Cress Definition CIRIA

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A COMPANY OF

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SUMMARY

CONTENTS

1 INTRODUCTION

1.1 Project Background

In cooperation with CetMef, CUR prepares a new Manual as replacement for the CUR Manual. In line with these developments, additional rules have been proposed for implementation in CRESS. In this report, the proposed rules are reviewed. If the rule is partly presented in Cress, than the new rule of CIRIA should be implemented. In some cases, present rules in CRESS contain other definitions, or additional definitions, not presented in the CIRIA manual. Some of the proposed definitions are after review, not proposed for implementation. Taking into account the purpose and nature of the software program CRESS, some rules are too complicated or not suitable for a “one formula” program.

1.2 Contents of this report

This report provides an overview of the definitions which are proposed for implementation in CRESS. The suitability of some of the formulations is discussed. Secondly locations of the new rules are proposed in the CRESS structure as it presently exists. Some rules will replace the old rules, other rules will be added. For every definition, an overview is given of the parameters used, an overview is given of boundary and mathematical values of the definitions, and finally an overview is given of the parameters which will requested by the user and which parameters have to be defined by the user. This will help the programmer of the rules in CRESS.

1.3 Some general comments

Some of the recommendations give rise to discussion, whether the definitions are suitable for implementation in CRESS. After the first review, the definitions can be finalised.
The final CIRIA manual is not finalised. Some of the references are not correct, or are missing. Nor can the reference to the manual be given already at this moment, as the page numbers are not known at the moment, at least not for certain. This will have to be added to the help files at the time the manual is finished.
The definitions can be converted in HTML, but this will be done if all comments are made and this report is finalized.

2 OVERVIEW OF DEFENITIONS

2.1 CIRIA Definitions

The following Definitions have been requested

Rule 1 Dispersion relation calculation of k, c, etc.
p 49-50
calculation of k (box 4.3)
calculation of c, c_g, L
criteria between deep, intermediate or shallow (3rd line of table 4.6)

Formulae from the CIRIA manual are proposed for calculation of the wave number, wave celerity, wave group celerity and wave length. The formulae however are explicit *approximations* of the linear dispersion relation. In Cress the *full* linear dispersion relation is calculated iteratively (Rule A5.1), and hence the need for explicit formulations is not logical. We therefore propose only to implement the wave number as an extra output variable, the other wave characteristics already exist in CRESS.

Rule 2 Wave kinematics

p 52-53
calculation of u_x, u_y, u_z and p

The formulae proposed here are the orbital velocities and the dynamic pressure of wave motions, formulated in vector format, both in space and time. To fully calculate this, the use of CRESS is questionable. The rule A5.2 is much better for practical use. It is advised not to implement this rule.

Rule 3 Wave growth for open ocean
p 69-70
formulae from Sverdrup-Munk-Bretschneider
formulae from Wilson
formulae from Kahma and Calkoen

At present, a formulation of the wave growth by wind is implemented in CRESS under rule Z11.1 The formulations are slightly different, and hence it is advices to use the rules as presented in CIRIA. In addition to the new formulations, the formulation of Krylov should be reviewed and implemented into a new rule, or if the rule is reviewed as relevant, deleted. Whether the user can choose separate rules which formulation to use, or whether the user is presented all definitions in one rule is questionable. Due to the fact that the amount of rules is large, it is advised to create separate rules.
Reference to Massel (1996) in CIRIA is missing. The reference is found on the internet, however should be checked.

Rule 3b Wave growth for reservoirs and lakes
p71-74
Saville method
Donelan method
Young and Verhagen method

Rule 4a. Deep water distribution
p57
Wave height ratio – Table 4.8

The Rayleigh Distribution is presented to the user in the helpfile more extensively than needed. This is mainly done to give some background for the table which is presented to the user. This rule should be programmed in a table. All wave parameters have a relationship with each other, which can be easily calculated by changing one of the wave heights. The values which are presented in the definition should be default in CRESS, so the user can see the relations. After this, the user can change one of the values.

Rule 4b Deep water distribution
Calculation of H_{max}

Again, the Rayleigh Distribution is again presented to the user, without using the formulae in CRESS. This is done to present the user a more detailed description of the formulae used, and some background reading.

Rules 5a & 5b Refraction and Shoaling
p75 (eq 4.95)
calculation of K_R
p76 (eq 4.98)
calculation of K_S

These rules are proposed to calculate the K_s and the K_r in Cress. However, the Shoaling parameter Ks in already implemented in rule z12.1 and the Refraction coefficient Kr in rule z12.1

Rule 6 Breaking
box 4.7 p80
calculation of Hb (depth-limited wave height) (different methods)

The calculation of the H_b is requested using this calculation rule. The user has to define a certain breaker depth, for which CRESS calculates the breaker wave height H_b. In some cases, if the user wants to know the breaker depth given an incident wave, the user has to iteratively find the breaker depth. Three different rules are described in CIRIA. The user can calculate the breaker wave height, or use the breaker index for input in for instance rule Z12.1 One of the rules (Equation 7) uses the shallow water wave equation. Within this rule, the dispersion relation has to be calculated too according to the iterative solver in rule A5.1

Rule 7 Wave height estimation within the surf zone
Box 4.9 p83
calculation of $H_{1/3}$ and H_{max} according to Goda

This is a rather complicated rule, in terms of expression, but should be easy to use for the user

Rule 8 The CWD of wave height in shallow water
Box 4.4 p59
calculation of $H_{1/10}$ and $H_{2\%}$

The formula proposed for this rule is Composed Weibull Distribution, taking depth induced wave breaking into account in the long-term wave height distribution. However, the formulae presented in CIRIA, cannot be solved analytically. The formula which is used, is not presented in CIRIA. Reference is made to a table in the report (Derived from the original publication) with wave height relations. It is not convenient, to let the user have to look up values in the table (this is rather inaccurate too). Discussion about this rule, is needed before finalization.

Rule 9 Wind set-up
eq 4.12 p27

Rule 10 : Wave set-up
p28-29

Rule 11: Lacey's regime equation
eq. 4.3 to 4.8 (p18-19)

Rule 12: Current velocity (Manning and Chézy formulation)
p110-115
calculation of U according to Manning and Chézy formulation

Introducing box 4.14 seems not to be appropriate for CRESS. The box contains empirical formulations of estimating k_s , but requires quite some parameters which have to be given by the user. Most of these parameters however are not known in most (practical) cases. It is advised to look for a table (like the table for the manning formulation) which contains different values for k_s which are far more practical to use, given the fact that in box 4.14. It is advised that the calculation rule is designed as such, that the user can either give a Chezy coefficient by himself or, calculate the Chezy coefficient from the roughness parameter γ, or calculate the Chezy parameter by the hydraulic roughness.

Rule 13: Return current, water level depression, front and stern waves
from eq 4.167 to 4.182 (p132-134)

A definition of the coefficient c_2 is not given and it is unclear what values to use

Rule 14 Secondary ship waves
from eq4.184 to 4.186 (p135-136)

Rule 15: Propeller jet velocities
p136-138

2.2 Location of defenitions in Cress

Table 2.1 provides an overview of the location of the proposed CIRIA rules in the CRESS structure.

Table 2-1 Overview of location of CIRIA rules in CRESS structure

Water movement				
	Wind waves and swell			
		<u>Wave growth</u> Rule 3a Wave growth for open ocean Rule 3b Wave growth for reservoir and lakes		
		<u>Basics of waves</u> Rule 1 Calculation of k (Rule 2: Wave kinematics (*Simplified*)) Rule 4 Calculation of Hmax Rule 7 Wave height estimation in the surf-zone (Rule 8: The CWD of wave height in shallow water)		
		Waves near the shore		
			Shoaling waters	
				<u>Wave breaking</u> Rule 6 Calculation of depth limited wave height according to; 1. Goda 2. Weggel 3. Rattanapatikon
				<u>Set-up/Set-down</u> Rule 10 : Wave set-up
			<u>Refraction, energy decay and longshore currents</u> (Rule 5a: Refraction) (Rule 5b: Shoaling)	
	Flow			
		Open Channel flow		
			<u>Determination equilibrium Depth</u> Rule 12: Current velocity (Manning and Chézy formulation)	
	Water Levels Rule 9: Wind set-up			
	NEW: **Ship induced forces** Rule 13: Return current, water level depression, front and stern waves Rule 14: Secondary ship waves Rule 15: Propeller jet velocities			
Sediment transport				
	Sediment transport and morphology			
		<u>River morphology</u> Rule 11: Lacey's regime equation		

3 CIRIA DEFENITIONS

3.1 Wave number (Y1)

The wave number k is calculated by

$$k = \frac{2\pi}{l} \tag{1}$$

An extra output option has to be added to rule A5.1 that calculates the wave number.

3.2 Sverdrup Munk Brettschneider (Y3_1)

These formulae for estimation of wave growth by wind were originally introduced by Sverdrup and Munk (1947) and further revised by Bretschneider (1954, 1970). They appear in the third edition of the Shore protection manual [SPM] (CERC, 1977). Prediction curves for significant wave height and significant wave period based on these formulae are given in SPM (CERC, 1977) (vol I, pp 3-36 and 3-37). Note that the fourth edition of SPM (CERC, 1984) contains different wave prediction formulae and curves, based on an intermediate calculation of wind stress and modified to conform to the JONSWAP formulas. The reliability for all situations of the SPM (CERC, 1984) formulae has recently been questioned, particularly for extreme events and/or short fetch conditions. They are now considered to be less reliable than the SMB formulae and should therefore not be used for practical applications.

Equations

They allow estimation of the significant wave height Hs (m) (see Equation 1) and significant wave period Ts (s) (see Equation 2) generated by a constant and homogeneous wind. Information required is the velocity at 10 m above MSL U_{10} (m/s) blowing over a fetch of length F (m), for fully developed conditions, i.e. if the duration of wind action is greater then t_{min} (hours), t_{min} can be calculated by Equation 3.

$$\frac{gH_s}{U_{10}^2} = 0.283\tanh\left(0.0125\left(\frac{gF}{U_{10}^2}\right)^{0.42}\right) \tag{1}$$

$$\frac{gT_s}{U_{10}} = 7.54\tanh\left(0.077\left(\frac{gF}{U_{10}^2}\right)^{0.25}\right) \tag{2}$$

$$\frac{gt_{\min}}{U_{10}} = 0.00183\exp\left[\left(0.0161x^2 - 0.3692x + 2.2024\right)^{1/2} + 0.8798x\right] \tag{3}$$

in which:

$$x = \ln\left(\frac{gF}{U_{10}^2}\right) \tag{4}$$

An overview of the used parameters is given below:

parameter	short description	unit
H_s	Significant wave height	[m]
U_{10}	Wind velocity at 10m above MSL	[m/s]
g	Gravitational acceleration	[m/s^2]
F	Length of Fetch	[Km]
T_s	Significant wave period	[s]

parameter	short description	unit
t_{min}	Minimum required duration of wind action	[s]
x	Dimensionless fetch	[-]

Input and output parameters

Input:	Output:
F, U_{10}	H_s, T_s, t_{min}

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
U_{10}	Wind velocity at 10m above MSL	0 – 30 (i)	>0
F	Length of Fetch	0 – 1000 (i)	>0

References

Sverdrup, H U and Munk, W H (1947). Wind, sea and swell: theory of relations for forecasting. *HO pub no 601*, US Navy Hydrographic Office

Bretschneider, C L (1954). Generation of wind waves over a shallow bottom. *Tech Memo no 51*, Beach Erosion Board, Office of the Chief of Engineer

Bretschneider, C L (1970). "Wave forecasting relations for wave generation". *Look Lab, Hawaii, vol 1, no 3*

CERC (1977). Shore protection manual [SPM]. Coastal Engineering Research Center (CERC), US Army Corps of Engineers, Vicksburg, MS

CERC (1984). Shore protection manual [SPM]. Coastal Engineering Research Center (CERC), US Army Corps of Engineers, Vicksburg, MS

3.3 Formulae from Wilson (1965), revisited by Goda (2003) (Y3_2)

Equations

Wilson (1965) produced a set of formulae to estimate the significant wave height $H_{1/3}$ (m) (see Equation 1), the significant wave period $T_{1/3}$ (s) (see Equation 2) and the minimum duration t_{min} (hours) (see Equation 3). Information required is the velocity at 10 m above MSL, U_{10} (m/s) blowing over a fetch of length F (m), for fully developed conditions, i.e. if the duration of wind action is greater then t_{min} (hours), t_{min} can be calculated by Equation 3.

$$\frac{gH_s}{U_{10}^2} = 0.3\left(1 - \left(1 + 0.004\left(\frac{gF}{U_{10}^2}\right)^{\frac{1}{2}}\right)^{-2}\right) \tag{1}$$

$$\frac{gT_s}{U_{10}} = 8.61\left(1 - \left(1 + 0.008\left(\frac{gF}{U_{10}^2}\right)^{\frac{1}{3}}\right)^{-5}\right) \tag{2}$$

$$\frac{gt_{\min}}{U_{10}} = 0.001194\left(\frac{gF}{U_{10}^2}\right)^{0.73} \tag{3}$$

An overview of the used parameters is given below:

parameter	short description	unit
H_s	Significant wave height	[m]
U_{10}	Wind velocity at 10m above MSL	[m/s]
g	Gravitational acceleration	[m/s^2]
F	Length of Fetch	[km]
T_s	Significant wave period	[s]
t_{min}	Minimum required duration of wind action	[s]

Input and output parameters

Input:	**Output:**
F, U_{10}	H_s,T_s,t_{min}

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
U_{10}	Wind velocity at 10m above MSL	0 – 30 (i)	>0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
F	Length of Fetch	0 – 1000 (i)	>0

References

Wilson, B W (1965). Numerical prediction of ocean waves in the North Atlantic for December 1959. *Deutsche Hydrographische Zeitschrift*, vol 18, no 3, pp 114–130

Goda, Y (2003). “Revisiting Wilson’s formulas for simplified wind-wave prediction”. *J Waterway, Port, Coastal and Ocean Engg*, vol 129, no 2, pp 93–95

3.4 Formulae from Kahma and Calkoen (1992) (Y3_3)

Kahma and Calkoen (1992) have performed a detailed analysis of wind wave growth by taking into account the stability of the air-sea interface. They showed that unstable conditions lead to an increase of wave height and period and proposed two sets of formulae: one for stable conditions and one for unstable conditions, as well as a composite formula for the entire dataset.

Equations

The composite formula is in quite close agreement with the SMB and Wilson formula. The formula for unstable conditions can be used to obtain conservative estimates of wave parameters. The three sets of formulae have the same form shown by Equations 1 and 2 with values of coefficients listed in Table 1.

$$\frac{gH_s}{U_{10}^2} = A\left(\frac{gF}{U_{10}^2}\right)^B \quad (1)$$

$$\frac{gT_s}{U_{10}} = C\left(\frac{gF}{U_{10}^2}\right)^D \quad (2)$$

An overview of the used parameters is given below:

parameter	short description	unit
H_s	Significant wave height	[m]
U_{10}	Wind velocity at 10m above MSL	[m/s]
g	Gravitational acceleration	[m/s^2]
F	Length of Fetch	[km]
A	Coefficient, see table 1	[-]
B	Coefficient, see table 1	[-]
T_s	Significant wave period	[s]
C	Coefficient, see table 1	[-]
D	Coefficient, see table 1	[-]

Input and output parameters

Input:	Output:
F, U_{10}	H_s,T_s

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
U_{10}	Wind velocity at 10m above MSL	0 – 30 (i)	>0
F	Length of Fetch	0 – 1000 (i)	>0

Coefficients in equation 1 and 2	**A**	**B**	**C**	**B**
Slope angle	$3.86\ 10^{-3}$	0.38	0.5236	0.24
Number of waves	$2.94\ 10^{-3}$	0.47	0.4425	0.28
Wave steepness based on T_m	$2.88\ 10^{-3}$	0.45	0.4587	0.27

Table 1: Coefficients in the wave prediction curves of Kahma and Calkoen (1992).

References

Kahma, K K and Calkoen, C J (1992). “Reconciling discrepancies in the observed growth of windgenerated waves”. *J Phys Oceanogr*, vol 22, pp 1389–1405

3.5 Wave growth for reservoirs and lakes with effective fetch length (Y3_4)

This method uses the SMB wave prediction formulae and curves for open waters (see Equations 1 and 2), and adapts them to reservoirs using the concept of effective fetch (Saville et al, 1962). The definition of the effective fetch is illustrated in Figure 1. A noticeable feature is that the effective fetch is independent of wind speed. The effective fetch from Saville should not be used with any other wave prediction formulae than SMB: serious underestimates of wave height will result otherwise.

The formulae for estimation of wave growth by wind were originally introduced by Sverdrup and Munk (1947) and further revised by Bretschneider (1954, 1970). They appear in the third edition of the Shore protection manual [SPM] (CERC, 1977). Prediction curves for significant wave height and significant wave period based on these formulae are given in SPM (CERC, 1977) (vol I, pp 3-36 and 3-37). Note that the fourth edition of SPM (CERC, 1984) contains different wave prediction formulae and curves, based on an intermediate calculation of wind stress and modified to conform to the JONSWAP formulas. The reliability for all situations of the SPM (CERC, 1984) formulae has recently been questioned, particularly for extreme events and/or short fetch conditions. They are now considered to be less reliable than the SMB formulae and should therefore not be used for practical applications.

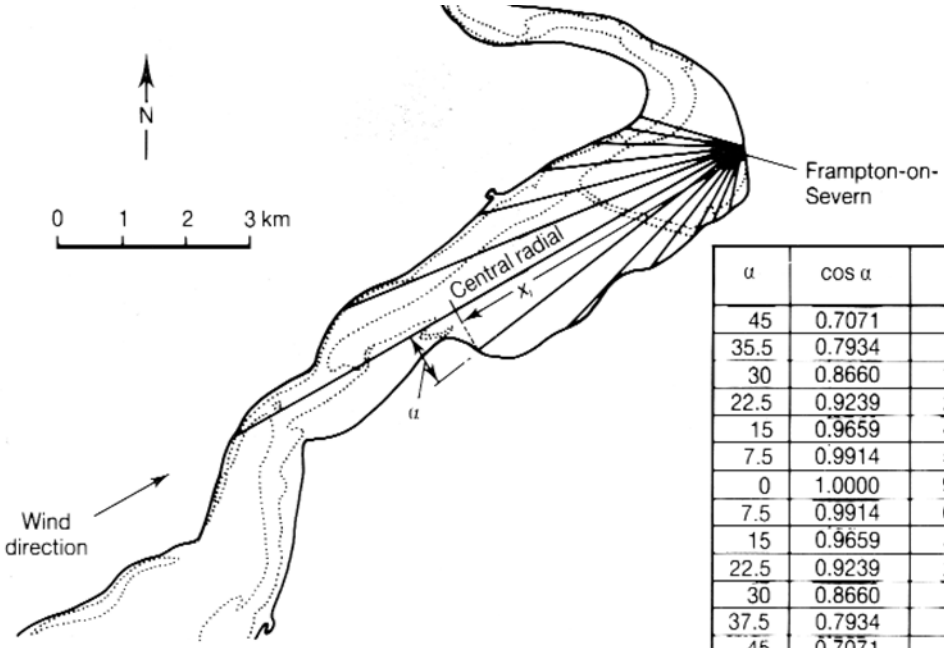
Equations

$$\frac{gH_s}{U_{10}^2} = 0.283 \tanh\left(0.0125\left(\frac{gF}{U_{10}^2}\right)^{0.42}\right) \tag{1}$$

$$\frac{gT_s}{U_{10}} = 7.54 \tanh\left(0.077\left(\frac{gF}{U_{10}^2}\right)^{0.25}\right) \tag{2}$$

in which:

$$F = \frac{\sum_{i=1}^{n} x_i \cos(\alpha)}{\sum_{i=1}^{n} \cos(\alpha)} \qquad \text{For } n \text{ radials} \tag{1}$$



α	cos α	x_i	x_i cos α
45	0.7071	800	566
35.5	0.7934	1025	813
30	0.8660	1550	1342
22.5	0.9239	2350	2171
15	0.9659	4050	3912
7.5	0.9914	5225	5180
0	1.0000	9000	9000
7.5	0.9914	6225	6171
15	0.9659	3975	3839
22.5	0.9239	2825	2610
30	0.8660	1925	1667
37.5	0.7934	1450	1150
45	0.7071	1175	831

$\Sigma = 11.4954$ $\Sigma = 39252$

$$F = \frac{\Sigma x_i \cos\alpha}{\cos\alpha} = \frac{39252}{11.4954} = 3415 \text{ m}$$

Figure 1: Example calculation of effective fetch length by Saville's method

An overview of the used parameters is given below:

parameter	**short description**	**unit**
$\mathbf{x_i}$	Length of radial *i*	[m]
$\boldsymbol{\alpha_i}$	Angle between radial *i* and central radial	[degrees]
$\mathbf{H_s}$	Significant wave height	[m]
$\mathbf{U_{10}}$	Wind velocity at 10m above MSL	[m/s]
g	Gravitational acceleration	[m/s^2]
F	Length of Fetch	[Km]
$\mathbf{T_s}$	Significant wave period	[s]

Input and output parameters

Input:	**Output:**
α, $x_{i,}$ U_{10}	H_s,T_s

Input and output parameters

Input:	**Output:**
F, U_{10}	H_s,T_s

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
U_{10}	Wind velocity at 10m above MSL	0 – 30 (i)	>0
α_i	Angle between radial *i* and central radial	-90 – 90 (f)	$-90<\alpha<90$
F	Length of Fetch	0 – 1000 (i)	>0

Bretschneider, C L (1954). Generation of wind waves over a shallow bottom. *Tech Memo no 51*, Beach Erosion Board, Office of the Chief of Engineer

Bretschneider, C L (1970). “Wave forecasting relations for wave generation”. *Look Lab, Hawaii, vol 1, no 3*

CERC (1977). Shore protection manual [SPM]. Coastal Engineering Research Center (CERC), US Army Corps of Engineers, Vicksburg, MS

CERC (1984). Shore protection manual [SPM]. Coastal Engineering Research Center (CERC), US Army Corps of Engineers, Vicksburg, MS

Saville, T, McClendon, E W and Cochran, A L (1962). “Freeboard allowance for waves in inland reservoirs”. *Proc Am Soc Civ Engrs*, vol 18, no WW2

Sverdrup, H U and Munk, W H (1947). Wind, sea and swell: theory of relations for forecasting. *HO pub no 601*, US Navy Hydrographic Office

3.6 Donelan method (Y3_5)

The Donelan method is presented in a series of papers (Donelan, 1980; Bishop and Donelan, 1989; Bishop et al, 1992; Donelan et al, 1992). It is based on the idea that the fetch length should be measured along the wave direction rather than the wind direction and that the wind speed used for wave prediction should therefore be the component along the wave direction. The method does not assume coincident wind direction, φ_w, and wave direction, θ. If the gradient of fetch about wind direction is large, one can expect that the wave direction is biased towards longer fetches. For long and narrow water bodies the wave direction is probably along the water body axis for a wide range of wind directions (rather than the wind direction). Differences up to 50° for | φ_w - θ | have been observed on Lake Ontario. For fetches of general shape, the predominant wave direction was assumed to produce the maximum value of wave period (for a given wind speed). For a point with known fetch distribution F_θ (F_θ is the fetch along the direction θ), the relation between the wave direction, θ , and the wind direction, φ_w, can be obtained by maximising the product $\cos(\varphi_w - \theta)\, F_\theta^{0.426}$. For any irregular shoreline, and a given wind direction, the value of θ satisfying this condition can only be determined by trial and error (Bishop and Donelan, 1989; Massel, 1996). As θ is independent of wind speed only one set of calculations is needed for a particular water body. Once θ has been determined, the significant wave height, peak period and minimum wind duration are derived from Equations 1–3 (modified from the JONSWAP formulae).

$$\frac{gH_s}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2} = 0.00366\left(\frac{gF_\theta}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2}\right)^{0.38} \qquad (1)$$

$$\frac{gT_p}{U_{10}\cos(\theta-\varphi_w)} = 0.542\left(\frac{gF_\theta}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2}\right)^{0.23} \qquad (2)$$

$$\frac{gt_{\min}}{U_{10}\cos(\theta-\varphi_w)} = 30.1\left(\frac{gF_\theta}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2}\right)^{0.77} \qquad (3)$$

The value of the directional fetch, F_θ, is limited by the criterion expressed by Equation 4 to avoid over-development of wave energy.

$$\frac{gF_\theta}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2} \le 9.47\cdot 10^4 \qquad (4)$$

At this value of non-dimensional directional fetch, F_θ, fully development of waves is reached, resulting in Equations 5 and 6.

$$\frac{gH_s}{\left(U_{10}\cos(\theta-\varphi_w)\right)^2} = 0.285 \qquad (5)$$

$$\frac{gT_p}{U_{10}\cos(\theta-\varphi_w)} = 7.56 \qquad (6)$$

An overview of the used parameters is given below:

parameter	short description	unit
H_s	Significant wave height	[m]
U_{10}	Wind velocity at 10m above MSL	[m/s]
g	Gravitational acceleration	[m/s^2]
F_θ	Length of Fetch along direction θ	[km]
T_p	Peak wave period	[s]
θ	Wave direction	[Degrees]
φ_w	Wind direction	[Degrees]

Input and output parameters

Input:	Output:
F, U_{10}	H_s,T_p,t_{min}

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
U_{10}	Wind velocity at 10m above MSL	0 – 30 (i)	>0
F	Length of Fetch	0 – 1000 (i)	>0
θ	Wave direction		0 - 360
φ_w	Wind direction		0 - 360
$\|\theta- \varphi_w\|$	Difference in wave- and wind direction		<50

References

Bishop, C T and Donelan, M A (1989). Wave prediction models. In: V C Lakhan and A S Trenhaile (eds), Applications in coastal modelling. Elsevier, Amsterdam (ISBN 0-44487452-6), pp 75-106

Bishop, C T, Donelan, M A and Kahma, K K (1992). Shore Protection Manual's wave prediction reviewed. Coastal Engg, vol 17, no 1, pp 25-48

Donelan, M A (1980). Similarity theory applied to the sea forecasting of wave heights, periods and directions. In: Proc Canadian coastal conf, pp 47-61

Donelan, M A, Skafel, M, Graber, H, Liu, P, Schwab, D and Venkatesh, S (1992). On the growth rate of wind-generated waves. Atmos-Ocean, vol 30, pp 457-478

S.R. Massel, *Ocean Surface Waves; their Physics and Prediction*. World Scientific Publ., Singapore - New Jersey - London - Hong Kong, 1996, 491pp Reference missing in CIRIA.

3.7 Wave Height Relations (Rayleigh Distributed) (Y4_1)

In deep water the water surface elevation usually follows a Gaussian process and thus the individual wave heights closely follow the Rayleigh distribution. This distribution is fully defined by a single parameter, which may be either the mean wave height H_m or the root mean square (rms) wave height H_{rms} , or alternatively the variance of the free-surface elevation m_0 .

Equations

Equation 1 gives the equivalent forms of the cumulative distribution function.

$$P(H) = P(\underline{H} < H) = 1 - \exp\left(-\frac{H^2}{8m_0}\right) = 1 - \exp\left(-\frac{\pi}{4}\left(\frac{H}{H_m}\right)^2\right) \tag{1}$$

$$= 1 - \exp\left(-\left(\frac{H}{H_{rms}}\right)^2\right)$$

Equation 2 gives the corresponding probability density function

$$p(H) = \frac{H}{4m_0}\exp\left(-\frac{H^2}{8m_0}\right) = \frac{\pi}{2}\frac{H}{H_m^2}\exp\left(-\frac{\pi}{4}\left(\frac{H}{H_m}\right)^2\right) \tag{2}$$

$$= \frac{2H}{H_{rms}^2}\exp\left(-\left(\frac{H}{H_{rms}}\right)^2\right)$$

The variance m_0 can be computed from the free-surface elevation signal η(t) or from the wave spectrum E(f) (it corresponds to the area between spectrum and the x-axis).

$$m_0 = \eta_{rms}^2 = \frac{1}{T}\int_0^T \left(\eta(t) - \overline{\eta}\right)^2 dt \tag{3}$$

A shortcoming of the Rayleigh distribution is that it is not bounded by an upper maximum
value. Thus the maximum wave height can neither be defined nor computed in a deterministic way from this distribution. However, the representative wave heights $H_{P\%}$ and $H_{1/Q}$ can be computed analytically (see Equations 3 and 4) from the Rayleigh distribution (eg Massel, 1996; Goda, 2000).

$$\frac{H_{p\%}}{H_{rms}} = \sqrt{-\ln\left(p/100\right)} \tag{3}$$

$$\frac{H_{1/Q}}{H_{rms}} = Q\,erfc\left(\sqrt{\ln Q}\right) + \sqrt{\ln Q}, \qquad erfc(x) = \int_{x}^{+\infty} \exp\left(-t^2\right) dt \tag{4}$$

The most important and useful results are listed in Table 1.

Coefficients in equation 1 and 2	$H/\sqrt{m_0}$	H/H_m	H/H_{rms}	H/H_s
Standard deviation of free surface $\sigma_\eta = \sqrt{m_0}$	1	0.399	0.353	0.25
Mean wave height H_m	2.507	1	0.886	0.626
Root-mean-square wave height H_{rms}	2.828	1.128	1	0.706
Significant wave height $H_s = H_{1/3}$	4.004	1.597	1.416	1
Wave height $H_{1/10}$	5.09	2.031	1.8	1.273
Wave height $H_{1/100}$	6.673	2.662	2.359	1.668
Wave height $H_{2\%}$	5.594	2.232	1.978	1.397

Table 1: Characteristic wave height ratios for a sea-state with a Rayleigh distribution of wave heights

An overview of the used parameters is given below:

parameter	short description	unit
σ_η	Standard deviation of free surface	[m]
m_0	Zero-th order moment of the variance density spectrum	[m.m]
H_m	Mean wave height	[m]
H_{rms}	Root-mean-square wave height	[m]
$H_s = H_{1/3}$	Significant wave height	[m]
$H_{1/10}$	Mean of 1/10 of the highest wave heights	[m]
$H_{1/100}$	Mean of 1/100 of the highest wave heights	[m]
φ_w	Wind direction	[Degrees]

Input and output parameters

Input:	Output:
$\sigma_\eta = \sqrt{m_0}$, H_m, $H_s = H_{1/3}$, $H_{1/10}$, $H_{1/100}$	$\sigma_\eta = \sqrt{m_0}$, H_m, $H_s = H_{1/3}$, $H_{1/10}$, $H_{1/100}$

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Columns
σ_η	Standard deviation of free surface	0 – 2.5 (i)	>0	See table 1
m_0	Zero-th order moment of the variance density spectrum	0 – 10 (i)	>0	See table 1
H_m	Mean wave height	0 – 10 (i)	>0	See table 1

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Columns
H_{rms}	Root-mean-square wave height	0 – 10 (i)	>0	See table 1
$H_s = H_{1/3}$	Significant wave height	0 – 10 (i)	>0	See table 1
$H_{1/10}$	Mean of 1/10 of the highest wave heights	0 – 10 (i)	>0	See table 1
$H_{1/100}$	Mean of 1/100 of the highest wave heights	0 – 10 (i)	>0	See table 1
φ_w	Wind direction	0 – 10 (i)	>0	See table 1

References

Goda, Y (2000). Random seas and design of maritime structures. *Advanced Series on Ocean Engg, vol 15*, World Scientific, Singapore, 444 pp

S.R. Massel, *Ocean Surface Waves; their Physics and Prediction*. World Scientific Publ., Singapore - New Jersey - London - Hong Kong, 1996, 491pp

3.8 Rayleigh Distributed Maximum Wave Heights (Y4_2)

In deep water the water surface elevation usually follows a Gaussian process and thus the individual wave heights closely follow the Rayleigh distribution. This distribution is fully defined by a single parameter, which may be either the mean wave height H_m or the root mean square (rms) wave height H_{rms}, or alternatively the variance of the free-surface elevation m_0 .

Equations

Equation 1 gives the equivalent forms of the cumulative distribution function.

$$P(H) = P(\underline{H} < H) = 1 - \exp\left(-\frac{H^2}{8m_0}\right) = 1 - \exp\left(-\frac{\pi}{4}\left(\frac{H}{H_m}\right)^2\right) \qquad (1)$$

$$= 1 - \exp\left(-\left(\frac{H}{H_{rms}}\right)^2\right)$$

Equation 2 gives the corresponding probability density function

$$p(H) = \frac{H}{4m_0}\exp\left(-\frac{H^2}{8m_0}\right) = \frac{\pi}{2}\frac{H}{H_m^2}\exp\left(-\frac{\pi}{4}\left(\frac{H}{H_m}\right)^2\right) \qquad (2)$$

$$= \frac{2H}{H_{rms}^2}\exp\left(-\left(\frac{H}{H_{rms}}\right)^2\right)$$

The variance m_0 can be computed from the free-surface elevation signal η(t) or from the wave spectrum E(f) (it corresponds to the area between spectrum and the x-axis).

$$m_0 = \eta_{rms}^2 = \frac{1}{T}\int_0^T \left(\eta(t) - \overline{\eta}\right)^2 dt \qquad (3)$$

A shortcoming of the Rayleigh distribution is that it is not bounded by an upper maximum
value. Thus the maximum wave height can neither be defined nor computed in a deterministic way from this distribution. However, the representative wave heights $H_{P\%}$ and $H_{1/Q}$ can be computed analytically (see Equations 4 and 5) from the Rayleigh distribution (eg Massel, 1996; Goda, 2000).

$$\frac{H_{p\%}}{H_{rms}} = \sqrt{-\ln\left(\frac{p}{100}\right)} \qquad (4)$$

$$\frac{H_{1/Q}}{H_{rms}} = Q\,erfc\left(\sqrt{\ln Q}\right) + \sqrt{\ln Q}, \qquad erfc(x) = \int_x^{+\infty}\exp\left(-t^2\right)dt \qquad (5)$$

An important issue is the estimation of the maximum value of the wave height for the case of sea-states of finite duration. This maximum wave height cannot be determined in a deterministic manner. One can, however, derive a probability density function for the (statistical) ratio H_{max}/H_s (eg Massel, 1996; Goda, 2000). Two important representative values, namely the mode and the mean values, can be expressed analytically (see Equations 6 and 7).

Mode of distribution

The most probable value of the ratio H_{max}/H_s for a record consisting of N waves (see Equation 6).

$$\left[\frac{H_{max}}{H_s}\right]_{mode} \approx \sqrt{\frac{\ln N}{2}} \qquad (6)$$

Mean value of the distribution

The mean value of the ratio H_{max}/H_s for a record consisting of N waves (see Equation 7). The mean value is greater than the mode, because of the skewed shape of the distribution:

$$\left[\frac{H_{max}}{H_s}\right]_{mean} \approx \left(\sqrt{\frac{\ln N}{2}} + \frac{\gamma}{2\sqrt{2\ln N}}\right) \qquad (7)$$

where γ = Euler constant ≈ 0.5772.

An overview of the used parameters is given below:

parameter	short description	unit
H_{max}	Maximum wave height	[m]
H_s	Significant wave height	[m]
N	Number of wave height in a wave record of finite length	[-]
γ	Euler constant	[-]

Input and output parameters

Input:	Output:
H_s, N	H_{max}

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Default Value
H_s	Significant wave height	0 – 10 (i)	>0	See table 1
H_{max}	Mean of 1/10 of the highest wave heights	0 – 10 (i)	>0	See table 1
N	Number of wave height in a wave record of finite length	0 - 20000	>0	1000
γ	Euler constant	0.5772	0.5772	0.5772

References

Goda, Y (2000). Random seas and design of maritime structures. *Advanced Series on Ocean Engg, vol 15*, World Scientific, Singapore, 444 pp

S.R. Massel, *Ocean Surface Waves; their Physics and Prediction*. World Scientific Publ., Singapore - New Jersey - London - Hong Kong, 1996, 491pp

3.9 Breaking caused by water depth (Y_6)

Wave breaking occurs when the relative wave height (H/h) becomes too large. Both the depth and the steepness therefore limit the maximum wave height. In shallow water, depth-induced breaking is usually the dominant factor, while the limit of steepness should be considered mainly for the generation of waves. The breaking criterion attributable to water depth is normally given by a useful non-dimensional parameter called the breaker index γ_{br} , defined as the maximum wave height to depth ratio H/h (see Equation 1) where the subscript b stands for the value at the breaking point.

$$H/h \leq \gamma_{br} = [H/h]_{\max} = H_b/h_b \tag{1}$$

The breaker wave height is calculated using

$$H_b = \gamma_{br} h_b \tag{2}$$

For stable and progressive waves over a flat bottom γ_{br} has a theoretical maximal value of 0.78 (McCowan, 1894). Note, however, that γ_{br} is not constant, but ranges roughly between 0.5 and 1.5 depending on the bottom slope and the wave period of the incident waves. Numerous criteria to predict the value of γ_{br} have been proposed. A comprehensive review and comparison of most of them can be found in Rattanapitikon and Shibayama (2000). For regular waves normally incident on a uniform slope m (i.e. m = tan(α)), two criteria (see Equations 3 and 4) may be recommended for practical use:

$$\gamma_{br} = \frac{H_b}{h_b} = 0.17\frac{L_0}{h_b}\left\{1 - \exp\left[-1.5\pi\frac{h_b}{L_0}\left(1 + 15m^{4/3}\right)\right]\right\} \tag{3}$$

$$\gamma_{br} = \frac{H_b}{h_b} = \frac{b(m)}{1 + a(m)\frac{h_b}{L_0}} = b(m) - a(m)\frac{H_b}{L_0} \tag{4}$$

Where

$$a(m) = 6.96[1 - \exp(-19m)] \tag{5}$$

$$b(m) = 1.56[1 + \exp(-19.5m)]^{-1} \tag{6}$$

Other criteria and a comparison of them on a large set of data can be found in Rattanapitikon and Shibayama (2000) and in Rattanapitikon et al (2003), who also proposed a new criterion giving the best fit to the experimental points of the validation database (see Equation 7):

$$\frac{H_b}{L_b} = \left[-1.40m^2 + 0.57m + 0.23\right]\left(\frac{H_0}{L_0}\right)^{0.35} \tag{7}$$

Where: L_b = wavelength computed at the breaking point (depth h_b) by the linear theory (see Rule A5.1);

$$L_b = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi h_b}{L_b}\right) \qquad (8)$$

An overview of the used parameters is given below:

parameter	short description	unit
H_b	Wave height at the breaking point	[m]
h_b	Depth at the breaking point	[m]
L_0	Wave length computed at the breaking point (h_b) by the linear theory	[-]
γ_{br}	Breaker Index	[-]
a(m)	Coefficient	[-]
b(m)	Coefficient	[-]
m	Bottom slope	[-]
α	Bottom gradient	[-]
L_b	Wave length at the breaker line	[m]
T	Wave period on deep water	[s]

Input and output parameters

Input:	Output:
h_b, L_0, m,T	H_b (according to Weggel, according to Goda and according to Rattanapitikon) γ_{br}

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Columns
H_b	Wave height at the breaking point	0-10 (i)	>0	
h_b	Depth at the breaking point	0-10 (i)	>0	
L_0	Wave length computed at the breaking point (h_b) by the linear theory	0 – 200 (i)	>0	
γ_{br}	Breaker Index	0 – 2 (i)	>0	
a(m)	Coefficient		>0	
b(m)	Coefficient		>0	

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Columns
m	Bottom slope	0-0.01 (i)	0<m<1	

References

McCowan (1894). “On the highest wave of permanent type”. Philosophical Magazine, vol 38, Ser 5, pp 351–358

Rattanapitikon, W and Shibayama, T (2000). “Verification and modification of breaker height formulas”. Coastal Engg J, vol 42, no 4, pp 389–406

Rattanapitikon, W, Vivattanasirisak, T and Shibayama, T (2003). “A proposal of new breaker height formula”. Coastal Engg J, vol 45, no 1, pp 29–48

3.10 Wave height estimation in the surf zone (Y_7)

Equations

Goda (2000) developed formulae to estimate the significant wave height (see Equation 1) and the maximum wave height (see Equation 2) in the surf zone.

$$H_{1/3} = K_s H_0^{'} \qquad h/L_0 > 0.2 \tag{1a}$$

$$H_{1/3} = \min\{(\beta_0 H_0^{'} + \beta_1 h)(\beta_{\max} H_0^{'})(K_s H_0^{'})\} \qquad h/L_0 < 0.2 \tag{1b}$$

$$H_{\max} = H_{1/250} = 1.8 K_s H_0^{'} \qquad h/L_0 > 0.2 \tag{2a}$$

$$H_{\max} = H_{1/250} = \min\{(\beta_0^* H_0^{'} + \beta_1^* h)(\beta_{\max}^* H_0^{'})(1.8 K_s H_0^{'})\} \qquad h/L_0 < 0.2 \tag{2b}$$

$$H_0^{'} = K_d K_r H_{s0} \tag{3}$$

The coefficients β_0, β_1, … are given in Table 1 (note that min{a,b,c} and max{a,b,c} stand for the minimum and maximum values among a, b and c). $H_0^{'}$ is the equivalent deep-water significant wave height. This equivalent wave height is a hypothetical wave height obtained from the actual significant deep-water wave height $H_{so,}$ corrected for the effects of refraction and/or diffraction from offshore to the shoreline.
It is obtained as $H_0^{'} = K_d K_r, H_{so}$ where K_d and K_r are the diffraction and refraction coefficients respectively. The above shoaling coefficient K_s is obtained using linear wave theory. m is the beach gradient (i.e. $m = \tan(\alpha)$). Goda (2000) advises that this numerical formula may overestimate wave heights by several per cent. In particular, for waves of steepness greater than 0.04, the formulae overestimate significant wave heights by at least 10 per cent around the water depth at which the value of $H_{1/3} = \beta_0 H_0 + \beta_1 h$ becomes equal to the value of $H_{1/3} = \beta_{max} H_0$. A similar difference also appears for the case of H_{max}. Waves of large steepness may have a discontinuity in the estimated height of H_{max} at the boundary $h/L_o = 0.2$. Caution should be taken when applying Goda's formulae with regard to such differences and discontinuities.

Coefficients for $H_{1/3}$	Coefficients for H_{max}
$\beta_0 = 0.028(H_0^{'}/L_0)^{-0.38} \exp(20m^{1.5})$	$\beta_0^* = 0.052(H_0^{'}/L_0)^{-0.38} \exp(20m^{1.5})$
$\beta_1 = 0.52 \exp(4.2m)$	$\beta_0^* = 0.63 \exp(3.8m)$
$\beta_{\max} = \max\{0.92, 0.32(H_0^{'}/L_0)^{-0.29} \exp(2.4m)\}$	$\beta_{\max}^* = \max\{1.65, 0.53(H_0^{'}/L_0)^{-0.29} \exp(2.4m)\}$

Table 1: ***Coefficients for*** **$H_{1/3}$ *and* H_{max}**

An overview of the used parameters is given below:

parameter	short description	unit
H'_0	Deep-water significant wave height	[m]
$H_{1/3}$	Mean of one third of the highest waves, or significant wave height	[m]
H_{s0}	Significant wave height at deep water	[m]
H_{max}	Maximum wave height in the surf zone	
L_0	Deep-water wave length	[m]
m	Beach gradient	[-]
β_{max}	Coefficient	[-]
B_1	Coefficient	[-]
β_0	Coefficient	[-]
β^*_{max}	Coefficient	[-]
β^*_1	Coefficient	[-]
β^*_0	Coefficient	[-]
K_d	Diffraction coefficient	[-]
K_s	Shoaling coefficient	[-]
K_r	Refraction coefficient	[-]

Input and output parameters

Input:	**Output:**
H_{s0}, L_0,K_s, m,K_d,	H_{max}, $H_{1/3}$

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H'_0	**Deep-water significant wave height**	0 – 10 (i)	>0
$H_{1/3}$	**Mean of one third of the highest waves, or significant wave height**	0 – 10 (i)	>0
H_{s0}	**Significant wave height at deep water**	0 – 10 (i)	>0
H_{max}	**Maximum wave height in the surf zone**	0 – 10 (i)	>0
L_0	**Deep-water wave length**	0 – 200 (i)	>0
m	**Beach gradient**	0 – 1	0<m<1
β_{max}	**Coefficient**		>0
β_1	**Coefficient**		>0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
β_0	**Coefficient**		>0
β^*_{max}	**Coefficient**		>0
β^*_1	**Coefficient**		>0
β^*_0	**Coefficient**		>0
K_d	**Diffraction coefficient**		
K_s	**Shoaling coefficient**		
K_r			

References

Goda, Y (2000). Random seas and design of maritime structures. Advanced Series on Ocean Engg, vol 15, World Scientific, Singapore, 444 pp

3.11 Wind set-up (Y_9)

A closed water domain (eg lake, lagoon) of length F (m) with a constant water depth h (m) and a constant wind speed U_{10} (m/s) blowing over the water domain, the resulting maximum wind set-up η_w (m) at the downwind coast or shoreline is given by Equation 1. In the absence of calibration data, simplified results such as those following from Equation 1 can only provide a guide to the likely wind set-up, because of uncertainties about the value of C_D and the choice of representative values of *h* and *F*.

$$\eta_w = \frac{1}{2}\frac{\rho_{air}}{\rho_w} C_D \frac{U_{10}^2}{gh} F \tag{1}$$

where U_{10} = wind speed at an elevation of 10 m above MSL (m/s), ρ_{air} = mass density of air (1.21 kg/m3) and C_D = air/water drag coefficient with typical values of $0.8.10^{-3}$ to $3.0 \cdot 10^{-3}$ (-), this value increases with wind speed (eg Abraham et al, 1979; Wu, 1980).
If possible, site-specific measurements of surge, from which wind set-up can be estimated, should be made on a few windy days. This would enable site-specific calibration of the equations for use in subsequent predictions.

An overview of the used parameters is given below:

parameter	short description	unit
η_w	Maximum wind set-up	[m]
ρ_{air}	Mass density of air	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]
C_D	C_D = air/water drag coefficient with typical values of $0.8.10^{-3}$ to $3.0.10^{-3}$ (-)	[-]
g	Gravitational acceleration	m/s^2
U_{10}	Wind velocity at 10m above MSL	[m/s]
h	Water depth	[m]
F	Length of closed domain	[m]

9.2 input and output parameters

Input:	Output:
ρ_{air},ρ_w, C_D,U_{10} ,h, F	η_w

9.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Default Value
η_w	Maximum wind set-up	0-50 (i)	>0	See table 1

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	Default Value
ρ_{air}	Mass density of air	1,1-1.3	>0	1.21
ρ_w	Mass density of water	1000-1040	>0	1025
C_D	C_D = air/water drag coefficient with typical values of $0.8.10^{-3}$ to $3.0.10^{-3}$ (-)	$0.5.10^{-3}$-4.10^{-3}	>0	$1.5.10^{-3}$
U_{10}	Wind velocity at 10m above MSL	0-50	>0	10
h	Water depth	0-2000(i)	>0	100
F	Length of closed domain	[m]	1000000	100000

9.4 References

Abraham, G, Karelse, M and Van Os, A G (1979). “On the magnitude of interfacial shear of subcritical stratified flows in relation to interfacial stability”. *J Hydraulic Research*, vol 17, no 4, pp 273–284

Wu, J (1980). “Wind stress coefficients over the sea surface near neutral conditions. A revisit”. *J Phys Oceanogr*, vol 10, pp 727–740

3.12 Wave set-up (Y_10)

Wave set-up is localised near to the shoreline. It is mainly caused by energy dissipation caused by depth-induced breaking of the incoming waves (see Figure 1).

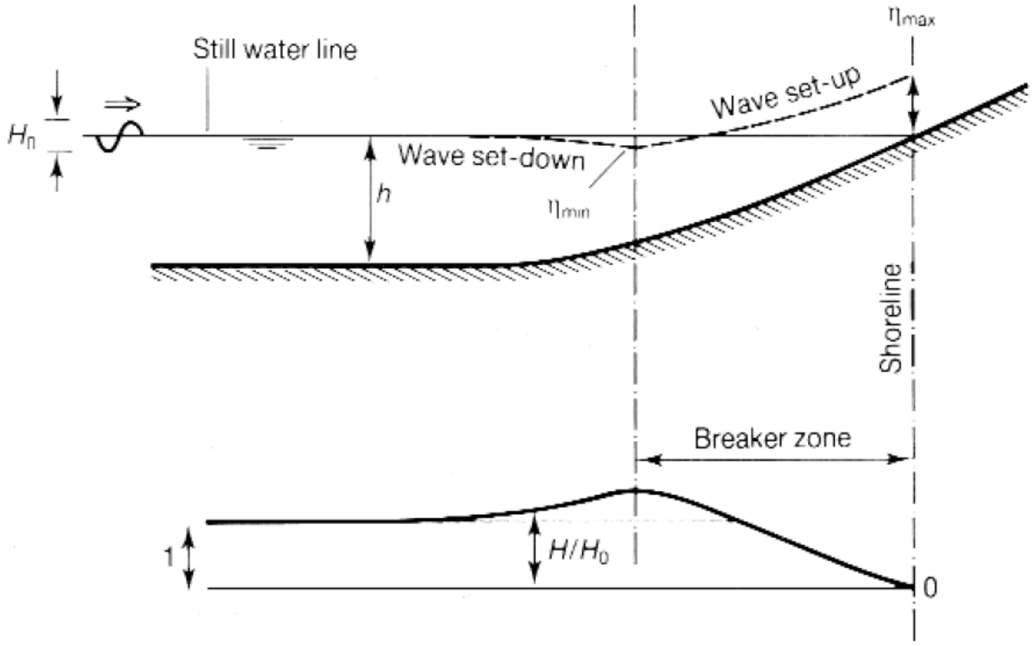


Figure 1 Wave set-up

Using linear wave theory for normally incident regular waves, Battjes (1974) derived a first estimate of wave set-up at the shoreline. Equation 4.15 gives the relationship between the wave set-up, η_{max} , and the wave conditions at the breaker line:

$$\eta_{max} = 0.3\gamma_{br}H_b \tag{1}$$

where γ_{br} = breaker index or maximum wave height to water depth ratio H/h (-) and H_b = wave height at the breaker line for regular waves (m). The value of H_b can be found by applying a wave model to the local bathymetry using deep-water waves as a boundary condition.

For the case of a planar beach, Bowen et al (1968) used the shallow-water linear wave theory for the radiation stress and made use of the approximate relationship H = γ_{br} (h+η) in the surf zone to derive Equation 4 for the set-up:

$$\eta - \eta_b = \frac{1}{K}(h_b - h) \tag{2}$$

where the subscript b again denotes values at the breaking point and K = 1+8/(3γ_{br}^2)

On the basis of field measurements and numerical simulations, some relationships have been established for irregular wave conditions. For example, Hanslow and Nielsen (1992) fitted the relationships given in Equations 3 and 4 to their measurements for the shoreline set-up:

$$\eta = 0.38 H_{orms} \tag{3}$$

$$\eta = 0.0488\sqrt{H_{orms} L_o} \tag{4}$$

where H_{orms} = incident (deep-water) root-mean-square wave height (m) and L_o = deep-water wavelength calculated from the wave period T as

$$L_0 = \frac{gT^2}{2\pi} \tag{5}$$

Equation 4 results in a slightly better fit of measurements than Equation 3, although a significant scatter of experimental points is still present.

An overview of the used parameters is given below:

parameter	short description	unit
η_{max}	Maximum wave set-up	[m]
η_b	Wave set-down at breaker line	[m]
γ_{br}	Breaker index	[-]
H_b	Wave height at the breaker line for regular waves	[-]
K	Calculated from breaker index	[-]
h_b	Water depth at the breaking point	[m]
h	Water depth	[m]
H_{orms}	Incident root mean square wave height	[m]
L_o	Incident wave length	[m]
T	Deep water wave height	[s]

Input and output parameters

Input:	Output:
$\gamma_{br}, H_b, h_b, h, H_{0rms}, T$	η_{max}, η-η_b, η

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
η_{max}	**Maximum wave set-up**	**0-50 (i)**	>0
η_b	**Wave set-down at breaker line**	**0-50 (i)**	>0
γ_{br}	**Breaker index**	**0-1 (f)**	>0
H_b	**Wave height at the breaker line for regular waves**	**0-10**	>0
K	**Calculated from breaker index**	**1 - 1+8/3**	1<K<1+8/3
h_b	**Water depth at the breaking point**	**>0-10000**	>0
h	**Water depth**	**>0-10000**	>0
H_{orms}	**Incident root mean square wave height**	**0-20**	>0
L_o	**Incident wave length**	**0-300**	>0
T	**Deep water wave period**	**0-15**	>0

References

Battjes, J A (1974). Computation of set-up, longshore currents, run-up and overtopping due to wind generated waves. Report 74-2, *Comm on Hydraulics*, Dept of Civil Engrs, Univ of Technology, Delft

Bowen, A J D, Inman, D L and Simons, V P (1968). "*Wave 'set-down' and 'set-up'* ". J Geophys Res, vol 73, pp 2569–2577

Hanslow, D J and Nielsen, P (1992). "*Wave setup on beaches and in river entrances*". In: Proc 23rd int conf coastal engg, Venice. ASCE, New York, pp 240–252

3.13 Lacey's regime equation (Y_11)

The need for design guidelines for stable irrigation canals in the Indian subcontinent led to the formulation of regime theory. Subsequently, the derived relationships were also used for other rivers. However, the empirical equations are strongly related to local circumstances and are not generally applicable to all situations. The various relationships enable a prediction of the width, water depth, flow velocity, hydraulic radius, hydraulic perimeter and bed gradient from overall hydraulic parameters.

Regime equations have been derived for many areas in the world, among others by Lacey (1930), Simons and Albertson (1960) and Henderson (1966). Regime theory is the classic procedure for the design of stable channels when sediment transport occurs. Its physical basis and historic development have been described in some detail in several publications on fluvial hydraulics (e.g. Chang, 1988; Yalin, 1992). Many authors have studied the topic and proposed equations – see Lacey (1930), Mahmood and Shen (1971), Simons and Albertson (1960), Chitale (1966) and Mahmood (1974).

The regime equations are supported by regime theories and, in this respect, the following definition of a river or flow regime seems to apply. A river regime is the range of river discharges, corresponding water levels and their respective (yearly or seasonally) averaged values and characteristic fluctuations around these values. Regime theories may be applied even if very little information of a river is available. It is recommended that the selected regime equations be calibrated using reliable local data. Most of the regime equations relate cross-sectional and longitudinal parameters to the discharge.

Many empirical formulae provide the width of the river B according to various morphological flows, which may be defined as equivalent permanent flows that would create the actual river morphology. These flows have a return period lower or equal to two years and are called morphologically dominant formative flow regimes. However, whatever the flow taken into account, the wavelength of the river bends λ (see also Figure 1) varies schematically according to the square root of the discharge (Dury, 1955, 1976; Carlston, 1965; Ackers and Charlton, 1970; Schumm, 1963, 1968, 1977). The bankfull discharge proves to be the best approach to characterise geometry and evolution of meandering rivers.

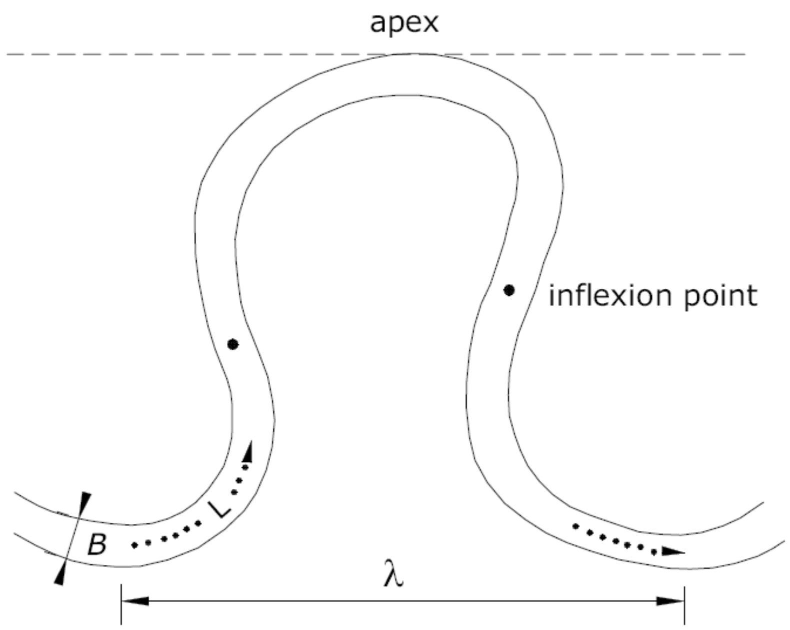


Figure 1 *Typical shape parameters of a meandering river (after Bravard and Petit, 2000)*

Equations

Lacey's regime equations (see Equations 1 to 6) are applied most widely to alluvial river channels and man-made canals with a low sediment transport, i.e. for sediment concentration of 100–2000 mg/l and grain size of bed material of 0.1–0.5 mm.

$$P = 4.87Q^{1/2} \qquad \text{(Equation 1)}$$

$$A = 2.38Q^{5/6} \,/\, f^{1/3} \qquad \text{(Equation 2)}$$

$$R = 0.47Q^{1/3} \,/\, f^{1/3} \qquad \text{(Equation 3)}$$

$$U = 0.64R^{1/2} f^{1/2} \qquad \text{(Equation 4)}$$

$$i_b = 0.00030 f^{5/3} \,/\, Q^{1/6} \qquad \text{(Equation 5)}$$

$$f = 1.59 D_{50}{}^{1/2} \qquad \text{(Equation 6)}$$

Suggested values for the Lacey's silt factor, *f* ,are given in Table 1.

Sediment	Silt	Sand	Gravel	Stones
Lacey's silt factor, *f*	0.3-1.0	1.3-1.5	2.4-4.5	6.4-40

Table 1: Lacey's silt factor, f

Lacey's equations do not distinguish between bed and bank material. Simons and Albertson (1960) extended the equations to include the effect of the soil properties of the banks. Regime equations have also been developed for rivers with gravel beds. Hey and Heritage (1988) give a summary. Further details on these equations and other regime theories are given in Henderson (1966).

parameter	short description	unit
P	**wetted perimeter**	[m]
A	**cross-sectional area**	[m^2]
R	**hydraulic radius**	[m]
U	**average flow velocity**	**[m/s]**
I_b	**average gradient of bed slope**	[-]
Q	**discharge**	**[m^3/s]**
f	**Lacey's silt factor see table 1**	[-]
D50	**median diameter of bed material**	**[mm]**

Input and output parameters

Input:	Output:
Q,*D_{50}*	***P,A,R,U,i,f***

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
P	wetted perimeter	0-10000 (i)	>0
A	cross-sectional area	0-10000 (i)	>0
R	hydraulic radius	0-10000 (f)	>0
U	average flow velocity	0-10	>0
I_b	average gradient of bed slope	1.10^{-6}	1.10^{-2}
Q	discharge	0-100000	>0
f	Lacey's silt factor	0-100	>0
D_{50}	median diameter of bed material	0-20	>0

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3.14 Current Velocity [Y12]

Manning-Strickler formulation

The flow velocity, U (m/s), can be calculated using the Manning-Strickler formula as given by Equation 1

$$U = \frac{R^{2/3} i^{1/2}}{n} \qquad \text{(Equation 1)}$$

In which

parameter	short description	unit
U	flow velocity,	[m/s]
R	hydraulic radius, the ratio of the water area and the wetted perimeter	[m]
i	slope of the energy line, or water surface slope	[-]
n	Manning's roughness coefficient.	[s/m$^{1/3}$]

Manning's roughness coefficient, n, takes into account that the roughness of the banks and the bottom results in head losses by friction. Consequently, head losses become more significant as roughness increases. Roughness depends mainly on the nature of the materials on the river bed and the vegetation. Using the Cowan (1956) procedure, Manning's roughness coefficient, n, can be computed using Equation 2:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad \text{(Equation 2)}$$

where:
n_0 = factor that depends on the constitutive material of the channel
n_1 = factor that depends on the degree of surface irregularity
n_2 = factor that depends on the variations of the cross-section form
n_3 = depends on the effects of obstruction (bridge etc)
n_4 = factor that depends on the vegetation which modifies the flow conditions
m_5 = coefficient that indicates the sinuosity degree of the channel.

Channel Conditions		Components of n	
Material involved	Earth Rock cut Fine gravel Coarse gravel	n_0	0.02 0.025 0.024 0.028
Degree of irregularity	Smooth Minor Moderate Severe	n_1	0.000 0.005 0.010 0.020

Variations of channel cross-section	Gradual Alternating occasionally Alternating frequently	n_2	0.000 0.005 0.010-0.015
Relative effect of obstructions	Negligible Minor Appreciable Severe	n_3	0.000 0.010-0.015 0.020-0.030 0.040-0.060
Vegetation	Low Medium High Very high	n_4	0.005-0.010 0.010-0.025 0.025-0.050 0.050-0.100
Degree of Meandering	Minor Appreciable Severe	n_5	1.00 1.15 1.300

Table 1: Values of Manning's coefficient proposed by the US Soil Conservation Service (Chow, 1959)

n_0 can either be determined with Strickler's formula: $n_0 = 0.048\ D_{50}^{1/6}$ where D_{50} = median particle diameter of the bed sediment (m); or with $n_0 = 0.038\ D_{90}^{1/6}$ (Simons and Senturk, 1977), with D_{90} = grain size not exceeded by 90 per cent (by mass) of the bed sediment. The relationship between n_0 and D_{90} is approximately constant for a range of depths given by $7< D_{90}/h <150$

Chézy

The flow velocity U (m/s) can also be calculated from the well-known Chézy equation given by Equation 3:

$$U = C\sqrt{Ri} \qquad \text{(Equation 3)}$$

In which

parameter	short description	unit
U	flow velocity,	[m/s]
R	hydraulic radius, the ratio of the water area and the wetted perimeter	[m]
i	slope of the energy line, or water surface slope	[-]
C	bed friction Chézy coefficient	[$m^{1/2}$/s]

The Chézy coefficient, C, is a measure of the riverbed and riverbank roughness and it has been defined by Bazin, as expressed by Equation 4.

$$C = \frac{87}{1 + \frac{\gamma}{\sqrt{R}}} \qquad \text{(Equation 4)}$$

In which:

parameter	short description	unit
C	bed friction Chézy coefficient	[$m^{1/2}$/s]
R	hydraulic radius, the ratio of the water area and the wetted perimeter	[m]
γ	parameter representative of the bed roughness varies from 0.06 for a smooth bed to 1.75 for a grassed ground bed and cobbles	[$m^{1/2}$]

It can also be determined by Equation 5 with the roughness length scale of Nikuradse, k_s (m):

$$C = 18\log(12h/k_s) \qquad \text{(Equation 5)}$$

The hydraulic roughness (k_s) is discussed below in Box 1.

It should be noted that for small water depths, Equation 6 cannot be used. For such cases, Christensen (1972) provides a practical alternative approach. By changing Prandtl's mixing length (Prandtl, 1925), Christensen (1972) determined an associated alternative formula for C given by Equation 4.133.

$$C = 18\log(1+12h/k_s) \qquad \text{(Equation 6)}$$

For $h/k_s > 2$, this formula is close to the common form given in Equation 5.

An overview of the used parameters is given below:

parameter	short description	unit
C	bed friction Chézy coefficient	[$m^{1/2}$/s]
h	waterdepth	[m]
ks	hydraulic roughness see box 1	[m]

.

Box 1

This box deals with methods based on bedform characteristics, particularly developed by Van Rijn (1989). The hydraulic roughness consists of two parts:

- grain roughness, k_{sg} (m)
- bedform roughness, $k_{s\Delta}$ (m).

The grain roughness, k_{sg}, can be approximated by Equation 7 (Van Rijn, 1982).

$$k_{sg} = 3D_{90} \qquad \text{(Equation 7)}$$

For engineering purposes, the scatter of k_{sg} in the case of *graded sediment* can be described by k_{sg} /D_{90} = 1 to 3. Somewhat arbitrarily assuming that D_{90}/D_{50} = 2, which implies k_{sg} /D_{50} = 4.

For uniform sediment the range of grain roughness is given by k_{sg} /D_{50} = 1 to 2. Despite scatter, on average the best results seem to be obtained using
$k_s = D_{90} \approx 2 D_{50}$ for fine sediments and
$k_s = 2 D_{90} \approx 4 D_{50}$ for coarse material, assuming no bedform roughness.

The bedform roughness, $k_{s\Delta}$, should be calculated using the roughness predictors given in Van Rijn, 1989. The empirical relation (see Equation 8) is based on the dimensions of the dune bedforms that are present in the river bed.

$$k_{s\Delta} = 1.1 D_b (1 - \exp(-25 D_b / L_b)) \qquad \text{(Equation 8)}$$

where :
D_b = average bedform height (m) and
L_b = average bedform length (m).

Values for D_b and L_b depend on the flow regime and should be determined from echo-soundings of the river bed. The overall hydraulic roughness is given by Equation 9.

$$k_s = k_{sg} + k_{s\Delta} \qquad \text{(Equation 9)}$$

In general, the contribution of k_{sg} to the hydraulic roughness is small compared with the contribution of $k_{s\Delta}$. Substituting k_s according to the above formulae in the equation for the Chézy coefficient should generally result in values in the range C = 25 to 60 m1/2/s. It is noted that for a silty bed (eg in estuaries) C may be up to 80–90. Other methods of determining hydraulic roughness exist; see for example
EDF et al (1992).

Determination of k_s and the resulting values of C are discussed below, where it appears again that practically C is a function of water depth, h, and sediment grain size, D. By using Equations 3 and 5 the depth-averaged velocity, U, can be found for given (average) water depth, h, water surface gradient, i, and hydraulic roughness, k_s statistical variations of these parameters may also be considered.

Clearly there is a close relationship between the Manning-Strickler formulation and Chézy equation through an appropriate description of C in terms of R and n in Equation 3. Historically, more complicated cross-sections have been analysed using the Manning-Strickler method and this is discussed below under the title of composite cross-sections.

Composite cross-section

A cross-section may need to be analysed as a composite section either if the geometry of the section is irregular such as a channel set within a floodplain or if the character of the hydraulic roughness varies significantly across the section. The starting point for the analysis is the definition of discharge, Q, in terms of the velocity across the section given in Equation 10.

$$Q = U_1 A_1 = U_2 A_2 = U_i A_i \qquad \text{(Equation 10)}$$

where: Q = volumetric flow rate (m3/s), U = average flow velocity (m/s), A = cross-sectional flow area (m²) and the subscripts on U and A designate different river section locations.

Two approaches are possible: traditional hand calculation procedures and computerbased methods.

The traditional approach for calculation by hand is to divide the section into several components, usually with plane boundaries, and to assume that the shear stresses in the planes between adjacent parts are zero. By making one or more assumptions, the effective mean value of the hydraulic resistance can be calculated as described below.

In more modern computational procedures the transverse velocity distribution is estimated from the shape (and possibly planform) of the section and the distribution of hydraulic roughness. Integration of this velocity distribution across the section then provides the total discharge. The computational procedures can also provide the velocities close to the boundary of the section for use in sizing bank protection materials.

Traditional calculation methods

For a composite cross-section the values of the hydraulic roughness for the various zones usually differ. Early publications on the approach used the Manning-Strickler method for irregular river cross-sections. In such cases, which are very common, the effects of banks and channels on the current distribution have to be considered. An irregular cross-section should be schematised using one of the following approaches.

1. A general method is to divide the cross-section into vertical slices parallel to the river axis, each with a more or less constant water depth, as shown in Figure 4.58. For the determination of the equivalent roughness, the water area is divided into N parts with the wetted perimeters P_1, P_2, …, P_N (m) and the Manning coefficients of roughness n_1, n_2, .., n_N (s/m1/3) are known.

   By assuming that each part of the area has the same mean velocity, the equivalent coefficient of roughness may be obtained by Equation 11 (Einstein, 1934; Yassin, 1954; Horton, 1933).

   $$n = (P_1 n_1^{1.5} + P_2 n_2^{1.5} + ... + P_N n_N^{1.5})^{3/2} / P^{2/3} \qquad \text{(Equation 11)}$$

   By assuming that the total force resisting the flow is equal to the sum of the forces resisting the flow developed in the subdivided areas (Pavlovski, 1931; Mülhofer, 1933; Einstein and Banks, 1950), the equivalent roughness coefficient is given by Equation 12.

$$n = (P_1 n_1{}^2 + P_2 n_2{}^2 + ... + P_N n_N{}^2)^{1/2} / P^{1/2} \quad \text{(Equation 11)}$$

Lotter (1933) assumed that the total discharge of the flow is equal to the sum of discharges of the subdivided areas. Thus the equivalent roughness coefficient can be computed from Equation 12.

$$n = PR^{5/3}(P_1 R_1{}^{5/3} / n_1 + P_2 R_2{}^{5/3} / n_2 + ... + P_N R_N{}^{5/3} / n_N) \quad \text{(Equation 12)}$$

2 Where a main channel and a floodplain can be clearly distinguished, the cross-section should be divided into two separate parts (see Figure 1). Then, using the Chézy formulation, the conditions of equal water surface gradient i and continuity yield to Equations 13 and 14.

$$i = U_1{}^2 / (R_1 C_1{}^2) = U_2{}^2 / (R_2 C_2{}^2) = U^2 / (RC^2) \quad \text{(Equation 13)}$$

$$UA = U_1 A_1 + U_2 A_2 \quad \text{(Equation 14)}$$

This results in Equations 4.142 and 4.143.

$$UA = UA_1(\sqrt{R_1 / R}\, C_1 / C) + UA_2(\sqrt{R_2 / R}\, C_2 / C) \quad \text{(Equation 15)}$$

$$\sqrt{R} = A_1(\sqrt{R_1}\, C_1) + A_2(\sqrt{R_2}\, C_2)(AC) \quad \text{(Equation 16)}$$

The overall C-value can be computed from Equation 17.

$$C = (b_1 C_1 + b_2 C_2) / b \quad \text{(Equation 17)}$$

where: b = b1 + b2 (see Figure 1).

3 If the area of the cross-sections (A_1 and A_2) cannot be estimated accurately, like in Figure 1, then the application of the hypothesis of Einstein is recommended. Einstein assumed $U_1 = U_2 = U$, resulting in Equation 18.

$$1/(R_1 C_1{}^2) = 1/(R_2 C_2{}^2) = 1/(RC^2) \quad \text{(Equation 18)}$$

Equation 4.145 results in the relationships given by Equations 19 and 20.

$$(R_1 C_1{}^2) = (R_2 C_2{}^2) = RC^2 = Q^2 / (A^2 i) \quad \text{(Equation 19)}$$

$$R_2 / R_1 = C_1{}^2 / C_2{}^2 \quad \text{(Equation 20)}$$

Especially for this schematisation (3) in Figure 1, Strickler provides a practical alternative (see Equation 21) to the Chézy friction coefficient, C ($m^{1/2}/s$), given by Equation 5.

$$C = 25(R / k_s)^{1/6} \quad \text{(Equation 21)}$$

Equation 21 gives a reasonable approximation for the original value of C in the range of C = 40 to 70 $m^{1/2}/s$ and transfers Equation 20 into Equation 22.

$$R_2 / R_1 = (k_{s2} / k_{s1})^{1/4} \qquad \text{(Equation 21)}$$

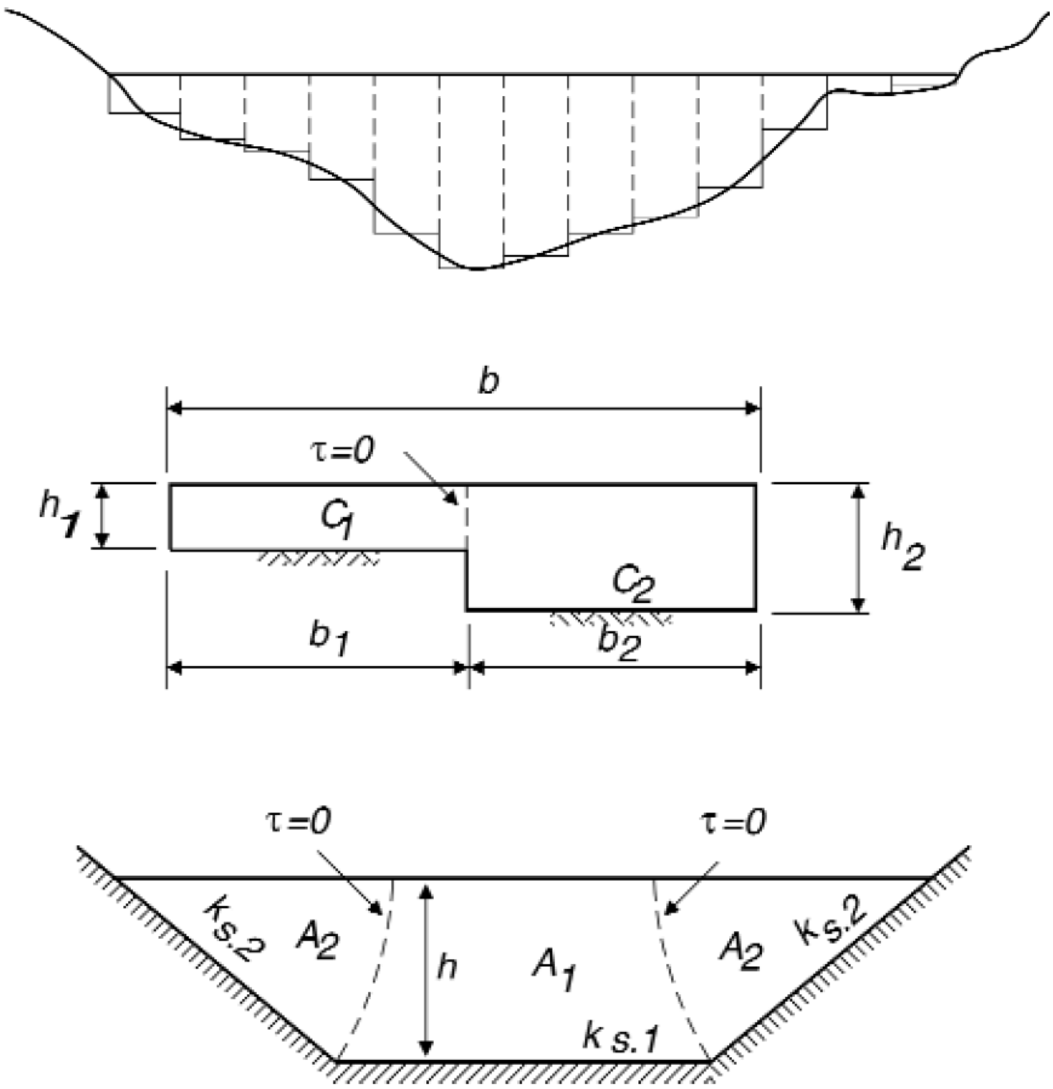

Figure 1: schematisation of composite crosssections

Many other hand traditional calculation methods exist, some separating the left floodplain and the right floodplain from the main channel (eg James and Wark, 1992)

Computational methods

All one-dimensional mathematical models of open channel hydraulics include computational methods for assessing the discharge capacity (conveyance) of cross-sections. They have increased in sophistication over the decades of increasing model development and use. Many models use the traditional calculation methods described above or some variant to represent the conveyance of the sections.

However, some models attempt to be more physics-based for the representation of conveyance. More recently, methods have been developed and are based upon differential equation models of the cross-stream variation of velocity. Refer to Vreugdenhil and Wijbenga (1982) or James and Wark (1992) for further information.

McGahey and Samuels (2003) provide a brief summary of approaches and describe the method adopted in the UK Environment Agency's Conveyance Estimation System (CES). The CES method is designed for both straight and meandering channels with associated floodplains. The resulting model can reproduce both the variation of discharge with water level and the transverse velocity distribution within the same model structure.

When interpreting results for the design of bank protection it is important to take account of the particular method and representation of velocity that underlies the calculation model, which should be available in the documentation of the software. It should also be noted that the value of resistance coefficient for the same circumstances may vary between models depending on the details of the calculation method. Section 4.3.5 below discusses modeling in more detail.

U	flow velocity,	[m/s]
R	hydraulic radius, the ratio of the water area and the wetted perimeter	[m]
i	slope of the energy line, or water surface slope	[-]
n	Manning's roughness coefficient.	[$s/m^{1/3}$]

Input and output parameters

Input:	Output:	Equation
R, i, n	U	Equation 1
R, i, C	U	Equation 3
R, γ	C	Equation 4
h, k_s	C	Equation 5
h, k_s	C	Equation 6

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	default values
R	hydraulic radius, the ratio of the water area and the wetted perimeter	0 –100	>0	10
i	slope of the energy line, or water surface slope	0 – 1	0<i<1	0.001
n	Manning's roughness coefficient.	0 -- 0.07	>0	0.012
C	bed friction Chézy coefficient	10 – 100	>0	40
h	waterdepth	0 –100	>0	10
ks	hydraulic roughness	0 – 2	>0	0.02
γ	parameter representative of the bed roughness	0 – 2	>0	1

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Vessel's submerged cross-section, Am (Y13_1)

The vessel's submerged cross-section, A_m (m^2), is evaluated by Equation 1.

$$A_m = C_m B_s T_s \tag{1}$$

where: C_m = midship coefficient related to the cross-section of the ship (-); B_s = beam width of the ship (m); T_s = draught of the ship (m). Appropriate values of C_m are:

- C_m = 0.9 to 1.0 for push units and inland vessels
- C_m = 0.9 to 0.7 for service vessels, tow boats and for marine vessels.

parameter	short description	unit
A_m	Vessel's submerged cross-section	[m^2]
C_m	Midship coefficient	[-]
B_s	Beam width of the ship	[m]
T_s	Draught of the ship	[m]

13a.2 input and output parameters

Input:	Output:
C_m, B_s, T_s	A_m

13a.3 boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
A_m	Vessel's submerged cross-section	0-100 (i)	>0
C_m	Midship coefficient	0.7-1 (i)	0-1
B_s	Beam width of the ship	30 (i)	>0
T_s	Draught of the ship	0-10 (i)	>0

3.15 Speed of vessel (Y13_2)

Equations

The limit speed of the vessel, V_L (m/s), is calculated by Equation 2

$$V_L = F_L \sqrt{gA_c / b_w} \tag{1}$$

where: F_L is determined implicitly by equation 3:

$$F_L = \left[2/3 \left(1 - \frac{A_m}{A_c} + 0.5 F_L^2 \right) \right]^{3/2} \tag{2}$$

And A_c = cross sectional area of the waterway (m^2); b_w=width of the waterway at the waterline (m).

Other relevant speed limits are given by Equations 4 and 5

$$V_L = \left(gL_s / 2\pi \right)^{1/2} \tag{3}$$

$$V_L = \left(gh \right)^{1/2} \tag{4}$$

The minimum value should be applied in further calculations. The actual speed of the vessel, V_s (m/s), is evaluated as a factor of the limit speed V_L (see Equation 6):

$$V_s = f_v V_L \tag{6}$$

where: f_v = 0.9 for unloaded ships and f_v = 0.75 for loaded ships.

parameter	short description	unit
V_L	Vessel's limit speed	[m/s]
V_S	Vessel's actual speed	[m/s]
F_L	Coefficient	[-]
A_m	The vessel's submerged cross-section	[m^2]
A_c	Cross sectional area of the waterway	[m^2]
L_s	Ship length	[m]
g	Gravitational acceleration	[m/s^2]
h	Water depth of fairway	[m]
b_w	Waterline width	[m]
f_v	Constant f_v = 0.9 for unloaded ships and f_v = 0.75 for loaded ships	[-]

Input and output parameters

Input:	Output:
A_c, b_w, A_m, h, f_v, L_s	**V_L. The minimum value of equation 3,4 and 1 should be selected by CRESS** **V_S**

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
V_L	Vessel's limit speed	0-20 (i)	>0
F_L	Coefficient	0.7-1 (i)	0-1
A_m	The vessel's submerged cross-section	0-500 (i)	>0
A_c	Cross sectional area of the waterway	0-500 (i)	>0
L_s	Ship length	0-300 (i)	>0
g	Gravitational acceleration	9.6-9.9 (i) Default 9.81	>0
h	Water depth of fairway	0 – 20	>0
b_w	Waterline width	0 – 300	>0

3.16 Water depression (Y13_3)

Equations

The mean water level depression, Δh (m), is calculated by Equation 1

$$\Delta h = \frac{V_s^2}{2g}\left[\alpha_s \left(A_c / A_c^*\right)^2 - 1\right] \tag{1}$$

Where the factor α to express the effect of the sailing speed Vs relative to its maximum (-) is described by

$$\alpha = 1.4 - 0.4V_s / V_L \tag{2}$$

factor A^*_c the cross-sectional area of the fairway next to the ship (m^2)

$$A_c^* = b_b\left(h - \Delta h\right) + \cot\alpha\left(h - \Delta h\right)^2 - A_m \tag{3}$$

factor A_c the cross-sectional area of the fairway in the undisturbed situation (m^2)

$$A_c = b_b h + \cot\alpha h^2 \tag{4}$$

And α is the slope of the bank

The mean return flow, U_r (m/s), is calculated by Equation 5.

$$U_r = V_s\left(A_c / A_c^* - 1\right) \tag{5}$$

The maximum water level depression, $\Delta\hat{h}$ (m/s) can be calculated by Equation 4.175:

$$\begin{aligned} &\Delta\hat{h} / \Delta h = 1 + 2A_w^* && b_w / L_s < 1.5 \\ &\Delta\hat{h} / \Delta h = 1 + 4A_w^* && b_w / L_s \geq 1.5 \end{aligned} \tag{6}$$

where:

$$A_w^* = yh / A_c \tag{7}$$

and y is the ship position, relative to the fairway axis. For ratios of A_c /A_m smaller than about 5 (i.e. comparable with b_w/B_s < 10) the flow field induced by sailing ships might be considered as one-dimensional. For these situations Equation 8 is applicable.

$$\begin{aligned} &\hat{U}_r / U_r = 1 + A_w^* && b_w / L_s < 1.5 \\ &\hat{U}_r / U_r = 1 + 3A_w^* && b_w / L_s \geq 1.5 \end{aligned} \tag{8}$$

For larger ratios, ie A_c /A_m > 5 or b_w/B_s > 10, the flow field is two-dimensional. Then, the gradient in the return current and the water level depression between the ship and the bank should be taken into account. In the computer program DIPRO these formulae are incorporated. At some places horizontal berms are present in embankments. Depending on the water depth, the water motion may become super-critical. More information on situations in which the Froude number related to ship speed and water depth above the berm plays a role can be found in Van der Wal (1989).

parameter	short description	unit
V_L	Vessel's limit speed	[m/s]
V_S	Vessel's actual speed	[m/s]
α_s	Coefficient	[-]
A_c^*	the cross-sectional area of the fairway next to the ship	[m^2]
A_c	cross-sectional area of the fairway in the undisturbed situation	[m^2]
b_b	width of fairway at the bed	[m]
Δh	mean water level depression	[m/s^2]
$\Delta\hat{h}$	maximum water level depression,	[m]
h	Water depth of fairway	[m]
b_w	Waterline width	[m]
A_m	The vessel's submerged cross-section	[-]
U_r	Return flow	[m/s]
$\hat{U}_r$	Maximum return flow	[m/s]
L_s	Length of vessel	[m]
y	Vessel's position, relative to the fairway axis	[m]

Input and output parameters

Input:	Output:
V_s , V_L , h, A_m, b_b, y	Δh, $\Delta\hat{h}$, U_r, $\hat{U}_r$

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
V_L	Vessel's limit speed	>0	>0
V_S	Vessel's actual speed	>0	>0
α_s	Coefficient	0-1.4 (f)	>0
A_c^*	the cross-sectional area of the fairway next to the ship	0-10000 (i)	>0
A_c	cross-sectional area of the fairway in the undisturbed situation	For ratios of A_c /A_m smaller than about 5) the flow field induced by sailing ships might be considered as one-dimensional For larger ratios the flow field is two-dimensional. (f)	>0
b_b	width of fairway at the bed	0-3000 (i)	>0
Δh	mean water level depression	0-5 (i)	>0
$\Delta\hat{h}$	maximum water level depression,	0-5 (i)	>0
h	Water depth of fairway	0-30 (i)	>0
b_w	Waterline width	0-500 (i)	>0
A_m	The vessel's submerged cross-section	For ratios of A_c /A_m smaller than about 5) the flow field induced by sailing ships might be considered as one-dimensional For larger ratios the flow field is two-dimensional. (f)	>0
U_r	Return flow	0-4 (i)	>0
$\hat{U}_r$	Maximum return flow	0-4 (i)	>0
L_s	Length of vessel	0-300 (i)	>0
y	Vessel's position, relative to the fairway axis	0-300 (i)	>0

3.17 Front wave height and Stern wave height (Y13_4)

The characteristics of the front wave can be calculated by Equations 1 and Equations 2.

$$\Delta h_f = 0.1\Delta h + \Delta\hat{h} \tag{1}$$

$$i_f = 0.03\Delta h_f \tag{2}$$

The characteristics of the stern wave can be calculated by Equations 3 to Equations 4:

$$z_{\max} = 1.5\Delta\hat{h} \tag{3}$$

$$i_{\max} = \left(z_{\max} / z_o\right)^2 \qquad i_{\max} < 0.15$$

where:

$$z_0 = 0.16 y_s - c_2 \tag{4}$$

$$y_s = 0.5 b_w - B_s - y \qquad i_{\max} < 0.15 \tag{5}$$

and c_2 = 0.2 to 2.6.

The maximum stern wave velocity is calculated using

$$u_{\max} = V_s\left(1 - \Delta D_{50} / z_{\max}\right) \tag{6}$$

where D_{50} = roughness of the bed (m) and Δ = relative buoyant density of the material (-).

$$\Delta = \left(\rho_s / \rho_w\right) - 1 \tag{7}$$

parameter	short description	unit
Δh	mean water level depression	[m/s^2]
$\Delta\hat{h}$	maximum water level depression,	[m]
Δh_f	The front wave height	[m]
i_f	The front wave steepness	[-]
i_{max}	Stern wave steepness	[-]
z_{max}	Stern wave height	[m]
u_{max}	Stern wave velocity	[m/s]
z_0	Coefficient	
c_2	Coefficient	
b_w	Waterline width	[m]

parameter	short description	unit
B_s	Ship beam	[m]
Δ	relative buoyant density of the material	[-]
D_{50}	roughness of the bed	[m]
y_s	Vessel's position, relative to the bank position	[m]
V_s	Vessel's actual speed	[m/s]
y	Vessel's position, relative to the fairway axis	[m]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

Input and output parameters

Input:	Output:
Δh, $\Delta\hat{h}$, z_0, B_s, b_w, c_2, V_s	Δh_f, i_f, i_{max}, z_{max}, u_{max}

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
Δh	mean water level depression	0-5 (i)	>0
$\Delta\hat{h}$	maximum water level depression,	0-5 (i)	>0
Δh_f	The front wave height	0-3 (i)	>0
i_f	The front wave steepness	0-0.2 (i)	>0
i_{max}	Stern wave steepness	0-0.2 (i)	>0
z_{max}	Stern wave height	0 – 3 (i)	>0
u_{max}	Stern wave velocity	0 – 3 (i)	>0
z_0	Coefficient	$<0.16y_s-c_2$ (f)	>0
c_2	Coefficient	0.2-0.6 not reference (?)	
b_w	Waterline width	0 – 400	>0
B_s	Ship beam	0 – 100	>0
Δ	relative buoyant density of the material	0-3 (f)	>0
D_{50}	median sieve diameter of the bed material	0-0.2 (i)	>0
y_s	Vessel's position, relative to the bank position	0-300 (i)	>0
V_S	Vessel's actual speed	0-10 (i)	>0
y	Vessel's position, relative to the fairway axis	0-300 (i)	>0
i_{max}	Stern wave steepness	0-0.2 (i)	>0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
z_{max}	Stern wave height	0-3 (i)	>0

Return currents in a groyne field (Y13_5)

The ship-induced return currents in a groyne field along a navigation canal or river can be estimated by Equation 8:

$$\frac{U_{local}}{U+U_r} = \alpha\left(\frac{h}{h_{ref}}\right)^{-1.4} \qquad (1)$$

It should be noted that Equation 1 is an empirical equation for the River Waal in the Netherlands that predicts the maximum flow velocity just downstream of the groyne when the stern of a push-tow unit passes. Designers should be aware that applying this equation for other rivers might not be valid.

parameter	short description	unit
U_{local}	maximum flow velocity at a location in a groyne field	[m/s]
U	average flow velocity in the river	[m/s]
U_r	average return current in front of the groyne heads exclusive the natural flow velocity	[m/s]
h	average water depth in the river	[m]
h_{ref}	average water depth in the river at a discharge at which the groynes submerge	[m]
α	coefficient depending on the location in the groyne field (-), α = 0.20 to 0.60.	[-]

3.18 Secondary ship waves H_i, L_i, T_i (Y_14)

Ships create transversal and longitudinal waves of which the interference peaks are called secondary waves. The interference peaks can be observed on lines making an angle of 19° with the vessel axis, their direction of propagation makes an angle of 35° with the axis of the ship (or 55° with the normal to the bank). Fast-moving ships, for example container vessels, loose tugs or freighters that are not fully loaded, generate the most severe secondary waves H_i. Ship wave heights vary between 0.25 m and 0.5 m, with maximum values of H_i of about 1.0 m. The wave period T_i is 2–4 s. Fast ferries also generate ship waves, but their characteristics differ from those of other types of ship because fast ferries sail above the critical speed limit. The height of the waves (often called wash) generated by a fast ferry can be up to 1.0 m, particularly if it is accelerating or decelerating close to the critical speed. A typical wave period for fast ferry waves is 9 s. The typical effect of secondary ship waves has some proven similarity with the effect of wind waves on rock structures, so basic equations for wind waves can be applied.

Characteristics of the largest secondary waves can be approximated (for $V_s / \sqrt{(gh)} < 0.8$) with Equations 1 to 3:

$$H_i = 1.2\alpha_i h\left(y_s / h\right)^{-1/3} V_s^4 / \left(gh\right)^2 \qquad \text{(Equation 1)}$$

$$L_i = 4.2 V_s^2 / g \qquad \text{(Equation 2)}$$

$$T_i = 5.1 V_s / g \qquad \text{(Equation 3)}$$

where:
α_i = coefficient depending on the type of ship with the following recommended values:
α_i = 1 for tugs and recreational craft and loaded conventional ships
α_i = 0.35 for unloaded conventional ships
α_i = 1 for unloaded push units.

α_i	1	tugs and recreational craft and loaded conventional ships
	0.35	unloaded conventional ships
	1	unloaded push units

Table 1: Coefficient depending on the type of ship

parameter	**short description**	**unit**
H_i	Secondary ship wave height	[m/s]
L_i	Secondary ship wave length	[m]
T_i	Secondary ship period	[-]
g	Gravitational acceleration	[m²]
y_s	Vessel's position, relative to the bank position	[m]
V_s	Vessel's actual speed	[m/s]

parameter	short description	unit
α_i	**coefficient depending on the type of ship**	[-]

Input and output parameters

Input:	Output:
V_s , α_i, h, y_s	***H_i, T_i, L_i***

Boundary values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H_i	Secondary ship wave height	0-2 (i)	>0
L_i	Secondary ship wave length	0-100 (i)	>0
T_i	Secondary ship period	2-15 (i)	>0
y_s	Vessel's position, relative to the bank position	0-300 (i)	>0
V_S	Vessel's actual speed	0-20 (i)	>0
α_i	coefficient depending on the type of ship	See table	0-1

Propeller jet velocities [Y15]

Near-bed velocities in the propeller jets of the main propulsion system behind a ship might reach 6 m/s or even higher. Flow velocities in bow and stern thrusters can reach up to about 3 m/s. These flow velocities occur if the ship is manoeuvring, ie they are usually found in or next to locks, near quay walls, or in swinging basins (see Figure 1). The water velocities in the propeller jets of a sailing ship can be ignored for most situations.

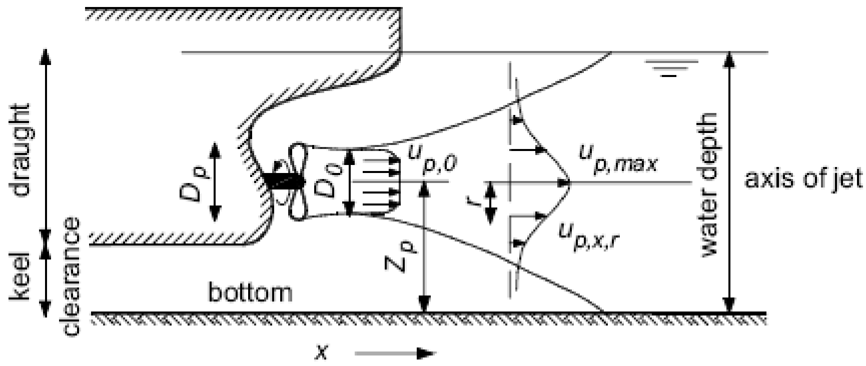

Figure 1 Water movements due to a main propeller

Equations

Equations 1 to 4 can be used to estimate the time-averaged current velocities in propeller jets caused by main propellers (see Figure 1, for ship speed V_s = 0 or otherwise relative to the ship when underway) or caused by bow or stern thrusters.

Velocity behind propeller (see Equation1):

$$u_{p,0} = 1.15(P / \rho_w D_0{}^2)^{1/3} \qquad \text{Equation 1}$$

Velocity along jet axis (see Equation 2):

$$u_{p,axis}(x) = a\,u_{p,0}(D_0 / x)^m \qquad \text{Equation 2}$$

Velocity distribution (see Equation 3):

$$u_{p,r}(x,r) = u_{p,axis}(x)\exp[-br^2 / x^2] \qquad \text{Equation 3}$$

Maximum bed velocity along horizontal bed (see Equation 4)

$$u_{p,\max bed} = c\,u_{p,0}(D_0 / z_p)^n \qquad \text{Equation 4}$$

An overview of the used parameters is given below:

parameter	short description	unit
P	applied power	[W]
D_0	effective diameter of propeller	[m]

parameter	short description	unit
	D0 = 0.7 (for free propellers without nozzle) to 1 (for propellers and thrusters in a nozzle) times the real diameter Dp (m),	
z_p	distance between the propeller axis and the bed	[m]
x	distance away from the propeller along the axis of the jet	[m]
r	distance perpendicular to the axis of the jet	[m]
$\boldsymbol{\rho_w}$	Mass density of water	[kg/m^3]
a	empirical coefficient	[-]
b	empirical coefficient	[-]
c	empirical coefficient	[-]
m	empirical coefficient	[-]
n	empirical coefficient	[-]

In addition to the approach presented below, reference is made to Fuehrer et al (1987), Römish (1993) and EAU (1996, 2004) where alternative values are presented. For more information, reference is also made to a future publication of the PIANC Working Group 48 (PIANC, 2006).

In the Netherlands these coefficients are generally used for design, neglecting the influence of rudders and confinements with the following values:
m =1
n = 1,
a = 2.8 and
b = 15.4,
which results in c = 0.3 (Blaauw and Van der Kaa, 1978).

In this approach the influence of lateral confinement by a quay wall in some cases is taken into account by increasing the velocity according to Equation 4 by 10–40 per cent. Blokland and Smedes (1996) measured a 40 per cent higher bottom velocity in the case of a jet that displays an angle of 16° with the quay wall. In the case of a propeller jet perpendicular or oblique against a sloping embankment, the velocities above the embankment can be estimated using Equation 3. In fact, the velocities in the jet are influenced by the presence of the embankment. In PIANC (1997) this influence is neglected for practical purposes. Hamill et al (1996) found that the velocities above the embankment are delayed.

In the case of a propeller jet perpendicular to a quay wall (eg caused by bow or stern thrusters, see Figure 2) the current velocity above the bottom in front of the quay wall can be estimated using Equation 2 for the velocity along the axis of the propeller jet. Blokland and Smedes (1996) propose to use Equation 2 with
m = 1,
a = 2.8 and
x = max ($z_{pq} + z_p$; 2,8 z_p),

where z_{pq} = distance between propeller (or the end of the propeller duct) and quay wall.

If the propeller or thruster is not close to the quay wall, $u_{p,bed}$ calculated by Equation 4 (n = 1) can be larger than $u_{p,axis}$ calculated by Equation 2 (see Figure 2).

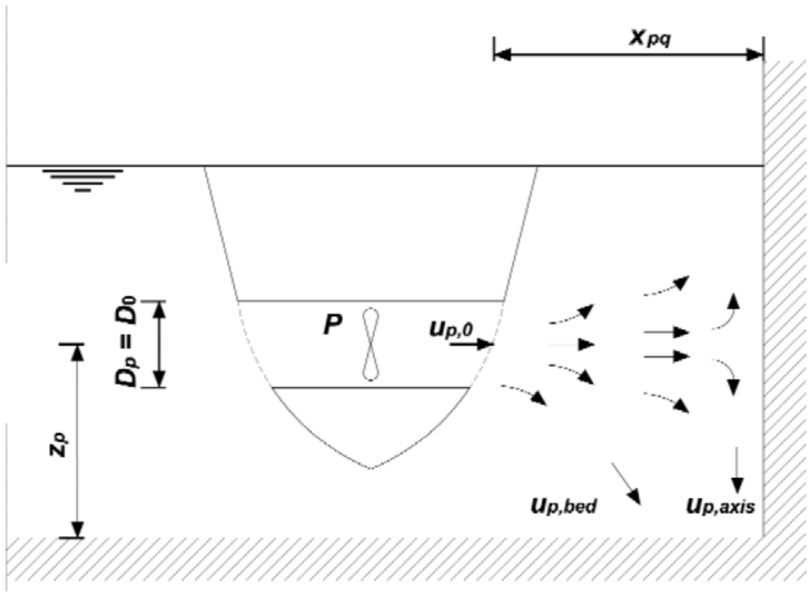

Figure 2 Flow field generated by a bow- or stern-thruster perpendicular to a quay wall

Very often the propeller diameter is not known. WL|Delft Hydraulics found an empirical relationship given by Equation 5 between propeller diameter D_p and installed engine power P (W); see also PIANC (2006).

$$D_p = 0.0133 P^{0.3651/3} \qquad \text{Equation 5}$$

This formula is valid for main propellers as well as bow and stern thrusters. Finally, some modern twin-hulled ships, such as ferries, have high-powered water jets located at the water level. These jets generate much higher flow velocities, up to 25 m/s at the outflow orifice. Being at the water level, these jets hardly affect the bed material but may affect slopes or quay walls behind ships. Bed stability is at greater risk when the ship is sailing backwards. In this situation the jet is directed not just to the bow of the ship but also to the bed under an angle of about 30° with the horizontal. This may result in flow velocities near the bed of about 10 m/s. Protection against these high-powered water jets requires particular care during the design.

Input and output parameters

Input:	**Output:**
P, D_0, z_p, x, r, ρ_w, , a, b, c, m, n	$u_{p,0}$, $u_{p,r}(x,r)$, $u_{p,\max bed}$, D_p

Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values	default values
P	applied power	0 – 10000 (i)	>0	1000
D_0	effective diameter of propeller	0 – 10 (i)	>0	1.5
z_p	distance between the propeller axis and the bed	0 – 100 (j)	>0	5
x	distance away from the propeller along the axis of the jet	0 – 1000 (j)	>0	0
r	distance away from the axis of the jet	0 – 1000 (j)	>0	0
ρ_w	Mass density of water	800 – 1200 (j)	>0	1025
a	empirical coefficient	0-100	>0	2.8
b	empirical coefficient	0-100	>0	15.4
c	empirical coefficient	0-100	>0	0.3
m	empirical coefficient	0-10	>0	1
n	empirical coefficient	0-10	>0	1

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