PHYSICAL MODELLING YIELDS AN INNOVATIVE BREAKWATER STRUCTURE PROTECTING A REED BED IN LAKE GENEVA FROM WIND-WAVE INDUCED HARMS

VECSERNYES ZSOLT
Laboratory for Applied Hydraulics HEPIA, University of Applied Sciences Western Switzerland, HES-SO, Geneva, Switzerland, zsolt.vecsernyes@hesge.ch

VENTURI ANDRÉ
CERA civil engineers SA, Geneva, Switzerland, andre.venturi@hesge.ch

WOHLWEND FLORENT
Laboratory for Applied Hydraulics HEPIA, University of Applied Sciences Western Switzerland, HES-SO, Geneva, Switzerland

GOBAT KARINE
Laboratory for Applied Hydraulics HEPIA, University of Applied Sciences Western Switzerland, HES-SO, Geneva, Switzerland

ANDREINI NICOLAS
Laboratory for Applied Hydraulics HEPIA, University of Applied Sciences Western Switzerland, HES-SO, Geneva, Switzerland, nicolas.andreini@hesge.ch

JAEGER AMANDINE
Laboratory for Applied Hydraulics HEPIA, University of Applied Sciences Western Switzerland, HES-SO, Geneva, Switzerland, amandine.jaeger@master.hes-so.ch

ABSTRACT

An innovative wave-braking structure was developed at the Laboratory for Applied Hydraulics (LHA) in Geneva-CH. Forming a part of the Lake Geneva shoreline revitalisation programme, the implementation of this structure will protect a future reed bed of Cologny. Raising an embankment-dike in the waters off the coast is not permitted, due to terms of the Suisse Federal Act on the Protection of Waters (RS 814.20) prohibiting the introduction of solid substances into lakes even if they would not lead to pollution. For this reason, CERA civil engineers designed a metal-wood breakwater structure which geometry had to be optimised to achieve high wave-energy dissipation performances. Hence, the *Service du lac, de la renaturation des cours d'eau et de la pêche of Geneva* mandated the LHA to carry out hydrodynamic and mechanical-structural analyses in an experimental wave tank (Jaeger 2017) and to develop a layout effectively preventing the future reed bed against wind-wave induced harms and erosion of its soil. The alternative analyses carried out at the LHA for 100-year return period wind conditions yielded an innovative breakwater geometry and the set of its optimal distance from the coast, guaranteeing an efficient wave-energy dissipation for the characteristic marling range of Lake Geneva. The mechanical stress and strain data measured on the model served CERA engineers in structural dimensioning. Since the physical hydraulic model analyses yielded robust results the innovative breakwater structure can be implemented along the Cologny coast of Lake Geneva.

Keywords: physical hydraulic model, metal-wood breakwater, wave breaking, lake shore revitalisation, ecosystem protection

1. INTRODUCTION

The reed bed of the Cologny will constitute a revitalisation measure for its artificialized shoreline and an ecological compensation for the future public access and leisure activities such as bathing in the Lake of Geneva. Hence, this project fulfils the Geneva renaturation master plan (PDCn 2030) and the Swiss federal legal requirements. The Figure 1 presents the projected measures and classifies the Cologny shoreline revitalisation programme as 1st priority.

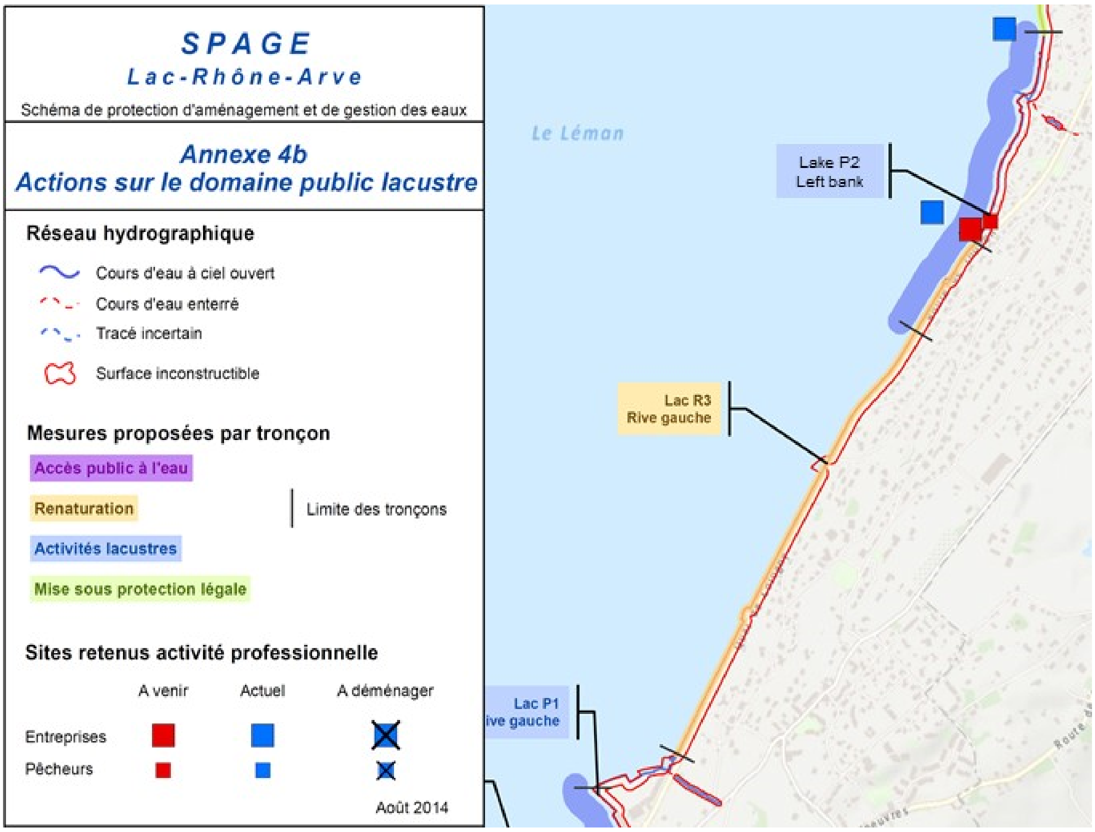

Figure 1. Excerpt of the Annexe 4b / SPAGE LRA. Proposed priority measures for the Cologny shoreline.

The project site extends over 650 m along the Cologny shoreline, including the Geneva Yacht Club to the north till the Port Tunnel belvedere to the south (see Figure 2). The lacustrine works aim the construction of a riprap belt protection of the future reed bed and a submerged wave-breaker parallel to the shoreline. Further structures are also planned such as and public access and a recreational platform.

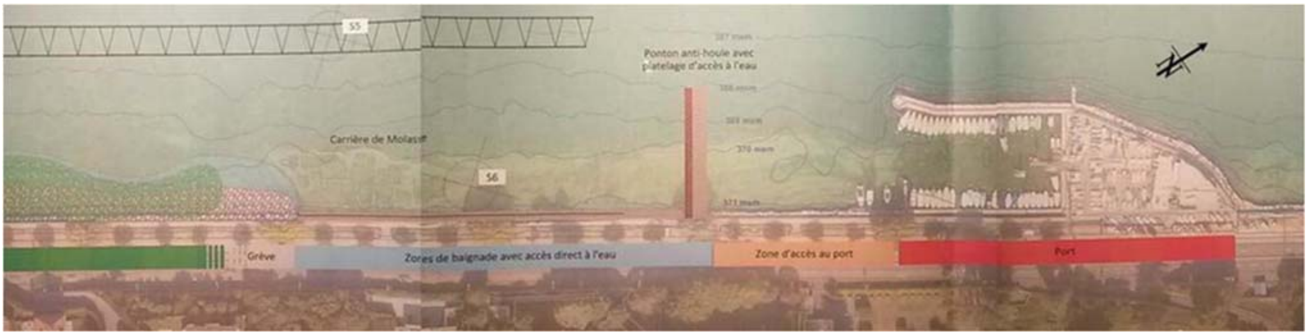

Figure 2. Excerpt of the project « Geneva Lake shore revitalisation and public access » of Cologny.

2. MAIN OBJECTIVES

The present experimental study carried out on a physical hydraulic model aims major goals in order to guarantee an efficient protection of the lacustrine reed bed and an adequate conception of the breakwater structure.

Objectives

1. Determine the most appropriate breakwater geometry to satisfy normal hydraulic conditions (up to 20-year return period) and for design return periods (50-year and 100-year return periods).
2. Protect the reed bed for 50-year dominant-wind induced waves.
3. Monitor the structural behaviour of the breakwater under 100-year return period waves.

3. EXPERIMENTAL SETUP

The present experimental study was carried on a physical hydraulic model (Figure 3), obeying Froude similitude at a 1:25 geometrical scale. Scales of typical linked hydraulic parameters are presented in Table 1. The 6.32 m long and 0.385 m wide wave tank was equipped with an air-piston wave generator. Lake bathymetry was reproduced over a path of 145 m (prototype) from the shoreline. In front of the shoreline, the reed bed and its riprap belt were modelled. The breakwater could be placed at variable distances from the shoreline. Its modular structure, presented in Figure 4, allowed variable geometrical configurations.

Table 1. Scales of typical hydraulic variables due to Froude similitude

	Scale function	Scale factor
Geometry: Distance [m], Wave length [m], wave height and amplitude [m], Pressure [m of water column]	$\frac{L_p}{L_m} = \frac{P_p}{P_m} = \lambda$	25
Wave kinematics: Speed [ms^{-1}], Time [s], Wave period [s]	$\frac{V_p}{V_m} = \frac{t_p}{t_m} = \lambda^{1/2}$	5
Frequency [s^{-1}]	$\frac{f_p}{f_m} = \lambda^{-1/2}$	0.20
Dynamic: Weight [N] Force [N]	$\frac{G_p}{G_m} = \lambda^{3}$	15'625

The wave tank was mounted with six UNAM ultrasound level probes placed at strategic points (Figure 3). A METFLOW ultrasound velocity probe was installed in the modelled riprap structure of the reed bed. The breakwater was equipped with HBM SG LE11 50 Hz 2.5 V extensomeric sensors glued at spots where the poles were embedded in the modelled lake-bed (Figure 4).

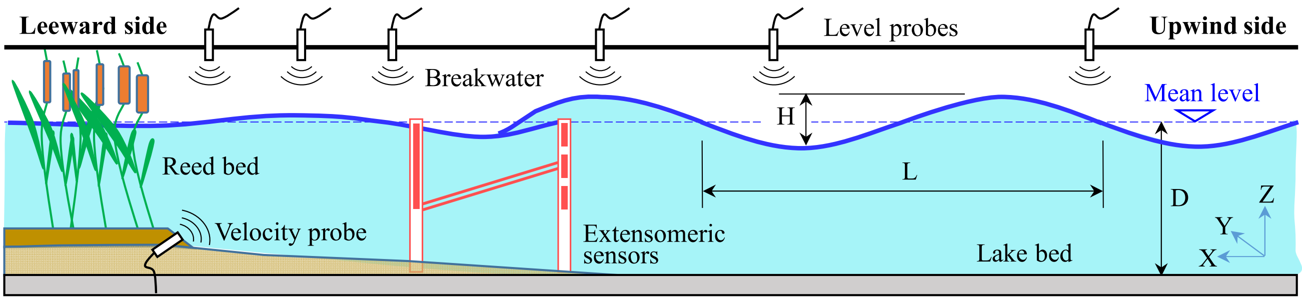

Figure 3. Hydraulic scheme of the physical model. Wave propagation from the right to left.

The primary geometry of the breakwater was provided by CERA engineers. The breakwater was designed with two separate vertical walls, facing the waves. While these walls on the prototype are planed with wooden planks, those of the model were built in metal. The leeward wall is designed with a solid surface and the upwind one with a definite hollow ratio (Figure 4). On the model, the steel structure of the frame, poles and beams was built of the same material as planned on the prototype (S235 European norm EN 10 025 and ECISS IC 1), in order to achieve a consistent physical comportment due to same specific mass, Young elasticity, Poisson coefficient.

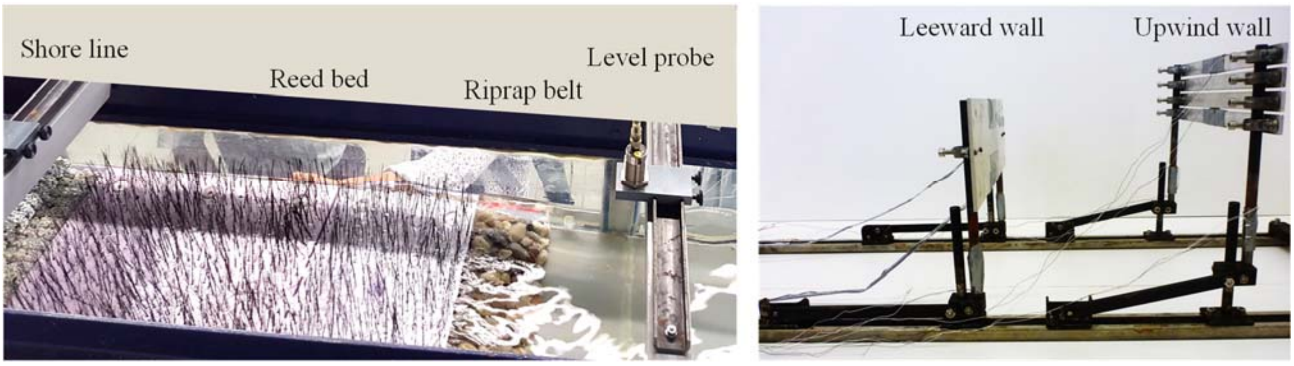

Figure 4. Left: model of the reed bed. Right: model of the breakwater with its modular metallic structure; the vertical poles are equipped with extensomeric sensors at their embedding in the modelled lake bed.

4. PRELIMINARY DATA

Lake Geneva bathymetry was obtained from SITG data base (Geographic information of the Geneva territory). Water level of the Lake is regulated at its emissary by the Seujet hydropower plant. The typical annual variation (marling) of the water level is presented in Figure 5. The mean-water level, 372.30 m a.s.l., was applied for all tests. Extreme conditions with 371.45 m a.s.l. and 372.70 m a.s.l. were also considered.

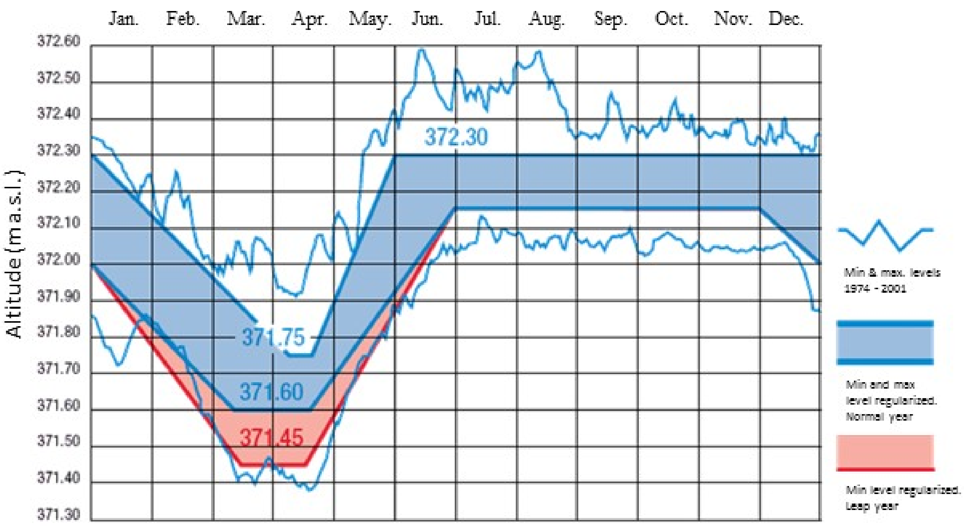

Figure 5. Characteristic water levels of Lake Geneva; mean: 372.30 m a.s.l., min.: 371.45 m, max.: 372.30 m.

Wind data were provided from MétéoSuisse. Wind rose of the Cointrin station is presented Figure 6.

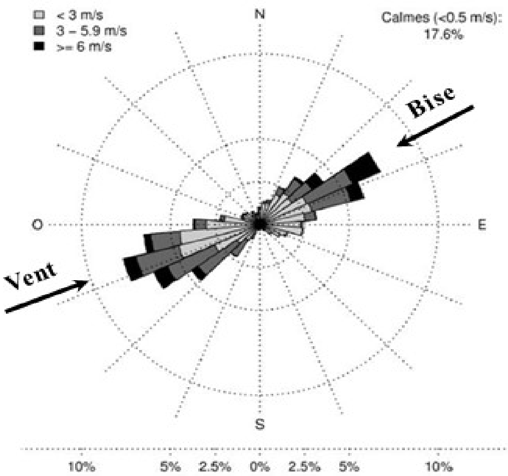

Figure 6. Characteristic wind rose of Cointrin (Geneva airport) MétéoSuisse station, with Bise as determinant at Cologny.

Due to its 16 km fetch, the determinant wind at Cologny-Geneva is the northeast dominant one, called Bise. Duration curves of distinct return periods (Tr) of Bise are presented in Figure 7 (c.f. Hughes, S.A. 1993).

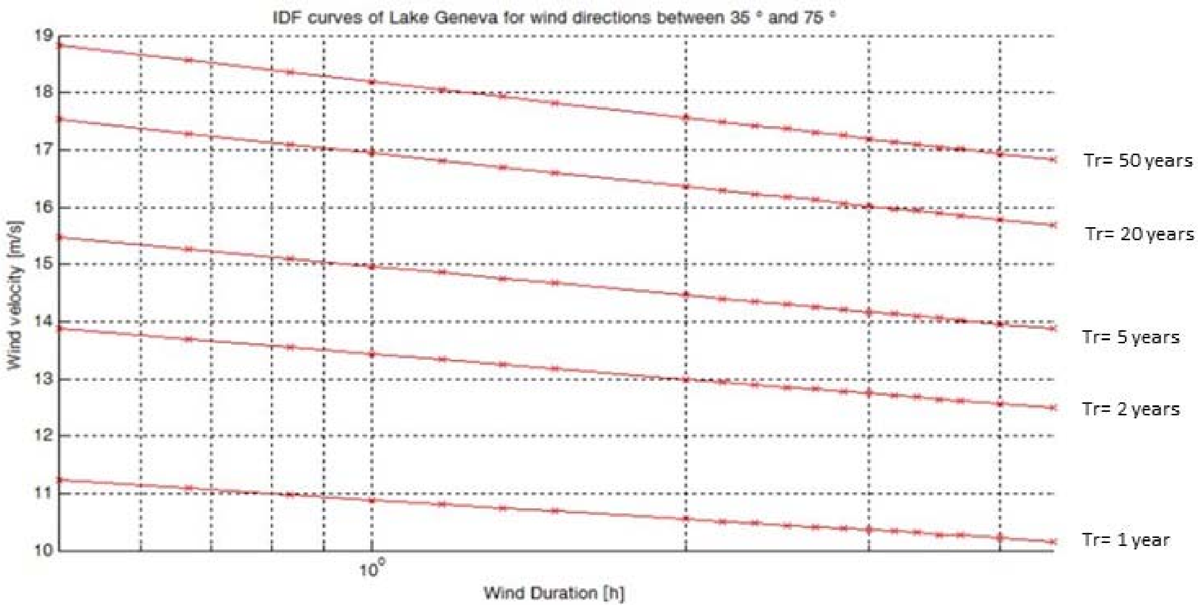

Figure 7. Duration curves for distinct return periods, Tr, of the northeast dominant wind, Bise, determinant at Cologny-Geneva due to its 16 km fetch.

Wind intensity and duration data were obtained from MétéoSuisse records. The dimensionless wave variables were determined with the JONSWAP method (Joint North Sea Wave Observation Project) (c.f. Bruschin and

Falvey 1975, and Kamphuis, J.W. 2010). They are expressed as a function of the determinant dominant wind, the local bathymetry and shoaling conditions, as follows:

$$H^*_{m0} = 0.0016(F^*)^{0.5} \tag{1}$$

$$T^*_p = 0.286(F^*)^{0.33} \tag{2}$$

$$t^* = 68.8(F^*)^{0.66} \tag{3}$$

where,

$$F^* = \frac{gF}{U^2}, \qquad H^*_{m0} = \frac{gF_{mo}}{U^2}, \qquad T^*_p = \frac{gTp}{U}, \qquad t^* = \frac{gt}{U}, \qquad d^* = \frac{gd}{U^2} \tag{4}$$

where,

g (m·s^{-2}) acceleration due du gravity
F (m) fetch
Tp (s) wave period (off shore)
H_{m0} (m) wave height
L_{m0} (m) wave length (deep water)
U (m·s^{-1}) wind velocity
D (m) water depth at location of the project
* for a dimensionless expression.

Prototype and calculated model values for Lake Geneva are presented in Table 2 for varying wind return periods and typical water levels.

Table 2. Prototype and model variables, for distinct wind return periods and typical water levels of Lake Geneva.

	Prototype						Model				
Tr [years]	Lake elevation [m.s.m]	Depth captor 6 [m]	Wind speed [m/s]	h real [m]	L real [m]	T_p real [s]	h [mm]	L [m]	T_p [s]	Nb. Of wave/min	Water level [cm]
20			14.30	0.87	21.66	3.82	35	0.87	0.76	78.60	
50	372.70	6.40	16.90	1.01	23.76	4.04	40	0.95	0.81	74.33	30.8
100			19.20	1.15	25.44	4.21	46	1.02	0.84	71.24	
20			14.30	0.87	21.44	3.82	35	0.86	0.76	78.60	
50	372.30	6.00	16.90	1.01	23.46	4.04	40	0.94	0.81	74.33	29.2
100			19.20	1.14	25.08	4.21	46	1.00	0.84	71.24	
20			14.30	0.86	21.37	3.82	35	0.85	0.76	78.60	
50	372.20	5.90	16.90	1.01	23.38	4.04	40	0.94	0.81	74.33	28.8
100			19.20	1.14	24.98	4.21	46	1.00	0.84	71.24	
20			14.30	0.86	21.02	3.82	34	0.84	0.76	78.60	
50	371.70	5.40	16.90	1.01	22.92	4.04	40	0.92	0.81	74.33	26.8
100			19.20	1.14	24.44	4.21	46	0.98	0.84	71.24	
20			14.30	0.86	20.94	3.82	34	0.84	0.76	78.60	
50	371.60	5.30	16.90	1.01	22.82	4.04	40	0.91	0.81	74.33	26.4
100			19.20	1.14	24.32	4.21	45	0.97	0.84	71.24	
20			14.30	0.86	20.81	3.82	34	0.83	0.76	78.60	
50	371.45	5.15	16.90	1.00	22.66	4.04	40	0.91	0.81	74.33	25.8
100			19.20	1.14	24.14	4.21	45	0.97	0.84	71.24	

5. MODELLING RESULTS

The calibration of the physical model was first carried out, on the basis of field wave data and JOHNSWAP calculations. The air piston course-amplitude and the frequency of the wave generator were fitted. Wave period, wave length and height were thus obtained for each return period to be tested.

The simulations were carried out under stationary condition with sinusoidal homogeneous waves. Wave propagation was defined as frontal to breakwater in order to achieve maximum mechanical stress on its structure. Three experimental series were run by consecutive adjustment of the breakwater's geometry in order to achieve the most efficient wave energy dissipation between the upwind and leeward sectors and therefore to protect effectively the reed bed (see Boillat *et al.* 2006).

6.1 Series n°1

Test configurations. Lake Geneva water level: 372.30 m a.s.l.. Wave return periods: 50, 100 years. Space between breakwater walls: 8, 12 m. Upwind wall off shore location: 85 m. Upwind wall hollow ratio: 10, 20%. Inclined hedge height of the upwind wall: -10, 30, 47.5, 90 cm. Inclined hedge height of the leeward wall: 30, 60, 90 cm.

The 48 tests carried out on the energy dissipation potential yielded the following results.

1. An 8 m spacing, between the walls of the dissipation chamber, is more efficient than a 12 m one.
2. The presence of a hedge on the top of booth wall has a positive impact on the wave energy reduction.
3. A 10% hollow of the upwind wall revels better than a 20% one.
4. A complementary riprap structure put at the foot of the upwind wall has no influence on wave energy reduction

The mechanical analyses of the breakwater structure were carried out for T = 50 years and 100 years. Typical mechanical stress results, for T = 100 years, are presented in Table 3, being slightly higher than those for T = 50 years.

Table 3: Characteristic stress results on steel poles S235, for T = 100 years return period.

Test n°	Deformation upwind wall µm	Deformation leeward wall µm	Momentum upwind kNm	Momentum leeward kNm	Stress upwind N/mm²	Stress leeward N/mm²
4	29	36	209	259	152	189
9	22	32	158	230	115	168
14	23	31	165	223	121	162

The observed T = 100 year stress, σ_d, is smaller than the elastic stress $f_{y,235} = 235\ N/mm^2$ and the ultimate stress $f_{u,235} = 360\ N/mm^2$. The structure can in such a configuration resist to 100 year waves.

6.2 Series n°2

Test configurations. Lake Geneva water level: 372.30 m a.s.l. Wave return periods: 50 years. Space between breakwater walls: 6 m, 8 m. Upwind wall off shore location: 75, 85 m. Upwind wall hollow ratio: 10%. Inclined hedge height of the upwind wall: 0, 30, 60, 90 cm. Inclined hedge height of the leeward wall: without, with 30, 60, 90 cm.

The 48 tests carried out on the energy dissipation potential yielded the following results.

1. An 8 m spacing of the dissipation chamber seems more efficient for T = 50 year waves than a 6 m one.
2. The presence of a hedge on the top of booth wall has a positive impact on the wave energy reduction.
5. A location of the breakwater at 75 m or 85 m off the shore line has no impact on the energy reduction at the modelled reed bed.

The mechanical analyses of the breakwater structure were carried out for T = 50 years. Typical mechanical stress results are presented in Table 4 and Table 5.

Table 4. Upwind steel pole S235 results, for T = 50 years return period. Moment, design force and distributed surface load.

Test n°	$\sigma_{d,\ mid}$ Max N/mm²	Min N/mm²	M_d + kNm	M_d - kNm	F_d + kN	F_d - kN	$q_{d,total}$ + kN/m²	$q_{d,total}$ - kN/m²	$q_{d,imp}$ + kN/m²	$q_{d,imp}$ - kN/m²
23	381	-173	523	-238	156	-71	14	-6	16	-7
24	326	-188	448	-258	134	-77	12	-7	14	-8
48	355	-203	488	-279	146	-83	13	-8	15	-9

Table 5. Leeward steel pole S235 results, for T = 50 years return period. Moment, design force and distributed surface load.

Test n°	$\sigma_{d,\ mid}$ Max N/mm²	Min N/mm²	M_d + kNm	M_d - kNm	F_d + kN	F_d - kN	$q_{d,total}$ + kN/m²	$q_{d,total}$ - kN/m²	$q_{d,imp}$ + kN/m²	$q_{d,imp}$ - kN/m²
23	381	-173	523	-238	156	-71	14	-6	16	-7
24	326	-188	448	-258	134	-77	12	-7	14	-8
48	355	-203	488	-279	146	-83	13	-8	15	-9

The observed T = 50 year stress, σ_d, are larger than the elastic stress $f_{y,235} = 235\ N/mm^2$ and in certain cases also than the ultimate stress $f_{u,235} = 360\ N/mm^2$. The structure in such a configuration do not resist to 50 year waves.

6.3 Series n°3

Test configurations. Lake Geneva water level: 371.45, 372.30, 372.70 m a.s.l. Wave return periods: 20, 50, 100 years. Space between breakwater walls: 6 m, 8 m. Upwind wall off shore location: 75 m. Upwind wall hollow ratio: 0, 10%. Inclined hedge height of the upwind wall: 0, 60 cm. Inclined hedge height of the leeward wall: 0, 30 cm. See a test configuration and resulting wave height reduction in Figure 8.

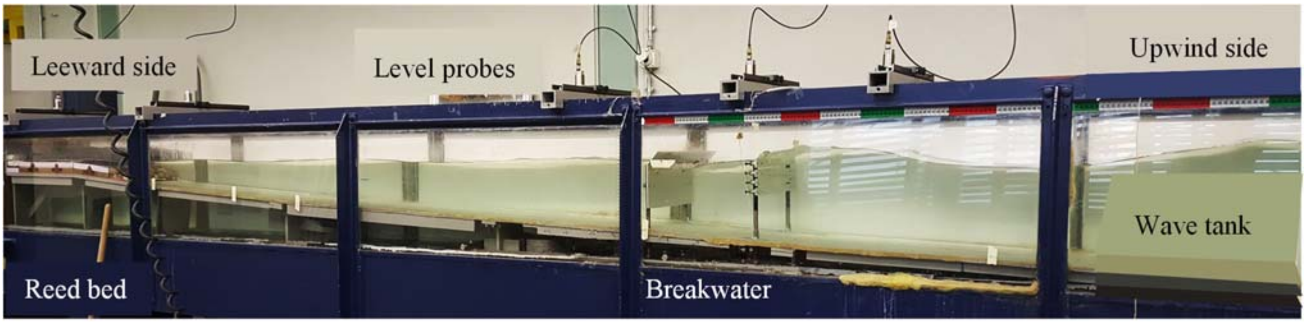

Figure 8. Test on the physical model, revealing the wave height reduction achieved by the breakwater.

The configuration of the 8 most relevant tests out of 72 realised are presented in Table 6. These results correspond to mean-water level conditions 372.30 m a.s.l. of Lake Geneva.

Table 6. Configuration of the eight most relevant test, under mean-water level conditions 372.30 m a.s.l. of Lake Geneva.

Configuration	Dissip. Chamber spacing		Upwind Wall hollow ratio		Hedge	
	6m	8m	0%	10%	without	0.6 / 0.3m upwind / leeward
A	x		x		x	
B	x		x			x
C	x			x	x	
D	x			x		x
E		x	x		x	
F		x	x			x
G		x		x	x	
H		x		x		x

The wave height reduction efficiency, K_{eff}, of the breakwater can reach 0.70, as expressed in Table 7.

Table 7. Wave height reduction rates, K_{eff}, for the eight most relevant configurations tested.

Configuration	$K_{efficiency}$
F	0.72
B	0.7
H	0.7
D	0.69
E	0.65
A	0.57
G	0.57
C	0.51

The investigations provided the following results for wave energy dissipation potential

1. An 8 m spacing of the dissipation chamber revels more efficient than a 6 m one.
2. The presence of a hedge on the top of the wall has a positive impact on the wave energy reduction.
3. The solid upwind wall revels better than a 10% hollow.

The mechanical analyses of the breakwater structure were carried out for T = 50 years. Typical mechanical stress results are presented in Table 8 and Table 9.

Table 8. Upwind steel pole S235 results, for T = 50 years return period. Moment, design force and distributed surface load.

Test n°	$\sigma_{d,\,mid}$ Positive N/mm²	$\sigma_{d,\,mid}$ Negative N/mm²	M_d + kNm	M_d - kNm	F_d + kN	F_d - kN	$q_{d,total}$ + kN/m²	$q_{d,total}$ - kN/m²	$q_{d,imp}$ + kN/m²	$q_{d,imp}$ - kN/m²
23	265	-121	364	-165	109	-49	10	-4	11	-5
24	233	-134	320	-184	95	-55	9	-5	10	-6
48	233	-133	320	-183	95	-55	9	-5	10	-6

Table 9. Leeward steel pole S235 results, for T = 100 year return period. Momentum, design and distributed surface load.

Test n°	$\sigma_{d,\,mid}$ positive N/mm²	$\sigma_{d,\,mid}$ negative N/mm²	M_d + kNm	M_d - kNm	F_d + kN	F_d - kN	$q_{d,total}$ + kN/m²	$q_{d,total}$ - kN/m²
23	216	-103	296	-141	99	-47	7	-3
24	206	-126	283	-172	94	-57	7	-4
48	210	-114	288	-157	96	-52	7	-4

The observed stress, σ_d , is smaller than the elastic stress $f_{y,235} = 235\,N/mm^2$ and the ultimate stress $f_{u,235} = 60\,N/mm^2$. The structure can in such a configuration resist to 100 year waves.

6. CONCLUSIONS

Since raising an embankment-dike in Suisse lakes is not permitted, the Laboratory for Applied Hydraulics (LHA) in Geneva-CH accomplished an experimental research on an innovative metal-wood breakwater structure designed by CERA civil engineers. The study carried out on the physical hydraulic model yielded the following achievements:

1. The breakwater should have two parallel vertical walls mounted on metallic poles imbedded in the lakebed.
2. An 8 m wide dissipation chamber, the space between the walls, is more efficient than a 6 m or 12 m one.
3. Solid impervious metal-wood walls are more efficient then hollowed ones.
4. Implementing a hedge inclined 45° leeward on the upwind wall and a hedge inclined 45° upward on the leeward wall increases considerably the wave energy dissipation.
5. The breakwater is the most efficient at mean-water condition of Lake Geneva, 372.30 m a.s.l., yielding up to 70% wave height reduction. The breakwater remains efficient at low- 371.45 m a.s.l. and high-water 372.70 m a.s.l. conditions, yielding respectively 55% and 60% wave height reduction.
6. The mechanical stress induced on the breakwater's structure by mean-water condition waves, are systematically larger than those observed under low- and high-water conditions.
7. In the studied range (75 m - 85 m), the location of the breakwater has a weak influence on wave energy dissipation.
8. The structure with only vertical steel poles imbedded in the lakebed would not resist 100-year waves and has to be reinforced.

In conclusion, the breakwater should be constructed with an 8 m large dissipation chamber, with vertical solid walls i.e. without hollow. The upwind wall should have a 60 cm high hedge inclined 45° leeward and the leeward wall a 30 cm high hedge inclined 45° upward. In this case, a 70% wave-height reduction can be achieved between the upwind and leeward sides. The structure must be supported by inclined beams embedded in the lakebed on the leeward side. A distance of the breakwater at 75 m from the Cologny shoreline would be optimal, accommodating favourably the reed-bed protection requirements and the public recreational goals.

REFERENCES

Boillat, J.-L. et al. (2006). Critères de dimensionnement et recommandations pratiques pour la protection des rives lacustres, Approche méthodologique pour l'établissement d'un projet de protection des rives lacustres par des mesures souples et écologiques, Erosee.

Bruschin, J., Falvey, H. (1975). Vagues de vent sur un plan d'eau confiné: considérations générales et application au Léman (Petit-Lac), in Bulletin technique de la Suisse romande, Cahier 14. Tiré de doi.org/10.5169/seals-72567

Hughes, S.A. (1993). Physical models and laboratory techniques in costal engineering. Advanced Series on Ocean Engineering, Volume 7, World Scientific, Singapore, 570 pp.

Jaeger, A. (2017). Étude sur modèle physique d'un brise-lames lacustre, Bachelor thesis HEPIA HES-SO, Suitzerland

Kamphuis, J.W. (2010). Introduction to Coastal Engineering and Management. Advanced Series on Ocean Engineering, Volume 30, World Scientific, Singapore, 502 pp.

PDCn 2030 (2015). Plan directeur cantonal 2030, Etat de Genève.