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Wave revetments for inland lakes

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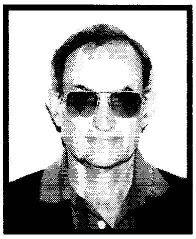

CHRIS WATERMEYER matriculated from Christian Brothers College in Kimberley in 1950, graduated at University of Cape Town in 1954, and completed a one-year postgraduate course in structures and strength of materials at University of Cambridge in 1956, and a one-year diploma course in environmental control engineering and resource utilisation at the Royal College of Science & Technology, Glasgow, in 1963. He joined Sir Alexander Gibb and Partners in 1955 and worked in London and on site on two dams in Scotland, before being transferred to Kariba Dam in 1958. Chris married Heather Greenwood in Kariba in 1965 and then ran the Gibb office in Salisbury, Rhodesia until early 1967. After four years with the Sabi Limpopo Authority, he joined Stewart Sviridov and Oliver in Rhodesia in 1971, was transferred to Johannesburg in 1978, became a partner in 1986, retired from Stewart Scott as a technical director in June 1999, and then set up Merman Engineering Enterprises cc, working mainly on dams.

The presentation of data comprises

- ***selected wave theory, sufficient for wave revetment and wave baffle boards design purposes***
- ***rock rip-rap wave revetments, which include the original derivation of a new modified Hudson formula (the design of the underlayers is included)***
- ***concrete-filled Hyson Cell wave revetments, which include the original derivation of a simple, safe method of design of such revetments***
- ***cement stabilised soil wave revetments, which provide the ingredients of a specification for such revetments***

INTRODUCTION

The cost of providing an adequate wave revetment on the upstream face of an earth embankment dam generally constitutes a significant portion of the overall cost of the dam. Failure of the wave revetment can result in compulsory lowering of the WL, together with high costs of repair and upgrading. Failures generally act as a spur for further investigation. The compilation of the data as presented was spurred by the failure of the concrete-filled Hyson Cell wave revetment at Bloemhoek Dam, Kroonstad, in 1996/97, the revetment having been speedily constructed and completed on 15 December 1995. Everything appeared to be fine during a post-completion inspection, because the construction collapse of most of the HDPE cells to between 30 % and 70 % of specified thickness was not discernible. The missing concrete was never accounted for.

This presentation of data comprises:

- selected wave theory, sufficient for wave revetment and wave baffle board design purposes
- rock rip-rap wave revetments, which include the derivation of a modified Hudson formula and the design of the underlayers
- concrete-filled Hyson Cell wave revetments, which include the derivation of a simple safe method of design of such revetments
- cement-stabilised soil wave revetments, which provide the ingredients of a specification for such revetments

Design of two essential requirements of all wave revetments on earth slopes is not included, namely

- a stable toe to the revetment, which ensures that sliding or base unravelling will not occur
- slope stability beneath the revetment, especially under rapid draw-down conditions

The symbols used are not defined in the text but are listed in alphabetical order and defined in the symbols section before the references. This facilitates unambiguous re-use of symbols in the different sections of the paper.

SELECTED WAVE THEORY

Keulegan (1950) wrote: 'A gravity wave is, strictly speaking, the most complex phenomenon of fluid motion that exists, since in general it is both unsteady and non-uniform and involves both normal acceleration and viscous resistance to at least some degree.'

Shallow water gravity waves

Per the *Shore protection manual* (1977), shallow water gravity waves occur when $y < 0{,}040 \,.\, L$ and then $C = \sqrt{g.y}$.

Deepwater gravity waves

These are the waves involved in the design of wave revetments, and according to Keulegan (1950), deepwater gravity waves occur when $y \geq 0{,}5L$. Przedwojski *et al* (1995) as well as Thompson *et al* (1976) consider that deepwater gravity wave theory can be used when $y > 0{,}25L$.

$$C = \sqrt{\frac{g.L}{2.\pi}} = \frac{L}{T} \tag{1}$$

$$L = \frac{g.T^2}{2.\pi} = 1{,}561.T^2 \tag{2}$$

$$Ls = 1{,}561.Ts^2 \tag{3}$$

Refer to equation 8 for Ts.

Based on Keulegan (1950), particle motion is negligible at depths greater than L. The crests of deepwater waves can be about 4.H/7 above SWL, and surface transfer of water occurs, leading to an undertow offshore.

Significant wave heights

Figure A4 of SANCOLD Report No 3 (1990) is reproduced from ICE, London (1978), and the following values of a and b, as used in calculating K, are based on figure A4 for Vw in the range of 10 to 30 m/s.

$$Hs = K \,.\, Vw \,.\, \sqrt{Fe} \qquad (4)$$

$$K = (a. + b.\, Vw) \qquad (5)$$

For Fe = 0,50 to 5,0 km, adopt b = 1/20 000
For Fe = 5,0 to 15,0 km, adopt b = 1/10 000

The values of a are dependent on Fe and are shown in table 1. They give identical values for K at Fe = 5,0 km when Vw = 20m/s.

Table 1 Values of a for the calculation of K

Fe in km	a	Fe in km	a
0,50	0,0235	5,0 to 5,5	0,0205
1,00	0,0225	10,0	0,0202
2,0	0,0220	15,0	0,0202
4,5 to 5,0	0,0215		

The hourly mean design wind speed for a 50-year return period can be adopted for Vw, as shown in figure A1 of SANCOLD Report No 3 (1990) for the RSA. These values of Vw are 20 m/s for the bulk of the inland region, 25 m/s for a 35 to 50 km wide coastal strip and 30 m/s for an area approximately 120 km in diameter centred over Beaufort West, at 175 km north of George. Wind durations required for full wave development are approximately as shown in table 2, which is based on data taken from CUR/RWS Report 169 (1995).

Table 2 Wind durations required for full wave development

Fetch	Duration in minutes	
	Vw = 10 m/s	Vw = 30 m/s
1 km	18	12
5 km	60	30
10 km	100	54
15 km	130	72

Per Saville *et al* (1962), wind speed over water is approximately 1,1 times wind speed overland at $F_e = 1{,}0$ km, 1,2 times at $F_e = 3$ km and 1,3 times at $F_e \geq 8$ km.

Wind set-up

$$S = \frac{Vw^2 . F}{Cs . Ya} \text{ , in m} \qquad (6)$$

SANCOLD Report No 3 (1990) gives Cs = 4 850.
CUR/RWS Report 169 (1995) gives Cs in the range 16 200 to 5 400, depending on the air/water friction coefficient.
For most dams S will be less than 100 mm.

Wave energy

The *Shore protection manual* (1977) provides information as follows: Relative to the SWL and an assumed sinusoidal wave profile, the potential energy, PE, equals the kinetic energy, KE, and the total energy, TE, equals (PE + KE). The total energy in one wave length per unit crest width is given as:

TE per unit crest width per wave length

$$= \frac{\rho w . g . H^2 . L}{8} \qquad (7)$$

Table 3 Ratios of Rus/Hs for wave run-up

Slope type	G =1,5	G=2,0	G=2,5	G=3,0	G=4,0	G=5,0	G=6,0	G=10
	Values of Rus/Hs							
Non-porous smooth	2,40	2,20	2,00	1,80	1,40	1,13	0,96	0,60
Graded rip-rap on non-porous core	1,95	1,85	1,75	1,60	1,24	1,03	0,90	0,60
Rip-rap on a porous rock core	1,20	1,10	1,01	0,93	0,80	0,68	0,60	0,40

Wave run-up

For a vertical face, Rus = 1,00 . Hs.
According to CUR/RWS Report 169 (1995), Ru 2% = 1,4 . Rus.
In table 3, the values of Rus/Hs for a non-porous smooth slope are based on figure A5 of SANCOLD Report No 3 (1990), but modified to pass through the ratios 1,8 at G = 3 and 0,6 at G = 10. The values of Rus/Hs for graded rip-rap on a non-porous core are based on Thompson *et al* (1976), and the values for rip-rap on a porous rock core are based on CUR/RWS Report 169 (1995).

Wave run-down

Based on Thompson *et al* (1976), the JONSWAP formula for Ts in seconds can be defined as follows:

$$Ts = 0{,}528 \,.\, Vw^{0,4} \,.\, F^{0,3} \qquad (8)$$

The JONSWAP values of Ts should never be used with values of Hs as calculated using equations 4 and 5, but should be multiplied by the following factor, Ks, in order to bring them in line with the Saville *et al* (1962) values of Ts as shown on figure A3 of SANCOLD Report No 3 (1990):

F (km)	0,5	1,0	5,0	10,0	15,0
Ks	1,34	1,31	1,28	1,25	1,24

Saville – Ts = Ks . JONSWAP – Ts

Based on CUR/RWS Report 169 (1995), the Iribarren No (or Surf Similarity Parameter or Breaker Parameter) for significant wave height can be defined as follows:

$$Irs = \frac{1{,}25 . Ts}{G . \sqrt{Hs}} \qquad (9)$$

For a non-porous smooth slope (CUR/RWS Report 169 [1995])

$$\frac{Rd2\%}{Hs} = 0{,}355 \,.\, Irs, \text{ but } \leq 1{,}5 \text{ for } Irs > 4{,}2 \qquad (10)$$

For graded rip-rap on non-porous core (Thompson et al *[1976])*

$$Rds = \frac{0{,}357 . Ts . \sqrt{Hs}}{G} \text{ in m} \qquad (11)$$

$$\frac{Rds}{Hs} = 0{,}286 \,.\, Irs, \text{ but } \leq 1{,}7 \text{ for } Irs > 6 \qquad (12)$$

Wind waves along rivers and canals

The generation of wind waves along rivers and canals differs significantly from that in large lakes and reservoirs due to relatively short lengths of fetch, the flow velocity and direction of flow relative to the wind, and the effect of friction due to relatively close banks. Based on data given in Przedwojski *et al* (1995), it appears that for a 10 m wide canal under no flow condition and 20 m/s wind velocity, the wave heights may range from 0,50.H at F = 0,1 km to 0,25.H at F = 1,0 km, where H refers to the open water wave height on a lake. The wave heights are considered proportional to $Ø.B^{0,45}$, where B is the channel width and Ø is a function of F/B.

Effective fetch, Fe, for calculation of Hs

The method of calculating Fe as illustrated in figure A2 of SANCOLD Report No 3 (1990) was developed by Saville *et al* (1962). Per Thompson *et al* (1976), the method was developed so as to cause the Saville *et al* (1962) recorded data to fall on the same line as Sverdrup-Munk-Bretschneider (SMB) data. According to the CUR/RWS Report 169 (1995), the method should only be used with SMB wave prediction formulae, or else serious underestimation of Hs will occur. Equations 4 and 5 of this paper replicate the values of Hs that would be obtained by using figure 9 of ICE, London (1978), to an accuracy of better than ± 3% for Fe in the range of 0,5 km to 15 km, and for Vw in the range of 10 m/s to 30 m/s. Figure 9 of ICE, London (1978) is reproduced in figure A4 of SANCOLD Report No 3 (1990), but the source of figure 9 is not stated. It is probable that the source is Saville *et al* (1962) and based on SMB wave prediction formulae. Hence it is probable that the method of calculating Fe as developed by Saville *et al* (1962) can be used correctly in conjunction with figure 9 of ICE, London (1978) in order to calculate Hs.

The JONSWAP formula for Hs is given as equation 3 in Thompson *et al* (1976) and can be reproduced as follows:

$$Hs = 16{,}154 \,.\, Vw \,.\, \sqrt{Fe} \,.\, 10^{-3} \qquad (13)$$

For values of Fe between 0,5 km and 15 km and values of Vw between 10 m/s and 30 m/s, the JONSWAP formula gives values of Hs that are between 67 % and 77 % (mean of 71 %) of the values of Hs obtained by using figure 9 of ICE, London (1978). Hence the value of Fe must be doubled for use in the JONSWAP formula for Hs in order to obtain a similar value to the value of Hs obtained by using figure 9 of ICE, London (1978).

In applying their design criteria for values of Hs/D^r_{50}, Thompson *et al* (1976) recommend that Saville *et al* (1962) be used to obtain Fe, Ts and Hs and that the result be checked by using the JONSWAP formula for Hs. However, there appears to be no point in using the JONSWAP formula if underestimation of Hs is to be avoided.

Proposed strip method for calculating Fe

The proposed method is illustrated in figure 1 and is intended to take account of the dampening effect of shore friction on mid-water wave heights. The strip width, Wf, is based on the premise (or principle) that the dampening effect of shore friction on wave heights diminishes with distance from the shore, but extends gradually further from the shore as the height of the waves increases.

$$Fe = Ae/Wf \qquad (14)$$

$$Wf = \sqrt{F}\ /Kf \qquad (15)$$

The author has no evidence that Kf is a constant, but adoption of Kf = 4 results in the following seemingly realistic values for Wf.

F (km)	0,5	1	4	8	16
Wf (km)	1,77	0,250	0,500	0,707	1,000

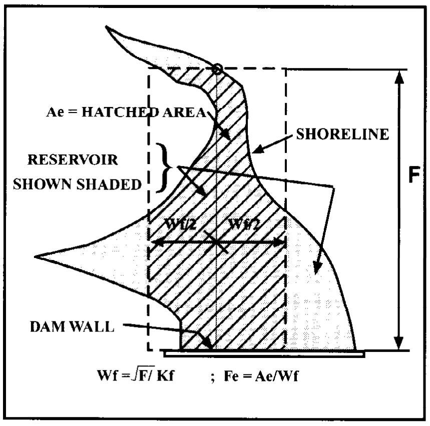

Figure 1 Strip method of calculating the effect fetch, Fe

ROCK RIP-RAP WAVE REVETMENTS

Derivation of a modified Hudson formula for sizing rock rip-rap

Adopting $Hd^n = Hd^3$, putting Hd = 1,25.Hs in accordance with Taylor (1973) and inserting these values into equation 6 of Thompson *et al* (1976), results in the following form of the Hudson formula for sizing rock rip-rap, with Kr = 1,82:

$$W^r_{50} = \frac{1{,}953.\gamma r.Hs^3}{Kr.(Sr-1)^3\ \cot\alpha} \qquad (16)$$

Consider the following relationships:

$$V^r_{50} = W^r_{50}/\gamma r \qquad (17)$$

$$D^r_{50} = \left[\frac{6.V^r_{50}}{\pi}\right]^{\frac{1}{3}} \qquad (18)$$

$$\text{Cot } \alpha = G, \text{ where slope is 1:G} \qquad (19)$$

$$Kc = \left(\frac{1}{Kr}\right)^{\frac{1}{3}} \qquad (20)$$

Equation 18 is based on the volume of a sphere.

Substituting equations 17, 18, 19 and 20 into equation 16 results in the Modified Hudson Formula as follows:

$$D^r_{50} = \frac{1{,}551.Kc.Hs}{(Sr-1).G^{0,333}} \qquad (21)$$

A value of Kr = 1,82 converts to a value of Kc = 0,819. According to Ben Belfadhel *et al* (1996), the Hudson Formula predicts realistic behaviour of rip-rap compatible with field observations and is on the safe side. Values of Kc in the Modified Hudson Formula can be adopted so as to take into account various degrees of damage during defined storm durations and this negates the criticism of the Hudson Formula by Van der Meer (1997).

Pitt *et al* (1982) recommend the continued use of the design criteria developed by Thompson *et al* (1976), who obtained for four different slopes at 1:G and for storm durations of 1 000 to 5 000 waves, values of the ratio Hs/D^r_{50} required for four different damage criteria, all based on a value of Sr = 2,70 and on a rip-rap layer thickness of Th = 2. D^r_{50}, over a filter layer on an impermeable core.

Required values of Kc for use in equation 21 have been calculated, which replicate exactly the values for D^r_{50} obtained in accordance with the design criteria developed by Thompson *et al* (1976) for the case of a storm duration of 5 000 waves and for two different damage criteria as follows:

Criterion A = no damage

Criterion C = 15 % of the damage that would occur at failure (at failure a 0,5.D^r_{50} hole occurs through the rip-rap to the filter layer)

The values of Kc are listed in table 4.

Observation of the values of Kc favours the adoption of Criterion C as an adequate practical compromise which necessitates some maintenance work. In addition, the values of Kc for Criterion C are comparable with Kc = 0,819, which converts to Kr = 1,82 for the Hudson Formula.

A storm of 5 000 waves has an approximate duration of a little less than 5 000.Ts (say about five hours).

Based on the model tests carried out by Thompson *et al* (1976), the values of Kc in table 4 apply to a rip-rap having Th = 2.D^r_{50}, over a filter layer having $D^f_{50} = D^r_{50}/4{,}5$, $D^f_{85} = D^r_{15}/2$, $D^f_{15} = D^r_{15}/4{,}5$ and filter Th = $D^r_{50}/2$. In addition, the rip-rap was graded as follows:

$$D^r_{85}/D^r_{50} = 1{,}5; \quad D^r_{15}/D^r_{50} = 0{,}67$$

It must be borne in mind, however, that the largest rip-rap stone size used in the model tests was D^r_{50} = 0,040 m and hence an underlayer between rip-rap and filter was not required.

The long axis of the rocks should not exceed 2,5 times the short axis.

First underlayer below rip-rap and the filter layers

Reference: CUR/RWS Report 169 (1995).

For each layer, adopt Th ≥ 2.D_{50}, when D_{50} ≥ 50 mm.

Weight criteria apply to the design of the rip-rap and of the first underlayer.

$$\text{Adopt } D''_{50} = D^r_{50}\ /Ku \qquad (22)$$

Ku is in the range of 2,2 to 2,5.

Filter criteria apply to the design of the filter layers below the first underlayer.

$$\text{Adopt } D^f_{85} \geq D^c_{15}/Kt \qquad (23)$$

Kt is in the range of 4 to 5.

Thompson *et al* (1976) recommend Kt ≤ 4 and the overall Th of filter layers ≥ 0,5 D^r_{50}, or the minimum allowed by construction.

Table 4 Values of Kc for use in equation 21 for a storm duration of 5 000 waves

Damage criterion	G = 2	G = 3	G = 4	G = 6
	Values of Kc			
A: No damage	1,726	1,580	1,243	1,992
C: 15 % of the damage at failure	0,863	0,878	0,696	0,586

Table 5 Materials and layer thicknesses

1 000	mm	thick layer of	(700 mm – 300 mm)	rocks over
400	mm	thick layer of	(250 mm – 150 mm)	rocks over
100	mm	thick layer of	(50 mm – 20 mm)	crushed stone over
75	mm	thick layer of	(10 mm – 2 mm)	gravel over
75	mm	thick layer of	(2,0 mm – 0,1 mm)	sand, laid on compacted earthfill

Example

Consider rip-rap D^{r}_{50} = 500 mm
The rock rip-rap plus underlayers will comprise the following materials and layer thicknesses:

Total thickness of rip-rap plus filter underlayers = 1,65 m

All layers should be compacted by vibration.

Grading of gravel, which can be crushed stone:

D_{15} to be between 2 and 4 mm
D_{50} to be between 5 and 8 mm
D_{85} to be between 8 and 10 mm

Grading of sand, which should preferably be silica:

Maximum passing	1,000 mm sieve	85 %
Maximum passing	0,600 mm sieve	75 %
Minimum passing	0,425 mm sieve	25 %
Minimum passing	0,300 mm sieve	15 %*
Maximum passing	0,075 mm sieve	10 %**

* A minimum of 15 % of the sand must be finer than 300 microns in order for the loss of fines from dispersive clays to be prevented, provided that $D^{f}_{85} \geq 0{,}033$ mm for the clay soil passing a 2,36 mm sieve.

** A maximum of 10 % of (silt plus clay) particles can be tolerated in the sand in order to ensure that the sand has adequate permeability.

A correctly designed and constructed rock rip-rap wave revetment over filter layers represents the Rolls-Royce of wave revetments.

Geofabrics as filters

The greater the flow conveyance of the combined first underlayer and filter layers, the greater the stability of the rip-rap. Geofabrics can provide no substitute for such flow conveyance as a stabilising factor. The size of underlayer laid upon a geofabric should be such that it will not cause damage to the geofabric during movement of the rip-rap under wave action. The geofabric must be capable of preventing the loss of fine material from the embankment earthfill, without becoming blocked with fine material (and hence impermeable) under the two-way flow that occurs during wave action. A geofabric that becomes impermeable will be subject to uplift during wave drawdown and during rapid drawdown of the reservoir. If the embankment earthfill comprises dispersive clayey material, then preferably a suitable sand filter should be used. Geofabrics (in particular non-woven needle-punched polyester geofabrics) can have very low coefficients of sliding friction and may introduce potential planes of sliding failure on steep revetment slopes. In short, the filter requirements

associated with wave revetments are much more onerous than the filter

requirements associated with many one-way flow drainage conditions and careful scrutiny of the appropriateness of a geofabric filter layer is necessary before it is incorporated in a wave revetment.

CONCRETE-FILLED HYSON CELL WAVE REVETMENTS

Description of Hyson Cells

Hyson Cells can be described as geocells manufactured from heat-welded 200 micron thick sheets of bonded three-layered HDPE, by processes developed and patented by Sally Hall (née Hyson). When stretched out and rigged ready for filling with the chosen fill material, the cells form squares of various side and depth dimensions. Each side of each cell incorporates a hemispherical bubble which produces a ball-and-socket interlocking key with the adjacent cell after filling with concrete or sand/cement mortar, grouted stone, soilcrete, vibrated wet sand, soil, ash, mine tailings, etc, and the partial flexibility of the Hyson Cell mattress at the joints is retained together with the facility of releasing uplift water pressures through the joints. Hence concrete-filled Hyson Cells can be safely laid on gravel over sand filter layers as erosion protection over compacted clayey earthfill embankments which may undergo differential settlement and which are subjected to scour, traffic or other damaging processes. The lateral containment of compacted columns of earthfill, sand, mine tailings, etc, in Hyson Cells results in a remarkably large increase in the safe-bearing capacity of the compacted fill material. This phenomenon has opened up the use of Hyson Cells in harbour walls, hard standing for heavily loaded areas, and mine props. Concrete-filled Hyson Cells are used in hard standing for heavy vehicles, haul roads on soft earth, and drains subject to scour damage.

400/280 x Dc deep Hyson Cells produce cells which are 0,20 m square x Dc m deep. The shrinkage of Grade 25 MPa concrete over an unrestrained length of 200 mm will be approximately as follows:

After 3 months	40 microns
After 3 years	80 microns

In practice, with concrete-filled Hyson Cell wave revetments on compacted earth embankment slopes, movement of the revetment can occur gradually and some joints between the cell concrete blocks may widen in time to several mm. Hence the cell concrete must be laid upon a gravel filter underlayer over a sand filter layer over the trimmed compacted face of the earth embankment.

Concrete blocks over permeable filter layer wave revetment behaviour and design methodology

Wave action associated with breaking or plunging waves on a sloping revetment results in repeated 'pore pressure' increases and dissipation in the filter layer and soil below the revetment, especially if the revetment incorporates joints which can transmit the sudden changes in water pressure. Unbalanced pore pressure is also caused by wave run-down, acting to push out the revetment blocks. The sudden increases in pore pressure in the interface between the soil and the revetment are probably the major cause of gradual disintegration of a revetment composed of particles (*including blocks*) of insufficient weight to resist the unbalanced pressure pulses. If this fluctuating pore pressure action leads to a widening of some of the joints in the revetment, so that the finer soil particles can be pushed by the unbalanced pressures through the joints and into the reservoir water, then cavities will develop below the jointed revetment and the rate of disintegration of the revetment will accelerate. Theory also indicates that if an adequate permeable filter layer is present and if no fine material is lost through the joints, then widening of joint gaps between the revetment blocks can reduce uplift on the blocks due to wave run-down, through more rapid drainage of the permeable filter layer through the widened joint gaps, and so stabilise the revetment. The pore pressure fluctuations can also lead to movement of fine material down the slope under the revetment and cause the revetment to hump up at lower levels and to sag at the water line if the revetment filter layer and blocks are too light to prevent this movement.

Hyson Cell concrete blocks are interlocked through ball-and-socket interlocking keys, which allow partial flexibility between the blocks. A careful study of Bezuijen *et al* (1996) led to the conclusion that a simple safe method of design of a concrete-filled Hyson Cell wave revetment, over a permeable gravel filter underlayer over a permeable sand filter layer, can be based on the maximum

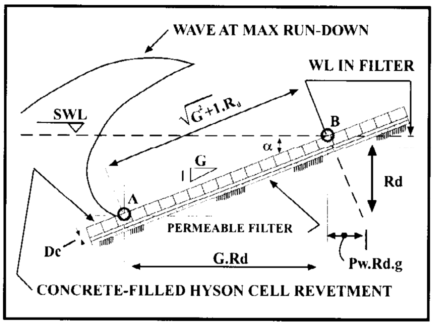

Figure 2** **Design parameters for concrete-filled Hyson Cell wave revetment

uplift on the cell concrete blocks during wave run-down and ignoring the permeability of the joints between the cell concrete blocks. The permeability of the jointed cell concrete block revetment must be less than the permeability of the filter underlayers for the method of analysis to be applicable.

Design criteria

The revetment, as analysed, is illustrated in figure 2.

The maximum value of Rd applicable to the revetment should be determined and a value of Rd less than 0,5.Hs should not be used. A value of $\rho c/\rho w = 2,4$ has been adopted. It is assumed that between points A and B on figure 2 the interlocked cell concrete blocks act as an isolated unit. (The support of the cell blocks above point B and below point A is ignored.)

The permeable filter layer must drain into the reservoir at the toe of the revetment. The assumption made is that the WL in the filter layer remains at SWL during wave rundown. A 1,0 m width of revetment between points A and B is analysed.

Criterion (a): Vertical uplift of revetment between A & B:

$$Pw = \text{vertical uplift} = \frac{G.Rd.g.\rho w.Rd}{2} = \frac{g.\rho w.G.Rd^2}{2} \qquad (24)$$

$$Pr = \text{vertical weight of revetment} = \sqrt{G^2+1}.Rd.D_c.\rho c.g \qquad (25)$$

$$FS = \text{factor of safety} = \frac{Pr}{Pw}$$

Adopt FS = 2

$$\text{Hence } D_c = 2.\frac{\rho w.g}{\rho c.g}.\frac{G}{2}.\frac{Rd^2}{Rd}.\frac{1}{\sqrt{G^2+1}} = \frac{G.Rd}{2,4.\sqrt{G^2+1}} \text{ (m)} \qquad (26)$$

Criterion (b): Rotation of revetment between A & B about A, ignoring revetment above B:

Mw = Moment at A due to uplift on concrete cell revetment
Mc = Moment at A due to weight of concrete cell revetment

$$Mw = \sqrt{G^2+1}.Rd.g.\frac{\rho w.Rd}{2}.\frac{\sqrt{G^2+1}.Rd}{3} = \frac{(G^2+1)Rd^3.\rho w.g}{6} \qquad (27)$$

$$Mc = \sqrt{G^2+1}.Rd.D_c.\rho c.g\frac{G.Rd}{2} = X \quad \frac{Dc.Rd^2.G.\sqrt{G^2+1}.\rho c.g}{2} \qquad (28)$$

$$\text{Adopt FS} = \frac{Mc}{Mw} = 2$$

$$\text{Hence } D_c = \frac{2.(G^2+1)Rd^3.\rho w.g}{6} \quad \frac{2}{Rd^2.G.\sqrt{G^2+1}.\rho c.g} \qquad (29)$$

$$\therefore D_c \frac{2.\sqrt{G^2+1}}{3.G}.Rd.\frac{\rho_w}{\rho_c} = \frac{\sqrt{G^2+1}.Rd}{3,6.G} \text{ (m)} \qquad (30)$$

Criterion (c): Push-out from the revetment of a concrete block of area 'a' at point A:

In this case the interlocking of the Hyson Cell blocks is ignored, but the FS is reduced to 1,0 for $G \geq 1,5$. In pushing the block out of the revetment, the water pressure on the block must be able to lift the block vertically to satisfy the kinematics of removal and to overcome bottom-edge friction.

If the block rotates, it will jam against the adjacent block.

$$Pw = g.\rho w.Rd.\frac{a.G}{\sqrt{G^2+1}} \qquad (31)$$

$$Pr = g.\rho c.D_c.a \qquad (32)$$

$$Fs = \frac{Pr}{Pw} = 1, \text{ for } G \geq 1,5$$

$$\therefore D_c = \frac{g.(FS).\rho w.Rd.a.G}{g.\rho c.a.\sqrt{G^2+1}} \qquad (33)$$

$$\text{Hence } D_c = \frac{Rd.G}{2,4\sqrt{G^2+1}} \text{ (m)} \qquad (34)$$

Equation 34 gives the same value for D_c as equation 26.

Adopted formula for D_c

For values of G greater or equal to 1,5, equation 26 gives larger values of D_c than does equation 30. Hence adopt:

$$D_c = \frac{G.Rd}{2,4.\sqrt{G^2+1}} \text{ (m)} \qquad (35)$$

Permeable gravel over sand filter layers

The permeable gravel over sand filter underlayers are essential for two purposes:

- for pressure relief purposes and for dissipation of water hammer type pressures caused by wave impacts
- for prevention of loss of fine material from the earth embankment (ie clay and silt fractions) which can otherwise be 'pumped' through the cell joints between concrete blocks, whose joints may develop open cracks 1 mm to 3 mm in width, depending upon the movement of the compacted earth embankment.

The permeable filter underlayers must be laid upon earth which has been compacted to 98 % Proctor maximum density. The steepness of the slope for wave revetment purposes should preferably not exceed 1v : 2h, but will also be a function of the strength of the embankment material. The gravel over sand filter underlayers must be compacted by vibration.

The permeable filter underlayers are best constructed in two separate layers comprising

- 75 mm thick layer of (10 mm – 2 mm) gravel, laid over
- 75 mm thick layer of (2,0 mm – 0,1 mm) sand, laid on compacted earthfill

The grading of the gravel and the grading of the sand should comply with the gravel and sand gradings given after the example of rock rip-rap plus underlayers in the section on rock rip-rap wave revetments.

The filter underlayers must drain easily into the reservoir at the toe of the revetment, with protection against loss of filter material.

Size and rigging of Hyson Cells for wave revetments

Preferred sizes of Hyson Cells for wave revetments are 600/420, giving 300 mm square blocks, or else 400/280, giving 200 mm square blocks for use when D_c is less than 200 mm.

The cell mattress should be rigged in strict accordance with the manufacturer's instructions and the drawings, especially with regard to the tautness of the cell walls. The completed cell mattress should be a series of squares, with parallelogram shapes not being acceptable.

Filling of cells and use of sacrificial depth-indicating rods and base plates

Grade 25 MPa/19 mm concrete should have adequate strength and durability as a fill material.

Concrete filling should be done by shovelling concrete into the individual cells. The cells should be overfilled by 10 % to compensate for volume reduction during compaction. The concrete should be compacted by means of poker vibrators.

The cell walls should not be depressed during filling with concrete. Sacrificial cast-in depth-indicating steel rods with base plates can be installed in the cells at ± 5 m centres prior to concrete filling. After filling the cells with concrete, a string can be strung across the tops of the steel rods in order to check the depth of concrete in the intervening cells.

Each depth-indicating steel rod with base plate comprises a D_c long x 12 mm dia mild steel rod and a 150 mm x 150 mm x 6 mm thick mild steel base plate with a central 12 mm dia hole through it. The 12 mm dia steel rod is fitted through the hole in the base plate, with its bottom face flush with the bottom face of the base plate. The rod is welded to the top face of the base plate. The base plate is placed in a cell on the surface of the gravel layer below the Hyson Cells.

After placement and compaction are complete, the top edge of the cell walls should be just visible with D_c depth of concrete filling.

Comparison of a concrete-filled Hyson Cell wave revetment with a rock rip-rap wave revetment

Consider G = 3; S_r = 2,7; K_C = 0,878; Vw = 22,6 m/s and F_e = 3 km

Per equations 5 and 4:
K = 0,0229 and H_s = 0,90 m

Per equation 8: JONSWAP – T_S = 2,555 sec
K_s = 1,295 and hence Saville – T_s = 3,31 sec

Per equation 9: Irs = 1,454

Per equation 10: Rd 2 % = 0,465 m
Adopt Rd = 0,47 m, which exceeds 0,5.H_s

Per equation 21: D'_{50} = 0,50 m = 500 mm

Per equation 35: D_C = 0,186 m
Adopt D_C = 200 mm

Refer to the example for rock rip-rap plus underlayers for D'_{50} = 500 mm, in the section on rock rip-rap wave revetments. The total thickness of rock rip-rap plus underlayers is 1,65 m. This should be compared with 200 mm thick concrete-filled Hyson Cells over a total 150 mm thickness of filter layers. Cost-wise, the concrete-filled Hyson Cell wave revetment appears to be very competitive. The concrete-filled Hyson Cells are much smoother than the rock rip-rap. In accordance with table 3 for G = 3 and H_s = 0,90 m, Rus = 1,44 m for the rip-rap revetment and Rus = 1,62 m for the concrete-filled Hyson Cell revetment. It should be borne in mind, however, that Ru2% = 1,4 . Rus.

CEMENT-STABILISED SOIL WAVE REVETMENTS

Reference

ICOLD Bulletin 54 (1986) has been used as a reference in drawing up a specification for a cement-stabilised soil wave revetment to the upstream face of an earth embankment.

Materials

(a) Cement: Ordinary Portland cement (OPC), less than 100 days old, is used. Rapid-hardening Portland cement is not permitted.

(b) Soil:

(i) The soil used shall be a silty sand meeting the grading and PI requirements as shown in table 6.

(ii) *pH and organic content:* The pH of the soil shall be ≥ 5,5 and ≤ 9,5 and the organic content of the soil shall be ≤ 2,0 %.

(iii) *Blended soil:*
In the case of a sandy clayey silt or a sandy silty clay, with more than 30 % passing the 0,075 mm sieve, the soil shall be blended with silica sand pass ing the 2,36 mm sieve and retained on the 0,150 mm sieve, in order to produce a blended soil which con forms with the requirements of sub-clause (b)(i) above.

(c) *Water*
Water suitable for use in concrete is used.

Table 6 Grading and P1 requirements

Maximum particle size	9,5 mm
Minimum passing 2,36 mm sieve	80 % by weight
Minimum passing 0,425 mm sieve	40 % by weight
Minimum passing 0,075 mm sieve	10 % by weight
Maximum passing 0,075 mm sieve	30 % by weight
PI	≤8 %

Strength and durability

(a) Symbol: UCS refers to the unconfined compressive strength of 150 mm cubes of the cement-stabilised soil compacted to 100 % Standard Proctor maximum dry density at optimum moisture content (OMC) for the cement-soil mix and cured between hessian sacks which are maintained wet and kept in the shade.

(b) *Locations where no frost, no freezing, no snow and no sea water occur*
Minimum 7 day UCS ≥ 2,7 MPa
Minimum 28 day UCS ≥ 4,0 MPa

(c) *Locations where frost, or freezing, or snow, or sea water occurs*

Minimum 7 day UCS ≥ 4,0 MPa
Minimum 28 day UCS ≥ 6,0 MPa

Revetment dimensions

(a) Symbols: Mean slope of upstream face of revetment = 1v : Gh

α = Angle of upstream face to the horizontal
Th = Compacted thickness of revetment perpendicular to the line of the upstream face
W = Compacted horizontal width of revetment
CW = Construction width of a horizontal layer of the revetment, as widened to provide for the under-compaction of the upstream edge of the layer,
CLT = Construction layer thickness measured vertically after compaction,
LLT = Construction loose layer thickness after spreading, measured vertically prior to compaction

(b) Formulae:

$$\tan \alpha = \frac{1}{G} \tag{36}$$

$$W = \frac{Th}{\sin \alpha} \tag{37}$$

$$LLT = \frac{4(CLT)}{3}, \text{ approximately} \tag{38}$$

(c) Criteria:

Minimum Th = 450 mm

Maximum CLT = 150 mm for W < 2,5 m

Minimum CW = W + 200 mm, for CLT = 150 mm

LLT = 200 mm, approximately, for CLT = 150 mm

Mix proportions and mixing cement stabilised soil

- The ingredients shall be weight batched and mixed in a concrete mixer as follows:
  Case of unblended silty sand soil:
  Add soil and cement to mixer and mix for 30 seconds
  Add required water to mixer and mix for 30 seconds
  Case of silty clayey soil blended with sand:
  Add sand and soil to mixer and mix for 30 seconds
  Add cement to mixer and mix for 30 seconds
  Add required water to mixer and mix for 30 seconds
- The amount of cement in the mix shall be sufficient to meet the strength requirements, as applicable. If the soil or blended soil meets the grading requirements, then 10 % by weight of cement relative to the dry weight of the soil or blended soil should be sufficient to meet the strength requirements for no frost.
- The total amount of water in the mix shall be between OMC and OMC + 3 % where:

OMC = optimum moisture content of the cement plus soil mix (or cement plus blended soil mix) for maximum dry density when compacted to Standard Proctor compaction.

The moisture content (MC) of the soil (or blended soil) shall be measured and the balance of the water to be added to the mixer shall be calculated.

Surface preparation and rate of construction

- Just prior to spreading a new layer of cement-stabilised soil, the in-situ surface of the previous layer shall be swept clean and the surface moistened with a fine spray of water as necessary to keep the surface moist. Washing out of material from the surface shall be strictly avoided.
- Cement-stabilised soil wave revetments shall be constructed with as little time delay between layers as is possible in order to increase bonding between layers. However, the minimum time permitted between the completion of compaction of the underlayer and the commencement of compaction of the overlayer shall be 12 hours. The intention is to minimise the formation of cracks in the underlayer during vibration of the overlayer.

Symbols (alphabetical order)

The following symbols are used in this paper:

A_e	=	Water surface area along fetch strip in km^2
B	=	Width of a flow channel
C	=	Wave celerity in m/s
C_s	=	Wind set-up factor
D_c	=	Thickness of concrete-filled Hyson Cell mattress
D^c_{15}	=	Particle size of coarse material layer relative to which 15 % of the particles are smaller
D^f_{85}	=	Particle size of fine material layer relative to which 85 % of the particles are smaller
D_{50}	=	Particle size relative to which 50 % of the particles are smaller
D'_{50}	=	Median rock size for rip-rap in m
D''_{50}	=	Median rock size for first underlayer in m
F	=	Fetch of wind along water surface in km
F_e	=	Effective fetch in km, allowing for shore retardation of development of wave heights
FS	=	Factor of safety
G	=	Cot α (revetment slope is 1 vert : G horiz)
g	=	9,81 m/s^2 (acceleration due to gravity)
H	=	Wave height in m = height of crest above trough
H_d	=	Design wave height in m
H_s	=	Significant wave height in m = average height of the highest one-third of the waves in a wave spectrum and is only equalled or exceeded by 13 % of the waves generated by a particular wind speed (SANCOLD Report No 3, 1990)
Irs	=	Iribarren No (or surf similarity parameter or breaker parameter) for significant wave height, in m and sec units
K	=	Significant wave height calculation factor
K_c	=	Modified rip-rap factor
K_d	=	Wave run-down factor
K_f	=	Effective fetch strip width factor (unit : km $^{-0,5}$)
K_r	=	Rip-rap K-factor
K_t	=	Filter size factor
K_u	=	Underlayer rock size factor
L	=	Wave length in m
L_s	=	Significant wave length in m
R_d	=	Wave run-down measured vertically below SWL in m
R_{ds}	=	Significant wave run-down below SWL
$R_d2\%$	=	Run-down of highest 2 % of waves
R_u	=	Wave run-up measured vertically above SWL in m
R_{us}	=	Significant wave run-up above SWL
$R_u2\%$	=	Run-up of highest 2 % of waves = 1,4.R_{us}
S	=	Rise in WL above SWL due to wind set-up, in m
S_r	=	Specific gravity of rock
SWL	=	Still water level
T	=	Wave period in secs
Th	=	Layer thickness
T_s	=	Significant wave period in seconds
V_w	=	Wind speed over water in m/s at 10 m elevation
V^r_{50}	=	Median rock volume m^3
W_f	=	Effective fetch strip width in km
W^r_{50}	=	Median rock weight
WL	=	Water level
y	=	Water depth below SWL
Y_a	=	Average water depth in m along fetch
α	=	Slope of embankment face to the horizontal in degrees, where slope is 1 : G
γ_r	=	Specific weight of rock
γ_w	=	Specific weight of water
ρc	=	Mass density of concrete
ρw	=	Mass density of water = γ_w /g

Compaction after mixing and spreading

Compaction of cement-stabilised soil preferably should commence within one hour of water being added to the mixer.

The cement-stabilised layer shall be compacted to 100 % Standard Proctor maximum dry density. The construction layer thickness (CLT) after compaction shall be a maximum of 150 mm + 20 mm oversize tolerance.

The stabilised layer shall be compacted against the in-situ compacted unstabilised soil on the downstream side of the wave revetment.

Curing

A compacted cement-stabilised soil layer shall be cured for at least seven days or until it is covered by the next layer. The surface of the layer shall be kept damp by means of a fine spray of water at intervals as required to keep the surface damp. On hot drying days, the surface should be covered with wet hessian. Washing out of material by a strong water spray shall be avoided.

Freezing of the surface shall be prevented, as necessary, by covering the surface with a 150 mm thick earth blanket for seven days.

Stepped upstream face

The stepped under-compacted upstream face should be trimmed by a spade at 34° to 45° to the vertical at the top outer edge of the layer immediately after completion of compaction, so as to produce horizontal steps at 150 mm vertical intervals. The stepped face impedes wave run-up.

Construction limitations

- No cement-stabilised soil construction work shall be carried out during wet weather or when the air temperature is less than 7 °C.
- The minimum time lapse between completion of compaction of an underlayer and commencement of compaction of an overlayer shall be 12 hours.
- Compaction of a layer shall be completed (after spreading the layer) within a time lapse of four hours reckoned from the time that water was added to the cement-stabilised soil in the mixer.

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