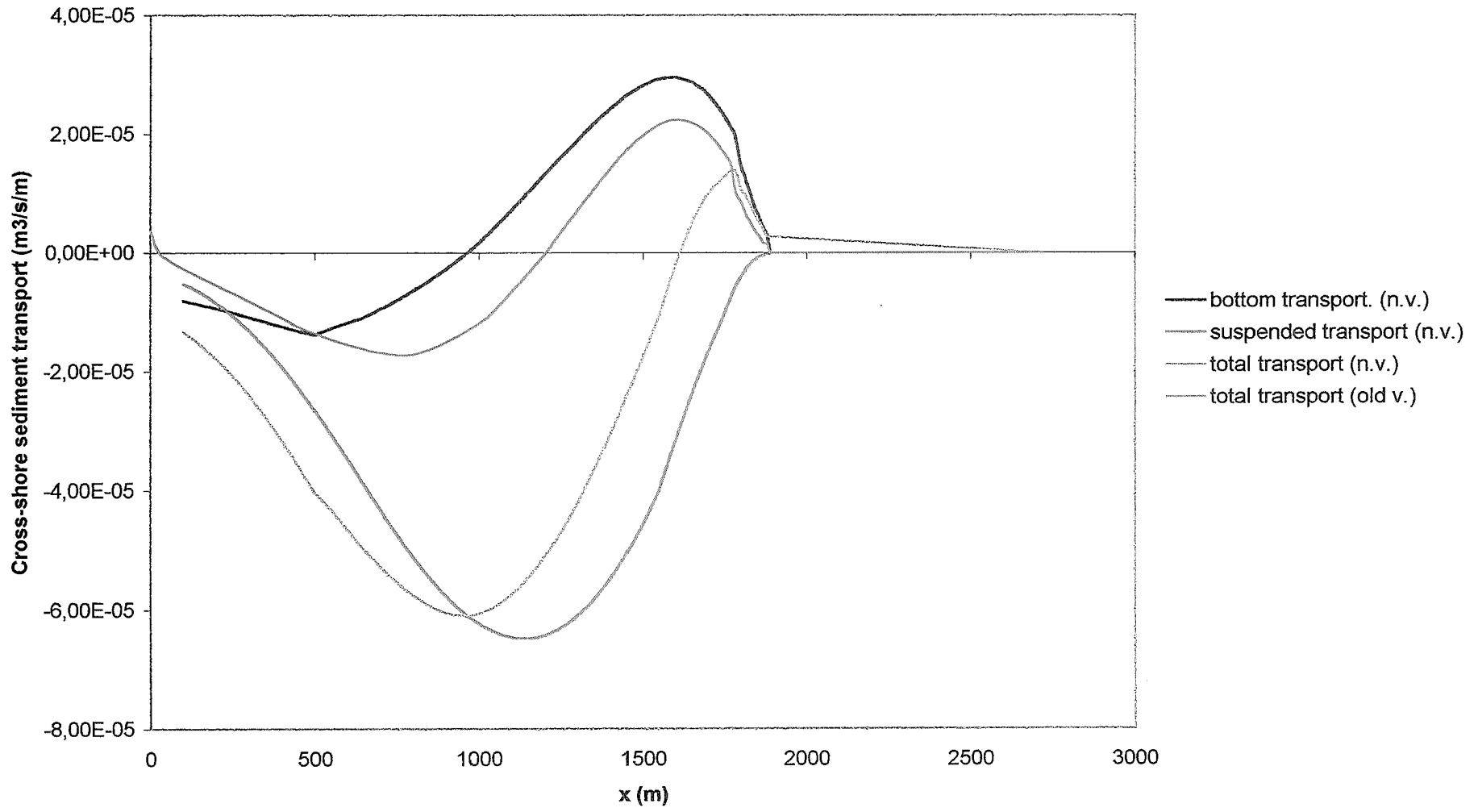
Figure 4.2.2: profile development (tg=0.015,H=1.86,T=10.76)
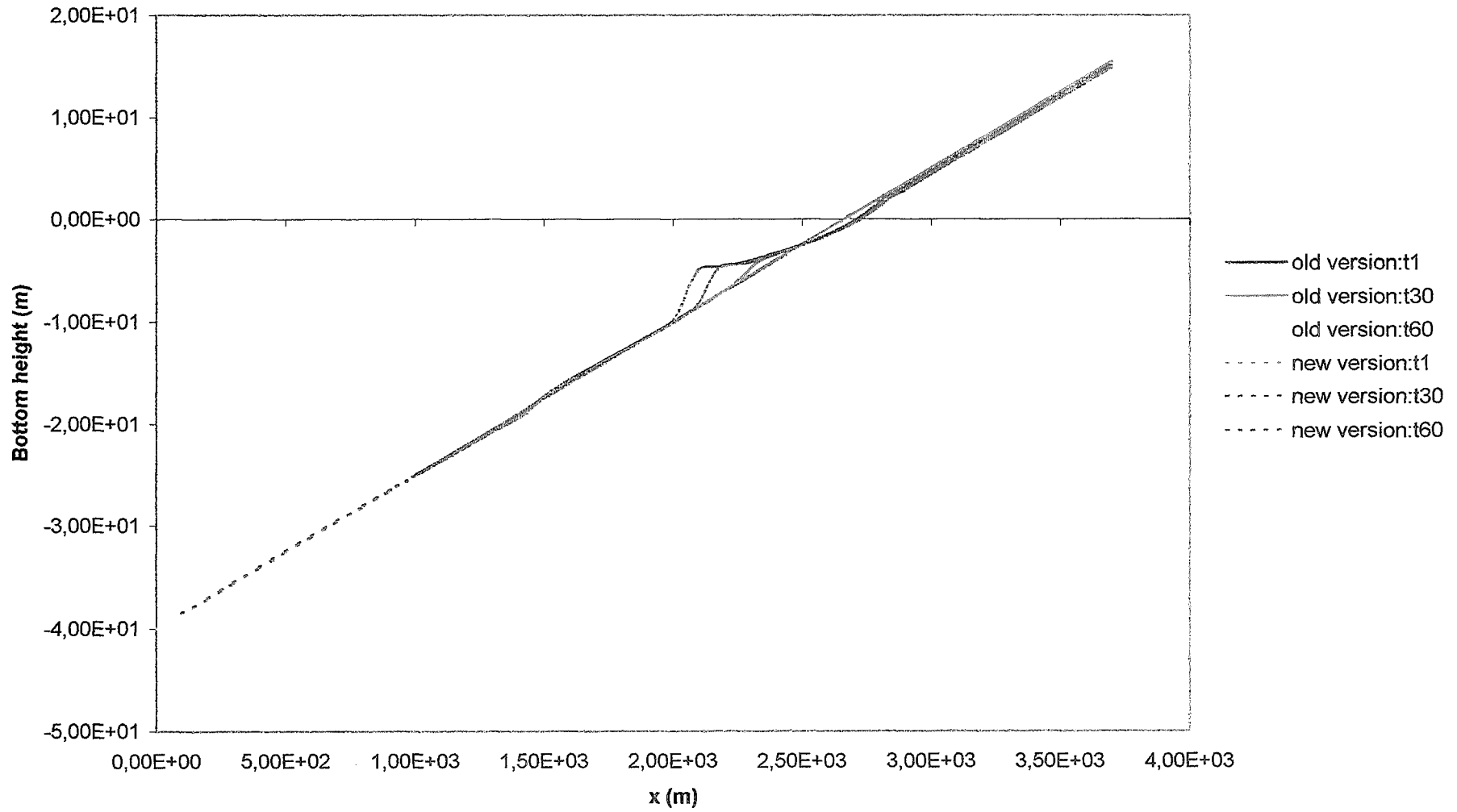
Figure 4.2.3: Sediment transport (tg=0.015,H=1.86,T=10.76)
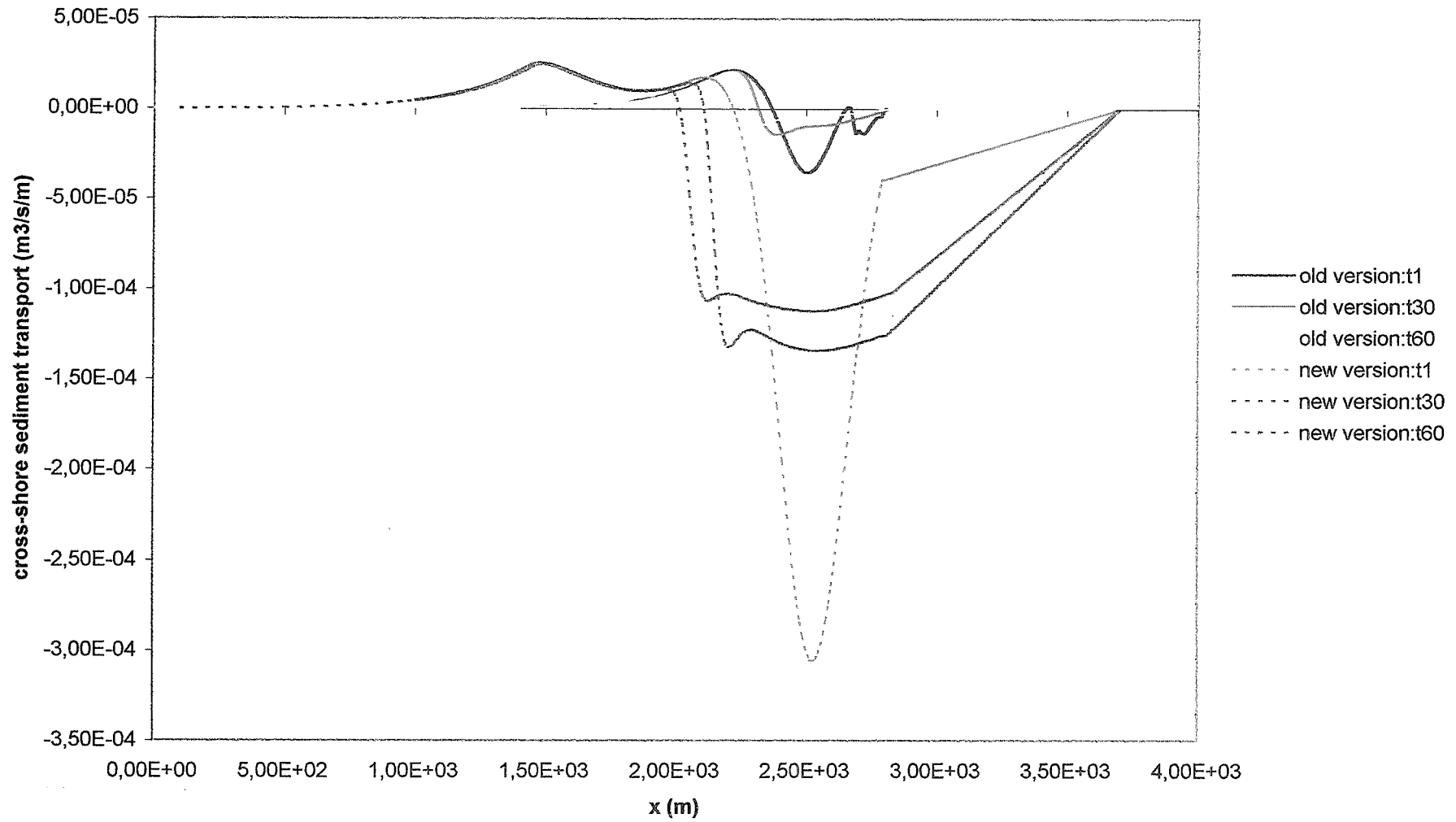
Figure 4.2.4: French flume studies conditions

Real conditions in the Bay of Biscay had been tried to reproduced in these studies and in the UNIBEST:

Slope: $tg\alpha= 0.015$

Sediment: French tests used sediment between 60 and 300 μm

                UNIBEST : $D_{50}= 225$ μm

Climate wave: cycle of waves showed in the report

                They had $H_{1/10}$ and in UNIBEST H_{rms} is needed so:

$$H_{rms} = \frac{H_{1/10}}{1.2\sqrt{2}}$$

                and the relation between periods and heights:

                $T_{mean}= H_{rms}+7.5$

                but in UNIBEST T_p is used:

                $T_p=1.15T_{mean}$

Tidal: it was represented by a sinusoidal law with amplitude

                of 3.65m

                between +4.30 m and +0.65 m

                mean see level: 2.45 m

Figure 4.2.5: Flume profile with trench at -12 m depth (old v.)

Nearshore sand extraction and coastal stability
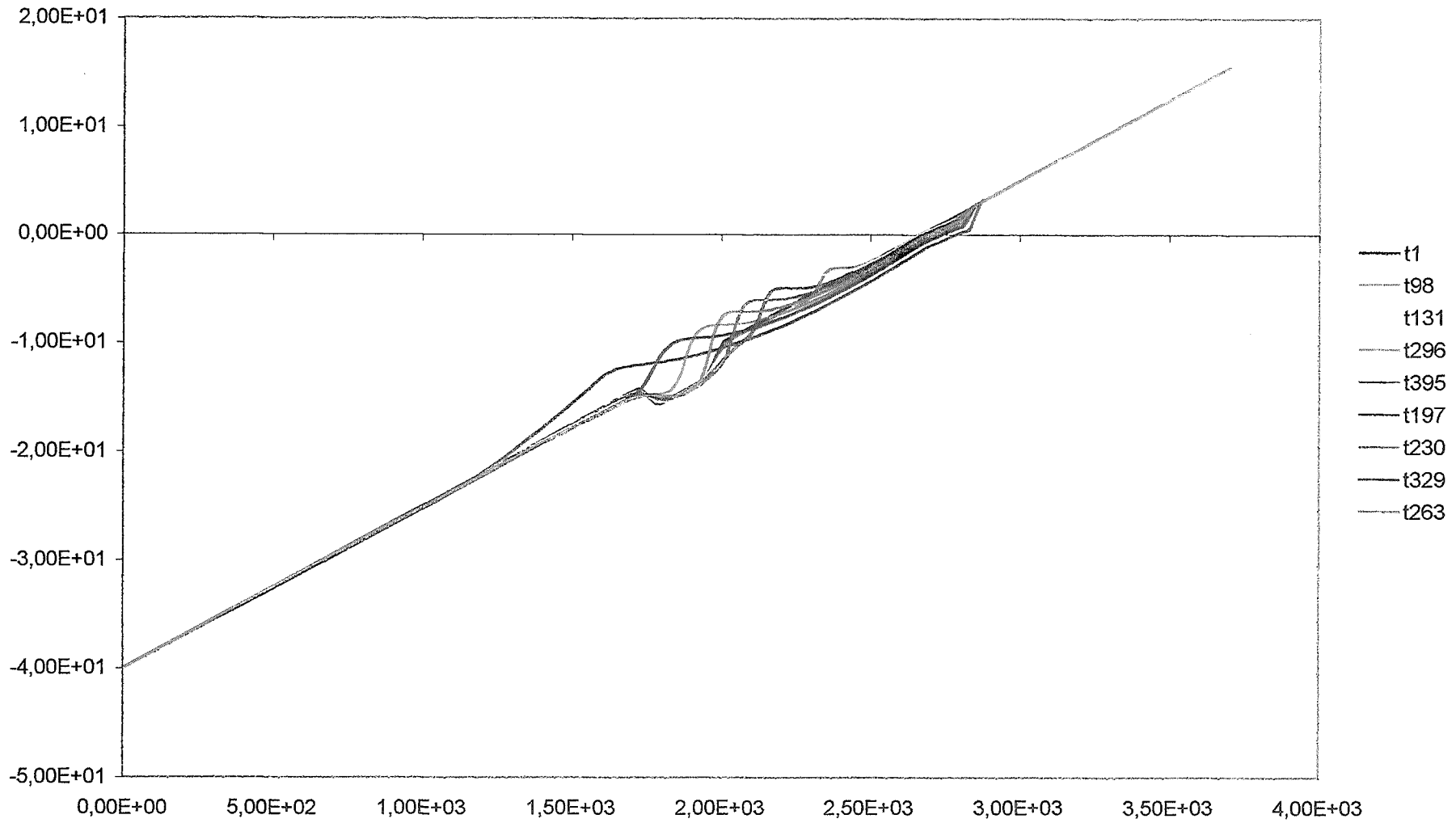
Figure 4.2.6: Flume profile with trench at -12 m depth (n.v.)
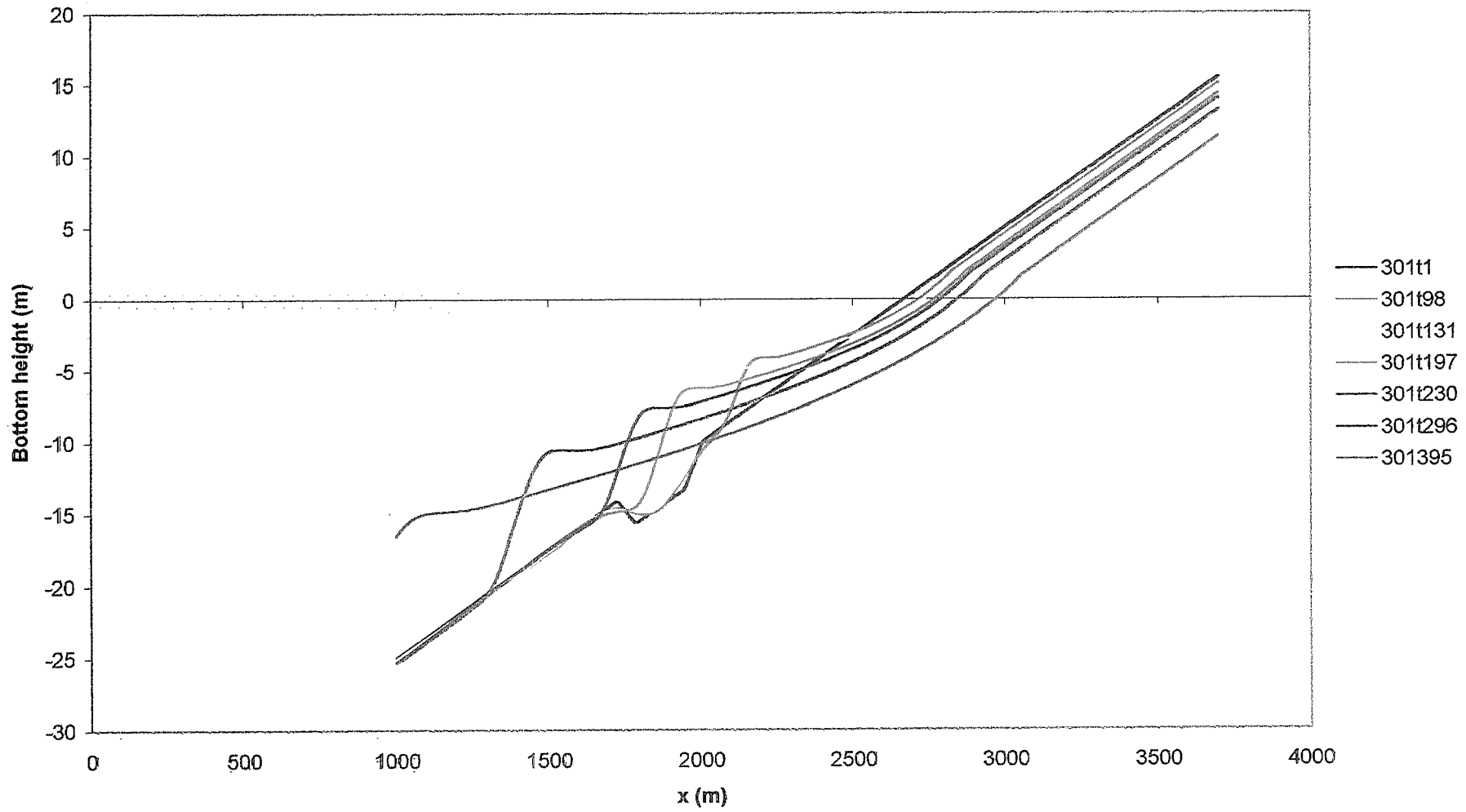
Figure 4.2.7: Theoretical beginning of the pit's filling
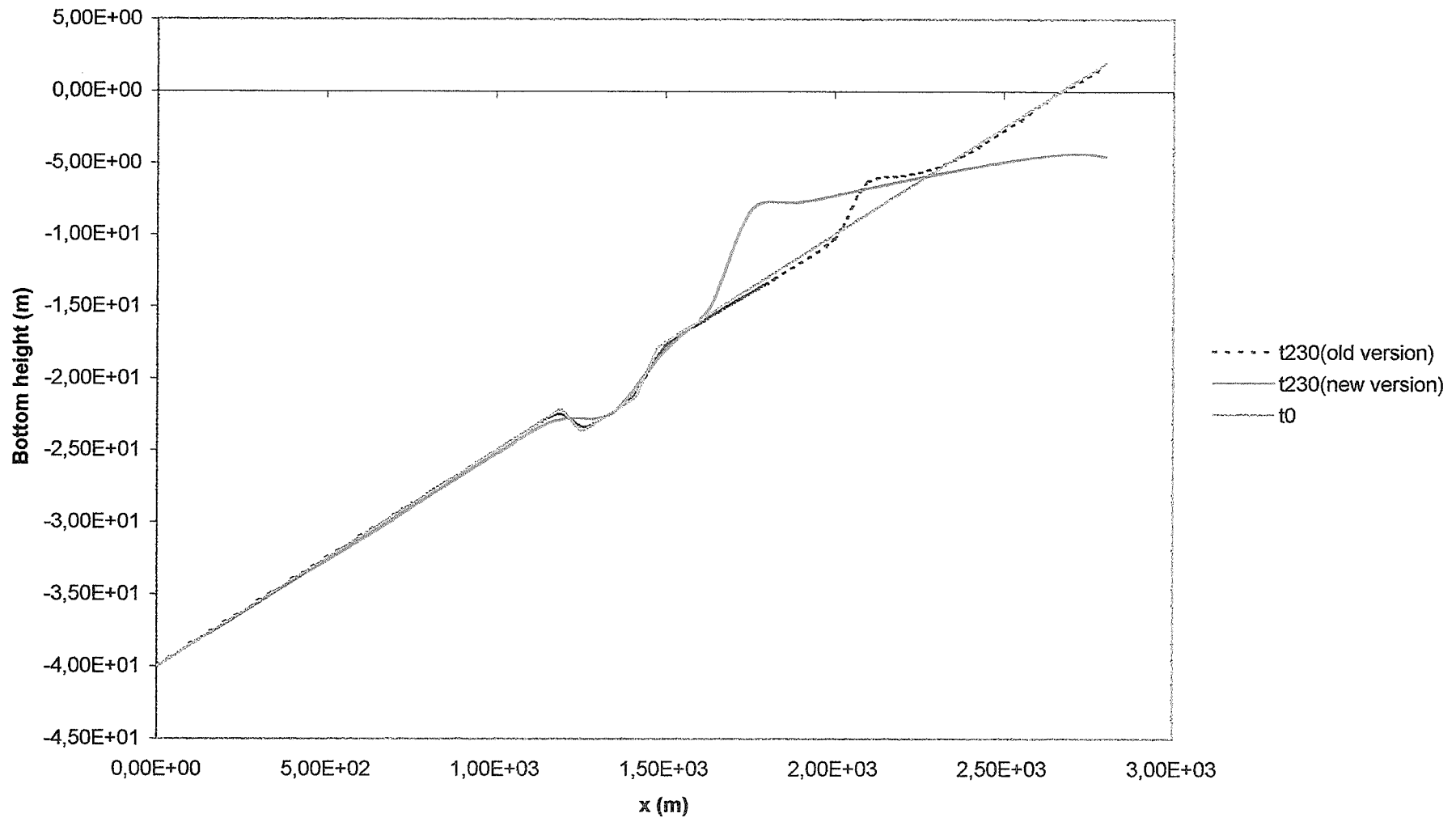
Figure 4.2.8: Beginning of the pit's filling
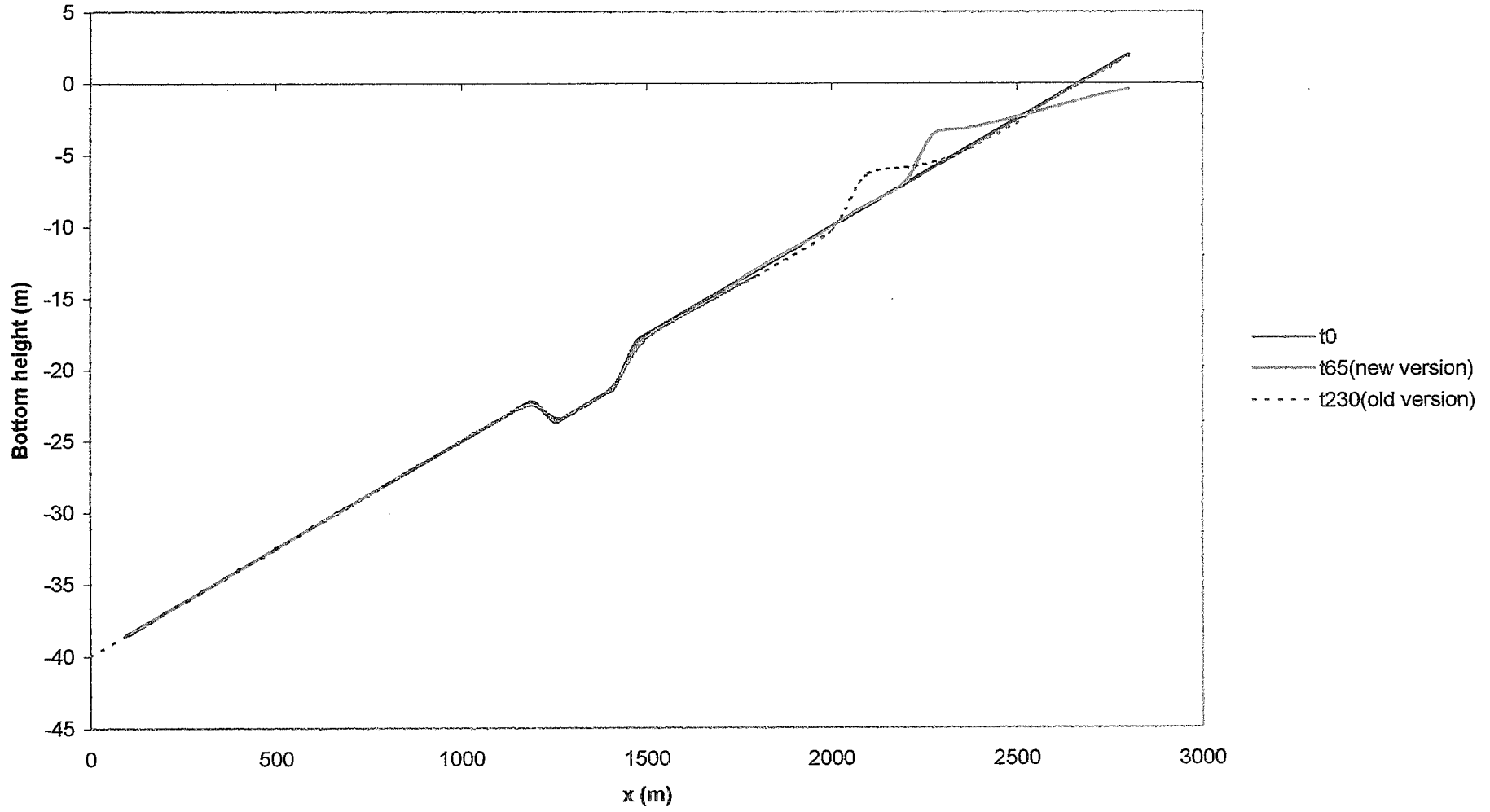
Figure 4.2.9: Flume profile with the real trench at -20 m depth
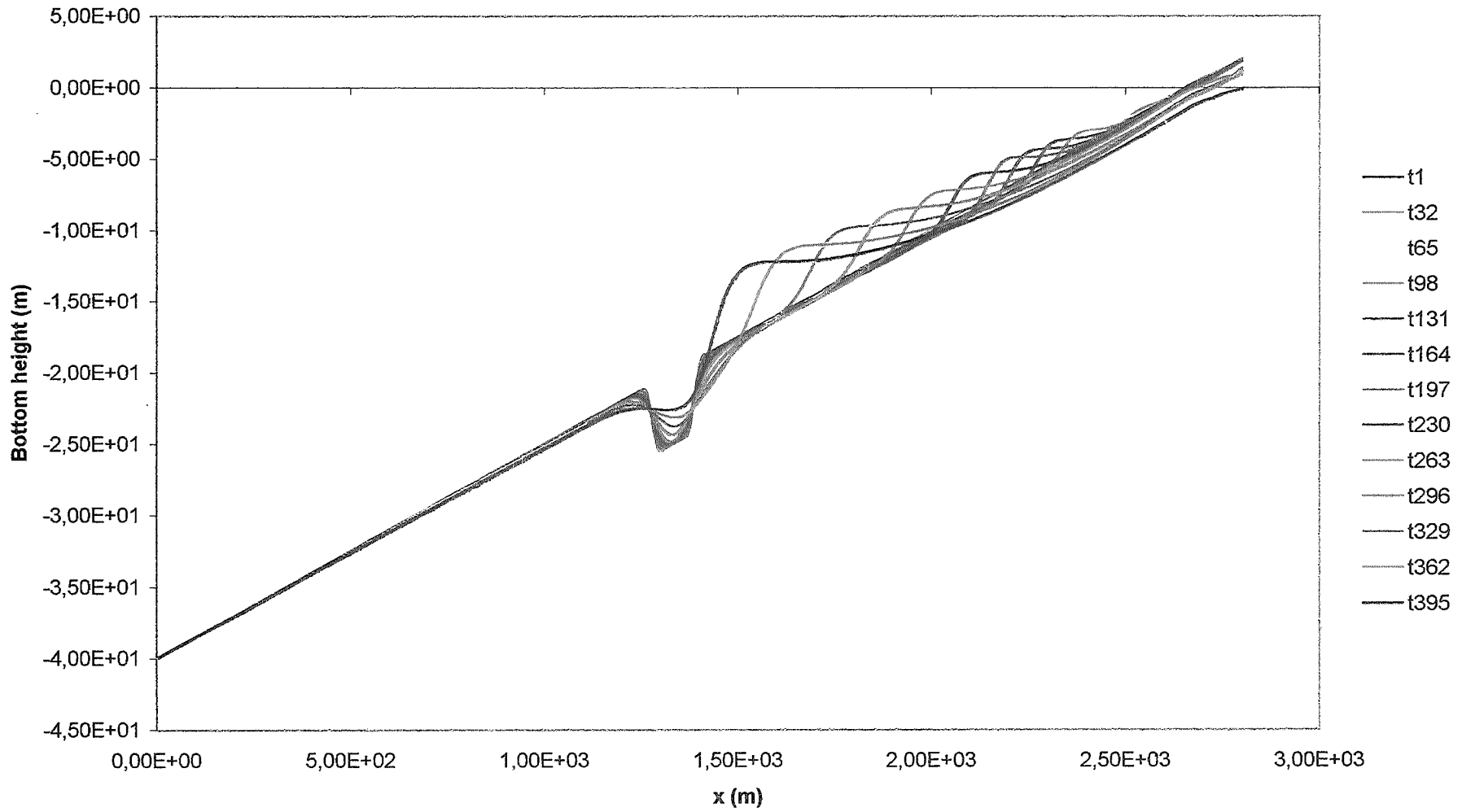
Figure 4.2.10: Root mean square orbital velocity (comparison: trench at -20m/no trench)
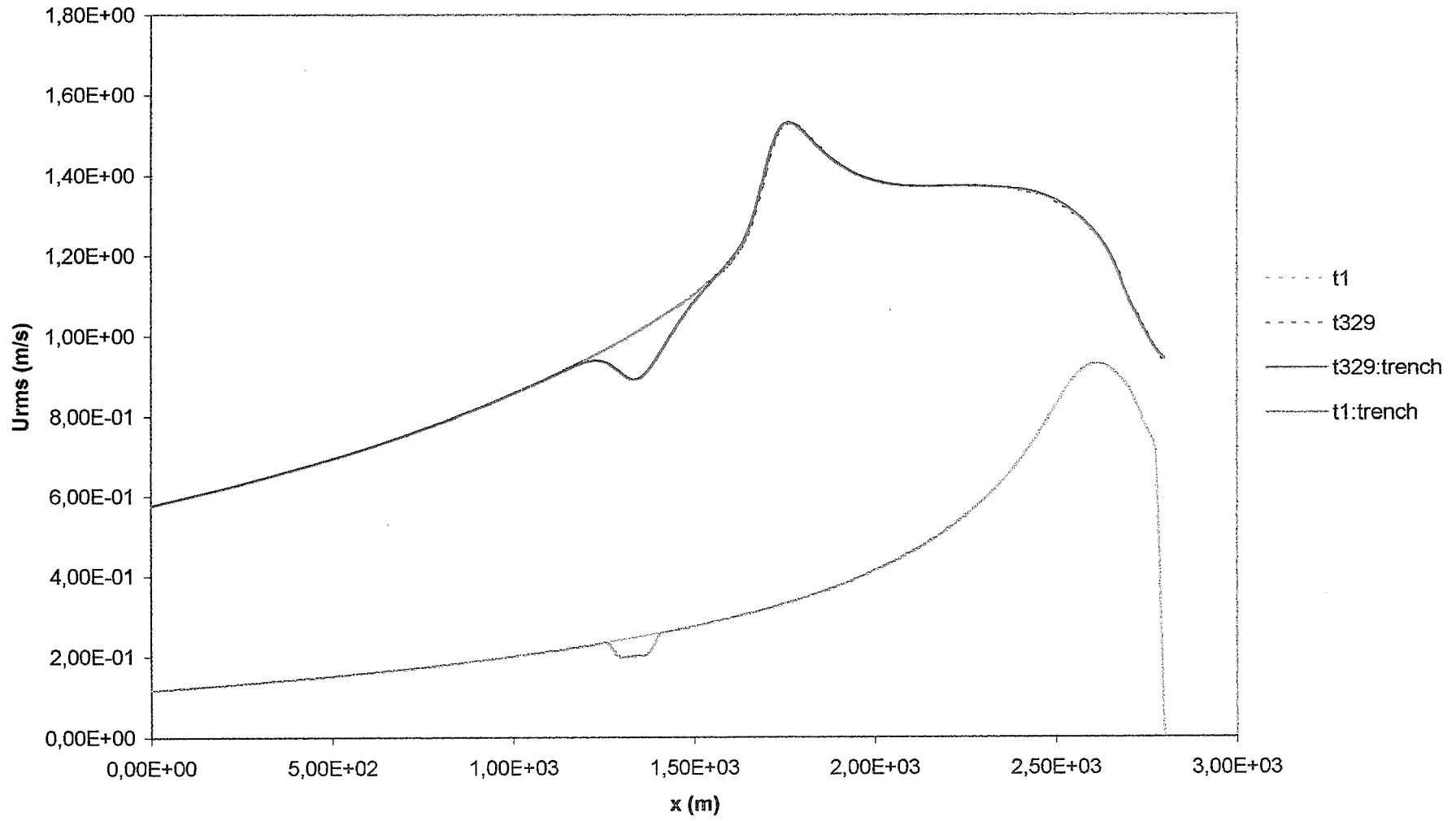
Figure 4.2.11: Profile's comparison (trench at -20m/no trench)
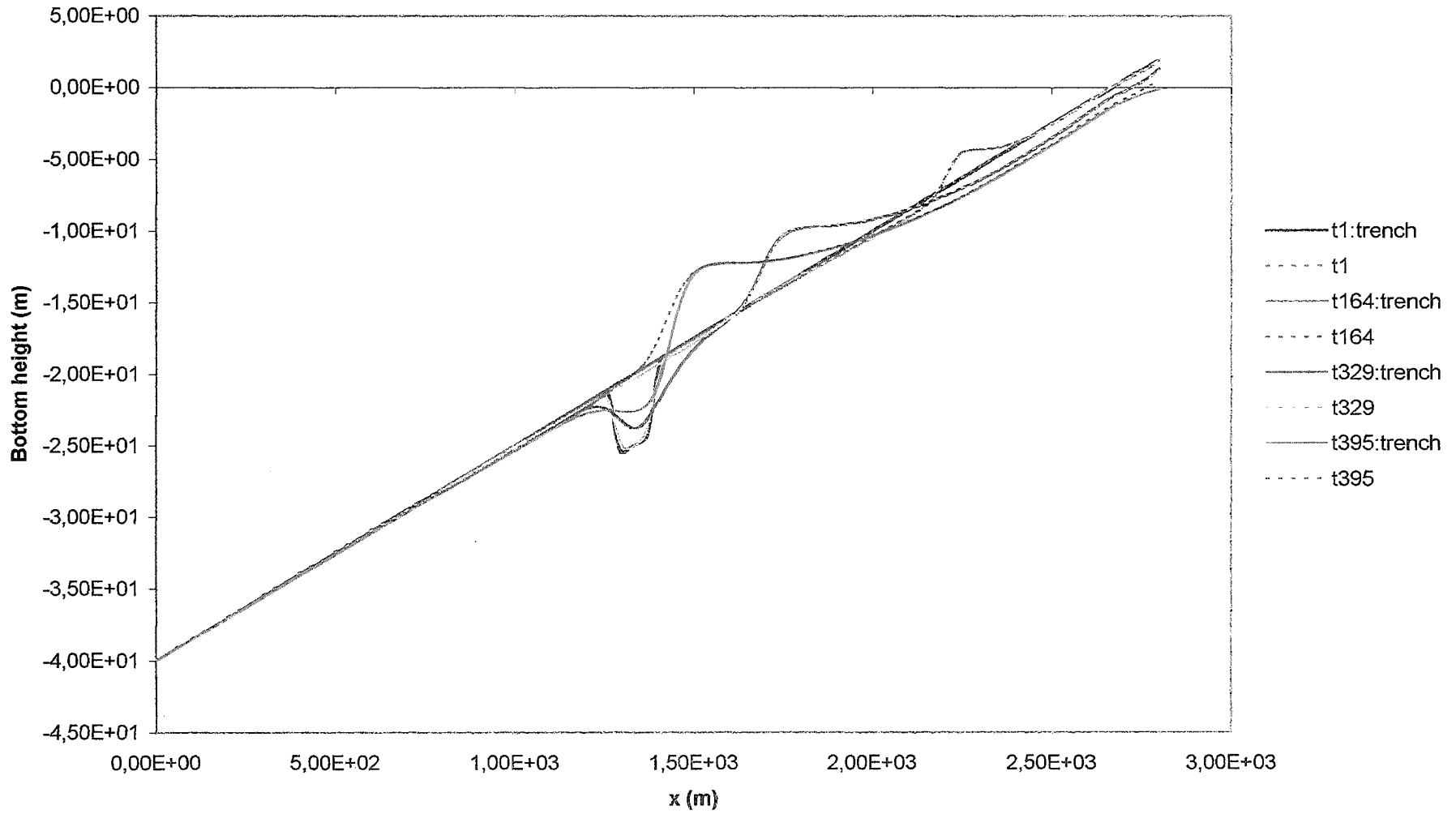
Figure 4.2.12: Profile's comparison (trench at -7 m/no trench)
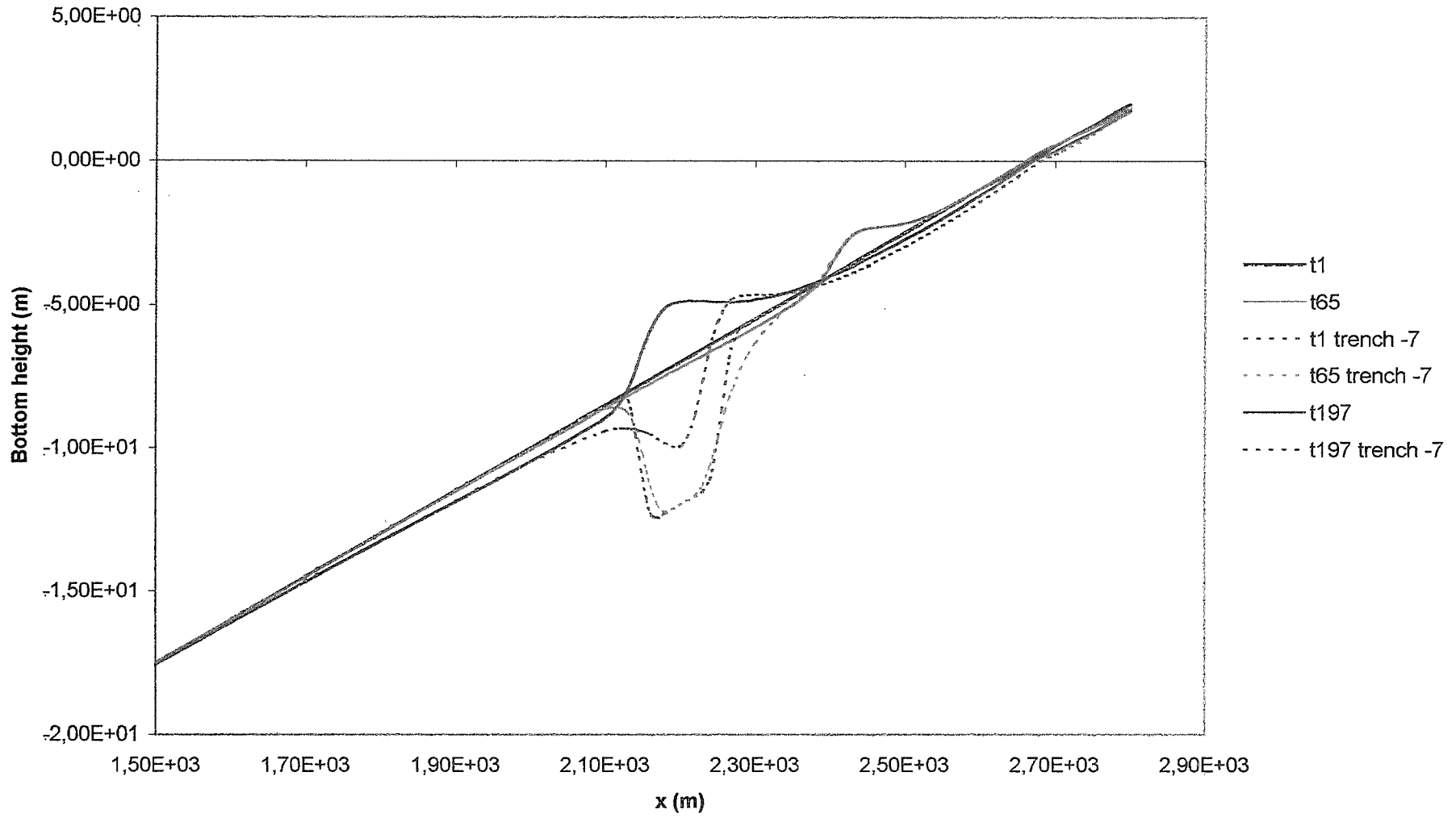
Figure 4.2.13: Development of a gentle slope (tg=0.005) with trench at -7 m depth
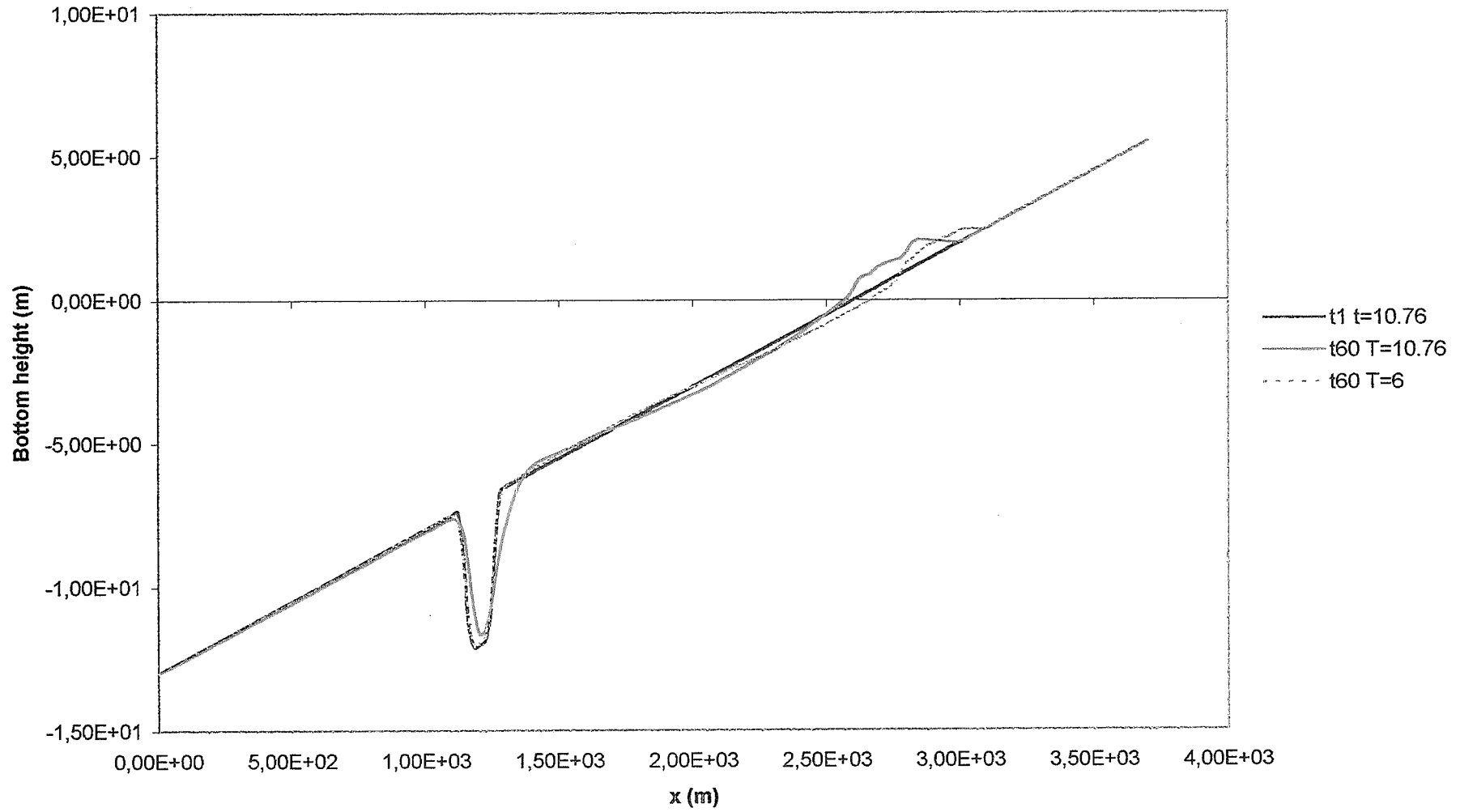
Figure 4.2.14: Total sediment transport (H=1.86,T=6). Comparison: trench at -7m/no trench
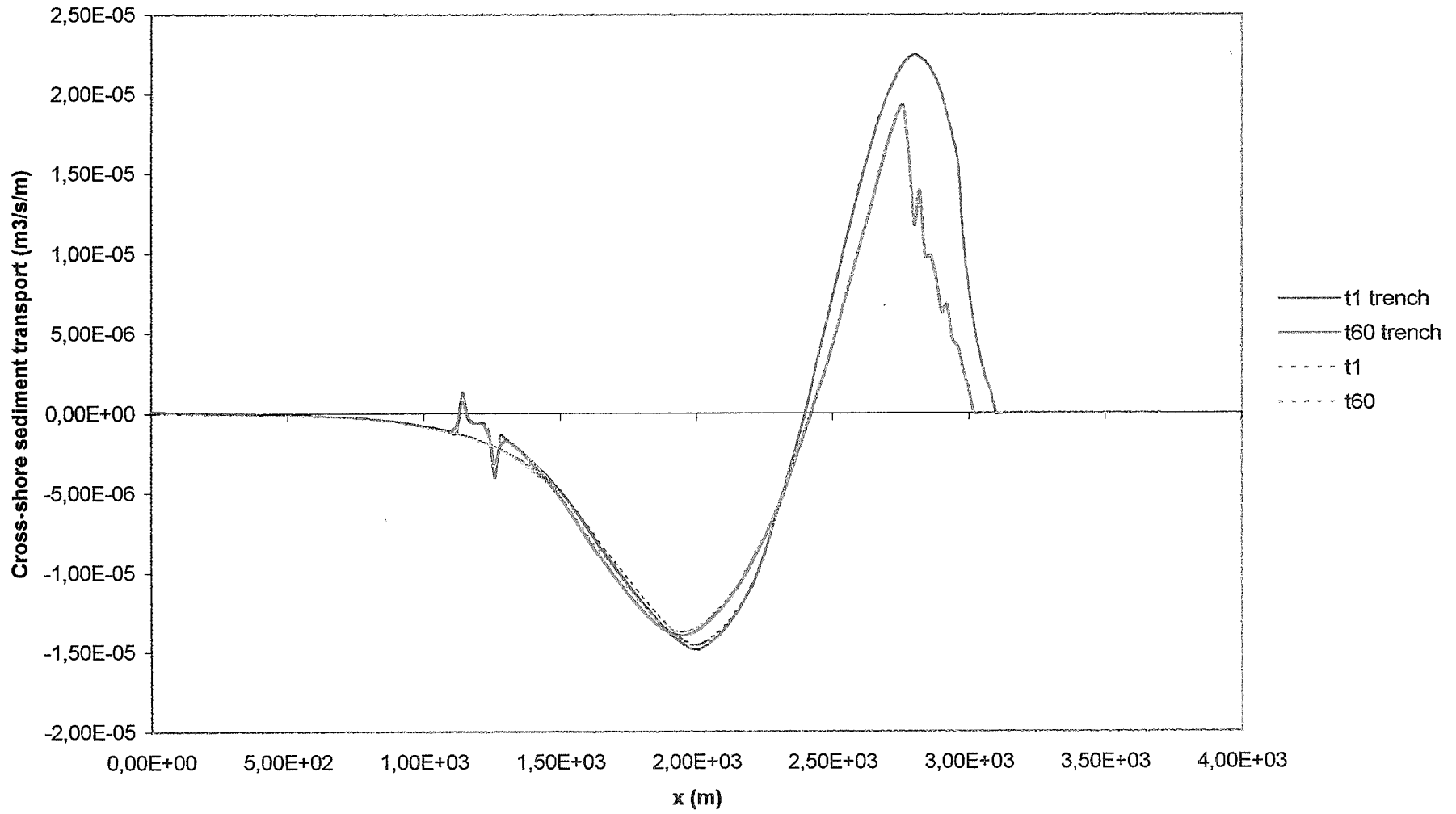
Figure 4.2.15: Profile's development (tg=0.005/trench at -7/H=1.86/T=6)
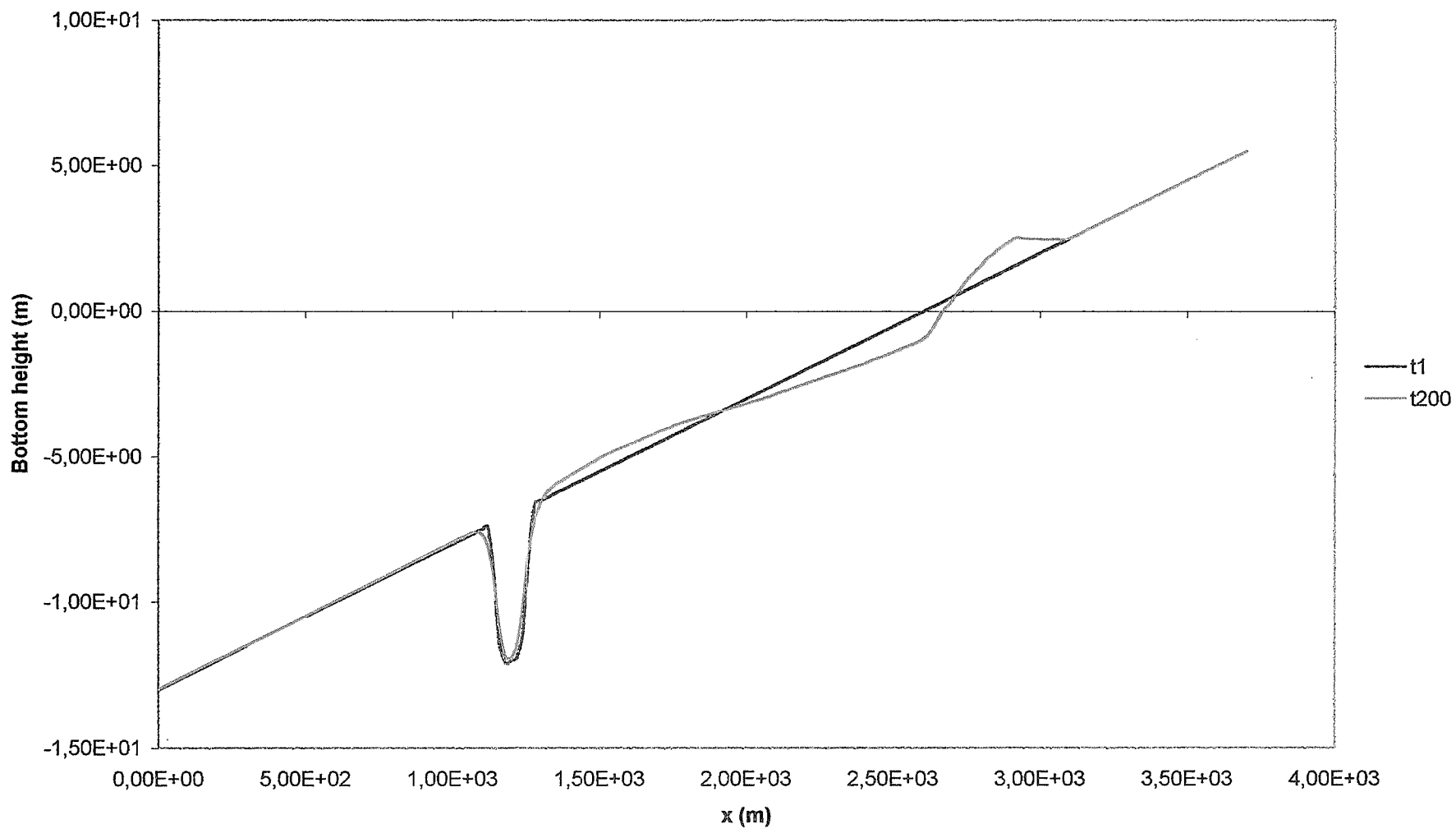
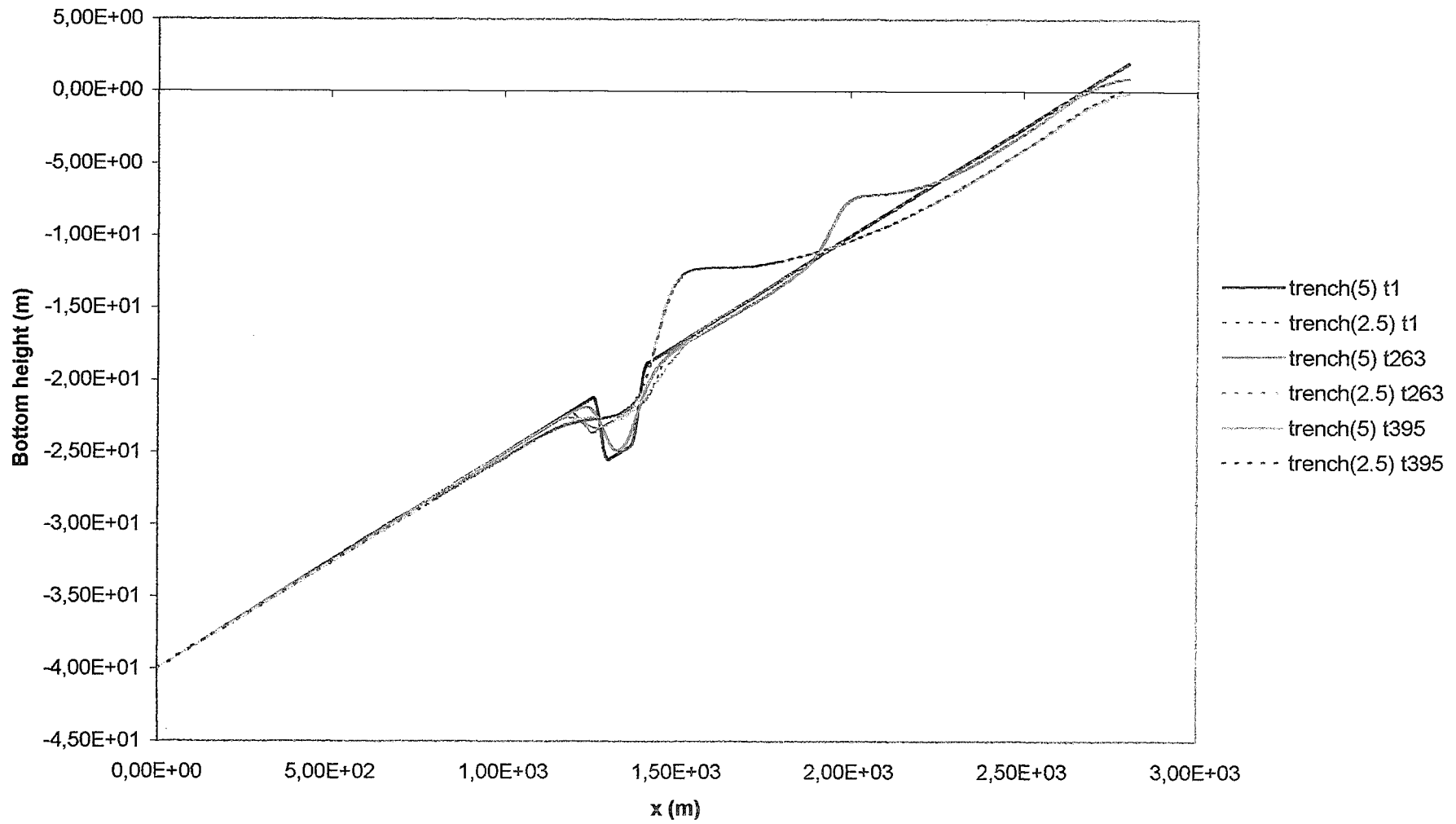**Figure 4.2.16:** Total sediment transport (tg=0.005/trench at -7/H=1.86/T=6)
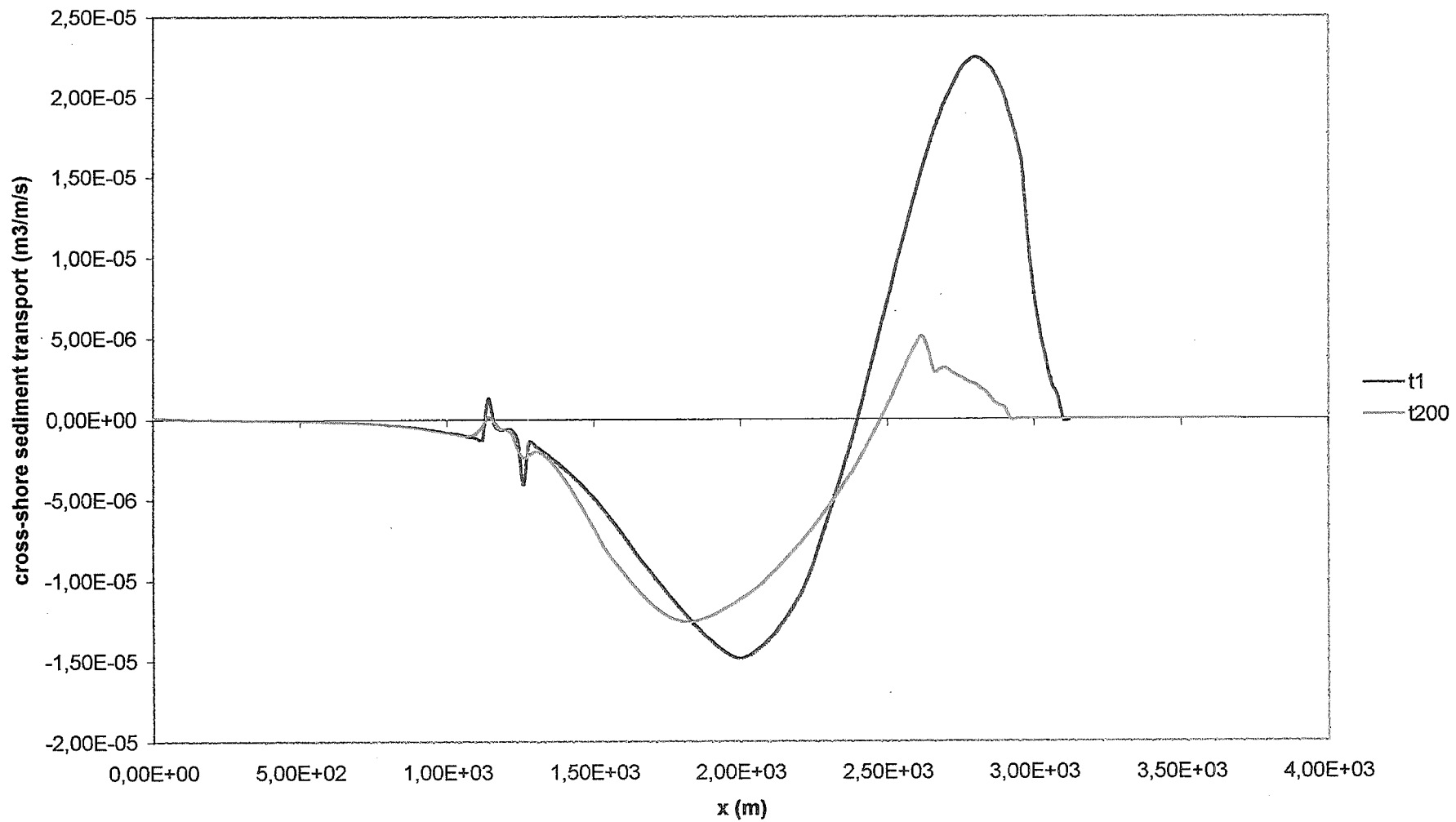
Figure 4.2.17: Profile's development with different depth trenches

4.3- Wave propagation over trenches:

This effect is one of the most important interaction mechanism between a trench and the coastal stability as it has been demonstrated in previous chapters (see chapters 2.4 and 3). It has been identified in a real case as a cause of the observed shore erosion (Combe & Soileau, chapter 3.6).

From the depth defined by the deep-water criterion (d/L$\cong$0.5) waves start "feeling" the bottom and if there is any variation start refracting. When a trench is dredged shorewards of this limit depth (Motyka & Willis, 1974) waves hydrodynamics change and also their angle of incidence to the coast (Fig.4.3.1). A depth increase supposes a local increase of wave height, but also an increase of wave celerity since the hole is placed in shallow waters (c = $\sqrt{gh}$). This modification of wave celerity over the trench induces a change in the orientation of wave front that means a change in the angle of wave incidence.

Both modifications mentioned above affect the longshore sediment transport rates. This transport depends on both height gradient ($S_l \cong \partial H / \partial y$) and angle of incidence ($S_l \cong H^2 \sin 2\theta$) so any of these variations will induce a change of it. And a change in the longshore sediment transport rates implies gradients that can yield erosion or accretion in the shoreline, i.e. changes in the coastal stability.
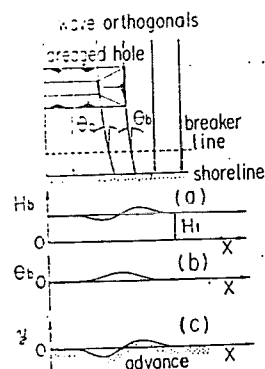

Figure 4.3.1: Changes of wave height, angle of incidence and shoreline position behind the hole formed by the offshore dredging (Uda et al., 1986).

To analyse these phenomena and their impacts to the beach a numerical model called MIKE 21 has been used. Actually its parabolic mild-slope module has been used, MIKE 21 PMS. It is based on a parabolic approximation to the elliptic mild-slope equation that is the governing equation for description of refraction, diffraction and reflection of linear time harmonic water waves on a gently sloping bottom. It was first derived by Berkhoff (1972). The dissipation function in the parabolic mild-slope equation includes dissipation due to

wave breaking and bottom friction. The dissipation function due to wave breaking is calculated using the method of Battjes and Janssen (1978), while the rate of energy dissipation due to bottom friction is formulated using the quadratic friction law to represent bottom shear stress following Dingemans (1983). The parabolic mild slope equation is solved using the Crank-Nicholson numerical scheme for parabolic differential equations and the resulting tridiagonal system of equations is solved using the double-sweep algorithm.

The original bathymetry that has been considered is shown in Figure 4.3.2. It is a coast rectangular cell of 5500*5000 m^2 with a slope of 1/100, between -50 m depth and 5 m. The grid used is $\Delta x = \Delta y = 10$ m.
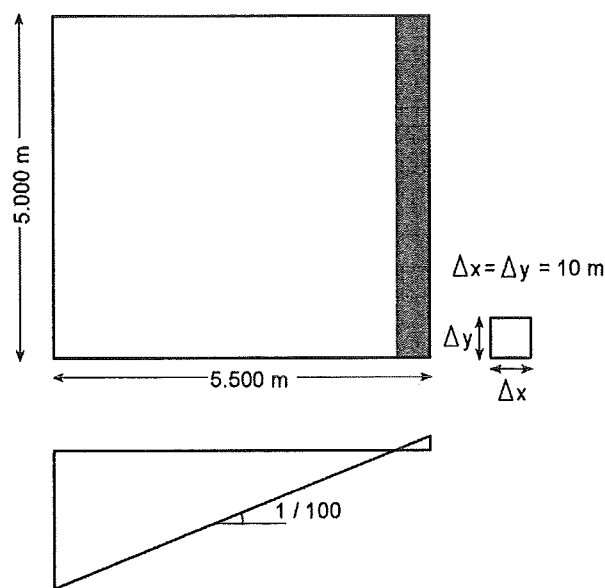

Figure 4.3.2: Outline of the original bathymetry.

There, two trenches of 500 m wide and 1000 m long have been represented between the -30 and -25 m isobath. The first with just 1 m of depth and the other one with 3 m, have been done to analyse the influence of trenches depth in the morphodynamic changes. And to also analyse the wave direction of propagation two different angles of incidence has been used, 0° representing the incidence of frontal waves, and 25° to represent oblique waves. It has been propagated an original wave height of 1 m and period of 10 s. First just the wave height has been observed, its variation while waves propagate over the original bathymetry and over both trenches, and afterwards, the effect of dredged holes presence on wave height have been measured as $H_{trench}/H_{no\ trench}$. Thus, since $H_{no\ trench}$ has been introduced as 1 m, increases and decreases of this coefficient mean the same magnitude of increases or decreases of H_{trench}. The different combinations that have been computed, have been presented as follows:

- Figure 4.3.3: wave height without trench, $\theta=25°$.

86

- Figure 4.3.4: wave height, $d_{trench}=1$ m, $\theta=25°$.

- Figure 4.3.5: wave height, $d_{trench}=3$ m, $\theta=25°$.

- Figure 4.3.6: $H_{trench}/H_{no\ trench}$, $d_{trench}=1$ m, $\theta=25°$.

- Figure 4.3.7: $H_{trench}/H_{no\ trench}$, $d_{trench}=3$ m, $\theta=25°$.

- Figure 4.3.8: $H_{trench}/H_{no\ trench}$, $d_{trench}=1$ m, $\theta=0°$.

- Figure 4.3.9: $H_{trench}/H_{no\ trench}$, $d_{trench}=3$ m, $\theta=0°$.

As it can be observed from all these figures, changes are more important when the depth of the trench is bigger. While for $d_{trench}=1$ m the variations in waves height are about 5% (Fig.4.3.6 & Fig.4.3.8), for $d_{trench}=3$ m they are about 15% (Fig.4.3.7 & Fig.4.3.9). If Figure 4.3.3, 4.3.4 and 4.3.5 are compared it is also clear that modifications are much more significant when the trench is deeper.

If the same type of trench is analysed by comparing the oblique incidence of wave or their frontal incidence, the most important difference is the direction of modifications. These modifications appear symmetric at both sides of the trench axe in the direction of waves incidence. In the lee of the trench, considered in the direction of wave propagation, there is a decrease in wave height while there is an increase in both extremes of the hole due to a concentration of wave energy.
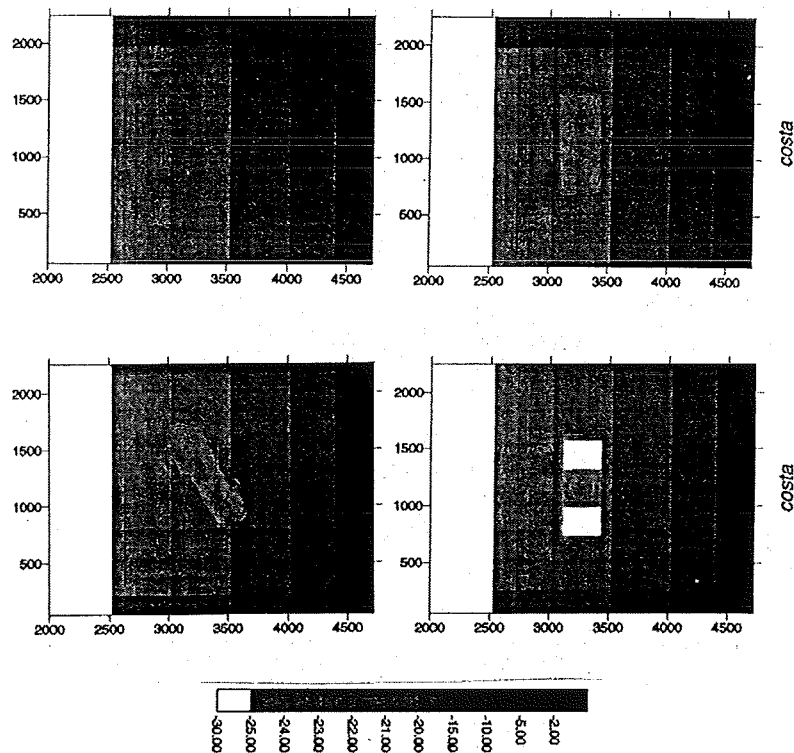

Figure 4.3.10: Bathymetrys for the simulation (Jiménez, 1997).

87

Another numerical experiment was done with different kinds of trenches using an irregular waves propagation model developed in the LIM/UPC (Jiménez, 1997). There, the wave height and the angle of incidence were studied for trenches parallel to the shoreline with constant depth, parallel to the coast but deeper in both extremes and oblique to the coast (Fig.4.3.10). The waves propagated were 2 m height and had a period of 7 s. Propagation was 25° oblique with a directional dispersion of 20°.

The results obtained were similar to the ones obtained by MIKE 21. In the trench with variable depth the behaviour is the same as for the trench with constant depth but more marked; there is more increase of wave height in the extremes because they are deeper. For the oblique trench the modifications of wave height follow the same pattern as the first one too, but turned and there are higher waves near the shoreline because one of the edges of the trench is closer to it. On the other hand, when the variations of the angle of incidence are observed ($\theta_{trench}-\theta_{no\ trench}$) it can be concluded that the angle also decrease in the lee of the trench and tends to increase in both ends (Fig.4.3.11).

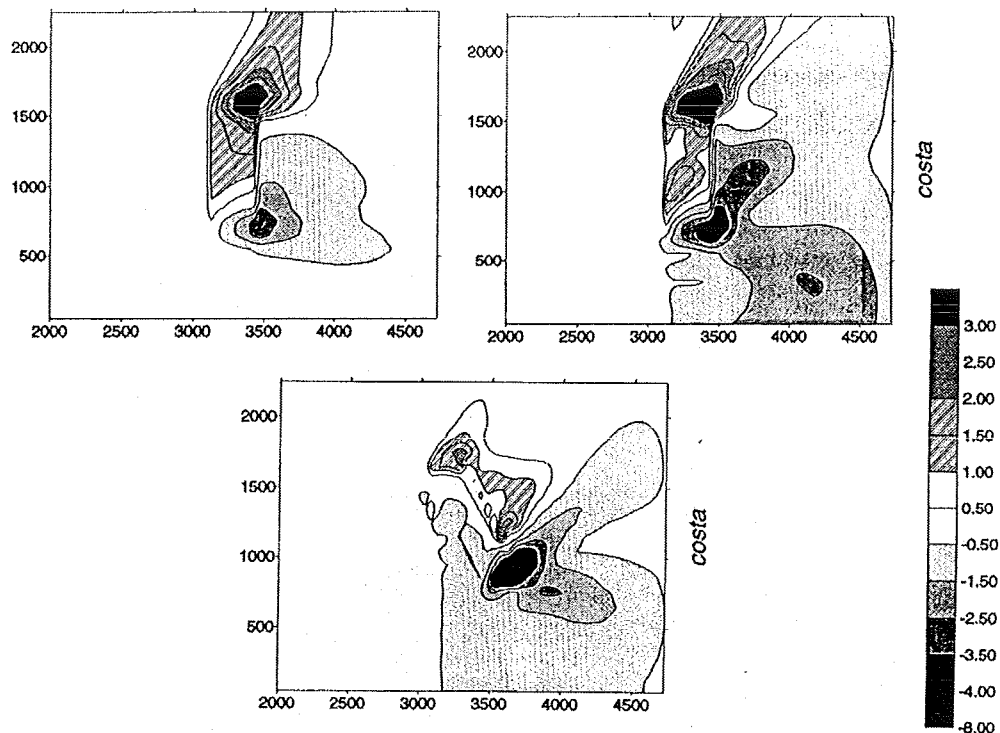**Figure 4.3.11**: Changes of the angle of incidence ($\theta_{trench}-\theta_{no\ trench}$) for the 3 different configurations showed in Figure 4.3.10 (Jiménez, 1997).

All these changes would induce gradients in the longshore transport rates that imply accumulations of material in the lee of the trenches and erosion in both extremes. It would induce the formation of structures tombolo-like as Horikawa et al. predicted (1977) and as the one that appeared in Grand Isle (see chapter 3.6, Combe & Solieau, 1987). There two

tombolos were formed in both extremes of the trench and it is easy to understand looking at the wave height changes in the figures obtained by using MIKE 21.

To sum up, although it will be necessary to make specific analysis for each case, it can be assume that the impact due to wave modification will be potentially important in long coasts where their development is conditioned by the longshore sediment transport. In rugged coasts with small beaches in between, this effect will be less significant, and if the beach were in the lee of one trench, no effect would be appreciable in the coastal evolution due to this fact.

Moreover, it seems that a general condition could be not to dredge deep holes and neither make them very wide (less than 400 m according to Viguier et al., 1984, see chapter 3.3).

To assess the modification of wave propagation, the waves that potentially more can be affected by trench presence should be chosen from the characteristic wave climate, i.e. waves with long periods and angles of incidence. And always their duration should be considered as well to characterise their effect in coastal evolution in an effective way.

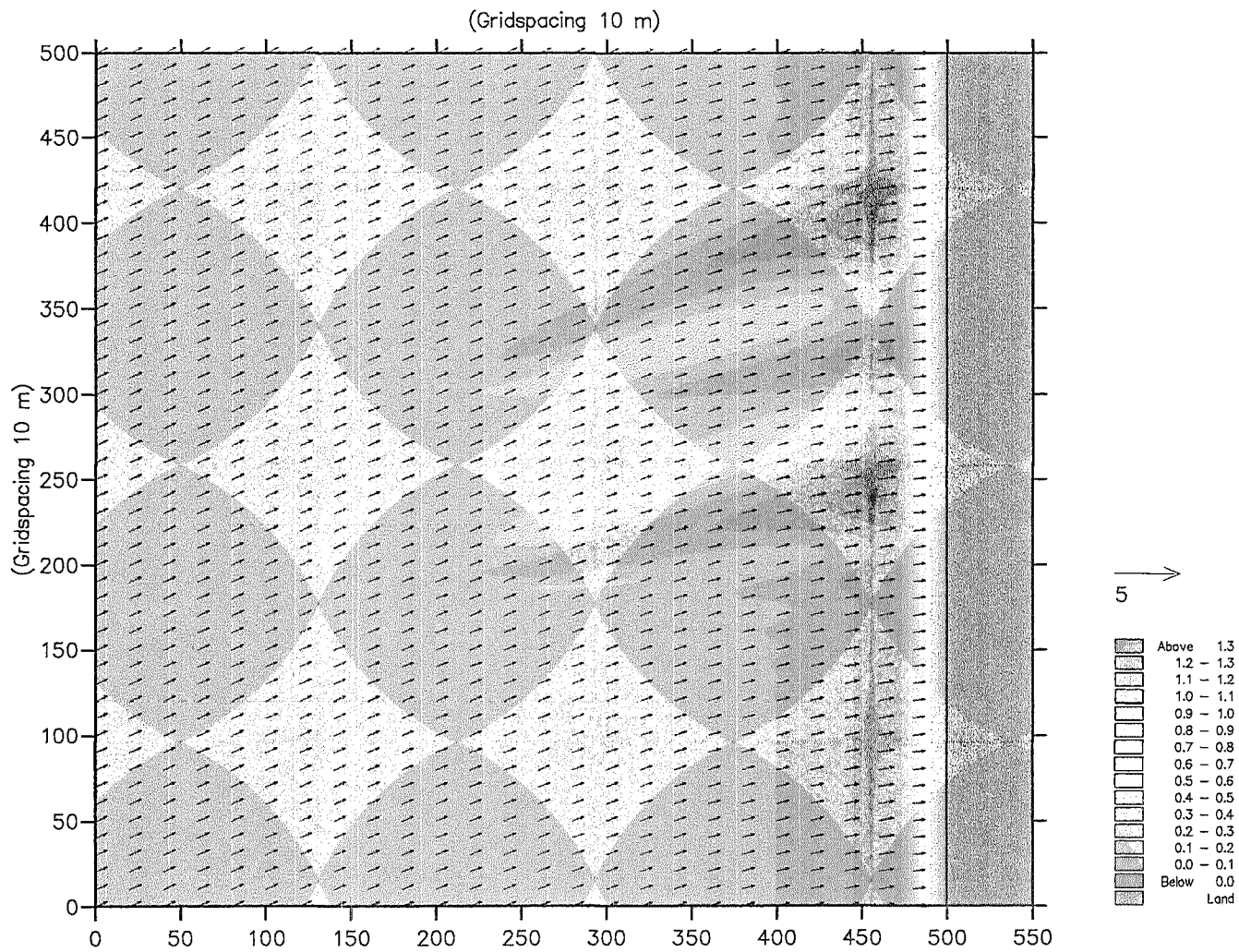**Figure 4.3.3**: Height wave propagation over the original bathymetry. H=1 m, T=10 s and θ=25°.

(Gridspacing 10 m)
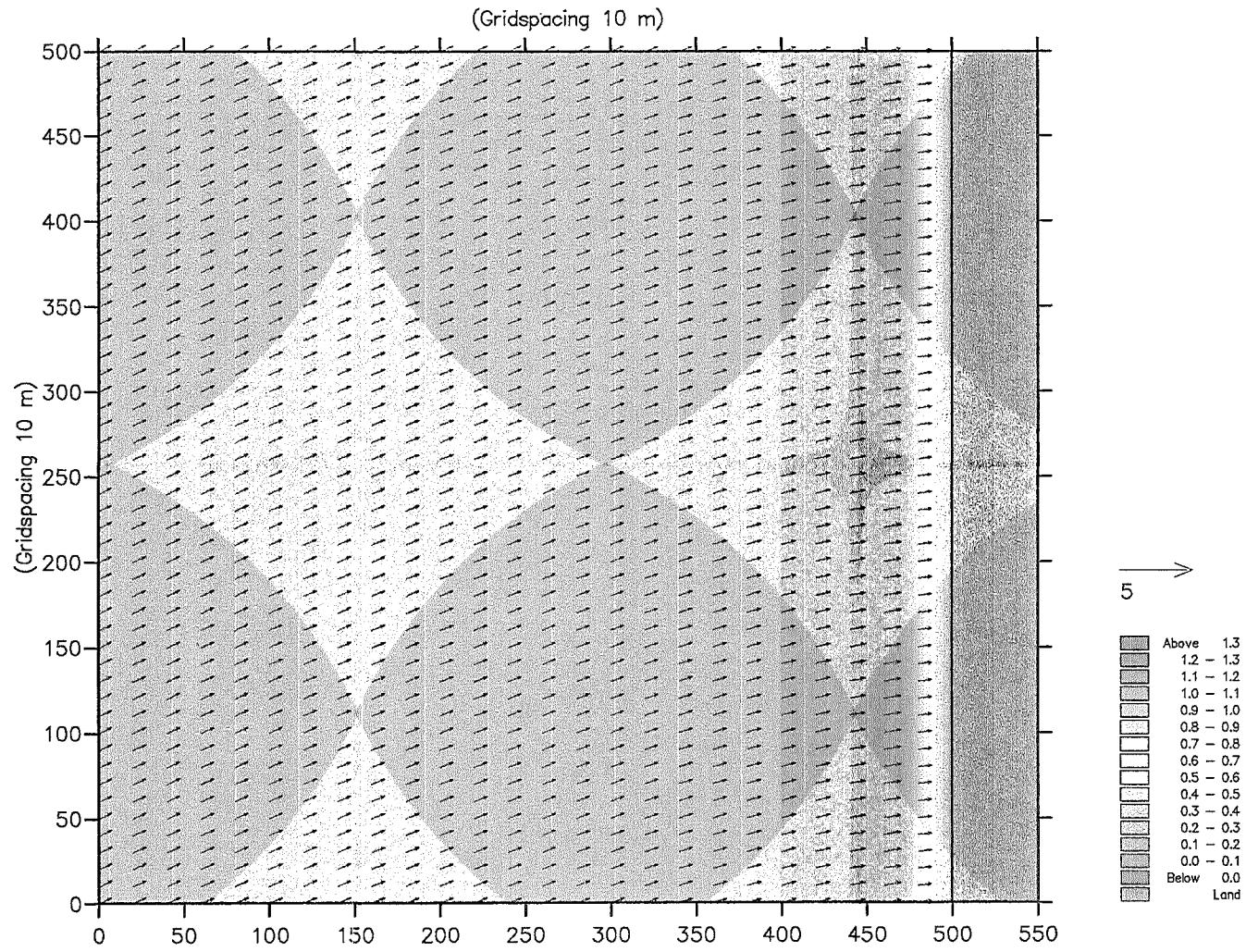
Figure 4.3.4: Height wave propagation when there is a 1 m depth trench . H=1 m, T=10 s and θ=25°.

(Gridspacing 10 m)
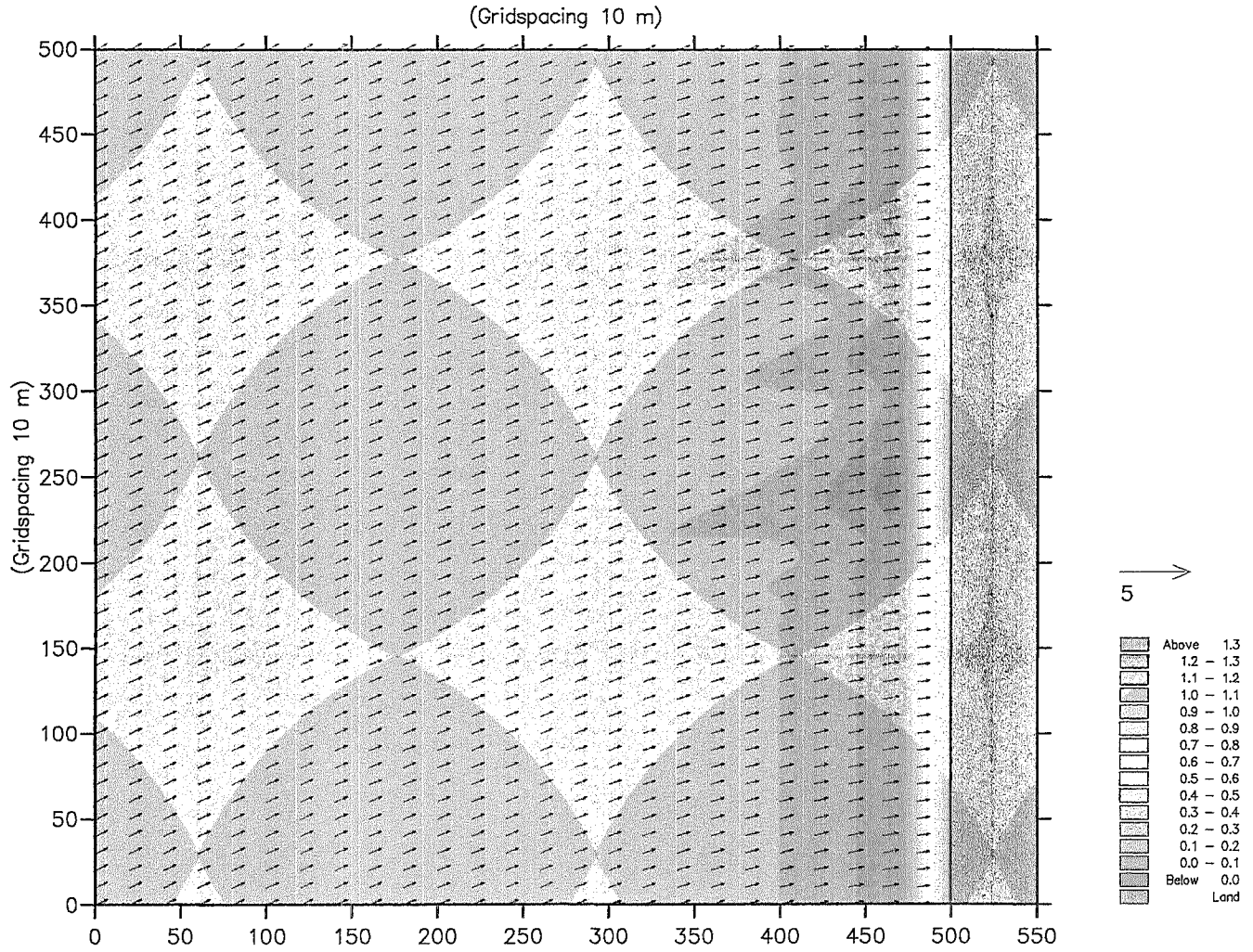
Figure 4.3.5: Height wave propagation when there is a 3 m depth trench . H=1 m, T=10 s and θ=25°.

(Gridspacing 10 m)

Figure 4.3.6: $H_{trench}/H_{no\ trench}$ when there is a 1 m depth trench . H=1 m, T=10 s and θ=25°.

(Gridspacing 10 m)

Above	1.05
1.00	1.05
0.95	1.00
Below	0.95
	Land

Figure 4.3.7: $H_{trench}/H_{no\ trench}$ when there is a 3 m depth trench . H=1 m, T=10 s and θ=25°.

(Gridspacing 10 m)

Figure 4.3.8: $H_{trench}/H_{no\ trench}$ when there is a 1 m depth trench . H=1 m, T=10 s and θ=0°.

(Gridspacing 10 m)
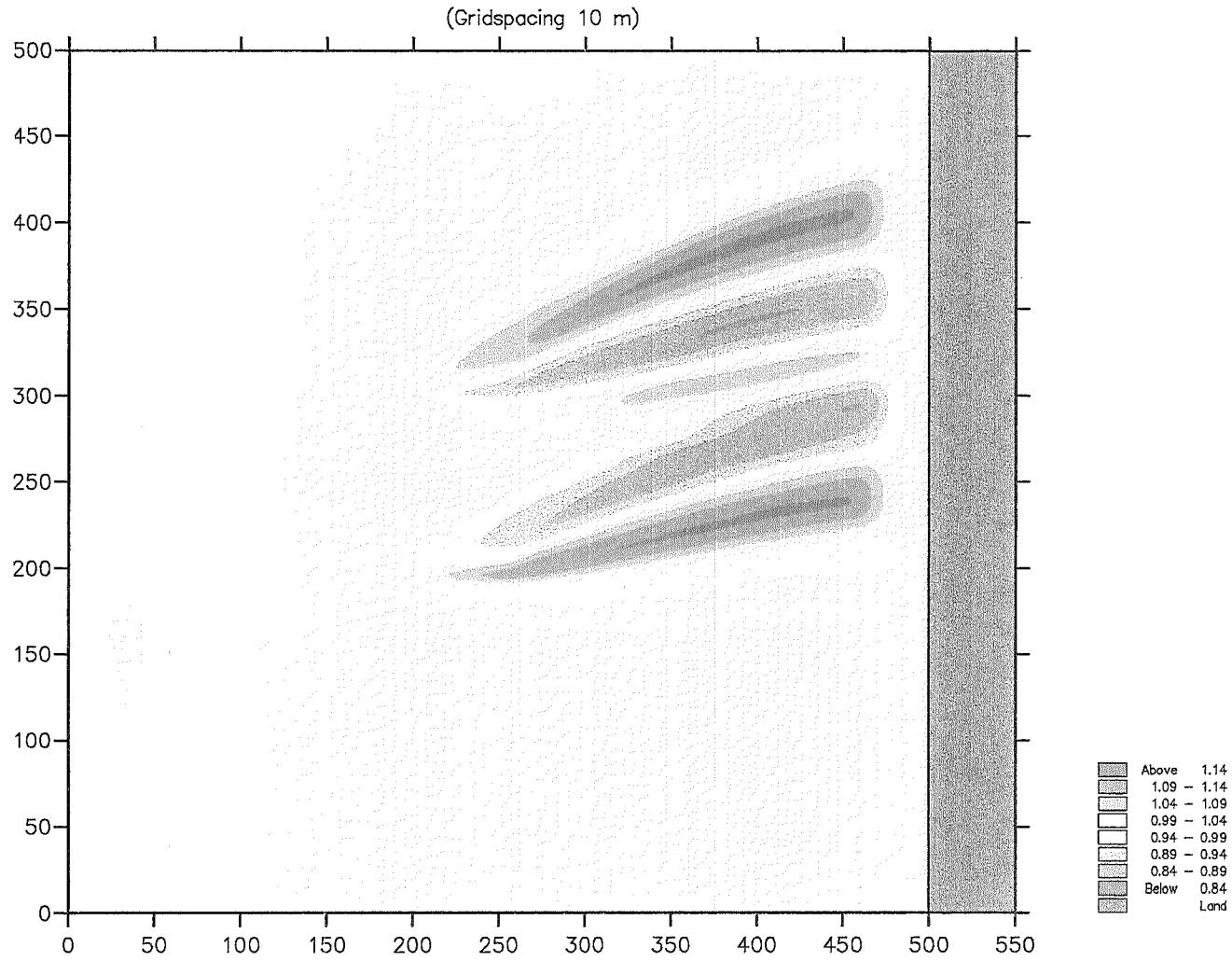
Figure 4.3.9: $H_{trench}/H_{no\ trench}$ when there is a 3 m depth trench . H=1 m, T=10 s and $\theta=0°$.

5. CONCLUSIONS:

From the analysis presented in this work a series of conclusions about the influence of nearshore sand extraction on coastal stability can be obtained. In what follows these conclusions are presented.

After the theoretical analysis the potential impacts due to nearshore sand extraction and the resulting trench on coastal stability have been identified: the interaction with longshore sediment transport, the beach drawdown, the interception of the onshore sediment transport in the inner shelf, the modification of waves characteristics, the modification of wave's field of velocity and the trench propagation to the coast.

There are different depth criterion to prevent each of these impacts but at the same time, the ones associated to some of them are exceeded by others. Thus, the way to select the depth criterion to prevent all of them must be based on the most restrictive one.

On the other hand, it has been observed that other impacts just would verify in particular kind of beaches or coasts (e.g. the interception of onshore transport) or within long time scales (e.g. trench propagation) and although potentially they would be able to affect the coastal stability, they will mainly verify in very specific coastal stretches and, in consequence, they are not very common.

Finally, three effects have been identified as the most likely to happen and therefore, some rules are necessary to be sure they will be avoided: beach drawdown, wave's modification and also trench propagation.

By analysing experimental data, which theoretically related the coastal behaviour with dredging, no clear conclusion can be obtained about the location of trenches to avoid real impacts. In most of the cases, final conclusions have not been fully proved and results obtained (and also data used) are so different and disperse that is not possible to obtain any global conclusion or neither to assure that the observed coastal response is due to the dredging works. Just wave refraction could be checked so it has been considered as an actual effect of borrow pits nearshore. Also trench evolution during storms, i.e. the infilling effect of sand eroded from the beach that yield beach drawdown, appeared in several of the analysed studies. In some modelled and theoretical analysis trench propagation was presented as important, increasing its importance as the time scale increased (instantaneously it is negligible). As a summary, according to the analysed field

cases few real conclusions about the influence of dredging works on coastal stability can be obtained.

Looking at the three identified most dangerous effects in more detail, some general considerations have been found to prevent the influence on coastal stability.

To avoid the called beach drawdown, trenches should be done seawards of the depth of closure. Therefore it is necessary to find this threshold depth of significant vertical changes, but since it has been demonstrated that it is time and space dependant it is not a trivial task. At decadal scales the weak morphodynamic changes in the middle and lower shoreface become significant and suppose a re-opening of the profiles. This re-opening was understood as a consequence of a cumulative process rather than of infrequent extreme events. Thus, since the importance of this behaviour is known, it should be done a large-scale analysis. In spite of this, a first assessment of the minimum depth can be done using Hallermeier's equation (Eq.4.1.1) fed by extremal wave conditions, selected to be representative of a return period according to the life period of the trench, e.g. $T_{return} > 25$ years.

The idea that theoretical studies and process-based models could not well represent real phenomena has been tested trying to represent trench propagation with a process-based model. Its representation through UNIBEST-TC has been just qualitative and not quantitative. No trench propagation has been obtained without any on-shore constant current to guarantee a steady on-shore sediment transport, which just would really exist in specific kind of coasts. On the other hand, it can represent the depth where trenches still do not induce more beach erosion than the one that would appear if they did not exist. Therefore, these kinds of models can be used to have a first order of magnitude of where trenches should be dredged to prevent coastal erosion. Moreover, UNIBEST-TC could predict depth of closure at small-scales as a storm event.

Although trench propagation can not be verified by UNIBEST-TC, it does not mean that it can not occur in nature. Consequently some criterion should be considered to prevent this impact. If trenches were done seawards of the deep-water limit, when waves start "feeling" the bottom and some hydrodynamic changes could appear, no propagation would be possible. But this depth ,d_0, is really deep and far from the coast so, since the important is to not let local hydrodynamics changes to be bigger than a little proportion, a shallower depth can be considered. A depth where sediment transport rates variation are almost negligible or little enough to assure that if there were trench migration, it would be

really slow and it would not affect coastal stability in a century. To keep sediment transport gradients small, trenches should have little depth (which means a minimum alteration of the local hydrodynamics) or be in depths where the transport rates are small. A general criterion could be to dredge seawards of the depth that represents the beginning of significant sediment transport, which at the same time would avoid the interception of the onshore transport in the inner shelf.

The third effect due to nearshore trenches is the modification of wave characteristics when they go over one of these dredged holes. By theoretical studies it was known that such an alteration of the sea bottom in depths where waves "feel" it could induced changes in wave height and angle of incidence, which imply longshore sediment transport gradients. These gradients induce erosion or accumulation of sediment alongshore so they suppose a change in the shoreline development, especially in long coasts where their development is conditioned by the longshore sediment transport. In rugged coasts with small beaches in between, this effect will be less significant.

To find the influence of the shape and the depth of trenches to the wave height changes some studies have been carried out by using numerical models. Through them it has been found that changes are more important when the depth of the trench is bigger and also how they are. Modifications appear symmetric at both sides of the trench axe in the direction of waves incidence. In the lee of the trench, considered in the direction of wave propagation, there is a decrease in wave height while there is an increase in both extremes of the hole due to a concentration of wave energy. On the other hand, when the variations of the angle of incidence are observed (θ_{trench}-$\theta_{no\ trench}$) it can be concluded that the angle also decrease in the lee of the trench and tends to increase in both ends.

Due to all these modifications, structures tombolo-like can be induced by the longshore sediment transport.

Actually, to avoid this impact, it will be necessary to make specific analysis for each case, but it seems that a general condition could be to not make them deep and neither very wide (less than 400 m according to Viguier et al., 1984). To assess the modification of wave propagation, the waves that potentially more can be affected by trench presence should be chosen from the characteristic wave climate, i.e. waves with long periods and angles of incidence. And always their duration should be considered as well to characterise their effect in coastal evolution in an effective way.

To finish and as a final summary, although nearshore sand extraction can become a problem for the coastal stability, there are some methods to minimise or to eliminate the induced impacts. They can and should be used to make an optimum design that would avoid damages in the coastal system and the use of unnecessary precautions (i.e. excessive constraints that would make dredging works more expensive) at the same time. Thus, the balance between uses and coastal resources would be achieved.

6. REFERENCES:

Anctil,F. and Ouellet,Y. (1990). Preliminary evaluation of impacts of sand extraction near Illes-de-la-Madeleine Archipielago, Québec, Canada. *Journal of Coastal Research*, 6,1,37-51.

Bailard,J. (1981). An energetics total load sediment transport model for a plane sloping beach. *Journal of Geophysical Research*, 86, C11, 10938-10954.

Basco, D.R. and Lonza, F.R. (1997). Nearshore wave transformation altered by sand volumes removed from borrow areas for beach nourishment. *Ocean wave measurement and analysis*, ASCE, 37-48.

Battjes, J.A. and Janssen, J.P.F.M. (1978). Energy loss and set-up due to breaking of random waves. *Proc. 16th Coastal Engineering Conf.*, ASCE, 569-587.

Berkhoff, J.C.W. (1972). Computation of combined refraction-diffraction. *Proc. 13th Coastal Engineering Conf.*, ASCE, 471-490.

Bodge, K.R. (1989) A literature review of the distribution of longshore sediment transport across the surfzone. *Journal of Coastal Research*, 5, 2, 307-328.

Boletín Oficial del Estado (1994). *Costas*. Colección Textos Legales, Madrid, 394 pp.

Capobianco, M., Larson, M., Nicholls, R.J. and Kraus, N.C. (1997). Depth of closure: a contribution to the reconciliation of theory and practise. *Proc. Coastal Dynamics'97*, ASCE, 506-515.

CERC (1984). *Shore Protection Manual*, US Army Corps of Engineers, Washington DC.

Combe, A.J. and Soileau, C.W. (1987). Behaviour of man-made beach and dune Grand Isle, Louisiana. *Coastal Sediments'87*, ASCE, 1232-1242.

Cowell, P.J., Roy, P.S. and Jones, R.A. (1995). Simulation of LSCB using a morphological behaviour model. *Marine Geology*, 126, 450-461.

Danish Hydraulic Institute (1996), *User's manual MIKE 21*.

Delft Hydraulics (1992), *User's manual MS-DOS PC-program, UNIBEST-TC version 1.00*.

Delft Hydraulics (1997), *User's manual UNIBEST-TC version 2.0*.

Dingemans, M.W. (1983). *Verification of numerical wave equation models with field measurements, CREDIZ verification Haringvliet*. Delft Hydraulics Laboratory, Rep.nº W488, Delft.

Douglass, S.L. (1995). Estimating landward migration of nearshore constructed sand mounds. *Journal of Waterway, Port, Coastal and Ocean Engineering*, 121, 5, 247-250.

Dovers, S.R. and Handmer, J.W. (1995). Ignorance, the Precautionary Principle and Sustainability. *Ambio*, 24, 2, 92-97.

Gracia, V. Jiménez, J.A. and Sánchez-Arcilla, A. (1997). Temporal and spatial variability of the depth of closure in the Ebro delta. *First Annual PACE Workshop, Abstracts-In-Depth*, Abstract 1.4, 4pp.

Hallermeier, R.J. (1980). Sand motion initiation by water waves: two asymptotes. *Journal of Waterway, Port, Coastal and Ocean Division*, 106 (WW3), 299-318.

Hallermeier, R.J. (1981). A profile zonation for seasonal sand beaches from wave climate. *Coastal Engineering*, 4, 3, 253-277.

Hands, E.B. and Allison, M.C. (1991). Mound migration in deeper water and methods of categorizing active and stable depths. *Coastal Sediments'91*, ASCE, 1985-1999.

Hands, E.B., Ahrens, J.P. and Resio, D.T. (1996). Predicting large-scale, cross-shore sediment movement from orbital speeds. *Proc. 25th Coastal Engineering Conf.*, ASCE, (in press).

Hanson, H. and Kraus, N.C. (1989). *Genesis: a generalized model for simulating shoreline change.* Technical Report CERC-89-19, USACE, Washington DC.

Hilton, M.J. and Hesp, P. (1996b). Nearshore-surfzone system limits and the impacts of sand extraction. *Journal of Coastal Research*, 12, 3, 726-747.

Hinton, C. and Nicholls, R.J. (1998). Spatial and temporal behaviour of depth of closure along the Holland Coast. *Proc. 26th Coastal Engineering Conf.*, ASCE, 2913-2925.

Hinton, C., Nicholls, R.J. and Dunsbergen, D. (1999). Profile re-opening on the shoreface of the Holland Coast. *Coastal Sediments'99*, ASCE, 535-550.

Horikawa, K., Sasaki, T. and Sakuramoto, H. (1977). Mathematical and laboratory models of shoreline changes due to dredged holes. *Journal of the Faculty of Engineering*, University of Tokyo (B), XXXIV,1.

Hsu, T.-W. And Chang, H.-K. (1994). Experimental studies on the effect of the dredging on Chang-Hwa reclamation area, Taiwan. *Proc. 24th Coastal Engineering Conf.*, ASCE, 2972-2986.

Inman, D.L. and Rusnak, G.S. (1956). *Changes in sand level on the beach and shelf at La Jolla, California.* U.S.Army, Corps of Engineers. Beach erosion board, Thecnical Memorandum nº 82 July 1956.

Jackson, L.A. and Tomlinson, R.B. (1990). Nearshore nourishment implementation, monitoring and model studies of 1.5 M³ at Kirra Beach. *Proc. 22nd Coastal Engineering Conf.*, ASCE, 2241-2254.

Jiménez, J.A. (1996). *Evolución costera en el Delta del Ebro. Un proceso a diferentes escalas de tiempo y espacio.* Tesis doctoral, Universitat Politècnica de Catalunya, 274 pp.

Jiménez, J.A. (1997). *Influencia de dragados offshore sobre la estabilidad costera.* Research paper, LIM/UPC.

Kojima, H., Ijima, T. and Nakamuta, T. (1986). Impact of offshore dredging on beaches along the Genkai Sea, Japan. *Proc. 20th Coastal Engineering Conf.*, ASCE, 1281-1295.

Kraus, N.C. and Harikai, S. (1983). Numerical model of shoreline change at Orai beach. *Coastal Engineering*,7,1-28.

Lee, A.C. and Osborne, R.H. (1995). Relative fluxes of sand for Southern California Beaches: Fourier grain-shape analysis. *Shore and Beach*, 63, 3, 9-19.

Madsen, O.S. and Grant, W.G. (1976). Quantitative description of sediment transport by waves. *Proc. 17th Coastal Engineering Conf.*, ASCE, 1093-1112.

Marsh, S.W., Nicholls, R.J., Kroon, A. and Hoekstra, P. (1988). Assessment of depth of closure on a nourished beach: Terschelling, The Netherlands. *Proc. 26th Coastal Engineering Conf.*, ASCE, 3110-3123.

Meyer-Peter, E. and Müller, R. (1948). Formulas for Bed-Load Transport. Report on *Second Meeting of International Association for Hydraulic Research*, 39-64.

Migniot, C. And Viguier, J. (1980). Influence de l'extraction des granulats en mer sur l'équilibre du littoral. *La Houille Blanche*, 3, 177-193.

MOPT. (1992). Recomendaciones para Obras Marítimas. ROM 0.3-91. Oleaje. Anejo I. Clima Marítimo en el Litoral Español. Dirección General de Puertos, Madrid, 76 pp.

Motyka, J.M. and Willis, D.H. (1974). The effect of wave refraction over dredged holes. *Proc. 14th Coastal Engineering Conf.*, ASCE, 615-625.

Nicholls, R.J. and Birkemeier, W.A. (1997). Morphological and sediment budget controls on depth of closure at Duck, NC. *Proc. Coastal Dynamics 1997*, ASCE, 496-505.

Nicholls, R.J., Birkemeier, W.A. and Hallermeier, R.J. (1996). Application of the depth of closure concept. *Proc. 25th Coastal Engineering Conf.*, ASCE, 3874-3887.

Nicholls, R.J., Larson, M., Capobianco, M. and Birkemeier, W.A. (1998). Depth of closure: improving understanding and prediction. *Proc. 26th Castal Engineering Conf.*, ASCE, 2888-2901.

Niedoroda, A.W., Reed, C.W., Swift, D.J.P., Arato, H. and Hoyanagi, K. (1995). Modelling shore-normal large scale coastal behaviour. *Marine Geology*, 126,181-199.

París, C.; Wibbelink, H. and Basabe, J.M. (1995). *Construcción de playas. Experiencias en España. Ingeniería del Agua, Proyectos y construcción de playas artificiales y regeneración de playas*, 2, 167-180.

Price, W.A, Motyka, J.M. and Jaffrey, L.J. (1978). The effect of offshore dredging on coastlines. *Proc. 16th Coastal Engineering Conf.*, ASCE, 1347-1358.

Roelse, P. (1990). Beach and dune nourishment in the Netherlands. *The Dutch coast; report of a session on the 22nd International Conference on Coastal Engineering 1990*, paper n°11, 14 pp.

Roelvink, J.A. and Stive, M.J.F. (1990). Sand transport on the shoreface of the Holland coast. *Proc. 22nd Coastal Engineering Conf.*, ASCE, 1909-1921.

Rózynski, G., Zielder, R.B. and Pruszak, Z. Depth of closure and seabed variability patterns. *Proc. 26th ICCE,* Copenhagen, ASCE, in press.

Saville, T. (1981). Letters-to-the-Editor. *Shore and Beach*, October 81, 10-11.

Stive, M.J.F. (1985). *Cross-shore sediment transport*. European workshop on Coastal Zones, Sept/Oct. 1985, Greece, 9 pp.

Stive, M.J.F. and de Vriend, H.J. (1995). Modelling shoreface profile evolution. *Marine Geology*, 126, 235-248.

Stive, M.J.F., de Vriend, H.J., Nicholls, R.J. and Capobianco, M. (1992). Shore nourishment and the active zone: a time scale dependant view. *Proc. 23th Coastal Engineering Conf.*, ASCE, 2464-2473.

Stive, M.J.F., Roelvink, D.J.A. and De Vriend, H.J. (1990). Large-Scale Coastal Evolution Concept. *The Dutch coast; report of a session on the 22nd International Conference on Coastal Engineering 1990*, paper nº 9, 13 pp.

Terwindt, J.H. and Battjes, J.A. (1990). Research on large-scale coastal behaviour. *The Dutch coast; report of a session on the 22nd International Conference on Coastal Engineering 1990*, paper nº 10, 9 pp.

Uda, t., Agemori, C. and Chujo, N. (1986). Beach changes caused by offshore dredging. *Coastal Engineering in Japan,*29, 215-226.

Van Alphen, J.S.L.J., Hallie, F.P., Ribberink, J.S., Roelvink, J.A. and Louisse, C.J. (1990). Offshore sand extraction and nearshore profile nourishment. . *The Dutch coast; report of a session on the 22nd International Conference on Coastal Engineering 1990*, paper nº 12, 12 pp.

Viguier, J. , Roques, J.M. and Migniot, C. (1984). Influence de l'extraction des granulats en mer sur l'équilibre du littoral. *XVIII Journées de l'Hydraulique, L'Hydraulique et la maîtrise du littoral*, Question III, Rapport 13, III.13.1-III.13.8.

Vongvissesomjai, S. (1994). Shoreline erosion due to offshore tin mining. *Proc. 24th Coastal Engineering Conf.*, ASCE, 3393-3407.

Watts, G.M. (1963). Behaviour of offshore borrow zones in beach fill operations. *Proc. 10th Congress of the International Association for Hydraulic Research*, Vol 1, 17-24.