ENVIRONMENTAL LOADS DESIGN CRITERIA FOR NEARSHORE STRUCTURES

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The standards for the design of nearshore structures usually refer to loading induced by currents, waves, wind and water level. The design of structures, including coastal structures located nearshore (e.g. brekwaters, piers, tresles, intakes and outfalls) deals normally with two critical loading states. The first design state referes to the critical loading occuring in the ultimate service state and the second design state refers to the critical loading occuring in the ultimate survival state. For the first design state the standards appears to be well defined. For the second design state however, for structures sited near the surf zone edge, particularly for these close but beyond the "normal" surf zone, the present standards consulted, seem to lack proper consideration of an important loading factor due to the development of very strong currents, induced by wind and breaking waves. The insufficient definition of the design loading criteria in the survival state due to the combined wind and wave induced longshore current in a number of international standards is indicated. An assessment of the resulting loading in the ultimate design state due to the wind and wave induced currents is shown to result in significantly increased velocities and accelerations, hence increased structural loading (as the structures are in this state at the edge of or within the breakers' zone),.

INTRODUCTION

The environmental loading design criteria for nearshore structures are defined by various standards and design manuals. These criteria usually refer to the loading due to currents, waves, wind and water level. The usual design of coastal structures, including those located nearshore refers to two critical loading states. The first referres to the ultimate service (operation) state of the structure and the second one referres to the ultimate survival state of the structure (see for example the American Petroleum Institute; Det Norske Veritas; Eurocode and International Standards Organization respective standards).

The design criteria for the first condition are well documented in our opinion. However, the standards' criteria for the ultimate (survival) design state, for the situation when the coastal structure is located at the edge of the surf zone, and in particular close but beyond the "normal" surf zone, seem to be lacking proper consideration of an important loading element. This element is due to very large wave induced longshore currents, developing during the ultimate design condition, normally defined by the standards as the one corresponding to the 100 years average return period sea state. The present guidance for the design of

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nearshore structures (e.g. Det Norske Veritas 1998, Det Norske Veritas 2000) recommends to account for the extreme combined loading of currents and waves as follows:

"If sufficient information is available on joint probability of waves and current, then the combined wave and steady current with 100 year recurrence interval should be used. If inadequate information is available on the joint probability of waves and current, then the following are suggested for the operational condition:

If waves forces dominates	Waves :	100 year return condition of near bottom wave-induced particle velocity normal to the pipeline
	Current :	10 year return condition.
If current forces dominates	Waves :	10 year return condition.
	Current :	100 year return condition."

However, nowadays it is feasibile to conduct hindcasting of wind and wave induced currents using numerical hydrodynamic models based on long term atmospheric and wind forcing from global data bases of 40 years and more. This has been already recognized in the new ISO draft standard (International Standards Organization 2006), recommending that in case of lack of data numerical modeling can be carried out. Consequently, this enables to determine reliably the 100 year current statistics in most places on the globe and hence the recommended practice quoted above is in our view not justified anymore, and the design should be based at least on the 100 year average return period, particularly in the case of design of major/key coastal structures.

The 100 years average return period is in our view a compromise due to the lower reliability for assessing the values of the sea level, wind, wave and currents for larger average return periods, as usually advised by normal coastal engineering manuals, where the design parameter is determined based on the economical life time of the structure and the risk accepted to encounter the design parameter (wave height, current speed, wind speed, sea level) during the lifetime of the structure, expressed by the following formula:

$$R(years) = \frac{1}{1-(1-r)^{1/L}} \tag{1}$$

Because that for an average return period **R** (expressed in years) of the design wave height equal to the lifetime of the structure (**L** in years) there is a risk (**r** in %) that the design wave height will be exceeded during the economical lifetime of the structure of about 64% (unacceptable), one has to select the design wave height with a much lower risk of encounter. This is possible to compute by formula (1). Table 1 presents a number of cases for various r, L and R values, which show that using the 100 years average return period for a structure with and economic lifetime of 20 years or more poses a relatively high risk of encountering the design parameter of higher.

Table 1. Design wave height versus encounter risk and economical lifetime of the marine structure

Risk to Encounter of Design Wave [percentages]	Economical Life Time of Structure (years)							
	2	4	6	8	10	15	20	50
	Average Return Period to be Used							
1	200	398	597	796	995	1493	1990	4975
5	39	78	117	156	195	293	390	975
10	19	38	57	76	95	143	190	475
20	10	18	27	36	45	68	90	225
50	4	6	9	12	15	22	29	73
64	2	4	6	8	10	15	20	49

On the other hand, even the new ISO draft standard does not account for large currents developing in the ultimate design state due to the shift in the position of the structure into the surf zone or at the surf zone edge, when it may be loaded also, (depending on the wave incidence with the contour lines in this state) by very strong wave induced longshore current.

This is the present recommended practice in the quoted standards, in spite of coastal engineering general guidance, provided by a number of reputed manuals (Mei 1992, Goda 2000, U.S. Army Engineer R&D Center 2004). However, even these manuals lack in authors' oppinion sufficiently detailed guidance on this subject. Goda 2005, provides guidance for the assessment of the maximum longshore current developing in a given sea state. However, his assessment has been derived by calibration against relatively very low sea states (< 2m), without validation against measurements of longshore current in high sea states (>7m), although such measurements are very difficult to carry out in the field.

The design and construction project of 3 marine intakes for the world largest reverse osmosis desalination plant have been recently built at the Ashkelon coast, Israel, is used as demonstation example of the importance of proper consideration of the wind and wave induced longshore currents developing during extreme (ultimate) sea state. A relatively simple and rapid method of estimation of the design longshore current in this state is also presented.

CASE STUDY

The site for the desalination plant has been selected at Ashkelon coast, located at the Southeastern Mediterranean coast of Israel (Fig. 1). The desalination plant has a total capacity of desalination of 100 million cubic meters per year. The desalination water is pumped from the sea via 3 HDPE burried pipelines connected to 3 intake heads placed on the -15m water depth contour (Fig.2). The perpendicular to the coast and contour lines makes an angle of about 34 deg with the predominant wave direction, approaching in deep water from West. The technical specifications for the design and construct tender which were prepared by the first author, required the intakes to be designed for the most critical conditions arriving for: (a) a design wave height of Hmax associated with a sea state with a deep water characteristic (significant) wave height sea state with a 100 year average return period or (b) the maximum depth limited wave height (at the site of the structures) with an average return period of 100 years and identified a deep water significant wave height of 8.7m for a 100 year average return period.

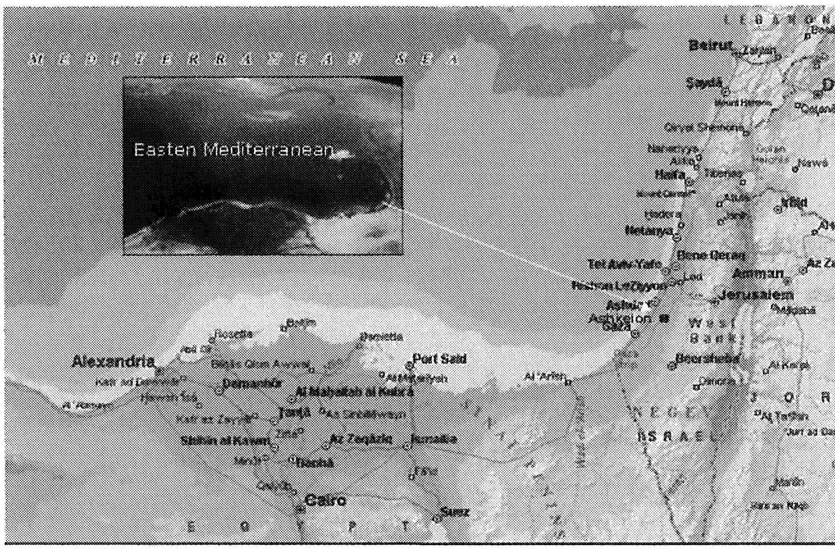

Figure 1. General location map of the Ashkelon desalination plant

Figure 2. Plan of the desalination intake at Ashkelon

Additional elements of the requested design are described below:

The maximum total intake discharge is 35,200 m^3/h, i.e. 308 M m^3/year, the top of the heads are 5m below MSL., the 3 HDPE pipelines have an outer diameter of 1.6 m and the economical lifetime of the intake system was chosen by the project owner as 25 years. Statistical data regarding sea level, wind, waves and currents are presented below to show normal selection of environmental loading for intake structures.

Sea levels

The tidal (astronomic) range on the Mediterranean coast of Israel is characteristic of the low-tide range of the Eastern-Mediterranean basin, being induced by the combined effect of the attraction forces of the moon and of the sun, and by the location of this area on the globe. The tide usually varies between 0.40m during spring tides (occurring in spring and autumn), and 0.15m during neap tides (occurring in winter and summer). The tide contribution exhibits the usual semi-diurnal periodicity (twice a day highs and lows) and fortnight (14 days) periodicity. Extreme sea levels may occur in combination with extreme meteorological conditions. Low sea-levels occur in winter during February-March months, while high sea-levels occur in August-September, with a second maximum in December. The average return periods of extreme sea levels (excluding sea level rise) are presented in Table 2.

Table 2. Average recurrence of extreme wind gust velocities at Israel coast

Average Return Period	Low Sea Level	High Sea Level
[years]	[m]	[m]
1	-0.38	0.64
50	-0.74	1.04
100	-0.87	1.10

Wind

~77% of the **fresh** winds blow from directions W to N through NW.

~77% of the **strong** winds blow from directions SW to W trough WSW.

Table 3. Average recurrence of high wind velocities at Israel the coast

Average recurrence	Wind speed (average of 10 highest minutes in 1 hour)	
once/year	23.6 m/sec	46 knots
once/50 years	31.5 m/sec	61 knots
once/100 years	32.6 m/sec	63 knots

Table 3 presents the recurrence of high wind velocities as estimated using the Weibull distribution while the extreme gusts statistics is shown in Table 4. The directional wind distribution is shown in Fig. 3.

Table 4. Average recurrence of extreme wind gust velocities at Israel coast

Average recurrence	Upper gust wind speed	
once/year	34.7 m/sec	67 knots
once/50 years	46.3 m/sec	90 knots
once/100 years	47.9 m/sec	93 knots

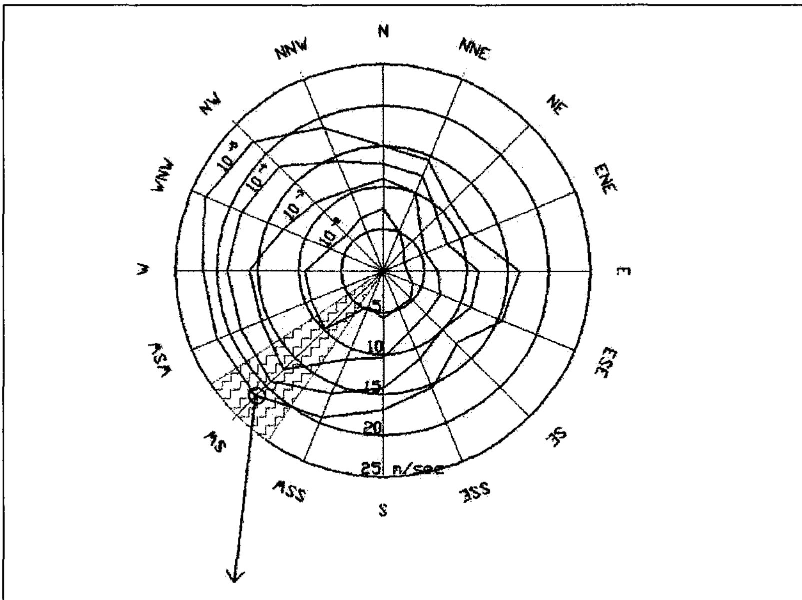

Figure 3. Weibull Extreme Directional Statistics of Hourly Wind Speeds

Waves

The year may also be divided in only two wave seasons with a transitional type beginning and end. In such a case the following division is obtained: (a) extended winter season ranging from mid November through mid April (5 months); (b) extended-summer season ranging from mid April through mid November (7 months).

The winter wave climate is characterized by alternating periods of high sea states (storms) and low sea states (calms). The storms are in general induced by cyclones passing slowly over the Mediterranean from West to East. The strongest storms usually occur in the period between mid December through the first week of March. Lower sea states occur at the beginning and at the end of that season. The prevailing wave direction is WNW, but the predominant wave direction is W, corresponding to the longest wind fetch.

During the summer season the wave climate is characterised by relatively calm seas with waves induced by the weak local winds (mainly by the breeze). Therefore, the waves are usually of "sea" type, which direction usually varies during the day in a clockwise direction from WSW in the early morning to WNW at noon and to N-NW in the afternoon.

The average yearly directional deep water significant wave distribution is characterized by:

-All moderate and higher sea states come from WSW to NNW through W

-66% of all waves approach from W trough WNW directions.

-The highest sea states approach from W direction, but storm development occurs by veering from WSW to NW through W directions.

Peak wave periods range between 3 and 16 seconds. During high sea states they range usually between 10 and 13 seconds, and very high sea states have peak periods between 12 and 16 seconds.

Extreme sea states and average return periods are presented in Table 5 below:

Table 5 Recurrence of extreme deep water significant wave heights off Israel

Average Return Period [years]	Deep Water Significant Wave Height [meters]
2	5.15
4	5.95
5	6.15
6	6.25
8	6.60
10	6.80
15	7.15
20	7.40
50	8.20
100	8.70
500	10.15

Currents

Detailed general current statistics were gathered for a period of 1 year on the -27m contour at 10m below surface. It indicated weak currents of 5-10 cm for most of the time. However, measurements carried out off Ashdod (about 15km north to the site) with an ADCP on the -24m contour and on the -15m contour showed strong currents during wave storms associated with strong local winds. An extreme value of 1m/s current was recorded at Hadera, some 65 km north to the site on the -27m contour at -11m below surface during a high sea state with deep water significant height of about 7.2m. Later on, strong currents of about 0.8m/s were recorded off the Tel-Aviv coast with an ADCP during a relatively low sea state, clearly being induced mainly by wind. This record is shown in Figure 4 in the next page.

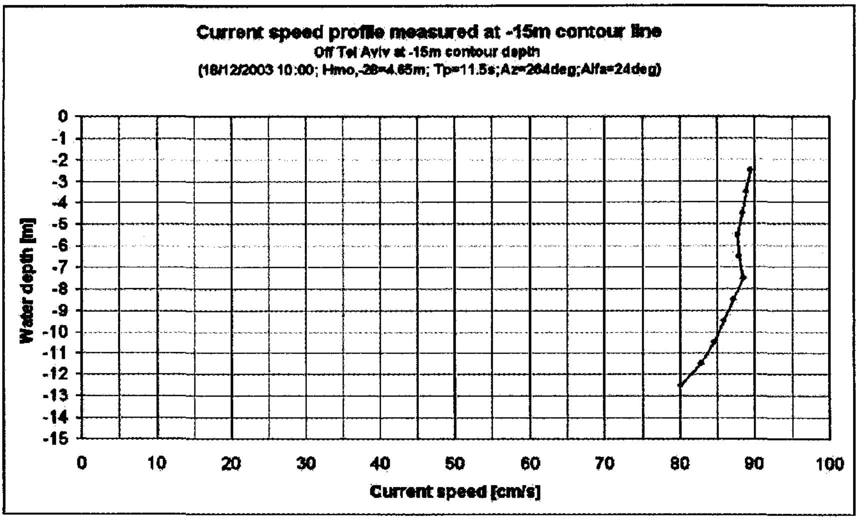

Figure 4. Record of current speed vertical profile off Tel Aviv in December 2003.

Values assessment of the environmental ultimate loading parameters

In the following we will show the insufficient definition of design loading criteria in a number of international standards under the ultimate design state.

Under such environmental conditions, simultaneous loading due to extreme wind induced currents in combination with near breaking/breaking wave loading and with large wave induced longshore currents in certain cases such as that at Ashkelon coast, where the deep water angle of the Westerly waves with the coast make an angle of 34 degrees (and because the structures are now near or within the breaking zone) lead to significantly increased resultant velocities and accelerations and consequently increased loading on the structures.

The design and build contract was awarded to and carried out by O.G. Pipeline Partnership contractor. The contractor produced a design report which assessed the forces due to the combined contributions of the (b) design condition and a general current speed of 0.7m/s, assessed as the general extreme current developing beyond the prevailing surf zone. The design review carried by the first author identified the lack of consideration of the longshore current developing under the ultimate (design) sea state. and consequently the final design was based on a total current speed of 2.0m/s in combination with the wave loading derived from (b).

The contractor assessment of the wave and current loading was based on:

- The 100 year design wave was specified to have a significant wave height of $H_{s,o} = 8.7$ m.
- An increased sea level due to sea level rise and storm surge of 1.25m was agreed to be used.
- The maximum wave height for this scenario was derived as depth limited of H_{max}-15=12.40 m.

- The maximum wave height will depend on the angle between wave direction and coast normal. In the design it was assumed that the wave height is reduced by the square of the cosine of the angle.
- The associated peak wave period was assumed between 10 to 15.3 seconds. It was concluded that the linear Airy theory is under the present circumstances equally applicable as the non-linear Stokes 5th order theory in determining the wave induced velocity and acceleration field around the intake structure.
- A maximum current speed of 0.75m/s was initially assessed at the intake site, which was changed later to 2 m/s, following the design review carried by the author, which identified the lack of consideration of the joint wind induced current and wave induced longshore current developing in the ultimate (design) sea state as the intake heads became near the surf zone edge.

It should be mentioned that in a recent paper by Kunitsa et al. 2005, stating to deal with continental shelf wind-driven currents, have assessed at this location a maximum current value induced by wind of 1.28m/s for an average return period of 100 years. However, the current values of Kunitsa et al. were obtained without considering important modelling problems of wave and wind hindcasting indicated by Cavalleri and Bertoti 2004. In this way, unaware engineers may be lead to believe that the extreme currents assessed include also the wave contributions, when in fact these are completely discarded.

The total current during the 100 year average return period sea state for the case study was assessed by a number of numerical models (Rosen 2004, Sladkevitch et al. 2004, leading to speeds of 2m/s and more, significantly larger than the original value used by the contractor (0.75m/s), or that assessed by Ronaess and Nestergard 2004 using Det Norske Veritas recommended practice and discarding extreme wind induced currents, or those assessed by Kunitsa et al. (1.28m/s).

Rosen 2004, used the NMLONG model developed by US Corps of Engineers, part of the CEDAS software package using both basically monochromatic waves equivalent to the significant and maximum wave heights assessed as well as wind induced current (see Fig. 5 and 6).

Sladkevitch et al. 2004, used both monochromatic and irregular waves derived to correspond to the Hrms of the design sea state and with wind induced current, see examples in Fig. 7a,b.

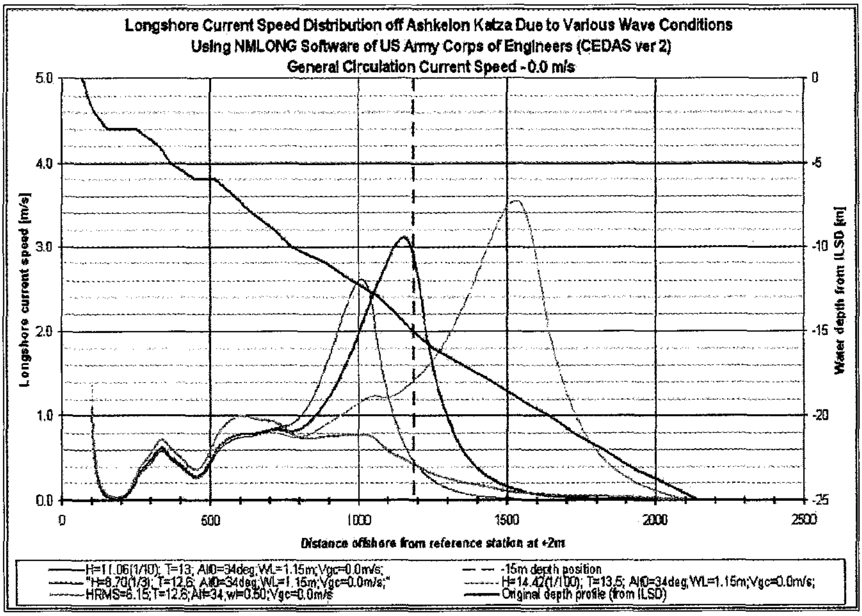

Figure 5 - Wave induced current speed at intake site without current for various H.

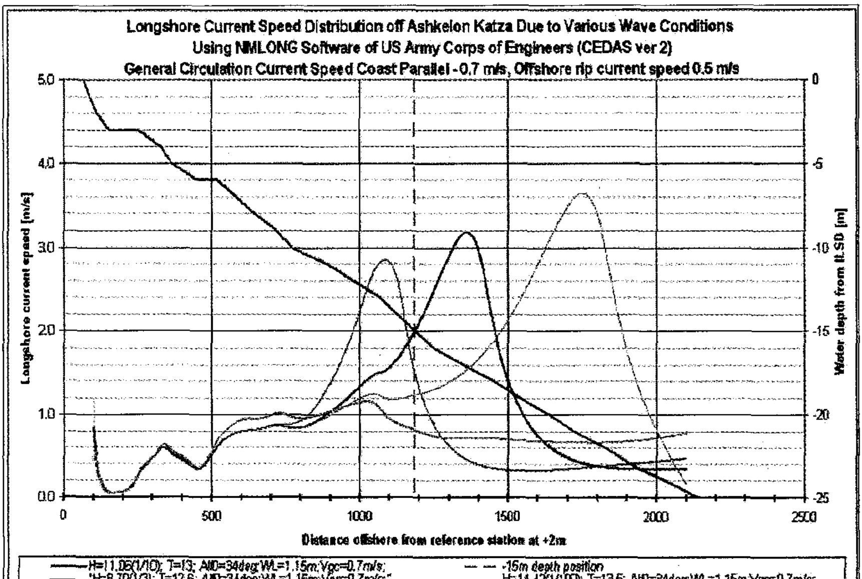

Figure 6 - Wave induced current speed at intake site with current for various H.

One of the usual problems encountered by numerical models for the assessment of the wave induced currents is that most of them use the Hrms wave height to assess the average currents induced by a sea state. However, the use of

Hrms is correct for assessing the average currents (radiation stresses), used to transport sediments or polluting materials. It is incorrect when assessing extreme wave induced currents and their loading for design at ultimate survival state.

Goda 2005, published results and empirical formulations for the computation of the distribution of the wave induced longshore current due to wave spectra and the position of the maximum longshore current speed. However, his method does not account for the wind induced currents in the extreme sea state, when it was estimated that strong winds will blow over the study area. Nevertheless, using Goda's formulation, similar values of the wave induced current to those assessed by the author were obtained. Nevertheless since the Goda formulations were calibrated for relatively low wave conditions another method for rapid assessment was seek by the author.

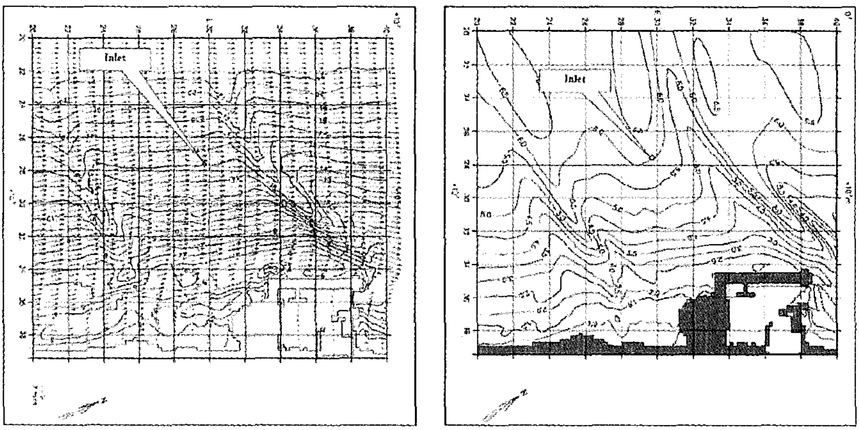

Figure 7. Current velocity (left) and Hrms (right) fields at sea bottom; random waves; CAMERI 3D HD model. Hs=8.7m (Hrms=6.2m), Tp=16.1s, waves direction 284.6,; Result U=1.7m/s

It is obvious that the maximum current velocity to be used in the Morison loading formula in the survival state is the vectorial summation of the geostrophic current, wave induced current and wind induced current (and tide if relevant).

Thus it is found that assessment of the combined wind induced and wave induced current by the recommended practice in extreme sea states is not accounted. Furthermore, the proper assessment of the maximum longshore current speed to be used in the survival state is yet difficult to assess because the usual model assessments based on the Longuett-Higgins formula uses the Hrms height value, which derives an average, not maximum current speed and because the Goda formulas were developed and calibrated against low height waves (<2m).

To correctly assess the total loading one needs to assess resulting current vector and magnitude which includes the joint contribution due to the wind induced current and to the longshore current and the orbital velocities. In this case a numerical hydrodynamic model preferably of Bousinesque type is necessary.

PROPOSED ASSESSMENT METHOD

However, a simpler assessment of the loading by a different approach is proposed, not involving a full hydrodynamic model, but merely analytic computations. This method integrates the local longshore current contributions due to each wave in the refracted waves spectrum, and selects the design longshore current speed as that due to the average of the highest 13.6% of all currents occurring during the ultimate sea state at the structure site as described shortly below.

It is proposed to assess the maximum longshore current velocity at a site located within or near but beyond the normal surf zone for loading assessment in the ultimate (survival) state as follows:

1. Determine local Hs or Hb in the survival state.
2. Use Beta- Rayleigh wave height distribution to assess the various wave heights at the site for the survival state.
3. Compute the local longshore current speed for each wave height
4. Compute the local maximum longshore current speed as the average of the highest 13.6% current values, in a similar way to the evaluation of Hs in the Rayleigh distribution.
5. Compute the resulting total current vector and magnitude by vectorial summation of the wind induced current assessed via NMLONG model using almost zero wave height and the extreme wind speed selected with the extreme wave induced longshore current.

CONCLUSION

Improved guidance on the selection of the values of the potential loading environmental parameters (sea level, wave statistics, wind statistics, tsunami and extreme lonshore currents) is provided. It is expected that some/all of the suggested guidance will be integrated in new versions of design standards of nearshore structures.

ACKNOWLEDGEMENTS

Part of the data used were gathered during the consultance provided by the author to OTID Desalination partnership for the Ashkelon desalination plant. Permission to use these data for the present paper from OTID Desalination Partnership and the assistance provided by Eng. A. Kaplan, Project Manager of the Ashkelon Desalination plant are acknowledged with thanks.

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