CHAPTER 139

Extreme Erosion Event on an Artificial Beach

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Abstract

At Oranjemund just north of the mouth of the Orange on the South-West African/ Namibian coastline the Consolidated Diamond Mines (Pty) Limited (CDM) is mining for diamonds in the inshore area. They use an artificially-built seawall of sand to keep the sea out of the paddocks which are being stripped and mined at bedrock level, which is well below sea level. The seawall runs parallel to the original shoreline at a distance of up to 350 m offshore. The beach profile is correspondingly very steep and under most conditions offshore sediment losses occur, which are compensated for by artificial nourishment. During February 1987 offshore losses of about 120 m^3 per running metre of seawall were recorded from above the waterline. In this paper a data set is presented to serve as a basis for the calibration of on-offshore sediment models and some simulations are reported on.

1. Background

Diamonds have been mined on the west coast of South-West Africa/Namibia for more than 50 years (see Figure 1). As mining techniques have improved more and more diamonds have been taken out of what used to be the surf-zone area. CDM has perfected the technique of inshore mining by using seawalls or bunds constructed of sand (Figure 2). Details of the coastal engineering aspects of the mining technique have been given by Möller **et al** (1986). In essence the construction of a seawall of sand in the nearshore zone and its maintenance against wave attack amounts to a large-scale sand suppletion or beach management project. Figure 3 shows how a typical nearshore beach profile which has a nearly horizontal platform up to 400 m wide at 6 m below MSL has been modified by the construction of the seawall to a beachface with a nearly uniform slope of 1 in 25. At the top of this very steep beach profile sits the seawall proper with its foot at about 4 m above MSL and its crest - at about 10 m above MSL. The crest width of the wall

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FIG. 1 LOCALITY MAP

FIG. 2 AERIAL VIEW OF MINING AREA BEHIND SEAWALL

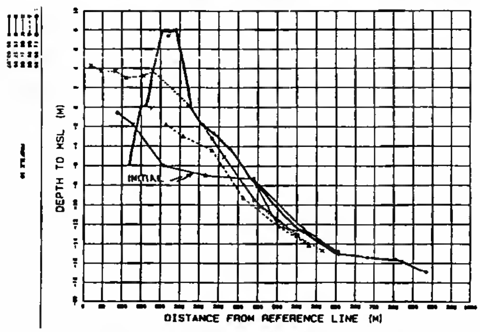

FIG. 3 BEACH AND NEARSHORE PROFILE DEVELOPMENT DUE TO SEAWALL CONSTRUCTION

varies between 17 and 25 m and the seaward slope is at the angle of repose or steeper. During extreme wave events classical dune erosion takes place and the seawall ("dune") has to be maintained on a round-the-clock basis by large earthmoving equipment. During periods when the earthmoving equipment can concentrate solely on beachwall building and maintenance it is possible to supply 16 000 m^3 of sand to the seawall per day. The seawall is being extended northwards in the direction of the littoral drift by active dumping at a rate of approximately 800 m per year. To minimise maintenance costs the area behind the seawall is divided into paddocks which are, once mined out, left to be inundated by the sea again (see Figure 2). This means that only a localised area needs to be maintained. Nevertheless the length to be maintained is of the order of 800 m at times but could also be as short as 200 m if localised erosion problems are encountered. This means that the volume of artificial nourishment to the seawall can amount to between 20 and 80 m^3/day per running metre of seawall. Under "normal" storm conditions this should be sufficient to counteract wave-induced erosion.

During a routine visit to Oranjemund on 24 and 25 February 1987 extreme wave conditions were observed. Four cross-sections were taken in front of the seawall during a 23-hour period to record the wave-induced erosion of the beach in front of the seawall and the seaward face of the seawall. This paper discusses the results obtained and relates them to available predicted techniques in literature.

2. Data

The data relevant to the particular storm is summarised here to serve as basis for use by researchers/engineers who want to verify onshore-offshore sediment models. The data can be subdivided into four types, namely, wave data, profile data, sand suppletion rates and grain size data.

2.1 Wave data

An offshore Waverider station at the site is situated in 106 m of water. The significant wave height with a 1 in 20 year return period, based on 8 years' worth of data at 6 hourly recording intervals is 4.5 m. Unfortunately it was out of commission during the event. Three other sources of wave data are available; (1) data from a Waverider situated to the south of the study area in the region from where the waves originate, (2) data on the format of that obtained by voluntary observing ships (VOS) gathered by lighter vessels operating in the area off Oranjemund and (3) visually observed wave heights at breaking by using the horizon as a reference.

Figure 4 shows the synoptic chart for 14h00 on the 22nd February 1987, which clearly shows the origin of the storm event.

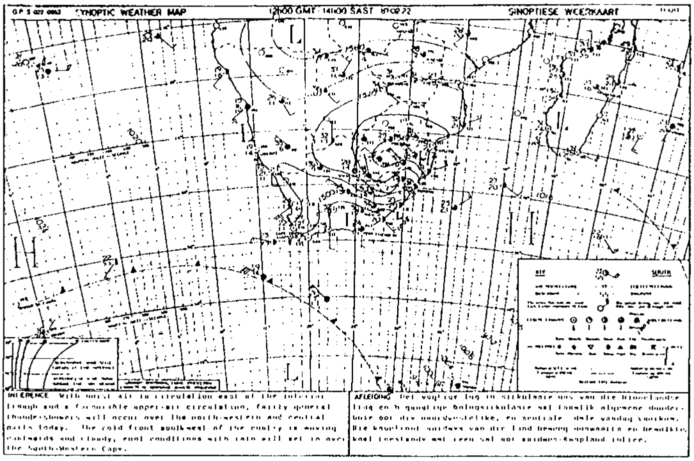

FIG. 4 SYNOPTIC CHART 14H00 : 22.02.1987

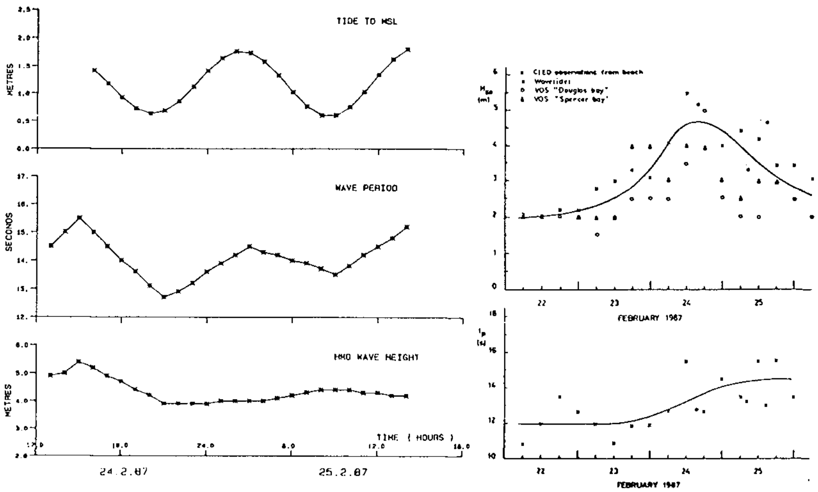

FIG. 5 WAVE DATA FROM VARIOUS SOURCES FOR FEBRUARY 1987 STORM AT ORANJEMUND

Waverider Waves in the area arrive predominantly from the south to southwesterly sectors. This is consistent with the main wave generating area which is situated in the so-called "Roaring Forties" to the southwest of South Africa. Studies have shown that wave heights attained during storm events at recording stations along the west coast are virtually the same but that a lag occurs as one moves up the coast, as is to be expected. The mean lag between Slangkop near Cape Town (see Figure 1 for location) and Oranjemund is about 24 hours. After applying the appropriate time lag the wave height variation at Oranjemund was found to be as shown in Figure 5. For reference the predicted tidal variation for this period is also shown on the figure.

Voluntary Observing Ships' data (VOS) By comparison with Waverider measurements for earlier storm events it has been found that VOS data in the area is extremely reliable. This is most probably due to the fact that two lighter vessels owned by to the De Beers Marine group operate in the area on a regular basis. The mariners have a long experience of working in this particular area. The VOS data obtained from these lighters for the February storm are also shown in Figure 5.

Visually observed maximum wave breaker heights (CLEO) As part of a low-level monitoring programme (CLEO) along similar lines as the COPE programme in Queensland, Australia and the LEO programme in the USA, maximum wave heights are estimated by using the horizon as a frame of reference. Comparison to Waverider observations has shown that provided that the appropriate conversion is made to allow for refraction and shoaling between the Waverider and the breaker line and that a suitable factor is used to account for the correlation between the maximum wave in the spectrum and the significant wave height, reliable wave heights can be estimated by this technique. In the present case with a known crest level for the seawall and the extremely high maximum pre-breaker heights this technique was easy to apply (see Figure 6 for breaking wave fury observed from an elevation of MSL + 10 m). The exposed beachface below the seawall did not exceed a width of 20 m during the event. Therefore one was in close proximity to the breaker line. By taking shots to the extreme trough of the wave just before it broke as a virtually totally plunging wave as well as to the crest and to the horizon it was established that the maximum wave height at breaking was 8.5 m. Obviously the maximum height varied with time but during the late afternoon on 24 February, the maximum height varied between 7.5 m and 8.5 m. It would appear that these heights are far in excess of those values quoted earlier although the wave periods are roughly in line. The wave heights observed in this manner represents a maximum height over a 2-4 minute period. It is possible to use the relationship developed by Longuett-Higgins (1952) to transfer the observed wave heights to significant wave heights at breaking for 13 sec waves. On the basis of the synoptic chart on Figure 4 it is possible to calculate a combined shoaling/refraction coefficient of 1.1 between

FIG. 6 BREAKING WAVE FURY AS OBSERVED FROM SEAWALL (MSL + 10M)

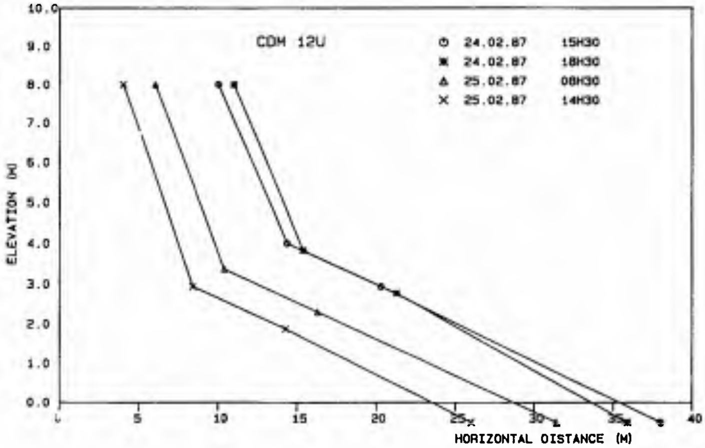

FIG. 7 BEACH FACE PROFILES MEASURED ON 24/25 FEBRUARY 1987

deep-sea and a water depth of 10 m. Therefore the wave height variation observed at the breaker line can be transferred to deep-sea significant wave heights. The results obtained in this manner are shown in Figure 5.

Summary It is clear that the data from the three sources show a similar trend, although there is some scatter, as is to be expected. The solid lines through the data represent the best estimate of the mean trend depicted in the data.

2.2 Profile data

As can be seen in Figure 3 the beach profiles are steep and the exposed beachface is at best fairly restricted. At 4 occasions during the 23-hour period a cross-section of the seawall and beachface was taken. This was done at extreme risk to the observers, because the beach face was only exposed for about two minutes at a time during which the staff-bearer had to go down a 6 m unconsolidated sand-slope onto the beach, taking off only metres ahead of the next onrushing wave. The four profiles are given in Figure 7. In the assessment of the profile changes observed due cognizance should be taken of the general morphological behaviour of this area.

The dominant wave direction in the area is approximately south-westerly which leads to a net northerly drift of sediment of about 1.4 x 10^6 m^3/yr. The seawall acts as an obstruction and material is deposited on the beach south of the seawall. The fact that the seawall is maintained more than 200 m from the original waterline results in high sediment losses and extremely high sediment concentrations in the water. The suspended sediment is moved northwards and is deposited on the downdrift beach. This results in the unusual situation that the shoreline grows on both the updrift and downdrift ends of the seawall in the long term (see Figure 8).

During the event the seawall was perfectly straight. In addition, the area where the observations were done was more than a kilometre from the then northern extremity of the seawall. Waves broke parallel along a fairly long section of seawall. It can be concluded that the effect of longshore transport on the profile development can be neglected as a first approximation.

2.3 Sand nourishment

For the reasons outlined above, the nourishment rates along the seawall are fairly high, as can be seen on Figure 9, which contains monthly nourishment rates over a three year period. The volume of material given in this figure is the total volume needed for (1) so-called seawall building to advance the seawall to the north and (2) for maintenance purposes.

It was obvious that the seawall was steadily being eroded. Therefore although the general policy has been to

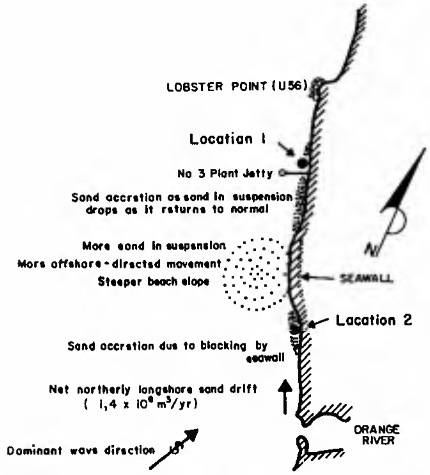

FIG. 8 SCHEMATIC REPRESENTATION OF NEARSHORE PROCESSES ADJACENT TO SEAWALL

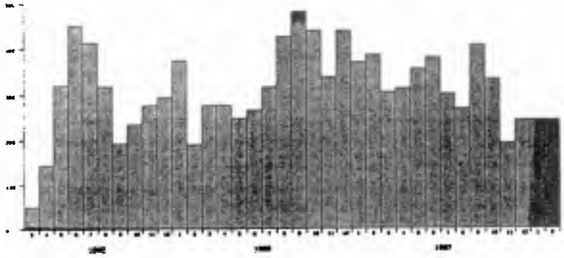

FIG. 9 INSHORE MINING PROJECT : MONTHLTY FEEDING RATES

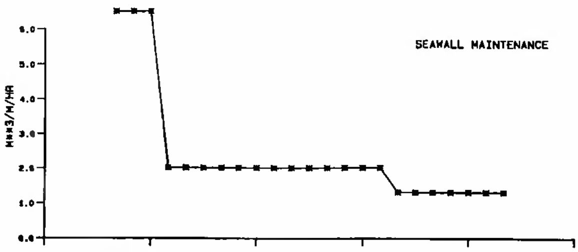

FIG. 10 SEAWALL MAINTENANCE DURING FEBRUARY 1987 STORM EVENT

work on a 24-hour a day basis with the effort being split between beachwall building and beachwall maintenance, all earth-moving equipment was diverted onto maintaining the paddock being stripped and mined at the time. The seaward face of this paddock is 500 m long. The material was being actively dumped on 400 m of sea wall along this paddock during the 24-hour period to try and curtail wave-induced erosion. A total of 1021 15 m^3 truckloads of sand was dumped onto the sea wall and dozed into the water by bulldozer.

On the basis of the actual area renourished over time during the 24 hour period it was possible to construct Figure 10, which is the best estimate of seawall maintenance during the event, represented in cubic metres maintenance per metre of wall.

2.4 Grain size data

Available grain size data for the seawall area indicate that the median grain size is about 200 microns.

3. Comparison with predictive models

The data set described herein represents an extreme erosion event. It creates the opportunity to test onshore-offshore predictive models against such an extreme offshore sediment loss event. In this section two such comparisons are given, namely, with the D-profile concept of Swart (1974a, 1974b, 1976, 1986) and with the energetics approach of Nairn (1988). The full data set is available from the authors.

3.1 Profile concept of Swart

The empirical approach of Swart has been well documented. It uses of the concept that an equilibrium profile will eventually be formed if a given wave condition persists long enough. At any time during the transition from the initial conditions to the final (equilibrium) condition the quantity of onshore or offshore sediment transport taking place is proportional to the difference between the schematized profile at that time and the schematized equilibrium profile. In order to facilitate the profile schematization, the beach profile is broken into essentially three parts, namely, (1) that portion of the profile which is above (or landward of) the point of maximum uprush, the so-called **backshore**, (2) the area between the point of maximum uprush and the lower limit of the actively developing profile, which is characterized by very active sediment transport (mostly as suspended load) and by well-developed bars and other bedforms, the so-called developing profile or D-profile, and (3) the area below (or seawards of) the lower limit of the D-profile, where a transition basically takes place between the newly developing profile and the original profile. This last area is called the transition area and transport here usually takes place as bed load. Swart (1974a) compared his theoretically based framework with extensive model and field data on profile development and presented empirical relationships for all parameters required to apply the technique in a predictive mode.

Over the past 14 years the technique has been extensively applied to a variety of coastal applications, with a large degree of success. Seymour (1988) concluded in a paper presented at this (ICCE) conference that the technique has the highest success rate of the onshore-offshore predictive models tested in his study.

Using the data described above a set of model input was prepared. Application of the model yielded satisfactory profile remoulding (Figure 11) and loss rates. Figures 12 and 13 which are highly compatible with the observed values, as deduced from the data presented earlier. Volumetric loss from above the water line is exceptionally close to the observed value, namely 130 m^3/m as opposed to 123 m^3/m over the 23 hour period. However, variations in the 0 m and +5 m contours are not as accurately predicted, specifically over the last 5 to 10 hours of the period. Nevertheless, the overall comparison is good and it can be concluded that this fairly simplistic method can give reliable results. A strong disadvantage of this technique is the fact that large-scale bedforms such as profile perturbations and breaker bars cannot be predicted.

3.2 Energetics Approach of Nairn

Bailard (1981) developed a model for the prediction of cross-shore sediment transport which utilised an energetics approach. Bailard assumed that instantaneous sediment transport is proportional to some power of the instantaneous orbital velocity at the bed. Stive (1987) used the same technique for random waves, and used the Stokes II wave theory to compute odd velocity moments needed as input for the model. Both bed load and suspended load are catered for, in three modes of transport, namely, asymmetric (onshore) transport, mean flow (offshore) transport and downslope (offshore) transport due to gravity. Nairn (1988) used the same techniques as Stive. As far as waves are concerned, the dissipation model for random breaking waves of Battjes and Janssen (1978) was used, whilst transfer of individual regular wave components in a probability density function was done according to the method of Dally (1987). Bottom changes were calculated using a Lax-Wrendroff second-order scheme based on the flux of sediment across a finite difference section of the model (Nairn 1988). Rob Nairn graciously ran his model on the Oranjemund data set. The results obtained when applying the Nairn model are given in Figure 14. It shows that a very realistic profile readjustment took place, although reference to Figure 15 shows that the volumetric loss rates predicted by the model are too low by a factor of approximately two. This is not considered an insurmountable problem as the loss rates would be appreciably improved if the model could cater adequately for sediment removal from above the water line, which is obviously not a function of the mathematical technique used

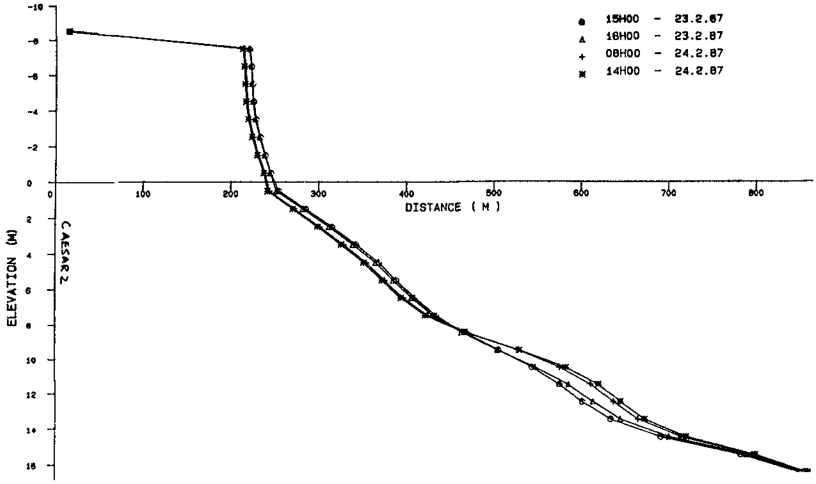

FIG. 11 MODEL PROFILE REMOULDING

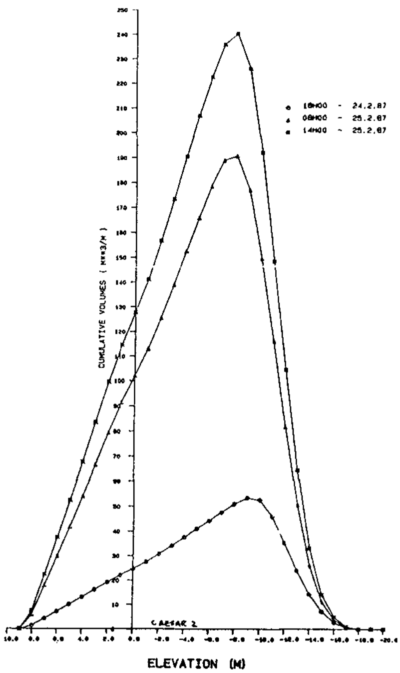

FIG. 12 MODEL EROSION RATES

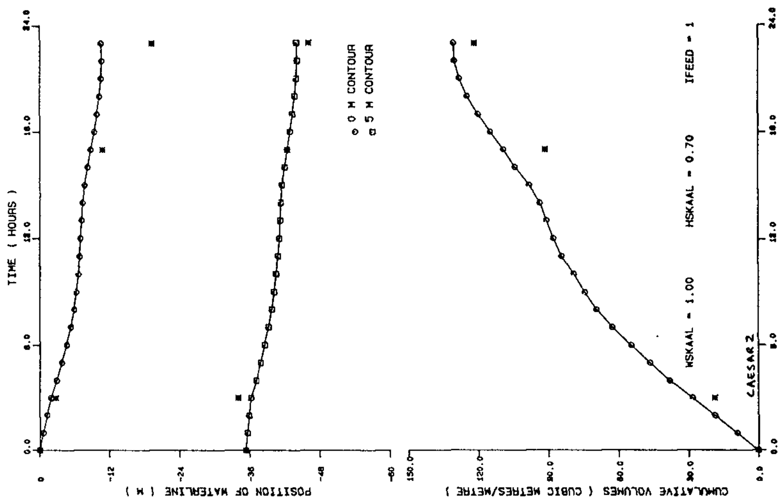

FIG. 13 WATERLINE VARIATION AND VOLUMETRIC LOSS

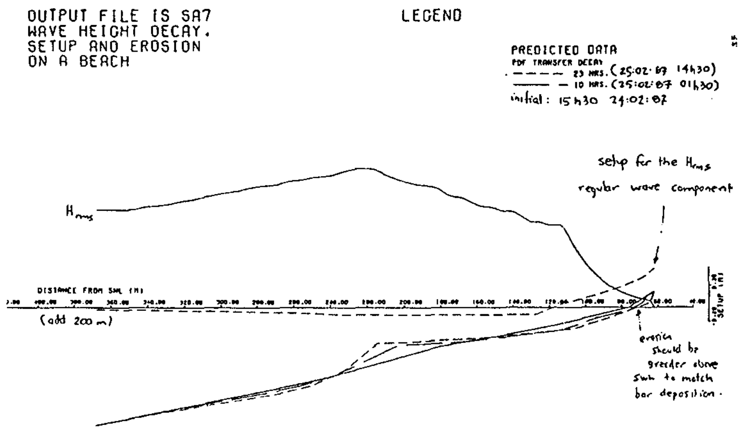

FIG. 14 MODEL PROFILE REMOULDING (NAIRN)

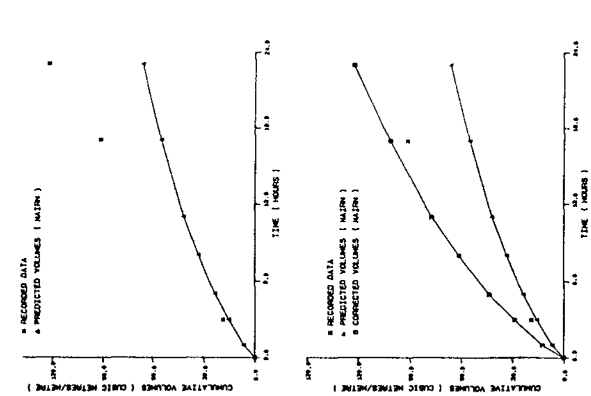

FIG. 15 RECORDED AND PREDICTED EROSION RATES

to calculate sediment movement seawards of the water line.

The results depicted in Figure 14 exhibit only a monotonic bar formation. It may be the result of the method in which bed profile changes are calculated presently which needs some pursuing.

Discussion of the results with Nairn lead to a quite plausible explanation for the discrepancy (Nairn, pers. comm.).

The band width of the input bivariate distribution of wave height and period does not have any significant effect on the predicted volumetric rates and can thus not explain the discrepancy. However, de Vriend (1987) suggests that in severe erosion conditions the suspended load exhibits a retardation in its response to a change in the transport capacity. Consequently, the sediment load at a point is partly determined by what has happened further upstream. This can lead to either underloading (actual transport < potential transport) or conversely, overloading (the so-called autosuspension condition). It is further stated that during the erosion of an unprotected dune a large overload of sediment is carried seawards by the undertow and the material is spread very quickly over a large area, which has a considerable influence on the dune erosion process.

The presently used theoretical development of the energetics sediment transport model only relates the sediment transport to the energetics of the flow. In lieu of the comments by de Vriend (1987) autosuspension will occur if

$$w < \bar{u}_s \tan \beta$$

where w is the fall velocity of the sand grains, $\bar{u}_s$ is the mean flow velocity advecting the suspended load and tan ß is the profile slope. It can be shown easily that autosuspension conditions could occur for many of the large waves in the spectrum during the storm event. Reasons for this include (1) the unnaturally steep slope of 1:20, even for a grain size of 0.5 mm (2) the wave exposure and potential for rapid return flows or undertow and (3) a steep dune face inducing avalanching which will trigger autosuspension. Moreover, autosuspension offers a plausible explanation of how so much material could be rapidly eroded and not show up as a pronounced bar deposit. Autosuspension can lead to self-maintaining turbidity currents which will carry and spread the sediment well outside the surfzone.

In conclusion, the Nairn approach yields a promising comparison with the observed profile changes, although the rate of change is at present still underpredicted.

4. Conclusions

* A data set was presented which represents an extreme wave-induced erosion event, with a volumetric loss rate from above the water line of approximately 130 m^3/day. The data are available on request.

* Computations with the semi-empirical D-profile approach of Swart (1986) and with the energetics model of Nairn (1988) show that it is possible to predict such extreme events and open some interesting avenues for more detailed research on the mechanism of extreme wave-induced sediment transport.

5. Acknowledgments

The logistic support of the Consolidated Diamond Mines (Pty) Ltd in compiling the data set is gratefully acknowledged. Thanks are also due to Rob Nairn of the Imperial College of Science and Technology who spent a substantial amount of time and effort to model this event with his technique.

6. References

BAILARD, J.A. (1981). An energetics bedload model for a plane sloping beach: Local transport. Journal of Geophysical Research. Vol. 86, No C3

BATTJES, J.A. and JANSSEN, J. (1978). Energy loss and setup due to breaking of random waves. Proc 17th International Conference on Coastal Engineering

DALLY, W.R. (1987). Wave transformation in the surf zone. Ph.D. Thesis, University of Florida

de VRIEND, H.J. (1987). Two and three-dimensional modeling of coastal morphology. Recent developments at the Delft Hydraulics Laboratory. Report No 377

LONGUETT-HIGGINS, M.S. (1952). On the statistical distribution of the heights of sea waves. Journal of Marine Research, Vol XI, 1952

MÖLLER, J.P., OWEN, K.C. and SWART, D.H. (1986). Coastal engineering studies for inshore mining of diamonds at Oranjemund. Proc. 20th International Conference on Coastal Engineering. Taiwan

NAIRN, R.B. (1988). Prediction of wave height and mean return flow in cross shore sediment transport modeling. IAHR Symposium on Mathematical Modeling of Sediment Transport in the Coastal Zone. Copenhagen

SEYMOUR, R.J., CASTEL, D. (1988). Validation of cross-shore transport formulations. Proc. 21st International Conference on Coastal Engineering, Malaga

STIVE, M.J.F. (1987). A model for cross-shore sediment transport. Proc. 20th International Conference on Coastal Engineering, Taipei

SWART, D.H. (1974a). Offshore sediment transport and equilibrium beach profiles. Doctoral thesis, Technische Hogeschool, Delft

SWART, D.H. (1974b). A schematization of on-offshore transport. Proc. 14th International Conference on Coastal Engineering

SWART, D.H. (1976). Predictive equations regarding coastal transports. Proc. 15th International Conference on Coastal Engineering. Honolulu, Hawaii

SWART, D.H. (1986). Prediction of beach changes and equilibrium beach profiles. Lecture notes for short course on : Dynamics of Sand Beaches. Taipei, Taiwan