6. DISEÑO DE PROTECCIÓN DE TALUDES

6.1. REFERENCIAS

Para la elaboración del presente, han sido tomados como referencia los siguientes estudios:

- Farias, H. D., Pilán, M. T., Pece, F. J., & Olmos, L. A. (2003, November). Erosión General en Ríos con Lechos Arenosos. In Nuevas Tendencias en Hidráulica de Ríos. Primer Simposio Regional sobre Hidráulica de Ríos, Buenos Aires, Argentina.
- Burger, AM, Bezuijen, A., Pilarczyk, KW y Breteler, MK (1991). Verificación y uso práctico del método de diseño de revestimiento de bloques. Coastal Engineering 1990 (págs. 1431-1445).
- Pilarczyk, K. W. (1998). Dikes and revetments, Published by A. Balkema, Rotterdam.

6.2. CRITERIOS DE DISEÑO

El sistema de protección previsto se conoce como Colchones de Concreto Encapsulado (CCE), el cual consiste básicamente en el resultado del bombeo de un mortero de cemento de elevada resistencia y fluidez dentro de un geotextil tejido de alta resistencia con dos capas, que sirve como encofrado para la colocación de la mezcla. Por debajo de esta protección, se coloca un geotextil no tejido de 300 gr/m2 en concepto de filtro entre el suelo fino que conforma el talud y la cara expuesta posterior a la protección.

El mortero de cemento empleado para la inyección de los colchones debe poseer una resistencia característica a la compresión a los 28 días de por lo menos 9,7 MPa en una prueba puntual o alcanzar un promedio de 13,8 MPa en una prueba triple.

El dimensionamiento de las protecciones de taludes conforme a la bibliografía de referencia, viene dado principalmente por las solicitaciones de oleaje y condiciones de escurrimiento a los que se verá sometido en su vida útil. En el caso de esta obra, no existen condiciones de oleaje, por lo que la verificación deberá hacerse estrictamente a condiciones de escurrimiento.

Considerando el alcance máximo de las excavadoras s/ orugas a emplearse (CAT 320 o similar), se dispone una berma que servirá para lograr la conformación y protección de la parte inferior del talud. En tal sentido, se realiza la protección en dos tramos, la parte inferior (desde la berma hasta el pie) y en segunda instancia la superior, desde allí al coronamiento.

Como se requiere dar inicio inmediato a la protección de taludes, ya que se requiere terminar los trabajos previo a la próxima avenida, se prediseño la protección inferior con CCE de 15cm de espesor medio, por poseer stock disponible en el país, siendo, mientras que la parte de la protección superior, a ser ejecutada en segunda instancia ha sido prediseñada con CCE de 10cm. Para contemplar fenómenos de erosión al pie, se dispone una longitud horizontal de 4m a partir del pie del talud, de manera tal que, ante una eventual erosión localizada al pie del talud, la protección pueda descender y acomodarse a la nueva configuración protegiendo el talud bajo esta nueva posible configuración. Para cumplir este efecto, se incorporan a los CCE cables de acero en el interior de la manta de manera tal que admitan deformación manteniéndose vinculados según lo previsto.

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CARLOS MUÑOZ ARMAS
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Se dispone a continuación la verificación de los espesores y de la extensión necesaria al pie.

6.3. PROFUNDIDAD DE SOCAVACIÓN

Para los cálculos de erosión se ha trabajado sobre estimaciones basadas en las recomendaciones de Pilarczyk (1991 y 1998) y Farias et al. (2003).

1. El factor de limo, regido por la expresión ($f = 1{,}76 \sqrt{d_{50}}$) queda establecido en $f = 0{,}682$ a partir de un diámetro característico ($d_{50} = 0{,}15$ mm).

2. El caudal especifico, regido por la relación entre caudal y ancho de cauce (Q/W) queda establecido en $Q_e = 2{,}5$ m^2/s, a partir de un ancho de cauce medido de 200m y un caudal adoptado para el cálculo de 500 m3/s.

3. La profundidad de socavación media, se rige por la expresión $Ds = 1{,}34\ (Qe^2 / f)^{1/3}$, quedando establecida en un valor de 2,8m.

4. Para garantizar que la protección articulada se flexibilice y proteja también dicha superficie, es necesario que proteja el pie en un talud estable geotécnicamente (1H:1V). Esta extensión se determina a partir de la descomposición del seno del ángulo requerido (45°). Por lo que finalmente se requieren 3,97m de extensión al pie.

5. Como las ecuaciones de erosión al pie involucran régimen permanente, es decir que, ante una eventual socavación, el aumento de la profundidad produciría una disminución de la velocidad de fondo en el sitio, no retroalimentando el fenómeno, se considera que con 4m de extensión al pie se cumplen buenas condiciones.

En conclusión, se recomienda adoptar una longitud de 4m al pie.

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6.4. ESPESOR DE LA MANTA

El espesor de la manta, es verificado de acuerdo la relación de Pilarczyk, debido a que se conoce con cierta razonabilidad la velocidad crítica. Dicha relación se rige por la siguiente expresión:

$$D = 0{,}035\ \frac{\Phi}{\Psi}\frac{K_T K_H}{K_s}\frac{u_{cr}^2}{2g}\frac{1}{\Delta}$$

Donde:

D — Es el espesor medio mínimo requerido para la manta de CCE.

Φ — Es el parámetro de estabilidad, que para pendientes de CCE toma valor unitario (0,75).

Ψ — Es el parámetro dependiente del tipo de material de la protección, donde para CCE toma un valor de 0,07.

K_T — Es el factor de turbulencia, donde para ríos en curva toma valores de 1,5.

K_H — Es el factor dependiente de la profundidad, regido por la expresión $2[\log(12h/k_s)]^{-2}$. Donde h es la profundidad y h_s es la rugosidad equivalente de Nikuradse, que toma un valor de 0,05m para los CCE. Para ambos casos, tanto en la parte inferior como en la parte superior del talud este coeficiente toma el valores próximos a 0,5.

K_S — Es el factor del talud, dependiente del coseno del angulo perpendicular al flujo, que para el caso que nos ocupa, por los taludes 3H:1V, el coeficiente resulta de 0,94.

u_{cr} — Es la velocidad de escurrimiento crítica, que como la velocidad de equilibrio es de 2,5m/s, por tratarse de una barranca en proceso de erosión activa, resulta conservador verificar los espesores con velocidades 10% superiores como 2,8m/s.

g — Es la aceleración de la gravedad (9,81m/s).

Δ — Es la densidad relativa, con un valor de 1,25.

Finalmente, resolviendo la expresión anterior, se tiene un espesor medio mínimo de 9,5cm.

Si bien se requiere como mínimo un espesor de 9,5cm, a fines de prevenir el impacto de los "palos bobos" y los demás elementos arrastrados por el río Pilcomayo, se recomienda disponer CCE con espesores medios garantizados por el fabricante de por lo menos 15cm en todo el talud. En todos los casos, los CCE deberán ser articulados en el sentido longitudinal y permeables.

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ANEXO TOPOGRÁFICO

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TABLA N° 3: Planilla de nube de puntos.

ID.	ESTE (m)	NORTE (m)	ELEV. (m)	DESCRIPCION
1	548.890,005	7.526.019,014	277,690	FONDO
2	548.859,983	7.525.967,628	277,765	FONDO
3	548.854,900	7.525.963,151	277,981	FONDO
4	548.912,921	7.525.969,762	277,190	FONDO
5	548.882,898	7.525.947,434	277,331	FONDO
6	548.864,253	7.525.936,475	277,570	FONDO
7	548.898,681	7.525.926,041	276,893	FONDO
8	548.919,001	7.525.937,975	276,957	FONDO
9	548.876,418	7.525.910,998	277,261	FONDO
10	548.874,587	7.525.891,918	277,110	FONDO
11	548.909,712	7.525.890,821	277,030	FONDO
12	548.869,431	7.525.856,966	277,172	FONDO
13	548.913,671	7.525.862,930	277,090	FONDO
14	548.866,623	7.525.822,234	277,080	FONDO
15	548.911,213	7.525.825,041	277,042	FONDO
16	548.864,290	7.525.793,494	276,983	FONDO
17	548.909,244	7.525.796,302	276,904	FONDO
18	548.937,018	7.526.040,439	276,530	FONDO
19	548.941,493	7.526.017,828	276,280	FONDO
20	548.944,509	7.525.993,001	276,127	FONDO
21	548.945,614	7.525.974,506	276,041	FONDO
22	548.949,012	7.525.948,815	276,260	FONDO
23	548.969,284	7.525.949,008	286,215	BASE
24	548.949,364	7.525.815,906	282,091	TN
25	548.951,409	7.525.827,388	282,977	TN
26	548.959,746	7.525.861,679	284,848	TN
27	548.960,690	7.525.877,408	285,394	TN
28	548.957,954	7.525.914,691	286,015	TN
29	548.956,836	7.525.933,707	286,180	TN
30	548.956,067	7.525.943,004	286,200	TN
31	548.955,717	7.525.951,184	286,176	TN
32	548.954,319	7.525.958,245	286,065	TN
33	548.952,807	7.525.974,734	285,897	TN
34	548.949,268	7.525.997,385	285,371	TN
35	548.946,909	7.526.022,631	284,793	TN
36	548.940,774	7.526.043,629	283,695	TN
37	548.932,251	7.526.055,191	281,352	TN
38	548.899,691	7.526.087,397	281,292	TN
39	548.918,094	7.526.075,364	281,241	TN
40	548.906,415	7.526.117,126	281,208	TN
41	548.936,498	7.526.112,879	281,654	TN
42	548.955,609	7.526.086,335	282,034	TN

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TABLA N° 3: Planilla de nube de puntos.

ID.	ESTE (m)	NORTE (m)	ELEV. (m)	DESCRIPCION
43	549.014,654	7.526.189,059	282,059	TN
44	548.949,691	7.525.748,279	282,169	TN
45	548.966,207	7.525.760,862	282,296	TN
46	548.975,645	7.525.770,457	282,996	TN
47	548.990,430	7.525.779,738	285,387	TN
48	549.011,764	7.525.798,200	285,353	TN
49	549.024,976	7.525.813,536	282,936	TN
50	549.037,009	7.525.823,210	281,579	TN
51	549.060,722	7.525.924,960	281,829	TN
52	549.032,409	7.525.940,297	281,956	TN
53	549.013,297	7.525.953,273	282,357	TN
54	548.978,850	7.525.959,408	286,011	TN
55	549.020,513	7.526.229,139	281,751	TN
56	549.030,423	7.526.223,791	281,853	TN
57	549.048,669	7.526.214,668	284,617	TN
58	549.068,960	7.526.205,702	284,331	TN
59	549.083,431	7.526.200,197	282,261	TN
60	549.094,756	7.526.194,534	281,814	TN
61	549.009,778	7.526.193,748	281,562	TN
62	549.030,226	7.526.181,322	286,040	TN
63	549.049,416	7.526.173,929	285,953	TN
64	549.063,258	7.526.167,165	284,175	TN
65	549.073,325	7.526.159,615	281,972	TN
66	548.978,004	7.526.138,577	281,444	TN
67	548.983,903	7.526.133,386	282,090	TN
68	548.999,003	7.526.126,072	286,434	TN
69	549.017,878	7.526.118,286	286,377	TN
70	549.033,451	7.526.112,623	284,173	TN
71	549.048,315	7.526.105,309	282,085	TN
72	548.959,365	7.526.085,018	282,396	TN
73	548.977,532	7.526.075,108	286,445	TN
74	548.997,351	7.526.070,625	286,567	TN
75	549.015,755	7.526.066,142	284,178	TN
76	549.035,810	7.526.065,906	282,148	TN
77	548.966,207	7.526.039,009	286,438	TN
78	548.986,970	7.526.033,818	286,499	TN
79	549.007,733	7.526.029,807	283,991	TN
80	549.025,429	7.526.025,560	282,285	TN
81	548.958,421	7.525.994,121	286,396	TN
82	548.979,892	7.525.993,413	286,231	TN
83	549.000,891	7.525.991,290	283,975	TN
84	549.018,350	7.525.987,043	281,933	TN

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TABLA N° 3: Planilla de nube de puntos.

ID.	ESTE (m)	NORTE (m)	ELEV. (m)	DESCRIPCION
85	548.996,762	7.525.959,201	284,156	TN
86	548.980,501	7.525.915,670	285,892	TN
87	548.996,073	7.525.914,412	284,710	TN
88	549.013,533	7.525.914,254	281,740	TN
89	548.984,434	7.525.880,121	285,748	TN
90	549.002,051	7.525.882,638	284,155	TN
91	549.015,106	7.525.880,908	281,919	TN
92	548.970,316	7.525.843,668	285,760	TN
93	548.993,439	7.525.847,679	285,697	TN
94	549.008,775	7.525.849,095	283,741	TN
95	549.020,336	7.525.848,859	281,931	TN
96	548.964,300	7.525.786,334	282,273	TN
97	548.983,175	7.525.798,367	285,484	TN
98	549.004,882	7.525.812,524	285,515	TN
99	549.023,285	7.525.826,444	282,288	TN
100	549.014,556	7.525.821,726	284,054	TN
101	548.998,747	7.525.725,461	282,283	TN
102	549.011,960	7.525.736,078	285,221	TN
103	549.031,307	7.525.747,168	285,282	TN
104	549.045,464	7.525.751,651	284,119	TN
105	549.056,317	7.525.757,549	282,020	TN
106	549.034,021	7.525.699,507	282,382	TN
107	549.041,571	7.525.707,294	284,336	TN
108	549.053,368	7.525.719,091	284,499	TN
109	549.061,154	7.525.723,574	283,925	TN
110	549.072,951	7.525.729,708	282,081	TN
111	548.969,484	7.525.922,681	286,033	DRON
112	548.957,439	7.525.922,561	286,064	DRON
113	548.989,281	7.525.818,569	285,589	DRON
114	548.981,042	7.526.040,475	286,348	DRON
115	549.013,209	7.526.121,026	286,245	DRON
116	549.077,694	7.526.186,883	283,314	DRON
117	549.050,520	7.526.198,117	285,474	DRON
118	549.032,482	7.526.204,833	283,869	DRON
119	549.031,907	7.526.107,844	284,118	DRON
120	549.010,803	7.526.031,078	283,570	DRON
121	548.954,609	7.526.046,711	284,878	DRON
122	549.088,026	7.526.183,629	281,658	TN
123	549.073,294	7.526.190,930	283,942	TN
124	549.062,902	7.526.193,954	285,205	TN
125	549.049,839	7.526.197,648	285,478	TN
126	549.041,187	7.526.201,816	285,074	TN

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TABLA N° 3: Planilla de nube de puntos.

ID.	ESTE (m)	NORTE (m)	ELEV. (m)	DESCRIPCION
127	549.033,091	7.526.206,581	283,764	TN
128	549.022,475	7.526.212,764	281,891	TN
129	548.946,282	7.526.049,722	284,077	TN
130	548.957,505	7.526.045,440	285,145	TN
131	548.967,368	7.526.041,792	286,336	TN
132	548.977,026	7.526.040,631	286,341	TN
133	548.925,373	7.526.057,318	276,974	FONDO
134	548.919,228	7.526.051,875	277,300	FONDO
135	548.908,410	7.526.040,525	277,615	FONDO
136	548.916,705	7.526.035,384	277,606	FONDO
137	548.928,260	7.526.040,047	276,881	FONDO
138	548.928,276	7.526.040,013	276,876	FONDO
139	548.932,237	7.526.011,404	276,749	FONDO
140	548.924,463	7.526.007,736	277,643	FONDO
141	548.931,695	7.525.984,616	276,515	FONDO
142	548.931,695	7.525.984,631	276,507	FONDO
143	548.932,069	7.525.978,292	276,447	FONDO
144	548.932,458	7.525.963,315	276,555	FONDO
145	548.936,549	7.525.943,311	276,642	FONDO
146	548.944,922	7.525.918,560	277,040	FONDO
147	548.950,187	7.525.918,595	276,935	FONDO
148	548.951,973	7.525.901,823	277,102	FONDO
149	548.940,875	7.525.894,392	277,248	FONDO
150	548.940,968	7.525.872,796	277,156	FONDO
151	548.954,137	7.525.876,268	277,120	FONDO
152	548.953,213	7.525.858,650	276,939	FONDO
153	548.941,059	7.525.857,204	277,184	FONDO
154	548.933,941	7.525.830,091	276,918	FONDO
155	548.945,633	7.525.827,735	276,530	FONDO
156	548.935,748	7.525.807,652	276,509	FONDO
157	548.942,377	7.525.808,233	276,478	FONDO
158	548.958,870	7.525.809,012	283,333	TN
159	548.967,852	7.525.811,267	284,366	TN
160	548.977,286	7.525.814,673	285,625	TN
161	548.989,564	7.525.817,331	285,601	TN
162	549.002,091	7.525.819,925	285,547	TN
163	549.009,406	7.525.821,796	284,649	TN
164	549.022,292	7.525.824,991	282,943	TN
165	549.024,261	7.525.825,262	282,267	TN
166	549.062,287	7.525.742,990	282,538	TN
167	549.060,881	7.525.742,584	283,172	TN
168	549.052,551	7.525.738,853	284,146	TN

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TABLA N° 3: Planilla de nube de puntos.

ID.	ESTE (m)	NORTE (m)	ELEV. (m)	DESCRIPCION
169	549.042,195	7.525.732,706	284,983	TN
170	549.032,513	7.525.727,201	285,002	TN
171	549.024,264	7.525.721,967	284,772	TN
172	549.018,375	7.525.717,420	283,894	TN
173	549.007,105	7.525.709,865	282,635	TN
174	548.956,932	7.525.937,170	286,117	TN
175	548.978,219	7.525.937,930	285,897	TN
176	548.993,024	7.525.936,572	284,763	TN
177	549.013,147	7.525.937,565	281,913	TN
178	549.016,074	7.525.963,529	281,865	TN
179	549.012,200	7.525.964,011	282,602	TN
180	549.000,728	7.525.967,055	283,593	TN
181	548.979,119	7.525.972,892	285,861	TN
182	548.952,059	7.525.975,996	285,722	TN
183	548.968,741	7.525.943,516	285,618	TN

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ANEXO GEOTECNICO

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FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

FR & ASOCIADOS	OBRA: PROYECTO PROTECCION DE COSTA	SONDEO P1	RESPONSABLE: ING. FELIPE RAMIREZ	HOJA: 1
	UBICACIÓN: POZO HONDO - DEPTO. BOQUERON	Cota de Boca (m) +286,34	Profundidad de sondeo (m): 10,45 Nivel Freatico (m): -	

ENSAYO DE CAMPO / ENSAYOS DE LABORATORIO

PROF (m)	COTA (m)	N F	NUMERO DE GOLPES (0 10 20 30 40 50)	N	NB	WN (%)	LIMITES DE ATTERBERG L.L. (%)	L.P. (%)	I.P. (%)	GRANULOMETRIA #4	#10	#40	#200	PROF (m)	C.U.S.	DESCRIPCION
1																
				13												
2																
				15												
3																
				3												
4																
	281,84			22			38,1	19,5	18,6	100	100	99	88	4,50	CL	Arcilla limosa
5																
				25												
6																
				11												
7																
				11												
8																
				8		--										
9																
	276,89			13			38,2	19,7	18,5	100	100	100	90	9,45	CL	Arcilla limosa
10																
	275,89			16			37,2	19,4	17,7	100	100	98	89	10,45	CL	Arcilla limosa
11																
12																
13																
14																
15																
16																
17																
18																

Coordenadas UTM 548975 m E - 7526048 m N

Fecha de Inicio de sondeo 27-07-2021
Fecha de finalización de sondeo 27-07-2021

N F nivel freatico N numero de golpes SPT
n.p. napa potente NB Indice de Bosio
n.d. napa debil

CARLOS MUÑOZ ARMAS
Representante Legal

Sara Blanco
Representante Legal

CONSORCIO CANALES DEL CHACO
RUC: 80115...

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

FR & ASOCIADOS	OBRA: POZO HONDO	SONDEO P2	RESPONSABLE: ING. FELIPE RAMIREZ	HOJA: 1
	UBICACIÓN:BOQUERON	Cota de Boca (m) +286,06	Profundidad de sondeo (m): 10,45 Nivel Freatico (m):	ESTUDIO

ENSAYO DE CAMPO						ENSAYOS DE LABORATORIO										
PROF. (m)	COTA (m)	N.F.	NUMERO DE GOLPES 0 10 20 30 40 50	N	NB	WN (%)	LIMITES DE ATTERBERG L.L. (%)	L.P.(%)	I.P.(%)	GRANULOMETRIA #4	#10	#40	#200	PROF. (m)	C.U.S	DESCRIPCION
1				11												
2	283,61			15			38,7	20,2	18,6	100	100	100	91	2,45	CL	Arcilla limosa
3				7												
4				8												
5				9												
6	280,06			7			37,4	19,6	17,9	100	100	98	89	6,00	CL	Arcilla limosa
7				10												
8				12		--										
9				15												
10	275,61			21			37,7	19,7	18,0	100	100	99	90	10,45	CL	Arcilla limosa
11																
12																
13																
14																
15																
16																
17																
18																

Coordenadas UTM: 548959 m E - 7525928 m N

Fecha de Inicio de sondeo: 28-08-2021
Fecha de finalización de sondeo: 28-07-2021

N.F.: nivel freatico
n.p.: napa potente
n.d.: napa debil

N: numero de golpes SPT
NB: Indice de Bosio

Sara Blanco
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CARLOS MUÑOZ ARMAS
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CONSORCIO CANALES DE [illegible]
RUC: 801160[illegible]

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTECNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

FR & ASOCIADOS	OBRA: POZO HONDO	SONDEO P3	RESPONSABLE: ING. FELIPE RAMIREZ	HOJA: 1
	UBICACIÓN:BOQUERON	Cota de Boca (m) +285,89	Profundidad de sondeo (m): 10,45 Nivel Freatico (m):	ESTUDIO

ENSAYO DE CAMPO | **ENSAYOS DE LABORATORIO**

PROF. (m)	COTA (m)	N.F.	NUMERO DE GOLPES 0 10 20 30 40 50	N	NB	WN (%)	LIMITES DE ATTERBERG L.L. (%)	L.P.(%)	I.P.(%)	GRANULOMETRIA #4	#10	#40	#200	PROF. (m)	C.U.S	DESCRIPCION
1				9												
2				15												
3	282,44			11			38,0	19,4	18,7	100,0	100,0	100,0	92,0	3,45	CL	Arcilla limosa
4				7												
5				10												
6	279,44			7			37,5	20,5	17,0	100,0	100,0	98,0	90,0	6,45	CL	Arcilla limosa
7				9												
8				13		–										
9				19												
10	275,44			16			37,2	20,3	16,9	100,0	100,0	98,0	89,0	10,45	CL	Arcilla limosa
11																
12																
13																
14																
15																
16																
17																
18																

Coordenadas UTM: 548986 m E - 7525929 S m N

Fecha de Inicio de sondeo: 28-08-2021
Fecha de finalización de sondeo: 28-07-2021

N.F.: nivel freatico
n.p.: napa potente
n.d.: napa debil

N: numero de golpes SPT
NB: Indice de Bosio

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CONSORCIO CANALES DEL CHACO
RUC: 80116057-5

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

FR & ASOCIADOS INGENIERÍA CIVIL Y GEOTÉCNICA	OBRA: POZO HONDO	SONDEO P4	RESPONSABLE: ING. FELIPE RAMIREZ	HOJA: 1
	UBICACIÓN:BOQUERON	Cota de Boca (m) +281,91	Profundidad de sondeo (m): 10,45 Nivel Freatico (m):	ESTUDIO

ENSAYO DE CAMPO / ENSAYOS DE LABORATORIO

PROF. (m)	COTA (m)	N.F.	NUMERO DE GOLPES 0 10 20 30 40 50	N	NB	WN (%)	LIMITES DE ATTERBERG L.L. (%)	L.P. (%)	I.P. (%)	GRANULOMETRIA #4	#10	#40	#200	PROF. (m)	C.U.S	DESCRIPCION
1																
				6												
2																
				8												
3																
				9												
4																
	277,41			7			37,2	20,5	16,7	100,0	100,0	98,0	88,0	4,50	CL	Arcilla limosa
5																
				11												
6																
				11												
7																
				13												
8																
	272,46			14		–	38,2	20,5	17,7	100,0	100,0	100,0	92,0	9,45	CL	Arcilla limosa
9																
				15												
10																
	271,46			17			37,7	20,1	17,5	100,0	100,0	99,0	89,0	10,45	CL	Arcilla limosa
11																
12																
13																
14																
15																
16																
17																
18																

Coordenadas UTM: 549015 m E - 7525931 S m N

Fecha de inicio de sondeo: 28-08-2021
Fecha de finalización de sondeo: 28-07-2021

N.F.: nivel freatico
n.p.: napa potente
n.d.: napa debil

N: numero de golpes SPT
NB: Indice de Bosio

CARLOS MUÑOZ ARMAS
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Sara Blanco
Representante Legal

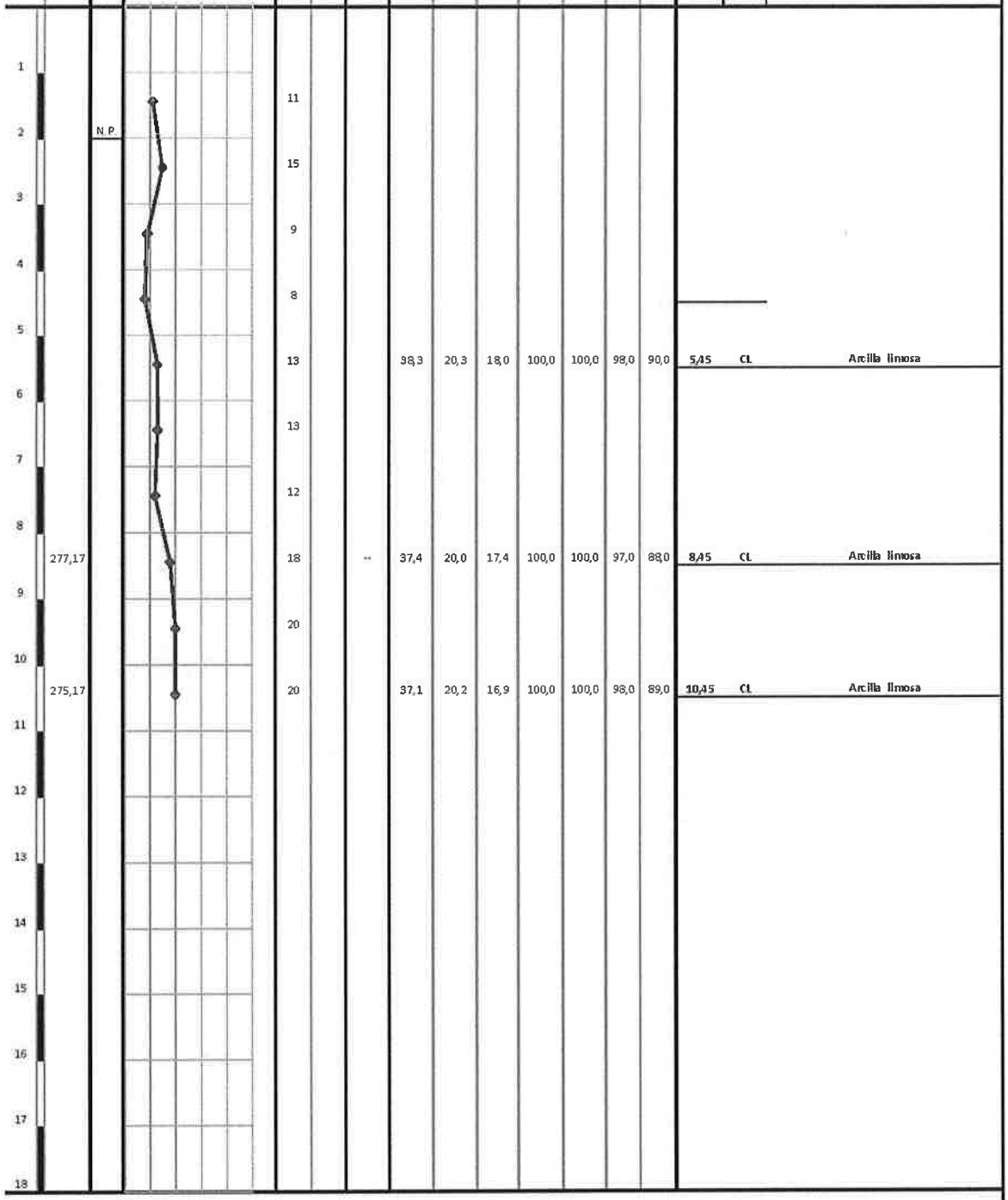
CONSORCIO CANALES DEL CHACO
RUC: 80118057-0

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

FR & ASOCIADOS	OBRA: POZO HONDO	SONDEO: P5		RESPONSABLE: ING. FELIPE RAMIREZ	HOJA: 1
	UBICACIÓN: BOQUERON	Cota de Boca (m): +285,62		Profundidad de sondeo (m): 10,45 / Nivel Freatico (m):	ESTUDIO



PROF (m)	N	NB	W.N (%)	L.L. (%)	L.P (%)	I.P (%)	#4	#10	#40	#200	PROF (m)	C.U.S	DESCRIPCION
1	11												
2	15												
3	9												
4	8												
5	13			38,3	20,3	18,0	100,0	100,0	98,0	90,0	5,45	CL	Arcilla limosa
6	13												
7	12												
8	18		--	37,4	20,0	17,4	100,0	100,0	97,0	88,0	8,45	CL	Arcilla limosa
9	20												
10	20			37,1	20,2	16,9	100,0	100,0	98,0	89,0	10,45	CL	Arcilla limosa
11													
12													
13													
14													
15													
16													
17													
18													

Coordenadas UTM: 548978 m E - 7525815 S m N

Fecha de Inicio de sondeo: 27-07-2021
Fecha de finalización de sondeo: 27-07-2021

N.F.: nivel freatico
n.p.: napa potente
n.d.: napa debil

N: numero de golpes SPT
NB: Indice de Bosio

CARLOS MUÑOZ ARMAS
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Sara Blanco
Representante Legal

CONSORCIO CANALES DEL CHACO
RUC: 80116067-0

LABORATORIO DE SUELOS	INFORME DE ENSAYO LIMITES DE ATTERBERG
UTM: 548975 7526048	FECHA: 02/08/2021
OBRA: POZO HONDO.	CAPA:
TRAMO: P 1	MUESTRA:

MÉTODO: AASHTO T89-68 LÍMITE LÍQUIDO

	Profundidad (mts)			1,00	4,45	4,45	9,45	9,45	10,45	
1	Cápsula.		N°	1	2	3	4	5	6	
2	Golpes.		N°	20	25	25	25	22	26	
3	Peso Cáps. + Suelo Húmedo.		gr.	39,55	40,01	40,05	40,35	40,02	40,52	
4	Peso Cáps. + Suelo Seco.		gr.	32,25	33,43	33,8	33,81	33,92	34	
5	Peso Agua.	(3 - 4)	gr.	7,3	6,58	6,25	6,54	6,1	6,52	
6	Peso Cápsula.		gr.	13,36	16,36	17,66	16,47	17,66	16,47	
7	Peso Suelo Seco.	(4 - 6)	gr.	18,89	17,07	16,14	17,34	16,26	17,53	
8	HUMEDAD	5/7 * 100	%	38,6	38,5	38,7	37,7	37,5	37,2	
9	Factor de Corrección			1,029	1,000	1,000	1,000	1,016	0,995	
10	Humedad Correguida		%	37,6	38,6	38,7	37,7	36,9	37,4	
11	**Humedad Promedio**		%	**38,1**		**38,2**		**37,2**		

MÉTODO: AASHTO T90-70 LÍMITE PLÁSTICO

1	Cápsula.		N°	7	8	9	10	11	12
2	Peso Cáps.+ Suelo Húmedo.		gr.	20,14	21,08	19,56	21,33	20,14	21,08
3	Peso Cáps. + Suelo Seco.		gr.	18,35	19,30	17,95	16,80	18,5	16,65
4	Peso Agua.	(2 - 3)	gr.	1,79	1,78	1,61	1,30	1,64	1,30
5	Peso Cápsula.		gr.	9,36	9,96	10,03	9,99	10,03	9,99
6	Peso Suelo Seco.	(3 - 5)	gr.	8,99	9,34	7,92	6,81	8,47	6,66
7	HUMEDAD	4/6 * 100	%	19,9	19,1	20,3	19,1	19,4	19,5
9	**Humedad Promedio**		%	**19,5**		**19,7**		**19,4**	

RESULTADOS:					
LL = %	38,1	38,2	37,2		
LP = %	19,5	19,7	19,4		
IP = %	18,6	18,5	17,7		
GRANULOMETRÍA:					
Pas. # 4: %	100	100	100		
Pas. # 10: %	100	100	100		
Pas. # 40: %	99	100	98		
Pas. # 200: %	88	90	89		
Clasificacion Norma AASHTO	A - 6	A - 6	A - 6		
Indice de Grupo	16	17	16		
Caracterizacion del material	Arcilla limo	Arcilla limo	Arcilla limo		

Lab. Carlos Ortiz

CARLOS MUÑOZ ARMAS
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RUC 20601166657-0

Sara Blanco
Representante Legal

ING. FELIPE RAMIREZ

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

LABORATORIO DE SUELOS	INFORME DE ENSAYO LIMITES DE ATTERBERG
UTM: 548975 7526048	FECHA: 02/08/2021
OBRA: POZO HONDO.	CAPA:
TRAMO: P 2	MUESTRA:

MÉTODO: AASHTO T89-68 LÍMITE LÍQUIDO

	Profundidad (mts)			1,00	2,45	2,45	6,00	6,00	10,45
1	Cápsula.		N°	13	14	15	16	17	18
2	Golpes.		N°	21	26	21	27	25	25
3	Peso Cáps. + Suelo Húmedo.		gr.	41,05	40,9	39,75	39,97	39,90	40,09
4	Peso Cáps. + Suelo Seco.		gr.	33,2	34,07	33,7	33,55	33,8	33,65
5	Peso Agua.	(3 - 4)	gr.	7,85	6,83	6,05	6,42	6,1	6,44
6	Peso Cápsula.		gr.	13,36	16,36	17,66	16,47	17,66	16,47
7	Peso Suelo Seco.	(4 - 6)	gr.	19,84	17,71	16,04	17,08	16,14	17,18
8	HUMEDAD	5/7 * 100	%	39,6	38,6	37,7	37,6	37,8	37,5
9	Factor de Corrección			1,022	0,995	1,022	0,990	1,000	1,000
10	Humedad Correguida		%	38,7	38,8	36,9	38,0	37,8	37,5
11	**Humedad Promedio**		%	**38,7**		**37,4**		**37,7**	

MÉTODO: AASHTO T90-70 LÍMITE PLÁSTICO

1	Cápsula.		N°	19	20	21	22	23	24
2	Peso Cáps.+ Suelo Húmedo.		gr.	21,10	20,01	19,55	21,03	19,35	20,14
3	Peso Cáps. + Suelo Seco.		gr.	19,1	18,35	18	16,60	17,85	16,45
4	Peso Agua.	(2 - 3)	gr.	2,00	1,66	1,55	1,30	1,50	1,30
5	Peso Cápsula.		gr.	9,36	9,96	10,03	9,99	10,03	9,99
6	Peso Suelo Seco.	(3 - 5)	gr.	9,74	8,39	7,97	6,61	7,82	6,46
7	HUMEDAD	4/6 * 100	%	20,5	19,8	19,4	19,7	19,2	20,1
9	**Humedad Promedio**		%	**20,2**		**19,6**		**19,7**	

RESULTADOS:				
LL = %	38,7	37,4	37,7	
LP = %	20,2	19,6	19,7	
IP = %	18,6	17,9	18,0	
GRANULOMETRÍA:				
Pas. # 4: %	100	100	100	
Pas. # 10: %	100	100	100	
Pas. # 40: %	100	98	99	
Pas. # 200: %	91	89	90	
Clasificacion Norma AASHTO	A - 6	A - 6	A - 6	q
Indice de Grupo	17	16	16	
Caracterizacion del material	Arcilla limo	Arcilla limo	Arcilla limo	

Lab. Carlos Ortiz

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CARLOS MUÑOZ ARMAS
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CONSORCIO CANALES DEL CHACO
RUC: 80116057-0

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

LABORATORIO DE SUELOS	INFORME DE ENSAYO LIMITES DE ATTERBERG
UTM: 548986 7525929	FECHA: 03/08/2021
OBRA: POZO HONDO.	CAPA:
TRAMO: P 3	MUESTRA:

MÉTODO: AASHTO T89-68 LÍMITE LÍQUIDO

	Profundidad (mts)			1,00	3,45	3,45	6,45	6,45	10,45	
1	Cápsula.		N°	25	26	27	28	29	30	
2	Golpes.		N°	21	25	25	26	22	26	
3	Peso Cáps. + Suelo Húmedo.		gr.	39,55	41,02	39,86	40,05	39,66	41,11	
4	Peso Cáps. + Suelo Seco.		gr.	32,25	34,2	33,8	33,65	33,62	34,45	
5	Peso Agua.	(3 - 4)	gr.	7,3	6,82	6,06	6,4	6,04	6,66	
6	Peso Cápsula.		gr.	13,36	16,36	17,66	16,47	17,66	16,47	
7	Peso Suelo Seco.	(4 - 6)	gr.	18,89	17,84	16,14	17,18	15,96	17,98	
8	HUMEDAD	5/7 * 100	%	38,6	38,2	37,5	37,3	37,8	37,0	
9	Factor de Corrección			1,022	1,000	1,000	0,995	1,016	0,995	
10	Humedad Correguida		%	37,8	38,2	37,6	37,5	37,2	37,2	
11	**Humedad Promedio**		%	**38,0**		**37,5**		**37,2**		

MÉTODO: AASHTO T90-70 LÍMITE PLÁSTICO

1	Cápsula.		N°	31	32	33	34	35	36		
2	Peso Cáps.+ Suelo Húmedo.		gr.	21,05	21,45	19,66	19,2	20,45	20,04		
3	Peso Cáps. + Suelo Seco.		gr.	19,12	19,62	18	16,45	18,7	16,35		
4	Peso Agua.	(2 - 3)	gr.	1,93	1,83	1,66	1,30	1,75	1,30		
5	Peso Cápsula.		gr.	9,36	9,96	10,03	9,99	10,03	9,99		
6	Peso Suelo Seco.	(3 - 5)	gr.	9,76	9,66	7,97	6,46	8,67	6,36		
7	HUMEDAD	4/6 * 100	%	19,8	18,9	20,8	20,1	20,2	20,4		
9	**Humedad Promedio**		%	**19,4**		**20,5**		**20,3**			

RESULTADOS:				
LL = %	38,0	37,5	37,2	
LP = %	19,4	20,5	20,3	
IP = %	18,7	17,0	16,9	
GRANULOMETRÍA:				
Pas. # 4: %	100	100	100	
Pas. # 10: %	100	100	100	
Pas. # 40: %	100	98	98	
Pas. # 200: %	92	90	89	
Clasificacion Norma AASHTO	A - 6	A - 6	A - 6	
Indice de Grupo	16	15	15	
Caracterizacion del material	Arcilla limo	Arcilla limo	Arcilla limo	

Lab. Carlos Ortiz

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RUC: 20416057-0

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FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

LABORATORIO DE SUELOS	INFORME DE ENSAYO LIMITES DE ATTERBERG
UTM: 549015 7525931	FECHA: 03/08/2021
OBRA: POZO HONDO.	CAPA:
TRAMO: P 4	MUESTRA:

MÉTODO: AASHTO T89-68 LÍMITE LÍQUIDO

	Profundidad (mts)			1,00	4,45	4,45	8,45	8,45	10,45
1	Cápsula.		N°	37	38	39	40	41	42
2	Golpes.		N°	25	25	22	25	21	26
3	Peso Cáps. + Suelo Húmedo.		gr.	40,22	41,62	39,85	39,66	40,08	41,23
4	Peso Cáps. + Suelo Seco.		gr.	32,86	34,85	33,65	33,25	33,85	34,48
5	Peso Agua.	(3 - 4)	gr.	7,36	6,77	6,2	6,41	6,23	6,75
6	Peso Cápsula.		gr.	13,36	16,36	17,66	16,47	17,66	16,47
7	Peso Suelo Seco.	(4 - 6)	gr.	19,50	18,49	15,99	16,78	16,19	18,01
8	HUMEDAD	5/7 * 100	%	37,7	36,6	38,8	38,2	38,5	37,5
9	Factor de Corrección			1,000	1,000	1,016	1,000	1,022	0,995
10	Humedad Correguida		%	37,8	36,6	38,2	38,2	37,6	37,7
11	Humedad Promedio		%	37,2		38,2		37,7	

MÉTODO: AASHTO T90-70 LÍMITE PLÁSTICO

1	Cápsula.		N°	43	44	45	46	47	48
2	Peso Cáps.+ Suelo Húmedo.		gr.	21,07	21,55	20,07	19,55	19,74	20,46
3	Peso Cáps. + Suelo Seco.		gr.	19,1	19,56	18,35	16,40	18,08	16,60
4	Peso Agua.	(2 - 3)	gr.	1,97	1,99	1,72	1,30	1,66	1,30
5	Peso Cápsula.		gr.	9,36	9,96	10,03	9,99	10,03	9,99
6	Peso Suelo Seco.	(3 - 5)	gr.	9,74	9,6	8,32	6,41	8,05	6,61
7	HUMEDAD	4/6 * 100	%	20,2	20,7	20,7	20,3	20,6	19,7
9	Humedad Promedio		%	20,5		20,5		20,1	

RESULTADOS:			
LL = %	37,2	38,2	37,7
LP = %	20,5	20,5	20,1
IP = %	16,7	17,7	17,5
GRANULOMETRÍA:			
Pas. # 4: %	100	100	100
Pas. # 10: %	100	100	100
Pas. # 40: %	98	100	99
Pas. # 200: %	88	92	89
Clasificacion Norma AASHTO	A - 6	A - 6	A - 6
Indice de Grupo	15	16	15
Caracterizacion del material	Arcilla limo	Arcilla limo	Arcilla limo

Lab. Carlos Ortiz

CARLOS MUÑOZ ARMAS
Representante Legal
CONSORCIO
CANALES DEL CHACO
RUC: 80116057

Sara blanco
Representante

ING. FELIPE RAMIREZ

FR & ASOCIADOS
INGENIERÍA CIVIL Y GEOTÉCNICA

INFORME DE INGENIERÍA

PROYECTO DE PROTECCIÓN DE COSTA – POZO HONDO

LABORATORIO DE SUELOS	INFORME DE ENSAYO LIMITES DE ATTERBERG
UTM: 548978 7525815	FECHA: 03/08/2021
OBRA: POZO HONDO.	CAPA:
TRAMO: P 5	MUESTRA:

MÉTODO: AASHTO T89-68 — LÍMITE LÍQUIDO

	Profundidad (mts)			1,00	5,45	5,45	8,45	8,45	10,45
1	Cápsula.		N°	49	50	51	52	53	54
2	Golpes.		N°	22	27	21	26	22	26
3	Peso Cáps. + Suelo Húmedo.		gr.	41,78	38,82	39,12	40,33	41,21	40,56
4	Peso Cáps. + Suelo Seco.		gr.	33,89	32,6	33,2	33,85	34,75	34,07
5	Peso Agua.	(3 - 4)	gr.	7,89	6,22	5,92	6,48	6,46	6,49
6	Peso Cápsula.		gr.	13,36	16,36	17,66	16,47	17,66	16,47
7	Peso Suelo Seco.	(4 - 6)	gr.	20,53	16,24	15,54	17,38	17,09	17,60
8	HUMEDAD	5/7 * 100	%	38,4	38,3	38,1	37,3	37,8	36,9
9	Factor de Corrección			1,016	0,990	1,022	0,995	1,016	0,995
10	Humedad Correguida		%	37,8	38,7	37,3	37,5	37,2	37,1
11	**Humedad Promedio**		%	**38,3**		**37,4**		**37,1**	

MÉTODO: AASHTO T90-70 — LÍMITE PLÁSTICO

1	Cápsula.		N°	55	56	57	58	59	60
2	Peso Cáps.+ Suelo Húmedo.		gr.	20,14	21,35	20,54	19,99	20,41	20,76
3	Peso Cáps. + Suelo Seco.		gr.	18,3	19,45	18,8	16,45	18,7	16,25
4	Peso Agua.	(2 - 3)	gr.	1,84	1,90	1,74	1,30	1,71	1,30
5	Peso Cápsula.		gr.	9,36	9,96	10,03	9,99	10,03	9,99
6	Peso Suelo Seco.	(3 - 5)	gr.	8,94	9,49	8,77	6,46	8,67	6,26
7	HUMEDAD	4/6 * 100	%	20,6	20,0	19,8	20,1	19,7	20,8
9	**Humedad Promedio**		%	**20,3**		**20,0**		**20,2**	

RESULTADOS:			
LL = %	38,3	37,4	37,1
LP = %	20,3	20,0	20,2
IP = %	18,0	17,4	16,9
GRANULOMETRÍA:			
Pas. # 4: %	100	100	100
Pas. # 10: %	100	100	100
Pas. # 40: %	98	97	98
Pas. # 200: %	90	88	89
Clasificacion Norma AASHTO	A - 6	A - 6	A - 6
Indice de Grupo	15	14	14
Caracterizacion del material	Arcilla limo	Arcilla limo	Arcilla limo

Lab. Carlos Ortiz

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ING. FELIPE RAMIREZ

DESIGN OF REVETMENTS

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1 INTRODUCTION

The use of revetments, such as riprap, blocks and block mats, various mattresses, and asphalt in civil engineering practice is very common. The granular filters, and more recently the geotextiles, are more or less standard components of the revetment structure (PIANC, 1987,1992).

Within the scope of the research on the stability of open slope revetments, much knowledge has been developed about the stability of placed (pitched) stone revetments under wave load (CUR/TAW, 1995) and stability of rock under wave and current load (CUR/CIRIA, 1991, CUR/RWS, 1995).

Until recently, no or unsatisfactory design tools were available for a number of other (open) types of revetment and for other stability aspects. This is why the design methodology for placed block revetments has recently been extended in applicability by means of a number of desk-studies for other (open) revetments:

- interlock systems and block mats;
- gabions;
- concrete mattresses;
- geosystems, such as sandbags and sand sausages;

and other stability aspects, such as: flow-load stability, soil-mechanical stability and residual strength.

This chapter, based partly on the paper by Klein Breteler et al., 1998, aims at giving a summary of the increased knowledge, especially that concerning the design tools that have been made available. The details behind it can be found in (Pilarczyk et al., 1998).

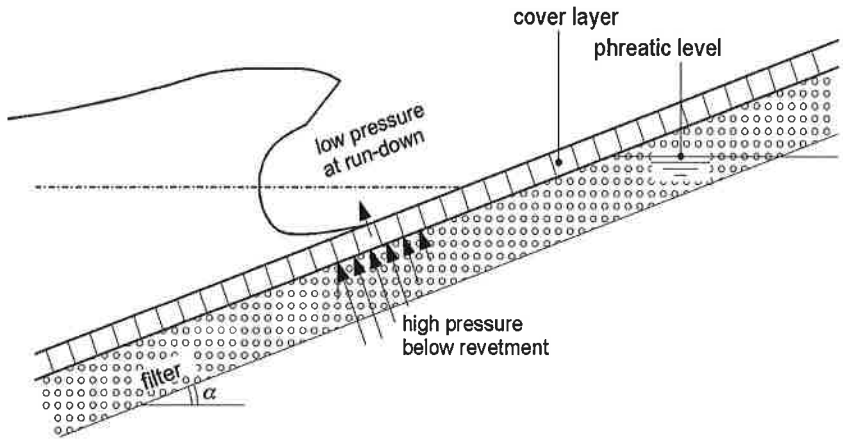

Figure 1 Pressure development in a revetment structure

2 THEORETICAL BACKGROUND OF WAVE LOADING

Wave attack on revetments will lead to a complex flow over and through the revetment structure (filter and cover layer). During wave run-up the resulting forces by the waves will be directed opposite to the gravity forces. Therefore the run-up is less hazardous then the wave run-down. Wave run-down will lead to two important mechanisms:

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- The downward flowing water will exert a drag force on the cover layer and the decreasing freatic level will coincide with a downward flow gradient in the filter (or in a gabion). The first mechanism can be schematised by a free flow in the filter or gabion with a typical gradient equalling the slope angle. It may result in sliding.
- During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a 'wall' of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the revetment is almost dry and therefore there is a low pressure on the structure. The high pressure front will lead to an upward flow in the filter or a gabion. This flow will meet the downward flow in the run-down region. The result is an outward flow and uplift pressure near the point of maximum wave run-down (Figure 1).

The schematised situation can be quantified on the basis of the Laplace equation for linear flow:

$$\frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (1)$$

with: $\phi = \phi_b$ = potential head induced in the filter or a gabion (m)
y = coordinate along the slope (m)
z = coordinate perpendicular to the slope (m)

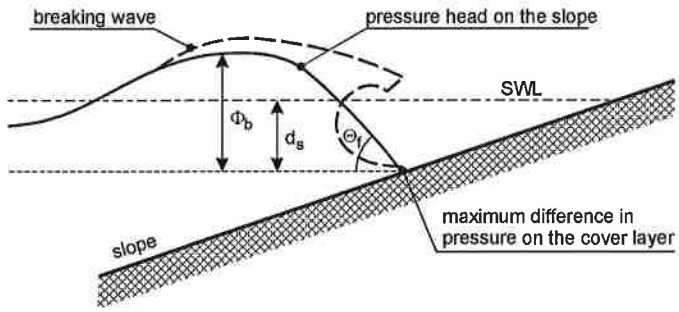

Figure 2 Schematization of pressure head on a slope

After complicated calculations the uplift pressure in the filter or a gabions can be derived. The uplift pressure is dependent on the steepness and height of the pressure front on the cover layer (which is dependent on the wave height, period and slope angle, see Figure 2), the thickness of the cover layer and the level of the phreatic line in the filter or a gabion. In case of riprap or gabions, it is not dependent on the permeability of the cover layer, if the permeability is much larger then the subsoil. For semi-permeable cover layers the equilibrium of uplift forces and gravity forces (defined by components of a revetment) leads to the following (approximate) design formula (Pilarczyk et al 1998):

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \quad \text{with} \quad \Lambda = \sqrt{\frac{bDk}{k'}} \qquad (2a)$$

or $$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \xi_{op}^{-0.67} \qquad (2b)$$

or $$\frac{H_{scr}}{\Delta D} = F\,\xi_{op}^{-0.67} \qquad (2c)$$

where H_{scr} = significant wave height at which blocks will be lifted out [m]; $\xi_{op} = \tan\alpha/\sqrt{(H_s/(1.56T_p^2))}$ = breaker parameter; T_p = wave period at the peak of the spectrum [s]; Λ = leakage length [m], $\Delta = (\rho_s - \rho)/\rho$ =

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relative volumetric mass of cover layer; b = thickness of a sublayer [m], D = thickness of a top (cover) layer [m], k = permeability of a sublayer [m/s], k' = permeability of a top layer [m/s], f = stability coefficient, mainly dependent on structure type, tanα and friction; F = total (black-box) stability factor.

The leakage length (Λ) and stability coefficient (F) are explained more in detail in the next sections.

3 STRUCTURAL RESPONSE

3.1 Wave-load approach

There are two practical design methods available: the black-box model and the analytical model. In both cases, the final form of the design method can be presented as a critical relation of the load compared to strength, depending on the type of wave attack:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \text{function of } \xi_{op} \qquad (3a)$$

For semi-permeable cover layers, the basic form of this relation is:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \frac{F}{\xi_{op}^{2/3}} \text{ with maximum } \left(\frac{H_s}{\Delta D}\right)_{cr} = 8.0 \text{ and } ctg\alpha \geq 2 \qquad (3b)$$

or, in more general form (also applicable for riprap and $ctg\alpha \geq 1.5$), as defined by Pilarczyk (1990, 1998):

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \frac{F\cos\alpha}{\xi_{op}^{b}} \qquad (3c)$$

In which: F = revetment (stability) factor, H_s = (local) significant wave height (m), Δ = relative density, D = thickness of the top layer (m), ξ_{op} = breaker parameter (-), and b = exponent; $0.5 \leq b \leq 1.0$.

The approximate values of stability factor F are: F = 2.25 for riprap, F = 2.5 for pitched stone of irregular shape, F = 3.0 to 3.5 for pitched basalt, F = 4.0 for geomattresses, $3.5 \leq F \leq 5.5$ for block revetments (4.5 as an average/usual value), $4.0 \leq F \leq 6.0$ for block mats (higher value for cabled systems), $6.0 \leq F \leq 8.0$ for gabions, and $6.0 \leq F \leq 10$ for (asphalt or concrete) slabs.
Exponent b refers to the type of wave-slope interaction and its value is influenced by the roughness and the porosity of a revetment. The following values of exponent b are recommended: $b = 0.5$ for permeable cover layers (i.e., riprap, gabions, pattern grouted riprap, very open block mats), $b = 2/3$ for semi-permeable cover layers (i.e., pitched stone and placed blocks, block mats, concrete- or sand-filled geomattresses, and $b = 1.0$ for slabs.

The relative density is defined as follows:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \qquad (4a)$$

with: ρ_s = density of the protection material and ρ_w = density of water (kg/m^3). For porous top layers, such as sand mattresses and gabions, the relative density of the top layer must be determined, including the water-filled pores:

$$\Delta_t = (1 - n) \cdot \Delta \qquad (4b)$$

In which: Δ_t = relative density including pores and n = porosity of the top layer material.

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D and Δ are defined for specific systems such as:

- for rock: $D = D_n = (M_{50}/\rho_s)^{1/3}$ (= nominal diameter) and $\Delta_t = \Delta = (\rho_s-\rho_w)/\rho_w$
- for blocks: D = thickness of block and $\Delta_t = \Delta$
- for mattresses: D = d = average thickness of mattress and $\Delta_t = (1-n)\Delta$, where n = bulk porosity of fill material and Δ = relative density of fill material. For common quarry stone (1-n) Δ ~1.

The breaker parameter is defined as follows:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s / L_{op}}} \tag{5}$$

The wave steepness S_{op} is defined as:

$$S_{op} = \frac{H_s}{L_o} = \frac{2\pi H_s}{g T^2} \tag{6a}$$

In which: $$L_{op} = \frac{g}{2\pi} T_p^2 \tag{6b}$$

with: α = slope angle (°), L_{op} = deep-water wavelength at the peak period (m), and T_p = wave period at the peak of the spectrum (s).

The advantage of this black-box design formula is its simplicity. The disadvantage, however, is that the value of F is known only very roughly for many types of structures.
The analytical model is based on the theory for placed stone revetments on a granular filter (pitched blocks). In this calculation model, a large number of physical aspects are taken into account. In short, in the analytical model nearly all physical parameters that are relevant to the stability have been incorporated in the "leakage length": $\Lambda = \sqrt{(bDk/k')}$. The final result of the analytical model may, for that matter, again be presented as a relation such as Eqs. 2c or 3c where $\mathbf{F} = f(\Lambda)$.

With a system without a filter layer (directly on sand or clay and geotextile) not the permeability of the filter layer, but the permeability of the subsoil (eventually with gullies/surface channels) is filled in. For the thickness of the filter layer it is examined to which depth changes at the surface affect the subsoil. One can fill in 0.5 m for sand and 0.05 m for clay. The values for D and Δ depend on the type of revetment.

In the case of a geotextile situated directly under the cover layer, the permeability of the cover layer decreases drastically. Since the geotextile is pressed against the cover layer by the outflowing water, it should be treated as a part of the cover layer. The water flow trough the cover layer is concentrated at the joints between the blocks, reaching very high flow velocities and resulting in a large pressure head over the geotextile. The presence of a geotextile may reduce k' by a factor 10 or more

To be able to apply the design method for placed stone revetments under wave load to other systems, the following items may be adapted:

- the revetment parameter **F**;
- the (representative) strength parameters Δ and D;
- the design wave height H_s;
- the (representative) leakage length Λ;
- the increase factor Γ (friction/interlocking between blocks) on the strength .

Only suchlike adaptations are presented in this summarising review. The basic formulas of the analytical model are not repeated here. For these, reader is referred to (CUR/TAW 1995).

The wave attack on a slope can be roughly transformed into the maximum velocity component on a slope during run-up and run-down, U_{max}, by using the following formula:

$$U_{max} = p\sqrt{gH_s\xi_{op}} \tag{7}$$

(for irregular waves and smooth slopes: $1 < p < 1.5$)

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3.2 Flow-load stability

There are two possible approaches for determining the stability of revetment material under flow attack. The most suitable approach depends on the type of load:

- flow velocity: 'horizontal' flow, flow parallel to dike;
- discharge: downward flow at slopes steeper than 1:10, overflow without waves; stable inner slope.

When the flow velocity is known, or can be calculated reasonably accurately, Pilarczyk's relation (Pilarczyk, 1990, 1999, Pilarczyk et al. 1998) is applicable:

$$\Delta D = 0.035 \frac{\Phi}{\Psi} \frac{K_T K_h}{K_s} \frac{u_{cr}^2}{2g} \qquad (8)$$

in which: Δ = relative density, D = characteristic thickness (m): for riprap $D = D_n$= nominal diameter as defined previously, g = acceleration of gravity ($g=9.81\ m/s^2$), u_{cr} = critical vertically-averaged flow velocity (m/s), Φ = stability parameter, Ψ = critical Shields parameter, K_T = turbulence factor, K_h = depth parameter, and K_s = slope parameter.
These parameters are explained below.

Stability parameter Φ :
The stability parameter Φ depends on the application. Some guide values are:

Revetment type	Continuous toplayer	Edges and transitions
Riprap and placed blocks	1.0	1.5
Block mats, gabions, washed-in blocks, geobags, and geomattresses	0.5 to 0.75	0.75 to 1.0

Shields parameter Ψ :
With the critical Shields parameter Ψ the type of material can be taken into account:

- riprap, small bags $\Psi \approx 0.035$
- placed blocks, geobags $\Psi \approx 0.05$
- blockmats $\Psi \approx 0.07$
- gabions $\Psi \approx 0.07$
- geomattresses $\Psi \approx 0.07$

Turbulence factor K_T:
The degree of turbulence can be taken into account with the turbulence factor K_T. Some guide values for K_T are:

- Normal turbulence:
  abutment walls of rivers: $K_T \approx 1.0$
- Increased turbulence:
  river bends: $K_T \approx 1.5$
  downstream of stilling basins: $K_T \approx 1.5$
- Heavy turbulence
  hydraulic jumps: $K_T \approx 2.0$
  strong local disturbances: $K_T \approx 2.0$
  sharp bends: $K_T \approx 2.0$ (to 2.5)
- Load due to water (screw) jet: $K_T \approx 3.0$ (to 4.0)

Depth parameter K_h:
With the depth parameter K_h, the water depth is taken into account, which is necessary to translate the depth-

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averaged flow velocity into the flow velocity just above the revetment. The depth parameter also depends on the development of the flow profile and the roughness of the revetment.
The following formulas are recommended:

fully developed velocity profile: $$K_h = \frac{2}{\left(\log\left(\frac{12h}{k_s}\right)\right)^2} \quad (9a)$$

non-developed profile: $$K_h = \left(\frac{h}{k_s}\right)^{-0.2} \quad (9b)$$

very rough flow ($h/k_s < 5$): $$K_h = 1.0 \quad (9c)$$

In which: h = water depth (m) and k_s = equivalent roughness according to Nikuradse (m).
In the case of dimensioning the revetment on a slope, the water level at the toe of the slope must be used for h. The equivalent roughness according to Nikuradse depends on the type of revetment/geosystem. For riprap, k_s is equal usually to one or twice the nominal diameter of the stones, for bags it is approximately equal to the thickness (d), for mattresses it depends of the type of mattress: k_s of about 0.05 m for smooth types and about the height of the rib for articulating mats.

Slope parameter K_s:
The stability of revetment elements also depends on the slope gradient under which the revetment is applied, in relation to the angle of internal friction of the revetment. This effect on the stability is taken into account with the slope parameter K_s, which is defined as follows:

$$K_s = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\theta}\right)^2} = \cos\alpha \sqrt{1 - \left(\frac{\tan\alpha}{\tan\theta}\right)^2} \quad (10a)$$

or

$$K_s = \cos\alpha_b \quad (10b)$$

with: θ = angle of internal friction of the revetment material, α = transversal slope of the bank (°), and α_b = slope angle of river bottom (parallel along flow direction) (°).
The following values of θ can be assumed as a first approximation: 40° for riprap, 30° to 40° for sand-filled systems, and 90° for stiff and anchored mortar-filled mattresses and (cabled) blockmats ($K_s = \cos\alpha$). However, for flexible non-anchored mattresses and block mats (units without contact with the neighbouring units) this value is much lower, usually about 3/4 of the friction angle of the sublayer. In case of geotextile mattress and block mats connected to geotextile lying on a geotextile filter, θ is about 15° to 20°.

The advantage of this general design formula of Pilarczyk is that it can be applied in numerous situations. The disadvantage is that the scatter in results, as a result of the large margin in parameters, can be rather wide.

With a downward flow along a steep slope it is difficult to determine or predict the flow velocity, because the flow is very irregular. In such case formulas based on the discharge are developed (Pilarczyk et al 1998).

3.3 Soil-mechanical stability

The water motion on a revetment structure can also affect the subsoil, especially when this consists of sand.

Geotechnical stability is dependent on the permeability and stiffness of the grain skeleton and the compressibility of the pore water (the mixture of water and air in the pores of the grain skeleton). Wave pressures on the top layer are passed on delayed and damped to the subsoil under the revetment structure and to deeper layers (as seen perpendicular to the slope) of the subsoil. This phenomenon takes place over a larger distance or depth as the grain skeleton and the pore water are stiffer. If the subsoil is soft or the pore water more compressible (because of the presence of small air bubbles) the compressibility of the system increases and large damping of the water pressures over a short distance may occur. Because of this, alternately water under-tension and over-tension may develop in the subsoil and corresponding to this an increasing and decreasing grain pressure. It can lead to sliding or slip circle failure, see Figure 3.

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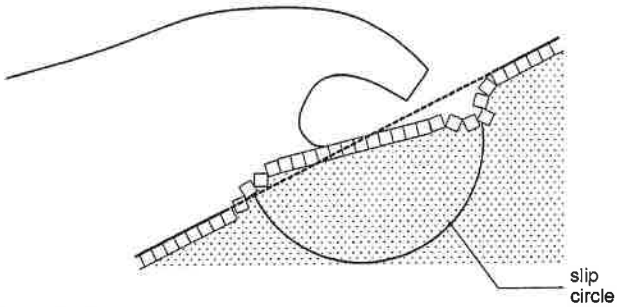

Figure 3 Schematised development of S-profile and possible local sliding in sand

The design method with regard to geotechnical instability is presented in the form of design diagrams. An example is given in Figure 4 (more diagrams and details: see Pilarczyk et al, 1998). The maximum wave height is a function of the sum of the cover layer weight (ΔD) and filter thickness (b_f).

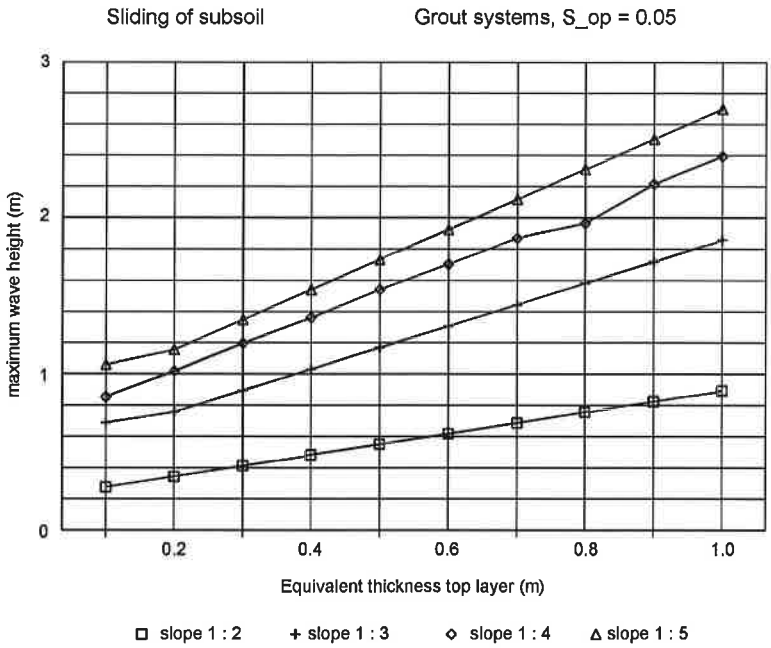

Figure 4 Geotechnical stability; design diagram for mattresses and $H_s/L_{op} = 0.05$

3.4 Filters

Granular and/or geotextile filters can protect structures subjected to soil erosion when used in conjunction with revetment armour such as riprap, blocks and block mats, gabions and mattresses, asphalt or concrete slabs, or any other conventional armour material used for erosion control (PIANC, 1987, 1992).

However, there is still a misunderstanding about the function of geotextiles in the total design of these structures, especially in comparison with the granular filters. In this Section the general principles of designing revetments incorporating granular or geotextiles are reviewed. Attention is paid to the replacing of a granular filter by a geotextile, which may often lead to geotechnical instability. Furthermore it appears that a thicker granular filter gives a larger geotechnical stability, but a lower cover layer stability (uplift of blocks). The conclusion is therefore that the wave loads must be distributed (balanced) adequately over the sand (shear stress) and the cover layer (uplift pressure). Too much emphasis on one failure mechanism can lead to another mechanism.

Filters have two functions: erosion prevention and drainage. Traditional design criteria for filters are that they should be "geometrically tight" and that the filter permeability should be larger than the base (soil) permeability. However, it results in a large number of layers which are often unnecessary, uneconomical and difficult to realize. In several cases a more economical filter design can be realized

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using the concept of "geometrically open filters" (e.g. when the hydraulic loads/gradients are too small to initiate erosion). Recently, some criteria for "geometrically open" filters including geotextiles were developed (and are still under further development). However, the application of these criteria requires the knowledge/prediction of the hydraulic loads.

In the cases when the erosion exceeds an acceptable level, a filter construction is a proper measure for solving this problem. In revetment structures geotextiles are mostly used to protect the subsoil from washing away by the hydraulic loads, such as waves and currents. Here the geotextile replaces a granular filter. Unfortunately, the mere replacing of a granular filter by a geotextile can endanger the stability of other components in the bank protection structure. The present section shows that designing a structure is more than just a proper choice of geotextile.

Filter structures can be realized by using granular materials (i.e. crushed stone), bonded materials (i.e. sand asphalt, sand cement), and geotextiles, or a combination of these materials. Typical filter compositions are shown in Figure 5. The choice between the granular filter, a bonded filter or geotextile depends on a number of factors. In general, a geotextile is applied because of easier placement and relatively lower cost. For example, the placement of granular filter underwater is usually a serious problem; the quality control is very difficult, especially when placement of thin layers is required.

When designing with geotextiles in filtration applications, the basic concepts are essentially the same as when designing with granular filters. The geotextile must allow the free passage of water (permeability function) whilst preventing the erosion and migration of soil particles into the armour or drainage system (retention function).

In principle, the geotextile must always remain more permeable than the base soil and must have pore sizes small enough to prevent the migration of the larger particles of the base soil. Moreover, concerning the permeability, not only the opening size but also the number of openings per unit area (Percent Open Area) is of importance (Pilarczyk, 1999).

It has to be stressed that geotextiles cannot always replace the granular filter completely. A granular layer can often be needed to reduce (damp) the hydraulic loadings (internal gradients) to an acceptable level at the soil interface. After that, a geotextile can be applied to fulfill the filtration function.

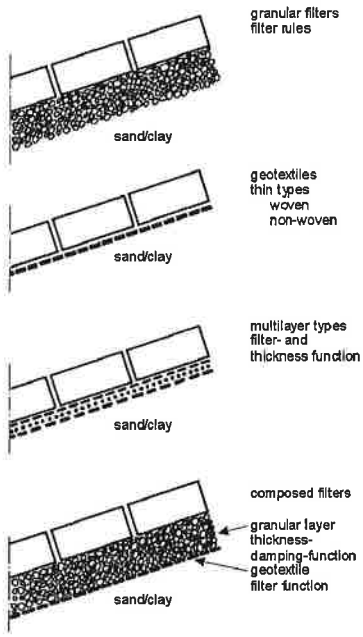

Figure 5 Examples of filters

In respect to the filters for erosion control (granular or geotextile) the distinction can be made between:

* geometrically tight filters,
* geometrically open filters, and
* transport filters (when a limited settlement is allowed).

Only geometrically tight filters are discussed. For other type of filters reader is referred to (Pilarczyk, 1999)

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3.4.1 Design criteria for geometrically tight granular filters

In this case there will be no transport of soil particles from the base, independent of the level of hydraulic loading. That means that the openings in the granular filter or geotextile are so small that the soil particles are physically not able to pass the opening. This principle is illustrated in Figure 6 for granular filters.

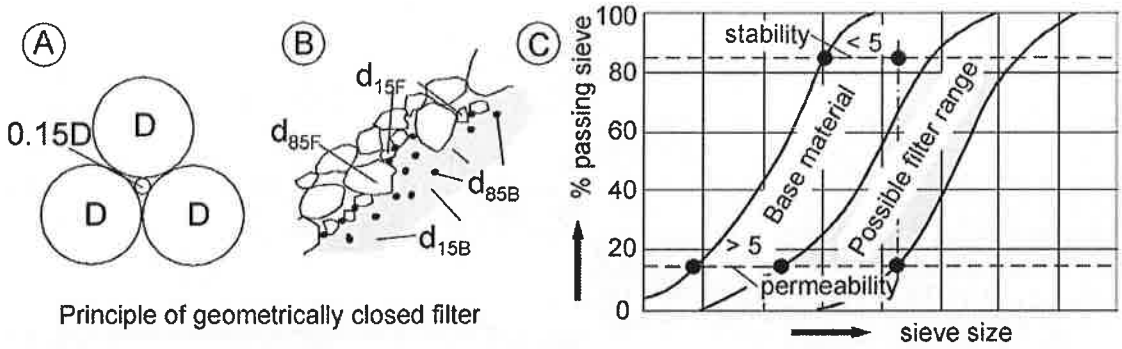

Figure 6 Principles of geometrically tight filters

The main design rules (criteria) for geometrically tight (closed) granular filters and geotextiles are summarized below. The more detailed information on design of geotextile filters is given in Pilarczyk (1999).

The soil tightness of the initial situation can be checked by means of the well-known criteria for granular filters:

- Interface stability (also called 'piping' criterion):

$$\frac{D_{f15}}{D_{b85}} \leq 4 \text{ to } 5 \qquad (11)$$

where:

D_{f15} is the grain size of the filter layer (or cover layer) which is exceeded by 15 % of the material by weight in m;

D_{b85} is the grain size of the base material (soil) which is exceeded by 85 % of the material by weight in m.

The factor 4 in Eq. (11) was given by TERZAGHI. The factor 5 is determined for normal wide-graded materials. Sometimes a similar equation is defined as:

$$\frac{D_{f50}}{D_{b50}} < 6 \text{ to } 10 \qquad (12)$$

However, Equation (12) is less general than Eq. (11) and can be used for 'small' gradation only. Therefore, Eq. (11) is recommended for general use. However, in the case of very 'wide' gradation the situation requires an additional check with respect to the internal migration. In this respect, an important parameter is the so-called 'uniformity coefficient' *Cu*, defined by Equation (13) and the shape of the sieve curve:

$$Cu = \frac{D_{b60}}{D_{b10}} \qquad (13)$$

where: *C*u is the coefficient of uniformity.

- Internal stability can be roughly judged by the following rules (Eq. 14):

$$D_{10} < 4\, D_5 \qquad (14a)$$
$$D_{20} < 4\, D_{10} \qquad (14b)$$
$$D_{30} < 4\, D_{15} \qquad (14c)$$
$$D_{40} < 4\, D_{20} \qquad (14d)$$

- permeability criterion

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$$\frac{D_{f15}}{D_{b15}} > 5 \qquad (15)$$

3.4.2 Summary of design rules for geotextiles

Current definitions for geotextile openings

There are a large number of definitions of the characteristic of geotextile openings. Moreover, there are also different test (sieve) methods for the determination of these openings (dry, wet, hydrodynamic, etc.) which depend on national standards. These all make the comparison of test results very difficult or even impossible. That also explains the necessity of international standarization in this field.
Some of the current definitions are listed below:

O_{90} corresponds with the average sand diameter of the fraction of which 90% of the weight remains on or in the geotextile (or 10 % passes the geotextile) after 5 minutes of sieving (method: dry sieving with sand);

O_{98} corresponds with the average sand diameter of the fraction of which 98% of the weight remains on or in the geotextile after 5 minutes of sieving. O_{98} gives a practical approximation of the maximum filter opening and therefore plays an important role in the sand tightness criterion for a geotextile in strong cyclic loading situations. O_{98} is also referred to as O_{max}.

O_f filtration opening size (FOS). O_f is comparable with O_{95} (hydrodynamic sieve method);

AOS apparent opening size (acc. to ASTM method), also called EOS (effective opening size). The AOS is determined by sieving spherical glass particles of known size through a geotextile. The AOS, also frequently referred to as O_{95} (dry sieve method), is defined as a standard sieve size, x, mm, for which 5% or less of the glass particles pass through the geotextile after a specified period of sieving;

D_w effective opening size which corresponds with the sand diameter of the fraction of which 10 %, determined by the wet sieve method, passes through the geotextile. D_w is comparable with O_{95}.

The transport of soil particles within a grain structure is possible when there is enough space and a driving force (groundwater pressure, hydraulic gradients within the soil). In most cases it is the intention to prevent the transport of small-sized soil particles in the subsoil and therefore the term soil tightness is used and not the term space for transport or pore volume (in the case of the transport of water the terms pore volume and water permeability are used). The relation between pore magnitude and grain diameter can be characterized by: pore diameter ≈ 20% of the grain diameter. Just as for the characterization of the performance of a grain structure with regard to the transport of soil particles, for geosynthetics, too, the term soil tightness is used.

As was mentiond before (Figure 6), in a theoretical case when the soil is composed of spheres of one-size diameter, all spheres can be retained if all apertures in the geosynthetic are smaller than the diameter of the spheres. Usually the soil consists of particles with different diameters and shapes, which is reflected in the particle-size distribution curves. Smaller particles can disappear straight across the geosynthetic by groundwater current. In this case the retained soil structure can function as a natural filter; see Figure 7. The better the soil particles are distributed, the better the soil tightness of the soil structure is effected. Smaller soil particles get stuck into the spaces between larger ones and the soil structure prevents the flow of fine particles. When certain particle-size fractions are lacking, the soil structure is not stacked very well and cavities develop through which erosion can occur. The displacement of soil particles not only depends on the soil tightness but also on the hydraulic gradient in the soil structure. Morever, the dynamic effects due to heavy wave loading may not allow the forming of a natural filter, and the process of washing-out may continue.

According to some researchers the forming of a natural filter is only possible for stationary flow (CUR, 1993). However, this is also possible for non-stationary flow, for small values of the hydraulic gradients. For heavy wave attack (i.e. exposed breakwaters) this is usually not the case. In extreme situations, soil liquefaction is even possible. In such situations the soil particles can still reach the surface of a geotextile and be washed out.

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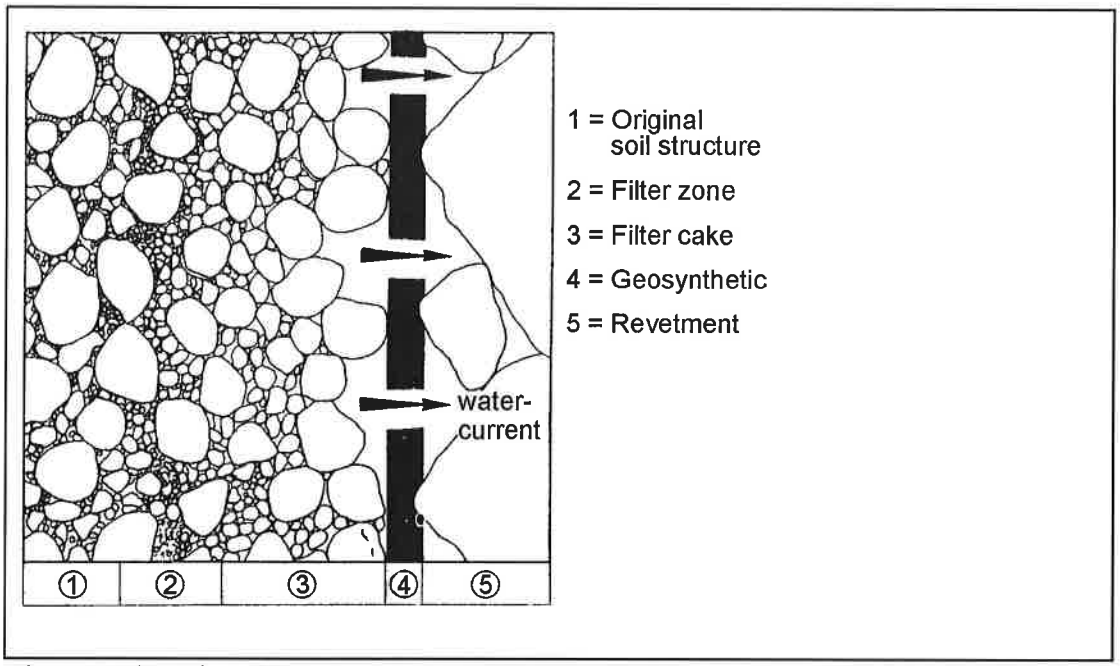

Figure 7 Schematic representation of a natural filter with a soil-retaining layer

In order to judge the risk of wash-out of soil particles through the geosynthetics, some aspects have to be considered. An important factor is the internal stability of the soil structure.

In the case of a loose particle stacking of the soil many small soil particles may pass through the geosynthetic before a stable soil structure is developed near the geosynthetic. Also, a proper compaction of soil is very important for the internal stability of soil. The internal stability is defined by the uniformity coefficient *Cu* (see Eq. 13). It is defined as D_{b60}/D_{b10}. If this ratio is smaller than 6 (to 10), the soil structure is considered internally stable.

In case of vibration, for instance caused by waves or by traffic, the stable soil structures can be disturbed. To avoid such situations, the subsoil has to be compacted in advance and a good junction between geosynthetic and subsoil has to be guaranteed and possibly, a smaller opening of geotextile must be chosen.

The shape of the sieve curve also influences the forming of a natural filter. Especially, when Cu > 6, the shape of the base gradation curve and its internal stability must be taken into account (Pilarczyk, 1999). For a self-filtering linearly graded soil, the representative size corresponds to the average grain size, D_{b50}. For a self-filtering gap graded soil, this size is equal to the lower size of the gap. For internally unstable soils, this size would be equivalent to D_{b30} in order to optimize the functioning of the filter system. It is assumed that the involved bridging process would not retrogress beyond some limited distance from the interface.

· *Soil tightness*

With respect to the soil tightness of geotextiles many criteria for geometric soil tightness have been developed and published in the past (Pilarczyk, 1999). An example of such design criteria, based on Dutch experience, is presented in Table 1. An additional requirement is that the soil should be internally stable. The internal stability of a grain structure is expressed in the ratio between D_{b60} and D_{b10}. As a rule this value has to be smaller than 10 to guarantee sufficient stability.

Table 1 Design requirements for geosynthetics with a filter and separation function

description filter	function/soil tightness
- stationary loading	$O_{90} \leq 1$ (to 2) D_{b90}
- cyclic loading with natural filter (stable soil structure)	$O_{98} \leq 1$ (to 2) D_{b85}
- cyclic loading without a natural filter	

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(unstable soil structure)	
. when wash-out effects acceptable	$O_{98} \leq 1.5\ D_{b15}$
. when wash-out effects not acceptable	$O_{98} \leq D_{b15}$

However, in many situations additional requirements will be necessary, depending on the local situation. Therefore, for design of geometrically tight geotextiles the method applied in Germany can be recommended (see HEERTEN, 1982, PIANC, 1987, BAW, 1993). In this method a distinction is made between so-called stable and unstable soils. Soils are defined as unstable (susceptible to down-slope migration) when the following specifications is fulfilled:
- a proportion of particles must be smaller than 0.06 mm;
- fine soil with a plasticity index ($\mathbf{I_p}$) smaller than 0.15 (thus, it is not a cohesive soil);
- 50 percent (by weight) of the grains will lie in the range $0.02 < D_b < 0.1$ mm;
- clay or silty soil with $Cu < 15$.

If the $\mathbf{I_p}$ is unknown at the preliminary design stage then the soil may be regarded as a problem soil if the clay size fraction is less than 50% of the silt size fraction.

The design criteria are presented in Table 2. More detailed information can be found in Pilarczyk (1999).

Table 2 Design criteria for geometrically soil-tight geotextiles

	soil type			
	$D_{b40} < 60$ µm		$D_{b40} > 60$ µm	
	stable soil	Instable soil	stable soil	instable soil
stationary loading	$O_{90} < 10\ D_{b50}$ and $O_{90} < 2\ D_{b90}$	$O_{90} < 10\ D_{b50}$ and $O_{90} < D_{b90}$	$O_{90} < 5\ D_{b10} Cu^{1/2}$ and $O_{90} < 2\ D_{b90}$	$O_{90} < 5\ D_{b10} Cu^{1/2}$ and $O_{90} < D_{b90}$
dynamic loading		$O_{90} < D_{b90}$ and $O_{90} < 0.3$ mm (300µm)	$O_{90} < D_{b90}$	$O_{90} < 1.5\ D_{b10} Cu^{1/2}$ and $O_{90} < D_{b50}$ $O_{90} < 0.5$ mm

(O_{90} is determined by wet sieve method)

In the case of fine sand or silty subsoils, however, it can be very difficult to meet these requirements. A more advanced requirement is based on hydrodynamic sand tightness, viz. that the flow is not capable of washing out the subsoil material, because of the minor hydrodynamical forces exerted (although the apertures of the geotextile are much larger than the subsoil grains).

Requirements concerning water permeability

To prevent the forming of water pressure (uplift) in the structure, causing loss of stability, the geotextile has to be water permeable. One has to strive for the increase of water permeability of a construction in the direction of the water current. In the case of a riverbank protection it means that the permeability of the geotextile has to be larger than the permeability of the soil on which the geotextile has to be applied. In the case of a dike slope or dike foundation the geosynthetic is often applied on an impermeable layer of clay. Proper permeability of geotextiles is very important in respect to the stability of relatively less permeable cover layers as, for example, block and block mats. When a geotextile lies directly under the cover layer it considerably reduces the open area of the cover layer, and as a result the uplift forces increase (see example in Section 4.2). The water permeability of woven fabrics and nonwovens may decrease in the course of time owing to the fact that fine soil particles, which are transported by the groundwater flow from the subsoil, **block** the openings in the geotextile, or migrate into the pores of the geosynthetic (**clogging**).

To prevent mineral clogging, the pore size of the geotextiles has to be chosen as large as possible; but, of course, this pore size has still to meet the requirements for soil tightness. The danger of clogging increases when the soil contains more than 20% of silt or in the case of gap-grading of a soil. On the other

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hand, there usually is no danger of clogging when the total hydraulic gradient (over the subsoil and geotextile together) is less than 3, or when the subsoil is well graded. In all situations it holds that the soil must be internally stable. For less critical situations no clogging can be expected if:

$Cu > 3$: $O_{95}/D_{b15} > 3$; (16a)

$Cu < 3$: criterion of internal stability of soil should be satisfied and/or geotextile with maximum opening size from soil-tightness criteria should be specified. (16b)

In respect to the water permeability of geosynthetics/geotextiles, a distinction should be made between "normal to the interface" and "parallel to the interface". For geotextile filters the permeability parallel to the interface is of importance, while for drainage structures the permeability normal to the interface is of most importance. As a general design criterion for flow normal to the interface one can hold that the water permeability of a geosynthetic/geotextile has to be greater than that of the soil at the side from where the water flow comes. As a rule one can keep to:

$$k_{geotextile\ (filter)} = k_{soil} \times \text{factor} \tag{17}$$

where k_s and k_g are usually (basically) defined for laminar conditions.

For normal (stationary) conditions and applications and clean sands a factor of 2 is sufficient to compensate the effect of blocking. If a geotextile is permeable with a factor of 10 more than the (non-cohesive) subsoil, overpressure will usually not occur, neither below the geosynthetic, nor in the case of reduced permeability caused by clogging or blocking. However, for special applications (i.e., for dam-clay cores with danger of clogging) this factor can be 50 or more.

The detailed treatment with respect to the effect of hydraulic loading on permeability characteristics of the geotextile and the possible interactions with the subsoil can be found in (Van Santvoort, 1994 and Pilarczyk, 1999)). However, the basic information on this aspect as far as it is needed for total design with geotextiles, is given hereafter.

Water permeability normal to the interface

The function of the water permeability requirement in the total approach of designing with geotextiles is to bring the design of the filter into harmony with the subsoil. The requirement that excess pressures should not occur means that the eventual loss of stability at the filter occurs no sooner than the similar loss of stability does in the subsoil (i.e. migration of particles, softening of the subsoil and resulting sliding, etc.) as a consequence of groundwater flow (critical gradients). The basic (starting) requirement is that the gradient over the geotextile should maximally be equal to the gradient of the subsoil:

$$i_n \leq i_b \tag{18}$$

where: i_b is the gradient in sublayer (or in subsoil, i_s) and i_n is the gradient normal to the geotextile.

The permeability of a geotextile can be characterized by the permeability coefficient $k_g = k_n$ (m/s) or by the permittivity ψ (1/s), see also Figure 8. Permittivity can be directly calculated from test results and expresses the rate of flow through the geotextile per unit area and per unit hydraulic head, and it is also defined as permeability per unit thickness of geotextile :

$$\psi = \frac{Q}{A\,\Delta h_g} = \frac{v_f}{\Delta h_g} = \frac{k_n}{T_g} \tag{19}$$

where:

- ψ is the permittivity in 1/s;
- Q is the flow rate through the geotextile in m^3/s;
- v_f is filtration velocity in m/s;
- A is the surface area of geotextile in m^2;
- Δh_g is the hydraulic head difference across the geotextile in m;
- k_n is the permeability coefficient of the geotextile (k_g), normal to the interface, in m/s;
- T_g is the thickness of the geotextile in m.

N.B. the term k_n/T_g in Eq. 19 is often used for rough estimation of the permeability coefficient for other (not tested) thicknesses for the same type of geotextile.

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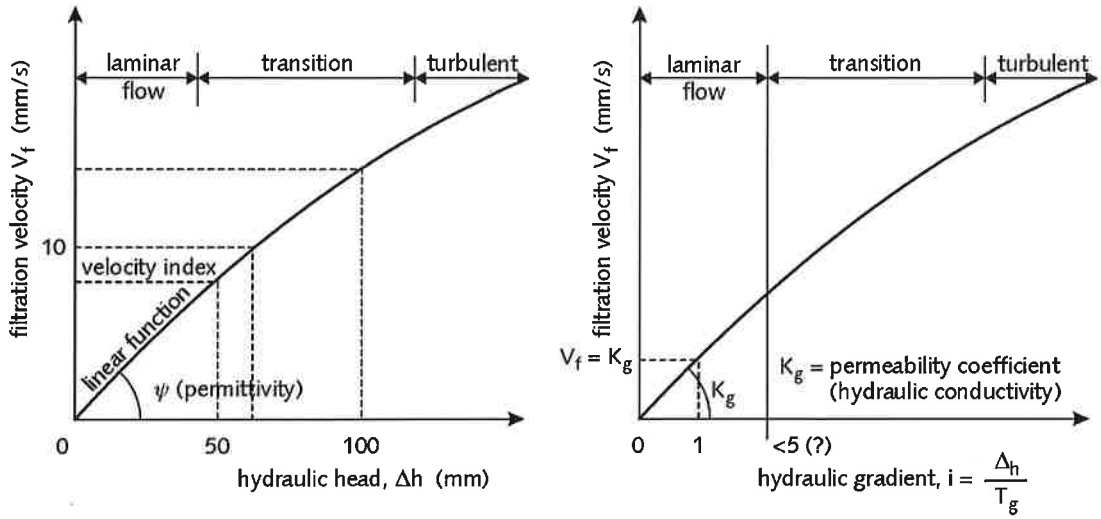
The main problem of using permittivity is the definition of the thickness of geotextile. Usually a thickness under the normal stress of 2 kPa is applied. The definitions and an example of test results concerning the determination of permittivity and permeability for geotextiles are presented in Figure 8.



Definitions

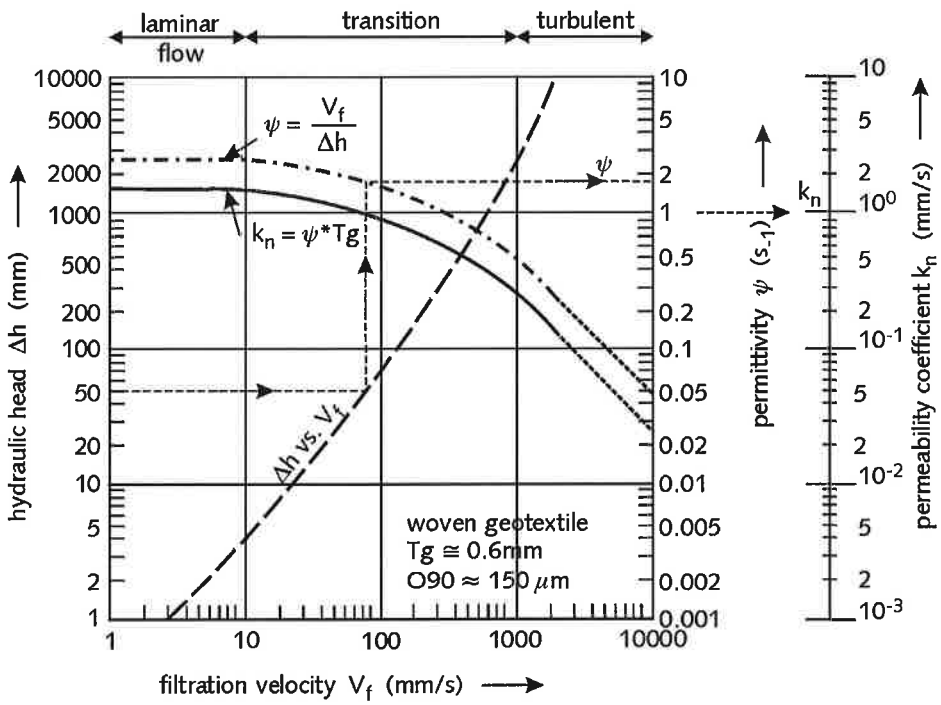


Figure 8 An example of the relationships between between permittivity and hydraulic conductivity (permeability coeff.) as a function of hydraulic head

Combining formulas (17) and (18) with formula (19), and applying a continuity principle,

$$k_n i_n = k_b i_b \qquad (20)$$

where: k_b is the permeability coefficient of the base material (subsoil, k_s) in m/s, and i_b is the gradient in the base material;

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it provides:

$$\psi = c\ k_b\ \frac{i_b}{\Delta h_g} \approx c_\psi\ k_b \qquad (21)$$

In general, permittivity of a certain geotextile is a function of the hydraulic head. Only in the zone of laminar flow the permittivity is more or less constant (see Figure 8). When permittivity is defined outside this zone, the associated hydraulic head (Dh) should be mentioned.

The total safety factor c_y incorporates a number of uncertainties (i.e., permeability of soil, the loss of permeability due to the effects of clogging and stress, etc.) and, depending on application, can be equal to 10^3 for clean sands and up to 10^5 for critical soils and severe applications in dams (see CFGG, 1986). The last value seems to be rather conservative.

Holtz et al. (1997) propose to use, additionally to other permeability criteria, the permittivity criterion directly related to the soil type defined by a certain percentage of passing the sieve of 0.075 mm. These criteria have originally been established by the U.S. Federal Highway Administration (FHWA, 1995). These permittivity requirements are:

$\psi \geq 0.5\ sec^{-1}$ for soils with < 15% passing the 0.075 mm sieve;

$\psi \geq 0.2\ sec^{-1}$ for soils with 15 to 50% passing the 0.075 mm sieve;

$\psi \geq 0.1\ sec^{-1}$ for soils with > 50% passing the 0.075 mm sieve.

The flow rate Q through the geotextile can be defined as:

$$Q = v_f A = \psi \Delta h_g A = k_n A i_g \qquad (22)$$

Discussion

There are a large number, often very unclear and confusing definitions of permeability of geotextiles, especially when the permeability of geotextile must match the permeability of a certain soil. The basic equation, both for soil and geotextile, is the Darcy's equation; $v_f = k \times i$, which is valid for laminar flow conditions.

The hydraulic gradient i is the average hydraulic gradient in the soil. For example, the gradient along (parallel to) geotextile in revetments with a thick granular layer above, loaded by run-up and run-down, often a tangent of slope angle can be applied (tana). However, for thin granular filters (layers) this gradient can be much larger than tanα. For other applications i can be estimated by using a conventional flow net analysis for seepage through dikes and dams or from a rapid drawdown analysis. The permeability of geotextiles is characterized by a number of different (national) index tests. Therefore, at this moment, the standard specification sheets provided by manufacturers include such definitions as: the permeability (filtration velocity, v_f) at the hydraulic head equal to 50, 100 or even 250 mm, permittivity defined at the standard filtration velocity (v_f) equal to 10 mm/s (at a certain hydraulic head), the head loss index corresponding to a filtration velocity of 20 mm/s, etc.

Actually, CEN (1998) prepared a European standard introducing only one index test, so-called 'Velocity Index', which defines the filtration velocity corresponding to a head loss of 50 mm across a specimen. The flow velocity v_f expressed in mm/s equals the unit discharge q expressed in l/m^2 s. However, to be able to draw conclusion about the proper choice of permeability for various conditions and applications, it is necessary to perform (to measure) the full permeability characteristics and prepare a collective plot of the velocity v_f and head loss Dh for each specimen. The test range must be sufficiently wide to allow also the determination of permeability parameters for laminar flow. If the full permeability characteristics of the geotextile product have previously been established, then for checking purposes it can be sufficient to determine the velocity index at a head loss of 50 mm only.

In case that only one or two test data with standard index specifications (v_f, Δh, and T_g) are known, the approximate estimation of permeability can be done by using the following equation: $v_f = k\ i^m$, where $i = \Delta h/T_g$ and $0.5 \leq m \leq 1.0$ ($m = 1$ for laminar flow and $m = 0.5$ for turbulent flow). When only one point is available the rough approximation can be obtained by applying $m = 0.7$.

By plotting a line on a log-log paper through two points the exponent m can be determined, and for $i = 1$, k can be approximated. By using this equation the permeability can be roughly extrapolated to the required conditions/definitions, for example, $k_g = v_f T_g/\Delta h$ or $k_g = \psi\ T_g$ at $\Delta h = 50$ mm, or k_g at $v_f = 10$ mm/s, or k_g or ψ for laminar conditions, and can be used as a first approximation. In case of doubt, more data or additional tests can be requested.

The permeabilities defined outside the laminar zone can also be interpreted in the following way, for

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example, if k_g established at v_f = 10 mm/s is equal or larger than $k_{s(soil)}$, the geotextile fulfills the requirement of permeability for filtration velocities (in the soil) lower than 10 mm/s. For larger filtration velocities the new estimation of k_g is needed, related to the higher v_f, to check the requirement $k_g > k_s$, because k_g will decrease for higher v_f in transition or turbulent flow zone.

With regard to the hydraulic efficiency of the geotextile filter, full advantage should be taken on the permissible upper limits of the opening size, provided the required mechanical filter effectiveness (soil-tightness) is ensured (DVWK, 1993). The reason for this is that an open (and possibly, thicker) structure is generally superior to a dense structure with regard to the filter stability. Thus, when the permeability is decesive for the design, the largest admissible opening size resulting from the soil-tightness criteria should be used to ensure the permeability as high as possible. There are usually no problems with obtaining a sufficient permeability when $1 \leq O_{90}/D_{90} < 2$ is applied as a soil-tightness criterion. Research results (MLYNAREK, 1994) indicate that the permeability of a soil-geotextile system is mainly defined by the permeability of a soil. However, the geotextile permeability will decrease in soil due to compression. It is of special importance for thick non-wovens. Therefore, the factor of safety should be increased accordingly to account for the critical nature of the application, type of geotextile, and the severity of the soil and hydraulic conditions.

This discussion indicates that there are still much uncertainties in the proper application of the permeability criteria. However, for normal (stationary) conditions and stable soils it is usually not a real problem. For critical projects, performance tests simulating soil-geotextile interaction can be recommended.

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4 STABILITY CRITERIA FOR PLACED BLOCKS AND BLOCK MATS

4.1 System description

Placed block revetments (or stone/block pitching) are a form of protection lying between revetments comprised of elements which are disconnected, such as rubble, and monolithic revetments, such as asphalt/concrete slabs. Individual elements of a pitched block revetment are placed tightly together in a smooth pattern. This ensures that external forces such as waves and currents can exert little drag on the blocks and also that blocks support each other without any loss of flexibility when there are local subsoil irregularities or settlement.

A (concrete) block mat is a slope revetment made of (concrete) blocks that are joined together to form a "mat", see Figure 9. The interconnection may consist of cables from block to block, of hooks connecting the blocks, or of a geotextile on which the blocks are attached with pins, glue or other means. The spaces between the blocks are usually filled with rubble, gravel or slag.

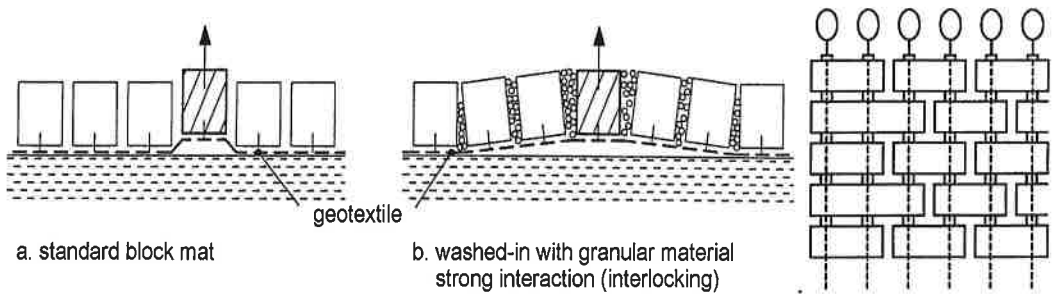

Figure 9 Examples of block mats

The major advantage of block mats is that they can to be laid quickly and efficiently and partly under water. Block mats are more stable than a setting of loose blocks, because a single stone cannot be moved in the direction perpendicular to the slope without moving other nearby stones. It is essential to demand that already with a small movement of an individual stone a significant interactive force with the surrounding stones is mobilised. Large movements of individual blocks are not acceptable, because transport of filter material may occur. After some time, this leads to a serious deformation of the surface of the slope.

The block mats are vulnerable at edges and corners. If two adjacent mats are not joined together, then the stability is hardly larger than that of pitched loose stones.

4.2 Design rules with regard to wave load

The usual requirement that the permeability of the cover layer should be larger than that of the underlayers cannot be met in the case of a closed block revetment and other systems with low permeable cover layer. The low permeable cover layer introduces uplift pressures during wave attack. In this case the permeability ratio of the cover layer and the filter, represented in the leakage length, is found to be the most important structural parameter, determining the uplift pressure. This is also the base of analytical model.

The analytical model is based on the theory for placed stone revetments on a granular filter (CUR/TAW, 1995). In this calculation model, a large number of physical aspects are taken into account (see Figures 1, 2 and 6). In short, in the analytical model nearly all physical parameters that are relevant to the stability have been incorporated in the "leakage length" factor. The final result of the analytical model may, for that matter, again be presented as a relation such as Eqs. 2 or 3 where $\mathbf{F} = f(\Lambda)$.For systems on a filter layer, the leakage length Λ is given as:

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$$\Lambda = \sqrt{\frac{bDk}{k'}} \quad \text{or} \quad \Lambda / D = \sqrt{\frac{bk}{Dk'}} \tag{23a}$$

where: Λ=leakage length [m] b_f = thickness of the filter layer (m), k_f = permeability of the filter layer or subsoil (m/s), and k' = permeability of the top (cover)layer (m/s).

With a system without a filter layer (directly on sand or clay, without gullies being formed under the top layer) not the permeability of the filter layer, but the permeability of the subsoil (eventually with gullies/surface channels) is filled in. For the thickness of the filter layer it is examined to which depth changes at the surface affect the subsoil. One can fill in 0.5 m for sand and 0.05 m for clay. The values for D and Δ depend on the type of revetment. When schematically representing a block on a geotextile on a gully in sand, the block should be regarded as the top layer and the combination of the geotextile and the small gully as the filter layer (Figure 10).

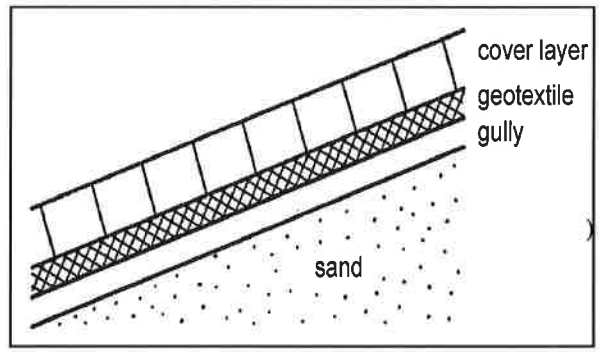

Figure 10 Schematization of a revetment with gully (cavity)

The leakage length can be calculated using:

$$\Lambda = \sqrt{\frac{(k_f d_g + k_g T_g) D}{k'}} \tag{23b}$$

with:
k_f = permeability of the filter layer (gully) (m/s), d_g = gully depth (m), k_g = permeability of the geotextile (m/s), T_g = thickness of the geotextile (m), D = thickness of the top layer (m), and k' = permeability of the top layer (m/s).

In the case of a geotextile situated directly under the cover layer, the permeability of the cover layer decreases drastically. Since the geotextile is pressed against the cover layer by the outflowing water, it should be treated as a part of the cover layer.

The water flow trough the cover layer is concentrated at the joints between the blocks, reaching very high flow velocities and resulting in a large pressure head over the geotextile. The presence of a geotextile may reduce k' by a factor 10 or more (see Figure 11).

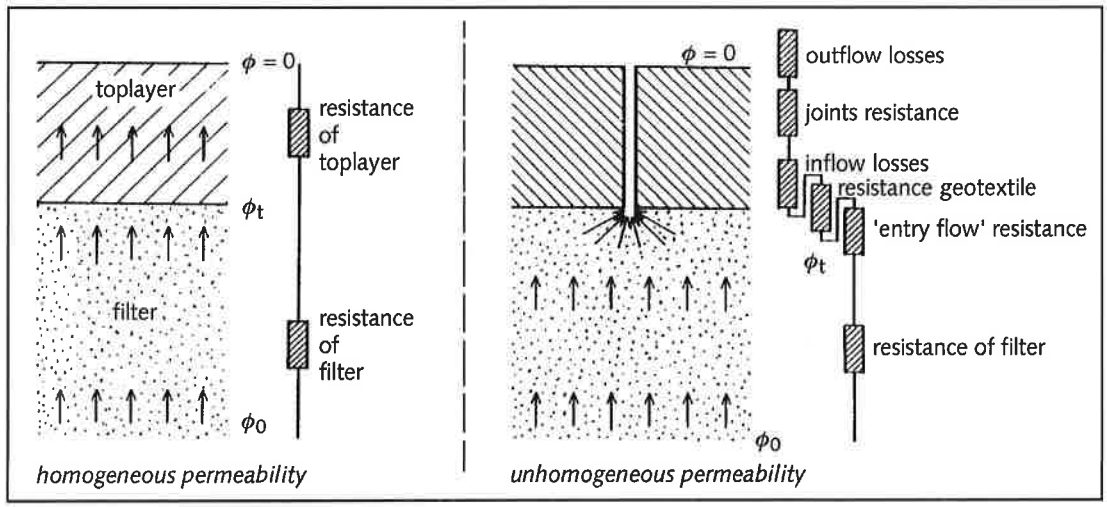

Figure 11 Combined flow resistance determining the permeability of a system

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The leakage length clearly takes into account the relationship between k_f and k' and also the thickness of the cover layer and the filter layer. For the theory behind this relationship, reference should be made to literature (see Klein Breteler et al, 1998 and CUR/TAW, 1995). The pressure head difference which develops on the cover layer is larger with a large leakage length than with a small leakage length. This is mainly due to the relationship k_f/k' in the leakage length formula. The effect of the leakage length on the dimensions of the critical wave for semi-permeable revetments is apparent from the following equations:

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} = \frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \xi_{op}^{-0.67} = \frac{H_{scr}}{\Delta D} = F\,\xi_{op}^{-0.67} \qquad (24)$$

where: H_{scr} = significant wave height at which blocks will be lifted out [m]; $\xi_{op} = \tan\alpha/\sqrt{(H_s/(1.56T_p^2))}$ = breaker parameter; T_p = wave period [s]; Δ = relative volumetric mass of cover layer = $(\rho_s - \rho)/\rho$; f = stability coefficient mainly dependent on structure type and with minor influence of Δ, $\tan\alpha$ and friction, and F = total (black-box) stability factor.

These equations indicate the general trends and have been used together with measured data to set up the general calculation model (CUR/TAW, 1995, Pilarczyk, 1998). This method works properly for placed/pitched block revetments and block mats within the following range: $0.01 < k'/k_f < 1$ and $0.1 < D/b_f < 10$. Moreover, when $D/\Lambda > 1$ use $D/\Lambda = 1$, and when $D/\Lambda < 0.01$ use $D/\Lambda = 0.01$. The range of the stability coefficient is: $5 < f < 15$; the higher values refer to the presence of high friction among blocks or interlocking systems. The following values are recommended for block revetments:

f = 5 for static stability of loose blocks (no friction between the blocks),

f = 7.5 for static stability of a system (with friction between the units),

f = 10 for tolerable/acceptable movement of a system at design conditions.

From these equations, neglecting the usually minor variations of 'f', it appears that:

- An increase in the volumetric mass, Δ, produces a proportional increase in the critical wave height. If ρ_b is increased from 2300 to 2600kg/m^3, H_{scr} is increased by about 23%,
- If the slope angle is reduced from 1:3 to 1:4 ($\tan\alpha$ from 0.33 to 0.25) H_{scr} is increased by about 20% (due to the breaker parameter, ξ_{op}),
- An increase of 20% in the thickness of the cover layer, D, increases H_{scr} by about 27%,
- A 30% reduction in the leakage length , Λ, increases H_{scr} by about 20%. This can generally be achieved by halving the thickness of the filter layer or by doubling the k'/k_f value. The latter can be achieved by approximation, by:

- reducing the grain size of the filter by about 50%, or
- by doubling the number of holes in (between) the blocks, or
- by making hole sizes 1.5 times larger, or
- by doubling joint width between blocks.

EXAMPLE: In 1983 the Armorflex mat on a slope 1:3 was tested on prototype scale at the Oregon State University: closed blocks with thickness D = 0.12 m and open area 10% on two types of geotextiles and very wide-graded subsoil (d_{15} = 0.27 mm, d_{85} = 7 mm).

In the case of a sand-tight geotextile the critical wave height (instability of mat) was only H_{scr} = 0.30 m. In the case of an open net geotextile (opening size about 1mm) the critical wave height was more than 0.75 m (maximum capacity of the wave flume).

The second geotextile was 20 times more permeable than the first one. This means that the stability increased by factor $20^{0.33}$ = 2.7.

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In most cases the permeabilities of the cover layer and sublayer(s) are not exactly known. However, based on the physical principles as described above, the practical 'black-box' method has been established where parameter Λ and coefficient 'f' are combined to one stability factor 'F'. F depends on the type of structure, characterised by the ratios of k'/k_f and D/b_f. With the permeability formulas from(CUR/TAW, 1995) it is concluded that the parameter $(k'/k_f)\bullet(D/b_f)$ ranges between 0.01 and 10, leading to a subdivision into 3 ranges of one decade each. Therefore the following types are defined:

a) Low stability: $(k'/k_f)(D/b_f) < 0.05 .. 0.1$
b) Normal stability: $0.05 .. 0.1 < (k'/k_f)(D/b_f) < 0.5 .. 1$
c) High stability: $(k'/k_f)(D/b_f) > 0.5 .. 1$

For a cover layer lying on a geotextile on sand or clay, without a granular filter, the leakage length cannot be determined because the size of b_f and k cannot be calculated. The physical description of the flow is different for this type of structure. For these structures there is no such a theory as for the blocks on a granular filter. However, it has been experimentally proved that Eqs. 3b or 24 are also valid for these structures.

It can be concluded that the theory has led to a simple stability formula (Eq. 24) and a subdivision into 4 types of (block) revetment structures:

a1) cover layer on granular filter possibly including geotextile, low stability;
a2) cover layer on granular filter possibly including geotextile, normal stability;
a3) cover layer on geotextile on sand;
a4) cover layer on clay or on geotextile on clay.

The coefficient, F, is quantified for each structure type by way of fitting Eq. 3b to the results of a large collection of results of model studies from all over the world. Only large-scale studies are used because both the waves and the wave induced flow in the filter should be well represented in the model. In the classification of structures according to the value of $(k'D/k_fb_f)$, the upper limit of $(k'D/k_fb_f)$ is 10 times the lower limit. Therefore the upper limit of F of each structure type (besides a1.1) is assumed $10^{0.33}= 2.14$ times the lower limit, since $F = f(k'D/k_fb_f)^{0.33}$. A second curve is drawn with this value of F. In Table 3 all available tests are summarised and for each type of structure a lower and upper boundary for the value of F is given (see also an example in Figure 12). The lower boundary gives with Eq. 3b a stability curve below which stability is guaranteed. Between the upper and lower boundary the stability is uncertain. It depends on various unpredictable influences whether the structure will be stable or not. The upper boundary gives a curve above which instability is (almost) certain.

Table 3 Lower and upper value for F

type	description	low F		high F	usual F (average value)
a1.1	pitched irregular natural stones on granular filter	2.0	3.0	2.5	
a1.2	loose blocks/basalt on granular filter, low stability		3.0	5.0	3.5
a2	loose blocks on granular filter, normal stability		3.5	6.0	4.5
a3	loose blocks on geotextile on compacted sand/clay		4.0	7.0	5.0
a4	linked/interlocked blocks on geotextile on good clay or on fine granular filter		5.0	8.0	6.0

The results for structure type a3 (blocks on geotextile on sand) may only be applied if the wave load is small ($H_s < 1$ or 1.5m (max.)) or to structures with a subsoil of coarse sand ($D_{50}> 0.3$ mm) and a gentle slope

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(tanα < 0.25), because geotechnical failure is assumed to be the dominant failure mechanism (instead of uplift of blocks). The good compaction of sand is essential to avoid sliding or even liquefaction. For loads higher than H = 1.2 m, a well-graded layer of stone on a geotextile is recommended (e.g. layer 0.3-0.5 m for 1.2 m < H < 2.5 m).

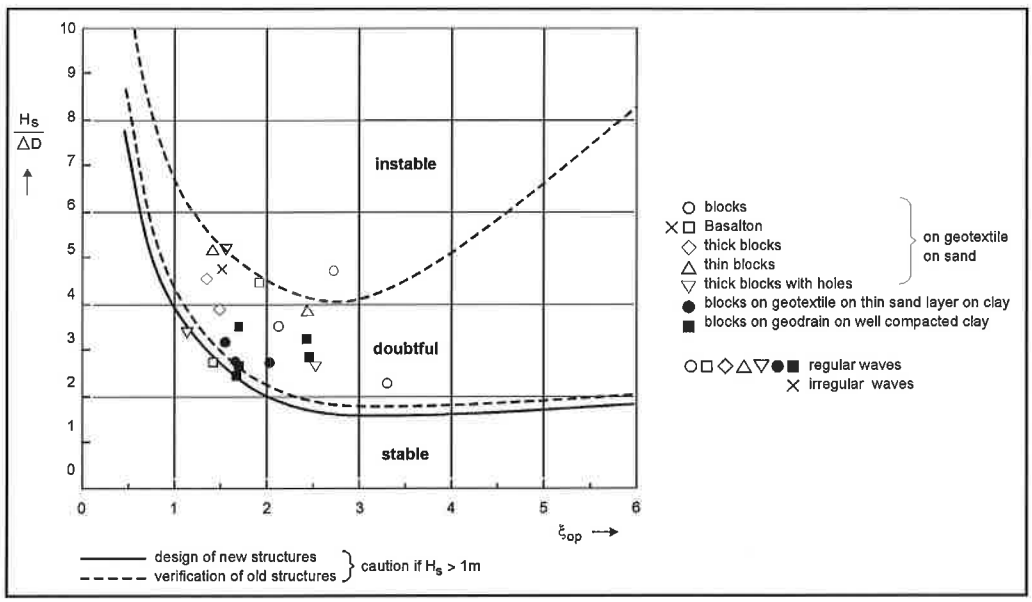

Figure 12 Example of a stability function for type a3 (loose blocks on geotextile on sand)

The results for structure type a4 can be applied on the condition that clay of high quality and with a smooth surface is used. A geotextile is recommended to prevent erosion during (long duration) wave loading. The general design criteria for geotextiles on cohesive soils are given by Pilarczyk (1999).

In the case of loose blocks an individual block can be lifted out of the revetment with a force exceeding its own weight and friction. It is not possible with the cover layers with linked or interlocking blocks. Examples of the second type are: block mattresses, ship-lap blocks and cable mats. However, in this case high forces will be exerted on the connections between the blocks and/or geotextile. In the case of blocks connected to geotextiles (i.e. by pins), the stability should be treated as for loose blocks in order to avoid the mechanical abrasion of geotextiles by moving blocks. The lower boundary of stability of cabled mats can be increased by a factor of 1.25 (or 1.5, if additionally grouted) in comparison with loose blocks. Such an increase of stability is only allowable when special measures are taken with respect to the proper connection between the mats. The upper boundary of stability (F = 8) remains the same for all systems. Application of this higher stability requires optimization of design. This optimization technique (incl. application of geometrically open but stable filters and geotextiles) can be found in (CUR, 1993 and CUR/TAW, 1995).

To be able to apply the design method for placed stone revetments under wave load to other semi-permeable systems, the following items may be adapted: the revetment parameter F, the (representative) strength parameters Δ and D, the design wave height H_s and the (representative) leakage length Λ. The basic formulas of the analytical model are presented in CUR/TAW, 1995 and Pilarczyk, 1998. Table 4 gives an overview of useable values for the revetment constant **F** in the black-box model for linked blocks (block mats).

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Table 4 Recommended values for the revetment parameter F for blockmats (the lower values refer to blocks connected to geotextile while the higher ones refer to cabled blocks).

Type of revetment		**F (-)**
Linked blocks on geotextile on sand		5 to 6
Linked blocks on geotextile on clay	good clay	5 to 6
	Mediocre (sandy) clay	4.5 to 5
Linked blocks on a granular filter	favourable construction	5 to 6
	normal construction	4 to 5
	unfavourable construction	3 to 4

The terms "favourable", "normal" and "unfavourable" refer to the composition of the granular filter and the permeability-ratio of the top layer and the filter layer (see CUR/TAW, 1995). In a case of fine granular filter and relatively permeable top layer the total composition can be defined as "favourable'. In a case of very coarse granular layer and less permeable top layer the composition can be defined as "unfavourable". In a case of blocks connected to a geotextile and concrete-filled mattresses on a filter layer the construction can be usually defined as between "unfavourable" and "normal", and the stability factor F = 3.0 to 3.5 (max. 4.0) can be applied. For blockmats and permeable mattresses on sand F = 5 (max. 6.0) can be applied. The higher values can also be used in cases that the extreme design loading is not very frequent or when the system is (repeatedly) washed in by coarse material providing additional interlocking.
This wide range of recommended values for **F** only gives a first indication of a suitable choice.
Furthermore it is essential to check the geotechnical stability with the design diagrams (see for example Figure 4 and for a full set of diagrams see Pilarczyk (1998, 1999).

5 STABILITY CRITERIA FOR CONCRETE-FILLED MATTRESSES

5.1 Concrete Mattresses

Characteristic of concrete mattresses are the two geotextiles with concrete or cement between them. The geotextiles can be connected to each other in many patterns, which results in a variety of mattress systems, each having its own appearance and properties. Some examples are given in Figure 13.

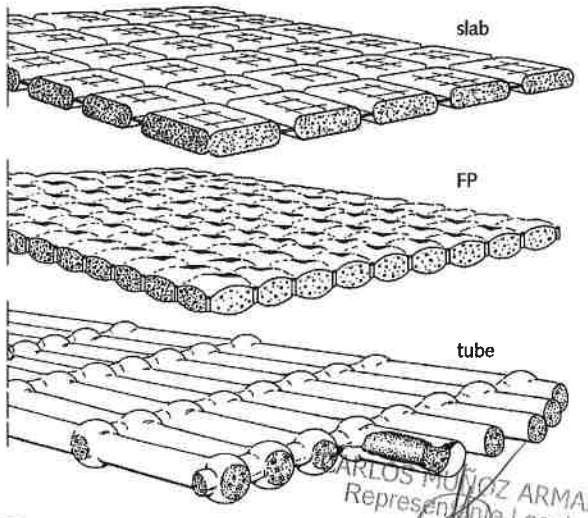

Figure 13 Examples of concrete filled mattresses

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The permeability of the mattress is one of the factors that determine the stability. It is found that the permeability given by the suppliers is often the permeability of the geotextile, or of the so-called Filter Points (Figure 14). In both cases, the permeability of the whole mattress is much smaller. A high permeability of the mattress ensures that any possible pressure build-up under the mattress can flow away, as a result of which the uplift pressures across the mattress remain smaller.
In general, with a subsoil of clay and silty sand the permeability of the mattress will be higher than the permeability of the subsoil. Therefore the water under the mattress can usually be discharged without excessive lifting pressures on the mattress.
The permeability of the mattress will be lower than the permeability of the subsoil or sub layers if a granular filter is applied, or with a sand or clay subsoil having an irregular surface (gullies/cavities between the soil and the mattress). This will result in excessive lifting pressures on the mattress during wave attack.

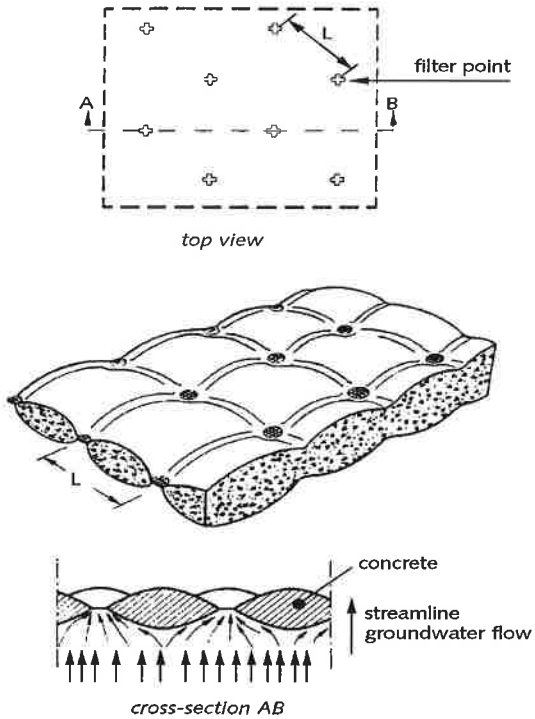

Figure 14, Principles of permeability of Filter Point Mattress

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5.2 Design rules with regard to wave load

The failure mechanism of the concrete mattress is probably as follows:

- First, cavities under the mattress will form as a result of uneven subsidence of the subsoil. The mattress is rigid and spans the cavities.
- With large spans, wave impacts may cause the concrete to crack and the spans to collapse. This results in a mattress consisting of concrete slabs which are coupled by means of the geotextile.
- With sufficiently high waves, an upward pressure difference over the mattress will occur during wave run-down, which lifts the mattress (Figure 1).
- The pumping action of these movements will cause the subsoil to migrate, as a result of which an S-profile will form and the revetment will collapse completely.

It is assumed that local settlement of the subsoil will lead to free spans of the concrete mattress. Then, the wave impact can cause the breaking of these spans, if the ratio of H_s/D is too large for a certain span length. A calculation method is derived on the basis of an empirical formula for the maximum wave impact pressure and the theory of simply supported beams. The collapsing of small spans (less then 1 or 2 m) is not acceptable, since these will lead to too many cracks.
The empirical formula for the wave impact is (Klein Breteler et al 1998):

$$\frac{F_{impact}}{\rho g} = 7.2 H_s^2 \tan\alpha \qquad (25)$$

With: F_{impact} = impact force per m revetment (N).

Calculation has resulted in an average distance between cracks of only 10 to 20 cm for a 10 cm thick mattress and wave height of 2 m. This means that at such a ratio of H_s/D the wave impacts will chop the mattress to pieces. For a mattress of 15 cm thick and a wave height of 1.5 m the crack distance will be in the order of 1 m.
Apart from the cracks due to wave impacts, the mattress should also withstand the uplift pressures due to wave attack. These uplift pressures are calculated in the same way as for block revetments. For this damage mechanism the leakage length is important.
In most cases the damage mechanism by uplift pressures is more important then the damage mechanism by impact.

The representative/characteristic values of the leakage length for various mattresses can be assumed as follow:

	Leakage length Λ (m)		
Mattress	on sand*)	on sand**)	on filter
Standard - FP	1.5	3.9	2.3
FPM	1.0	3.9	2.0
Slab	3.0	9.0	4.7
Articulated (Crib)	0.5	1.0	0.5

*) good contact of mattress with sublayer (no gullies/cavities underneath)
**) pessimistic assumption: poor compaction of subsoil and presence of cavities under the mattress

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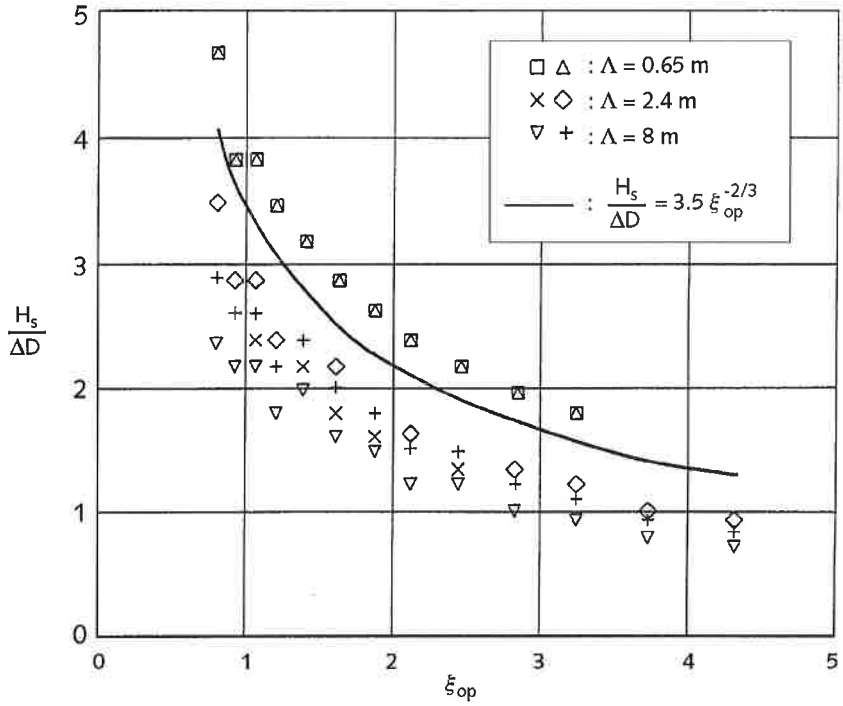

Figure 15 Calculation results for concrete mattresses ($H_s/\Delta D < 4$ because of acceptable crack distance due to impacts on spans).

Taking into consideration the above failure mechanisms, the following design (stability) formula has been derived for the mattresses (Eq. 3b):

$$\frac{H_s}{\Delta D} = \frac{F}{\xi_{op}^{2/3}} \quad with: \quad \left[\frac{H_s}{\Delta D}\right]_{max} = 4 \tag{26}$$

with:

$D = \dfrac{\text{mass per m}^2}{\rho_s}$ (which can be called $D_{effective}$ or $D_{average}$)

Δ = relative volumetric mass of the mattress (-) = $(\rho_s - \rho)/\rho$

ρ_s = volumetric mass of concrete (kg/m^3)

F = stability factor (see below)

For an exact determination of the leakage length, one is referred to the analytical model (Klein Breteler et al 1998). However, besides the mattresses of a type as, for example, the tube mat (Crib) with relative large permeable areas, the other types are not very sensitive to the exact value of the leakage length. It can be recommended to use the following values of F in design calculations:

- F = 2.5 or (≤ 3) - for low-permeable mattresses on (fine) granular filter,
- F = 3.5 or (≤ 4) - for low-permeable mattress on compacted sand,
- F = 4.0 or (≤ 5) - for permeable mattress on sand or fine filter (D_{f15} < 2 mm).

The higher values can be applied for temporary applications or when the soil is more resistant to erosion (i.e. clay), and the mattresses are properly anchored.

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